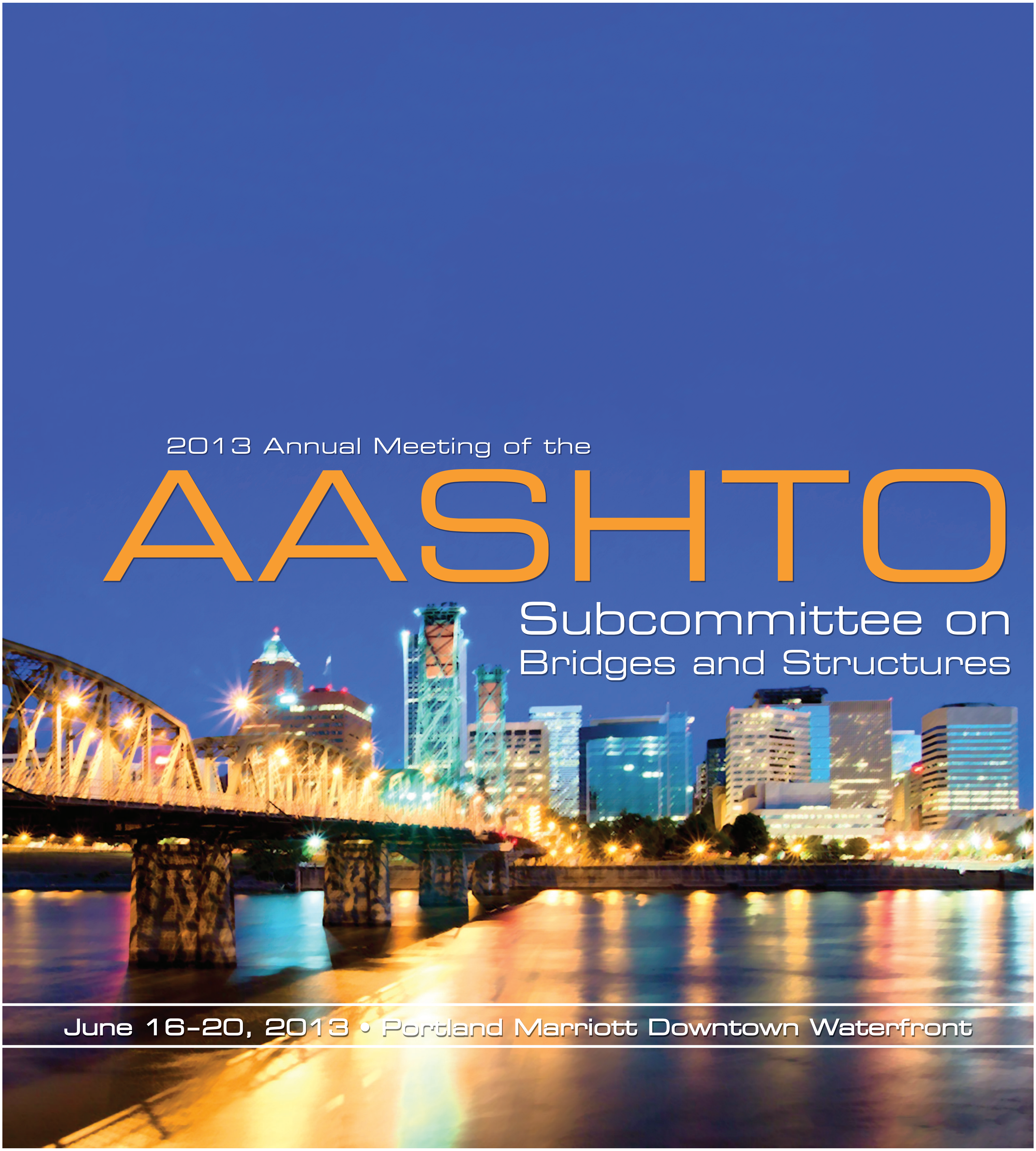

2013 Annual Meeting of the

AASHTO

Subcommittee on Bridges and Structures

June 16-20, 2013 • Portland Marriott Downtown Waterfront

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2013 AASHTO BRIDGE SUBCOMMITTEE MEETING

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(As of May 15, 2013)

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2013 AASHTO BRIDGE COMMITTEE AGENDA ITEM: 1

SUBJECT: LRFD Bridge Design Specifications: Section 3, Articles 3.10.1 & 3.16

TECHNICAL COMMITTEE: T-3 Seismic

☐ **REVISION** ☒ **ADDITION** ☐ **NEW DOCUMENT**

☒ **DESIGN SPEC** ☐ **CONSTRUCTION SPEC** ☐ **MOVABLE SPEC**
☐ **MANUAL FOR BRIDGE EVALUATION** ☐ **SEISMIC GUIDE SPEC** ☐ **COASTAL GUIDE SPEC**
☐ **OTHER**

DATE PREPARED: 2/1/13
DATE REVISED:

AGENDA ITEM:

Item#1

Add the following paragraph to the end of Article 3.10.1:

When seismic isolation is used, the design shall be in accordance with the *Guide Specifications for Seismic Isolation Design*, unless otherwise specified by the Owner.

Item #2

In Article 3.16, add the following reference:

AASHTO. 2010. *Guide Specifications for Seismic Isolation Design*, Third Edition, American Association of State Highway and Transportation Officials, Washington, DC.

OTHER AFFECTED ARTICLES:

None

BACKGROUND:

The intent of this item is to provide clear direction for the seismic design of seismically isolated bridges.

Item 1 requires the Engineer to design seismically isolated structures using the *Guide Specifications for Seismic Isolation Design.* The clause “unless otherwise specified by the Owner” permits project-specific criteria to be developed should an isolation system not covered by the Guide Specifications arise.

Item 2 adds the *Guide Specifications for Seismic Isolation Design* to the Section 3 Reference List.

ANTICIPATED EFFECT ON BRIDGES:

Improved seismic response

REFERENCES:

None

OTHER:

None

2013 AASHTO BRIDGE COMMITTEE AGENDA ITEM: 2

SUBJECT: LRFD Bridge Design Specifications: Section 6, Various Articles

TECHNICAL COMMITTEE: T-3 Seismic/T-14 Steel

☒ **REVISION** ☒ **ADDITION** ☐ **NEW DOCUMENT**

☒ **DESIGN SPEC** ☐ **CONSTRUCTION SPEC** ☐ **MOVABLE SPEC**
☐ **MANUAL FOR BRIDGE EVALUATION** ☐ **SEISMIC GUIDE SPEC** ☐ **COASTAL GUIDE SPEC**
☐ **OTHER**

DATE PREPARED: 2/1/13
DATE REVISED:

AGENDA ITEM:

Item#1

Revise the 1st paragraph of Article 6.5.5 as follows:

All applicable extreme event load combinations in Table 3.4.1-1 shall be investigated. ~~For Extreme Event I, γ_p for *DC* and *DW* loads shall be taken equal to 1.0.~~

Item #2

In Article 6.16.4.1, revise the bullets and the 2nd paragraph as follows:

- *Type ~~1~~A*~~—~~Design an elastic superstructure with a ductile substructure according to ~~the provisions of Article 6.16.4.4~~ these Specifications.
- *Type ~~2~~B*~~—~~With the approval of the Owner including the design methodology, d~~D~~esign an elastic superstructure and substructure with a fusing mechanism at the interface between the superstructure and substructure ~~according the provisions of Article 6.16.4.4~~.

The deck and shear connectors on bridges located in Seismic Zones 3 or 4 shall also satisfy the provisions of Articles 6.16.4.2 and 6.16.4.3, respectively. If Strategy Types ~~1~~A or ~~2~~B are invoked for bridges in Seismic Zone 2, the provisions of Articles 6.16.4.2 and 6.16.4.3 should be considered.

Item #3

Revise the 1st sentence of the 1st paragraph of Article C6.16.4.1 as follows:

~~An alternative~~ The conventional seismic ~~performance criterion~~ design strategy for slab-on-steel-girder bridges, denoted herein as Type A, is to provide an elastic superstructure in combination with a ductile substructure.

Item #4

Revise the 2nd paragraph of Article C6.16.4.1 as follows:

Providing an essentially elastic superstructure and substructure by utilizing response modification devices such as ~~base~~ seismic isolation as a fusing mechanism is a viable alternative strategy to Type A for designing steel-girder bridges ~~in Seismic Zones 3 and 4~~ to resist earthquake loading. When seismic isolation is used, the Engineer is referred to the *Guide Specifications for Seismic Isolation Design* (AASHTO, 2010).

Item #5

Revise the 3rd paragraph of Article C6.16.4.1 as follows:

The provision of an alternative fusing mechanism between the interface of the superstructure and substructure by shearing off the anchor bolts ~~is~~ may also comprise an adequate seismic strategy. However, care must be taken to provide adequate ~~seat width~~ support length and to stiffen the girder webs against out-of-plane forces at support locations. It is also anticipated that large deformations will occur in the superstructure at support locations during a seismic event where this strategy is employed.

Item #6

Revise the bullets in Article 6.16.4.2 as follows:

- For structures in Seismic Zone 2 designed using Strategy Type ~~1~~A, the elastic transverse base shears at the support under consideration divided by ~~the~~ a response modification factor, *R*, ~~specified in Table 3.10.7.1-1~~ equal to 1.0.

- For structures in Seismic Zones 3 or 4 designed using Strategy Type ~~1~~A, the lesser of:

  - The elastic transverse base shears at the support under consideration divided by ~~the~~ a response modification factor, *R*, ~~specified in Table 3.10.7.1-1~~ equal to 1.0, and

  - The inelastic hinging force determined as specified in Article 3.10.9.4.3 ~~multiplied by 1.2 or 1.4 for ASTM A706 or ASTM A615 Grade 60 reinforcement, respectively~~.

- For structures in Seismic Zones 2, 3 or 4 designed using Strategy Type ~~2~~B, the expected lateral resistance of the fusing mechanism multiplied by ~~the applicable~~ an appropriate overstrength factor.

Item #7

In Article C6.16.4.2, revise the 2nd sentence of the 5th paragraph as follows:

At skewed supports in structures designed using Strategy Type ~~1~~A, *F* should be taken as the sum of the absolute values of the components of the transverse and longitudinal base shears parallel to the skew combined as specified in Article 3.10.8, as shown in Figure C6.16.4.2-1.

Item #8

Revise the last paragraph of Article 6.16.4.4 as follows:

The lateral force, *F*, for the design of the support cross-frame members or support diaphragms shall be determined as specified in Article 6.16.4.2 for structures designed using Strategy Types ~~1~~A or ~~2~~B, as applicable.

Item #9

In Article 6.17, add the following reference:

AASHTO. 2010. *Guide Specifications for Seismic Isolation Design*, Third Edition, American Association of State Highway and Transportation Officials, Washington, DC.

OTHER AFFECTED ARTICLES:

None

BACKGROUND:

The changes proposed herein address several shortcomings included in the language that was passed in 2012.

Item 1 addressed the fact that the permanent load factors, γ_p, for *DC* and *DW* for Extreme Event I should be the same for all materials and as established in Article 3.4.1.

Item 2 addresses the fact that Article 6.16.4.4 does not cross reference all the provisions required to complete a Type A or Type B design. For Type A the Engineer must follow all the applicable provisions in the *LRFD Bridge Design Specifications*. For Type B the designer will need Owner's approval for use of the strategy and will need to agree with the Owner on a design criteria and methodology. The item also changes the Strategy Type designations from 1 and 2 to A and B to distinguish these from and avoid confusion with those used in the *Guide Specifications for LRFD Seismic Bridge Design*.

Item 3 recognizes that the conventional design methodology is for Type A designs.

Item 4 recognizes seismic isolation as a distinct strategy and refers the Engineer to the *Guide Specification for Seismic Isolation Design*. The item also corrects the term "base" isolation to "seismic" isolation.

Item 5 addresses the need for caution when applying alternative fusing mechanisms to seismic isolation. The item also corrects the older term "seat width" to the current term "support length".

Item 6 addresses the design shear forces used to design an elastic superstructure. The design shear forces may either be taken as the full unreduced R=1.0 forces or the inelastic hinging forces, whichever are less. The use of a normal substructure R value (R>1.0), as the current article permits, could lead to unintentionally weak systems where failure could occur in the superstructure, rather than yielding occurring in the substructure, as intended. Additionally, the item removes the 1.2 and 1.4 overstrength factors, which are redundant because Article 3.10.9.4.3 already includes a 1.3 factor, which serves as the overstrength factor in LRFD for reinforced concrete columns. Finally, the item changes the Strategy Type designations from 1 and 2 to A and B to distinguish these from and avoid confusion with those used in the *Guide Specifications for LRFD Seismic Bridge Design*.

Item 7 revises the Strategy Type designations from 1 to A for Article C6.16.4.2.

Item 8 revises the Strategy Type designations from 1 and 2 to A and B for Article 6.16.4.4.

Item 9 adds the *Guide Specifications for Seismic Isolation Design* to the Section 6 Reference List.

ANTICIPATED EFFECT ON BRIDGES:

Improved seismic response

REFERENCES:

None

OTHER:

None

2013 AASHTO BRIDGE COMMITTEE AGENDA ITEM: 3

SUBJECT: AASHTO Guide Specifications for LRFD Seismic Bridge Design, Sections 1, 2, 3, 4 & 7, Various Articles

TECHNICAL COMMITTEE: T-3 Seismic / T-14 Steel

☒ **REVISION**	☒ **ADDITION**	☐ **NEW DOCUMENT**
☐ **DESIGN SPEC**	☐ **CONSTRUCTION SPEC**	☐ **MOVABLE SPEC**
☐ **MANUAL FOR BRIDGE EVALUATION**	☒ **SEISMIC GUIDE SPEC**	☐ **COASTAL GUIDE SPEC**
	☐ **OTHER**	

DATE PREPARED: 2/1/13
DATE REVISED: 4/17/13

AGENDA ITEM:

Item #1

Revise the 4th paragraph of Article 1.3 as follows:

The flowcharts in Figure 1.3 address the design of bridges using the Type 1 design strategy. The flowchart in Figure 1.3-1 guides the designer on the applicability of the Guide Specifications and the seismic design procedure for bridges in SDC A and single-span bridges. Figures 1.3-2 through 1.3-4 show seismic design procedure flowcharts for bridges in SDC B through D, respectively. Figure 1.3-5 shows foundation design and detailing flowcharts.

Item #2

In Article 2.2, delete and revise the following notation:

~~K_{DED} = stiffness of the ductile end diaphragm (kip/in.) (7.4.6)~~

~~K_{SUB} = stiffness of the substructure (kip/in.) (7.4.6)~~

R = maximum expected displacement ductility of the structure; response modification factor (4.3.3) (7.2) (7.2.2) ~~(7.4.6)~~

Item #3

Revise the 2nd bullet in Article 3.3 as follows:

- **Type 2**—Essentially Elastic Substructure with a Ductile Superstructure: ~~This category applies only to steel superstructures, and ductility is achieved by ductile elements in the pier cross frames.~~ This category applies only to straight, nonskewed, steel I-Section composite girder superstructures with ductile cross frames at the supports. The use of this strategy shall be approved by the Owner and based on a case-specific design criteria and methodology.

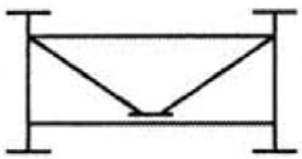
Item #4

In Article C3.3, add the following as the 2nd paragraph in "**Type 2—Essentially Elastic Substructure with a Ductile Superstructure**":

The use of this strategy requires the Owner's approval and a case-specific design criteria and methodology because the design guidelines are under development and there is a lack of practical experience with ductile cross frames.

Item #5

Replace identification label of element Number 3 in Figure 3.3-2 with the following:

3 

~~Ductile end diaphragms in superstructure (Article 7.4.6)~~

Ductile cross frame at the supports in the superstructure (Article 7.4.6)

Item #6

Add the following to the end of Article 4.3.3:

For steel substructures using a Type 1 design strategy, the response modification factor, *R*, may be used in lieu of the maximum local member displacement ductility demand, μ_D, in Eq. 4.3.3-1.

Item #7

Revise the last paragraph of Article 4.11.1 as follows:

For SDC C or D, exception to capacity design requirements using provisions for conventional plastic hinge formation in the substructure elements is permitted for the following:

- The seismic resisting system includes the fusing effects of an isolation device (Type 3 global design strategy),
- ~~A ductile end diaphragm is incorporated into the transverse response of a steel superstructure (Type 2 global design strategy; see Article 7.2.2), and~~
- A foundation situated in soft or potentially liquefiable soils where plastic hinging is permitted below ground.

Item #8

In Article C4.11.1, revise the 1st paragraph and add an new 2nd paragraph as follows:

The objective of these provisions for conventional design is that inelastic deformation (plastic hinging) occurs at the location in the columns (top or bottom or both) where they can be readily inspected and repaired. To achieve this objective, all members connected to the columns, the shear capacity of the column, and all members in the load path from the superstructure to the foundation, should be capable of transmitting the maximum (overstrength) force effects developed by plastic hinges in the columns. ~~The exceptions to the need for capacity design of connecting elements are:~~

- ~~Where seismic isolation design is used, and~~
- ~~In the transverse direction of columns when a ductile diaphragm is used and piers when ductile cross frames (Type 2 strategy) are used.~~

When seismic isolation (Type 3 strategy) is used, capacity design using provisions for conventional plastic hinge formation (Type 1 strategy) for connecting elements is not necessarily required. In this case, the substructure should be designed to resist the maximum lateral force generated by the isolation bearings (using the maximum System Property Modification Factors as defined in the *AASHTO Guide Specifications for Seismic Isolation Design*) and the inertial forces of the substructure elements in an elastic manner. However, the Designer is referred to the paragraphs below for design force increase consideration which are intended to provide additional conservatism for cases where Type 1 capacity design methods are not used.

Item #9

In Section 7, revise the Table of Contents as follows:

Item #10

Replace the term "end diaphragms" with "support diaphragms" in the 3rd paragraph and the 3rd bullet of Article 7.1.

Item #11

Replace the last paragraph of Article C7.1 with the following:

Bridge bearings may not resist lateral loads in a uniform manner especially in bridges with relatively flexible support diaphragms or cross frames. Bearings that are subjected to greater demands may be damaged prior to those subjected to lesser demands. Damaged bearings may transfer a different amount of load to the girders than undamaged bearings. Nonuniform bearing demands should be taken into account in the design of the support diaphragms or cross frames. Also, a significant change in the load distribution between abutments and piers may occur and should be considered in the design.

Item #12

Replace the 4th paragraph of Article 7.2 with the following:

Seismic design forces for individual members designed as earthquake resisting elements (ERE) to respond in a ductile manner shall be determined by dividing the unreduced elastic seismic forces by the appropriate response modification factor (R) specified in the *AASHTO LRFD Bridge Design Specifications*. A combination of orthogonal seismic forces equivalent to the orthogonal seismic displacement combination specified in Article 4.4 shall be used to obtain the seismic force demands.

Item #13

Add the following to the end of Article C7.2:

The Type 2 design strategy using ductile cross frames is intended to limit the transverse demands acting on the substructure but do not significantly alter the longitudinal demands. A Type 2 strategy alone may not be a feasible method of addressing both longitudinal and transverse seismic demands. In the longitudinal direction, a Type 1 or Type 3 strategy may be required to accommodate seismic demands.

Item #14

Replace Article 7.2.2 with the following:

7.2.2—Type 2

For Type 2 structures, the Designer may, with the Owner's approval, design a ductile superstructure and an essentially elastic substructure using a case-specific criteria and methodology.

When the Type 2 system consists of ductile cross frames at supports, the substructure shall be designed to resist a force in the transverse direction of the bridge that includes at least the sum of the following:

- The overstrength lateral resistance of the ductile cross frame using the expected material properties,
- The lateral resistance of the frame action generated between the girders and deck, and
- The inertial forces of the girders and substructure.

Where a Type 2 strategy is used, the substructure shall be detailed such that it has the capacity to respond in a ductile manner in both the longitudinal and transverse directions.

Item #15

Insert the following new commentary to Article 7.2.2:

C7.2.2

The use of ductile cross frames at the supports of steel superstructures may be an effective means of accommodating transverse seismic demands. Longitudinal demands must be accommodated using other means such as a Type 1 strategy (ductile substructure) or by utilizing the longitudinal restraint offered by the abutments (see Article 5.2.3). However, due to the lack of specific analytical or experimental data substantiating this strategy, the Designer is required to develop appropriate criteria and methodologies, which are to be approved by the Owner, before employing this strategy.

Seismic demands on the substructure must consider all of the applicable force components. These forces include those associated with inelastic resistance of ductile cross frames as well as inertial effects of the structure below the deck. Demands in the longitudinal direction, including potential inelastic demands, must also be considered. Because the design guidelines for a Type 2 strategy are still under development, ductile detailing of the substructure elements in both the transverse and longitudinal direction is required. The extent of the inelastic action in the substructure and subsequent ductile detailing requirements are developed in a case-specific manner.

Item #16

Replace Article 7.4.6 with the following:

7.4.6—Ductile Cross Frames

When permitted by the Owner, ductile cross frames may be used to accommodate transverse seismic demands. Ductile cross frames shall be designed using a case-specific criteria and methodology.

Ductile cross frames shall only be used for bridges satisfying the following requirements:

- I-Section girders (rolled or built up) with composite cast-in-place concrete decks,
- Bridges with shear stud connectors provided throughout the length of the girders including the negative moment regions of continuous structures,
- Bridges without skew,
- Bridges without horizontal curvature, and
- Bridges with uniform width and girder spacing

Item #17

Insert the following new commentary to Article 7.4.6:

C7.4.6

At this time, the use of ductile cross frames at the bridge supports to address transverse seismic demands is considered to be experimental and developments are ongoing. Refer to Articles 4.11.1 and C7.2.2 for additional information and design requirements.

OTHER AFFECTED ARTICLES:

None

BACKGROUND:

The *Guide Specifications for LRFD Seismic Bridge Design* (SGS) includes provisions for the use of ductile cross frames (Type 2 design strategy) in steel I-Section girder bridges. The use of ductile cross frames is intended to protect substructures from transverse seismic demands by permitting inelastic response in the superstructure. Experts in the development and use of ductile cross frames have identified problems with the current SGS provisions. These problems include:

- Special detailing requirements necessary to justify a force reduction factor, *R*, of 4 that are not presented in the provisions
- Lack of testing and verification of a force reduction factor of 3 for "regular" ductile cross frames that do not incorporate special detailing currently permitted in the SGS
- Longitudinal bridge performance would likely require a Type 1 (plastic hinging in the substructure) response that is not addressed
- Special member slenderness requirements are more restrictive than those presented in Section 7 necessary to accommodate large ductility in the cross frames
- Special detailing provisions of shear stud connectors on girders and top horizontal members of ductile cross frames
- Research limitations that would not permit the use of ductile cross frames on bridges with irregular geometry, skew, or horizontal curvature
- Lack of reliable overstrength factors for ductile cross frames necessary to design capacity protected substructure elements in the transverse direction

Owner approval has been and remains a requirement for the use of ductile cross frames. The proposed changes do not preclude the use of ductile cross frames but do require that case-specific design criteria and design methodology be developed when using ductile cross frames. Future development of ductile cross frames may be incorporated into future editions of the SGS as the design guidelines become available.

The changes proposed herein address the shortcomings in the SGS that may result in unacceptable performance when using ductile cross frames.

ANTICIPATED EFFECT ON BRIDGES:

Improved seismic response and clarification to the use and implementation of the Type 2 seismic strategy for steel I-Section slab-on-steel-girder bridges.

REFERENCES:

Carden, L.P., Buckle, I., and Itani, A., 2007, “Transverse Displacement Capacity and Stiffness of Steel Plate Girder Bridge Superstructures for Seismic Loads,” Journal of Constructional Steel Research, Vol. 63, 1546-1559.

Carden, L.P., A.M. Itani, and I. Buckle. 2006. "Seismic Performance of Steel Girder Bridges with Ductile End Cross Frames Using Single Angle X Braces," Journal of Structural Engineering, American Society of Civil Engineers, Reston, VA.

Carden, L.P., F. Garcia-Alverez, A.M. Itani, and I. Buckle. 2006. "Cyclic Behavior of Single Angles for Ductile End Cross Frames," Engineering Journal, American Institute of Steel Construction, Chicago, IL.

OTHER:

None

2013 AASHTO BRIDGE COMMITTEE AGENDA ITEM: 4

SUBJECT: LRFD Bridge Design Specifications: Section 3, Article C3.6.1.3.1 (WAI 37)

TECHNICAL COMMITTEE: T-5 Loads

☐ **REVISION** ☒ **ADDITION** ☐ **NEW DOCUMENT**

☒ **DESIGN SPEC** ☐ **CONSTRUCTION SPEC** ☐ **MOVABLE SPEC**
☐ **MANUAL FOR BRIDGE EVALUATION** ☐ **SEISMIC GUIDE SPEC** ☐ **BRIDGE ELEMENT INSP GUIDE**
☐ **OTHER**

DATE PREPARED: 12/5/12
DATE REVISED: 4/12/13

AGENDA ITEM:

Item #1

In Article C3.6.1.3.1, add the following between the existing 4th and 5th paragraphs:

The HL93 live load model was found to be appropriate for global analysis of long-span bridges (Nowak 2010). In general, the design lane load portion of the HL-93 design load, which is the major contributor to live load force effects for long loaded lengths, is conservative. The conservatism is generally acceptable since members with long loaded lengths typically have much larger dead load than the live load. The conservatism could be somewhat less where the dead load has been mitigated, such as with cambered stiffening trusses on suspension bridges.

Item #2

Add the following reference to Article 3.16:

Nowak, A.S., M. Lutomirska and F.I. Sheikh Ibrahim. 2010. "The Development of Live Load for Long Span Bridges," Bridge Structures. IOS Press, Amsterdam, Vol. 6, 2010, pp. 1-7.

OTHER AFFECTED ARTICLES:

None

BACKGROUND:

None

ANTICIPATED EFFECT ON BRIDGES:

None

REFERENCES:

Nowak, A.S., M. Lutomirska and F.I. Sheikh Ibrahim. 2010. "The Development of Live Load for Long Span Bridges," Bridge Structures. IOS Press, Amsterdam, Vol. 6, 2010, pp. 1-7. (also http://iospress.metapress.com/content/65278l2663530375/fulltext.pdf).

OTHER:

None

2013 AASHTO BRIDGE COMMITTEE AGENDA ITEM: 5

SUBJECT: LRFD Bridge Design Specifications: Section 4, Various Articles (WAI 48)

TECHNICAL COMMITTEE: T-5 Loads

☒ **REVISION** ☒ **ADDITION** ☐ **NEW DOCUMENT**

☒ **DESIGN SPEC** ☐ **CONSTRUCTION SPEC** ☐ **MOVABLE SPEC**
☐ **MANUAL FOR BRIDGE EVALUATION** ☐ **SEISMIC GUIDE SPEC** ☐ **BRIDGE ELEMENT INSP GUIDE**
☐ **OTHER**

DATE PREPARED: 11/2012
DATE REVISED: 5/7/13

AGENDA ITEM:

Item #1

In Article 4.6.2.2.1 after the 13th paragraph, insert the following:

The term *L* (length) shall be determined for use in the live load distribution factor equations given in Articles 4.6.2.2.2 and 4.6.2.2.3 as shown in Table 4.6.2.2.1-2.

Move Table C4.6.2.2.1-1 from Commentary to Specifications after Table 4.6.2.2.1-1 and change the title as follows:

Table ~~C~~4.6.2.2.1-2—*L* for Use in Live Load Distribution Factor Equations

Revise the 16th paragraph of Article C4.6.2.2.1as follows:

~~Table C4.6.2.2.1-1 describes how the term *L* (length) may be determined for use in the live load distribution factor equations given in Articles 4.6.2.2.2 and 4.6.2.2.3.~~ The value of *L* to be used for positive and negative moment distribution factors will differ within spans of continuous girder bridges as will the distribution factors for positive and negative flexure.

Item #2

Renumber existing Table 4.6.2.2.1-2 as Table 4.6.2.2.1-3. Also renumber the references to this table.

Item #3

Revise Article 4.6.2.2.2d as follows:

The live load flexural moment for exterior beams may be determined by applying the live load distribution factor, *g*, specified in Table 4.6.2.2.2d-1. However if the girders are not equally spaced and *g* for the exterior girder is a function of $g_{interior}$, $g_{interior}$ should be based on the spacing between the exterior and first-interior girder.

The distance, d_e, shall be taken as positive if the exterior web is inboard of the interior face of the traffic railing and negative if it is outboard of the curb or traffic barrier. However, if a negative value for d_e falls outside the range of applicability as shown in Table 4.6.2.2.2.d-1 d_e should be limited to -1.0.

In steel beam-slab bridge cross-sections with diaphragms or cross-frames, the distribution factor for the exterior beam shall not be taken to be less than that which would be obtained by assuming that the cross-section deflects and rotates as a rigid cross-section. The provisions of Article 3.6.1.1.2 shall apply.

Revise the 1st and 2nd paragraphs of Article C4.6.2.2.2d as follows:

T~~his additional investigation is required because t~~he distribution factor for girders in a multigirder cross-section, Types "a," "e," and "k" in Table 4.6.2.2.1-1, was determined without consideration of diaphragm or cross-frames, or parapets. ~~The recommended procedure is an interim provision until research provides a better solution.~~ Some research shows a minimal contribution to load transfer from diaphragms or cross-bracing and resultant increase in force effects in external girders. However, reactions may be calculated using a ~~The~~ procedure ~~outlined in this section is~~ similar to the ~~same as the~~ conventional approximation for loads on piles as shown below.

$$R = \frac{N_L}{N_b} + \frac{X_{ext}\sum^{N_L} e}{\sum^{N_b} x^2} \qquad \text{(C4.6.2.2.2d-1)}$$

where:

R = reaction on exterior beam in terms of lanes
N_L = number of loaded lanes under consideration
e = eccentricity of a design truck or a design lane load from the center of gravity of the pattern of girders (ft)
x = horizontal distance from the center of gravity of the pattern of girders to each girder (ft)
X_{ext} = horizontal distance from the center of gravity of the pattern of girders to the exterior girder (ft)
N_b = number of beams or girders

Item #4

Revise Article 4.6.2.2.3c as follows:

Shear in ~~the exterior beam at the obtuse corner of the~~ bridge girders shall be adjusted when the line of support is skewed. The value of the correction factor shall be obtained from Table 4.6.2.2.3c-1~~. It is~~ and applied to the live load distribution factors, g, specified in ~~Table 4.6.2.2.3a-1 for interior beams and in~~ Table 4.6.2.2.3b-1 for exterior beams at the obtuse corner of the span, and from Table 4.6.2.2.3a-1 for interior beams. If the beams are well connected and behave as a unit, only the exterior and first interior beam need to be adjusted. The shear correction factors should be applied between the point of support at the obtuse corner and mid-span, and may be decreased linearly to a value of 1.0 at mid-span, regardless of end condition. This factor should not be applied in addition to modeling skewed supports.

~~In determining the end shear in multibeam bridges, the skew correction at the obtuse corner shall be applied to all the beams.~~

In Table 4.6.2.2.3c-1, revise the 2nd row as follows:

Cast-in-Place Concrete Multicell Box	d	For exterior girder: $1.0+\left(0.25+\frac{12.0L}{70d}\right)\tan\theta$ For first interior girder: $1.0+\left(0.042+\frac{12.0L}{420d}\right)\tan\theta$	$0° < \theta \le 60°$ $6.0 < S \le 13.0$ $20 \le L \le 240$ $35 \le d \le 110$ $N_c \ge 3$

Revise Article C4.6.2.2.3c as follows:

Verifiable correction factors are not available for cases not covered in Table 4.6.2.2.3c-1, including large skews and skews in combination with curved bridge alignments. When torsional force effects due to skew become significant; load distribution factors are inappropriate.

The equal treatment of all beams in a multibeam bridge (box beams and deck girders) is conservative regarding positive reaction and shear. The contribution from transverse post-tensioning is conservatively ignored. However, it is not necessarily conservative regarding uplift in the case of large skew and short exterior spans of continuous beams. A supplementary investigation of uplift should be considered using the correction factor from Table 4.6.2.2.3c-1, i.e., the terms other than 1.0, taken as negative for the exterior beam ~~at~~ on the acute corner.

Item #5

Revise Article 4.9 References as follows:

Zokaie, T. 1998, 1999, 2000, 2012-13. Private Correspondence.

OTHER AFFECTED ARTICLES:

None

BACKGROUND:

The *AASHTO Manual for Bridge Evaluation* refers to load distribution in Section 4 of the *AASHTO LRFD Bridge Design Specifications.* This ballot item contains clarifications for application of the *Design Specifications* to existing as well as new bridges.

Items 1 and 2: In Article 4.6.2.2.1-1, the last two paragraphs of Commentary on the definition of span length '*L*' and Table C4.6.2.2.1-1 first appeared in the '96-'97 Interims. Since then, the definitions have become more widely accepted and thus more appropriate to be specified rather than suggested in the Commentary. These items are to ensure that appropriate values for span length are used to determine approximate load distribution factors.

Item 3: The purpose of this item is to remove the rigid cross-section check from structures that don't behave as rigid structures.

Item 4: This item clarifies the intent of NCHRP Report 12-26 *Distribution of Wheel Loads on Highway Bridges*. Skew factors are needed to increase the shear force effects not just at abutment supports when using approximate load distribution analysis, but also at intermediate bents. Skew factors are necessary for all girders when the cross-section does not deflect in a rigid manner, but unnecessary for interior girders if well-connected. The effectiveness of connectivity is dependent on the geometry and details used, and left to the judgment of the engineer.

ANTICIPATED EFFECT ON BRIDGES:

None

REFERENCES:

Davis, R. and Wallace, M. "Skew Parameter Studies, Volumes 1 & 2, California Department of Transportation, October 1976.

Zokaie, T., T. A. Osterkamp, and R. A. Imbsen. 1991. Distribution of Wheel Loads on Highway Bridges, NCHRP Report 12-2611. Transportation Research Board, National Research Council, Washington, DC.

OTHER:

Thanks to MNDOT, NCDOT, and TXDOT for submitting comments.

The previous draft of this ballot item proposed clarification of e_g as being the distance between the centers of gravity of the basic beam and deck for composite girders, and zero for noncomposite girders. This proposal was pulled after comments were received that some engineers feel some composite strength is obtainable even without studs or vertical bars extending into the deck.

2013 AASHTO BRIDGE COMMITTEE AGENDA ITEM: 6

SUBJECT: LRFD Bridge Design Specifications: Section 5, Articles 5.3, 5.8.3.6.3 & 5.8.6.4 (WAI 130-11)

TECHNICAL COMMITTEE: T-10 Concrete

☒ **REVISION** ☒ **ADDITION** ☐ **NEW DOCUMENT**

☒ **DESIGN SPEC** ☐ **CONSTRUCTION SPEC** ☐ **MOVABLE SPEC**
☐ **MANUAL FOR BRIDGE EVALUATION** ☐ **SEISMIC GUIDE SPEC** ☐ **BRIDGE ELEMENT INSP GUIDE**
☐ **OTHER**

DATE PREPARED: 10/9/11
DATE REVISED: 4/3/13

AGENDA ITEM:

Item #1

Revise the definition of A_ℓ in Article 5.3 as follows:

A_ℓ = area of longitudinal torsion reinforcement in ~~the exterior web of the~~ a box girder (in.^{2}); area of longitudinal column reinforcement (in.^{2}) (5.8.3.6.3) (5.11.5.2.1)

Item #2

Add the following as the last paragraph of Article 5.8.3.6.3:

A_ℓ shall be distributed around the outer-most webs and top and bottom slabs of the box girder.

Item #3

Add the following at the end of Article C5.8.3.6.3:

Torsion addressed in this Article, St. Venant's Torsion, causes an axial tensile force. In a nonprestressed beam this force is resisted by longitudinal reinforcement having an axial tensile strength of $A_l f_y$. This steel is in addition to the flexural reinforcement and is to be distributed uniformly around the perimeter so that the resultant acts along the axis of the member. In a prestressed beam, the same approach (providing additional reinforcing bars with strength $A_l f_y$) can be followed, or the longitudinal torsion reinforcement can be comprised of normal reinforcing bars and that portion of the longitudinal prestressing steel not required to provide cross-sectional flexural capacity at strength limit states.

For box girder construction, interior webs should not be considered in the calculation of the longitudinal torsion reinforcing required by this Article. The values of p_h and A_ℓ should be for the box shape defined by the outer-most webs and the top and bottom slabs of the box girder. In wide, multi-cell box girders, longitudinal stresses resulting from cross-section distortion may require more reinforcement that St. Venant's torsion. Special analyses could be performed to determine these longitudinal distortional stresses. Often, the longitudinal reinforcement to resist longitudinal stresses resulting from cross-section distortion is placed in the outer-most webs only.

The longitudinal tension due to torsion may be considered to be offset in part by compression at a cross-section resulting from longitudinal flexure, allowing a reduction in the longitudinal torsion steel in longitudinally compressed portions of the cross-section at strength limit states.

Item #4

Revise the definition of A_ℓ in Article 5.8.6.4 as follows:

A_ℓ = total area of longitudinal torsion reinforcement in ~~the exterior web of the~~ a box girder (in.^{2})

Revise the 5th paragraph of Article 5.8.6.4 as follows:

A_ℓ shall be distributed around the ~~perimeter of the closed stirrups~~ outer-most webs and top and bottom slabs of the box girder in accordance with Article 5.8.6.6.

OTHER AFFECTED ARTICLES:

None

BACKGROUND:

In Article 5.8.3.6.3, it isn't clear if the longitudinal torsion reinforcement A_ℓ should be distributed around the perimeter of the box, or just in the webs. Article 5.8.6.4 defines A_ℓ as being in the exterior web but it isn't clear if the value is to be split between two webs, or provided in both webs. This ballot item resolves the conflicting requirements and terminology--based on the rationale in the proposed new Commentary. ASBI assisted in developing this ballot item, although the provisions in Article 5.8.3 apply to all cellular cross-sections, and not just for segmental construction.

ANTICIPATED EFFECT ON BRIDGES:

Improved distribution of longitudinal reinforcement when torsion is to be considered.

REFERENCES:

AASHTO (2012) "*AASHTO LRFD Bridge Design Specifications*," American Association of State Highway and Transportation Officials, 6th Edition, Washington, DC, 2012.

OTHER:

None

2013 AASHTO BRIDGE COMMITTEE AGENDA ITEM: 7

SUBJECT: LRFD Bridge Design Specifications: Section 5, Articles 5.8.2.5, C5.8.2.5 & C5.8.2.7 (WAI 130-05)

TECHNICAL COMMITTEE: T-10 Concrete

☒ **REVISION** ☒ **ADDITION** ☐ **NEW DOCUMENT**

☒ **DESIGN SPEC** ☐ **CONSTRUCTION SPEC** ☐ **MOVABLE SPEC**
☐ **MANUAL FOR BRIDGE EVALUATION** ☐ **SEISMIC GUIDE SPEC** ☐ **BRIDGE ELEMENT INSP GUIDE**
☐ **OTHER**

DATE PREPARED: 10/9/11
DATE REVISED: 4/12/13

AGENDA ITEM:

<u>Item #1</u>

Revise Article 5.8.2.5 as follows:

~~Except for segmental post-tensioned concrete box girder bridges, w~~<u>W</u>here transverse reinforcement is required, as specified in Article 5.8.2.4, the area of steel shall satisfy:

$$A_v \geq 0.0316\sqrt{f'_c}\,\frac{b_v s}{f_y} \qquad (5.8.2.5.1)$$

where:

A_v = area ~~if a~~ <u>of</u> transverse reinforcement ~~with~~ <u>within</u> distance s (in.^{2})
b_v = width of web adjusted for the presence of ducts as specified in Article 5.8.2.9 (in)
s = spacing of transverse reinforcement (in.)
f_y = yield strength of transverse reinforcement (ksi) ≤ 100 ksi

~~For segmental post-tensioned concrete box girder bridges, where transverse reinforcement is required, as specified in Article 5.8.6.5, the area of transverse reinforcement shall satisfy:~~

$$\cancel{A_v \geq 0.05\frac{b_w s}{f_y}} \qquad \cancel{(5.8.2.5\text{-}2)}$$

~~where:~~

~~A_v = area of a transverse shear reinforcement per web within distance s (in.2)~~
~~b_w = width of web (in.)~~
~~s = spacing of transverse reinforcement (in.)~~
~~f_y = yield strength of transverse reinforcement (ksi)~~

For segmental post-tensioned concrete box girder bridges, where transverse reinforcement is not required, as specified in Articles 5.8.6.5, the minimum area of transverse shear reinforcement per web shall not be less than the equivalent of two No. 4 Grade 60 reinforcement bars per foot of length.

Item #2

Add the following paragraph to the end of Article C5.8.2.5:

Previous editions of these Specifications had a minimum transverse reinforcement requirement for segmentally-constructed, post-tensioned concrete box girders that was not a function of concrete strength. Observations in shear testing of prestressed concrete girders indicate that higher strength concrete with lightly reinforced girders can fail soon after cracking. Since the limit for segmental structures was both less conservative and not tied to concrete strength, it was eliminated.

Item #3

Revise the 1st paragraph of Article C5.8.2.7 as follows:

Sections that are highly stressed in shear require more closely spaced reinforcement to provide crack control. Some research (NCHRP Report 579) indicates that in prestressed girders the angle of diagonal cracking can be sufficiently steep that a transverse bar reinforcement spacing of $0.8d_v$ could result in no stirrups intersecting and impeding the opening of a diagonal crack. A limit of $0.6d_v$ may be appropriate in some situations. Reducing the transverse bar reinforcement diameter is another approach taken by some.

OTHER AFFECTED ARTICLES:

None

BACKGROUND:

Item # 1 and #2

A role of providing minimum shear reinforcement is to ensure that the nominal shear capacity after cracking, V_n, is greater than that of shear cracking, V_{cr}. This cracking stress is predicted by codes in North America as being proportional to the square root of the cylinder compressive strength, and therefore so should be the minimum reinforcement requirements.

$A_v \geq 0.0316\sqrt{f'_c}\,\frac{b_v s}{f_y}$ (where f'_c is in ksi units); $A_v \geq \sqrt{f'_c}\,\frac{b_v s}{f_y}$ (where f'_c is in psi units)

Any coefficient in front of $\sqrt{f'_c}$ controls the level of conservatism of the minimum shear reinforcement requirements. Dr. Dan Kuchma who drafted this change proposal led a similar effort for ACI318 Building Code Requirements that results in the minimum requirements of:

$$A_v \geq 0.75\sqrt{f'_c}\,\frac{b_v s}{f_y}$$ (where f'_c is in psi units)

In this study, it was observed, as shown in Figure 1, that members with reinforcement amount of $40 \leq \rho_v f_y \leq 60\ psi$ (the minimum for segmental is 50 psi) were the most likely to be under their calculated strength. In this case, the ratio of V_{test}/V_{code} is by the AASHTO Simplified Method. The data used to make this assessment were taken from the identified references at the end of this document.

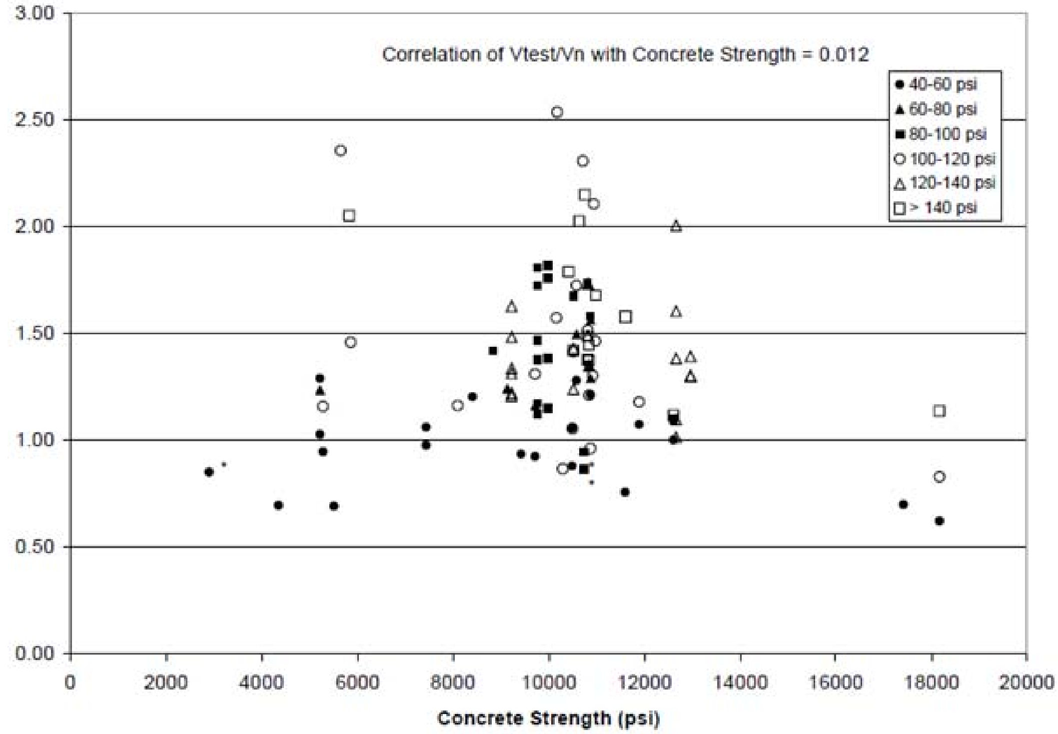


Figure 1 Variation of V_{test}/V_n with f'_c for all test results

Subsequent testing on prestressed concrete girders (NCHRP Report 579, 2007) illustrated the problem of shear reinforcement in lightly reinforced members yielding immediately after cracking. This provides additional justification for being conservative in setting minimum shear reinforcement levels and having them increase as a function of the tensile strength of the concrete. Thereby, leaving the minimum requirements for bridges proportional to $1\sqrt{f'_c}$ is justified rather than making them proportional to ACI's $0.75\sqrt{f'_c}$ (psi units).

Item #3

Dr. Kuchma made T-10 aware that the Canadian Code limit s_{max} to $0.7d_v$ in Eq. 5.8.2.7-1, rather than AASHTO's $0.8d_v$. Others cap v_u to $0.10f'_c$ rather than AASHTO's $0.125f'_c$. As test data is limited, T-10 instead added Commentary so the designer is made alert and can opt to use more conservative limits. The justification for being more conservative in selection of the maximum spacing of shear reinforcement for regions of members with lighter amounts of shear reinforcement stem from an examination of patterns of cracking in prestressed concrete girders (NCHRP Report 579) that indicates that the angle of diagonal cracking can be sufficiently steep that a spacing of 0.8dv of transverse reinforcement could result in essentially no stirrups intersecting and thereby controlling the opening of a diagonal cracking. See Figure 2.

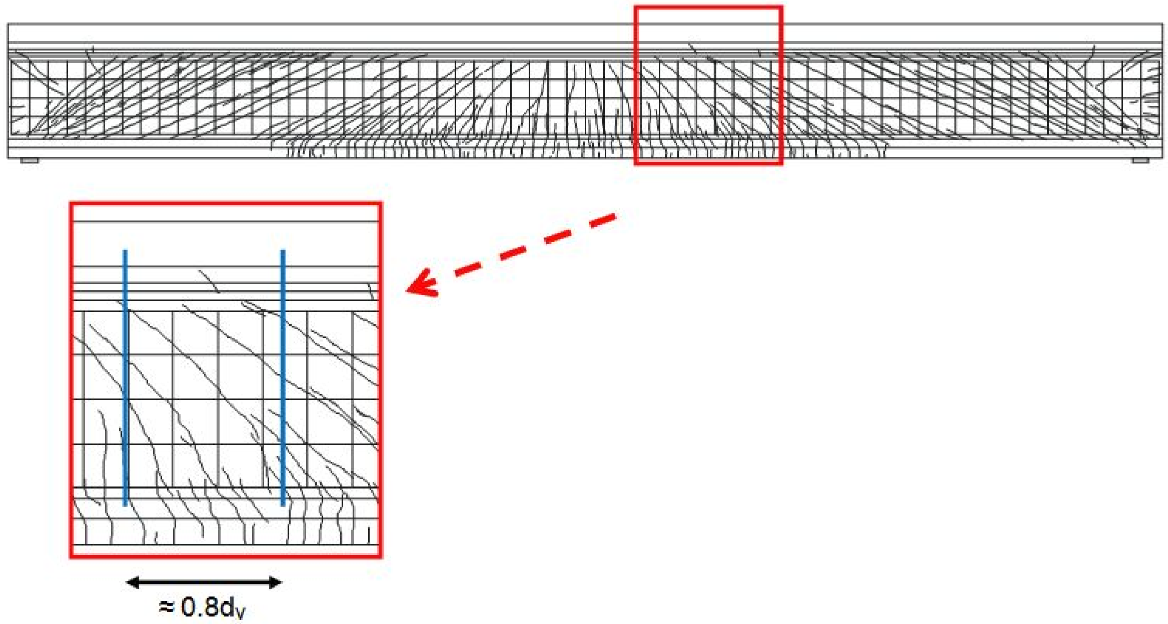


Figure 2: Spacing of transverse reinforcement and shear cracks

ANTICIPATED EFFECT ON BRIDGES:

Increase in minimum provided reinforcement in segmental provisions and decrease in spacing of lightly reinforced members designed in accordance to the general requirements. These two effects will increase the margin of safety against brittle shear failures.

REFERENCES:

AASHTO (2010) "AASHTO LRFD Bridge Design Specifications", American Association of State Highway and Transportation Officials, 5th Edition, with 2010 Interim Revisions, Washington, DC, 2010, 1822 pp.

ACI318-08 (2008) "Building Code Requirements for Structural Concrete (ACI 318-08) and Commentary (ACI 318R-08)", American Concrete Institute, Farmington Hills, Michigan, 2008, 521 pp.

ACI318-95 (1995) "Building Code Requirements for Reinforced Concrete (ACI318-95), and Commentary (ACI318R-95)", American Concrete Institute, Farmington Hills, Michigan

NCHRP Report 579 (2007) National Cooperative Highway Research Program Report 579 "Application of the LRFD Bridge Design Specifications to High-Strength Structural Concrete: Shear Provisions", http://onlinepubs.trb.org/onlinepubs/nchrp/nchrp_rpt_579.pdf , 206 pp., 2007.

List of Papers Reporting Tests on Lightly Reinforced Members Young-Soo Yoon, William D. Cook, and Denis Mitchell, "Minimum Shear Reinforcement in Normal, Medium, and High-Strength Concrete Beams", ACI Structural Journal, v.93, No.5, 1996, pp.576-584.

Michael Collins, Daniel Kuchma, "How Safe Are Our Large, Lightly Reinforced Concrete Beams, Slabs, and Footings?, ACI Structural Journal, July-August 1999, pp. 282-290.

John J. Roller, and Henry G. Russell, "Shear Strength of High-Strength Concrete Beams with Web Reinforcement", ACI Structural Journal, v.87, No.2, 1990, pp.191-198.

Kaiss F. Sarsam and Janan M.S. Al-Musawi, Shear Design of High and Normal Strength Concrete Beams with Web Reinforcement, ACI Structural Journal, Nov-Dec. 1992, pp. 658-664.

Paul Y. L. Kong, and B. Vijaya Rangan, "Shear Strength of High" -688.

Mark K. Johnson, and Julio A. Ramirez, "Minimum Shear Reinforcement in Beams with Higher Strength Concrete", ACI Structural Journal, v.86, No.4, 1989, pp.376-382.

Guney Ozcebe, Ugur Ersoy, and Tugrul Tankut, "Evaluation of Minimum Shear Reinforcement Requirements for Higher Strength Concrete", ACI Structural Journal, v.96, No.3, 1999, pp.361-368.

Dino Angelakos, M.A.Sc. Thesis, Department of Civil Engineering, University of Performance Concrete Beams", ACI Structural Journal, v.95, No.6, 1998, pp.677Toronto, 1999.

OTHER:

None

2013 AASHTO BRIDGE COMMITTEE AGENDA ITEM: 8

SUBJECT: LRFD Design Specifications: Section 5, Articles 5.8.3.6.2 & C5.8.3.6.2 (WAI 130-10)

TECHNICAL COMMITTEE: T-10 Concrete

☒ **REVISION** ☒ **ADDITION** ☐ **NEW DOCUMENT**

☒ **DESIGN SPEC** ☐ **CONSTRUCTION SPEC** ☐ **MOVABLE SPEC**
☐ **MANUAL FOR BRIDGE EVALUATION** ☐ **SEISMIC GUIDE SPEC** ☐ **BRIDGE ELEMENT INSP GUIDE**
☐ **OTHER**

DATE PREPARED: 10/26/11
DATE REVISED:

AGENDA ITEM:

Item #1

Revise the definition in Article 5.8.3.6.2 as follows:

θ = angle of crack <u>or diagonal compression</u> as determined in accordance with the provisions of Article 5.8.3.4 with the modifications to the expressions for v and V_u herein (degrees)

Item #2

Add the following at the end of Article 5.8.3.6.2

<u>While the general shear design procedure in Article 5.8.3.4.2 provides a best possible estimate of strain, it is acceptable to assume a strain of 0.0024 for prestressed concrete members and 0.0045 for reinforced concrete members.</u>

Item #3

Add new Commentary Article C5.8.3.6.2

<u>*C5.8.3.6.2*</u>

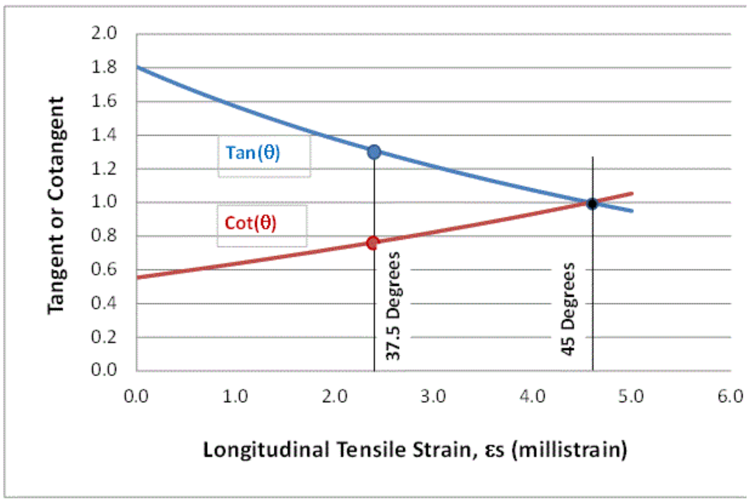
<u>The assumed strains suggested as an approximation for the procedure of Article 5.8.3.4.2 imply the angles of diagonal compression theta, of 37.5 and 45 degrees, respectively.</u>

OTHER AFFECTED ARTICLES:

None

BACKGROUND:

In the AASHTO Standard Specifications (AASHTO, 2004), the angle of diagonal compression is taken to be 45 degrees. The general design provisions of the first edition of the AASHTO LRFD Bridge Design Specifications introduced the use of a variable angle truss model for calculating the required levels of shear and longitudinal reinforcements. The equilibrium relationships presented in Equations 5.8.3.6.2-1 and 5.8.3.6.3-1 indicate that the flatter the angle of diagonal compression (θ), the lower the demand for shear reinforcement and the greater the demanded for longitudinal reinforcement. While the general shear design procedure in Article 5.8.3.4.2 of the current AASHTO LRFD Specifications (AASHTO, 2010) provide a best possible estimated of the angle of diagonal compression θ that considers compatibility, inelastic constitutive relationships, and equilibrium, assuming other reasonable values for θ is also acceptable. The traditional angle of 45 degrees is acceptable to use for reinforced concrete members and 37.50 degrees is acceptable for the use for prestressed concrete members. Such an approach for assuming angles is permitted in Eurocode2 (EC2-02, 2002).



The influence of the angle of diagonal compression/cracking on the longitudinal and shear reinforcement demands is presented in the figure above. The lower curved line presents the cotangent of θ as a function of the strain in the longitudinal tension reinforcement. Comparing this versus to the cotangents associated with θ = 37.5 ($\varepsilon_s = 2.4 \times 10^{-3}$) and 45 degrees ($\varepsilon_s = 4.6 \times 10^{-3}$) provides for an assessment of how assuming this value impacts the required amount of shear reinforcement. For most reinforced concrete members, ε_s is going to be well less than 4.6 millistrain and therefore the assumption of 45 degrees will lead to a larger required amount of shear reinforcement than by using 5.8.3.4.2 to determine θ. For most prestressed concrete member, the longitudinal strain is likely to be less than 2.4 millistrains and thereby the assumption of 37.50 degrees will lead to a larger required amount of shear reinforcement than by using 5.8.3.4.2 to determine θ. The opposite is true for determining the required amount of longitudinal reinforcements.

ANTICIPATED EFFECT ON BRIDGES:

Modest change in relative amounts of required shear and longitudinal reinforcements.

REFERENCES:

AASHTO (2010) "AASHTO LRFD Bridge Design Specifications", American Association of State Highway and Transportation Officials, 5th Edition with 2010 Interim Revisions, Washington, DC, 2010, 1822 pp.

AASHTO (2004) "AASHTO Standard Specifications for Highway Bridges, 17th Edition", American Association of State Highway and Transportation Officials, Washington, DC, 2004, 1028 pp.

EC2-02 (2002) Commite European de Normalisation (CEN), "Eurocode 2: Design of Concrete Structures. Part 1-General Rules and Rules for Buildings," EN 1992-1, 2002, 211 pp.

OTHER:

None

2013 AASHTO BRIDGE COMMITTEE AGENDA ITEM: 9

SUBJECT: LRFD Bridge Design Specifications: Section 5, Articles C5.8.4.1, 5.8.4.2 & C5.8.4.2 (WAI 172)

TECHNICAL COMMITTEE: T-10 Concrete

☒ **REVISION**	☒ **ADDITION**	☐ **NEW DOCUMENT**
☒ **DESIGN SPEC**	☐ **CONSTRUCTION SPEC**	☐ **MOVABLE SPEC**
☐ **MANUAL FOR BRIDGE EVALUATION**	☐ **SEISMIC GUIDE SPEC**	☐ **BRIDGE ELEMENT INSP GUIDE**
	☐ **OTHER**	

DATE PREPARED: 1/20/13
DATE REVISED: 4/1/13

AGENDA ITEM:

Item #1

Delete the 11th paragraph of Article C5.8.4.1 as follows:

~~Composite section design utilizing full-depth precast deck panels is not addressed by these provisions. Design specifications for such systems should be established by, or coordinated with the Owner.~~

Item #2

Revise the last paragraph of Article 5.8.4.2 as follows:

For beams and girders, the longitudinal center-to-center pitch ~~spacing~~ of nonwelded interface shear connectors ~~the rows of interface shear transfer reinforcing bars~~ shall not exceed ~~24.0~~ 48.0 in.

Item #3

Add the following at the end of Article C5.8.4.2:

Recent research (Markowski et al. 2005, Tadros & Girgis, 2006, Badie & Tadros 2008, Sullivan et al. 2011) has demonstrated that increasing interface shear connector spacing from 24.0 to 48.0 in. has resulted in no deficiency in composite action for the same resistance of shear connectors per foot, and girder and deck configurations. These research projects have independently demonstrated no vertical separation between the girder top and the deck under cyclic or ultimate loads.

As the spacing of connector groups increases, the capacities of the concrete and grout in their vicinity become more critical and need to be carefully verified. This applies to all connected elements at the interface. Eqs. 5.8.4.1-2 and 5.8.4.1-3 are intended to ensure that the capacity of the concrete component of the interface is adequate. Methods to enhance that capacity, if needed, include use of high strength materials and of localized confinement reinforcement.

Item #4

Add the following references to the list of REFERENCES in Section 5:

Badie, S.S., and Tadros, M.K., "*Guidelines for Design, Fabrication and Construction of Full-Depth, Precast Concrete Bridge Deck Panel Systems*," *National Cooperative Highway Research Program (NCHRP)* Report 584, Transportation Research Board (TRB), Washington, D.C., 2008.

Tadros, M.K., and Girgis, A.F., "Concrete Filled Steel Tube Arch", Nebraska Department of Roads, SPR-P1 (04) P571, 2006.

Markowski, S.M.; Ehmke, F.G., Oliva; M.G.; Carter III, J.W.; Bank, L.C.; Russell, J.S.; Woods, S.; and Becker; R.; "Full-Depth, Precast, Prestressed Bridge Deck Panel System for Bridge Construction in Wisconsin," Proceeding of The PCI/National Bridge Conference, Palm Springs, CA, October 16-19, 2005.

Sullivan, S.R.; Wollman, C.L.R.; and Swenty, M.K., "Composite Behavior of Precast Concrete Bridge Deck-panel Systems," PCI Journal, Summer 2011, V. 56, No. 3, pp. 43-59.

OTHER AFFECTED ARTICLES:

None

BACKGROUND:

The proposed revisions were developed in the NCHRP 12-65 research project "*FULL-DEPTH, PRECAST-CONCRETE BRIDGE DECK PANEL SYSTEM,*" (NCHRP Report 584). Also, independent studies were conducted at the University of Nebraska and the University of Wisconsin to study connections of bridge decks with floor beams and stringer systems. These studies showed no detrimental effect when the connectors spacing increased from 24.0 to 48.0 in. However, when the spacing increased to 96.0 in., a slight separation between the deck and the girder top flange was observed near the failure load.

The current 24.0 in maximum spacing appears to be based on an empirical spacing of four times a slab thickness of 6.0 in. originally adopted in the AC1318-08 building code.

The current limits specified by Equations 5.8.4.1-2 and 5.8.1-3 are adequate to ensure satisfactory resistance to localized concrete crushing near the connector reinforcement. If necessary, confinement of the concrete at each cluster of connectors could be provided to enhance the capacity of the concrete portion of the interface connection. NCHRP Report 584 has demonstrated that precast concrete deck panels reinforced with 4#4 closed loops immediately around the grout pockets were adequate to overcome possible concrete compressive strength deficiency indicated by Equations 5.8.4.1-2 and 5.8.1-3. Other options to satisfy these two equations would be to increase the concrete strength in the pockets and to increase the pocket size.

The following references are cited in Chapter 17 of the AC1318-08, which give the background of the 24.0 in. spacing:

17.2 Kaar, P.H., Kriz, L.B.; and Hognestad, E., "Precast-Prestressed Concreted Bridges: (1) Pilot Tests of Continuous Girders," *Journal*, PCA Research and Development Laboratories, V.2, No. 2, May 1960, pp. 21-37.

17.3 Saemann, J.C., and Washa, G.W., "Horizontal Shear Connections between Precast beams and Cast-in-Place Slabs," ACI JOURNAL, *Proceedings* V. 61, No. 11, Nov. 1964, pp. 1383-1409. Also see discussion, ACI JOUNAL, June 1965.

17.4 Hanson, N.W., "Precast-Prestressed Concrete Bridges: Horizontal Shear Connections," *Journals*, PCA Research and Development Laboratories, V.2, No. 2, May 1960, pp. 38-58.

17.5 Grossfield, B., and Birnstiel, C., "Tests of T-Beams with Precast Webs and Cast-in-Place Flanges," ACI JOURNAL, *Proceedings* V. 59, No. 6, June 1962, pp 843-851.

17.6 Mast, R.F., "Auxiliary Reinforcement in Concrete Connections," *Proceedings*, ASCE, V. 94, No. ST6, June 1968, pp. 1485-1504.

ANTICIPATED EFFECT ON BRIDGES:

The proposed change allows for more user friendly details when accelerated construction requires use of prefabricated deck systems. Also, the proposed change will expedite deck removal and simplifies fabrication of the prefabricated deck panels.

REFERENCES:

See Item #4 Above

OTHER:

None

2013 AASHTO BRIDGE COMMITTEE AGENDA ITEM: 10

SUBJECT: LRFD Bridge Design Specifications: Section 5, Article 5.8.6.5 (WAI 130-07)

TECHNICAL COMMITTEE: T-10 Concrete

☒ **REVISION** ☐ **ADDITION** ☐ **NEW DOCUMENT**

☒ **DESIGN SPEC** ☐ **CONSTRUCTION SPEC** ☐ **MOVABLE SPEC**
☐ **MANUAL FOR BRIDGE EVALUATION** ☐ **SEISMIC GUIDE SPEC** ☐ **BRIDGE ELEMENT INSP GUIDE**
☐ **OTHER**

DATE PREPARED: 10/26/12
DATE REVISED:

AGENDA ITEM:

Revise the 1st paragraph of Article 5.8.6.5 as follows:

5.8.6.5 Nominal Shear Resistance

In lieu of the provisions of Article 5.8.3, the provisions herein ~~shall~~ may be used to determine the nominal shear resistance of post-tensioned concrete box girders in regions where it is reasonable to assume that plane sections remain plane after loading.

OTHER AFFECTED ARTICLES:

None

BACKGROUND:

The agenda item permits the use of the general modified-compression field theory (MCFT) shear-resistance provisions of Article 5.8.3, in addition to the more traditional and more conservative shear resistance provisions specifically for segmental post-tensioned concrete box girders of Article 5.8.6.5.

ANTICIPATED EFFECT ON BRIDGES:

This may increase the permitted load-carrying capacity of some segmental bridges. The serviceability limit state check in Article 5.8.5 will maintain a check on serviceability performance.

REFERENCES:

None

OTHER:

None

2012 AASHTO BRIDGE COMMITTEE AGENDA ITEM: 11

SUBJECT: LRFD Bridge Design Specifications: Section 5, Articles 5.10.9.3.4b & C5.10.9.3.4b (WAI 152)

TECHNICAL COMMITTEE: T-10 Concrete

☒ **REVISION** ☒ **ADDITION** ☐ **NEW DOCUMENT**

☒ **DESIGN SPEC** ☐ **CONSTRUCTION SPEC** ☐ **MOVABLE SPEC**
☐ **MANUAL FOR BRIDGE EVALUATION** ☐ **SEISMIC GUIDE SPEC** ☐ **COASTAL GUIDE SPEC**
☐ **OTHER**

DATE PREPARED: 12/28/11
DATE REVISED:

AGENDA ITEM:

Item#1

Revise Article 5.10.9.3.4b as follows:

5.10.9.3.4b ~~—Tie-Backs~~ Crack Control Behind Intermediate Anchors

Unless otherwise specified herein, bonded reinforcement shall be provided to tie-back at least 25 percent of the ~~intermediate anchorage unfactored stressing~~ tendon force ~~into the concrete section~~ behind the intermediate anchor into the concrete section at service limit states and any stage of construction. Stresses in this bonded reinforcement shall not exceed a maximum of 0.6 f_y or 36 ksi. If ~~permanent~~ compressive stresses are generated behind the anchor ~~from other loads~~, the amount of tie-back reinforcement may be reduced using Eq. 5.10.9.3.4b-1.

$$T_{ia} = 0.25 P_s - f_{cb} A_{cb} \qquad (5.10.9.3.4b\text{-}1)$$

where:

T_{ia} = the tie-back tension force at the intermediate anchorage (kip)

P_s = the ~~maximum unfactored~~ tendon force (es) at the anchorage ~~stressing force~~ (kip)

f_{cb} = the unfactored ~~dead load~~ minimum compressive stress in the region behind the anchor at service limit states and any stage of construction (ksi)

A_{cb} = the area of the continuing cross-section within the extensions of the sides of the anchor plate or blister, i.e., the area of the blister or rib shall not be taken as part of the cross-section (in^2)

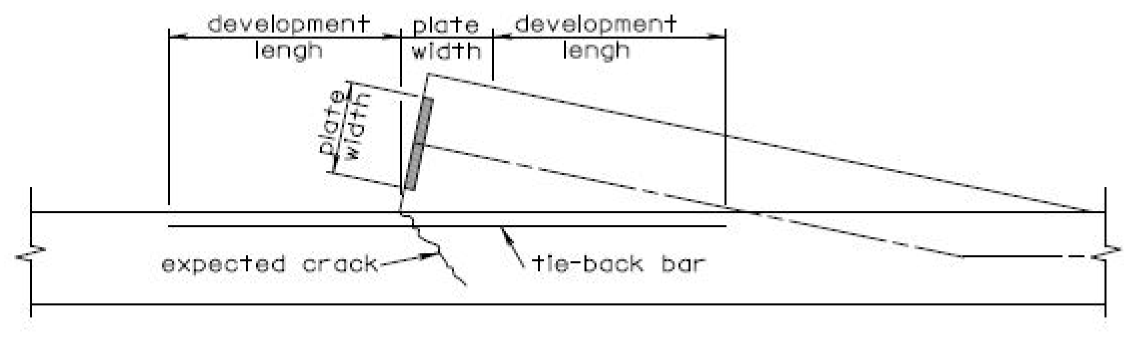
Tie-back reinforcement shall be placed no further than one plate width from the tendon axis. It shall be fully anchored so that the yield strength can be developed at ~~a distance of one plate width or half the length of the blister or rib ahead of the anchor as well as at the same distance behind the anchor~~ the base of the blister as well as a distance of one plate width ahead of the anchor. The centroid of this reinforcement shall coincide with the tendon axis, where possible. For blister and ribs, the reinforcement shall be placed in the continuing section near the face of the flange or web from which the blister or rib is projecting.

Item #2

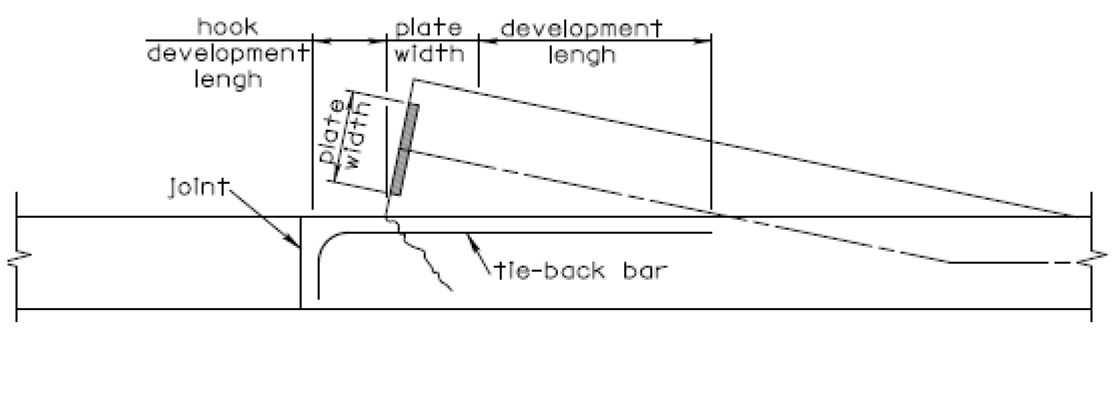
Add new Commentary Article C5.10.9.3.4b:

C5.10.9.3.4b

Cracks may develop in the slab and/or web walls immediately behind blisters and ribs due to stress concentrations caused by the anchorage force. Reinforcement proportioned to tie back 25 percent of the unfactored jacking force has been shown to provide adequate crack control (Wollmann, 1992). To ensure that the reinforcement is adequately developed at the crack location, a length of bar must be provided equal to one plate width plus one development length ahead of and one development length behind the anchor plate. This is illustrated in Figure 1(a). For precast segmental bridges in which the blister is close to a joint, a hook may be used to properly develop the bar. This is illustrated in Figure C5.10.9.3.4b -1(b).



(a) Condition with no nearby precast joints



(b) Condition with precast joint near anchor plate

Figure C5.10.9.3.4b-1—Required Length of Crack Control Reinforcement

The amount of tie-back reinforcing can be reduced by accounting for the compression in the concrete cross-section behind the anchor. Note that if the stress behind the anchor is tensile, the tie-back tension force will be $0.25P_s$ plus the tensile stress times A_{cb}. The area, A_{cb}, is illustrated in Figure C5.10.9.3.4b-2, along with other detailing requirements for the tie-back reinforcement.

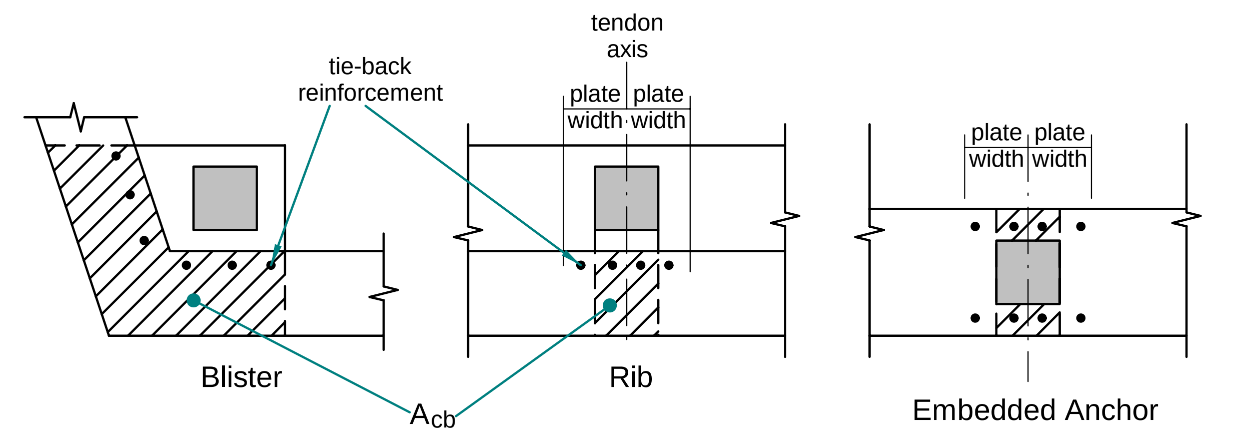


Figure C5.10.9.3.4b-2—Examples of A_{cb}

For bridges with multiple tendons and anchorages, the order of stressing should be taken into account and specified in the plans. For spans in which shorter tendons are wholly encompassed by longer tendons, it is frequently prudent to stress the longer tendons first.

OTHER AFFECTED ARTICLES:

None

BACKGROUND:

Wollmann (1992) performed a series of tests on intermediate anchorages as part of NCHRP 10-29 (Breen et al., 1994). Typically the first crack that developed in each test was a hairline crack at the base of the blister or rib. Wollmann varied the amount of tie back reinforcement, and found that providing 15 to 25% of the ultimate strength of the tendon in tie-back reinforcement resulted in well controlled cracks. Based on his tests, and previous work by Eibl and Ivanyi (1973), he proposed that 25% of the unfactored jacking force be tied back, with a working stress in the tie-back reinforcement of no more than $0.6f_y$.

ANTICIPATED EFFECT ON BRIDGES:

The changes proposed herein are primarily to clarify the existing provisions. It is anticipated that there will be less confusion about the amount of tie-back reinforcement required. Also, the required length of the tie-back reinforcement has been shortened and more clearly explained.

REFERENCES:

Breen, J.E., Burdet, O., Roberts, C., Sanders, D., and Wollmann, G. (1994). "Anchorage Zone Reinforcement for Post-Tensioned Concrete Girders," NCHRP Report 356, Transportation Research Board, Washington, D.C.

Eibl, J., and Ivanyi, G. (1973). "Innenverankerungen in Spannbetonbau", Deutscher Ausschuss fur Stahlbeton, Heft 223, pp. 35-39.

Wollmann, G.P. (1992). "Anchorage Zones in Post-Tensioned Concrete Structures", Ph.D. Dissertation, The University of Texas at Austin.

OTHER:

None

2013 AASHTO BRIDGE COMMITTEE AGENDA ITEM: 12

SUBJECT: LRFD Bridge Design Specifications: Section 5, Various Articles (WAI 145)

TECHNICAL COMMITTEE: T-10 Concrete

☒ **REVISION**	☒ **ADDITION**	☐ **NEW DOCUMENT**
☒ **DESIGN SPEC**	☐ **CONSTRUCTION SPEC**	☐ **MOVABLE SPEC**
☐ **MANUAL FOR BRIDGE EVALUATION**	☐ **SEISMIC GUIDE SPEC**	☐ **BRIDGE ELEMENT INSP GUIDE**
	☐ **OTHER**	

DATE PREPARED: 2/1/08
DATE REVISED: 4/27/13

AGENDA ITEM:

Item #1

Add or revise the following to Article 5.3 Notation:

A_{tr} = total cross-sectional area of all transverse reinforcement which is within the spacing s and which crosses the potential plane of splitting through the reinforcement being developed (in^2); area of concrete deck slab with transformed longitudinal deck reinforcement (in.^{2}) (5.11.2.1.3) (C5.14.1.4.3)

c_b = the smaller of distance from center of bar or wire being developed to the nearest concrete surface and one-half the center-to-center spacing of the bars or wires being developed (in.) (5.11.2.1.3)

k_{tr} = the transverse reinforcement index (5.11.2.1.3)

ℓ_s = ~~Class C tension lap splice length of column longitudinal reinforcement (in.)~~ lap splice length + s, based on assumed strut angle of 45° (in.) (5.11.5.2.1)

n = modular ratio = E_s/E_c or E_p/E_c; number of anchorages in a row; projection of base plate beyond the wedge hole or wedge plate, as appropriate (in.); number of bars or wires developed along plane of splitting; modular ratio between deck concrete and reinforcement (5.7.1) (5.10.9.6.2) (5.10.9.7.2) (5.11.2.1.3) (C5.14.1.4.3)

s = average spacing of mild steel reinforcement in layer closest to tension face (in.); spacing of reinforcing bars (in.); spacing of row of ties (in.); anchorage spacing (in.); center-to-center spacing of anchorages (in.); maximum center-to-center spacing of transverse reinforcement within ℓ_d, (in.); spacing of hanger reinforcing bars (in.) (5.7.3.4) (5.8.2.5) (5.8.4.1) (5.10.9.3.6) (5.10.9.6.2) (5.11.2.1.3) (5.13.2.5)

λ_{cf} = coating factor (5.11.2.1.1)

λ_{er} = excess reinforcement factor (5.11.2.1.1)

λ_{lw} = lightweight aggregate factor (5.11.2.1.1)

λ_{rc} = reinforcement confinement factor (5.11.2.1.1)

λ_{rl} = reinforcement location factor (5.11.2.1.1)

Item #2

Delete Article C5.11.1.1.

Item #3

Add the following as Paragraph 1 to Article C5.11.2:

Most of the provisions in the Article are based on ACI 318-08 and its attendant commentary. In addition, results of NCHRP Report 603 on Transfer, Development, and Splice Length for Strand/Reinforcement in High Strength Concrete (Ramirez and Russell, 2008) are incorporated to include applications with specified concrete strengths up to 15 ksi. The NCHRP 603 Report examined an extensive database of previous tests compiled by ACI Committee 408. Previous tests (Azizinamini et al. 1993, and 1999) had indicated that in the case of concrete with compressive strengths between 10 and 15 ksi, a minimum amount of transverse reinforcement was needed to ensure yielding of reinforcement splices of bottom bars with less than 12.0 in. of concrete placed below them. NCHRP Report recommended replacing the minimum transverse reinforcement with a development modification factor of 1.2. The bottom bar factor is not needed for epoxy coated bars, because of the single modification factor of 1.5. The bar size factor of 0.8 for No. 6 and smaller bars was recommended to be removed to generalize application to concrete strength higher than 10 ksi. The procedure described here is more conservative to use with higher steel strength than 60 ksi than recently published reports such as that by Hosny et al. (2012), and by Darwin et al. (2005).

Item #4

Revise Article 5.11.2.1.1 as follows:

5.11.2.1.1—Tension Development Length

The tension development length, ℓ_d, shall not be less than the product of the basic tension development length, ℓ_{db}, specified herein and the modification factor or factors specified in Articles 5.11.2.1.2 and 5.11.2.1.3. The tension development length shall not be less than 12.0 in., except for lap splices specified in Article 5.11.5.3.1 and development of shear reinforcement specified in Article 5.11.2.6.

The ~~basic~~ tension development length, ~~ℓ_{db}~~ ℓ_d, in in. shall be taken as:

$$\ell_d = \ell_{db}\left(\lambda_{rl} \times \lambda_{cf} \times \lambda_{lw} \times \lambda_{rc} \times \lambda_{er}\right) \qquad (5.11.2.1.1\text{-}1)$$

- ~~For No. 11 bar and smaller ....~~ $\cancel{\dfrac{1.25\,A_b f_y}{\sqrt{f'_c}}}$

  ~~but not less than~~ $\cancel{0.4\,d_b f_y}$

- ~~For No. 14 bars ....~~ $\cancel{\dfrac{2.70\,f_y}{\sqrt{f'_c}}}$

~~For No. 18 bars~~ $\cancel{\dfrac{3.5\,f_y}{\sqrt{f'_c}}}$

- ~~For deformed wire ....~~ $\cancel{\dfrac{0.95\,d_b f_y}{\sqrt{f'_c}}}$

in which:

$$\ell_{db} = 2.4 d_b \frac{f_y}{\sqrt{f'_c}} \qquad (5.11.2.1.1\text{-}2)$$

where:

ℓ_{db} = basic development length (in.)
λ_{rl} = reinforcement location factor
λ_{cf} = coating factor
λ_{lw} = lightweight aggregate factor
λ_{rc} = reinforcement confinement factor

$\underline{\lambda_{er}}$ = <u>excess reinforcement factor</u>
~~A_b = area of bar or wire (in.^{2})~~
f_y = specified yield strength of reinforcing bars <u>or wire</u> (ksi)
d_b = diameter of bar or wire (in.)
f'_c = specified compressive strength of concrete at 28 days, unless another age is specified (ksi)

<u>Modification factors shall be applied to the basic development length to account for the various effects specified herein. They shall be taken equal to 1.0 unless they are specified to increase ℓ_d in Article 5.11.2.1.2, or to decrease ℓ_d in Article 5.11.2.1.3.</u>

<u>Item #5</u>

Revise Article 5.11.2.1.2 as follows:

5.11.2.1.2—Modification Factors which Increase ℓ_d

The basic development length, ℓ_{db}, shall be multiplied by the following factor or factors, as applicable:

- For top horizontal ~~or nearly horizontal~~ reinforcement, ~~so~~ placed <u>such</u> that more than 12.0 in. of fresh concrete is cast below the reinforcement, $\underline{\lambda_{rl} =}$ ~~……………………………………1.4~~ <u>1.3.</u>

- <u>For bottom horizontal reinforcement, placed such that no more than 12.0 in. of concrete is cast below the reinforcement and f'_c is greater than 10 ksi, λ_{rl} = 1.3.</u>

- ~~For lightweight aggregate concrete where f_{ct} (ksi) is specified~~ $\cancel{\dfrac{0.22\sqrt{f'_c}}{f_{ct}} \geq 1.0}$

- For ~~all~~ lightweight concrete ~~where f_{ct} is not specified,~~ $\underline{\lambda_{lw} =}$ ~~……………………………~~ 1.3<u>.</u>

- ~~For sand-light weight concrete where fct is not specified ……………………………….. 1.2~~

~~Linear interpolation may be used between all lightweight and sand-lightweight provisions when partial sand replacement is used.~~

- For epoxy-coated bars with cover less than $3d_b$ or with clear spacing between bars less than $6d_b$, $\underline{\lambda_{cf} =}$ ~~………………………..~~ 1.5<u>.</u>

- For epoxy-coated bars not covered above, $\underline{\lambda_{cf} =}$ ~~…………………………………………~~1.2<u>.</u>

<u>Either the λ_{rl} factor for bottom reinforcement or the λ_{rl} factor for top reinforcement shall be applied as appropriate, but not both simultaneously. The product $\lambda_{rl} \times \lambda_{cf}$ need not be taken to be greater than 1.7.</u>

~~The product obtained when combining the factor for top reinforcement with the applicable factor for epoxy-coated bars need not be taken greater than 1.7.~~

<u>Item #6</u>

Revise Article 5.11.2.1.3 as follows:

5.11.2.1.3—*Modification Factors which Decrease ℓ_d*

The basic development length, ℓ_{db}, <u>specified in Article 5.11.2.1.1,</u> modified by the factors ~~as~~ specified in Article 5.11.2.1.2, <u>as appropriate,</u> may be multiplied by the following factor or factors~~, where~~:

- <u>The value of the confinement factor, λ_{rc} where, for the r</u>~~R~~einforcement being developed in the length under consideration, λ_{rc} satisfies the following: ~~is spaced laterally not less than 6.0 in. center-to-center, with not less than 3.0 in. clear cover measured in the direction of the spacing ……0.8~~

$$1.0 \geq \lambda_{rc} = \frac{d_b}{c_b + k_{tr}} \geq 0.4 \qquad (5.11.2.1.3\text{-}1)$$

in which:

$$k_{tr} = 40A_{tr}/(sn) \qquad (5.11.2.1.3\text{-}2)$$

where:

c_b = the smaller of the distance from center of bar or wire being developed to the nearest concrete surface and one-half the center-to-center spacing of the bars or wires being developed (in.)
k_{tr} = transverse reinforcement index
A_{tr} = total cross-sectional area of all transverse reinforcement which is within the spacing s and which crosses the potential plane of splitting through the reinforcement being developed (in.^{2})
s = maximum center-to-center spacing of transverse reinforcement within ℓ_d (in.)
n = number of bars or wires developed along plane of splitting

- For reinforcement being developed in the length under consideration that is confined laterally by reinforcement spaced such that $c_b \geq 2.5$ in., regardless of existence of stirrups, $\lambda_{rc} = 0.4$.
- Anchorage or development for the full yield strength of reinforcement is not required, or where reinforcement in flexural members is in excess of that required by analysis, ~~……………………… $\frac{(A_s\ required)}{(A_s\ provided)}$~~

$$\lambda_{er} = \frac{(A_s\ required)}{(A_s\ provided)} \qquad (5.11.2.1.3\text{-}3)$$

- ~~Reinforcement is enclosed within a spiral composed of bars of not less than 0.25 in. in diameter and spaced at not more than a 4.0 in. pitch …………………………………………………………0.75~~

Item #7

Revise Article 5.11.2.4.1 as follows:

5.11.2.4.1—Basic Hook Development Length

The development length, ℓ_{dh}, in in., for deformed bars in tension terminating in a standard hook specified in Article 5.10.2.1 shall not be less than:

- The product of the basic development length l_{hb}, as specified by Eq. 5.11.2.4.1-1, and the applicable modification factors, as specified in Article 5.11.2.4.2;
- 8.0 bar diameters; or
- 6.0 in.

Basic development length, ℓ_{hb}, for a hooked-bar ~~with yield strength, f_y, not exceeding 60.0 ksi~~ shall be taken as:

~~$l_{hb} = \frac{38.0 d_b}{\sqrt{f'_c}}$~~ ~~(5.11.2.4.1-1)~~

$$\ell_{hb} = \frac{38.0 d_b}{60.0}\left(\frac{f_y}{\sqrt{f'_c}}\right) \qquad \text{(5.11.2.4.1-1)}$$

where:

d_b = diameter of bar or wire (in.)
f'_c = specified compressive strength of concrete for use in design ~~at 28 days, unless another age is specified,~~ not to be taken greater than 15 ksi for normal weight concrete and 10 ksi for lightweight concrete (ksi)

Item #8

Revise Article 5.11.2.4.2 as follows:

5.11.2.4.2—Modification Factors

Basic hook development length, ℓ_{hb}, shall be multiplied by the following factor or factors, as applicable, ~~where~~ for:

- ~~Reinforcement has a yield strength exceeding 60.0 ksi……………~~ ~~$\frac{f_y}{60.0}$~~
- ~~Side cover for No. 11 bar and smaller, normal to plane of hook, is not less than 2.5 in., and 90° hook, cover on bar extension beyond hook not less than 2.0 in.…………..0.7~~
- ~~Hooks for No. 11 bar and smaller enclosed vertically or horizontally within ties or stirrup ties which are spaced along the full development length, ℓ_{dh}, at a spacing not exceeding $3d_b$…..……0.8~~
- Lightweight aggregate concrete, with a specified compressive strength not exceeding 10 ksi, is used~~……………………………………………………………~~, λ_{lw} = 1.3
- Epoxy-coated reinforcement is used~~………~~, λ_{cf} = 1.2
- For No. 11 bar and smaller, hooks with side cover normal to plane of the hook not less than 2.5 in., and for 90 deg hook with cover on the bar extension beyond hook not less than 2.0 in., λ_{rc} = 0.8
- For 90 deg hooks of No. 11 and smaller bars that are either enclosed within ties or stirrups perpendicular to the bar being developed, spaced not greater than $3d_b$ along the development length ℓ_{hd} of the hook; or enclosed within ties or stirrups parallel to the bar being developed spaced not greater than $3d_b$ along the length of the tail extension of the hook plus bend, λ_{rc} = 0.8
- For 180 deg hooks of No. 11 and smaller bars that are enclosed within ties or stirrups perpendicular to the bar being developed, spaced not greater than $3d_b$ along the development length ℓ_{hd}, of the hook = 0.8
- Anchorage or development of full yield strength is not required, or where reinforcement is provided in excess of that required by analysis~~…..~~ , $\lambda_{er} = \frac{(A_s\, required)}{(A_s\, provided)}$

Item #9

Add the following Commentary to Article 5.11.2.4:

C5.11.2.4

Article 5.11.2.4 was verified for specified concrete compressive strength up to 15 ksi in NCHRP Report 603 with the exception of the lightweight aggregate factor. The previous limit of 10 ksi has been retained for lightweight concrete. Based on the analysis of NCHRP Report 603 and of tests of additional specimens in the literature, the approach in the 318-11 ACI Code provision for anchorage of bars terminated with standard hooks, black and epoxy-coated, can be extended to normal weight concrete with compressive strengths of up to 15 ksi. A minimum amount of transverse reinforcement, at least No. 3 U bars at $3d_b$ spacing, is recommended in NCHRP Report 603 be provided to improve the bond strength of No. 11 and larger bars in tension anchored by means of standard hooks. A modification factor of 0.8 instead of the current previous factor of 0.7 for No. 11 and smaller hooks with side cover not less than 2.5 in., and for 90 degree hook with cover on bar extension beyond hook not less than 2.0 in., was found to be adequate. Similar to the position of ACI 318-11, hooks are not considered effective in developing bars in compression.

Item #10

In Article 5.11.2.5.1, revise the 3rd paragraph and replace Equations 1 and 2 as follows:

The basic development length, ℓ_{hd}, for welded deformed wire fabric, with not less than one cross wire within the development length at least 2.0 in. from the point of critical section, shall satisfy the larger of:

$$\ell_{hd} \geq 0.95 d_b \frac{f_y - 20.0}{\sqrt{f'_c}} \text{, or} \qquad (5.11.2.5.1\text{-}1)$$

$$\ell_{hd} \geq 6.30 \frac{A_w f_y}{s_w \sqrt{f'_c}} \qquad (5.11.2.5.1\text{-}2)$$

Item #11

In Article 5.11.5.2.1, replace the following definition:

ℓ_s = lap splice length + s, based on assumed strut angle of 45° (in.)

Item #12

Revise Article 5.11.5.3.1 as follows:

5.11.5.3.1—Lap Splices in Tension

The minimum length of lap for tension lap splices shall ~~not~~ be as required for Class A or B splice, but not less than ~~either~~ 12.0 in. ~~or the following for Class A, B or C splices~~, where:

Class A splice .... $1.0\ell_d$
Class B splice .... $1.3\ell_d$
~~Class C splice .... 1.7ℓd~~

The tension development length, ℓ_d, for the specified yield strength shall be taken in accordance with Article 5.11.2.

~~The class of lap splice required for deformed bars and deformed wire in tension shall be as specified in~~

~~Table 5.11.5.3.1-1.~~

Delete **Table 5.11.5.3.1-1—Classes of Tension Lap Splices**

Lap splices of deformed bars and deformed wire in tension shall be Class B splices except that Class A splices may be used where:

(a) the area of reinforcement provided is at least twice that required by analysis over the entire length of the splice; and
(b) one-half or less of the total reinforcement is spliced within the required lap length.

For splices having f_y > 75.0 ksi, transverse reinforcement satisfying the requirements of Article 5.8.2.5 in beams and Article 5.10.6.3 in columns shall be provided over the required development length.

Item #13

Add the following paragraph to the end of Article C5.11.5.3.1:

Tension lap splices were evaluated under NCHRP Report 603. Splices of bars in compression were not part of the experimental component of the research. Class C splices were eliminated based on the modifications to development length provisions. The modifications to Article 5.11.2.1 in 2013 contain several changes that eliminated many of the concerns regarding tension splices due to closely spaced bars with minimal cover. However, the development lengths, on which splice lengths are based, have in some cases increased. A two-level splice length was retained primarily to encourage designers to splice bars at points of minimum stress and to stagger splices to improve behavior of critical details, but does not reflect the increased strength of the splice.

Item #14

Add the following to Article 5.15—References:

Azizinamini, A., M. Stark, J. R. Roller, and S. K. Ghosh. 1993. "Bond Performance of Reinforcing Bars Embedded in Concrete," *ACI Structural Journal*, Vol. 90, No. 5, September-October 1993, pp. 554-561.

Darwin, D, Lutz, L, Zuo, J, "Recommended Provisions and Commentary on Development and Lap Splice Lengths for Deformed Reinforcing bars in Tension," ACI Structural Journal, Vol. 102, No. 6, Nov-Dec. 2005, pp. 892-900.

Hosny, A, Seliem HM, Rizkalla, SH, and Zia, P, "Development Length of Unconfined Conventional and High Strength Steel Reinforcing Bars, ACI Structural Journal, Vol. 109, No. 5, Sept-Oct. 2012, pp. 655-664.

OTHER AFFECTED ARTICLES:

None

BACKGROUND:

Article 5.4.2.1 limits the applicability of the specifications for concrete compressive strengths of 10 ksi or less unless physical tests are made to establish the relationships between concrete strength and other properties. A Ballot Item passed in 2012 (WAI 145A) extended the provision of Articles 5.11.2.1, 5.11.2.4, and 5.11.5.3.1 to 15.0 ksi. Also, the current provisions of the reinforcement development and splice length provisions are based on the ACI 318-89 Building Code, which has undergone considerable revisions up to the 2011 Edition.

A comprehensive article-by-article review of Section 5 of these Specifications pertaining to transfer, development, and splice length for strand, reinforcing bars and reinforcing wire was performed under NCHRP Project 12-60, and described in Report 603. This review was conducted to identify all the provisions that directly or indirectly had to be revised to extend their use to specified concrete strengths up to 15 ksi.

The proposed recommendations combine the recommendations of Report 603 on Transfer, Development, and Splice Length for Strand/Reinforcement in High Strength Concrete (Ramirez and Russell, 2008) and ACI 318-11 to include applications with specified concrete strengths up to 15 ksi.

An extensive comparison among development lengths calculated by current AASHTO, ACI 318-11 and the proposed revisions is appended as Attachment A.

ANTICIPATED EFFECT ON BRIDGES:

In general, ACI 318-11 is yielding longer development lengths than the ACI 318-89 Code. Experimental data supports the change.

REFERENCES:

NCHRP 12-60, Report 603. Transfer, Development, and Splice Length for Strand/ Reinforcement in High strength Concrete.

ACI 318-11, Building Code Requirements for Reinforced Concrete, American Concrete Institute, Box 19150, Redford Station, Detroit, Michigan 48219.

Hosny, A, Seliem HM, Rizkalla, SH, and Zia, P, "Development Length of Unconfined Conventional and High Strength Steel Reinforcing Bars, ACI Structural Journal, Vol. 109, No. 5, Sept-Oct. 2012, pp. 655-664.

Darwin, D, Lutz, L, Zuo, J, "Recommended Provisions and Commentary on Development and Lap Splice Lengths for Deformed Reinforcing bars in Tension," ACI Structural Journal, Vol.102, No. 6, Nov. Dec. 2005, pp. 892-900.

OTHER:

None

Agenda 12 (WAI 145) Development of Mild Steel – Attachment A

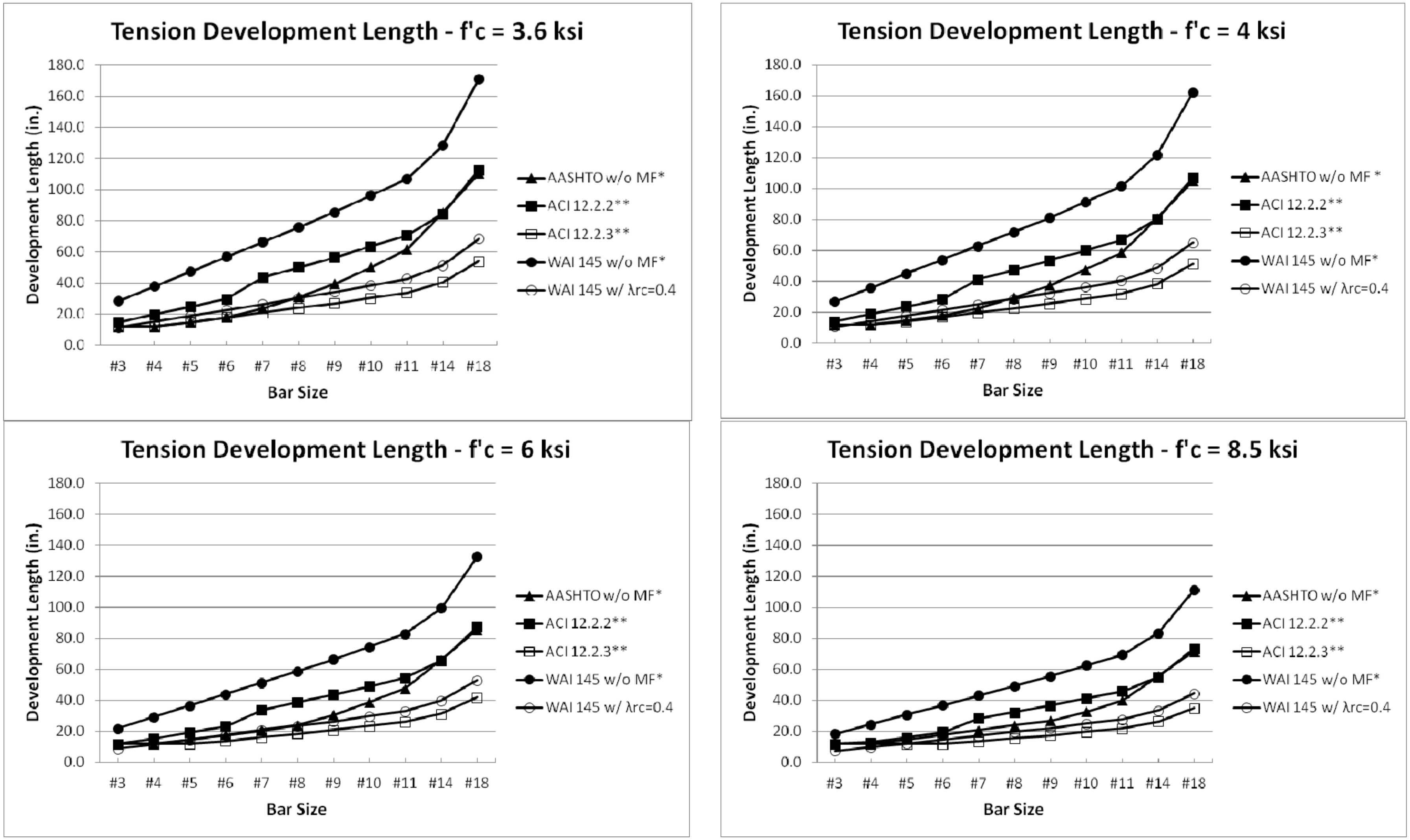
- Page 1 - Summary

- Page 2 - 3
  Graphs illustrate the tension development lengths from the different methods listed below for (8) concrete strengths (3.6ksi, 4ksi, 6ksi, 8ksi, 9ksi, 10ksi, 12ksi, and 15ksi) and various bar sizes.
  - AASHTO 6th Edition (without any modification factor)
  - ACI 318-11
    - ✓ Section 12.2.2 (using Tables - preselected confinement terms included)
    - ✓ Section 12.2.3 (using Equation 12-1 with confinement term = 2.5 and ψs = 0.8 for bar sizes #6 or smaller)
  - WAI 145
    - ✓ Without any modification factor (or confinement factor = 1.0)
    - ✓ Use confinement factor = 0.4 (similar to ACI 318-11 Section 12.2.3 method)

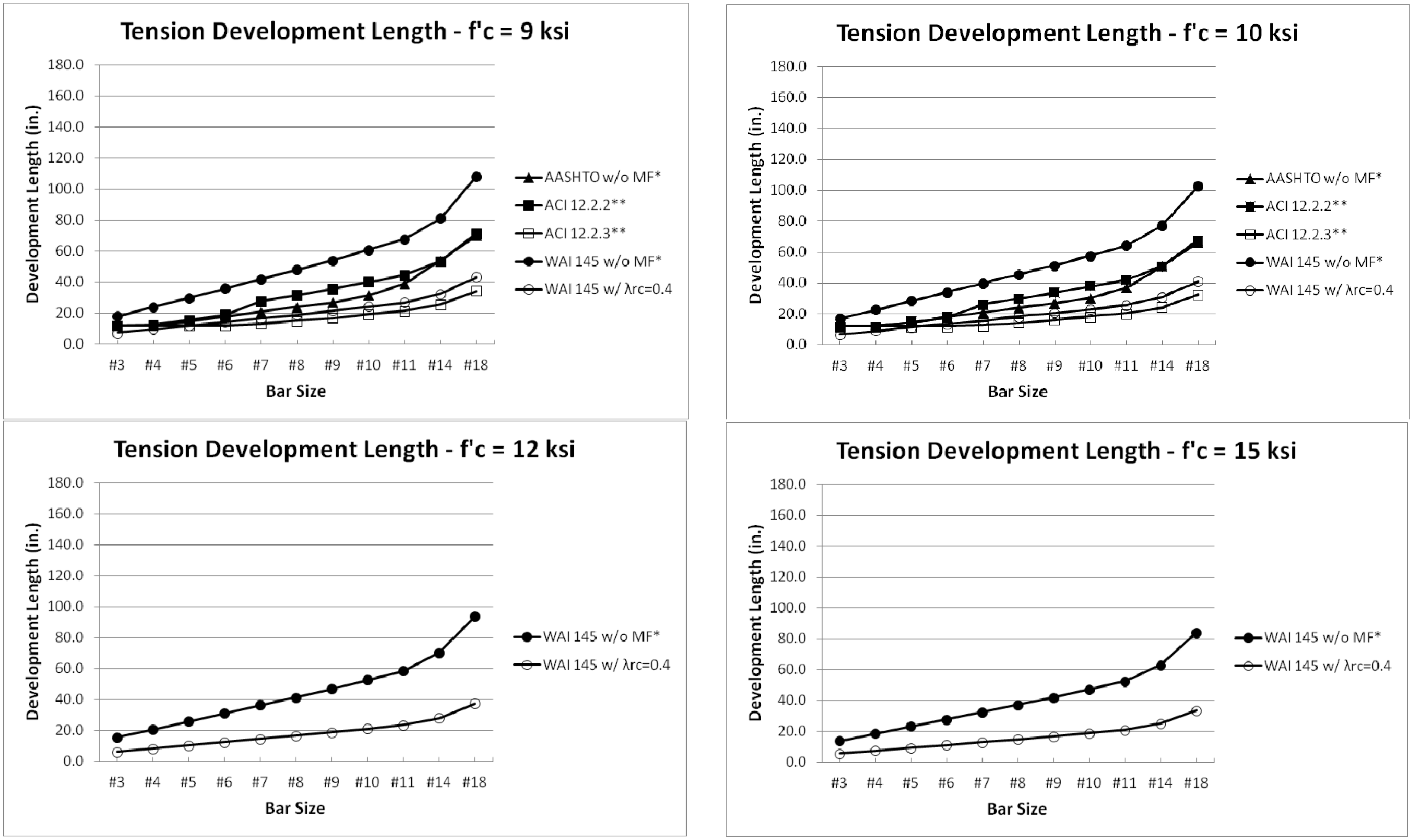
- Page 4 - 5
  Graphs illustrate the tension development lengths for each of the methods listed above with various concrete strengths (3.6ksi, 4ksi, 6ksi, 8ksi, 9ksi, 10ksi, 12ksi, and 15ksi) and bar sizes.

- Page 6-7
  Design examples and comparison of results

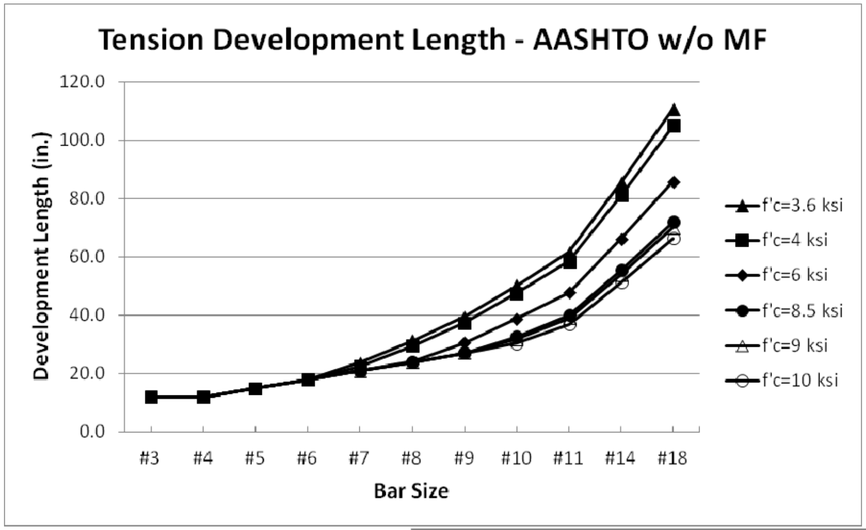
- An excel worksheet, "WAI 145 excel worksheet", has been developed to calculate the development length and lap splice length for all methods listed above and for any design conditions. The excel worksheet can be downloaded from the link below.
  http://www.dotd.la.gov/highways/project_devel/design/bridge_design/documents.aspx?key=2

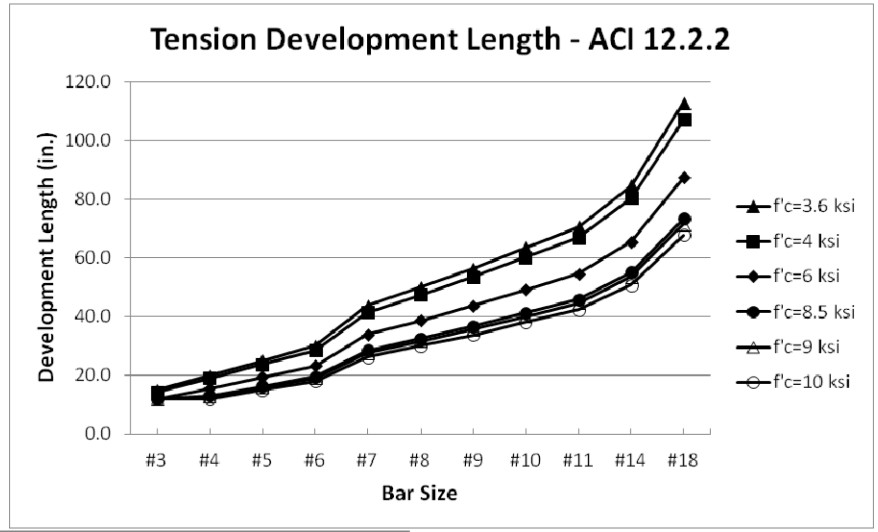


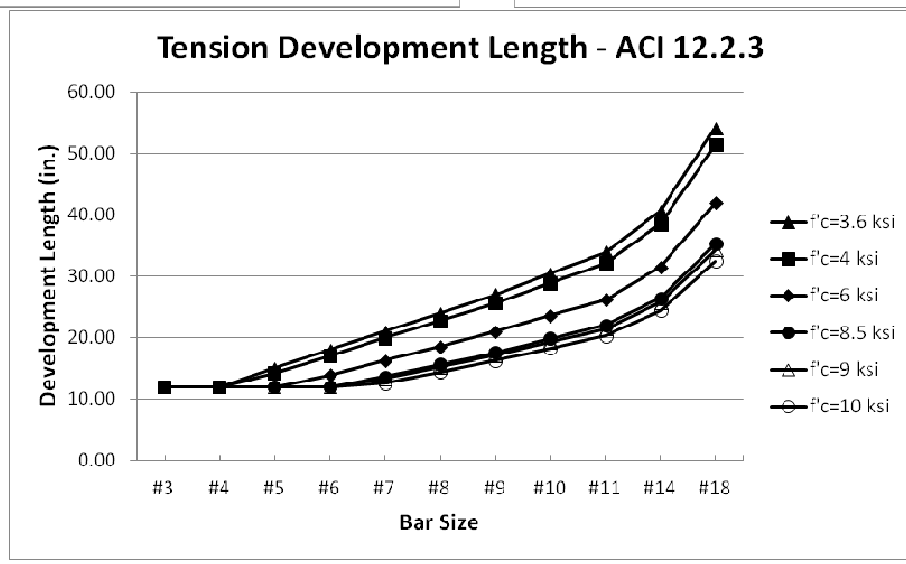
MF* - Modification Factor, ACI 12.2.2** – Preselected Confinement Term $\frac{c_b+K_{tr}}{d_b}$ Included, ACI 12.2.3** - Confinement Term $\frac{c_b+K_{tr}}{d_b}$ = 2.5. Ψ_s = 0.8 for #6 bars or smaller



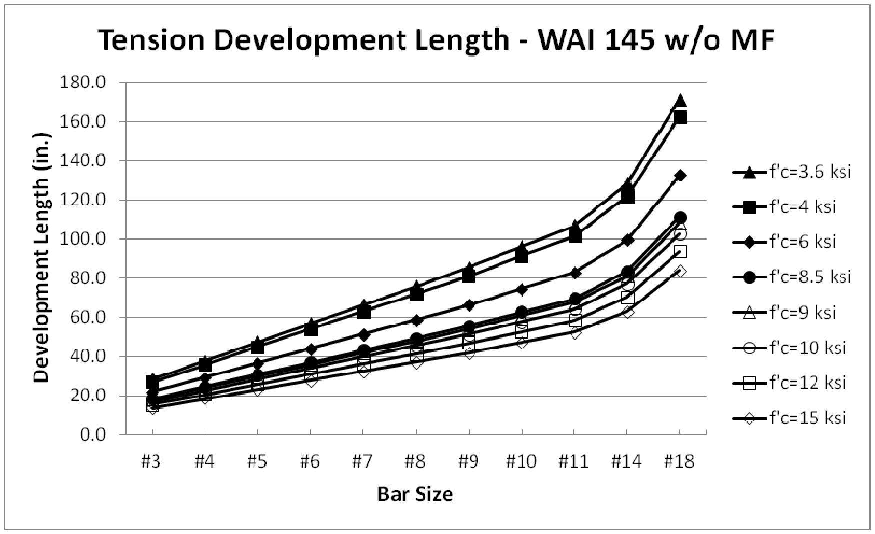
MF* - Modification Factor, ACI 12.2.2** – Preselected Confinement Term $\frac{c_b+K_{tr}}{d_b}$ Included, ACI 12.2.3** - Confinement Term $\frac{c_b+K_{tr}}{d_b}$ = 2.5. Ψ_s = 0.8 for #6 bars or smaller

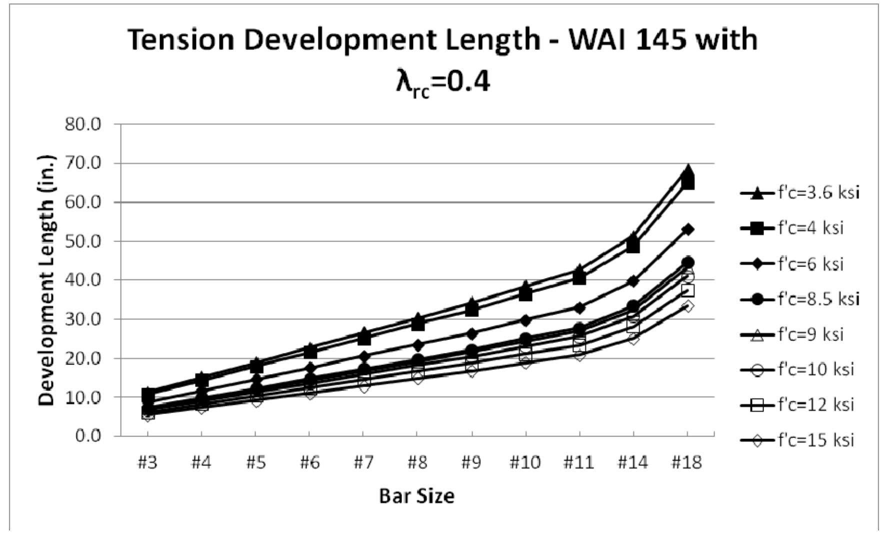






MF* - Modification Factor, ACI 12.2.2** – Preselected Confinement Term $\frac{c_b+K_{tr}}{d_b}$ Included, ACI 12.2.3** - Confinement Term $\frac{c_b+K_{tr}}{d_b}$ = 2.5. Ψ_s = 0.8 for #6 bars or smaller





MF* - Modification Factor

No.	Examples provided by	Example Description	Assumed Design Information	Tension Development Length (in)							Hook Development Length (in)					Lap Splices in Tension (in)						
				AASHTO 6 Ed.	ACI 318-11				WAI 145		AASHTO 6 Ed.	ACI 318-11		WAI 145		AASHTO 6 Ed.	ACI 318-11				WAI 145	
					12.2.2		12.2.3										12.2.2		12.2.3			
1	Louisiana	Transverse #5 top rebar at deck	Deck thickness = 8", #5@12", 2.5" top clear cover, f'c = 4ksi, fy = 60ksi, As $_{\text{required}}$ = As $_{\text{provided,}}$ 50% As spliced, confinement factor = 0.4	15	24	58%	15	0%	18	20%						20	31	55%	19	-5%	24	20%
2	Louisiana	Transverse #6 top rebar at deck	Deck thickness = 8", #6@12", 2.5" cover, f'c = 4ksi, fy = 60ksi, As $_{\text{required}}$ = As $_{\text{provided,}}$ 50% As spliced, confinement factor = 0.4	18	29	61%	17	-6%	22	22%						24	37	54%	22	-8%	29	21%
3	Louisiana	Longitudinal top rebar #7 at deck continuity	Deck thickness = 8", #7@12", 2.5" cover, f'c = 4 ksi, fy = 60 ksi, As $_{\text{required}}$ = As $_{\text{provided,}}$ 50% As spliced, confinement factor = 0.4	23	42	83%	25	9%	25	9%						30	54	80%	33	10%	33	10%
4	Louisiana	Longitudinal mild reinforcement #5 at top flange of BT-78	#5 @ 6", 1.375" cover, f'c = 8.5ksi, fy = 60ksi, As required = As provided , 50% As spliced, confinement factor = 0.4	15	17	13%	10	-33%	13	-13%						20	22	10%	13	-35%	17	-15%
5	Louisiana	Shear Reinforcement #5 at BT-78	#5 @6", f'c = 8.5 ksi, fy = 60 ksi, As $_{\text{required}}$ = As $_{\text{provided}}$								9	9	0%	9	0%							
6	Louisiana	Top rebar #11 at pier cap	Cap size (5'-6" wide and 7'-6" deep), top bar #11@6", total 24 #11 top bars to be developed (n=24), 3" cover, As $_{\text{required}}$ = As $_{\text{provided}}$, 50% As spliced, f'c = 4 ksi, fy = 60ksi, stirrup #6@7" (with four legs, Atr=4x0.44=1.76), , calculate confinement factor using equation 5.11.2.1.3-1 = 0.41 (also meet requirements for 0.4), used 0.41	66	87	32%	54	-18%	55	-17%	19	19	0%	19	0%	86	114	33%	70	-19%	71	-17%
7	Louisiana	Vertical rebar #11 at Column	6'-0" square column, #11@6", total 11-#11 bars on each face to be developed (n=11), 3" cover, f'c = 4ksi, fy = 60ksi, As required = As provided, 100% As spliced, stirrup #5 @12" (4 legs, Atr=4x0.31=1.24), calculate confinement factor using equation 5.11.2.1.3-1 = 0.42 (also meet requirements for 0.40), used 0.42	47	67	43%	42	-11%	43	-9%						61	87	43%	55	-10%	56	-8%
8	Louisiana	Top rebar #8 in footing	Footing size (100'-0" long and 43'-6" wide), top bar #8@7.5", total 70 #8 bars to be developed (n=70), 4" cover, As $_{\text{required}}$ = As $_{\text{provided}}$, 50% As spliced, f'c = 4.0 ksi, fy = 60ksi, confinement factor = 0.4	34	62	82%	37	9%	38	12%						44	81	84%	49	11%	49	11%
9	Louisiana	Vertical rebar #10 at Column	4'-0" diameter column, #10@5.5" 1-#10 bars to be developed (n=1), 2" cover, f'c = 3.0ksi, fy = 60ksi, As required = As provided, 100% As spliced, spiral #4 @6" pitch (2 legs, Atr=2x0.20=0.40), calculate confinement factor using equation 5.11.2.1.3-1 = 0.43	55	70	27%	42	-24%	43	-22%						94	91	-3%	55	-41%	55	-41%

No.	Examples provided by	Example Description	Assumed Design Information	Tension Development Length (in)							Hook Development Length (in)					Lap Splices in Tension (in)						
				AASHTO 6 Ed.	ACI 318-11				WAI 145		AASHTO 6 Ed.	ACI 318-11		WAI 145		AASHTO 6 Ed.	ACI 318-11				WAI 145	
					12.2.2		12.2.3										12.2.2		12.2.3			
10	Texas	Top rebar #11 at cap	Cap size (4'-0" wide and 4'-0" deep), top bar #11@8.3", total 5 #11 top bars to be developed (n=5), 2.25" cover, $As_{required}$ = $As_{provided}$, 50% As spliced, f'c = 3.6 ksi, fy = 60ksi, stirrup #5@6.75" (with two legs, Atr=2x0.31=0.62), calculate confinement factor using equation 5.11.2.1.3-1 = 0.40 (Note: top bar location factor was not included in the calculation sheet provided by Texas. This example used the same input as shown in their calculation)	62	71	15%	43	-31%	43	-31%	28.2	28.2	0%	28.2	0%	80	92	15%	55	-31%	56	-30%
11	Texas	Vertical rebar #9 at Column	3'-6" diameter column, #9@7.74" 1-#9 bars to be developed (n=1), 3" cover, f'c = 3.6ksi, fy = 60ksi, As required = As provided, 100% As spliced, spiral #3 @6" pitch (2 legs, Atr=2x0.11=0.22), calculate confinement factor using equation 5.11.2.1.3-1 = 0.4	40	57	43%	34	-15%	34	-15%						68	74	9%	44	-35%	45	-34%
12	Texas	Transverse #5 top rebar at deck	#5@6", 2.0" top clear cover, f'c = 4ksi, fy = 60ksi, As required = As provided, 50% As spliced, confinement factor = 0.4	15	24	58%	15	0%	18	20%						20	31	55%	19	-5%	24	20%
13	Nebraska	Transverse #6 top rebar at deck	Deck thickness = 7.5", #6@12", 2.5" cover, f'c = 4ksi, fy = 60ksi (epoxy -coated), $As_{required}$ = $As_{provided,}$ 50% As spliced, confinement factor = 0.4	22	34	55%	21	-5%	26	18%						28	45	61%	27	-4%	34	21%
14	Nebraska	Transverse #7 top rebar at deck	Deck thickness = 7.5", #7@12", 2.5" cover, f'c = 4ksi, fy = 60ksi (epoxy-coated), $As_{required}$ = $As_{provided,}$ 50% As spliced, confinement factor = 0.4	34	62	82%	38	12%	38	12%						44	81	84%	49	11%	49	11%
15	Nebraska	Transverse #7 top rebar at deck (3" Spacing)	Deck thickness = 7.5", #7@3", 2.5" cover, f'c = 4ksi, fy = 60ksi (epoxy-coated), $As_{required}$ = $As_{provided,}$ 50% As spliced, confinement factor = 0.58	34	62	82%	55	62%	55	62%						44	81	84%	71	61%	72	64%

2012 AASHTO BRIDGE COMMITTEE AGENDA ITEM: 13

SUBJECT: Committee Report and Recommendations for Approval

TECHNICAL COMMITTEE: T-11 Research

☐ **REVISION** ☐ **ADDITION** ☐ **NEW DOCUMENT**

☒ **DESIGN SPEC** ☐ **CONSTRUCTION SPEC** ☐ **MOVABLE SPEC**
☐ **MANUAL FOR BRIDGE EVALUATION** ☐ **SEISMIC GUIDE SPEC** ☐ **COASTAL GUIDE SPEC**
☐ **OTHER**

DATE PREPARED: 5/7/13
DATE REVISED:

AGENDA ITEM:

A list of recommended research statements will be presented for approval.

OTHER AFFECTED ARTICLES:

None

BACKGROUND:

Research statements that were reviewed and submitted by Technical Committee Chairs or State Bridge Engineers are discussed and recommended for the next NCHRP Program cycle.

ANTICIPATED EFFECT ON BRIDGES:

Will depend if research statements are approved for NCHRP funding and on the results from that research.

REFERENCES:

None

OTHER:

None

2013 AASHTO BRIDGE COMMITTEE AGENDA ITEM: 14

SUBJECT: LRFD Bridge Design Specifications: Section 12, Articles 12.8.9.3.1 & 12.8.9.6, Tables A12-14 & A12-15

TECHNICAL COMMITTEE: T-13 Culverts

☒ **REVISION** ☒ **ADDITION** ☐ **NEW DOCUMENT**

☒ **DESIGN SPEC** ☐ **CONSTRUCTION SPEC** ☐ **MOVABLE SPEC**
☐ **MANUAL FOR BRIDGE EVALUATION** ☐ **SEISMIC GUIDE SPEC** ☐ **BRIDGE ELEMENT INSP GUIDE**
☐ **OTHER**

DATE PREPARED: 1/1/13
DATE REVISED: 4/15/13

AGENDA ITEM:

Item #1

Revise Article 12.8.9.3.1 as follows:

Deep corrugated structural plate used to manufacture structures designed under this section shall meet the requirements of AASHTO M 167M/M 167.

Sections may be stiffened. If stiffening is provided by ribs, the ribs shall be bolted to the structural plate corrugation prior to backfilling using a bolt spacing of not more than ~~16~~ 16.0 in. for 15.0 by 5.5 in. corrugations or 20.0 in. for 20.0 by 9.5 in. corrugations. The cross-section properties in Table A12-14 shall apply.

Item #2

Revise the definition of M_s in Article 12.8.9.6 as follows:

M_s = constrained modulus of embedment computed based on the free field vertical stress at a depth halfway between the top and springline of the structure (Table 12.12.3.5-1)

Item #3

Add the following commentary as a new paragraph in Article C12.8.9.6:

Global buckling of deep corrugated structures occurs over a long wavelength, thus the soil modulus computed at a depth halfway between the top and springline of the structure is representative of the overall soil resistance to buckling.

Item #4

In Tables A12-14 and A12-15, revise the heading in the 1st column as follows:

~~Coating Thickness~~ Coated Thickness

Item #5

Revise the title and add a new section to Table A12-14 as follows:

Table A12-14—Steel Structural Plate with Deep Corrugations—Cross-Section Properties

20 x 9 1/2 in. Corrugations			
Coated Thickness (in.)	A (in^2/ft)	r (in.)	I ($in.^4$/in.)
0.280	5.021	3.21	4.321
0.319	5.737	3.22	4.945
0.380	6.855	3.22	5.921

Item #6

Add a new section to Table A12-15 as follows:

Table A12-15—Minimum Longitudinal Seam Strengths, Deep Corrugated Structures—Bolted

20 x 9 1/2 in. Corrugations		
Coated Thickness (in.)	Bolt Diameter (in.)	12 Bolts/Corrugation (lb/ft of seam)
0.280	7/8	197,000[a]
0.319	7/8	218,000[a]
0.380	7/8	277,000[a]

[a] The number of bolts per corrugation includes the bolts in the corrugation crest, tangent, and valley; the number of bolts within one pitch. The ultimate seam strengths listed are based on tests of staggered seams in assemblies fabricated from panels with a nominal width of 40 in. and include the contribution of additional bolts at the stagger. The listed ultimate seam strengths are only applicable for panels with a nominal width of 40 in. and staggered seams.

OTHER AFFECTED ARTICLES:

None

BACKGROUND:

This item proposes changes to *AASHTO LRFD Bridge Design Specifications* to incorporate design for structural steel plate with 20 by 9½ in. corrugations under Article 12.8.9 *Deep Corrugated Structural Plate Structures*. AASHTO Materials Committee 4b has been asked to modify Standard M167 to incorporate this same corrugation. ASTM is currently balloting to incorporate the corrugation in Standard ASTM A761.

ANTICIPATED EFFECT ON BRIDGES:

This proposal gives bridge engineers a stiffer corrugated steel plate for use in long-span culverts, and allowing longer spans and reduced construction sensitivity.

REFERENCES:

Evaluation of Ultra•Cor Seam Strength, R.L. Brockenbrough & Associates, 23 November 2012.

OTHER:

None

2013 AASHTO BRIDGE COMMITTEE AGENDA ITEM: 15

SUBJECT: LRFD Bridge Design Specifications: Section 3, Articles 3.4.2.1 & 3.4.2.2 (T-5 WAI 47); Section 6, Article 6.5.4.1

TECHNICAL COMMITTEE: T-14 Steel/T-5 Loads

☒ **REVISION** ☒ **ADDITION** ☐ **NEW DOCUMENT**

☒ **DESIGN SPEC** ☐ **CONSTRUCTION SPEC** ☐ **MOVABLE SPEC**
☐ **MANUAL FOR BRIDGE EVALUATION** ☐ **SEISMIC GUIDE SPEC** ☐ **COASTAL GUIDE SPEC**
☐ **OTHER**

DATE PREPARED: 12/19/12
DATE REVISED: 4/22/13

AGENDA ITEM:

Item #1

Revise Article 3.4.2.1 as follows:

All appropriate strength limit state load combinations in Table 3.4.1-1, modified as specified herein, shall be investigated.

When investigating Strength Load Combinations I~~,~~ and III~~, and V~~ for maximum force effects during construction, load factors for the weight of the structure and appurtenances, *DC* and *DW*, shall not be ~~taken to be~~ less than 1.25.

Unless otherwise specified by the Owner, ~~the load factor for~~ construction loads including ~~and for any associated~~ dynamic effects (if applicable) shall be added ~~not be less than 1.5~~ in Strength Load Combination I with a load factor not less than 1.5 when investigating for maximum force effects. ~~The load factor for wind in Strength Load Combination III shall not be less than 1.25.~~

Unless otherwise specified by the Owner, the load factor for wind during construction in Strength Load Combination III shall not be less than 1.25 when investigating for maximum force effects. Any applicable construction loads shall be included with a load factor not less than 1.25.

Unless otherwise specified by the Owner, primary steel superstructure components shall be investigated for maximum force effects during construction for an additional load combination consisting of the applicable *DC* loads and any construction loads that are applied to the fully erected steelwork. For this additional load combination, the load factor for *DC* and construction loads including dynamic effects (if applicable) shall not be less than 1.4.

Item #2

Revise Article C3.4.2.1 as follows:

The load factors presented here should not relieve the contractor of responsibility for safety and damage control during construction.

Construction loads are ~~permanent loads and other~~ loads that act on the structure only during construction. Often the construction loads are not accurately known at the time of design. Construction loads include but are not limited to the weight of materials, removable forms, personnel, and equipment such as deck finishing machines or loads applied to the structure through falsework or other temporary supports. The Owner may consider noting the construction loads assumed in the design on the contract documents. ~~Often the construction loads are not accurately known at design time; however, the magnitude and location of these loads considered in the design should be noted on the contract documents~~. The weight of the wet concrete deck and any stay-in-place forms should be considered as *DC* loads.

For steel superstructures, the use of higher-strength steels, composite construction, and limit-states design approaches in which smaller factors are applied to dead load force effects than in previous service-load design approaches, have generally resulted in lighter members overall. To ensure adequate stability and strength of primary steel superstructure components during construction, an additional strength limit state load combination is specified for the investigation of loads applied to the fully erected steelwork.

Item #3

Revise Article 3.4.2.2 as follows:

In the absence of special provisions to the contrary, where evaluation of construction deflections are required by the contract documents, Service Load Combination ~~Service~~ I shall apply. Except for segmentally constructed bridges, ~~C~~construction ~~dead loads shall be considered as part of the permanent load and construction transient loads considered part of the live load~~ loads shall be added to the Service Load Combination I with a load factor of 1.00. Appropriate load combinations and allowable stresses for segmental bridges are addressed in Article 5.14.2.3. The associated permitted deflections shall be included in the contract documents**.**

Item #4

Add the following paragraph to the end of Article 6.5.4.1:

A special load combination for investigating the constructibility of primary steel superstructure components for loads applied to the fully erected steelwork shall be considered, as specified in Article 3.4.2.1.

OTHER AFFECTED ARTICLES:

None

BACKGROUND:

This item clarifies the load factors used with construction loads and the associated loading combination. It also introduces a separate load combination for checking the constructibility of primary steel superstructure components, in which a load factor of 1.4 is to be applied to the force effects due to the applicable component (or *DC*) dead loads for the construction condition under consideration, acting in conjunction with any construction loads that may be considered, acting on the fully erected steelwork. Previous service-load design approaches effectively applied a load factor ranging from about 1.67 (1/0.60) to 1.82 (1/0.55) to the dead load force effects, with the AASHTO service-load design method effectively applying the latter (often discounted as much as 25 percent for temporary construction conditions). The base strength load combinations in more recent limit-state design approaches have applied a load factor ranging from about 1.25 to 1.3 to these force effects. With the advent of higher-strength steels and composite construction also generally contributing to the use of lighter members, it is felt that this special load combination should be applied when checking the constructibility of primary steel girder superstructure components to ensure a level of strength and stability during critical construction stages that at least approaches that attained in the past using previous design approaches.

ANTICIPATED EFFECT ON BRIDGES:

Investigation of the constructibility of primary steel superstructure components for this special load combination is anticipated to lead to greater strength and stability of steel components under loads acting on the fully erected steelwork during construction, where unintended events could potentially lead to significantly larger force effects than those predicted during the design.

REFERENCES:

None

OTHER:

None

2013 AASHTO BRIDGE COMMITTEE AGENDA ITEM: 16

SUBJECT: LRFD Bridge Design Specifications: Section 4, Various Articles

TECHNICAL COMMITTEE: T-14 Steel / T-5 Loads

☒ **REVISION** ☒ **ADDITION** ☐ **NEW DOCUMENT**

☒ **DESIGN SPEC** ☐ **CONSTRUCTION SPEC** ☐ **MOVABLE SPEC**
☐ **MANUAL FOR BRIDGE EVALUATION** ☐ **SEISMIC GUIDE SPEC** ☐ **COASTAL GUIDE SPEC**
☐ **OTHER**

DATE PREPARED: 1/20/13
DATE REVISED: 4/15/13

AGENDA ITEM:

Item # 1

Delete the 1st sentence of the sixth bullet item of Article C4.6.3.3.1. Move the 7th bullet item and the remainder of the 6th bullet item to the two paragraphs at the end of the new Article C4.6.3.3.4 as shown below.

Item # 2

Revise the 1st sentence of the last bullet item of Article C4.6.3.3.1 as follows:

- The St. Venant torsional inertia may be determined using the appropriate equation ~~in~~ from Article C4.6.2.2.1.

Item # 3

Add the following to the end of the last bullet item of Article C4.6.3.3.1:

For the analysis of composite loading conditions using plate and eccentric beam structural analysis models, the St. Venant torsional inertia of steel I-girders should be calculated using Eq. 4.6.2.2.1-1 without the consideration of any torsional interaction with the composite deck.

Item # 4

Add the following new Article 4.6.3.3.4:

Article 4.6.3.3.4—Cross-frames and Diaphragms

When modeling a cross-frame with a single line of equivalent beam elements, both the equivalent beam flexure and shear deformation shall be considered.

The influence of end connection eccentricities shall be considered in the calculation of the equivalent axial stiffness of single-angle and flange-connected tee-section cross-frame members.

Item # 5

Add the following new Article C4.6.3.3.4:

C4.6.3.3.4

Due to their predominant action as trusses, cross-frames generally exhibit substantial beam shear deformations when modeled using equivalent beam elements in a structural analysis. The modeling of cross-frames using Euler-Bernoulli beam elements, which neglect beam shear deformation, typically results in substantial misrepresentation of their physical stiffness properties. Timoshenko beam elements, or other types of beam elements that include explicit modeling of beam shear deformations, provide a significantly improved approximation of the cross-frame stiffnesses (White et al., 2012).

The axial rigidity of single-angle members and flange-connected tee-section cross-frame members is reduced due to end connection eccentricities, as illustrated in Figure C4.6.3.3.4-1. An upper-bound for this reduction may be estimated by assuming zero bending restraint at the ends of the member, and equating the relative displacements between the ends of the member at the plane of the connection, due to axial deformation plus bending about the geometric axis parallel to the plane of the connection, to the axial deformation based on an equivalent axial rigidity, $(AE)_{eq}$, which accounts for the bending effects as follows:

$$\Delta = \frac{PL}{AE} + \frac{PeL}{EI}e = \frac{PL}{(AE)_{eq}} \qquad \text{(C4.6.3.3.4-1)}$$

where:

A = gross area of the member (in.^{2})
e = eccentricity of the connection plate relative to the member centroidal axis (in.)
L = member length (in.)
P = member axial load (kip)

In lieu of a more accurate analysis, $(AE)_{eq}$ of equal leg single angles, unequal leg angles connected to the long leg, and flange-connected tee-section members may be taken as 0.65AE. This is an approximate median value of the stiffnesses measured in detailed finite element and experimental studies discussed by Wang et al. (2012). The value 0.50AE is a lower bound to the detailed FEA and experimentally measured stiffnesses. In many bridges, the response is insensitive to the specific values selected for $(AE)_{eq}$.

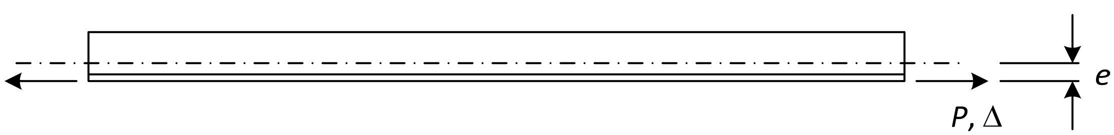


Figure C4.6.3.3.4-1—Eccentrically Loaded Single-angle or Flange-connected Tee-section Member

For bridges with widely spaced cross-frames or diaphragms, it may be desirable to use notional transverse beam members to model the deck when using grid analysis methods. The number of such beams is to some extent discretionary. The significance of shear lag in the transverse beam-slab width as it relates to lateral load distribution can be evaluated qualitatively by varying the stiffness of the beam-slab elements within reasonable limits and observing the results. Such a sensitivity study often shows this effect is not significant.

Live load force effects in cross-frames and diaphragms should be calculated by grid or finite element analysis. The easiest way to establish extreme force effects is by using influence surfaces analogous to those developed for the main longitudinal members.

Item # 6

Add the following references to Article 4.9:

White, D.W., Coletti, D., Chavel, B.W., Sanchez, A., Ozgur, C., Jimenez Chong, J.M., Leon, R.T., Medlock, R.D., Cisneros, R.A., Galambos, T.V., Yadlosky, J.M., Gatti, W.J., and Kowatch, G.T. 2012. "Guidelines for Analytical Methods and Construction Engineering of Curved and Skewed Steel Girder Bridges," NCHRP Report 725, Transportation Research Board, National Research Council, Washington, D.C.

Wang, W.H., Battistini, A.D., Helwig, T.A., Engelhardt, M.D. and Frank, K.H. (2012). "Cross Frame Stiffness Study by Using Full Size Laboratory Test and Computer Models," *Proceedings of the Annual Stability Conference*, Structural Stability Research Council, Grapevine, TX, April 18-21, 11 pp.

Item # 7

Add the following definitions to Article 4.2 **(Note: these same definitions also are proposed in the Agenda Item- pertaining to Article 4.6.3.3.2 in case one or the other of these items is not approved.)**:

Grid Method—A grillage analogy method of analysis of girder bridges in which the longitudinal girders are modeled individually using beam elements, including a width of the deck tributary to the individual girders in the calculation of composite beam cross-section properties, and the cross-frames are typically modeled as equivalent beam elements. For composite girders, a tributary deck width is considered in the calculation of individual girder cross-section properties.

Plate and Eccentric Beam Method—A method of analysis of composite girder bridges in which the bridge deck is modeled using shell finite elements, the longitudinal girders are modeled using beam elements, and the cross-frames are typically modeled as equivalent beam elements. The girder and cross-frame elements are offset from the deck elements to account for the structural depth of these components relative to the deck.

Item # 8

Add the following to Article 4.3:

$(AE)_{eq}$ = equivalent axial rigidity of single-angle members and flange-connected tee-section cross-frame members that accounts for bending effects due to end connection eccentricities (kip) (C4.6.3.3.1)

Revise the following in Article 4.3:

e = correction factor for distribution; eccentricity of a lane from the center of gravity of the pattern of girders (ft); eccentricity of the connection plate relative to the member centroidal axis (in.); rib spacing in orthotropic steel deck (in.) (4.6.2.2.1) (C4.6.2.2.2d) (C4.6.3.3.1) (4.6.2.6.4)
L = span length of deck (ft); span length (ft); span length of beam (ft); member length (in.); length of bridge deck (ft) (4.6.2.1.3) (4.6.2.1.8) (4.6.2.2.1) (C4.6.3.3.1) (4.7.4.4)
P = axle load (kip); member axial load (kip) (4.6.2.1.3) (C4.6.3.3.1)

OTHER AFFECTED ARTICLES:

None

BACKGROUND:

In bridges where the elastic deformation of the cross-frames provides a significant influence on the structural response, traditional modeling of cross-frames using Euler-Bernoulli beam theory can lead to significant inaccuracies in the analysis. In addition, the end eccentricities on single-angle and flange-connected tee-section cross-frame members can have a significant influence on the effective axial stiffness of these components and should be considered in all types of structural analysis. The proposed new Article 4.6.3.3.4 requires the consideration of both shear and flexure deformations in equivalent beam element models of cross-frames. The median value of the stiffnesses determined from the research by Wang et al. (2012) is recommended. The value 0.5*AE* is a lower bound to the measured stiffnesses. The new Article 4.6.3.3.4 also requires the consideration of bending deformations of the cross-frame members due to end eccentricities in all types of cross-frame structural analysis models. Guidance is provided for satisfying these requirements in the corresponding commentary. Furthermore, a sentence about modeling the equivalent beam flexure and shear stiffness in K-frame and X-frame diaphragms is removed from the Commentary Article C4.6.3.3.1 because of the proposed explicit requirement to consider the equivalent beam flexure and shear stiffness in Article 4.6.3.3.4. Lastly, two bullet items from the current Article C4.6.3.3.1 pertaining to cross-frames and diaphragms are moved as paragraphs to the end of the new Article 4.6.3.3.4. The first of these items is specified to apply to grid analysis methods. The bullet item in the current Specification does not provide this qualification. The second bullet item is moved from Article C4.6.3.3.1 without any modification except that the terminology “cross-frames and diaphragms” is used.

A sentence is added to the last bullet item in Article C4.6.3.3.1 to clarify how the St. Venant torsional inertia of the I-girders should be calculated in plate and eccentric beam structural analysis models.

Definitions of the common terms “grid method” and “plate and eccentric beam method” are proposed for addition to Article 4.2. The current AASHTO Section refers to “grid” methods at a number of places, but this term is never defined. The terminology “plate and eccentric beam method” is a more recent addition to the lexicon which defines a specific type of structural analysis commonly utilized in modern practice for composite girder bridges.

ANTICIPATED EFFECT ON BRIDGES:

Better modeling of the response of cross-frames in steel girder bridges.

REFERENCES:

See Item #6

OTHER:

None

2013 AASHTO BRIDGE COMMITTEE AGENDA ITEM: 17

SUBJECT: LRFD Bridge Design Specifications: Section 4, Various Articles; Section 6, Article 6.10.1.5

TECHNICAL COMMITTEE: T-14 Steel / T-5 Loads

☒ **REVISION** ☒ **ADDITION** ☐ **NEW DOCUMENT**

☒ **DESIGN SPEC** ☐ **CONSTRUCTION SPEC** ☐ **MOVABLE SPEC**
☐ **MANUAL FOR BRIDGE EVALUATION** ☐ **SEISMIC GUIDE SPEC** ☐ **COASTAL GUIDE SPEC**
☐ **OTHER**

DATE PREPARED: 1/20/13
DATE REVISED: 4/15/13

AGENDA ITEM:

Item # 1

Add the following new Article 4.6.3.3.2 and renumber the current Articles 4.6.3.3.2 and C4.6.3.3.2 to Articles 4.6.3.3.3 and C4.6.3.3.3:

4.6.3.3.2—Grillage and Plate and Eccentric Beam Analyses of Curved and/or Skewed Steel I-Girder Bridges

For the analysis of curved and/or skewed steel I-girder bridges where either $I_C > 1$ or $I_S > 0.3$ the warping rigidity of the I-girders shall be considered in grillage methods and in plate and eccentric beam methods of structural analysis.

in which:

$$I_C = \frac{15,000}{R(n_{cf}+1)m} \quad (4.6.3.3.2\text{-}1)$$

$$I_S = \frac{w_g \tan\theta}{L_s} \quad (4.6.3.3.2\text{-}2)$$

where:

I_C = I-girder bridge connectivity index
m = bridge type constant, equal to 1 for simple-span bridges or bridge units, and equal to 2 for continuous-span bridges or bridge units, determined at the construction stage and/or loading condition being evaluated
n_{cf} = minimum number of intermediate cross-frames or diaphragms within the individual spans of the bridge or bridge unit at the construction stage and/or loading condition being evaluated
R = minimum radius of curvature at the centerline of the bridge cross-section throughout the length of the bridge or bridge unit at the construction stage and/or loading condition being evaluated (ft)
I_S = bridge skew index, taken equal to the maximum of the values of Eq. 4.6.3.3.2-2 determined for each span of the bridge
w_g = maximum width between the girders on the outside of the bridge cross-section at the completion of the

construction or at an intermediate stage of the steel erection

L_S = span length at the centerline

θ = maximum skew angle of the bearing lines at the end of a given span, measured from a line taken perpendicular to the span centerline

Item # 2

Add the following new Article C4.6.3.3.2:

C4.6.3.3.2

Unless otherwise stated, this Article applies to curved and/or skewed steel I-girder bridges analyzed by grillage or plate and eccentric beam analysis. A 3D finite element analysis of a steel I-girder bridge in which the girder webs are modeled using shell elements and the girder flanges are modeled using beam, shell, or solid elements is capable of directly capturing the contribution of the girder warping rigidity to the torsional stiffness. In a grillage analysis or a plate and eccentric beam analysis of a steel I-girder bridge, the use of only the St. Venant torsional stiffness GJ/L_b can result in a substantial underestimation of the girder torsional stiffness. This is due to neglect of the contribution from girder cross-section warping, or the corresponding flange lateral bending, to the torsional response. For I-girders, the torsional contribution from the girder warping rigidity, EC_w, is often substantial compared to the contribution from the St. Venant torsional rigidity, GJ. When the contribution from the girder warping rigidity is not accounted for in the analysis, the vertical deflections in curved I-girder systems can be substantially over-estimated due to the coupling between the girder torsional and flexural response where $I_C > 1$. Furthermore, the cross-frame forces can be substantially underestimated in straight or curved skewed I-girder bridges due to the under-estimation of the torsional stiffness provided by the girders where $I_S > 0.3$.

White et al. (2012) present a simplified approximate method of considering the girder warping rigidity, applicable for I-girder bridges or bridge units in their final constructed condition, as well as in intermediate noncomposite conditions during steel erection, when at least two I-girders are connected together by support cross-frames, cantilevered girder units have a cross-frame line near the cantilevered girder ends, at least one intermediate cross-frame is located between each of the above locations, and, where the girders are curved, and $I_C < 20$.

For steel I-girder bridges under noncomposite loading conditions, the behavior of grillage models and plate and eccentric beam models can be particularly sensitive to the contribution from the warping rigidity to the girder torsional stiffness. The behavior tends to be less sensitive to the girder warping rigidity under composite loading conditions. For the analysis of composite loading conditions using plate and eccentric beam structural analysis models, it is sufficient to calculate the warping rigidity of the steel I-girders, EC_w, using solely the steel cross-section with Eq. C6.9.4.1.3-1 and without the consideration of any composite torsional interaction with the composite deck.

Other methods of considering the warping rigidity of steel I-girders include the explicit use of open-section thin-walled beam theory, or the use of a general-purpose 3D finite element analysis in which the I-girder is modeled as described previously. Additional information on the modeling of torsion in I-girder bridges may be found in AASHTO/NSBA (2011).

Item # 3

Add the following to the end of Article 6.10.1.5:

The requirement for the modeling of girder torsional stiffness in curved and/or skewed I-girder bridges specified in Article 4.6.3.3.2 shall be satisfied for grillage analyses or plate and eccentric beam analyses of steel I-girder bridges.

Item # 4

Add the following references to Article 4.9:

AASHTO/NSBA Steel Bridge Collaboration. 2011. *Guidelines for Steel Girder Bridge Analysis, G13.1*, 1st edition, NSBASGBA-1, American Association of State Highway and Transportation Officials, Washington, DC.

White, D.W., Coletti, D., Chavel, B.W., Sanchez, A., Ozgur, C., Jimenez Chong, J.M., Leon, R.T., Medlock, R.D., Cisneros, R.A., Galambos, T.V., Yadlosky, J.M., Gatti, W.J., and Kowatch, G.T. 2012. "Guidelines for Analytical Methods and Construction Engineering of Curved and Skewed Steel Girder Bridges," NCHRP Report 725, Transportation Research Board, National Research Council, Washington, D.C.

Item # 5

Add the following definitions to Article 4.2 **(Note: these same definitions also are proposed in the Agenda Item pertaining to Article 4.6.3.3.1 in case one or the other of these items is not approved.)**:

Grid Method—A grillage analogy method of analysis of girder bridges in which the longitudinal girders are modeled individually using beam elements and the cross-frames are typically modeled as equivalent beam elements. For composite girders, a tributary deck width is considered in the calculation of individual girder cross-section properties.

Plate and Eccentric Beam Method—A method of analysis of composite girder bridges in which the bridge deck is modeled using shell finite elements, the longitudinal girders are modeled using beam elements, and the cross-frames are typically modeled as equivalent beam elements, The girder and cross-frame elements are offset from the deck elements to account for the structural depth of these components relative to the deck.

Item # 6

Add the following to Article 4.3:

C_w = girder warping constant (in.^{6}) (C4.6.3.3.2)
I_C = I-girder bridge connectivity index (C4.6.3.3.2)
J_{eq} = equivalent St. Venant torsion constant accounting for the influence of I-girder cross-section warping (in^4) (C4.6.3.3.2)
m = bridge type constant, equal to 1 for simple-span bridges or bridge units, and equal to 2 for continuous-span bridges or bridge units, determined at the construction stage being evaluated (C4.6.3.3.2)
n_{cf} = minimum number of intermediate cross-frames or diaphragms within the individual spans of the bridge or bridge unit at the stage of construction being evaluated (C4.6.3.3.2)

Revise the following in Article 4.3:

L_b = spacing of brace points (ft.) (C4.6.2.7.1) (in.)
R = girder radius (ft); load distribution to exterior beam in terms of lanes; radius of curvature; R-Factor for calculation of seismic design forces due to inelastic action; minimum radius of curvature at the centerline of the bridge cross-section throughout the length of the bridge or bridge unit at the stage of construction being evaluated (ft) (C4.6.1.2.4b) (C4.6.2.2.2d) (C4.6.6) (4.7.4.5) (C4.6.3.3.2)

OTHER AFFECTED ARTICLES:

None

BACKGROUND:

The traditional use of only the steel I-girder St. Venant torsional stiffness in grillage or plate and eccentric beam structural analyses can lead to substantial overestimation of the structural displacements and underestimation of certain internal forces within curved and/or skewed bridge structural systems. These errors can be particularly problematic when estimating structural displacements and internal forces due to noncomposite loadings. The proposed additions require that the girder warping rigidity be considered in any grillage or plate and eccentric beam structural analysis of steel I-girder bridges with significant horizontal curvature and/or skew. One method of satisfying this requirement is referenced in the commentary. The proposed commentary explicitly states that a 3D finite element analysis in which the I-girder webs are modeled using shell elements and the flanges are modeled using beam, shell or solid elements is capable of directly capturing the contribution of the girder warping rigidity to the torsional stiffness.

The terms “Grid Method” and “Plate and Eccentric Beam Method” are proposed for inclusion in Article 4.2 to more clearly define these terms within the context of current structural analysis practices. The current AASHTO Specification refers to “grid” methods at a number of places, but this term is never defined. The terminology “plate and eccentric beam method” is a more recent addition to the lexicon which defines a specific type of structural analysis commonly utilized in modern practice for composite girder bridges.

ANTICIPATED EFFECT ON BRIDGES:

The proposed changes provide requirements and guidance that will reduce the likelihood of issues related to the inadequate modeling of the girder warping torsional rigidity in grid or grillage analyses or plate and eccentric beam analyses of steel I-girder bridges.

REFERENCES:

See Item #4

OTHER:

None

2013 AASHTO BRIDGE COMMITTEE AGENDA ITEM: 18

SUBJECT: LRFD Bridge Design Specifications: Section 6, Article 6.5.4.2

TECHNICAL COMMITTEE: T-14 Steel

☒ **REVISION** ☐ **ADDITION** ☐ **NEW DOCUMENT**

☒ **DESIGN SPEC** ☐ **CONSTRUCTION SPEC** ☐ **MOVABLE SPEC**
☐ **MANUAL FOR BRIDGE EVALUATION** ☐ **SEISMIC GUIDE SPEC** ☐ **COASTAL GUIDE SPEC**
☐ **OTHER**

DATE PREPARED: 9/25/12
DATE REVISED:

AGENDA ITEM:

In Article 6.5.4.2, revise the 3rd bullet as follows:

- For axial compression, steel only ϕ_c = ~~0.90~~ <u>0.95</u>

OTHER AFFECTED ARTICLES:

None

BACKGROUND:

The resistance factor for steel members (or components) subject to axial compression, ϕ_c, was increased from 0.85 to 0.90 in the 2005 AISC Specification. This increase gave recognition to the changes in industry practice combined with the substantial numbers of additional column strength analyses and tests that had taken place since the original calibrations were performed in the 1970s and 1980s. In the original research on the probability-based strength of steel columns (Bjorhovde, 1972, 1978, 1988), three column curves were recommended. The three column curves were the approximate means of bands of strength curves for columns of similar manufacture based on extensive analyses and confirmed by full-scale tests. Hot-formed and cold-formed heat treated HSS columns fell into the data band of highest strength (SSRC Column Category 1P), while built-up wide-flange columns made from universal mill plates were included in the data band of lowest strength (SSRC Column Category 3P). The largest group of data, however, clustered around SSRC Column Category 2P. Probabilistic analysis would have resulted in a resistance factor ϕ_c = 0.90 or even slightly higher had the original AISC LRFD Specification opted for using all three column curves for the respective column categories (Galambos, 1983; Bjorhovde, 1988; Ziemian, 2010). However, it was decided to use only one column curve, SSRC Column Category 2P, for all column types. The AASHTO LRFD Specification followed suit. The use of only one column curve results in a larger data spread and thus a larger coefficient of variation, and so a resistance factor ϕ_c = 0.85 was adopted in the original AISC LRFD Specification for the column equations to achieve a level of reliability comparable to that of beams. Resistance factors in the AASHTO LRFD Specification are typically set at a level that is 0.05 higher than those in the AISC LRFD Specification; thus, ϕ_c was set to 0.90 in the original AASHTO LRFD Specification.

Since that time, significant additional analyses and tests, as well as changes in practice, have demonstrated that the increase in ϕ_c from 0.85 to 0.90 in the AISC LRFD Specification was warranted, indeed even somewhat conservative (Bjorhovde, 1988). Significant changes in industry practice since that time have included the following: (1) built-up shapes are no longer manufactured from universal mill plates; (2) the most commonly used structural steel is now ASTM A 709 Grade 50 or 50W, with a specified minimum yield stress of 50 ksi; and (3) changes in steelmaking practice have resulted in materials of higher quality and much better defined properties. The level and variability of the yield stress thus have led to a reduced coefficient of variation for the relevant material properties (Bartlett et al., 2003). As a result, for consistency, it is recommended that ϕ_c for steel members (or components) subject to axial compression be raised from 0.90 to 0.95 in the AASHTO LRFD Specification.

ANTICIPATED EFFECT ON BRIDGES:

A slight increase in the factored compressive resistance of steel members (or components) subject to axial compression.

REFERENCES:

Bartlett, R.M., Dexter, R.J., Graeser, M.D., Jelinek, J.J., Schmidt, B.J. and Galambos, T.V. (2003), "Updating Standard Shape Material Properties Database for Design and Reliability," *Engineering Journal*, AISC, Vol. 40, No. 1, pp. 2–14.

Bjorhovde, R. (1972), "Deterministic and Probabilistic Approaches to the Strength of Steel Columns," Ph.D. Dissertation, Lehigh University, Bethlehem, PA, May.

Bjorhovde, R. (1978), "The Safety of Steel Columns," *Journal of the Structural Division*, ASCE, Vol. 104, No. ST9, September, pp. 1371–1387.

Bjorhovde, R. and Birkemoe, P.C. (1979), "Limit States Design of HSS Columns," *Canadian Journal of Civil Engineering*, Vol. 6, No. 2, pp. 276–291.

Bjorhovde, R. (1988), "Columns: From Theory to Practice," *Engineering Journal*, AISC, Vol. 25, No. 1, 1st Quarter, pp. 21–34.

Galambos, T.V. (1983), "Reliability of Axially Loaded Columns," *Engineering Structures*, Vol. 5, No. 1, pp. 73–78.

Ziemian, R.D. (ed.) (2010), *Guide to Stability Design Criteria for Metal Structures*, 6th Ed., John Wiley & Sons, Inc., Hoboken, NJ.

OTHER:

None

2013 AASHTO BRIDGE COMMITTEE AGENDA ITEM: 19

SUBJECT: LRFD Bridge Design Specifications: Section 6, Table 6.6.1.2.3-1

TECHNICAL COMMITTEE: T-14 Steel/T-12 Structural Supports for Highway Signs, Luminaires and Traffic Signals

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☒ **REVISION** ☐ **ADDITION** ☐ **NEW DOCUMENT**

☒ **DESIGN SPEC** ☐ **CONSTRUCTION SPEC** ☐ **MOVABLE SPEC**
☐ **MANUAL FOR BRIDGE EVALUATION** ☐ **SEISMIC GUIDE SPEC** ☐ **COASTAL GUIDE SPEC**
☐ **OTHER**

DATE PREPARED: 1/18/13
DATE REVISED: 4/4/13

AGENDA ITEM:

In Table 6.6.1.2.3-1, revise the description for Condition 2.3 as follows:

2.3 Base ~~metal at the net section of all bolted connections in hot dipped galvanized members (Huhn and Valtinat, 2004); base~~ metal at the ~~appropriate section defined in Condition 2.1 or 2.2, as applicable,~~ <u>net or gross section</u> of high-strength bolted joints with pretensioned bolts installed in holes punched full size (Brown et al., 2007); and base metal at the net section of other mechanically fastened joints, except for eyebars and pin plates, e.g., joints using ASTM A307 bolts or non-pretensioned high-strength bolts.
(Note: see Condition 2.5 for bolted angle or tee section member connections to gusset or connection plates).

OTHER AFFECTED ARTICLES:

Delete the following reference from the Section 6 reference list in Article 6.17:

Huhn, H., and G. Valtinat. 2004. "Bolted Connections with Hot Dip Galvanized Steel Members with Punched Holes." *Proceedings of the ECCS/AISC Workshop, Connections in Steel Structures V: Innovative Steel Connections*, June 3–5, 2004. European Convention for Constructional Steelwork/American Institute of Steel Construction, Amsterdam.

BACKGROUND:

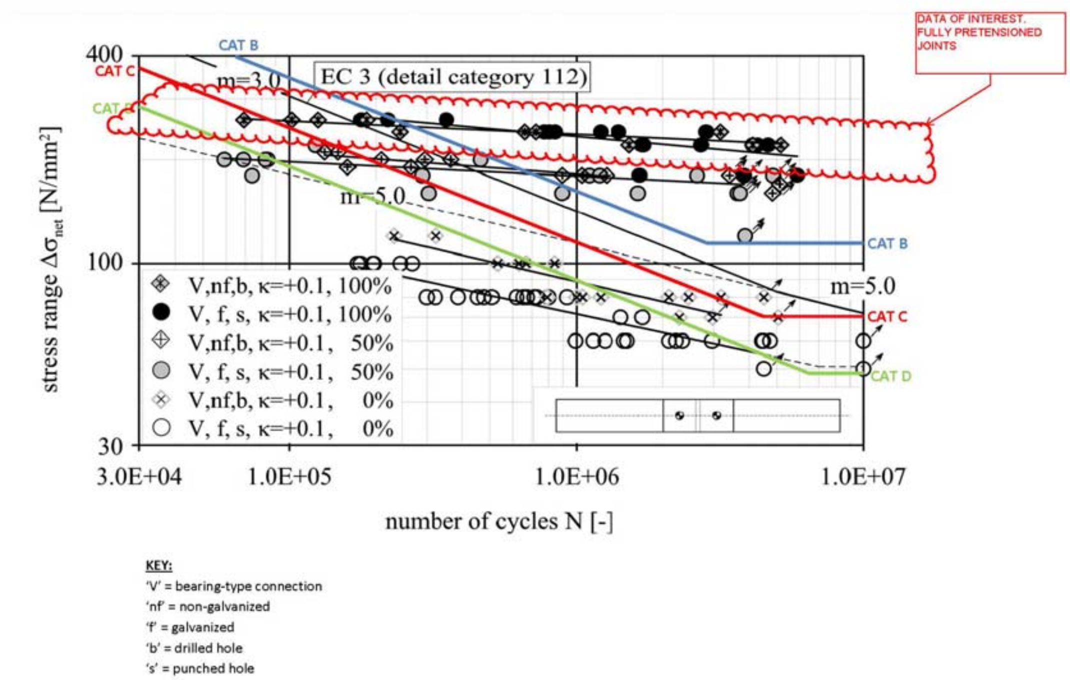
Condition 2.3 in Table 6.6.1.2.3-1 deals with the fatigue resistance of mechanically fastened joints. The types of joints included under this condition are those using pretensioned high-strength bolts installed in holes punched full size, those using A307 bolts or non-pretensioned high-strength bolts, and also all bolted joints used in hot-dipped galvanized members. For these joints, the fatigue Detail Category is reduced from the base fatigue Detail Category B that is typically applied to mechanically fastened joints with fully pretensioned HS bolts to fatigue Detail Category D. The application of this reduction to bolted joints in hot-dipped galvanized members was based previously on the research described in Huhn and Valtinat, 2004, and was introduced in the 2008 Interims when an extensive update was made to Table 6.6.1.2.3-1. Previous versions of the specifications did not make a distinction between hot-dipped galvanized and non-galvanized mechanically fastened joints. The proposed revision in this item reflects the results of a further examination of the data contained in this report, as described below.

The data of interest included in this report were based on fatigue tests on connections with fully tightened fasteners installed in drilled holes with and without galvanized plates. These data are plotted against the AASHTO

Category B, C, and D design S-N curves below. Note there is no significant difference in the fatigue resistance of the hot-dipped galvanized and non-galvanized plates where fully pretensioned fasteners were used. Almost all of the data fall well above Category B, except where the applied stress range was unusually high, i.e., about 240 MPa or about 35 ksi. Interestingly, even the non-galvanized plates failed below Category B at these extremely high stress ranges. The cause for this is believed to be due to the applied stress range exceeding the slip resistance of the joint.

Since the data from both the hot-dipped galvanized and non-galvanized tests are essentially the same, it is proposed to delete the reference to hot-dipped galvanized members in the description for Condition 2.3. This would result in the inclusion of all bolted connections with fully pretensioned high-strength bolts in galvanized members with the bolts installed in holes drilled full size or subpunched and reamed to size being covered under Condition 2.1 or 2.2, as applicable, and classified as Category B. Bolted connections in galvanized members not satisfying the preceding conditions would still be classified under the modified Condition 2.3 as Category D.

Under very high stress ranges, say greater than 35 ksi, the data from the report would appear to indicate that fatigue Detail Category C might be more appropriate for all fully pretensioned joints; i.e. both galvanized and non-galvanized. However, the database used to develop the existing requirement that all fully pretensioned joints satisfy Category B was not examined to determine if such an additional restriction is indeed appropriate. Considering the actual magnitude of in-situ stress ranges in bridges and ancillary structures, the use of Category B still seems appropriate for these joints with no further commentary or restrictions.



ANTICIPATED EFFECT ON BRIDGES

More economical fatigue designs for bridges and ancillary structures utilizing pretensioned high-bolted connections in drilled holes on hot-dipped galvanized members.

REFERENCES:

Huhn, H., and G. Valtinat. 2004. "Bolted Connections with Hot Dip Galvanized Steel Members with Punched Holes." *Proceedings of the ECCS/AISC Workshop, Connections in Steel Structures V: Innovative Steel Connections*, June 3–5, 2004. European Convention for Constructional Steelwork/American Institute of Steel Construction, Amsterdam.

OTHER:

None

2013 AASHTO BRIDGE COMMITTEE AGENDA ITEM: 20

SUBJECT: LRFD Bridge Design Specifications: Section 6, Article 6.7.4.1

TECHNICAL COMMITTEE: T-14 Steel

☒ **REVISION** ☐ **ADDITION** ☐ **NEW DOCUMENT**

☒ **DESIGN SPEC** ☐ **CONSTRUCTION SPEC** ☐ **MOVABLE SPEC**
☐ **MANUAL FOR BRIDGE EVALUATION** ☐ **SEISMIC GUIDE SPEC** ☐ **COASTAL GUIDE SPEC**
☐ **OTHER**

DATE PREPARED: 1/28/13
DATE REVISED:

AGENDA ITEM:

Revise the 4th paragraph of Article 6.7.4.1 as follows:

~~Diaphragms or cross-frames not required for the final condition may be specified to be temporary bracing.~~ Metal stay-in-place deck forms should not be assumed to provide adequate stability to the top flange in compression prior to curing of the deck.

OTHER AFFECTED ARTICLES:

None

BACKGROUND:

The proposed revision removes a sentence from the specification that essentially states the obvious and that can lead to potential confusion, misinterpretation and/or debate as to whether or not select cross-frames or diaphragms in a steel-girder bridge are necessarily required for the final condition in a particular situation and might potentially be removed. In general, cross-frames and diaphragms still carry significant dead, live and wind load forces in most steel-girder bridges in the final condition, and are also necessary to provide stability to bottom flanges in compression.

ANTICIPATED EFFECT ON BRIDGES:

Less potential to remove cross-frames and diaphragms in steel-girder bridges that may actually be performing a significant function in the final condition.

REFERENCES:

None

OTHER:

None

2013 AASHTO BRIDGE COMMITTEE AGENDA ITEM: 21

SUBJECT: LRFD Bridge Design Specifications: Section 6, Article C6.7.4.2

TECHNICAL COMMITTEE: T-14 Steel

☐ **REVISION** ☒ **ADDITION** ☐ **NEW DOCUMENT**

☒ **DESIGN SPEC** ☐ **CONSTRUCTION SPEC** ☐ **MOVABLE SPEC**
☐ **MANUAL FOR BRIDGE EVALUATION** ☐ **SEISMIC GUIDE SPEC** ☐ **COASTAL GUIDE SPEC**
☐ **OTHER**

DATE PREPARED: 1/20/13
DATE REVISED: 4/8/13

AGENDA ITEM:

Add the following paragraph after the 4th paragraph in Article C6.7.4.2:

In skewed bridges where end support lines are skewed more than 20 degrees from normal, the first intermediate cross-frames or diaphragms placed normal to the girders adjacent to a skewed end support ideally should be offset by a minimum of the larger of 1.5D or 0.4L_{b2} from the end supports along each of the girders, where D is the girder web depth and L_{b2} is the unbraced length between the first and the second intermediate cross-frame or diaphragm connected to the girder under consideration within the span (White et al., 2012). This practice helps to alleviate the introduction of a stiff load path that will attract and transfer large transverse forces to the skewed end supports. In some cases, the limit of 0.4L_{b2} may be difficult to achieve, in which case, the offset should be made as large as practicable, but not less than 1.5D.

OTHER AFFECTED ARTICLES:

None

BACKGROUND:

The positioning of the first intermediate cross-frame or diaphragm normal to the girder at a close distance from (or directly connected to) an end support skewed more than 20 degrees from normal tends to produce a stiff load path that will attract and transfer large transverse forces to that support. The proposed paragraph provides recommendations taken from the NCHRP 725 Report (White et al., 2012) that help to limit the development of this "nuisance stiffness" within the structural system, which can result in the potential for excessively large cross-frame members adjacent to severely skewed end supports in I-girder bridges.

ANTICIPATED EFFECT ON BRIDGES:

The proposed addition provides useful guidance that should help engineers in laying out cross-frames in severely skewed I-girder bridges in a manner that results in more economical proportions of these components.

REFERENCES:

White, D.W., Coletti, D., Chavel, B.W., Sanchez, A., Ozgur, C., Jimenez Chong, J.M., Leon, R.T., Medlock, R.D., Cisneros, R.A., Galambos, T.V., Yadlosky, J.M., Gatti, W.J., and Kowatch, G.T. 2012. “Guidelines for Analytical Methods and Construction Engineering of Curved and Skewed Steel Girder Bridges,” NCHRP Report 725, Transportation Research Board, National Research Council, Washington, D.C.

OTHER:

None

2013 AASHTO BRIDGE COMMITTEE AGENDA ITEM: 22

SUBJECT: LRFD Bridge Design Specifications: Section 6, Articles 6.9.2.2, 6.9.5 & 6.17

TECHNICAL COMMITTEE: T-14 Steel

☒ **REVISION** ☒ **ADDITION** ☐ **NEW DOCUMENT**

☒ **DESIGN SPEC** ☐ **CONSTRUCTION SPEC** ☐ **MOVABLE SPEC**
☐ **MANUAL FOR BRIDGE EVALUATION** ☐ **SEISMIC GUIDE SPEC** ☐ **COASTAL GUIDE SPEC**
☐ **OTHER**

DATE PREPARED: 9/3/12
DATE REVISED: 4/14/13

AGENDA ITEM:

Item #1

Revise the 1st sentence of Article 6.9.2.2 as follows:

Except as permitted otherwise in Articles 6.9.4.4 and 6.9.5.3, the axial compressive load, P_u, and concurrent moments, M_{ux} and M_{uy}, calculated for the factored loadings by elastic analytical procedures shall satisfy the following relationship:

Item #2

Replace Article 6.9.5 and the associated commentary with the following:

6.9.5—Concrete-Filled Steel Tube (CFST) Components

The provisions of this article shall be taken to apply to the design of composite CFST construction by the plastic stress distribution method (PSDM). Other design methods may be used with the owner's approval.

C6.9.5

These provisions are not specifically applicable to seismic design, although some aspects of the provisions may be useful for that purpose.

A number of approaches have been used to predict the nominal resistance of CFST members including fiber models, strain-compatibility methods and a cross-sectional analysis known as the plastic stress distribution method (PSDM). Strain compatibility methods and fiber models are permitted by these provisions. Strain compatibility and fiber models are quite accurate for reinforced concrete members, but research evaluating a wide range of prior CFST test data, show that these methods are significantly less accurate for evaluation of composite CFST members, because concrete strain limits are less meaningful for CFST. For CFST, resistances predicted by the strain compatibility methods and fiber models have greater variation in the mean prediction with larger standard deviation than the PSDM. For example, the use of strain compatibility method with a 0.003 compressive strain limit in the concrete, results in a mean error which is more than twice as large as that achieved with the PSDM and a standard deviation that is more than four times that provided by the PSDM, because this compressive strain limit is based upon spalling of the concrete, which cannot occur with CFST (Roeder, Lehman and Bishop 2010). The strain compatibility method is permitted by these provisions, but its use is not preferred for the reasons noted above.

6.9.5.1—General

The resistance of CFST components shall be determined as:

$R_r = \phi R_n$ (6.9.5.1-1)

where:

R_n = nominal resistance based on cross-sectional capacity and adjusted for bending moment and stability effects, as specified herein

ϕ_c = resistance factor for compression and combined axial load and flexure = 0.9

ϕ_v = resistance factor for shear = 1.0

C6.9.5.1

The resistance factors used for compression loading are somewhat greater than the resistance factors used for reinforced concrete. This is justified as a result of the smaller co-variance of 0.05 with axial loads less than 0.6Po noted for CFST members relative to RC members coupled with a conservative predication of the flexural resistance prediction provided by the plastic stress distribution method (PSDM) (as noted in Article C6.9.5.3.3 the moment capacity averages 1.24 times the predicted PSDM moment for a given axial load). Further, circular CFST have uniformly distributed and higher confining stresses provided to the concrete fill, which provide superior resistance and performance relative to reinforced concrete sections with discrete confinement. The strain compatibility method is very accurate for reinforced concrete members, and has been used in past CFST practice. This method is permitted by these provisions, but its use is discouraged, because it results in less accurate predictions with a much large standard deviation than the PSDM (Roeder, Lehman and Bishop 2010). Finally, these CFST provisions directly address member buckling capacity, while reinforced concrete design address instability primarily through a minimum eccentricity for the combined load design (see Articles 5.7.4.3 and 5.7.4.4 and commentary).

6.9.5.2—Limitations

These provisions shall be taken to apply to circular steel tubes, where the specified minimum yield stress of the steel tube and any internal longitudinal reinforcement shall not be greater than 70.0 ksi nor less than 36.0 ksi. The design shall comply with the following:

- The steel tubes shall be spiral welded tubes formed from coil steel or straight seam welded tubes formed from flat plates formed into a tube with longitudinal seam welds.

- The spiral welds shall be complete joint penetration welds of matching metal formed by the double submerged arc process and shall have a minimum CVN toughness of 40 ft-lbs at 70^{o} F.

- Longitudinal welds shall be complete joint penetration welds of matching weld metal.

- Unless specifically noted herein, the local slenderness of the tube shall satisfy:

  $$\frac{D}{t} \leq 0.15\frac{E_{st}}{F_{yst}} \quad (6.9.5.2\text{-}1)$$

  CFST columns with Option 1 or Option 2 foundation connections as specified in Article 6.9.5.5 that develop the maximum bending moment demand at the connection shall satisfy the following:

  $$\frac{D}{t} \leq 0.22\frac{E_{st}}{F_{yst}} \quad (6.9.5.2\text{–}2)$$

- The specified minimum 28-day compressive strength of the concrete shall be at least 2.5 ksi and shall be greater than $.075F_{yst}$.

- The concrete mix shall be designed with the addition of a low-shrinkage admixture to achieve a maximum of 0.04 percent shrinkage at 28 days as tested in accordance with ASTM C 157 Modified Standard Test Method for Length Change of Hardened Hydraulic-Cement Mortar.

- CFSTs shall be designed to assure full composite action in the design.

- Connections of CFST columns into the foundation and/or pier cap shall satisfy the requirements of Article 6.9.5.5.

where:

D = outside diameter of the tube (in.)
E_{st} = elastic modulus of steel tube (ksi)
F_{yst} = minimum specified yield stress of steel tube (ksi)
t = wall thickness of the tube (in.)

C6.9.5.2

Circular CFSTs provide continuous confinement of the concrete, which is superior to that achieved with rectangular CFST. Rectangular steel tubes are specifically excluded from these provisions. Binding action resulting from bending and mechanical interlock of the spiral weld provides shear stress transfer between the steel tube and the concrete fill. These confinement and stress transfer mechanisms are only possible if concrete shrinkage is minimized.

The data used to define and verify these recommended provisions are based upon experimental and nonlinear analytical results that had minimum yield stress values of the steel between 36.0 and 70.0 ksi, and specified compressive strengths of the concrete between 2.5 and 10.0 ksi. The research notes that a balance between the steel and concrete strength provides optimum results, therefore a concrete strength greater than or equal to $0.075F_{yst}$ is required. In addition, the concrete strength must be designed to meet the applicable temperature and environmental conditions within the tube (AASHTO LRFD Article 5.4.2).

Large diameter tubes are commonly required for bridge columns, piles and caissons or drilled shafts. Circular steel tubes fall under ASTM standards including A53 or A500, but these steel tubes are unlikely to be useful for the majority of bridge construction because of the limitations in the maximum diameter, and therefore they are not the focus of these specifications.

The large diameter tubes referred to herein are commonly formed by one of two methods. Coil steel may be unrolled in a helical fashion to form a spirally welded tube. The spiral welds are made as butt joints and formed from both the inside and outside of the tube by the submerged arc process. This manufacturing process is limited to steel tubes with wall thickness of about 1.0 in. or less and diameters greater than about 20.0 in. The spiral welds are subjected to direct stresses under axial load and flexure, and these welds are important to the development of the resistance of the CFST. The double (inside and outside) submerged arc process has been shown to provide good performance if proper weld metal and processes are employed and the minimum tensile strength of the weld metal matches the yield strength of the steel tube. The welds may also be inspected by radiographic methods over the entire length of the weld if increased quality control is needed.

Large diameter tubes may also be formed by bending flat plate around a mandrel and forming the tube with longitudinal seam welds. The longitudinal welds are important, but they are not directly stressed under axial load and flexure and therefore the requirements are not as stringent as those required for spiral welds.

The local slenderness limit of Eq. 6.9.5.2-1 has been shown to experimentally achieve the full plastic capacity with substantial inelastic deformation capacity for CFSTs (Roeder, Lehman, and Bishop 2010). The foundation connections described in article 6.9.5.5 develops a large portion of their inelastic deformation capacity by tensile and compressive yielding of the steel tube both within the lower portion of the pier and the foundation connection. Buckling of the tube within the connection region cannot occur, and hence the slenderness limit for tubes where the maximum moment occurs at these connections is increased to that of Eq. 6.9.5.2-2 to encourage extensive yielding as required for development of large inelastic deformations and design to extreme loadings. This limit based upon prior experimental results (Lehman and Roeder 2012), and significant inelastic deformation capacity was consistently developed. As a result, CFST piers employing this connection at their proposed plastic hinge locations are clearly suitable for seismic design requirements.

Stress transfer between the steel tube and concrete fill is essential for developing full composite action. For CFSTs subjected to combined loading, even small bending moments combined with axial load cause internal binding between the steel and concrete fill, because of their differential stiffness and the arc of the compression strut which acts to engage the tube. Tests on tubes that are greased over the entire inside surface to eliminate friction developed the same composite action and slip resistance as tubes with friction and mechanical shear transfer when the member was subjected to bending (Roeder, Lehman, and Thody 2009).

With large axial load and little or no bending, mechanical shear stress transfer may be required, because the concrete shrinkage may result in separation of the steel tube and concrete fill under loading. With spiral-welded tubes, the inner weld provides mechanical interlock if a low-shrinkage concrete is used, and this may reduce the need for mechanical shear stress transfer. This benefit has not been demonstrated for straight seam welded tubes. Internal shear studs may be used for mechanical shear stress transfer, but the studs may result in damage to the concrete at the steel interface and reduce normal friction (Roeder, Lehman, and Thody 2009). Hence, applications with large axial force and small bending moments are best designed with connections that directly distribute load to both the steel and concrete fill or with internal annular rings. As a guide, a member may be conservatively defined as having high axial load if the bending moment is less than 0.2 M_o while the simultaneous applied axial compression force is greater than 0.2 P_o where P_o and M_o are defined in Article 6.9.5.3.2.

6.9.5.3—Combined Axial Compression and Flexure

6.9.5.3.1—General

The nominal resistance of CFST columns shall be based on a rational method of analysis. A cross-sectional analysis using the constituent materials as specified in Articles 6.9.5.3.2 and 6.9.5.3.3 and adjusted for stability of the column as specified in Article 6.9.5.3.4 is one method that satisfies these requirements. Other methods may be used with the owner's approval.

The connection of the column into the foundation and/or pier cap shall satisfy the requirements of Article 6.9.5.5.

6.9.5.3.2—Nominal Compressive Resistance

The factored compressive resistance, P_r, of a CFST column shall be determined as:

$$P_r = \phi_c P_n = \phi_c P_{cr} \quad (6.9.5.3.2\text{-}1)$$

where:

ϕ_c = resistance factor for compression and combined axial load and flexure specified in Article 6.9.5.1
P_n = nominal compressive resistance (kip)

The nominal compressive resistance, P_n, of a CFST column supporting only an axial load shall be determined using Eqs. 6.9.5.3.2-2 through 6.9.5.3.2-7.

If $P_e \geq 0.44 P_o$ then $P_{cr} = 0.658^{\frac{P_o}{P_e}} P_o$ (6.9.5.3.2-2)

If $P_e \leq 0.44 P_o$ then $P_{cr} = 0.877 P_e$ (6.9.5.3.2-3)

in which:

$$P_o = 0.95 f'_c A_c + F_{yst} A_{st} + F_{yb} A_{sb} \quad (6.9.5.3.2\text{-}4)$$

$$P_e = \frac{\pi^2 EI_{eff}}{\left(Kl^2\right)} \quad (6.9.5.3.2\text{-}5)$$

$$EI_{eff} = E_{st} I_{st} + E_{si} I_{si} + C' E_c I_c \quad (6.9.5.3.2\text{-}6)$$

$$C' = 0.15 + \frac{P}{P_o} + \frac{A_{st} + A_{si}}{A_{st} + A_{si} + A_c} \leq 0.9 \quad (6.9.5.3.2\text{-}7)$$

where:

A_{st} = cross-sectional area of the steel tube (in.^{2})
A_{sb} = total cross-sectional area of the internal reinforcement bars (in.^{2})
A_c = net cross-sectional area of the concrete (in.^{2})
E_c = elastic modulus of the concrete (ksi)
E_{st} = elastic modulus of the steel tube (ksi)
E_{si} = elastic modulus of the internal steel reinforcement (ksi)
EI_{eff} = effective composite flexural cross-sectional stiffness of the CFST
F_{yst} = specified minimum yield stress of the steel tube (ksi)
F_{yb} = specified minimum yield stress of the internal steel reinforcing bars (ksi)
f'_c = specified minimum 28-day compressive strength of the concrete (ksi)
I_c = uncracked moment of inertia of the concrete about the centroidal axis (in.^{4})
I_{st} = moment of inertia of the steel tube about the centroidal axis (in.^{4})
I_{si} = moment of inertia of the internal steel reinforcement about the centroidal axis (in.^{4})
K = effective length coefficient as specified in Article 4.6.2.5
l = unbraced length of the column (in.)
P = applied axial dead load (kips)
P_{cr} = nominal compressive resistance (kips)
P_e = Euler buckling load (kips)
P_o = maximum compressive load resistance of the column without consideration of buckling (kips)

Wherever practical, use of internal reinforcement should be avoided.

C6.9.5.3.2

The provisions of this article pertain to CFSTs with and without internal reinforcement. Research has demonstrated that the strength contribution provided by the steel tube is significantly more than internal reinforcement (Roeder and Lehman, 2012). Further, internal reinforcement interferes with placement of the concrete fill. Therefore, internal reinforcement should be used only when required by other design constraints.

The procedure for designing composite CFST columns for axial compression is similar to that used for design of steel columns, except that a composite flexural stiffness, EI_{eff}, is employed. The composite flexural stiffness increases with increasing compressive load and, hence, the factor C', which provides an estimate of the effective stiffness of the concrete as a function of the axial load. The flexural tangent stiffness of composite concrete-filled steel tubes depends on the degree of cracking in the concrete, and therefore depends on the strain levels. The flexural stiffness values provided by Eqs. 6.9.5.3.2-6 and 7 correspond to approximately 90 percent of the maximum resistance of the member, since this provides a conservative estimate of buckling load. The flexural stiffness equations have been developed by comparison with past experimental results on concrete filled steel tubes where the members have been loaded to loss of lateral load carrying capacity (Roeder, Lehman and Bishop, 2010).

The use of internal reinforcement is discouraged for CFST, because internal reinforcement is significantly less efficient in developing resistance than is the steel tube. However, it is recognized that many connection methods require internal reinforcement to transfer forces and moments to adjacent elements. These provisions do not prevent the use of such connections, but it should be noted that such connections are reinforced concrete connections rather than CFST connections and should be designed as such.

6.9.5.3.3—Nominal Composite Resistance

The nominal composite resistance of a CFST subjected to axial load and moment shall be determined by the plastic stress distribution method. The compressive stress distribution in the concrete shall be defined as a rectangular stress block with stress equal to $0.95f'_c$ and shall include the full depth of the concrete in compression. The tensile strength of the concrete shall be neglected. The stress in the steel tube and any reinforcement at any location in the cross section shall be equivalent to the minimum specified yield stress, F_{yst}, in both tension and compression. CFSTs subjected to axial compressive loads that are affected by buckling, secondary moments, or P-δ effects shall also satisfy the requirements of Article 6.9.5.3.4.

C6.9.5.3.3

Research shows that the PSDM is simple to use and is consistently more accurate than other methods (Roeder, Lehman and Bishop, 2010). The PSDM is illustrated in Figure C6.9.5.3.3-1. The method uses the full yield strength of the steel in tension and compression. Even under higher axial stress, the full yield strength is available because the concrete fill restrains local buckling of the steel and permits development of the full plastic capacity of the steel. A uniform concrete stress distribution with a magnitude of stress equal to $0.95f'_c$ over the entire compressive region is used. The coefficient of 0.95 is higher than the typical coefficient of 0.85 used for reinforced concrete flexural strength calculations in recognition of the increased confinement to the concrete (and resulting increased deformation capacity) provided by the circular steel tube. The compressive block is also taller, with the equivalent uniform stress assumed acting over the entire compressive region. The axial load, P, and bending moment, M, are in equilibrium with the stress state and the resulting P and M values define one point on the P-M interaction curve. Other points are defined for other neutral axis locations to fully establish the PSDM interaction curve.

Past research has shown that the PSDM provides a conservative estimate of the composite resistance of the CFST, and for a given specified axial load, the experimental flexural resistance is on average 24 percent larger than that predicted by the PSDM (Roeder, Lehman and Bishop, 2010). This overstrength is not used in strength design. However, seismic design requires that less ductile elements be designed for the expected maximum plastic capacity of the ductile members. Given a specified axial load, the expected maximum bending moment of the CFST will be on average 1.24 times the moment obtained from the interaction curve and this value should be used to approximate the maximum demand transferred to any less-ductile connecting elements in a capacity-based design approach. This procedure has been developed based upon comparison with prior experimental results and extensive nonlinear calculations of the resistance of CFST members (Moon et al. 2012).

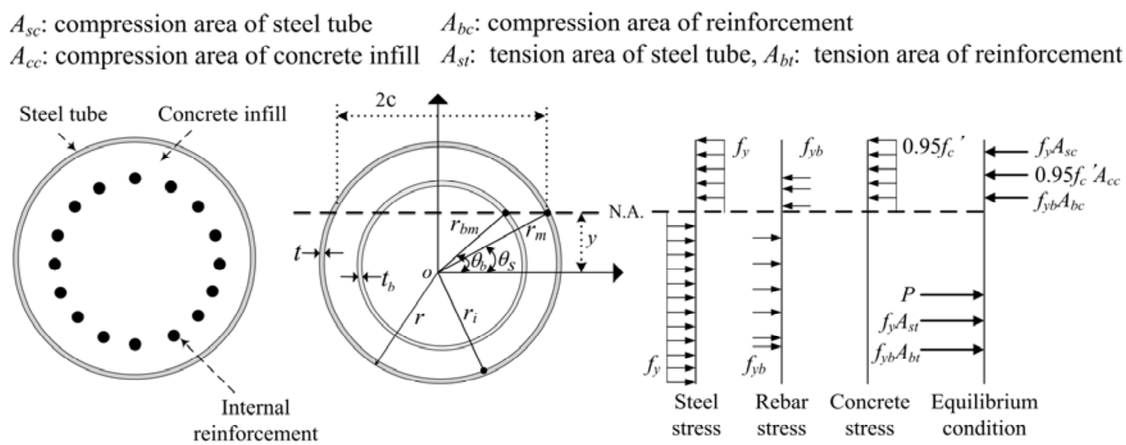


Figure C6.9.5.3.3-1—PSDM Model

For combined axial load and bending moment, the PSDM should be defined for multiple assumed locations of the neutral axis to define a material-based axial force-moment interaction curve, such as illustrated in Figure C6.9.5.3.3-2. This development is analogous to determining interaction curves for concrete columns. This interaction curve defines the-material based resistance of CFST components that are not affected by buckling, secondary moments or P-δ effects.

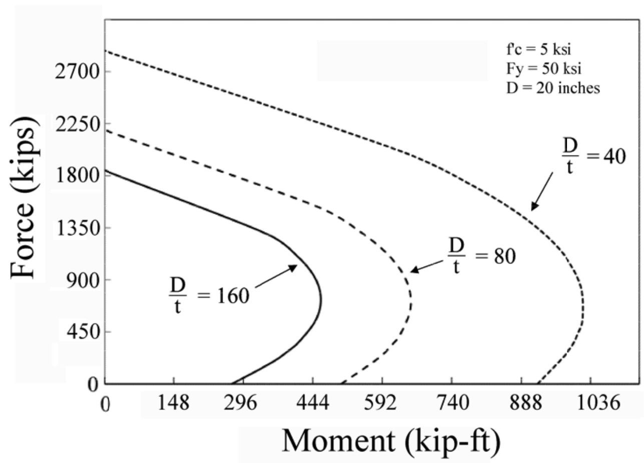


Figure C6.9.5.3.3-2—Interaction Curve

Closed-form solutions of the complete material based interaction curves have been developed for the PSDM and are provided in Eqs. C6.9.5.3.3-1 through C6.9.5.3.3-8 for circular CFST with no internal reinforcement or with one radially symmetric row of internal reinforcement:

$$P_n = F_{yst} t r_m \{(\pi-2\theta_s)-(\pi+2\theta_s)\} + t_b r_b \{F_{yb}(\pi-2\theta_b)-(F_{yb}-0.95f'_c)(\pi+2\theta_b)\}$$

$$+\frac{0.95 f'_c}{2}\{(\pi-2\theta_s)r_i^2-2yc\} \qquad \text{(C6.9.5.3.3-1)}$$

$$M_n = 0.95 f'_c c\left\{\left(r_i^2-y^2\right)-\frac{c^2}{3}\right\}+4F_{ys}tc\frac{r_m^2}{r_i}+4F_{yb}t_b c_b r_b \qquad \text{(C6.9.5.3.3-2)}$$

in which:

$$r_m = r - \frac{t}{2} \qquad \text{(C6.9.5.3.3-3)}$$

$$\theta_s = \sin^{-1}\left(\frac{y}{r_m}\right) \qquad \text{(C6.9.5.3.3-4)}$$

$$\theta_b = \sin^{-1}\left(\frac{y}{r_b}\right) \qquad \text{(C6.9.5.3.3-5)}$$

$$c = r_i \cos\theta_s \qquad \text{(C6.9.5.3.3-6)}$$

$$c_b = r_b \cos\theta_b \qquad \text{(C6.9.5.3.3-7)}$$

$$t_b = \frac{nA_b}{2\pi r_b} \qquad \text{(C6.9.5.3.3-8)}$$

where:

A_b = area of a single bar for the internal reinforcement (in.^{2})
C = one half the chord length for a given stress state as shown in Figure 6.9.5.3.3-1 (in.)
c_b = one half the chord length for a given stress state of a fictional tube modeling the internal reinforcement (in.)
F_{yb} = specified minimum yield stress of the steel bars used for internal reinforcement (ksi)
F_{yst} = specified minimum yield stress of the steel tube (ksi)
f'_c = 28-day compressive strength of the concrete (ksi)

M_o = composite plastic moment resistance of the CFST without axial load (kip-in.)
M_n = nominal flexural resistance as function of nominal axial resistance, P_n, for a given stress state (kip-in.)
n = number of uniformly spaced internal reinforcing bars as shown in Figure 6.9.5.3.3-1
P_o = plastic crush capacity of the CFST column without flexural moment (kips)
P_n = nominal compressive resistance of the member as function of nominal bending moment for a given stress state (kips)
r = radius to the outside of the steel tube as shown in Figure 6.9.5.3.3-1 (in.)
r_b = radius to the center or the internal reinforcing bars as shown in Figure 6.9.5.3.3-1 (in.)
r_i = radius to the inside of the steel tube as shown in Figure 6.9.5.3.3-1 (in.)
r_m = radius to the center or the steel tube as shown in Figure 6.9.5.3.3-1 (in.)
t = thickness of steel tube as shown in Figure 6.9.5.3.3-1 (in.)
t_b = thickness of a fictional steel tube used to model the contribution of the internal reinforcement as shown in Figure 6.9.5.3.3-1 (in.)
y = distance from the center of the tube to neutral axis for a given stress state as shown in Figure 6.9.5.2.3-1 (in.)
θ_b = angle used to define the c_b for a given stress state (radians)
θ_s = angle used to define the c for a given stress state (radians)

Smaller D/t values result in larger resistance, because the area of steel is larger. Larger D/t ratios result in significantly increased bending moment for modest compressive loads, because of the increased contribution of concrete fill.

For the case where no internal reinforcement is used inside the tube, A_b and t_b are equal to zero and several terms do not contribute to the resistance. A positive value of P implies a compressive force, and y and θ are positive according to the sign convention shown in Figure C6.9.5.3.2-1. The variable y varies between plus and minus r_i. The P-M interaction curve is generated by solving the equations for discrete values of y, and connecting those points.

6.9.5.3.4—Stability

A series of points defined herein shall be joined to form the interaction curve for CFST members, and Figure 6.9.5.3.3-1 illustrates the construction. The points shall be defined as follows:

- Point A is P_o, determined as specified in Article 6.9.5.3.2.
- Point *A*' is obtained by multiplying the axial load associated with point A by the ratio, P^n/P_o, where P_n is determined in Article 6.9.5.3.3.
- Point *A*'' is the intersection of the material-based interaction curve determined as specified in Article 6.9.5.3.3 and a horizontal line through Point *A*'.
- Point B is the flexural resistance without an axial load, M_o, as determined by the PSDM specified in Article 6.9.5.3.3.
- Point C corresponds to the axial force, PC, on the material-based interaction curve determined as specified in Article 6.9.5.3.3 that corresponds to the moment capacity without axial load, M_o (Point *B*).
- Point D is located on the material based interaction curve specified in Article 6.9.5.3.3 with the axial load, P_D, such that:

$$P_D = 0.5 P_C \, P_n / P_o \quad (6.9.5.3.4\text{-}1)$$

The stability based interaction curve defining the nominal resistance shall be constructed by joining points A', A'', D, and B, and shall be taken to define the nominal composite resistance, R_n, at the strength limit state.

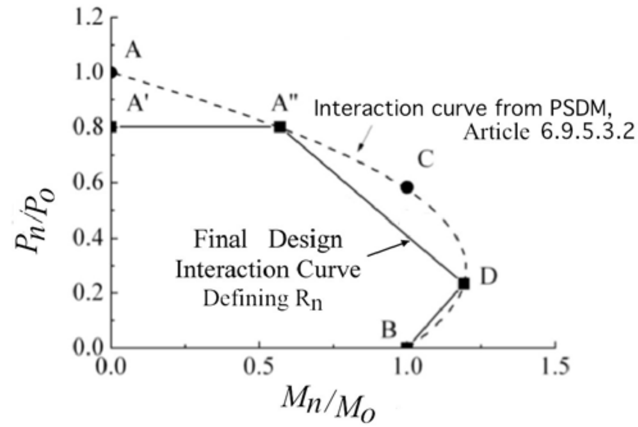


Figure 6.9.5.3.4-1—Construction of the Stability-Based Interaction Curve Defining R_n

C6.9.5.3.4

The interaction curve of Figure 6.9.5.3.4-1 accounts for global buckling of the CFST. The axial resistance is limited by the computed buckling capacity. The stability-based interaction curve includes stability effects and is a modified version of the material interaction curve in Figure 2.12.3.3.3-2, where the modification is based upon the buckling load computed from Eqs. 6.9.5.3.2-1 and 6.9.5.3.2-2. This interaction curve should then be used for strength design of the CFST for all load conditions.

6.9.5.4—Shear Resistance of CFST

The factored shear resistance, V_r, of circular CFST components shall be determined by:

$$V_r \leq \phi 0.6 F_{yst} A_{shear} = \phi 0.6 F_{yst} 0.5 \pi t D \qquad (6.9.5.4\text{-}1)$$

where:

D = diameter of the steel tube (in.)
F_{yst} = minimum specified yield stress of the steel tube (ksi)
t = thickness of the steel tube (in.)
ϕ = resistance factor for shear specified in Article 6.9.5.1

C6.9.5.4

Few experiments have been designed to specifically evaluate the shear resistance of CFSTs. Eq. 6.9.5.4-1 neglects all shear strength of the concrete fill. Nonlinear analysis and very limited experimental data shows that the observation that the equation is a conservative prediction, because it considers only half the steel section and neglects all contribution of the concrete, and also demonstrates that shear forces below this limit do not adversely affect the flexural or axial resistance of the member (Lehman et al., 2012). However, there is insufficient data to justify a larger shear resistance at this time. While the predicted shear resistance is conservative, since it neglects composite behavior, the shear resistance provided by this equation for CFST members designed by these provisions exceeds the maximum shear resistance provided by a reinforced concrete member of the same diameter by a large amount.

6.9.5.5—End Connections

Connections joining CFST to other structural members shall be designed by rational design methods. Articles 6.9.5.5.1 through 6.9.5.5.6 define a full-strength, fully-restrained, composite CFST connection for joining CFST columns-to-reinforced concrete footings, pile caps or pier caps.

6.9.5.5.1—General

Full-strength and fully-restrained embedded connections of CFST columns-to-footings, pile caps or pier caps shall be designed by either Option 1 or 2.

- Option 1 is a monolithic fully embedded connection, as illustrated in Figure 6.9.5.5.1-1.
- Option 2 is a grouted, embedded connection, as illustrated in Figure 6.9.5.5.1-2.

For both options, the steel tube shall have an annular ring welded to the end of the tube satisfying the provisions of Article 6.9.5.5.2. Option 1 connections shall be embedded into the reinforced concrete foundation component with an embedment depth and footing depth specified in Articles 6.9.5.5.3 and 6.9.5.5.4 and with the foundation design meeting the requirements of Article 6.9.5.5.5. Connections meeting the Option 2 requirements shall satisfy the requirements for Option 1 and shall also satisfy Article 6.9.5.5.6.

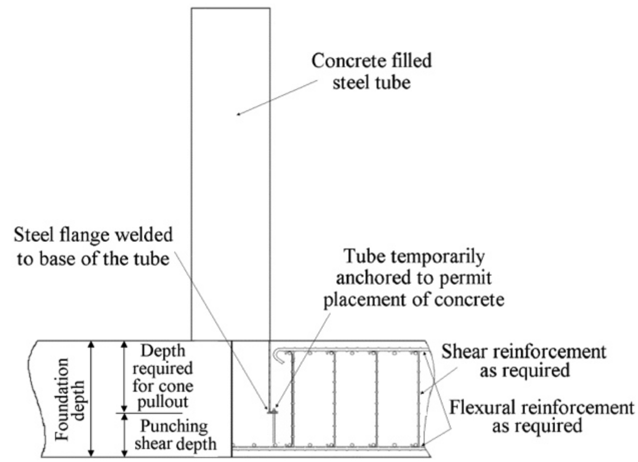


Figure 6.9.5.5.1-1—Option 1 - Monolithic Fully Embedded Connection

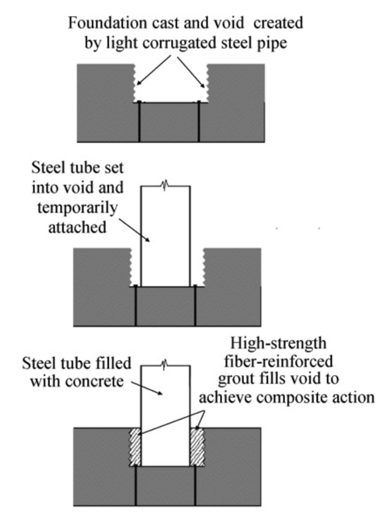


Figure 6.9.5.5.1-2—Option 2 – Grouted Embedded Connection

C6.9.5.5.1

Two fully restrained connections for CFST columns to concrete foundation elements including footings and pile caps are illustrated in Figures 6.9.5.5.1-1 and 6.9.5.5.1-2. These connections have been researched extensively

(Lehman and Roeder, 2012). Both options employ a flange or annular ring, which is attached to the bottom of the concrete filled tube and then embedded into the reinforced concrete component. These anchored connections resist flexural loading from the column through strutting action to the top and bottom of the foundation. Tests show this connection is both simple to construct and fully effective in transferring the flexural strength of the CFST column. Further, the connection develops significant inelastic deformation and is suitable for seismic design.

Option 1 uses monolithic construction, as illustrated in Figure 6.9.5.5.1-1. The tube and annular ring are temporarily anchored and cast into the reinforced concrete footing or pile cap. Option 2 isolates the tube from the foundation and permits independent placement of the reinforcement in the foundation element. The footing or pile cap is cast with a recess formed by light gauge corrugated metal pipe for later placement of the tube and annular ring as depicted in Figure 6.9.5.5.1-2. High-strength, fiber-reinforced grout is required between the corrugated metal duct and the CFST column to permit transfer of high anchorage stresses and minimize crack propagation. The concrete fill should be cast following placement of the grout.

6.9.5.5.2—Annular Ring

The annular ring shall have the same thickness of the steel tube and a minimum specified yield stress equal or greater than that of the steel tube. The ring shall extend outside the tube 16 times the thickness of the tube and project inside the tube 8 times the thickness of the tube, as shown in Figure 6.9.5.5.2-1.

The annular ring shall be welded to the tube with complete joint penetration (CJP) welds of matching filler metal or fillet welds on the both the inside and outside of the tube. The minimum size, w, of the fillets shall be determined as:

$$w > (F_u t)/(0.75 (2)\, 1.2\, (0.6)\, F_{EXX} (.707)) = 1.31 F_u t/F_{EXX} \qquad (6.9.5.5.2\text{–}1)$$

where F_{EXX} and F_u are the minimum tensile strength of the weld metal and tube steel, respectively. The welds shall provide a minimum CVN toughness of 40 ft-lbs at 70^{o}F.

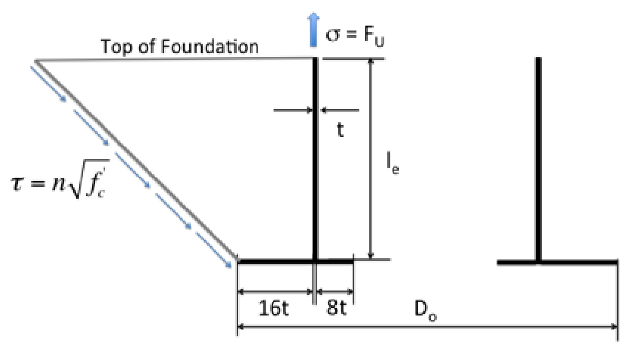


Figure 6.9.5.5.2-1—Geometry of Annular Ring

C6.9.5.5.2

The annular ring is welded to the end of the tube to provide anchorage and stress distribution, as illustrated in Figure 6.9.5.5.2-1. The thickness, projections, and material properties of the annular ring are based upon extensive experimental results (Lehman and Roeder, 2012).

The annular ring provides temporary attachment of the tube during construction of the footing. The ring then develops the overturning resistance of the finished connection. Tensile stress is developed in the tube penetrating into the connection. As a result, the welds joining the tube to the annular ring are designed to develop the tensile resistance of the steel tube.

6.9.5.5.3—Embedment Depth

The minimum embedment depth, l_e, shall be determined as:

$$l_e = \sqrt{\frac{D_o^2}{4} + \frac{5.27DtF_u}{\sqrt{f_{cf}^{'}}}} - \frac{D_o}{2} \qquad (6.9.5.5.3\text{-}1)$$

where:

F_u = the ultimate tensile strength of steel tube (ksi)
f'_{cf} = the minimum compressive strength of the concrete in the footing (ksi)
D = outside diameter of the steel tube (in)
D_o = the outside diameter of the annular ring for Option 1 connections and the inside diameter of corrugate pipe of Option 2 connections (in)
F_u = the specified minimum tensile strength of the steel tube (ksi)

C6.9.5.5.4

The embedded depth provisions are based upon developing the full tensile capacity of the CFST in flexure prior to developing cone pullout failure of the connection, as depicted in Figure 6.9.5.5.2-1. The failure cone is assumed to form at a 45-degree angle and can sustain a maximum shear stress of 0.189 $\sqrt{f'_c}$ (in ksi units). The cone depth and concrete shear stress limits were derived using results from an extensive experimental program (Lehman and Roeder, 2012). Some test specimens with embedment depths shallower than required by Eq. 6.9.5.5.3-1 provided good inelastic performance with considerable inelastic deformation capacity, but some connections with inadequate embedment suffered sudden and dramatic failures with severe cracking of the reinforced concrete footing. All specimens with embedment depth greater than Eq. 6.9.5.5.3-1 provided full strength connections with ductile inelastic performance and large inelastic deformation capacity.

6.9.5.5.4—Punching Shear and Total Foundation Depth

The tube shall have adequate foot depth, d_f, and concrete depth, *h*, below the CFST to avoid punching through the base of the footing. As a minimum, the following requirements shall be satisfied:

$$h \geq 32t \qquad (6.9.5.5.4\text{-}1)$$

$$d_f = l_e + h \geq \sqrt{\frac{D_o^2}{4} + \frac{7.91C_{max}}{\sqrt{f_{cf}^{'}}}} - \frac{D}{2} \qquad (6.9.5.5.4\text{-}2)$$

$$C_{max} = C_s + C_c \qquad (6.9.5.5.4\text{-}3)$$

where:

C_c = the compression forces in the concrete due to the combined bending and axial load as computed by the PSDM for the most extreme combined load case (kips)
C_s = the compression forces in the steel due to the combined bending and axial load as computed by the PSDM for the most extreme combined load case (kips)
D = outside diameter of the tube (in.)
f'_{cf} = the specified minimum 28-day compressive strength of the concrete in the footing (ksi)

C6.9.5.5.4

The footing depth, d_f, will normally be controlled by foundation design requirements such as those defined in Section 10 of these provisions. In addition, the combination of cone pullout requirements and depth requirement to meet the punching shear requirements also establish a minimum footing or pile cap thickness. Several methods may be used for punching shear evaluation. The punching shear procedure for single shear is a conservative approach and is similar to that employed in ACI 381-11. In compression, the column carries the axial force, P_r, and the compression force from the moment couple; these forces correspond to the same load case. The forces C_c and C_s capture the combined compressive force (see Figure C6.9.5.3.2-1). However, unlike the tension case, the data show that a portion of the compressive force is distributed to the foundation through bond stress. Hence, Eq. 6.9.5.5.4-2 defines the full depth of the footing. The minimum value of *h* in Eq. 6.9.5.5.4-1 is chosen to assure that the CFST is not set too deep into the footing and is consistent with the minimum depth used in prior experimental research. This is similar to the force transfer mechanism for a reinforced concrete column.

6.9.5.5.5—Special Requirements for Foundation Design

6.9.5.5.5a—General

The footing shall have minimum thickness equal or greater than d_f specified in Article 6.9.5.5.4 and shall be designed to meet the requirements of Section 10 of these *AASHTO LRFD Specification* provisions.

6.9.5.5.5b—Minimum Foundation Width and Length

The width and length of the footing, b_f, shall satisfy:

$$b_f \geq D_o + 3.5 l_e \tag{6.9.5.5.5b-1}$$

where:

D_o = the outside diameter of the annular ring for Option 1 connections and the inside diameter of corrugate pipe of Option 2 connections (in.)
l_e = minimum embedment depth defined by Article 6.9.5.5.3

C6.9.5.5.5b

The minimum width and length of the footing given by Eq. 6.9.5.5.5b-1 is required to assure that there is adequate reinforced concrete in footing to prevent cone-pullout and punching-shear failures. The thickness of the footing must be designed to develop the load resistance required by Section 10 of these specifications, and to also satisfy the minimum thickness requirements of Article 6.9.5.5.4.

6.9.5.5.5c—Flexural Reinforcement

Longitudinal and transverse flexural reinforcement shall be spaced across the length and width of the footing to meet the flexural and shear demands and detailing requirements. Where top bars are interrupted to accommodate the tube, the continuous longitudinal bars shall be designed with adequate capacity to develop the required flexural resistance of the foundation.

Figure 6.9.5.5.5c-1 shows the configuration of the longitudinal reinforcing bars that do not penetrate the tube but are placed within the anchorage zone of the tube. The interrupted bars are needed; their contribution to the flexural resistance is limited. Each of the interrupted bars shall be hooked adjacent to the steel tube. The hook dimensions shall satisfy Article 5.11.2.4 of these specifications.

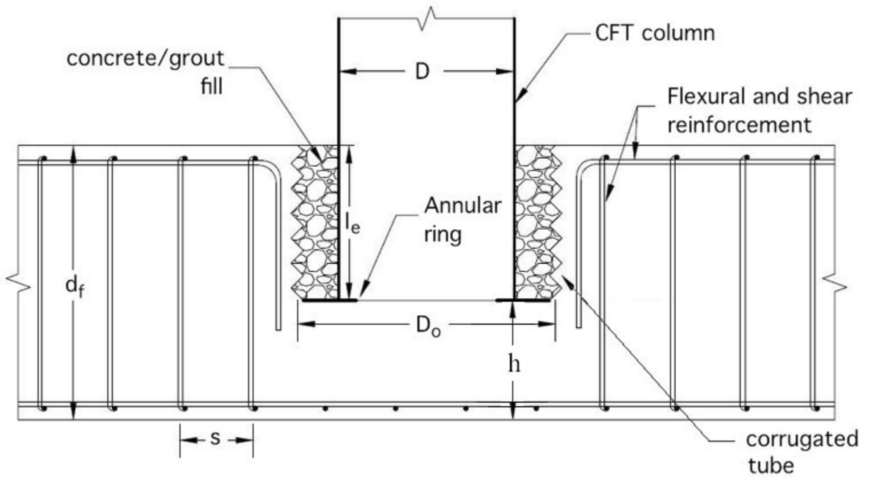


Figure 6.9.5.5.5c-1—Detailing of Reinforcement Adjacent to the Tube

C6.9.5.5.5c

The shear and flexural reinforcement in the footing must be designed for the normal shear and flexural loadings based upon the bridge loads, the soil conditions, and the expected capacity of the CFST. The top layer of flexural reinforcement will be interrupted by the concrete tube. The longitudinal bars that are not interrupted by the tube can be relied upon for their full tensile strength and therefore the flexural design must be based on them alone. The interrupted bars are needed to assure good transfer of stress around the connection, but these bars do not contribute to the flexural resistance of the footing.

The transverse reinforcement defined in Article 6.9.5.5.5c is required only within the radial distance of $1.5l_e$ of the steel tube, but the reinforcement within this region must be spaced no greater than s in the two orthogonal directions, and the reinforcing bars must be sized for the selected spacing to develop a uniform shear stress in the concrete according to the provisions of Article 5.8.3.3. It is recommended that the spacing of the transverse reinforcement outside of this region meet the shear strength provisions with a maximum spacing of 3*s*.

6.9.5.5.5d—Minimum Shear Reinforcement in Connection Region

Shear reinforcement of the footing shall have stirrups that are spaced no greater than *s* in both principal directions in the region within $1.5l_e$ of the outside of the tube. The spacing, s, shall satisfy:

$$s \leq \frac{l_e}{2.5} \qquad (6.9.5.5.5d\text{-}1)$$

where:

l_e = minimum embedment depth defined by Article 6.9.5.5.3

The vertical shear reinforcing bars shall be designed to resist a uniform shear stress of $0.126\sqrt{f'_c}$ (in ksi units).

6.9.5.5.6—Grout Requirements for Option 2

The foundation for Option 2 connections shall be cast with a recess formed to a depth, l_e, by a corrugated steel pipe having no less than 1.0-in. deep corrugations and meeting the requirements of AASHTO M218. The corrugated pipe shall be centered about the location of the CFST, and shall have an inside diameter larger than the outside diameter of the annular ring. The inside diameter of the corrugated pipe shall be at least 4.0 in. larger than the outside diameter of the annular ring, and not more than 10.0 in. larger than the outside diameter of the annular ring.

The contract documents shall require that after placement of the reinforced concrete footing, the tube shall then be grouted into the recess with a high strength grout reinforced with 0.2 percent (by volume) structural fiber reinforcement. The reinforced grout shall meet the requirements of ASTM Standard C-1107 and shall provide a minimum 28-day compressive strength that is at least $1.1f'_{cf}$, where f'_{cf} is the specified minimum 28-day compressive strength of the foundation concrete.

C6.9.5.5.6

This grouted connection has been tested in past research (Lehman and Roeder, 2012) and has been shown to provide superior anchorage into the footing, and good resistance to crack growth in the footing. A higher grout strength is required to provide optimal stress transfer in this critical region. The structural fibers mitigate cracking in the grout, also improving stress transfer.

Item # 3

Insert the following references to Article 6.17:

Moon, Jiho, Lehman, D.E., Roeder, C.W, and Lee, H-K, (2012) "Strength of Circular Concrete-Filled Tubes (CFT) with and without Internal Reinforcement under Combined Loading," *Journal of Structural Engineering*, ASCE, Reston, VA, DOI Information: 10.1061/(ASCE)ST.1943-541X.0000788.

Roeder, C. W., and Lehman, D.E., (2012) "Initial Investigation of Reinforced Concrete Filled Tubes for use in Bridge Foundations," *Research Report WA-RD 776.1*, Washington Department of Transportation, Olympia, WA, June 2012.

Roeder, Lehman and Bishop (2010),"Strength and Stiffness of Circular Concrete Filled Tubes," ASCE, *Journal of Structural Engineering*, Vol 135, No. 12, pgs 1545-53, Reston, VA.

Roeder, C.W, Lehman, D.E., and Thody, R. (2009) "Composite Action in CFT Components and Connections," AISC, *Engineering Journal*, Vol. 46, No. 4, Chicago, IL, pgs 229-42.

Lehman, D.E., and Roeder, C.W (2012) "Rapid Construction of Bridge Piers with Improved Seismic Performance," *California Department of Transportation* Report CA12-1972, Sacramento, CA.

Lehman, D.E., and Roeder, C.W., (2012) "Foundation Connections for Circular Concrete-Filled Tubes," *Journal of Constructional Steel Research* (2012), pp. 212-225 DOI information: 10.1016/j.jcsr.2012.07.001.

Item #4

Modify the Notation in Article 6.3 as necessary.

OTHER AFFECTED ARTICLES:

Delete Articles 6.12.2.3 and 6.12.3 and all associated commentary.

BACKGROUND:

First major update of composite column provisions in *AASHTO LRFD Bridge Design Specifications*.

ANTICIPATED EFFECT ON BRIDGES:

These provisions will permit the use of concrete filled steel tubes (CFST) for bridge piers and other structural elements. The use of CFST piers permits rapid construction of the pier, since no formwork or internal reinforcement is required. Further, CFST piers result in the less weight and material since the diameter of the pier will be 25% to 35% smaller than a comparable reinforced concrete pier of the same strength and stiffness.

CFST is also very suitable for seismic design, since it leads to lighter members with less material and smaller seismic design forces, while CFST with the connection defined in Article 6.9.5.5 provides ductility and inelastic deformation capacity equal to or greater than that achieved with a properly detailed reinforced concrete member.

REFERENCES:

Moon, Jiho, Lehman, D.E., Roeder, C.W, and Lee, H-K, (2012) "Strength of Circular Concrete-Filled Tubes (CFT) with and without Internal Reinforcement under Combined Loading," *Journal of Structural Engineering*, ASCE, Reston, VA, DOI Information: 10.1061/(ASCE)ST.1943-541X.0000788.

Roeder, C. W., and Lehman, D.E., (2012) "Initial Investigation of Reinforced Concrete Filled Tubes for use in Bridge Foundations," *Research Report WA-RD 776.1*, Washington Department of Transportation, Olympia, WA, June 2012.

Roeder, Lehman and Bishop (2010),"Strength and Stiffness of Circular Concrete Filled Tubes," ASCE, *Journal of Structural Engineering*, Vol 135, No. 12, pgs 1545-53, Reston, VA.

Roeder, C.W, Lehman, D.E., and Thody, R. (2009) "Composite Action in CFT Components and Connections," AISC, *Engineering Journal*, Vol. 46, No. 4, Chicago, IL, pgs 229-42.

Lehman, D.E., and Roeder, C.W (2012) "Rapid Construction of Bridge Piers with Improved Seismic Performance," *California Department of Transportation Report CA12-1972*, Sacramento, CA.

Lehman, D.E., and Roeder, C.W., (2012) "Foundation Connections for Circular Concrete-Filled Tubes," *Journal of Constructional Steel Research* (2012), pp. 212-225 DOI information: 10.1016/j.jcsr.2012.07.001.

OTHER:

None

2013 AASHTO BRIDGE COMMITTEE AGENDA ITEM: 23

SUBJECT: LRFD Bridge Design Specifications: Section 6, Articles C6.10.1 & 6.17

TECHNICAL COMMITTEE: T-14 Steel

☒ **REVISION** ☒ **ADDITION** ☐ **NEW DOCUMENT**

☒ **DESIGN SPEC** ☐ **CONSTRUCTION SPEC** ☐ **MOVABLE SPEC**
☐ **MANUAL FOR BRIDGE EVALUATION** ☐ **SEISMIC GUIDE SPEC** ☐ **COASTAL GUIDE SPEC**
☐ **OTHER**

DATE PREPARED: 1/20/13
DATE REVISED: 4/8/13

AGENDA ITEM:

Item # 1

Add the following after the 4th paragraph of Article C6.10.1:

White et al. (2012) present one suggested method of estimating I-girder flange lateral bending moments and stresses in straight-skewed I-girder bridges and curved I-girder bridges with or without skew. This method is particularly useful to account in a rational manner for the flange lateral bending moments and stresses resulting from a grillage or plate and eccentric beam analysis since these results cannot be obtained directly from such analyses.

Item #2

Revise the current 4th paragraph of Article C6.10.1 as follows:

For horizontally curved bridges, in addition to the potential sources of flange lateral bending discussed ~~in the preceding paragraph~~ previously, flange lateral bending effects due to curvature must always be considered at all limit states and also during construction.

Item # 3

Add the following reference to Article 6.17:

White, D.W., Coletti, D., Chavel, B.W., Sanchez, A., Ozgur, C., Jimenez Chong, J.M., Leon, R.T., Medlock, R.D., Cisneros, R.A., Galambos, T.V., Yadlosky, J.M., Gatti, W.J., and Kowatch, G.T. 2012. "Guidelines for Analytical Methods and Construction Engineering of Curved and Skewed Steel Girder Bridges," NCHRP Report 725, Transportation Research Board, National Research Council, Washington, D.C.

OTHER AFFECTED ARTICLES:

None

BACKGROUND:

White et al. (2012) suggests one method of estimating I-girder flange lateral bending moments and stresses primarily for use with refined analysis methods from which these results are not obtained directly (e.g. grid or grillage and plate and eccentric beam analysis methods) by directly using the calculated cross-frame forces determined from the analysis. This new paragraph provides a reference to this work.

ANTICIPATED EFFECT ON BRIDGES:

More accurate estimation of I-girder flange lateral bending moments and stresses, particularly when grillage or plate and eccentric beam analysis methods are employed.

REFERENCES:

See Item #3

OTHER:

None

2013 AASHTO BRIDGE COMMITTEE AGENDA ITEM: 24

SUBJECT: LRFD Bridge Design Specifications: Section 6, Article C6.10.1.6 & 6.17

TECHNICAL COMMITTEE: T-14 Steel

☒ **REVISION** ☒ **ADDITION** ☐ **NEW DOCUMENT**

☒ **DESIGN SPEC** ☐ **CONSTRUCTION SPEC** ☐ **MOVABLE SPEC**
☐ **MANUAL FOR BRIDGE EVALUATION** ☐ **SEISMIC GUIDE SPEC** ☐ **COASTAL GUIDE SPEC**
☐ **OTHER**

DATE PREPARED: 1/20/13
DATE REVISED: 4/8/13

AGENDA ITEM:

Item #1

In Article C6.10.1.6, revise the 1st sentence of the 8th paragraph as follows:

Eq. 6.10.1.6-2, or equivalently Eq. 6.10.1.6-3 as applicable, ~~simply~~ gives a maximum value of L_b for which $f_\ell = f_{\ell 1}$ in Eq. 6.10.1.6-4 or 6.10.1.6-5.

Item #2

In Article C6.10.1.6, revise the 3rd sentence of the 8th paragraph as follows:

Eqs. 6.10.1.6-4 and 6.10.1.6-5, which are ~~This equation, which is~~ an established form for estimating the maximum second-order elastic moments in braced beam-column members whose ends are restrained by other framing, ~~tends~~ tend to be significantly conservative for larger ~~unsupported~~ unbraced lengths associated with f_{bu} approaching F_{cr} (White et al., 2001).

Item #3

Add the following after the 9th paragraph of Article C6.10.1.6:

When determining the amplification of $f_{\ell 1}$ in horizontally-curved I-girders with $L_b/R \geq 0.05$, F_{cr} in Eqs. 6.10.1.6-4 and 6.10.1.6-5 may be determined from Eq. 6.10.8.2.3-8 or Eq. A6.3.3-8 by replacing L_b with $KL_b = 0.5L_b$. For girders with $L_b/R < 0.05$, L_b may be used. The use of $KL_b = 0.5L_b$ for $L_b/R \geq 0.05$ gives a better estimate of the amplification of the bending deformations associated with the boundary conditions for the flange lateral bending at intermediate cross-frame locations, which are approximately symmetrical, and assumes that an unwinding stability failure of the compression flange is unlikely for this magnitude of the girder horizontal curvature. Figure C6.10.1.6-1 illustrates qualitatively, using a straight elastic member for simplicity, the amplified second-order elastic flange lateral deflections associated with horizontal curvature effects as well as the unwinding stability failure mode.

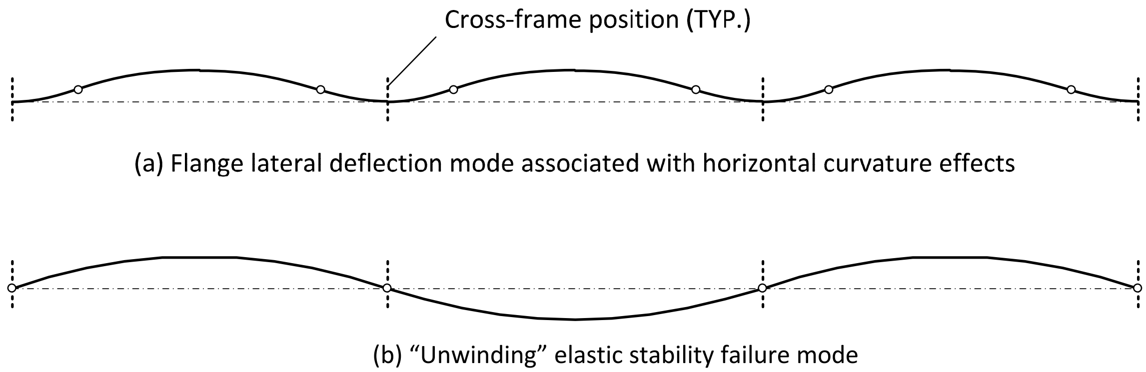


Figure C6.10.1.6-1—Second-order Elastic Lateral Deflections due to Horizontal Curvature Effects Versus the Unwinding Stability Failure Mode of the Compression Flange

Item #4

In Article 6.17, add the following reference:

White, D.W., Coletti, D., Chavel, B.W., Sanchez, A., Ozgur, C., Jimenez Chong, J.M., Leon, R.T., Medlock, R.D., Cisneros, R.A., Galambos, T.V., Yadlosky, J.M., Gatti, W.J., and Kowatch, G.T. 2012. "Guidelines for Analytical Methods and Construction Engineering of Curved and Skewed Steel Girder Bridges," NCHRP Report 725, Transportation Research Board, National Research Council, Washington, D.C.

OTHER AFFECTED ARTICLES:

The specification and commentary language proposed for inclusion in Articles 6.10.3.4 and C6.10.3.4 supplements the requirements of Article 2.5.3, which are referenced by Article 6.10.3.1, by providing specific guidelines for checking the global stability of certain multiple I-girder bridge units interconnected by cross-frames or diaphragms when in their noncomposite condition during the deck placement.

BACKGROUND:

The proposed additions are intended to clarify the application of Eqs. 6.10.1.6-4 or 6.10.1.6-5 for approximating second-order elastic lateral flange bending stresses. Furthermore, guidance is provided on when an effective length factor of 0.5 may be assumed appropriately in Eqs. 6.10.1.6-4 and 6.10.1.6-5.

ANTICIPATED EFFECT ON BRIDGES

The guidance provided on the use of $KL_b = 0.5L_b$ in Eqs. 6.10.1.6-4 and 6.10.1.6-5 leads to more accurate estimates of the actual second-order amplification of the flange lateral bending stresses in horizontally curved I-girder bridges in cases where these approximate equations are employed to determine the amplification effect.

The proposed changes to Articles 6.10.3.4 and C6.10.3.4 alert the Engineer to potential situations where global second-order amplification of the girder vertical and lateral displacements may lead to construction difficulties during the deck placement, and provide requirements to avoid these situations.

REFERENCES:

See Item #4

OTHER:

None

2013 AASHTO BRIDGE COMMITTEE AGENDA ITEM: 25

SUBJECT: LRFD Bridge Design Specifications: Section 6, Various Articles

TECHNICAL COMMITTEE: T-14 Steel

☒ **REVISION** ☒ **ADDITION** ☐ **NEW DOCUMENT**

☒ **DESIGN SPEC** ☐ **CONSTRUCTION SPEC** ☐ **MOVABLE SPEC**
☐ **MANUAL FOR BRIDGE EVALUATION** ☐ **SEISMIC GUIDE SPEC** ☐ **COASTAL GUIDE SPEC**
☐ **OTHER**

DATE PREPARED: 1/11/13
DATE REVISED: 4/15/13

AGENDA ITEM:

Make the revisions and additions to the indicated articles in Section 6 shown in Attachment A.

OTHER AFFECTED ARTICLES:

None

BACKGROUND:

Following the I-35W Bridge collapse investigation, the National Transportation Safety Board (NTSB) made five recommendations to the Federal Highway Administration (FHWA) and AASHTO. One of these recommendations was to require bridge owners to include main truss member gusset plates as part of the load rating process for these bridges.

To assist the states with this process, FHWA issued a Guidance document in February 2009. This document required, at a minimum, for main truss member gusset plates to be evaluated for five limit states using either the Load Factor Rating (LFR) or Load and Resistance Factor Rating (LRFR) philosophies.

The Guidance document was based on existing provisions in the AASHTO LRFD Bridge Design Specifications and the older AASHTO Standard Specifications for Highway Bridges along with engineering judgment. The FHWA Guidance document was thought to yield conservative gusset plate ratings. As States began to evaluate their inventory with the Guidance document, a need for more direction on some checks was identified, while some facets of other checks were thought to be too conservative. This was the case particularly for the shear reduction factor (Ω) associated with the shear yielding check, and the K-factor selection for use in the column analogy compressive buckling resistance check.

To address these concerns, FHWA initiated a research project collaboratively with the AASHTO-sponsored National Cooperative Highway Research Program (NCHRP) to evaluate the shear, tensile and compressive resistance of gusset plates at the strength limit state (NCHRP Project 12-84). The project tested 12 full-scale experimental gusset plate connections, and used finite element analysis to explore a variety of geometric parameters that could not be experimentally investigated. Primarily, the goal of NCHRP Project 12-84 was to derive new load rating provisions for inclusion in the MBE to satisfy NTSB Recommendation H-08-23, "*When the findings of the Federal Highway Administration–American Association of State Highway and Transportation Officials joint study on gusset plates become available, update the Manual for Bridge Evaluation accordingly*." A separate companion Agenda item is taking care of addressing these recommendations with significant proposed additional content to the MBE. A decision was made to ensure that the LRFR and LFR gusset plate rating specifications in the MBE are reasonably self-sufficient and do not refer back to the LRFD Bridge Design

Specifications or the Standard Specifications to a significant extent for determining the factored resistance of the gusset plate and its connections. Therefore, it is not imperative that the gusset plate design provisions derived from the research findings be included in the LRFD Bridge Design Specifications, although it makes sense to unify the two specifications for consistency, and to ensure that gusset plates on new truss bridges are designed based on the latest state-of-the-art knowledge in order to provide a more uniform reliability.

ANTICIPATED EFFECT ON BRIDGES:

The current specification provisions in LRFD Bridge Design Specification Article 6.14.2.8 allow the Engineer significant discretion in the design of truss member gusset plates. The new provisions are much more comprehensive and should result in a more unified design approach and a more uniform reliability for these gusset plate designs. The new provisions may result in thicker gusset plates than would be required using the current specifications, but the cost associated with thickening gusset plates is relatively marginal, and should be the only significant perceived difference when the proposed design specifications are employed.

REFERENCES:

See the revised Article 6.17 in Attachment A.

OTHER:

None

ATTACHMENT A – 2013 AGENDA ITEM 25 - T-14

Make the following revisions to Articles 6.2, 6.3, 6.5.4.2, 6.7.3, 6.13.6.1.5, 6.14.2.8 & 6.17 of the LRFD Bridge Design Specifications:

6.2—DEFINITIONS

Chord Splice—A connection between two discontinuous chord members in a truss structure, which may occur within or outside of a gusset plate.

Gusset Plate—Plate material used to interconnect vertical, diagonal, and horizontal truss members at a panel point, or to interconnect diagonal and horizontal cross-frame members for subsequent attachment of the cross-frame to transverse connection plates.

Whitmore Section—A portion of a truss gusset plate defined at the end of a member fastener pattern based on 30 degree dispersion patterns from the lead fastener through which it may be assumed for the purposes of design that all force from the member is evenly distributed into the gusset plate.

6.3—NOTATION

A_f = area of the inclined bottom flange (in.^{2}); area of a box flange including longitudinal flange stiffeners (in.^{2}); sum of the area of fillers on ~~the top and bottom~~ both sides of a connecting plate (in.^{2}); area of flange transmitting a concentrated load (in.^{2}) (C6.10.1.4) (C6.11.11.2) (6.13.6.1.5) (6.13.7.2)

A_g = gross area of a member (in.^{2}); gross cross-sectional area of the member (in.^{2}); gross area of the tension flange (in.^{2}); gross area of the section based on the design wall thickness (in.); gross cross-sectional area of the effective Whitmore section determined based on 30 degree dispersion angles (in.^{2}); gross area of all plates in the cross-section intersecting the spliced plane (in.^{2}) (6.6.1.2.3) (6.8.2.1) ~~(6.9.4.1.1) (6.9.4.1.3) (6.10.1.8)~~ (6.12.1.2.3c) (6.13.6.1.4c) (6.14.2.8.4) (6.14.2.8.6)

A_n = net cross-section area of a tension member (in.^{2}); net area of a flange (in.^{2}); net area of gusset and splice plates (in.^{2}) (6.6.1.2.3) (6.8.2.1) (6.10.1.8) (6.14.2.8.6)

A_p = smaller of either the connected plate area or the sum of the splice plate area on ~~the top and bottom~~ both sides of the connected plate (in.^{2}) (6.13.6.1.5)

A_{vg} = gross area along the cut carrying shear stress in block shear (in.^{2}); gross area of the connection element subject to shear (in.^{2}); gross area of gusset plate subject to shear (in.^{2}) (6.13.4) (6.13.5.3) (6.14.2.8.3)

e_p = distance between the centroid of the cross-section and the resultant force perpendicular to the spliced plane in gusset plates (in.) (6.14.2.8.6)

F_{cr} = critical buckling stress for plates (ksi); elastic lateral torsional buckling stress (ksi); shear buckling resistance (ksi); elastic local buckling stress (ksi); stress in the spliced section at the limit of usable resistance (ksi) (C6.9.4.2) (6.10.1.6) (6.12.1.2.3c) (6.12.2.2.3) ~~(6.12.2.2.5)~~ (6.14.2.8.6)

F_u = specified minimum tensile strength of steel (ksi); specified minimum tensile strength of a stud shear connector (ksi); specified minimum tensile strength of a connected part (ksi); tensile strength of a connected element (ksi); specified minimum tensile strength of a gusset plate (ksi) (6.4.1) (6.10.10.4.3) (6.13.2.9) (6.13.5.3) (6.14.2.8.6)

F_y = specified minimum yield strength of steel (ksi); specified minimum yield strength of a pin (ksi); specified minimum yield strength of a pin plate (ksi); specified minimum yield strength of a connected part (ksi); specified minimum yield strength of a splice plate (ksi); specified minimum yield strength of a gusset plate (ksi) (6.4.1) (6.7.6.2.1) (6.8.7.2) ~~(6.9.4.1.1) (6.12.2.2.4) (6.12.2.2.5) (6.12.2.2.7)~~ (6.13.4) (6.13.6.1.4c) (6.14.2.8.3)

K = effective length factor; effective length factor in the plane of buckling determined as specified in Article 4.6.2.5; effective column length factor taken as 0.50 for chord splices (6.9.3) (6.9.4.1.2) (6.14.2.8.6)

L_{mid} = in a gusset plate connection, the distance from the last row of fasteners in the compression member under consideration to the first row of fasteners in the closest adjacent connected member, measured along the line of action of the compressive axial force (6.14.2.8.4)

L_{splice} = in a gusset plate connection, the center-to-center distance between the first lines of fasteners in the adjoining chords at a chord splice (6.14.2.8.6)

P_e = elastic critical buckling resistance determined as specified in Article 6.9.4.1.2 for flexural buckling, ~~and~~ as specified in Article 6.9.4.1.3 for torsional bucking or flexural-torsional buckling, as applicable (kips); and as specified in Article 6.14.2.8.4 for gusset plate buckling (6.9.4.1.1) (6.14.2.8.4)

P_n = nominal bearing resistance on pin plates (kip); nominal axial compressive resistance (kip); total longitudinal force in the concrete deck over an interior support for the design of the shear connectors at the strength limit state, taken as the lesser of either P_{1n} or P_{2n} (kip); nominal compressive resistance of an idealized Whitmore section (kip) (6.8.7.2) (6.9.2.1) (6.10.10.4.2) (6.14.2.8.4)

P_r = factored axial tensile or compressive resistance (kip); factored bearing resistance on pin plates (kip); factored axial resistance of bearing stiffeners (kip); ~~nominal flexural resistance of an orthotropic deck, with consideration of the effective width of the deck (kip)~~; factored compressive resistance of gusset plates (kip); factored axial compressive resistance of a steel pile (kip)~~; factored compressive resistance determined as specified in Article 6.9.2.1 (kip)~~ (6.8.2.1) (6.8.7.2) ~~(6.9.2.2) (6.9.4.2.1) (6.9.4.3.2)~~ (6.10.11.2.4a) (6.14.2.8.4) (6.15.3.1)

R_r = factored resistance of a bolt, connection or connected material (kip) or (ksi); factored resistance in tension of connection elements (kip); factored tensile resistance of gusset plates (kip) (6.13.2.2) (6.13.5.2) (6.14.2.8.5)

S_g = elastic gross section modulus of gusset plates and splice plates (in.^{3}) (6.14.2.8.6)

S_n = elastic net section modulus of gusset plates and splice plates (in.^{3}) (6.14.2.8.6)

V_r = factored shear resistance (kip); factored shear resistance of gusset plates (kip) (6.12.1.2.3) (6.14.2.8.3)

t_g = gusset plate thickness (6.14.2.8.4)

ϕ_{cg} = resistance factor for truss gusset plate compression (6.5.4.2)

ϕ_{cs} = resistance factor for truss gusset chord splice (6.5.4.2)

ϕ_{vy} = resistance factor for truss gusset plate shear yielding (6.5.4.2)

θ = angle of inclination of the bottom flange of a variable web depth member (degrees); angle of inclination of the web plate of a box section to the vertical (degrees); framing angle of compression member relative to an adjoining member in a gusset-plate connection (C6.10.1.4) (6.11.9) (6.14.2.8.4)

Ω = shear yield reduction factor for gusset plates (6.14.2.8.3)

6.5.4.2—Resistance Factors

Resistance factors, ϕ, for the strength limit state shall be taken as follows:

- For flexure $\phi_f = 1.00$
- For shear $\phi_v = 1.00$
- For axial compression, steel only $\phi_c = 0.90$
- For axial compression, composite $\phi_c = 0.90$
- For tension, fracture in net section $\phi_u = 0.80$
- For tension, yielding in gross section $\phi_y = 0.95$
- For bearing on pins in reamed, drilled or bored holes and on milled surfaces $\phi_b = 1.00$
- For bolts bearing on material $\phi_{bb} = 0.80$
- For shear connectors $\phi_{sc} = 0.85$
- For A 325 and A 490 bolts in tension $\phi_t = 0.80$
- For A 307 bolts in tension $\phi_t = 0.80$
- For F 1554 bolts in tension $\phi_t = 0.80$
- For A 307 bolts in shear $\phi_s = 0.75$
- For F 1554 bolts in shear $\phi_s = 0.75$
- For A 325 and A 490 bolts in shear $\phi_s = 0.80$
- For block shear $\phi_{bs} = 0.80$
- For shear, rupture in connection element $\phi_{vu} = 0.80$
- <u>For truss gusset plate compression $\phi_{cg} = 0.75$</u>
- <u>For truss gusset plate chord splices $\phi_{cs} = 0.65$</u>
- <u>For truss gusset plate shear yielding $\phi_{vy} = 0.80$</u>
- For web crippling $\phi_w = 0.80$
- For weld metal in complete penetration welds:
  - shear on effective area $\phi_{e1} = 0.85$
  - tension or compression normal to effective area same as base metal
  - tension or compression parallel to axis of the weld same as base metal
- For weld metal in partial penetration welds:
  - shear parallel to axis of weld $\phi_{e2} = 0.80$
  - tension or compression parallel to axis of weld same as base metal
  - compression normal to the effective area same as base metal

    tension normal to the effective area $\phi_{e1} = 0.80$
- For weld metal in fillet welds:
  - tension or compression parallel to axis of the weld same as base metal
  - shear in throat of weld metal $\phi_{e2} = 0.80$
- For resistance during pile driving $\phi = 1.00$

C6.5.4.2

Base metal ϕ as appropriate for resistance under consideration.

<u>The resistance factors for truss gusset plates were developed and calibrated to a target reliability index of 4.5 for the Strength I load combination at a dead-to-live ratio, DL/LL, of 6.0. More liberal ϕ factors could be justified at a DL/LL less than 6.0.</u>

- For axial resistance of piles in compression and subject to damage due to severe driving conditions where use of a pile tip is necessary:
  - H-piles $\phi_c = 0.50$
  - pipe piles $\phi_c = 0.60$
- For axial resistance of piles in compression under good driving conditions where use of a pile tip is not necessary:
  - H-piles $\phi_c = 0.60$
  - pipe piles $\phi_c = 0.70$
- For combined axial and flexural resistance of undamaged piles:
  - axial resistance for H-piles $\phi_c = 0.70$
  - axial resistance for pipe piles $\phi_c = 0.80$
  - flexural resistance $\phi_f = 1.00$
- For shear connectors in tension $\phi_{st} = 0.75$

The basis for the resistance factors for driven steel piles is described in Article 6.15.2. Further limitations on usable resistance during driving are specified in Article 10.7.8.

Indicated values of ϕ_c and ϕ_f for combined axial and flexural resistance are for use in interaction equations in Article 6.9.2.2.

6.7.3—Minimum Thickness of Steel

Structural steel, including bracing, cross-frames, and all types of gusset plates, except for gusset plates used in trusses, webs of rolled shapes, closed ribs in orthotropic decks, fillers, and in railings, shall not be less than 0.3125 in. in thickness. The thickness of gusset plates used in trusses shall not be less than 0.375 in.

For orthotropic decks, the web thickness of rolled beams or channels and of closed ribs in orthotropic decks shall not be less than 0.25 in., the deck plate thickness shall not be less than 0.625 in. or four percent of the larger spacing of the ribs, and the thickness of closed ribs shall not be less than 0.1875 in.

Where the metal is expected to be exposed to severe corrosive influences, it shall be specially protected against corrosion or sacrificial metal thickness shall be specified.

6.13.6.1.5—Fillers

When bolts carrying loads pass through fillers 0.25 in. or more in thickness in axially loaded connections, including girder flange splices, either:

- The fillers shall be extended beyond the gusset or splice material, and the filler extension shall be secured by enough additional bolts to distribute the total stress in the member uniformly over the combined section of the member and the filler or

- As an alternative, the fillers need not be extended and developed provided that the factored resistance of the bolts in shear at the strength limit state, specified in Article 6.13.2.2, is reduced by the following factor:

$$R = \left[\frac{(1+\gamma)}{(1+2\gamma)} \right] \qquad (6.13.6.1.5\text{-}1)$$

where:

γ = A_f/A_p

A_f = sum of the area of the fillers on ~~the top and bottom of~~ both sides of the connected plate (in.^{2})

A_p = smaller of either the connected plate area or the sum of the splice plate areas on ~~the top and bottom~~ both sides of the connected plate (in.^{2}). For truss gusset plate chord splices, when considering the gusset plate(s), only the portion of the gusset plate(s) that overlaps the connected plate shall be considered in the calculation of the splice plate areas.

For slip-critical connections, the factored slip resistance of a bolt, specified in Article 6.13.2.2, shall not be adjusted for the effect of the fillers.

Fillers 0.25 in. or more in thickness shall consist of not more than two plates, unless approved by the Engineer.

For bolted web splices with thickness differences of 0.0625 in. or less, no filler plates are required.

The specified minimum yield strength of fillers 0.25 in. or greater in thickness should not be less than the larger of 70 percent of the specified minimum yield strength of the connected plate and 36.0 ksi.

6.14—PROVISIONS FOR STRUCTURE TYPES

6.14.2.8—Gusset Plates

6.14.2.8.1—General

~~The provisions of Articles 6.13.4 and 6.13.5 shall apply, as applicable.~~

Gusset or connection plates should be used for connecting ~~main~~ truss members, except where the members are pin-connected. The fasteners connecting each member shall be symmetrical with the axis of the member, so far as practicable, and the ~~full development of~~ connection of all the elements of the member should be given consideration to facilitate the load transfer.

Re-entrant cuts, except curves made for appearance, should be avoided as far as practicable.

~~The maximum stress from combined factored flexural and axial loads shall not exceed~~ $\cancel{\phi_f F_y}$ ~~based on the gross area.~~

~~The maximum shear stress on a section due to the factored loads shall be~~ $\cancel{\phi_v F_y/\sqrt{3}}$ ~~for uniform shear and~~ $\cancel{\phi_v\, 0.74\, F_y/\sqrt{3}}$ ~~for flexural shear computed as the factored shear force divided by the shear area.~~

~~If the length of the unsupported edge of a gusset~~

~~C6.14.2.8~~

~~Following the 2007 collapse of the I-35W bridge in Minneapolis, the traditional procedures for designing gusset plates, including the provisions of this Article, have been under extensive review. As of Spring 2008, new design procedures have not been codified. Guidance from FHWA is expected shortly. Designers are advised to obtain the latest approved recommendations from Owners.~~

C6.14.2.8.1

The provisions provided in this article are intended for the design of double gusset-plate connections used in trusses. The validity of the requirements for application to single gusset-plate connections has not been verified.

These provisions are based on the findings from NCHRP Project 12-84 (NCHRP, 2013). Example calculations illustrating the application of the resistance equations for gusset-plate connections contained herein are provided in NCHRP (2013).

~~plate exceeds 2.06(E/F_y)$^{1/2}$ times its thickness, the edge shall be stiffened. Stiffened and unstiffened gusset edges shall be investigated as idealized column sections.~~

Gusset plates shall satisfy the minimum plate thickness requirement for gusset plates used in trusses specified in Article 6.7.3. Gusset plates shall be designed for shear, compression, and/or tension occurring in the vicinity of each connected member, as applicable, according to the requirements specified in Articles 6.14.2.8.3 through 6.14.2.8.5. Gusset plates serving as a chord splice shall also be independently designed as a splice according to the provisions of Article 6.14.2.8.6. The edge slenderness requirement specified in Article 6.14.2.8.7 shall be considered.

Bolted gusset plate connections shall satisfy the applicable requirements of Articles 6.13.1 and 6.13.2. Where filler plates are required, the provisions of Article 6.13.6.1.5 shall apply.

6.14.2.8.2—Multi-Layered Gusset and Splice Plates

Where multi-layered gusset and splice plates are used, the resistances of the individual plates may be added together when determining the factored resistances specified in Articles 6.14.2.8.3 through 6.14.2.8.6 provided that enough fasteners are present to develop the force in the layered gusset and splice plates.

C6.14.2.8.2

Kulak et al. (1987) contains additional guidance on determining the number of fasteners required to develop the force in layered gusset and splice plates.

6.14.2.8.3—Shear Resistance

The factored shear resistance, V_r, of gusset plates shall be taken as the smaller value based on shear yielding or shear rupture.

For shear yielding, the factored shear resistance shall be taken as:

$$V_r = \phi_{vy} 0.58 F_y A_{vg} \Omega \quad (6.14.2.8.3\text{-}1)$$

where:

ϕ_{vy} = resistance factor for truss gusset plate shear yielding specified in Article 6.5.4.2

Ω = shear reduction factor for gusset plates taken as 0.88

A_{vg} = gross area of the shear plane (in.^{2})

F_y = specified minimum yield strength of the gusset plate (ksi)

For shear rupture, the factored shear resistance shall be determined from Eq. 6.13.5.3-2.

Shear shall be checked on relevant partial and full failure plane widths. Partial shear planes shall only be checked around compression members and only Eq. 6.14.2.8.3-1 shall apply to partial shear planes. The partial shear plane length shall be taken along adjoining member fastener lines between plate edges and other fastener lines. The following partial shear planes, as

C6.14.2.8.3

The Ω shear reduction factor is used only in the evaluation of truss gusset plates for shear yielding. This factor accounts for the nonlinear distribution of shear stresses that form along a failure plane as compared to an idealized plastic shear stress distribution. The nonlinearity primarily develops due to shear loads not being uniformly distributed on the plane and also due to strain hardening and stability effects. The Ω-factor was developed using shear yield data generated in NCHRP Project 12-84 (NCHRP, 2013). On average, Ω was 1.02 for a variety of gusset-plate geometries; however, there was significant scatter in the data due to proportioning of load between members, and variations in plate thickness and joint configuration. The specified Ω-factor has been calibrated to account for shear plane length-to-thickness ratios varying from 85 to 325.

Failure of a full width shear plane requires relative mobilization between two zones of the plate, typically along chords. Mobilization cannot occur when a shear plane passes through a continuous member; for instance, a plane passing through a continuous chord member that would require shearing of the member itself.

Research has shown that the buckling of connections with tightly spaced members is correlated with shear yielding around the compression members. This is important because the buckling criteria used in Article 6.14.2.8.4 would overestimate the compressive applicable, shall be evaluated to determine which shear plane controls:

- The plane that parallels the chamfered end of the compression member, as shown in Figure 6.14.2.8.3-1;
- The plane on the side of the compression member that has the smaller framing angle between the that member and the other adjoining members, as shown in Figure 6.14.2.8.3-2; and
- The plane with the least cross-sectional shear area if the member end is not chamfered and the framing angle is equal on both sides of the compression member.

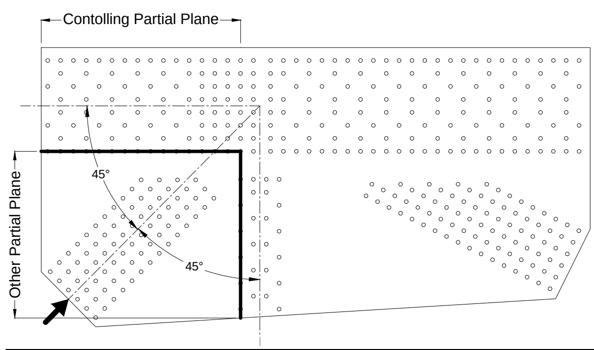


Figure 6.14.2.8.3-1—Example of a Controlling Partial Shear Plane that Parallels the Chamfered End of the Compression Member Since that Member Frames in at an Angle of 45 Degrees to Both the Chord and the Vertical

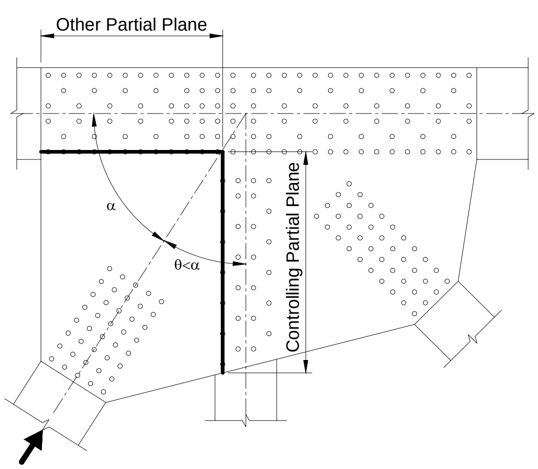


Figure 6.14.2.8.3-2—Example of a Controlling Partial Shear Plane on the Side of a Compression Member Without a Chamfered End that has the Smaller Framing Angle between that Member and the Other Adjoining Members (i.e. $\theta < \alpha$)

buckling resistance of these types of connections. Once a plane yields in shear, the reduction in the plate modulus reduces the out-of-plane stiffness such that the stability of the plate is affected. Generally, truss verticals and chord members are not subject to the partial plane shear yielding check because there is no adjoining member fastener line that can yield in shear and cause the compression member to become unstable. For example, the two compression members shown in Figure C6.14.2.8.3-1 would not be subject to a partial plane shear check.

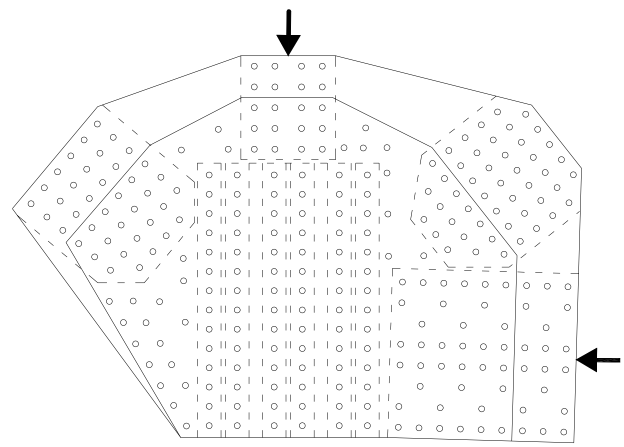


Figure C6.14.2.8.3-1—Example Showing Truss Vertical and Chord Members in Compression that Do Not Require a Partial Shear Plane Check

6.14.2.8.4—Compressive Resistance

The factored compressive resistance, P_r, of gusset plates shall be taken as:

$$P_r = \phi_{cg} P_n \quad (6.14.2.8.4\text{-}1)$$

where:

ϕ_{cg} = resistance factor for truss gusset plate compression specified in Article 6.5.4.2

P_n = nominal compressive resistance of a Whitmore section determined from Eq. 6.9.4.1.1-1 or 6.9.4.1.1-2, as applicable (kip)

In the calculation of P_n, the slender element reduction factor, Q, shall be taken as 1.0, and the elastic critical buckling resistance, P_e, shall be taken as:

$$P_e = \frac{3.29E}{\left(\frac{L_{mid}}{t_g}\right)^2} A_g \quad (6.14.2.8.4\text{-}2)$$

where:

A_g = gross cross-sectional area of the Whitmore section determined based on 30 degree dispersion angles, as shown in Figure 6.14.2.8.4-1 (in.^{2}). The Whitmore section shall not be reduced if the section intersects adjoining member bolt lines

L_{mid} = distance from the middle of the Whitmore section to the nearest member fastener line in the direction of the member, as shown in Figure 6.14.2.8.4-1 (in.)

t_g = gusset-plate thickness (in.)

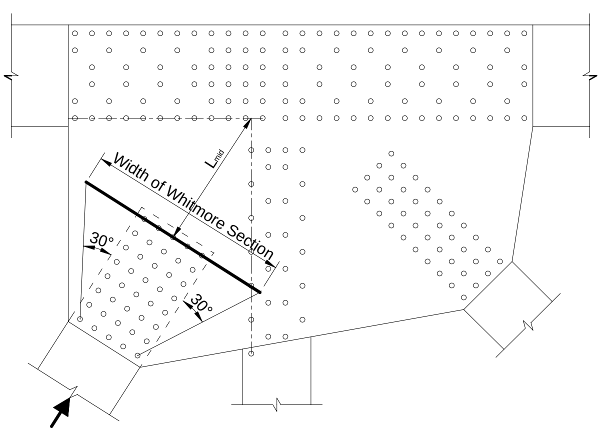


Figure 6.14.2.8.4-1—Example Connection Showing the Whitmore Section for a Compression Member Derived from 30 Degree Dispersion Angles and the Distance L_{mid}

C6.14.2.8.4

Gusset plate zones in the vicinity of compression members are to be designed for plate stability. Experimental testing and finite element simulations performed as part of NCHRP Project 12-84 (NCHRP, 2013) and by others (Yamamoto et al., 1988; Hafner et al., 2012) have found that truss gusset plates subject to compression always buckle in a sidesway mode in which the end of the compression member framing into the gusset plate moves out-of-plane. The buckling resistance is dependent upon the chamfering of the member, the framing angles of the members entering the gusset, and the standoff distance of the compression member relative to the surrounding members; i.e. the distance, L_{mid}. An example connection showing a typical chamfered member end and member framing angle is provided in Figure C6.14.2.8.4-1. The research found that the compressive resistance of gusset plates with large L_{mid} distances was reasonably predicted using modified column buckling equations and Whitmore section analysis. When the members were heavily chamfered reducing the L_{mid} distance, the buckling of the plate was initiated by shear yielding on the partial shear plane adjoining the compression member causing a destabilizing effect, as discussed in Article C6.14.2.8.3.

Eq. 6.14.2.8.4-2 is derived by substituting plate properties into Eq. 6.9.4.1.2-1 along with an effective length factor of 0.5 that was found to be relevant for a wide variety of gusset-plate geometries (NCHRP, 2013).

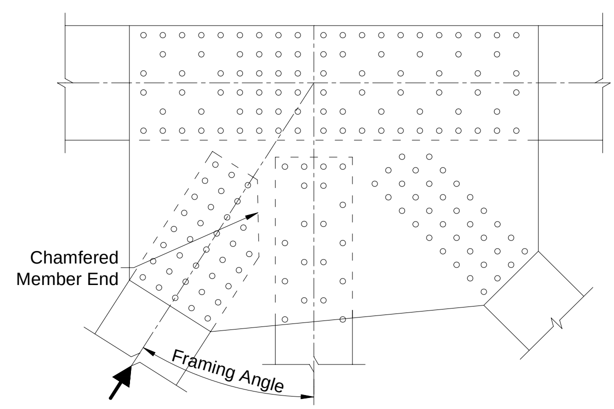


Figure C6.14.2.8.4-1–Example Connection Showing a Typical Chamfered Member End and Member Framing Angle

The provisions of this Article shall not be applied to compression chord splices.

6.14.2.8.5—Tensile Resistance

The factored tensile resistance, R_r, of gusset plates shall be taken as the smallest factored resistance in tension based on yielding, fracture or block shear rupture determined according to the provisions of Article 6.13.5.2. When checking Eqs. 6.8.2.1-1 and 6.8.2.1-2, the Whitmore section defined in Figure 6.14.2.8.5-1 shall be used to define the effective area. The Whitmore section shall not be reduced if the width intersects adjoining member bolt lines.

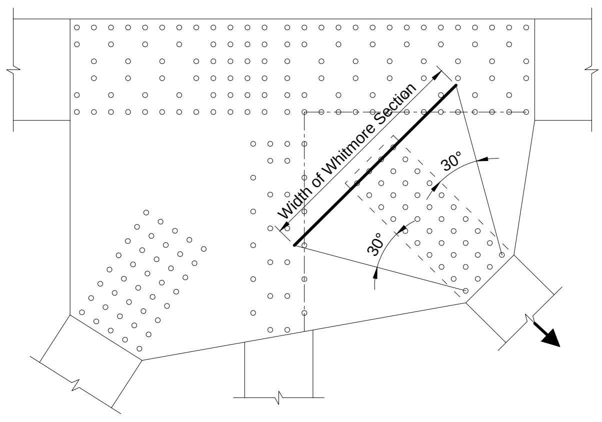


Figure 6.14.2.8.5-1—Example Connection Showing the Whitmore Section for a Tension Member Derived from 30 Degree Dispersion Angles

The provisions of this Article shall not be applied to tension chord splices.

6.14.2.8.6—Chord Splices

Gusset plates that splice two chord sections together shall be checked using a section analysis considering the relative eccentricities between all plates crossing the splice and the loads on the spliced plane.

For compression chord splices, the factored compressive resistance, P_r, of the spliced section shall be taken as:

$$P_r = \phi_{cs} F_{cr} \left(\frac{S_g A_g}{S_g + e_p A_g} \right) \qquad (6.14.2.8.6\text{-}1)$$

in which:

F_{cr} = stress in the spliced section at the limit of usable resistance (ksi). F_{cr} shall be taken as the specified minimum yield strength of the gusset plate when the following equation is satisfied:

C6.14.2.8.6

This Article is not intended to cover the design of chord splices that occur outside of the gusset plates; this situation is covered by the provisions of Article 6.13.6.1.2 or 6.13.6.1.3, as applicable. For gusset plates also serving the role of a chord splice, the forces from all members framing into the connection must be considered. The chord splice forces are the resolved axial forces acting on each side of the spliced section, as illustrated in Figure C6.14.2.8.6-1. The chord splice should be investigated for the larger of the two resolved forces on either side of the splice.

$$\frac{Kl\sqrt{12}}{t_g} < 25 \qquad (6.14.2.8.6\text{-}2)$$

where:

ϕ_{cs} = resistance factor for truss gusset plate chord splices specified in Article 6.5.4.2

A_g = gross area of all plates in the cross-section intersecting the spliced plane (in.^{2})

e_p = distance between the centroid of the cross-section and the resultant force perpendicular to the spliced plane (in.)

K = effective column length factor taken as 0.50 for chord splices

L_{splice} = center-to-center distance between the first lines of fasteners in the adjoining chords as shown in Figure 6.14.2.8.6-1 (in.)

S_g = gross section modulus of all plates in the cross-section intersecting the spliced plane (in.^{3})

t_g = gusset plate thickness (in.)

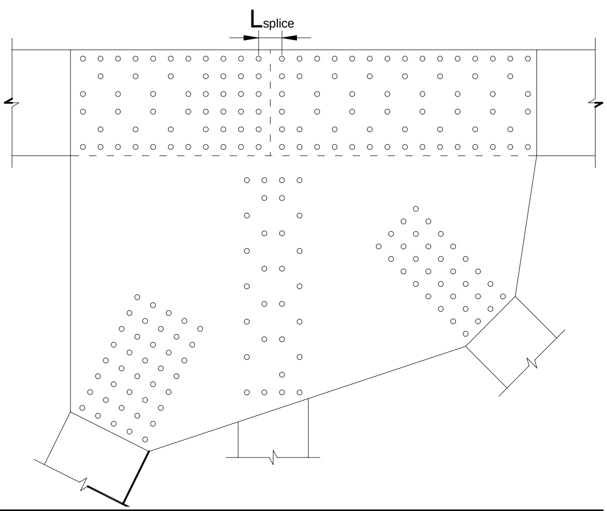


Figure 6.14.2.8.6-1—Example Connection Showing Chord Splice Parameter, L_{splice}

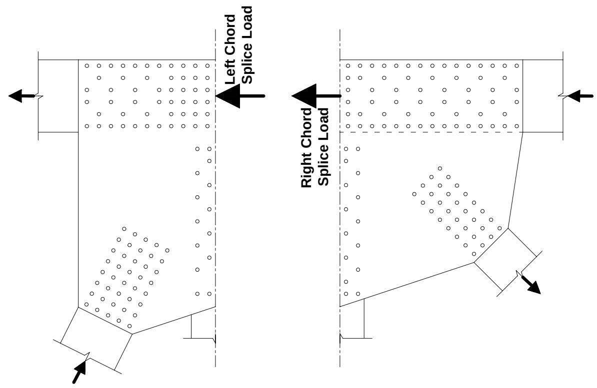


Figure C6.14.2.8.6-1—Example Connection Showing the Resolution of the Member Forces into Forces Acting on Each Side of a Chord Splice

The resistance equations in this Article assume the gusset and splice plates behave as one combined spliced section to resist the applied axial load and eccentric bending that occurs due to the fact that the resultant forces on the section are offset from the centroid of the combined section, as illustrated in Figure C6.14.2.8.6-2. The combined spliced section is treated as a beam and the factored resistance at the strength limit state is determined assuming the stress in the combined section at the limit of usable resistance is equal to the specified minimum yield strength of the gusset plate if the slenderness limit for the spliced section given by Eq. 6.14.2.8.6-2 is met, which will typically be the case. If not, the Engineer will need to derive a reduced value of F_{cr} to account for possible elastic buckling of the gusset plate within the splice.

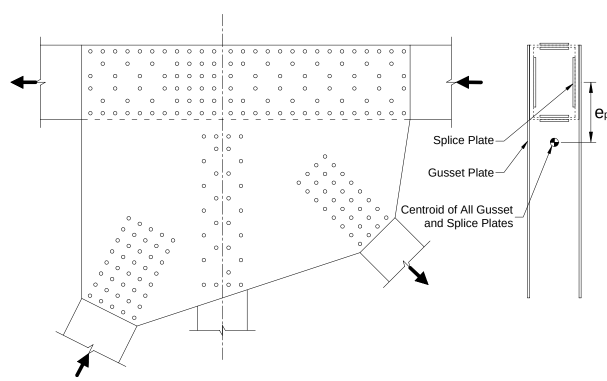


Figure C6.14.2.8.6-2—Illustration of the Combined Spliced Section at a Chord Splice

For tension chord splices, the factored tensile resistance, P_r, shall be taken as the lesser of the values given by Eqs. 6.14.2.8.6-3 and 6.14.2.8.6-4.

$$P_r = \phi_{cs} F_y \left(\frac{S_g A_g}{S_g + e_p A_g} \right) \quad (6.14.2.8.6\text{-}3)$$

$$P_r = \phi_{cs} F_u \left(\frac{S_n A_n}{S_n + e_p A_n} \right) \quad (6.14.2.8.6\text{-}4)$$

where:

ϕ_{cs} = resistance factor for truss gusset plate chord splices specified in Article 6.5.4.2
A_g = gross area of all plates in the cross-section intersecting the spliced plane (in.^{2})
A_n = net area of all plates in the cross-section intersecting the spliced plane (in.^{2})
e_p = distance between the centroid of the cross-section and the resultant force perpendicular to the spliced plane (in.)
F_y = specified minimum yield strength of the gusset plate (ksi)
F_u = specified minimum tensile strength of the gusset plate (ksi)
S_g = gross section modulus of all plates in the cross-section intersecting the spliced plane (in.^{3})
S_n = net section modulus of all plates in the cross-section intersecting the spliced plane (in.^{3})

Tension chord splice members shall also be checked for block shear rupture as specified in Article 6.13.4.

The Whitmore section check specified in Article 6.14.2.8.4 is not considered applicable for the design of a compression chord splice.

The yielding and net section fracture checks on the Whitmore section specified in Article 6.14.2.8.5 are not considered applicable for the design of a tension chord splice.

6.14.2.8.7—Edge Slenderness

If the length of the unsupported edge of a gusset plate exceeds $2.06t_g(E/F_y)^{1/2}$, where t_g is the gusset plate thickness and F_y is the specified minimum yield strength of the gusset plate, the edge should be stiffened.

C6.14.2.8.7

This Article is intended to provide good detailing practice to reduce deformations of free edges during fabrication, erection, and service versus providing an increase in the member compressive buckling resistance at the strength limit state. NCHRP Project 12-84 (NCHRP, 2013) found no direct correlation between the buckling resistance of the gusset plate and the free edge slenderness. There are no criteria specified for sizing of the edge stiffeners, but the traditional practice of using angles with leg thicknesses of 0.50 in. has generally provided adequate performance.

6.17—REFERENCES

Add the following references:

Hafner, A., O. T. Turan, and T. Schumacher. 2012. "Experimental Tests of Truss Bridge Gusset Plates Connections with Sway-Buckling Response," *Journal of Bridge Engineering*, American Society of Civil Engineers, Reston, VA (accepted for publication).

NCHRP (web only document No. 197). 2013. *Guidelines for the Load and Resistance Factor Design and Rating of Welded, Riveted and Bolted Gusset-Plate Connections for Steel Bridges*, Transportation Research Board, National Research Council, Washington D.C.

Yamamoto, et al. 1998. "Buckling Strengths of Gusseted Truss Joints," *Journal of Structural Engineering*, American Society of Civil Engineers, Reston, VA, Vol. 114.

2013 AASHTO BRIDGE COMMITTEE AGENDA ITEM: 26

SUBJECT: LRFD Bridge Construction Specifications: Section 11, Article 11.4.12.2.2 and LRFD Bridge Design Specifications: Section 6, Articles 6.7.7.1 and 6.7.7.2

TECHNICAL COMMITTEE: T-14 Steel / T-4 Construction

☒ **REVISION** ☒ **ADDITION** ☐ **NEW DOCUMENT**

☒ **DESIGN SPEC** ☒ **CONSTRUCTION SPEC** ☐ **MOVABLE SPEC**
☐ **MANUAL FOR BRIDGE EVALUATION** ☐ **SEISMIC GUIDE SPEC** ☐ **COASTAL GUIDE SPEC**
☐ **OTHER**

DATE PREPARED: 7/5/11
DATE REVISED: 4/5/13 **(Based on comments received, Heat Curving Limitations will be placed in the LRFD Bridge Construction Specifications)**

AGENDA ITEM:

Item #1

In the LRFD Bridge Construction Specifications renumber Articles 11.4.12.2.2 through 11.4.12.2.7 as 11.4.12.2.3 through 11.4.12.2.8.

Item #2

Insert new Article 11.4.12.2.2 Et Sequa as follows:

11.4.12.2.2—Geometric Limitations

11.4.12.2.2a—Cross-Sectional Criteria

Rolled beams and constant depth welded I-section plate girders satisfying all of the following criteria may be heat curved to obtain a horizontal curvature:

- R > 1,000 ft if (t_f > 3.0 in.) or (b > 30.0 in.), otherwise R > 150 ft
- $\Psi \leq 2.0$
- $\Psi_f \geq 0.20$
- $t_{nf} \leq t_f$

in which:

$$\psi = \frac{b_{nf}t_{nf} + bt_f + D_w t_w}{b_{nf}t_{nf} + bt_f} \qquad (11.4.12.2.2a\text{-}1)$$

$$\psi_f = \frac{b_{nf}t_{nf}}{bt_f} \leq 1.0 \qquad (11.4.12.2.2a\text{-}2)$$

where:

R = horizontal radius of curvature measured to the centerline of the girder web

b = width of wider flange (in.)
b_{nf} = width of narrower flange (in.)
D_w = clear distance between flanges (in.)
t_f = thickness of wider flange (in.)
t_{nf} = thickness of narrower flange (in.)
t_w = web thickness (in.)

Insert the following commentary to Article 11.4.12.2.2a:

C11.4.12.2.2a

The development of the minimum radius limits in Articles 11.4.12.2.2b and 11.4.12.2.2c was limited to cases with ψ less than or equal to 2, ψ_f greater than or equal to 0.2 and t_{nf} less than or equal to t_f (Sause et al 2013). The 150 ft limit is traditional.

For nonprismatic beams and girders the largest radius determined by applying the applicable equations in Articles 11.4.12.2.2b and 11.4.12.2.2c to each cross-section controls.

Guidance on shop implementation of continuous and V-heating to heat curve girders is given in U.S. Steel (2001) and (2002).

Insert new Article 11.4.12.2.2b as follows:

11.4.12.2.2b—Minimum Radius for Doubly-Symmetric Beams and Girders

For heat-curved doubly-symmetric beams and girders, the horizontal radius of curvature measured to the centerline of the girder web, in inches, shall not be less than the following:

- $If\ \frac{D_w}{t_w} > \frac{592}{\sqrt{F_{yw}}}, then$

$$R = 0.0365\frac{b}{\psi}\left(\frac{D_w}{t_w}\right)^2 \quad (11.4.12.2.2b\text{-}1)$$

- Otherwise

$$R = \frac{12{,}800b}{F_{yw}\psi} \quad (11.4.12.2.2b\text{-}2)$$

where:

F_{yw} = specified minimum yield strength of a web (ksi)

Insert the following commentary to Article 11.4.12.2.2b:

C11.4.12.2.2b

The stress analysis performed by Brockenbrough (1970) to develop the equations in earlier editions of AASHTO specifications was based on the following assumptions:

- The heat curving process introduces heat continuously along the girder length, resulting in a heated portion of the flange which is the same at every cross-section along the length.

- The girder cross-section is a doubly-symmetric I-shaped section.

- A tensile shrinkage force P develops in the heated portion of each flange near the flange edge on the inside of the curve. P is eccentric to the girder centroid.
- The heated width of the flange is 0.20b, i.e. 20 percent of the flange width.
- The entire cross-section of the girder resists P, resulting in transverse bending stresses associated with the horizontal curvature about a vertical axis and axial compressive stresses on the cross-section. The entire cross-section remains elastic and plane sections remain plane.

The resulting equation for the compressive stress in the web due to heat curving, normalized to the yield stress, was

$$\frac{\sigma_w}{F_{yw}} = \frac{-6,000}{F_{yw}} \frac{1}{\psi} \frac{1}{R/b} \qquad \text{(C11.4.12.2.2b-1)}$$

The equations in this Article are based on numerical simulations that overcame three limitations of the stress analysis by Brockenbrough (Sause et al 2013). These three limitations are:

- The stress in the cross-section was permitted to exceed the yield stress,
- The stress analysis was limited to doubly-symmetric cross-sections, and
- The heated width was fixed at 0.20b, rather than varying with R.

The studies which varied the heated flange width with R and considered the effect of yielding in the flanges, found that the compressive stress in the web due to heat curving could be adequately represented by the empirical equation of Eq. C11.4.12.2.2b-1 with the constant 6,000 replaced by 6,670 resulting in Eq. C11.4.12.2.2b-2 (Sause et al 2013).

$$\frac{\sigma_w}{F_{yw}} = \frac{-6,670}{F_{yw}} \frac{1}{\psi} \frac{1}{R/b} \qquad \text{(C11.4.12.2.2b-2)}$$

With the stress in the web from the heat curving operation now quantified, limits on R were established following the process developed by Brockenbrough. The basis for Eq. C11.4.12.2.2b-1 is limiting stress in web to the buckling stress instead of the post buckling strength previously used. Eq. C11.4.12.2.2b-2 is based on Von Mises yield criteria but revised assuming greater web shear stresses (0.425 F_y) under current design practice and an allowable stress of 0.90 F_y, while the original development assumed a web shear stress of 0.33 F_y and an allowable stress of F_y.

Insert new Article 11.4.12.2.2c as follows:

11.4.12.2.2c—Minimum Radius of Curvature for Singly-Symmetric Girders

For heat-curved singly-symmetric beams or girders, the horizontal radius of curvature measured to the centerline of the girder, in inches, shall not be less than the values calculated from Eqs. 11.4.12.2.2b-1 and 11.4.12.2.2b-2. Additionally, for singly-symmetric girders with ψ greater than or equal to 1.46 and with ψ_f less than ψ_{fo}, the radius shall not be less than that determined as:

$$R = \left[1.43\psi\left(1-\frac{\psi_f}{\psi_{fo}}\right)^2 + 1\right]\left(\frac{12800\ b}{F_{yw}\ \psi}\right) \qquad \text{(11.4.12.2.2c-1)}$$

in which:

$$\psi_{fo} = 0.68\psi - 0.79 \quad \text{(11.4.12.2.2c-2)}$$

Insert the following commentary to Article 11.4.12.2.2c:

C11.4.12.2.2c

Singly-symmetric cross-sections were included in the study described in Article C11.4.12.2.2b with the limitation that the moment of the heated area of the narrower flange about the elastic neutral axis is equal to the moment of the heated area of the wider flange about the elastic neutral axis. Parametric studies demonstrated that Eq. C11.4.12.2.2b-2 was adequate for the singly-symmetric case as well, provided that the flange width in the equation is taken as the width of the wider flange.

The stress analysis of singly-symmetric heat-curved girders that was used to develop Eq. C11.4.12.2.2b-1 and Eq. C11.4.12.2.2b-2 is valid when R is greater than the radius at which the heated width of the narrower flange equals the flange half width. This limit on R is provided by Eq. 11.4.12.2.2c-1 which is also considered to be a practical limit on the radius of singly-symmetric heat curved beams and girders. For many cases, this limit on heated flange width is reached when the web stresses are quite high, so it is not of practical concern, as the radius will be limited by the two equations in Article 11.4.12.2.2b. However, for highly unsymmetrical cases, this limit will be reached when the web stresses are not large, so that a limit on the radius of heat curved girders based on this limit on heated flange width is needed.

Insert new Article 11.4.12.2.2d as follows:

11.4.12.2.2d—Minimum Radius of Curvature for Hybrid Girders

Hybrid girders which meet the following criteria may be heat curved:

- $\eta_w \leq \eta_f$, and
- $\eta_w = \eta_f$ if $\eta_f < 1$
- $b_{yf} \geq b_{yfw}$ if $\eta_f < 1$

in which:

$$\eta_f = \frac{F_{yfw}}{F_{yf}} \quad \text{(11.4.12.2.2d-1)}$$

$$\eta_w = \frac{F_{yw}}{F_{yf}} \quad \text{(11.4.12.2.2d-2)}$$

where:

F_{yfw} = yield stress of flange with lower yield stress (ksi)
F_{yf} = yield stress of flange with higher yield stress (ksi)
F_{yw} = yield stress of web (ksi)
b_{yfw} = width of flange with lower yield stress (in.)
b_{yf} = width of flange with higher yield stress (in.)

For hybrid sections with $\eta_f = 1$ and $\eta_w < 1$, the horizontal radius of curvature measured to the centerline of the girder, in inches, shall not be less than the minimum radius determined from Articles 11.4.12.2.2b and 11.4.12.2.2c with F_{yf} substituted for F_{yw} in Eq. 11.4.12.2.2c-1.

For hybrid sections with $\eta_f < 1$, the horizontal radius of curvature measured to the centerline of the girder, in inches, shall not be less than the minimum radius determined from Article 11.4.12.2.2b. Additionally, for girders with ψ_{fo} greater than or equal to $0.2\sqrt{\eta_f}$ and with ψ_f less than $\frac{\psi_{fo}}{\sqrt{\eta_f}}$, the radius shall not be less than that determined as:

$$R = \left(1.43\left(1 - \frac{\psi_f\sqrt{\eta_f}}{\psi_{fo}}\right)^2 \cdot \psi + 1\right)\left(\frac{12800\ b}{F_{yw}\psi}\right) \qquad (11.4.12.2.2d\text{-}3)$$

Insert the following commentary to Article 11.4.12.2.2d:

C11.4.12.2.2d

Hybrid cross-sections were also included in the study described in Article C11.4.12.2.2b with the limitation that the moment of the heated area of the weaker flange about the elastic neutral axis is equal to the moment of the heated area of the stronger flange about the elastic neutral axis.

The results of the study showed that Eqs. 11.4.12.2.2b-1 and 11.4.12.2.2b-2 are also valid for hybrid sections, subject to the limitations outlined in this Article. The stress analysis of hybrid singly-symmetric heat-curved girders that was used to validate Eqs. 11.4.12.2.2b-1 and 11.4.12.2.2b-2 is valid when R is greater than the radius at which the heated width of the weaker flange equals the flange half width. This limit on R is provided by Eq. 11.4.12.2.2d-3.

Item #3

Add the following references to Article 11.10 and delete three references to "ASCE 1970".

- Brockenbrough, R. L. 1970. "Criteria for Heat Curving Steel Beams and Girders," *Journal of the Structural Division*. American Society of Civil Engineers, Vol. 96, October 1970.

- Brockenbrough, R. L. 1970a. "Experimental Stresses and Strains from Heat Curving," *Journal of the Structural Division*, Volume 96, No. ST7, American Society of Civil Engineers, New York.

- Brockenbrough, R. L. 1970b. "Theoretical Stresses and Strains from Heat Curving," *Journal of the Structural Division*, Volume 96, No. ST7, American Society of Civil Engineers, New York.

- Sause, R., H. Ma, and J. M. Kulicki. 2013. "Residual Stresses in Heat-Curved I-Girders and Associated Limits on Radius of Curvature," ATLSS Report No. 13-01, Center for Advanced Technology for Large Structural Systems, Lehigh University, Bethlehem, PA, April 2013.

Item #4

Revise Article 6.7.7.1 as follows:

This section pertains to rolled beams and constant depth welded I-section plate girders heat-curved to obtain a horizontal curvature. Structural steels conforming to AASHTO M 270M/M 270 (ASTM A709/A709M), Grades 36, 50, 50S, 50W, HPS 50W, HPS 70W or HPS 100W (Grades 250, 345, 345S, 345W, HPS 345W, HPS 485W or HPS 690W) may be heat-curved.

Item #5

In the LRFD Bridge Design Specifications delete Article 6.7.7.2 and replace with the following:

6.7.2.2—Geometric Limitations

The provisions of Article 11.4.12.2 of the LRFD Bridge Construction Specifications regarding cross-sectional limitations and radius limitations shall apply.

Item #6

Delete the following Notation references to Article 6.7.7.2: b, ψ, D, F_{yw}

OTHER AFFECTED ARTICLES:

None

BACKGROUND:

The limits on the radius of heat-curved steel bridge girders, Eq. 6.7.7.2-1 and Eq. 6.7.7.2-2 in the 2010 Edition AASHTO LRFD given as Eq. 1 and Eq. 2 below, are based on research by Brockenbrough (reference 1).

$$R_{min} = \frac{14\ b\ D_w}{\sqrt{F_{Yw}}\ \psi\ t_w} \qquad (1)$$

$$R_{min} = \frac{7{,}500\ b}{F_{Yw}\ \psi} \qquad (2)$$

where:

ψ = ratio of the total cross-sectional area to the total cross-sectional area of the two flanges
b = widest flange width (in.)
t_w = web thickness (in.)
D_w = clear distance between flanges (in.)
F_{yw} = specified minimum yield stress of web (ksi)
R_{min} = minimum radius of heat-curved girder (in.), where the larger result from Eq. 1 and Eq. 2 controls

The axial compressive stress in the web due to heat curving (reference 1) was given by Eq. 3.

$$\frac{\sigma_w}{F_{yw}} = \frac{-6{,}000}{F_{yw}} \frac{1}{\psi} \frac{1}{R/b} \qquad (3)$$

Development and use of Eq. 3 is referred to herein as “Method B” to denote the work by Brockenbrough.

Method B considers the magnitude of the stresses only in the web where stresses will be controlled by existing Eq. 6.7.7.2-1 and Eq. 6.7.7.2-2. Experimental research conducted by Brockenbrough (reference. 2) showed that the final stresses in the heated portion of the flanges were relatively constant and close to the yield stress. Method B does not give this result.

An improved analysis of residual stresses in heat curved steel girders (called Method A to denote the alternative method used in the supporting calculations) was developed (Sause et al 2013), in which it is assumed that the final stress in the heated portion of the flanges equals the yield stress in tension. The remainder of the section, called the elastic section herein (see Fig. 1), develops stresses which are in equilibrium with the tensile stress in the heated portion of the flanges. The width of the heated portion of the flanges is treated as a variable, which increases as R decreases. Specifically, the assumptions are:

- The heat curving process introduces heat continuously along the girder length, resulting in a heated portion of the flange which is the same at every cross-section along the length.

- The girder cross-section is a doubly-symmetric I-shaped section with a single yield stress (i.e., is not a hybrid cross-section).
- The stress in the heated portion of the flanges equals the yield stress in tension.
- The remainder of the cross-section is elastic.

Method A does not provide a simple relationship between web stress σ_W and the curvature radius R, in a form similar to Eq. 1. However data which relate σ_W to R, were generated and plotted as the width of the heated portion of the flange is varied. This is as shown in Fig. 2 for F_Y = 50 ksi and various values of ψ. Similarly, data for the stresses in the flange away from the heated portion of the flange (at point O on the flange edge on the outside of the curve, Fig. 1) can be generated and plotted various values of ψ.

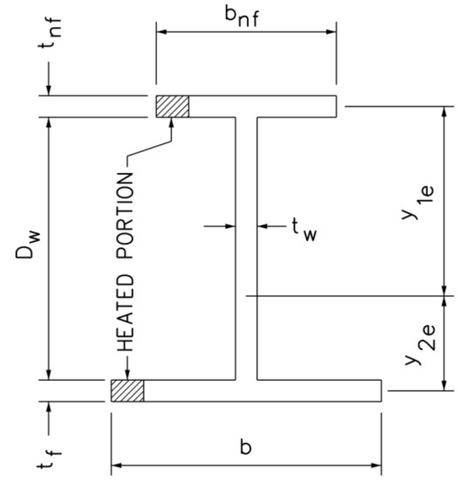


Fig. 1 Elastic section

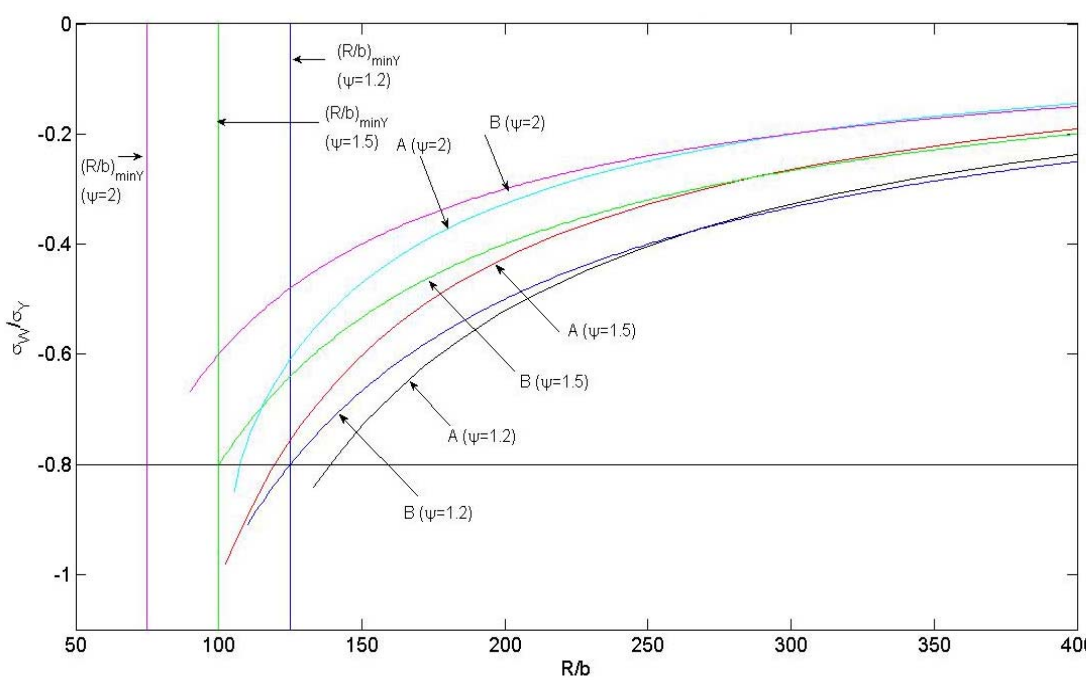


Fig. 2 Web stress (F_Y=50 ksi)

A comparison of the results from Method A and Method B indicated the following:

- Method A estimates larger web stress than Method B.
- Method A specifies the stress in the heated width of the flange, while Method B does not control this stress. Method B gives stresses in the heated width of the flange that are well above yield stress.

From Method A, it was observed that tensile flange stresses at point O on the flange edge on the outside of the curve exceed the yield stress before the minimum value of R is reached. Similar results (not shown) indicate that compressive yield stresses in the flange in the elastic region adjacent to the heated portion may also exceed the yield stress before the minimum value of R is reached.

To account for this, a nonlinear version of Method A was developed in which yielding in the flanges was considered. As the flanges yield, the section properties of the elastic section are recalculated. There are two flange yielding conditions: (1) the outside flange edge (point O) yields first; and (2) the flange adjacent to the heated portion (point I) yields first. Both conditions are considered in the analysis.

A comparison of results for wide range of cross-section shapes and yield strengths led to empirical Eq. 4 which gives the web axial compressive stress induced by heat-curving, with stresses in the flanges limited to the yield stress. Eq. 4 has the same form as Eq. 1 derived by Brockenbrough (reference 1). The value of 0.23E, with E=29000 ksi, is 6670ksi, which is about 11% larger than the constant (6000 ksi) in Eq. 3.

$$\frac{\sigma_W}{F_{yw}} = -0.23\frac{E}{F_{yw}}\frac{1}{\psi}\frac{1}{R/b} = -\frac{6670}{F_{yw}}\frac{1}{\psi}\frac{1}{R/b} \qquad (4)$$

The similarity of these results is a little surprising since Brockenbrough's analysis used a constant fraction of the flange width as the heated portion, did not consider yielding on the cross-section, and was correlated to a limited data set from experiments on beams with only one yield stress (36 ksi, nominal). These results, however, show that Brockenbrough's analysis produced relatively reasonable results.

The work was extended to include the singly-symmetric section. The shrinkage forces in the heated portions of the flanges should not generate a bending moment about the primary axis of bending (i.e., about the horizontal elastic neutral axis) of the elastic part of the section (after removing the heated portions of the flanges from the section). Therefore, the heated portion of the flanges should have the following relationship:

$$b_{th} t_{nf} y_{1e} = b_h t_f y_{2e}$$

where as shown in Figure 1:

$b_{th}t_{nf}$ = heated area of narrower flange (e.g., the top flange) (in^2)
$b_h t_f$ = heated area of wider flange (e.g., the bottom flange) (in^2)
b_{th} = heated width of narrower flange (in.)
b_h = heated width of wider flange (in.)
t_{nf} = thickness of narrower flange (in.)
t_f = thickness of wider flange (in.)
y_{1e} = distance from centerline of narrower flange to horizontal ENA for elastic section (in.)
y_{2e} = distance from centerline of wider flange to horizontal ENA for elastic section (in.)

Three parameters are needed to describe the singly-symmetric section: φ, ν, and ψ. φ is the ratio of narrower flange width to wider flange width; ν is the ratio of the thickness of the narrower flange to the thickness of the wider flange; ψ is ratio of total girder area to total flange area.

$$\varphi = \frac{b_{nf}}{b}$$

$$\nu = \frac{t_{nf}}{t_f}$$

$$\psi = \frac{b_{nf}t_{nf} + bt_f + D_w t_w}{b_{nf}t_{nf} + bt_f}$$

where as shown in Figure 1:

D_w = clear distance between the flanges (in.)

t_w = web thickness (in.)

b_{nf} = width of narrower flange (in.)

b = width of wider flange

Analysis of a wide range of cross-section shapes and yield strengths showed that the following empirical equation accurately represents the web axial compressive stress induced by heat-curving for singly-symmetric sections.

$$\frac{\sigma_w}{F_{yw}} = -0.23\frac{1}{\lambda_2} = -0.23\frac{E}{F_{yw}}\frac{1}{\psi}\frac{1}{R/b}\frac{1}{\varphi^{0.043-0.133\nu}} \quad (5)$$

Figs. 3 and 4 compare Eq. 5 with Eq. 4 for several singly-symmetric cases. Figs. 3 and 4 show analysis data for singly-symmetric and doubly symmetric cases, with ψ=1.2 and ψ=2, respectively. Together these figures and others developed as part of the numerical study, show that Eq. 4 alone provides sufficient accuracy for estimating the web stress from heat curving for singly-symmetric sections, as long as the width of the widest flange is used for the parameter b. For the cases that have been studied, the term $\varphi^{0.043-0.133\nu}$ is close enough to 1, so that the effect of singly-symmetry on the heat-induced stress in the web can be neglected.

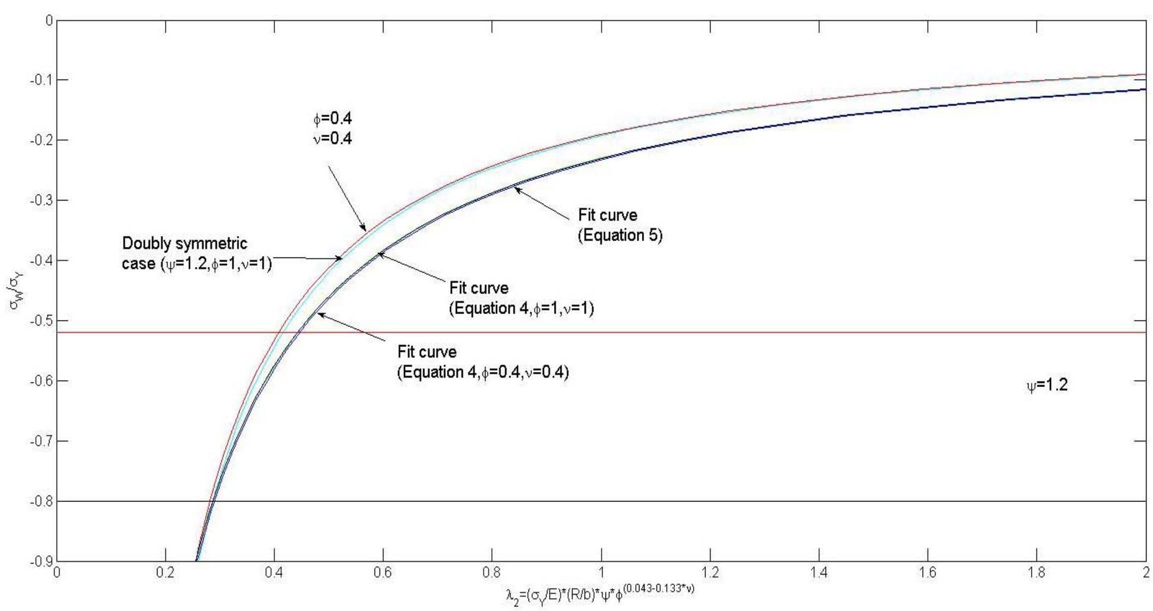


Fig. 3 Singly-symmetric curve fit for ψ=1.2

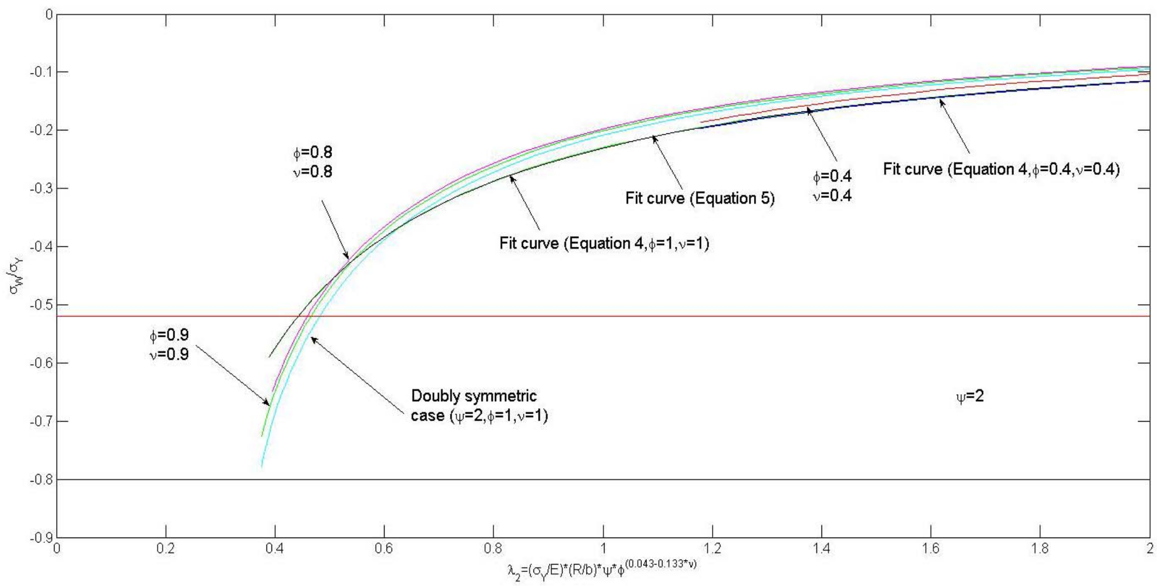


Fig. 4 Singly-symmetric curve fit for ψ=2.0

The original stability criteria given by current Eq. 6.7.7.2-1 was derived by Brockenbrough by setting the web axial stress induced by heat curving from Eq. 3 equal to the web post-buckling stress taken as:

$$f_u = \sqrt{f_c F_{yw}} \qquad (6)$$

in which:

$$f_c = \frac{k_c \pi^2 E}{12\left(1-\mu^2\right)\left(\dfrac{D_w}{t_w}\right)^2} \qquad (7)$$

where:

f_u = post-buckling buckling stress (ksi)
f_c = elastic buckling stress (ksi)
k_c = buckling coefficient taken as 6.97 (for fixed edge conditions)
μ = Poisson's ratio

Since Eq. 6 provides a post-buckling stress, the current provisions permit the theoretical elastic web buckling stress to be exceeded by the heat curving operation. It was understood that the actual web behavior will not be bifurcation as assumed for elastic buckling theory, but will be a growth of the initial out-of-flatness of the web present before heat curving. However, several other provisions in AASHTO LRFD limit calculated web stresses to below a theoretical elastic buckling stress. Taking a similar approach where the web stress from heat curving is limited to the theoretical elastic buckling stress given by Eq. 7 rather than the post-buckling stress given by Eq. 6 leads the proposed stability criteria.

The proposed equation is derived by substituting $\sigma_w = f_c$ from Eq. 7 into Eq. 4 using k_c = 6.97 and μ = 0.30, neglecting the minus sign, and solving for R which gives the minimum R that causes elastic buckling of the web under stress from heat curving:

$$R_{\min} = 0.23\frac{12\left(1-\mu^2\right)}{k_c\pi^2}\frac{b}{\psi}\left(\frac{D_w}{t_w}\right)^2 = 0.0365\frac{b}{\psi}\left(\frac{D_w}{t_w}\right)^2 \qquad (8)$$

The table below compares the results from Eq. 1 (the current Eq. 6.7.7.2-1) and Eq. 8 (the proposed Eq. 11.4.12.2.2b-1) for girders with F_{yw} = 50 ksi, and shows that the proposed Eq. 8 limits the heat curving radius to 1.3 to 3 times the current R_{min}.

$\frac{D_w}{t_w}$	$\frac{R_{\min}}{\left(\frac{b}{\psi}\right)}$ from Eq. 1	$\frac{R_{\min}}{\left(\frac{b}{\psi}\right)}$ from Eq. 8
70	139	179
80	158	234
90	178	296
100	198	365
110	218	442
120	238	526
130	257	617
140	277	715
150	297	821
160	317	934

Current Eq. 6.7.7.2-2 in AASHTO LRFD was derived from Eq. 1 by Brockenbrough as follows. The web axial stress induced by heat curving in combination with an estimated shear stress under service load was limited to yield. The Von Mises yield criterion was used to establish the yield condition under these combined stresses.

$$\left(F_{yw}\right)^2 = \left(\sigma_c\right)^2 + 3\left(\sigma_s\right)^2 \qquad (9)$$

where:

σ_c = web axial stress due to heat curving
σ_s = estimated web shear stress under service load

Brockenbrough assumed $\sigma_s = 0.33F_{yw}$ and solved Eq. 9 for $\sigma_c = 0.80F_{yw}$. Eq. 2 (the current Eq. 6.7.7.2-2) was obtained by substituting $\sigma_w = \sigma_c = 0.80F_{yw}$ into Eq. 3, neglecting the minus sign, and solving for R. The result is the minimum R that causes web yielding under the combination of axial stress from heat curving with the estimated shear stress under service load of $0.33\ F_{yw}$.

An improved version of the current Eq. 6.7.7.2-2 is proposed, considering the web stress results given by Eq. 4 and an updated estimate of the web stress under service load. For design according to AASHTO LRFD, the expected maximum shear stress under the Service II load combination is estimated from the expected maximum shear stress under the Strength I load combination, using a representative load factor for Strength I = 1.5 and a representative load factor for Service II = 1.1.

$$\sigma_{s-Service\,II} = \left(\frac{1.1}{1.5}\right)\sigma_{s-Strength\,I} \qquad (10)$$

If $\sigma_{s\text{-Strength I}}$ is assumed to be equal to $0.58\ F_{yw}$, then $\sigma_{s\text{-Service II}} = 0.425\ F_{yw}$. Assuming that $\sigma_s = \sigma_{s\text{-Service II}} = 0.425\ F_{yw}$ and limiting the maximum Service II stress to $0.90\ F_{yw}$, an average of that currently used for composite and noncomposite girders, and then solving Eq. 9 for σ_c results in $\sigma_c = 0.52\ F_{yw}$.

Substituting $\sigma_w = \sigma_c = 0.52\ F_{yw}$ into Eq. 4, neglecting the minus sign and solving for R gives the minimum R that causes web yielding under the combination of axial stress from heat curving with the estimated shear stress under service load:

$$R_{\min} = \frac{0.23E}{0.52F_{yw}}\frac{b}{\psi} = \frac{12800}{F_{yw}}\frac{b}{\psi} \qquad (11)$$

Comparing Eq. 11 with Eq. 2 shows that the revised Eq. 11 limits the heat curving radius to 1.7 times the current R_{min} from Eq. 2.

The limits on the radius of heat-curved steel I-girders (Eq. 8 and 11) are derived from limits on the residual stress in the web. For certain highly unsymmetrical singly-symmetric sections, the heated width of the narrower flange reaches half the flange width $b_{nf}/2$ before the radius R becomes small. Therefore, $b_{th} = b_{nf}/2$ was a limit on the validity of the analyses used in this study. This limit is also considered to be a practical limit on heat curving of I-girders. Data points were generated for a large number of singly-symmetric cross-sections showing the value of R $b_{th} = b_{nf}/2$. This radius R was plotted versus $\psi_f = \varphi\nu$, which is the ratio of the area of the narrower flange to the area of the wider flange.

A new nonlinear equation was developed to fit this data, to provide a limit on R (i.e.,R_{min}) corresponding to $b_{th} = b_{nf}/2$. The data points for a given value of ψ that are less than R_{min} from Eq. 11 are inconsequential, because R_{min} will be controlled by Eq. 8 or Eq. 11. So only the data points for R corresponding to $b_{th} = b_{tf}/2$ which exceed R_{min} from Eq. 11 were considered in developing the nonlinear equation. Only data with $\psi_f \geq 0.2$ were considered. The intersection between the data points and Eq. 11 was identified for each value of ψ that was studied. A linear equation of the form $\psi_{fo} = f(\psi)$ was fit to these intersection points, and the resulting equation was $\psi_{fo} = 0.68\psi - 0.79$. Since the intersection points represent the intersection of the new nonlinear equation for R_{min} with Eq. 11, the nonlinear equation for R_{min} includes the constant term $\frac{0.23\,E\,b}{0.52\,\sigma_Y\psi}$ which is reached when $\psi_f = \psi_{fo}$. The intersection of the nonlinear equation for R_{min} with Eq. 11 was achieved by including the term $1-\frac{f}{fo}$ in the nonlinear term. A conservative curve fit to all the data points considered was obtained with the following equation for R_{min} is:

$$R_{\min} = \left(0.63\left(1-\frac{f}{fo}\right)^2 + \frac{0.23}{0.52\psi}\right)\left(\frac{E}{\sigma_Y}\,b\right) = (1.43\,\psi\left(1-\frac{\psi_f}{\psi_{f0}}\right)^2 + 1)\left(\frac{12800\,b}{\sigma_Y\,\psi}\right) \qquad (12)$$

Fig. 5 compares Eq. 11 with the data points generated for various values of ψ and ψ_f.

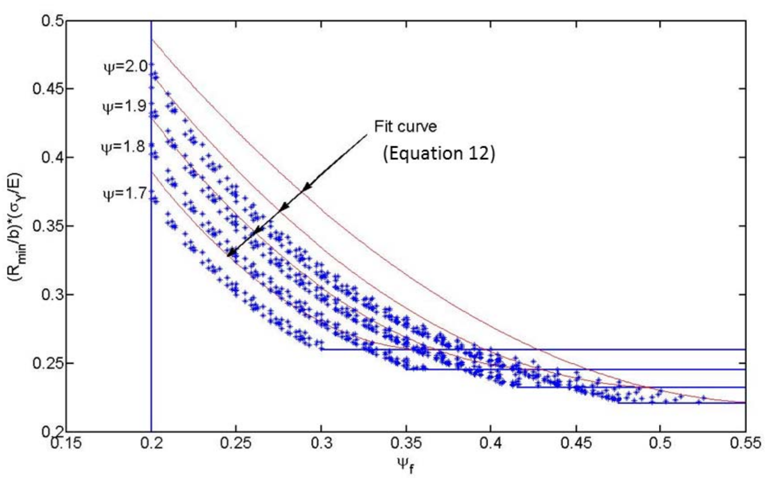


Fig. 4 Data for radius when heated flange width equals half flange width plotted versus ψ_f with Eq.12 and line representing Eq. 11

ANTICIPATED EFFECT ON BRIDGE:

This will restrict the ability to heat curve some girders that would have been allowed under the previous criteria.

REFERENCES:

Brockenbrough, R. L. 1970. "Criteria for Heat Curving Steel Beams and Girders," *Journal of the Structural Division*. American Society of Civil Engineers, Vol. 96, October 1970.

Brockenbrough, R. L. 1970. "Theoretical Stresses and Strains from Heat Curving," *Journal of the Structural Division*. American Society of Civil Engineers, Vol. 96, July 1970.

Sause, R., H. Ma, and J. M. Kulicki. 2013. "Residual Stresses in Heat-Curved I-Girders and Associated Limits on Radius of Curvature," ATLSS Report No. 13-01, Center for Advanced Technology for Large Structural Systems, Lehigh University, Bethlehem, PA, April 2013.

OTHER:

None

2013 AASHTO BRIDGE COMMITTEE AGENDA ITEM: 27

SUBJECT: LRFD Bridge Design Specifications: Section 10, Various Articles

TECHNICAL COMMITTEE: T-15 Substructures and Retaining Walls

☒ **REVISION**	☐ **ADDITION**	☐ **NEW DOCUMENT**
☒ **DESIGN SPEC**	☐ **CONSTRUCTION SPEC**	☐ **MOVABLE SPEC**
☐ **MANUAL FOR BRIDGE EVALUATION**	☐ **SEISMIC GUIDE SPEC**	☐ **BRIDGE ELEMENT INSP GUIDE**
	☐ **OTHER**	

DATE PREPARED: 1/14/13
DATE REVISED: 4/17/13

AGENDA ITEM:

In **Section 10 - Foundations**, make the revisions as shown in Attachment A.

OTHER AFFECTED ARTICLES:

None

BACKGROUND:

The impetus for this agenda item was the availability of the recently completed Federal Highway Administration Geotechnical Engineering Circular 10 – Drilled Shafts: Construction Procedures and LRFD Design Methods (Brown et al., 2010). The FHWA recently completed a comprehensive update to the recommended technical guidance for design and construction of drilled shaft foundations. This update is the first to FHWA recommended technical guidance since the manual developed by O'Neill and Reese (1999), which is heavily referenced in the current Section 10.

Advances in construction equipment and procedures, and updates to design methodologies based on knowledge advances and performance data have been well documented in the literature. These advances include routine design and construction of large diameter foundation elements (e.g., $\geq$ 5 feet), analysis of more comprehensive load test databases, and incorporation of recent published research to address gaps in the specification. This agenda item will serve to better represent the state of practice for drilled shaft design.

The specific changes to Section 10 are shown in the attachment to this agenda item as underlined and stricken through text. The key changes are summarized as follows:

1. Table 10.4.2-1 has been adjusted to correct a problem with potentially excessive exploration depths for drilled shaft groups. In addition, language has been added to address potential design and construction risk due to subsurface condition variability and construction claims, especially with large diameter shafts socketed into bedrock.
2. The rock mass classification system used to assess rock mass strength, currently the Rock Mass Rating (RMR) system, is being replaced with the Geological Strength Index (GSI) system in Article 10.4.6.4. This will apply for Section 10 except as noted in Articles 10.6.2.4.4, 10.6.2.6.2and 10.6.3.2. As stated by Marinos and Hoek (2000), the GSI index is based upon an assessment of the lithology, structure and condition of discontinuity surfaces in the rock mass and it is estimated from visual examination of the rock

mass exposed in surface excavations such as roadcuts, in tunnel faces and in borehole core. The GSI, by the combination of the two fundamental parameters of geological process, the blockiness of the mass and the condition of the discontinuities, respects the main geological constraints that govern a formation and is thus both a geologically friendly index and practical to assess. The GSI provides a more direct correlation to the Hoek-Brown strength parameters than the RMR. Furthermore, the new drilled shaft specifications use the GSI to classify the rock mass to assess rock mass strength for design purposes.

3. The design procedures for spread footings in rock have been developed using the RMR system. Although GSI is specified for rock characterization in Section 10, RMR is still to be used for spread footing design in rock. Reference to Sabatini et al (2002) is provided for RMR.
4. A key issue is the design of shafts in intermediate geo-materials (IGM). Section 10 is rewritten to consider design methods only for cohesive IGM's. The design method for tip resistance in cohesionless IGM's has been deleted to remove confusion amongst designers when evaluating dense sands versus transitional materials. The term cohesionless IGM was used previously by O'Neill and Reese (1999) to describe granular tills or granular residual soils with $N1_{60}$ greater than 50 blows/ft. The use of this term is discontinued and all cohesionless geomaterials are included in a single category of cohesionless soils. This addresses designer confusion with results when trying to interpret whether very dense cohesionless soils (e.g. N=50) were to be considered cohesionless or IGM. Furthermore, the cohesionless IGM equation included in the current specifications tended to produce results that were more conservative than the equation for sand, likely due to the limited regionally applicable data used as its basis. The cohesionless IGM equation was not applicable for wider use nationally. Therefore, it was removed from the specifications in this agenda item.
5. Articles 10.8.1.6.2 and 10.8.2.3 have been significantly expanded to address downdrag for shafts with tip bearing in soil versus bedrock. The Articles have been rewritten to consider that downdrag occurs in response to relative movement of a drilled shaft and may not exist if shaft response to axial load exceeds vertical deformation of the soil. The response of a drilled shaft to downdrag in combination with the other forces acting at the head of the shaft is complex and a realistic evaluation of actual limit states that may occur requires careful consideration of two issues: (1) drilled shaft load-settlement behavior, and (2) the time period over which downdrag occurs relative to the time period over which non-permanent components of load occur. When these factors are taken into account, it is appropriate to consider different downdrag forces for evaluation of geotechnical strength limit states than for structural strength limit states. These issues are further addressed in Brown et al. (2010).
6. In the past, there has not been guidance on how to handle horizontal movement of shafts in rock. Furthermore, for fractured rock masses, the best method for calculating the rock shear strength may not be the same as in Article 10.4. For shafts socketed into rock, Article 10.8.2.3 has been significantly expanded to provide guidance on design for horizontal movement for shafts in rock, including a reference for guidance on developing hyperbolic P-y curves for fractured rock masses using the GSI (Liang et al., (2009).
7. Article 10.8.3.5.1b has been adjusted based on knowledge advances from field load test data. In accordance with O'Neil and Reese (1999), it has traditionally been recommended to neglect side resistance over a distance of one diameter above the base of drilled shafts where this portion derives its resistance from a cohesive soil. The recommendation is based on numerical modeling that predicts a zone of tension at the soil-shaft interface in the zone immediately above the base. This is not supported by field load test data and the agenda item has adjusted the text and Figure 10.8.3.5.1b-1 to show that this side resistance is not to be neglected over the bottom one diameter.
8. The method for determination of side resistance of shafts in cohesionless soils (the β-Method) has been updated. The method in Article 10.8.3.5.2b is based on axial load tests on drilled shafts as presented by Chen and Kulhawy (2002) and updated by Kulhawy and Chen (2007). The method provides a rational approach for relating unit side resistance to N-values and to the state of effective stress acting at the soil-shaft interface. This approach replaces the previously used depth-dependent β-method developed by O'Neill and Reese (1999), which does not account for variations in N-value or effective stress on the calculated value of β. Further discussion, including the detailed development of Eq. 10.8.3.5.2b-2, is provided in Brown et al. (2010).
9. The method for determination of side resistance of shafts socketed into bedrock has been updated based on analysis of load test data reported by Kulhawy et al., (2005). The adjustment of the method incorporates data from previous studies by Horvath and Kenney (1979), Rowe and Armitage (1987), Kulhawy and Phoon (1993), and others. The updated equation in Article 10.8.3.5.4b provides a more realistic estimate

of unit side shear that is based on a comprehensive load test database.

10. A key issue in drilled shaft design has been the design decision to neglect one or the other component of resistance (side or base) in a rock socket. C10.8.3.5.4d has been updated to better assist designers with this. The commentary focuses on the use of quality construction practices, as opposed to omitting tip resistance in shafts. It is noted that in many cases, the cost of quality control and assurance is offset by the economies achieved in socket design by including tip resistance. Reasons cited for neglecting side resistance of rock sockets include (1) the possibility of strain-softening behavior of the sidewall interface (2) the possibility of degradation of material at the borehole wall in argillaceous rocks, and (3) uncertainty regarding the roughness of the sidewall. Brittle behavior along the sidewall, in which side resistance exhibits a significant decrease beyond its peak value, is not commonly observed in load tests on rock sockets. If there is reason to believe strain softening will occur, laboratory direct shear tests of the rock-concrete interface can be used to evaluate the load-deformation behavior and account for it in design. These cases would also be strong candidates for conducting field load tests. Investigating the sidewall shear behavior through laboratory or field testing is generally more cost-effective than neglecting side resistance in the design. Application of quality control and assurance through inspection is also necessary to confirm that sidewall conditions in production shafts are of the same quality as laboratory or field test conditions. Materials that are prone to degradation at the exposed surface of the borehole and are prone to a "smooth" sidewall generally are argillaceous sedimentary rocks such as shale, claystone, and siltstone. Degradation occurs due to expansion, opening of cracks and fissures combined with groundwater seepage, and by exposure to air and/or water used for drilling. Hassan and O'Neill (1997) note that this behavior is most prevalent in cohesive IGM's and that in the most severe cases degradation results in a smear zone at the interface. Smearing may reduce load transfer significantly. As reported by Abu-Hejleh et al. (2003), both smearing and smooth sidewall conditions can be prevented in cohesive IGM's by using roughening tools during the final pass with the rock auger or by grooving tools. Careful inspection prior to concrete placement is required to confirm roughness of the sidewalls. Only when these measures cannot be confirmed would there be cause for neglecting side resistance in design.

ANTICIPATED EFFECT ON BRIDGES:

These revisions will affect the design of drilled shaft foundations for support of transportation structures through update of design methodologies to incorporate recently available information and knowledge advances construction equipment and procedures.

REFERENCES:

See Attachment A

OTHER:

None

ATTACHMENT A — 2013 AGENDA ITEM 27 - T-15

10.1—SCOPE – *NO CHANGES – NOT SHOWN*

10.2—DEFINITIONS

ONE ADDITION BELOW – THE REMAINDER STAYS THE SAME

GSI—Geologic Strength Index

10.3—NOTATION

ONE ADDITION BELOW – THE REMAINDER STAYS THE SAME

s, m, a = fractured rock mass parameters (10.4.6.4)

10.4—SOIL AND ROCK PROPERTIES

10.4.1—Informational Needs – *NO CHANGES – NOT SHOWN*

10.4.2—Subsurface Exploration

Subsurface explorations shall be performed to provide the information needed for the design and construction of foundations. The extent of exploration shall be based on variability in the subsurface conditions, structure type, and any project requirements that may affect the foundation design or construction. The exploration program should be extensive enough to reveal the nature and types of soil deposits and/or rock formations encountered, the engineering properties of the soils and/or rocks, the potential for liquefaction, and the groundwater conditions. The exploration program should be sufficient to identify and delineate problematic subsurface conditions such as karstic formations, mined out areas, swelling/collapsing soils, existing fill or waste areas, etc.

Borings should be sufficient in number and depth to establish a reliable longitudinal and transverse substrata profile at areas of concern such as at structure foundation locations and adjacent earthwork locations, and to investigate any adjacent geologic hazards that could affect the structure performance.

C10.4.2

The performance of a subsurface exploration program is part of the process of obtaining information relevant for the design and construction of substructure elements. The elements of the process that should precede the actual exploration program include a search and review of published and unpublished information at and near the site, a visual site inspection, and design of the subsurface exploration program. Refer to Mayne et al. (2001) and Sabatini et al. (2002) for guidance regarding the planning and conduct of subsurface exploration programs.

The suggested minimum number and depth of borings are provided in Table 10.4.2-1. While engineering judgment will need to be applied by a licensed and experienced geotechnical professional to adapt the exploration program to the foundation types and depths needed and to the variability in the subsurface conditions observed, the intent of Table 10.4.2-1 regarding the minimum level of exploration needed should be carried out. The depth of borings indicated in Table 10.4.2-1 performed before or during design should take into account the potential for changes in the type, size and depth of the planned foundation elements.

As a minimum, the subsurface exploration and testing program shall obtain information adequate to analyze foundation stability and settlement with respect to:

- Geological formation(s) present,
- Location and thickness of soil and rock units,
- Engineering properties of soil and rock units, such as unit weight, shear strength and compressibility,
- Groundwater conditions,
- Ground surface topography, and
- Local considerations, e.g., liquefiable, expansive or dispersive soil deposits, underground voids from solution weathering or mining activity, or slope instability potential.

Table 10.4.2-1 shall be used as a starting point for determining the locations of borings. The final exploration program should be adjusted based on the variability of the anticipated subsurface conditions as well as the variability observed during the exploration program. If conditions are determined to be variable, the exploration program should be increased relative to the requirements in Table 10.4.2-1 such that the objective of establishing a reliable longitudinal and transverse substrata profile is achieved. If conditions are observed to be homogeneous or otherwise are likely to have minimal impact on the foundation performance, and previous local geotechnical and construction experience has indicated that subsurface conditions are homogeneous or otherwise are likely to have minimal impact on the foundation performance, a reduced exploration program relative to what is specified in Table 10.4.2-1 may be considered.

If requested by the Owner or as required by law, boring and penetration test holes shall be plugged.

Laboratory and/or in-situ tests shall be performed to determine the strength, deformation, and permeability characteristics of soils and/or rocks and their suitability for the foundation proposed.

This Table should be used only as a first step in estimating the number of borings for a particular design, as actual boring spacings will depend upon the project type and geologic environment. In areas underlain by heterogeneous soil deposits and/or rock formations, it will probably be necessary to drill more frequently and/or deeper than the minimum guidelines in Table 10.4.2-1 to capture variations in soil and/or rock type and to assess consistency across the site area. For situations where large diameter rock socketed shafts will be used or where drilled shafts are being installed in formations known to have large boulders, or voids such as in karstic or mined areas, it may be necessary to advance a boring at the location of each shaft. Even the best and most detailed subsurface exploration programs may not identify every important subsurface problem condition if conditions are highly variable. The goal of the subsurface exploration program, however, is to reduce the risk of such problems to an acceptable minimum.

In a laterally homogeneous area, drilling or advancing a large number of borings may be redundant, since each sample tested would exhibit similar engineering properties. Furthermore, in areas where soil or rock conditions are known to be very favorable to the construction and performance of the foundation type likely to be used, e.g., footings on very dense soil, and groundwater is deep enough to not be a factor, obtaining fewer borings than provided in Table 10.4.2-1 may be justified. In all cases, it is necessary to understand how the design and construction of the geotechnical feature will be affected by the soil and/or rock mass conditions in order to optimize the exploration.

Borings may need to be plugged due to requirements by regulatory agencies having jurisdiction and/or to prevent water contamination and/or surface hazards.

Parameters derived from field tests, e.g., driven pile resistance based on cone penetrometer testing, may also be used directly in design calculations based on empirical relationships. These are sometimes found to be more reliable than analytical calculations, especially in familiar ground conditions for which the empirical relationships are well established.

Table 10.4.2-1—Minimum Number of Exploration Points and Depth of Exploration (modified after Sabatini et al., 2002)

Application	Minimum Number of Exploration Points and Location of Exploration Points	Minimum Depth of Exploration
Retaining Walls	A minimum of one exploration point for each retaining wall. For retaining walls more than 100 ft in length, exploration points spaced every 100 to 200 ft with locations alternating from in front of the wall to behind the wall. For anchored walls, additional exploration points in the anchorage zone spaced at 100 to 200 ft. For soil-nailed walls, additional exploration points at a distance of 1.0 to 1.5 times the height of the wall behind the wall spaced at 100 to 200 ft.	Investigate to a depth below bottom of wall at least to a depth where stress increase due to estimated foundation load is less than ten percent of the existing effective overburden stress at that depth and between one and two times the wall height. Exploration depth should be great enough to fully penetrate soft highly compressible soils, e.g., peat, organic silt, or soft fine grained soils, into competent material of suitable bearing capacity, e.g., stiff to hard cohesive soil, compact dense cohesionless soil, or bedrock.
Shallow Foundations	For substructure, e.g., piers or abutments, widths less than or equal to 100 ft, a minimum of one exploration point per substructure. For substructure widths greater than 100 ft, a minimum of two exploration points per substructure. Additional exploration points should be provided if erratic subsurface conditions are encountered. To reduce design and construction risk due to subsurface condition variability and the potential for construction claims, at least one exploration per shaft should be considered for large diameter shafts (e.g., greater than 5 ft in diameter), especially when shafts are socketed into bedrock.	Depth of exploration should be: • great enough to fully penetrate unsuitable foundation soils, e.g., peat, organic silt, or soft fine grained soils, into competent material of suitable bearing resistance, e.g., stiff to hard cohesive soil, or compact to dense cohesionless soil or bedrock; • at least to a depth where stress increase due to estimated foundation load is less than ten percent of the existing effective overburden stress at that depth; and • if bedrock is encountered before the depth required by the second criterion above is achieved, exploration depth should be great enough to penetrate a minimum of 10 ft into the bedrock, but rock exploration should be sufficient to characterize compressibility of infill material of near-horizontal to horizontal discontinuities. Note that for highly variable bedrock conditions, or in areas where very large boulders are likely, more than 10 ft or rock core may be required to verify that adequate quality bedrock is present.
Deep Foundations	For substructure, e.g., bridge piers or abutments, widths less than or equal to 100 ft, a minimum of one exploration point per substructure. For substructure widths greater than 100 ft, a minimum of two exploration points per substructure. Additional exploration points should be provided if erratic subsurface conditions are encountered, especially for the case of shafts socketed into bedrock. To reduce design and construction risk due to subsurface condition variability and the potential for construction claims, at least one exploration per shaft should be considered for large diameter shafts (e.g., greater than 5 ft in diameter), especially when shafts are socketed into bedrock.	In soil, depth of exploration should extend below the anticipated pile or shaft tip elevation a minimum of 20 ft, or a minimum of two times the ~~maximum~~ minimum pile group dimension, whichever is deeper. All borings should extend through unsuitable strata such as unconsolidated fill, peat, highly organic materials, soft fine-grained soils, and loose coarse-grained soils to reach hard or dense materials. For piles bearing on rock, a minimum of 10 ft of rock core shall be obtained at each exploration point location to verify that the boring has not terminated on a boulder. For shafts supported on or extending into rock, a minimum of 10 ft of rock core, or a length of rock core equal to at least three times the shaft diameter for isolated shafts or two times the ~~maximum~~ minimum shaft group dimension, whichever is greater, shall be extended below the anticipated shaft tip elevation to determine the physical characteristics of rock within the zone of foundation influence. Note that for highly variable bedrock conditions, or in areas where very large boulders are likely, more than 10 ft or rock core may be required to verify that adequate quality bedrock is present.

10.4.3—Laboratory Tests – ***NO CHANGES – NOT SHOWN***

10.4.4—In-Situ Tests – ***NO CHANGES – NOT SHOWN***

10.4.5—Geophysical Tests – ***NO CHANGES- NOT SHOWN***

10.4.6—Selection of Design Properties

10.4.6.1—General – ***NO CHANGES – NOT SHOWN***

10.4.6.2—Soil Strength – ***NO CHANGES – NOT SHOWN***

10.4.6.3—Soil Deformation – ***NO CHANGES – NOT SHOWN***

10.4.6.4—Rock Mass Strength

The strength of intact rock material should be determined using the results of unconfined compression tests on intact rock cores~~, splitting tensile tests on intact rock cores, or point load strength tests on intact specimens of rock~~.

~~The rock should be classified using the rock mass rating system (RMR) as described in Table 10.4.6.4-1. For each of the five parameters in the Table, the relative rating based on the ranges of values provided should be evaluated. The rock mass rating (RMR) should be determined as the sum of all five relative ratings. The RMR should be adjusted in accordance with the criteria in Table 10.4.6.4-2. The rock classification should be determined in accordance with Table 10.4.6.4-3.~~ Except as noted for design of spread footings in rock, for a rock mass that contains a sufficient number of "randomly" oriented discontinuities such that it behaves as an isotropic mass, and thus its behavior is largely independent of the direction of the applied loads, the strength of the rock mass should first be classified using its geological strength index (GSI) as described in Figures 10.4.6.4-1 and 10.4.6.4-2 and then assessed using the Hoek-Brown failure criterion.

C10.4.6.4

Point load strength index tests may be used to assess intact rock compressive strength in lieu of a full suite of unconfined compression tests on intact rock cores provided that the point load test results are calibrated to unconfined compression strength tests. Point load strength index tests rely on empirical correlations to intact rock compressive strength. The correlation provided in the ASTM point load test procedure (ASTM D 5731) is empirically based and may not be valid for the specific rock type under consideration. Therefore, a site specific correlation with uniaxial compressive strength test results is recommended. Point load strength index tests should not be used for weak to very weak rocks (< 2200 psi /15 MPa).

~~Because of the importance of the discontinuities in rock, and the fact that most rock is much more discontinuous than soil~~Because the engineering behavior of rock is strongly influenced by the presence and characteristics of discontinuities, emphasis is placed on visual assessment of the rock and the rock mass. The application of a rock mass classification system essentially assumes that the rock mass contains a sufficient number of "randomly" oriented discontinuities such that it behaves as an isotropic mass, and thus its behavior is largely independent of the direction of the applied loads. It is generally not appropriate to use such classification systems for rock masses with well defined, dominant structural fabrics or where the orientation of discrete, persistent discontinuities controls behavior to loading.

The GSI was introduced by Hoek et al. (1995) and Hoek and Brown (1997), and updated by Hoek et al. (1998) to classify jointed rock masses. Marinos et al. (2005) provide a comprehensive summary of the applications and limitations of the GSI for jointed rock masses (Figure 10.4.6.4-1) and for heterogeneous rock masses that have been tectonically disturbed (Figure 10.4.6.4-2). Hoek et al. (2005) further distinguish heterogeneous sedimentary rocks that are not tectonically disturbed and provide several diagrams for determining GSI values for various rock mass conditions. In combination with rock type and uniaxial compressive strength of intact rock (q_u), GSI provides a practical means to assess rock mass strength and rock mass modulus for foundation design using the Hoek-Brown failure criterion (Hoek et al. 2002).

The design procedures for spread footings in rock provided in Article 10.6.3.2 have been developed using the rock mass rating (RMR) system. For design of foundations in rock in Articles 10.6.2.4 and 10.6.3.2, classification of the rock mass should be according to the RMR system. For additional information on the RMR system, see Sabatini et al. (2002).

Other methods for assessing rock mass strength, including in-situ tests or other visual systems that have proven to yield accurate results may be used in lieu of the specified method.

Table 10.4.6.4-1 Geomechanics Classification of Rock Masses

	Parameter		Ranges of Values						
1	Strength of intact rock material	Point load strength index	>175 ksf	85–175 ksf	45–85 ksf	20–45 ksf	For this low range, uniaxial compressive test is preferred		
		Uniaxial compressive strength	>4320 ksf	2160–4320 ksf	1080–2160 ksf	520–1080 ksf	215–520 ksf	70–215 ksf	20–70 ksf
	Relative Rating		15	12	7	4	2	1	0
2	Drill core quality RQD		90% to 100%	75% to 90%	50% to 75%	25% to 50%	<25%		
	Relative Rating		20	17	13	8	3		
3	Spacing of joints		>10 ft	3–10 ft	1–3 ft	2 in.–1 ft	<2 in.		
	Relative Rating		30	25	20	10	5		
4	Condition of joints		• Very rough surfaces • Not continuous • No separation • Hard joint wall rock	• Slightly rough surfaces • Separation <0.05 in. • Hard joint wall rock	• Slightly rough surfaces • Separation <0.05 in. • Soft joint wall rock	• Slicken-sided surfaces or • Gouge <0.2 in. thick or • Joints open 0.05–0.2 in. • Continuous joints	• Soft gouge >0.2 in. thick or • Joints open >0.2 in. • Continuous joints		
	Relative Rating		25	20	12	6	0		
5	Groundwater conditions (use one of the three evaluation criteria as appropriate to the method of exploration)	Inflow per 30 ft tunnel length	None	<400 gal./hr.	400–2000 gal./hr.	>2000 gal./hr.			
		Ratio = joint water pressure/ major principal stress	0	0.0–0.2	0.2–0.5	>0.5			
		General Conditions	Completely Dry	Moist only (interstitial water)	Water under moderate pressure	Severe water problems			
	Relative Rating		10	7	4	0			

Table 10.4.6.4-2 Geomechanics Rating Adjustment for Joint Orientations

Strike and Dip Orientations of Joints		Very Favorable	Favorable	Fair	Unfavorable	Very Unfavorable
Ratings	Tunnels	0	–2	–5	–10	–12
	Foundations	0	–2	–7	–15	–25
	Slopes	0	–5	–25	–50	–60

~~**Table 10.4.6.4-3—Geomechanics Rock Mass Classes Determined from Total Ratings**~~

~~RMR Rating~~	~~100–81~~	~~80–61~~	~~60–41~~	~~40–21~~	~~<20~~
~~Class No.~~	~~I~~	~~II~~	~~III~~	~~IV~~	~~V~~
~~Description~~	~~Very good rock~~	~~Good rock~~	~~Fair rock~~	~~Poor rock~~	~~Very poor rock~~

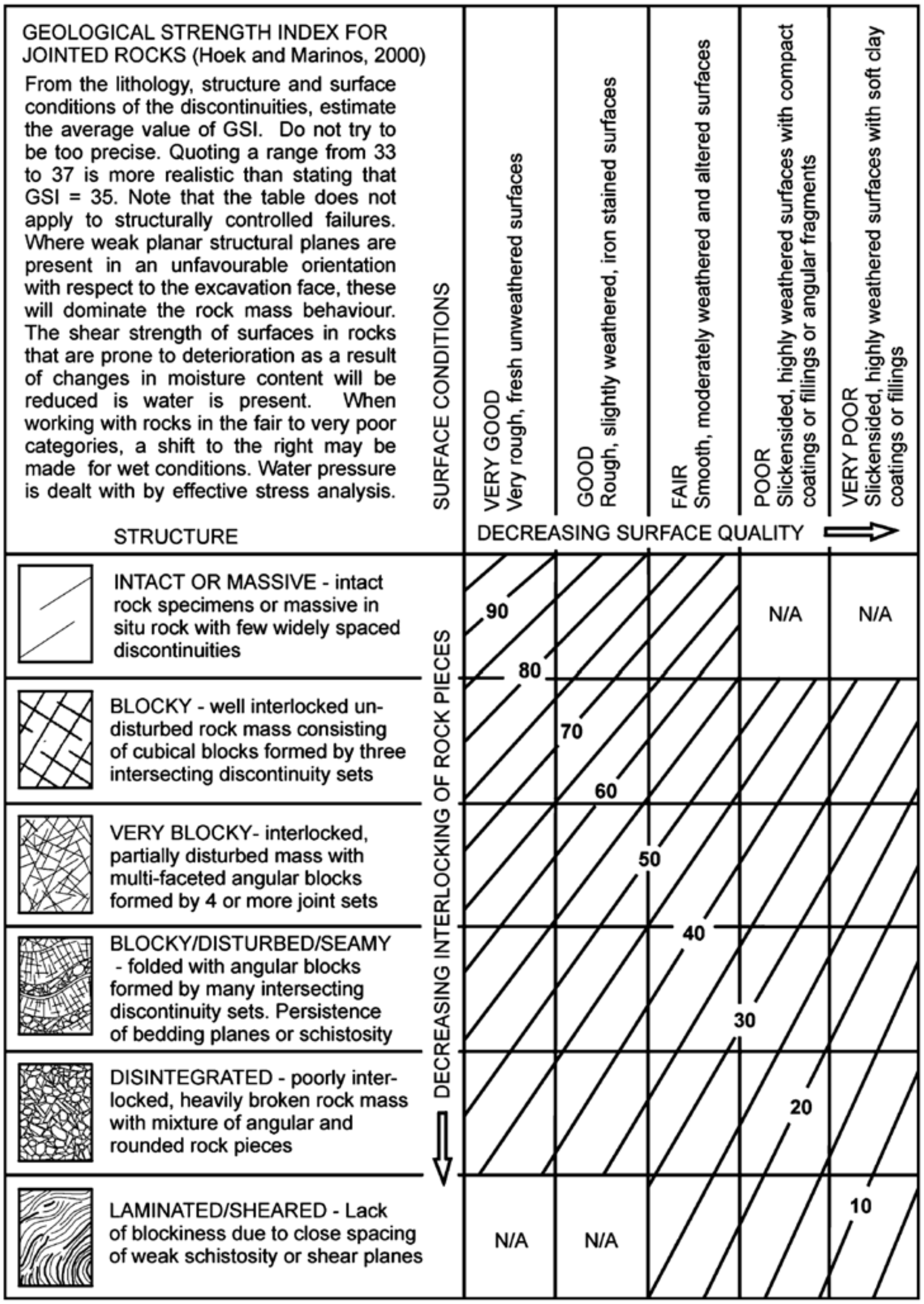


Figure 10.4.6.4-1—Determination of GSI for Jointed Rock Mass (Hoek and Marinos, 2000)

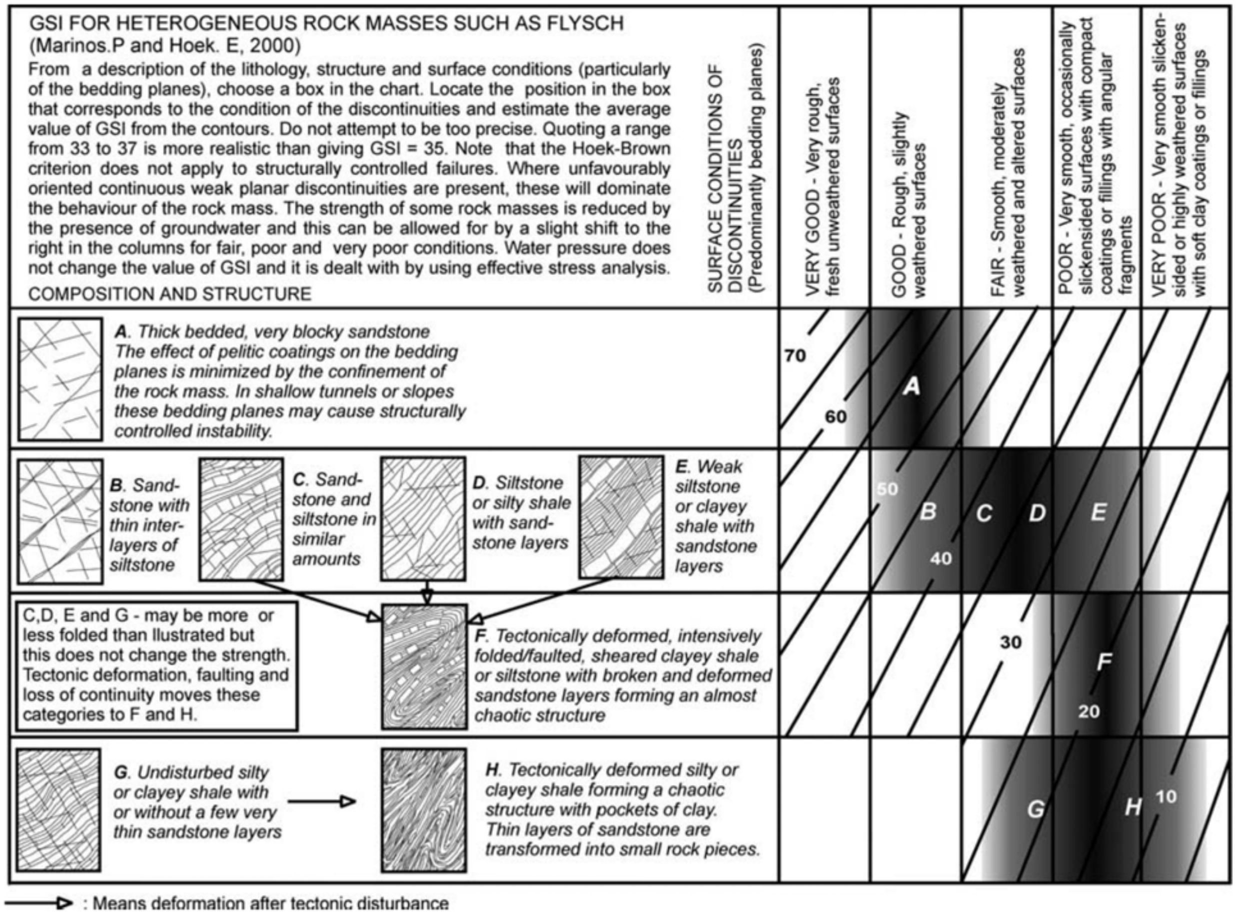


Figure 10.4.6.4-2—Determination of GSI for Tectonically Deformed Heterogeneous Rock Masses (Marinos and Hoek 2000)

The ~~shear~~ strength of ~~fractured~~jointed rock masses should be evaluated using the ~~Hoek and Brown~~ Hoek-Brown failure criterion (Hoek et al., 2002). This nonlinear strength criterion is expressed in its general form as: ~~criteria in which the shear strength is represented as a curved envelope that is a function of the uniaxial compressive strength of the intact rock, q_u, and two dimensionless constants m and s. The values of m and s as defined in Table 10.4.6.4-4 should be used.~~

~~The shear strength of the rock mass should be determined as:~~

$$\tau = \left(\cot \phi_i' - \cos \phi_i'\right) m \frac{q_u}{8} \qquad (10.4.6.4\text{-}1)$$

~~in which:~~

$$\phi_i' = \tan^{-1}\left\{4h \cos^2\left[30 + 0.33 \sin^{-1}\left(h^{\frac{-3}{2}}\right)\right] - 1\right\}^{\frac{-1}{2}}$$

$$h = 1 + \frac{16\left(m\sigma_n' + sq_u\right)}{(3m^2 q_u)}$$

~~where:~~

~~This method was developed by Hoek (1983) and Hoek and Brown (1988, 1997). Note that the instantaneous cohesion at a discrete value of normal stress can be taken as:~~

$$c_i = \tau - \sigma_n' \tan \phi_i' \qquad (C10.4.6.4\text{-}1)$$

~~The instantaneous cohesion and instantaneous friction angle define a conventional linear Mohr envelope at the normal stress under consideration. For normal stresses significantly different than that used to compute the instantaneous values, the resulting shear strength will be unconservative. If there is considerable variation in the effective normal stress in the zone of concern, consideration should be given to subdividing the zone into areas where the normal stress is relative constant and assigning separate strength parameters to each zone. Alternatively, the methods of Hoek (1983) may be used to compute average values for the range of normal stresses expected.~~

~~τ = the shear strength of the rock mass (ksf)~~

~~ϕ'_i = the instantaneous friction angle of the rock mass (degrees)~~

~~q_u = average unconfined compressive strength of rock core (ksf)~~

~~σ'_n = effective normal stress (ksf)~~

~~m, s = constants from Table 10.4.6.4-4 (dim)~~

$$\sigma'_1 = \sigma'_3 + q_u\left(m_b \frac{\sigma'_3}{q_u} + s\right)^a \qquad (10.4.6.4\text{-}1)$$

in which:

$$s = e^{\left(\frac{GSI-100}{9-3D}\right)} \qquad (10.4.6.4\text{-}2)$$

$$a = \frac{1}{2} + \frac{1}{6}\left(e^{\frac{-GSI}{15}} - e^{\frac{-20}{3}}\right) \qquad (10.4.6.4\text{-}3)$$

where:

e = 2.718 (natural or Naperian log base)

D = disturbance factor (dim)

σ'_1 and σ'_3 = principal effective stresses (ksf)

q_u = average unconfined compressive strength of rock core (ksf)

m_b, s, and a = empirically determined parameters

The value of the constant m_i should be estimated from Table 10.4.6.4-1, based on lithology. Relationships between GSI and the parameters m_b, s, and a, according to Hoek et al. (2002) are as follows:

$$m_b = m_i e^{\left(\frac{GSI-100}{28-14D}\right)} \qquad (10.4.6.4\text{-}4)$$

~~Table 10.4.6.4-4—Approximate Relationship between Rock-Mass Quality and Material Constants Used in Defining Nonlinear Strength (Hoek and Brown, 1988)~~

~~Rock Quality~~	~~Constants~~	~~Rock Type~~ ~~A = Carbonate rocks with well developed crystal cleavage—*dolomite, limestone and marble*~~ ~~B = Lithified argrillaceous rocks—*mudstone, siltstone, shale and slate (normal to cleavage)*~~ ~~C = Arenaceous rocks with strong crystals and poorly developed crystal cleavage—*sandstone and quartzite*~~ ~~D = Fine grained polyminerallic igneous crystalline rocks—*andesite, dolerite, diabase and rhyolite*~~ ~~E = Coarse grained polyminerallic igneous & metamorphic crystalline rocks—*amphibolite, gabbro gneiss, granite, norite, quartz-diorite*~~				
		~~A~~	~~B~~	~~C~~	~~D~~	~~E~~
~~INTACT ROCK SAMPLES~~ ~~Laboratory size specimens free from discontinuities.~~ ~~CSIR rating: *RMR* = 100~~	~~*m*~~ ~~*s*~~	~~7.00~~ ~~1.00~~	~~10.00~~ ~~1.00~~	~~15.00~~ ~~1.00~~	~~17.00~~ ~~1.00~~	~~25.00~~ ~~1.00~~
~~VERY GOOD QUALITY ROCK MASS~~ ~~Tightly interlocking undisturbed rock with unweathered joints at 3–10 ft~~ ~~CSIR rating: *RMR* = 85~~	~~*m*~~ ~~*s*~~	~~2.40~~ ~~0.082~~	~~3.43~~ ~~0.082~~	~~5.14~~ ~~0.082~~	~~5.82~~ ~~0.082~~	~~8.567~~ ~~0.082~~
~~GOOD QUALITY ROCK MASS~~ ~~Fresh to slightly weathered rock, slightly disturbed with joints at 3–10 ft~~ ~~CSIR rating: *RMR* = 65~~	~~*m*~~ ~~*s*~~	~~0.575~~ ~~0.00293~~	~~0.821~~ ~~0.00293~~	~~1.231~~ ~~0.00293~~	~~1.395~~ ~~0.00293~~	~~2.052~~ ~~0.00293~~
~~FAIR QUALITY ROCK MASS~~ ~~Several sets of moderately weathered joints spaced at 1–3 ft~~ ~~CSIR rating: *RMR* = 44~~	~~*m*~~ ~~*s*~~	~~0.128~~ ~~0.00009~~	~~0.183~~ ~~0.00009~~	~~0.275~~ ~~0.00009~~	~~0.311~~ ~~0.00009~~	~~0.458~~ ~~0.00009~~
~~POOR QUALITY ROCK MASS~~ ~~Numerous weathered joints at 2 to 12 in.; some gouge. Clean compacted waste rock.~~ ~~CSIR rating: *RMR* = 23~~	~~*m*~~ ~~*s*~~	~~0.029~~ $\cancel{3 \times 10^{-6}}$	~~0.041~~ $\cancel{3 \times 10^{-6}}$	~~0.061~~ $\cancel{3 \times 10^{-6}}$	~~0.069~~ $\cancel{3 \times 10^{-6}}$	~~0.102~~ $\cancel{3 \times 10^{-6}}$
~~VERY POOR QUALITY ROCK MASS~~ ~~Numerous heavily weathered joints spaced <2 in. with gouge. Waste rock with fines.~~ ~~CSIR rating: *RMR* = 3~~	~~*m*~~ ~~*s*~~	~~0.007~~ $\cancel{1 \times 10^{-7}}$	~~0.010~~ $\cancel{1 \times 10^{-7}}$	~~0.015~~ $\cancel{1 \times 10^{-7}}$	~~0.017~~ $\cancel{1 \times 10^{-7}}$	~~0.025~~ $\cancel{1 \times 10^{-7}}$

Table 10.4.6.4-1—Values of the Constant m_i by Rock Group (after Marinos and Hoek 2000; with updated values from Rocscience, Inc., 2007)

Rock type	Class	Group	Texture: Coarse	Texture: Medium	Texture: Fine	Texture: Very fine
SEDIMENTARY	Clastic		Conglomerate (21 ± 3) Breccia (19 ± 5)	Sandstone 17 ± 4	Siltstone 7 ± 2 Greywacke (18 ± 3)	Claystone 4 ± 2 Shale (6 ± 2) Marl (7 ± 2)
SEDIMENTARY	Non-Clastic	Carbonates	Crystalline Limestone (12 ± 3)	Sparitic Limestone (10 ± 5)	Micritic Limestone (8 ± 3)	Dolomite (9 ± 3)
SEDIMENTARY	Non-Clastic	Evaporites		Gypsum 10 ± 2	Anhydrite 12 ± 2	
SEDIMENTARY	Non-Clastic	Organic				Chalk 7 ± 2
METAMORPHIC	Non Foliated		Marble 9 ± 3	Hornfels (19 ± 4)) Metasandstone (19 ± 3)	Quartzite 20 ± 3	
METAMORPHIC	Slightly foliated		Migmatite (29 ± 3)	Amphibolite 26 ± 6	Gneiss 28 ± 5	
METAMORPHIC	Foliated*			Schist (10 ± 3)	Phyllite (7 ± 3)	Slate 7 ± 4
IGNEOUS	Plutonic	Light	Granite 32 ± 3 Granodiorite (29 ± 3)	Diorite 25 ± 5		
IGNEOUS	Plutonic	Dark	Gabbro 27 ± 3 Norite 20 ± 5	Dolerite (16 ± 5)		
IGNEOUS	Hypabyssal		Porphyries (20 ± 5)		Diabase (15 ± 5)	Peridotite (25 ± 5)
IGNEOUS	Volcanic	Lava		Rhyolite (25 ± 5) Andesite 25 ± 5	Dacite (25 ± 3)) Basalt (25 ± 5)	
IGNEOUS	Volcanic	Pyroclastic	Agglomerate (19 ± 3)	Volcanic breccia (19 ± 5)	Tuff (13 ± 5)	

Disturbance to the foundation excavation caused by the rock removal methodology should be considered through the disturbance factor D in Eqs. 10.4.6.4-2 through 10.4.6.4-4.

Where it is necessary to evaluate the strength of a

The disturbance factor, D, ranges from 0 (undisturbed) to 1 (highly disturbed), and is an adjustment for the rock mass disturbance induced by the excavation method. Suggested values for various tunnel and slope excavations can be found in Hoek et al. (2002). However, these values may not directly applicable to foundations. If using blasting techniques to remove the rock in a shaft foundation, due to its confined state, a disturbance factor approaching 1.0 should be considered, as the blast energy will tend to radiate laterally into the intact rock, potentially disturbing the rock. If using rock coring techniques, much less disturbance is likely and a disturbance factor approaching 0 may be considered. If using a down hole hammer to break up the rock, the disturbance factor is likely between these two extremes.

The range of typical friction angles provided in single discontinuity or set of discontinuities, the strength along the discontinuity should be determined as follows:

- For smooth discontinuities, the shear strength is represented by a friction angle of the parent rock material. To evaluate the friction angle of this type of discontinuity surface for design, direct shear tests on samples should be performed. Samples should be formed in the laboratory by cutting samples of intact core or, if possible, on actual discontinuities using an oriented shear box.
- For rough discontinuities the nonlinear criterion of Barton (1976) should be applied or, if possible, direct shear tests should be performed on actual discontinuities using an oriented shear box.

Table C10.4.6.4-1 may be used in evaluating measured values of friction angles for smooth joints.

Table C10.4.6.4-1—Typical Ranges of Friction Angles for Smooth Joints in a Variety of Rock Types (modified after Barton, 1976; Jaeger and Cook, 1976)

Rock Class	Friction Angle Range	Typical Rock Types
Low Friction	20–27°	Schists (high mica content), shale, marl
Medium Friction	27–34°	Sandstone, siltstone, chalk, gneiss, slate
High Friction	34–40°	Basalt, granite, limestone, conglomerate

Note: Values assume no infilling and little relative movement between joint faces.

When a major discontinuity with a significant thickness of infilling is to be investigated, the shear strength will be governed by the strength of the infilling material and the past and expected future displacement of the discontinuity. Refer to Sabatini et al. (2002) for detailed procedures to evaluate infilled discontinuities.

10.4.6.5—Rock Mass Deformation

The elastic modulus of a rock mass (E_m) shall be taken as the lesser of the intact modulus of a sample of rock core ~~(E_i)~~ (E_R) or the modulus determined from ~~one of the following equations:~~ Table 10.4.6.5-1.

$$\cancel{E_m = 145\left(10^{\frac{RMR-10}{40}}\right) \qquad (10.4.6.5\text{-}1)}$$

~~where:~~

~~E_m = Elastic modulus of the rock mass (ksi)~~

~~E_m ≤ E_i~~

C10.4.6.5

~~Table 10.4.6.5-1 was developed by O'Neill and Reese (1999) based on a reanalysis of the data presented by Carter and Kulhawy (1988) for the purposes of estimating side resistance of shafts in rock.~~ Methods for establishing design values of E_m include:

- Empirical correlations that relate E_m to strength or modulus values of intact rock (q_u or E_R) and GSI
- Estimates based on previous experience in similar rocks or back-calculated from load tests
- In-situ testing such as pressuremeter test

Empirical correlations that predict rock mass

~~E_i = Elastic modulus of intact rock (ksi)~~

~~RMR = Rock mass rating specified in Article 10.4.6.4.~~

~~or~~

$$\cancel{E_m = \left(\frac{E_m}{E_i}\right) E_i} \qquad \text{~~(10.4.6.5-2)~~}$$

~~where:~~

~~E_m = Elastic modulus of the rock mass (ksi)~~

~~E_m/E_i = Reduction factor determined from Table 10.4.6.5-1 (dim)~~

~~E_i = Elastic modulus of intact rock from tests (ksi)~~

For critical or large structures, determination of rock mass modulus (E_m) using in-situ tests ~~may be warranted~~ should be considered. Refer to Sabatini et al. (2002) for descriptions of suitable in-situ tests.

modulus (E_m) from GSI and properties of intact rock, either uniaxial compressive strength (q_u) or intact modulus (E_R), are presented in Table 10.4.6.5-1. The recommended approach is to measure uniaxial compressive strength and modulus of intact rock in laboratory tests on specimens prepared from rock core. Values of GSI should be determined for representative zones of rock for the particular foundation design being considered. The correlation equations in Table 10.4.6.5-1 should then be used to evaluate modulus and its variation with depth. If pressuremeter tests are conducted, it is recommended that measured modulus values be calibrated to the values calculated using the relationships in Table 10.4.6.5-1.

Preliminary estimates of the elastic modulus of intact rock may be made from Table C10.4.6.5-1. Note that some of the rock types identified in the Table are not present in the U.S.

It is extremely important to use the elastic modulus of the rock mass for computation of displacements of rock materials under applied loads. Use of the intact modulus will result in unrealistic and unconservative estimates.

~~Table 10.4.6.5-1 Estimation of E_m Based on RQD (after O'Neill and Reese, 1999)~~

~~*RQD* (percent)~~	~~E_m/E_i~~	
	~~Closed Joints~~	~~Open Joints~~
~~100~~	~~1.00~~	~~0.60~~
~~70~~	~~0.70~~	~~0.10~~
~~50~~	~~0.15~~	~~0.10~~
~~20~~	~~0.05~~	~~0.05~~

Table 10.4.6.5-1—Estimation of E_m Based on GSI

Expression	Notes/Remarks	Reference
$E_m(GPa) = \sqrt{\frac{q_u}{100}}\,10^{\frac{GSI-10}{40}}$ for $q_u \le 100$ MPa $E_m(GPa) = 10^{\frac{GSI-10}{40}}$ for $q_u > 100$ MPa	Accounts for rocks with $q_u < 100$ MPa; *note* q_u in MPa	Hoek and Brown (1997); Hoek et al. (2002)
$E_m = \frac{E_R}{100} e^{GSI/21.7}$	Reduction factor on intact modulus, based on GSI	Yang (2006)
Notes: E_R = modulus of intact rock, E_m = equivalent rock mass modulus, GSI = geological strength index, q_u = uniaxial compressive strength. 1 MPa = 20.9 ksf.		

Table C10.4.6.5-1—Summary of Elastic Moduli for Intact Rock (modified after Kulhawy, 1978)

Rock Type	No. of Values	No. of Rock Types	Elastic Modulus, ~~(E_i)~~ (E_R) (ksi $\times 10^3$)			Standard Deviation (ksi $\times 10^3$)
			Maximum	Minimum	Mean	
Granite	26	26	14.5	0.93	7.64	3.55
Diorite	3	3	16.2	2.48	7.45	6.19
Gabbro	3	3	12.2	9.8	11.0	0.97
Diabase	7	7	15.1	10.0	12.8	1.78
Basalt	12	12	12.2	4.20	8.14	2.60
Quartzite	7	7	12.8	5.29	9.59	2.32
Marble	14	13	10.7	0.58	6.18	2.49
Gneiss	13	13	11.9	4.13	8.86	2.31
Slate	11	2	3.79	0.35	1.39	0.96
Schist	13	12	10.0	0.86	4.97	3.18
Phyllite	3	3	2.51	1.25	1.71	0.57
Sandstone	27	19	5.68	0.09	2.13	1.19
Siltstone	5	5	4.76	0.38	2.39	1.65
Shale	30	14	5.60	0.001	1.42	1.45
Limestone	30	30	13.0	0.65	5.7	3.73
Dolostone	17	16	11.4	0.83	4.22	3.44

Poisson's ratio for rock should be determined from tests on intact rock core.

Where tests on rock core are not practical, Poisson's ratio may be estimated from Table C10.4.6.5-2.

Table C10.4.6.5-2—Summary of Poisson's Ratio for Intact Rock (modified after Kulhawy, 1978)

Rock Type	No. of Values	No. of Rock Types	Poisson's Ratio, ν			Standard Deviation
			Maximum	Minimum	Mean	
Granite	22	22	0.39	0.09	0.20	0.08
Gabbro	3	3	0.20	0.16	0.18	0.02
Diabase	6	6	0.38	0.20	0.29	0.06
Basalt	11	11	0.32	0.16	0.23	0.05
Quartzite	6	6	0.22	0.08	0.14	0.05
Marble	5	5	0.40	0.17	0.28	0.08
Gneiss	11	11	0.40	0.09	0.22	0.09
Schist	12	11	0.31	0.02	0.12	0.08
Sandstone	12	9	0.46	0.08	0.20	0.11
Siltstone	3	3	0.23	0.09	0.18	0.06
Shale	3	3	0.18	0.03	0.09	0.06
Limestone	19	19	0.33	0.12	0.23	0.06
Dolostone	5	5	0.35	0.14	0.29	0.08

10.4.6.6—Erodibility of Rock - *NO CHANGES – NOT SHOWN*

10.5—LIMIT STATES AND RESISTANCE FACTORS

10.5.1—General – *NO CHANGES – NOT SHOWN*

10.5.2—Service Limit States – *NO CHANGES – NOT SHOWN*

10.5.3—Strength Limit States – *NO CHANGES – NOT SHOWN*

10.5.4—Extreme Events Limit States – *NO CHANGES – NOT SHOWN*

10.5.5—Resistance Factors

10.5.5.1—Service Limit States – *NO CHANGES – NOT SHOWN*

10.5.5.2—Strength Limit States

10.5.5.2.1—General - ***NO CHANGES – NOT SHOWN***

10.5.5.2.2—Spread Footings - ***NO CHANGES – NOT SHOWN***

10.5.5.2.3—Driven Piles - ***NO CHANGES – NOT SHOWN***

10.5.5.2.4—Drilled Shafts

Resistance factors shall be selected based on the method used for determining the nominal shaft resistance. When selecting a resistance factor for shafts in clays or other easily disturbed formations, local experience with the geologic formations and with typical shaft construction practices shall be considered.

C10.5.5.2.4

The resistance factors in Table 10.5.5.2.4-1 were developed using either statistical analysis of shaft load tests combined with reliability theory (Paikowsky et al., 2004), fitting to allowable stress design (ASD), or both. Where the two approaches resulted in a significantly different resistance factor, engineering judgment was

Where the resistance factors provided in Table 10.5.5.2.4-1 are to be applied to a single shaft supporting a bridge pier, the resistance factor values in the Table should be reduced by 20 percent. Where the resistance factor is decreased in this manner, the η_R factor provided in Article 1.3.4 shall not be increased to address the lack of foundation redundancy.

The number of static load tests to be conducted to justify the resistance factors provided in Table 10.5.5.2.4-1 shall be based on the variability in the properties and geologic stratification of the site to which the test results are to be applied. A site, for the purpose of assessing variability, shall be defined ~~in accordance with Article 10.5.5.2.3.~~as a project site, or a portion of it, where the subsurface conditions can be characterized as geologically similar in terms of subsurface stratification, i.e., sequence, thickness, and geologic history of strata, the engineering properties of the strata, and groundwater conditions.

used to establish the final resistance factor, considering the quality and quantity of the available data used in the calibration. The available reliability theory calibrations were conducted for the Reese and O'Neill (1988) method, with the exception of shafts in cohesive intermediate geo-materials (IGMs), in which case the O'Neill and Reese (1999) method was used. In Article 10.8, the O'Neill and Reese (1999) method is recommended. See Allen (2005) for a more detailed explanation on the development of the resistance factors for shaft foundation design, and the implications of the differences in these two shaft design methods on the selection of resistance factors.

~~The information in the commentary to Article 10.5.5.2.3 regarding the number of load tests to conduct considering site variability applies to drilled shafts as well.~~

For single shafts, lower resistance factors are specified to address the lack of redundancy. See Article C10.5.5.2.3 regarding the use of η_R.

Where installation criteria are established based on one or more static load tests, the potential for site variability should be considered. The number of load tests required should be established based on the characterization of site subsurface conditions by the field and laboratory exploration and testing program. One or more static load tests should be performed per site to justify the resistance factor selection ~~as discussed in Article C10.5.5.2.3, applied to drilled shafts installed within the site. See Article C10.5.5.2.3 for details on assessing site variability as applied to selection and use of load tests~~.

Site variability is the most important consideration in evaluating the limits of a site for design purposes. Defining the limits of a site therefore requires sufficient knowledge of the subsurface conditions in terms of general geology, stratigraphy, index and engineering properties of soil and rock, and groundwater conditions. This implies that the extent of the exploration program is sufficient to define the subsurface conditions and their variation across the site.

A designer may choose to design drilled shaft foundations for strength limit states based on a calculated nominal resistance, with the expectation that load testing results will verify that value. The question arises whether to use the resistance factor associated with the design equation or the higher value allowed for load testing. This choice should be based on engineering judgment. The potential risk is that axial resistance measured by load testing may be lower than the nominal resistance used for design, which could require increased shaft dimensions that may be problematic, depending upon the capability of the drilled shaft equipment mobilized for the project and other project-specific factors.

For the specific case of shafts in clay, the resistance factor recommended by Paikowsky et al. (2004) is much lower than the recommendation from Barker et al. (1991). Since the shaft design method for clay is nearly the same for both the 1988 and 1999 methods, a resistance factor that represents the average of the two resistance factor recommendations is provided in Table 10.5.5.2.4-1. This difference may point to the differences in local geologic formations and local construction practices, pointing to the importance of taking such issues into consideration when selecting resistance factors, especially for shafts in clay.

Cohesive IGMs are materials that are transitional between soil and rock in terms of their strength and compressibility, such as ~~residual soils, glacial tills, or very weak rock. See Article C10.8.2.2.3 for a more detailed definition of an IGM.~~clay shales or mudstones with undrained shear strength between 5 and 50 ksf.

Since the mobilization of shaft base resistance is less certain than side resistance due to the greater deformation required to mobilize the base resistance, a lower resistance factor relative to the side resistance is provided for the base resistance in Table 10.5.5.2.4-1. O'Neill and Reese (1999) make further comment that the recommended resistance factor for tip resistance in sand is applicable for conditions of high quality control on the properties of drilling slurries and base cleanout procedures. If high quality control procedures are not used, the resistance factor for the O'Neill and Reese (1999) method for tip resistance in sand should be also be reduced. The amount of reduction should be based on engineering judgment.

Shaft compression load test data should be extrapolated to production shafts that are not load tested as specified in Article 10.8.3.5.6. There is no way to verify shaft resistance for the untested production shafts, other than through good construction inspection and visual observation of the soil or rock encountered in each shaft. Because of this, extrapolation of the shaft load test results to the untested production shafts may introduce some uncertainty. Statistical data are not available to quantify this at this time. Historically, resistance factors higher than 0.70, or their equivalent safety factor in previous practice, have not been used for shaft foundations. If the recommendations in Paikowsky, et al. (2004) are used to establish a resistance factor when shaft static load tests are conducted, in consideration of site variability, the resistance factors recommended by Paikowsky, et al. for this case should be reduced by 0.05, and should be less than or equal to 0.70 as specified in Table 10.5.5.2.4-1.

This issue of uncertainty in how the load test is applied to shafts not load tested is even more acute for shafts subjected to uplift load tests, as failure in uplift can be more abrupt than failure in compression. Hence, a resistance factor of 0.60 for the use of uplift load test results is recommended.

Table 10.5.5.2.4-1—Resistance Factors for Geotechnical Resistance of Drilled Shafts

Method/Soil/Condition			Resistance Factor
Nominal Axial Compressive Resistance of Single-Drilled Shafts, φ_{stat}	Side resistance in clay	α-method (~~O'Neill and Reese, 1999~~ Brown et al., 2010)	0.45
	Tip resistance in clay	Total Stress (~~O'Neill and Reese, 1999~~ Brown et al., 2010)	0.40
	Side resistance in sand	β-method (~~O'Neill and Reese, 1999~~ Brown et al., 2010)	0.55
	Tip resistance in sand	~~O'Neill and Reese (1999)~~ Brown et al., (2010)	0.50
	Side resistance in cohesive IGMs	~~O'Neill and Reese (1999)~~ Brown et al., (2010)	0.60
	Tip resistance in cohesive IGMs	~~O'Neill and Reese (1999)~~ Brown et al., (2010)	0.55
	Side resistance in rock	~~Horvath and Kenney (1979)~~ ~~O'Neill and Reese (1999)~~ Kulhawy et al. (2005) Brown et al. (2010)	0.55
	Side resistance in rock	Carter and Kulhawy (1988)	0.50
	Tip resistance in rock	Canadian Geotechnical Society (1985) Pressuremeter Method (Canadian Geotechnical Society, 1985) ~~O'Neill and Reese (1999)~~Brown et al. (2010)	0.50
Block Failure, φ_{b1}	Clay		0.55
Uplift Resistance of Single-Drilled Shafts, φ_{up}	Clay	α-method (~~O'Neill and Reese, 1999~~ Brown et al., 2010)	0.35
	Sand	β-method (~~O'Neill and Reese, 1999~~ Brown et al., 2010)	0.45
	Rock	~~Horvath and Kenney (1979)~~ ~~O'Neill and Reese (1999)~~ Kulhawy et al. (2005) Brown et al. (2010)	0.40
Group Uplift Resistance, φ_{ug}	Sand and clay		0.45
Horizontal Geotechnical Resistance of Single Shaft or Shaft Group	All materials		1.0
Static Load Test (compression), φ_{load}	All Materials		0.70
Static Load Test (uplift), φ_{upload}	All Materials		0.60

10.5.5.2.5—Micropiles - ***NO CHANGES – NOT SHOWN***

10.5.5.3—Extreme Limit States – ***NO CHANGES – NOT SHOWN***

10.6—SPREAD FOOTINGS

10.6.1—General Considerations – *NO CHANGES – NOT SHOWN*

10.6.2—Service Limit State Design

10.6.2.1—General – ***NO CHANGES – NOT SHOWN***

10.6.2.2—Tolerable Movements – ***NO CHANGES – NOT SHOWN***

10.6.2.3—Loads – ***NO CHANGES – NOT SHOWN***

10.6.2.4—Settlement Analyses

10.6.2.4.1—General - ***NO CHANGES – NOT SHOWN***

10.6.2.4.2—Settlement of Footings on Cohesionless Soils - ***NO CHANGES – NOT SHOWN***

10.6.2.4.3—Settlement of Footings on Cohesive Soils - ***NO CHANGES – NOT SHOWN***

10.6.2.4.4—Settlement of Footings on Rock

For footings bearing on fair to very good rock, according to the Geomechanics Classification system~~, as defined in Article 10.4.6.4,~~ and designed in accordance with the provisions of this Section, elastic settlements may generally be assumed to be less than 0.5 in. When elastic settlements of this magnitude are unacceptable or when the rock is not competent, an analysis of settlement based on rock mass characteristics shall be made.

Where rock is broken or jointed (relative rating of ten or less for *RQD* and joint spacing), the rock joint condition is poor (relative rating of ten or less) or the criteria for fair to very good rock are not met, a settlement analysis should be conducted, and the influence of rock type, condition of discontinuities, and degree of weathering shall be considered in the settlement analysis.

The elastic settlement of footings on broken or jointed rock, in feet, should be taken as:

- For circular (or square) footings:

$$\rho = q_o\left(1-\nu^2\right)\frac{rI_p}{144\,E_m} \qquad (10.6.2.4.4\text{-}1)$$

C10.6.2.4.4

In most cases, it is sufficient to determine settlement using the average bearing stress under the footing.

Where the foundations are subjected to a very large load or where settlement tolerance may be small, settlements of footings on rock may be estimated using elastic theory. The stiffness of the rock mass should be used in such analyses.

The accuracy with which settlements can be estimated by using elastic theory is dependent on the accuracy of the estimated rock mass modulus, E_m. In some cases, the value of E_m can be estimated through empirical correlation with the value of the modulus of elasticity for the intact rock between joints. For unusual or poor rock mass conditions, it may be necessary to determine the modulus from in-situ tests, such as plate loading and pressuremeter tests.

in which:

$$I_p = \frac{\left(\sqrt{\pi}\right)}{\beta_z} \qquad (10.6.2.4.4\text{-}2)$$

- For rectangular footings:

$$\rho = q_o\left(1-\nu^2\right)\frac{BI_p}{144\,E_m} \qquad (10.6.2.4.4\text{-}3)$$

in which:

$$I_p = \frac{\left(L/B\right)^{1/2}}{\beta_z} \qquad (10.6.2.4.4\text{-}4)$$

where:

q_o = applied vertical stress at base of loaded area (ksf)

ν = Poisson's Ratio (dim)

r = radius of circular footing or $B/2$ for square footing (ft)

I_p = influence coefficient to account for rigidity and dimensions of footing (dim)

E_m = rock mass modulus (ksi)

β_z = factor to account for footing shape and rigidity (dim)

Values of I_p should be computed using the β_z values presented in Table 10.6.2.4.2-1 for rigid footings. Where the results of laboratory testing are not available, values of Poisson's ratio, ν, for typical rock types may be taken as specified in Table C10.4.6.5-2. Determination of the rock mass modulus, E_m, should be based on the methods described in ~~Article 10.4.6.5~~ Sabatini (2002).

The magnitude of consolidation and secondary settlements in rock masses containing soft seams or other material with time-dependent settlement characteristics should be estimated by applying procedures specified in Article 10.6.2.4.3.

10.6.2.5—Overall Stability – *NO CHANGES – NOT SHOWN*

10.6.2.6—Bearing Resistance at the Service Limit State

10.6.2.6.1—Presumptive Values for Bearing Resistance ***– NO CHANGES – NOT SHOWN***

10.6.2.6.2—Semiempirical Procedures for Bearing Resistance

Bearing resistance on rock shall be determined using empirical correlation to the Geomechanic Rock Mass Rating System, RMR~~, as specified in Article 10.4.6.4~~. Local experience should be considered in the use of these semi-empirical procedures.

If the recommended value of presumptive bearing resistance exceeds either the unconfined compressive strength of the rock or the nominal resistance of the concrete, the presumptive bearing resistance shall be taken as the lesser of the unconfined compressive strength of the rock or the nominal resistance of the concrete. The nominal resistance of concrete shall be taken as 0.3 f'_c.

10.6.3—Strength Limit State Design

10.6.3.1—Bearing Resistance of Soil – ***NO CHANGES – NOT SHOWN***

10.6.3.2—Bearing Resistance of Rock

10.6.3.2.1—General

The methods used for design of footings on rock shall consider the presence, orientation, and condition of discontinuities, weathering profiles, and other similar profiles as they apply at a particular site.

For footings on competent rock, reliance on simple and direct analyses based on uniaxial compressive rock strengths and *RQD* may be applicable. For footings on less competent rock, more detailed investigations and analyses shall be performed to account for the effects of weathering and the presence and condition of discontinuities.

The designer shall judge the competency of a rock mass by taking into consideration both the nature of the intact rock and the orientation and condition of discontinuities of the overall rock mass. Where engineering judgment does not verify the presence of competent rock, the competency of the rock mass should be verified using the procedures for *RMR* rating ~~in Article 10.4.6.4~~.

C10.6.3.2.1

The design of spread footings bearing on rock is frequently controlled by either overall stability, i.e., the orientation and conditions of discontinuities, or load eccentricity considerations. The designer should verify adequate overall stability at the service limit state and size the footing based on eccentricity requirements at the strength limit state before checking nominal bearing resistance at both the service and strength limit states.

The design procedures for foundations in rock have been developed using the RMR rock mass rating system. Classification of the rock mass should be according to the RMR system. For additional information on the RMR system, see Sabatini et al. (2002).

10.6.3.2.2—Semiempirical Procedures - ***NO CHANGES – NOT SHOWN***

10.6.3.2.3—Analytic Method - ***NO CHANGES – NOT SHOWN***

10.6.3.2.4—Load Test - ***NO CHANGES – NOT SHOWN***

10.6.3.3—Eccentric Load Limitations – ***NO CHANGES – NOT SHOWN***

10.6.3.4—Failure by Sliding – ***NO CHANGES – NOT SHOWN***

10.6.4—Extreme Event Limit State Design – ***NO CHANGES – NOT SHOWN***

10.6.5—Structural Design – ***NO CHANGES – NOT SHOWN***

10.7—DRIVEN PILES – ***NO CHANGES – NOT SHOWN***

10.8—DRILLED SHAFTS

10.8.1—General

10.8.1.1—Scope - ***NO CHANGES – NOT SHOWN***

10.8.1.2—Shaft Spacing, Clearance, and Embedment into Cap - ***NO CHANGES – NOT SHOWN***

10.8.1.3—Shaft Diameter and Enlarged Bases - ***NO CHANGES – NOT SHOWN***

10.8.1.4—Battered Shafts - ***NO CHANGES – NOT SHOWN***

10.8.1.5—Drilled Shaft Resistance

Drilled shafts shall be designed to have adequate axial and structural resistances, tolerable settlements, and tolerable lateral displacements.

C10.8.1.5

The drilled shaft design process is discussed in detail in Drilled Shafts: Construction Procedures and Design Methods (~~O'Neill and Reese, 1999~~ Brown, et al., 2010).

The axial resistance of drilled shafts shall be determined through a suitable combination of subsurface investigations, laboratory and/or in-situ tests, analytical methods, and load tests, with reference to the history of past performance. Consideration shall also be given to:

- The difference between the resistance of a single shaft and that of a group of shafts;
- The resistance of the underlying strata to support the load of the shaft group;
- The effects of constructing the shaft(s) on adjacent structures;
- The possibility of scour and its effect;
- The transmission of forces, such as downdrag forces, from consolidating soil;
- Minimum shaft penetration necessary to satisfy the requirements caused by uplift, scour, downdrag, settlement, liquefaction, lateral loads and seismic conditions;
- Satisfactory behavior under service loads;
- Drilled shaft nominal structural resistance; and
- Long-term durability of the shaft in service, i.e., corrosion and deterioration.

Resistance factors for shaft axial resistance for the strength limit state shall be as specified in Table 10.5.5.2.4-1.

The method of construction may affect the shaft axial and lateral resistance. The shaft design parameters shall take into account the likely construction methodologies used to install the shaft.

The performance of drilled shaft foundations can be greatly affected by the method of construction, particularly side resistance. The designer should consider the effects of ground and groundwater conditions on shaft construction operations and delineate, where necessary, the general method of construction to be followed to ensure the expected performance. Because shafts derive their resistance from side and tip resistance, which is a function of the condition of the materials in direct contact with the shaft, it is important that the construction procedures be consistent with the material conditions assumed in the design. Softening, loosening, or other changes in soil and rock conditions caused by the construction method could result in a reduction in shaft resistance and an increase in shaft displacement. Therefore, evaluation of the effects of the shaft construction procedure on resistance should be considered an inherent aspect of the design. Use of slurries, varying shaft diameters, and post grouting can also affect shaft resistance.

Soil parameters should be varied systematically to model the range of anticipated conditions. Both vertical and lateral resistance should be evaluated in this manner.

Procedures that may affect axial or lateral shaft resistance include, but are not limited to, the following:

- Artificial socket roughening, if included in the design nominal axial resistance assumptions.
- Removal of temporary casing where the design is dependent on concrete-to-soil adhesion.
- The use of permanent casing.
- Use of tooling that produces a uniform cross-section where the design of the shaft to resist lateral loads cannot tolerate the change in stiffness if telescoped casing is used.

It should be recognized that the design procedures provided in these Specifications assume compliance to construction specifications that will produce a high quality shaft. Performance criteria should be included in the construction specifications that require:

- Shaft bottom cleanout criteria,
- Appropriate means to prevent side wall movement or failure (caving) such as temporary casing, slurry, or a combination of the two,
- Slurry maintenance requirements including minimum slurry head requirements, slurry testing requirements, and maximum time the shaft may be left open before concrete placement.

If for some reason one or more of these performance criteria are not met, the design should be reevaluated and the shaft repaired or replaced as necessary.

10.8.1.6—Determination of Shaft Loads

10.8.1.6.1—General - ***NO CHANGES – NOT SHOWN***

10.8.1.6.2—Downdrag

The provisions of Articles 10.7.1.6.2 and 3.11.8 shall apply for determination of load due to downdrag.

For shafts with tip bearing in a dense stratum or rock where design of the shaft is structurally controlled, and downdrag shall be considered at the strength and extreme event limit states.

For shafts with tip bearing in soil, downdrag shall not be considered at the strength and extreme limit states if settlement of the shaft is less than failure criterion.

C10.8.1.6.2

See commentary to Articles ~~10.7.1.6.2 and~~ 3.11.8.

Downdrag loads may be estimated using the α-method, as specified in Article 10.8.3.5.1b, ~~for calculating~~ to calculate negative shaft ~~resistance~~ friction. As with positive shaft resistance, ~~the top 5.0 ft and a bottom length taken as one shaft diameters~~ shaft length assumed to not contribute to nominal side resistance should also be assumed to not contribute to downdrag loads.

When using the α-method, an allowance should be made for a possible increase in the undrained shear strength as consolidation occurs. Downdrag loads may also come from cohesionless soils above settling cohesive soils~~, requiring granular soil friction methods be used in such zones to estimate downdrag loads~~. The downdrag caused by settling cohesionless soils may be estimated using the β method presented in Article 10.8.3.5.2.

Downdrag occurs in response to relative downward deformation of the surrounding soil to that of the shaft, and may not exist if downward movement of the drilled shaft in response to axial compression forces exceeds the vertical deformation of the soil. The response of a drilled shaft to downdrag in combination with the other forces acting at the head of the shaft therefore is complex and a realistic evaluation of actual limit states that may occur requires careful consideration of two issues: (1) drilled shaft load-settlement behavior, and (2) the time period over which downdrag occurs relative to the time period over which nonpermanent components of load occur. When these factors are taken into account, it is appropriate to consider different downdrag forces for evaluation of geotechnical strength limit states than for structural strength limit states. These issues are addressed in Brown et al. (2010).

10.8.1.6.3—Uplift - ***NO CHANGES – NOT SHOWN***

10.8.2—Service Limit State Design

10.8.2.1—Tolerable Movements - *NO CHANGES – NOT SHOWN*

10.8.2.2—Settlement

10.8.2.2.1—General - ***NO CHANGES – NOT SHOWN***

10.8.2.2.2—Settlement of Single-Drilled Shaft

The settlement of single-drilled shafts shall be estimated ~~in consideration of~~ as a sum of the following:

- Short-term settlement resulting from load transfer,
- Consolidation settlement if constructed ~~in~~ where cohesive soils~~s~~ exists beneath the shaft tip, and
- Axial compression of the shaft.

The normalized load-settlement curves shown in Figures 10.8.2.2.2-1 through 10.8.2.2.2-4 should be used to limit the nominal shaft axial resistance computed as specified for the strength limit state in Article 10.8.3 for service limit state tolerable movements. Consistent values of normalized settlement shall be used for limiting the base and side resistance when using these Figures. Long-term settlement should be computed according to Article 10.7.2 using the equivalent footing method and added to the short-term settlements estimated using Figures 10.8.2.2.2-1 through 10.8.2.2.2-4.

~~Other methods for evaluating shaft settlements that may be used are found in O'Neill and Reese (1999).~~

C10.8.2.2.2

O'Neill and Reese (1999) have summarized load-settlement data for drilled shafts in dimensionless form, as shown in Figures 10.8.2.2.2-1 through 10.8.2.2.2-4. These curves do not include consideration of long-term consolidation settlement for shafts in cohesive soils. Figures 10.8.2.2.2-1 and 10.8.2.2.2-2 show the load-settlement curves in side resistance and in end bearing for shafts in cohesive soils. Figures 10.8.2.2.2-3 and 10.8.2.2.2-4 are similar curves for shafts in cohesionless soils. These curves should be used for estimating short-term settlements of drilled shafts.

The designer should exercise judgment relative to whether the trend line, one of the limits, or some relation in between should be used from Figures 10.8.2.2.2-1 through 10.8.2.2.2-4.

The values of the load-settlement curves in side resistance were obtained at different depths, taking into account elastic shortening of the shaft. Although elastic shortening may be small in relatively short shafts, it may be substantial in longer shafts. The amount of elastic shortening in drilled shafts varies with depth. O'Neill and Reese (1999) have described an approximate procedure for estimating the elastic shortening of long-drilled shafts.

~~Settlements induced by loads in end bearing are different for shafts in cohesionless soils and in cohesive soils. Although drilled shafts in cohesive soils typically have a well-defined break in a load-displacement curve, shafts in cohesionless soils often have no well-defined failure at any displacement. The resistance of drilled shafts in cohesionless soils continues to increase as the settlement increases beyond five percent of the base diameter. The shaft end bearing~~ $\cancel{R_p}$ ~~is typically fully mobilized at displacements of two to five percent of the base diameter for shafts in cohesive soils. The unit end bearing resistance for the strength limit state (see Article 10.8.3.3) is defined as the bearing pressure required to cause vertical deformation equal to five percent of the shaft diameter, even though this does not correspond to complete failure of the soil beneath the base of the shaft.~~

Induced settlements for isolated drilled shafts are different for elements in cohesive soils and in cohesionless soils. In cohesive soils, the failure threshold, or nominal axial resistance corresponds to mobilization of the full available side resistance, plus the full available base resistance. In cohesive soils, the failure threshold has been shown to occur at an average normalized deformation of 4 percent of the shaft diameter. In cohesionless soils, the failure threshold is the force corresponding to mobilization of the full side resistance, plus the base resistance corresponding to settlement at a defined failure criterion. This has been traditionally defined as the bearing pressure required to cause vertical deformation equal to 5 percent of the shaft diameter, even though this does not correspond to complete failure of the soil beneath the base of the shaft. Note that nominal base resistance in cohesionless soils is calculated according to the empirical correlation given by Eq. 10.8.3.5.2c-1 in terms of N-value. That relationship was developed using a base resistance corresponding to 5 percent normalized displacement. If a normalized displacement other than 5 percent is used, the base resistance calculated by Eq. 10.8.3.5.2c-1 must be corrected.

The curves in Figures 10.8.2.2.2-1 and 10.8.2.2.2-3 also show the settlements at which the side resistance is mobilized. The shaft skin friction R_s is typically fully mobilized at displacements of 0.2 percent to 0.8 percent of the shaft diameter for shafts in cohesive soils. For shafts in cohesionless soils, this value is 0.1 percent to 1.0 percent.

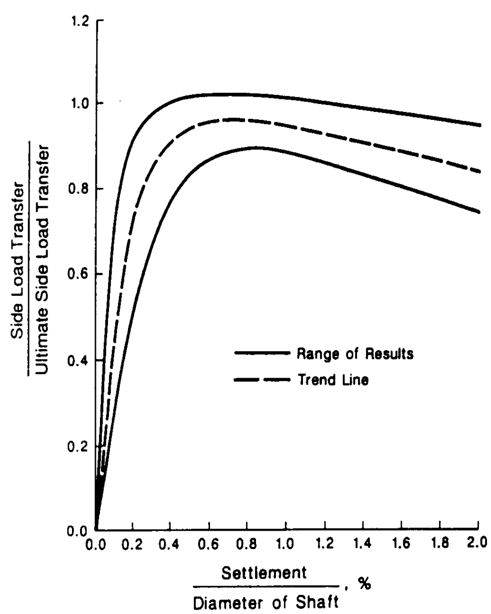


Figure 10.8.2.2.2-1 Normalized Load Transfer in Side Resistance versus Settlement in Cohesive Soils (from O'Neill and Reese, 1999)

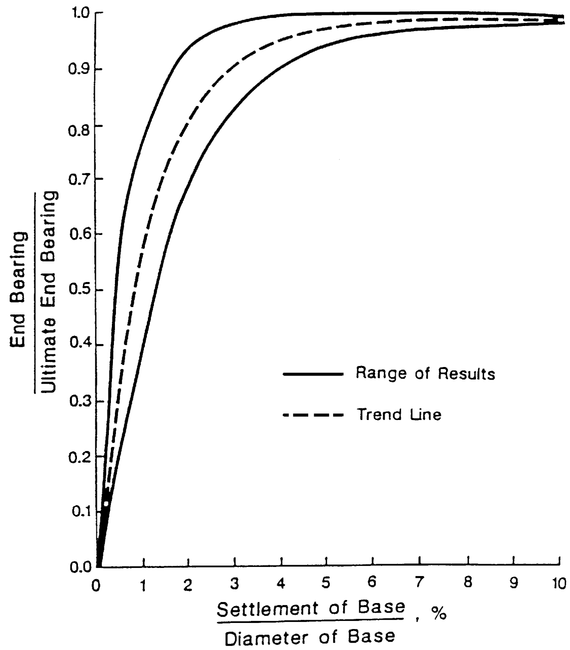


Figure 10.8.2.2.2-2—Normalized Load Transfer in End Bearing versus Settlement in Cohesive Soils (from O'Neill and Reese, 1999)

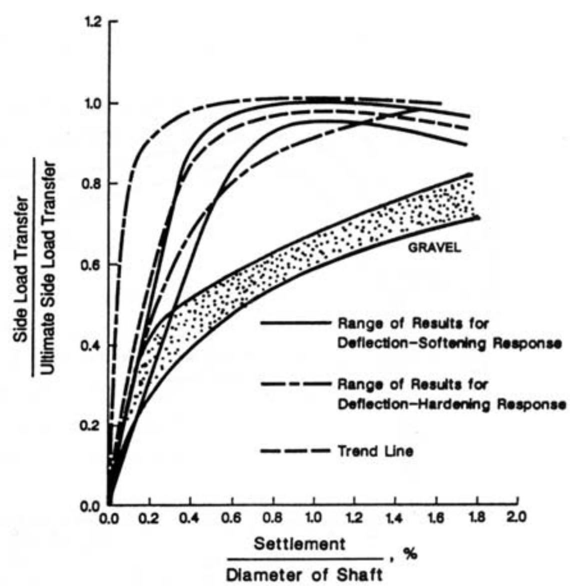


Figure 10.8.2.2.2-3—Normalized Load Transfer in Side Resistance versus Settlement in Cohesionless Soils (from O'Neill and Reese, 1999)

The deflection-softening response typically applies to cemented or partially cemented soils, or other soils that exhibit brittle behavior, having low residual shear strengths at larger deformations. Note that the trend line for sands is a reasonable approximation for either the deflection-softening or deflection-hardening response.

The normalized load-settlement curves require separate evaluation of an isolated drilled shaft for side and base resistance. Brown et al. (2010) provide alternate normalized load-settlement curves that may be used for estimation of settlement of a single drilled shaft considering combined side and base resistance. The method is based on modeling the average load deformation behavior observed from field load tests and incorporates the load test data used in development of the curves provided by O'Neill and Reese (1999). Additional methods that consider numerical simulations of axial load transfer and approximations based on elasto-plastic solutions are available in Brown et al. (2010).

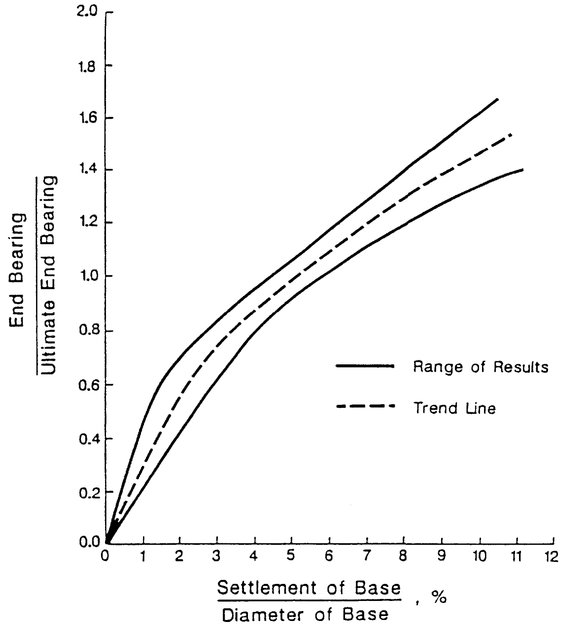


Figure 10.8.2.2.2-4—Normalized Load Transfer in End Bearing versus Settlement in Cohesionless Soils (from O'Neill and Reese, 1999)

10.8.2.2.3—Intermediate Geomaterials (IGMs)

For detailed settlement estimation of shafts in IGMs, the procedures ~~provided by O'Neill and Reese (1999)~~ described by Brown et al. (2010) should be used.

C10.8.2.2.3

IGMs are defined by ~~O'Neill and Reese (1999)~~ Brown et al. (2010) as follows:

- *Cohesive IGM*—clay shales or mudstones with an S_u of 5 to 50 ksf, and
- ~~*Cohesionless* granular tills or granular residual soils with $N1_{60}$ greater than 50 blows/ft.~~

10.8.2.2.4—Group Settlement

The provisions of Article 10.7.2.3 shall apply. Shaft group effect shall be considered for groups of 2 shafts or more.

C10.8.2.2.4

See commentary to Article 10.7.2.3.

O'Neill and Reese (1999) summarize various studies on the effects of shaft group behavior. These studies were for groups that consisted of 1 × 2 to 3 × 3 shafts. These studies suggest that group effects are relatively unimportant for shaft center-to-center spacing of 5*D* or greater.

10.8.2.3—Horizontal Movement of Shafts and Shaft Groups

The provisions of Articles 10.5.2.1 and 10.7.2.4 shall apply.

For shafts socketed into rock, the input properties used to determine the response of the rock to lateral loading shall consider both the intact shear strength of the rock and the rock mass characteristics. The designer shall also consider the orientation and condition of discontinuities of the overall rock mass. Where specific adversely oriented discontinuities are not present, but the rock mass is fractured such that its intact strength is considered compromised, the rock mass shear strength

C10.8.2.3

See commentary to Articles 10.5.2.1 and 10.7.2.4.

For shafts socketed into rock, approaches to developing p-y response of rock masses include both a weak rock response and a strong rock response. For the strong rock response, the potential for brittle fracture should be considered. If horizontal deflection of the rock mass is greater than 0.0004b, a lateral load test to evaluate the response of the rock to lateral loading should be considered. Brown et al. (2010) provide a summary of a methodology that may be used to estimate parameters should be assessed using the procedures for *GSI* rating in Article 10.4.6.4. For lateral deflection of the rock adjacent to the shaft greater than 0.0004b, where b is the diameter of the rock socket, the potential for brittle fracture of the rock shall be considered.

the lateral load response of shafts in rock. Additional background on lateral loading of shafts in rock is provided in Turner (2006).

These methods for estimating the response of shafts in rock subjected to lateral loading use the unconfined compressive strength of the intact rock as the main input property. While this property is meaningful for intact rock, and was the key parameter used to correlate to shaft lateral load response in rock, it is not meaningful for fractured rock masses. If the rock mass is fractured enough to justify characterizing the rock shear strength using the GSI, the rock mass should be characterized as a c-ϕ material, and confining stress (i.e., σ'_3) present within the rock mass should be considered when establishing a rock mass shear strength for lateral response of the shaft. If the P-y method of analysis is used to model horizontal resistance, user-specified P-y curves should be derived. A method for developing hyperbolic P-y curves is described by Liang et al. (2009).

10.8.2.4—Settlement Due to Downdrag - *NO CHANGES – NOT SHOWN*

10.8.2.5—Lateral Squeeze - *NO CHANGES – NOT SHOWN*

10.8.3—Strength Limit State Design

10.8.3.1—General - *NO CHANGES – NOT SHOWN*

10.8.3.2—Groundwater Table and Buoyancy - *NO CHANGES – NOT SHOWN*

10.8.3.3—Scour - *NO CHANGES – NOT SHOWN*

10.8.3.4—Downdrag

~~The provisions of Article 10.7.3.7 shall apply.~~

The foundation should be designed so that the available factored axial geotechnical resistance is greater than the factored loads applied to the shaft, including the downdrag, at the strength limit state. The nominal shaft resistance available to support structure loads plus downdrag shall be estimated by considering only the positive skin and tip resistance below the lowest layer contributing to the downdrag. The drilled shaft shall be designed structurally to resist the downdrag plus structure loads.

C10.8.3.4

~~See commentary to Article 10.7.3.7.~~

The static analysis procedures in Article 10.8.3.5 may be used to estimate the available drilled shaft nominal side and tip resistances to withstand the downdrag plus other axial force effects.

Nominal resistance may also be estimated using an instrumented static load test provided the side resistance within the zone contributing to downdrag is subtracted from the resistance determined from the load test.

As stated in Article C10.8.1.6.2, that it is appropriate to apply different downdrag forces for evaluation of geotechnical strength limit states than for structural strength limit states. A drilled shaft with its tip bearing in stiff material, such as rock or hard soil, would be expected to limit settlement to very small values. In this case, the full downdrag force could occur in combination with the other axial force effects, because downdrag will not be reduced if there is little or no downward movement of the shaft. Therefore, the factored force effects resulting from all load components, including full factored downdrag, should be used to check the structural strength limit state of the drilled shaft.

A rational approach to evaluating this strength limit state will incorporate the force effects occurring at this magnitude of downward displacement. This will include the factored axial force effects transmitted to the head of the shaft, plus the downdrag loads occurring at a downward displacement defining the failure criterion. In many cases, this amount of downward displacement will reduce or eliminate downdrag. For soil layers that undergo settlement exceeding the failure criterion (for example, 5 percent of B for shafts bearing in sand), downdrag loads are likely to remain and should be included. This approach requires the designer to predict the magnitude of downdrag load occurring at a specified downward displacement. This can be accomplished using the hand calculation procedure described in Brown et al. (2010) or with commercially available software.

When downdrag loads are determined to exist at a downward displacement defining failure, evaluation of drilled shafts for the geotechnical strength limit state in compression should be conducted under a load combination that is limited to permanent loads only, including the calculated downdrag load at a settlement defining the failure criterion, but excluding nonpermanent loads, such as live load, temperature changes, etc. See Brown et al. (2010) for further discussion.

When analysis of a shaft subjected to downdrag shows that the downdrag load would be eliminated in order to achieve a defined downward displacement, evaluation of geotechnical and structural strength limit states in compression should be conducted under the full load combination corresponding to the relevant strength limit state, including the non-permanent components of load, but not including downdrag.

10.8.3.5—Nominal Axial Compression Resistance of Single Drilled Shafts - *NO CHANGES – NOT SHOWN*

10.8.3.5.1—Estimation of Drilled Shaft Resistance in Cohesive Soils

10.8.3.5.1a—General - ***NO CHANGES – NOT SHOWN***

10.8.3.5.1b—Side Resistance

The nominal unit side resistance, q_s, in ksf, for shafts in cohesive soil loaded under undrained loading conditions by the α-Method shall be taken as:

$$q_s = \alpha S_u \qquad (10.8.3.5.1b\text{-}1)$$

C10.8.3.5.1b

The α-method is based on total stress. For effective stress methods for shafts in clay, see ~~O'Neill and Reese (1999)~~ Brown et al. (2010).

The adhesion factor is an empirical factor used to correlate the results of full-scale load tests with the

in which:

$$\alpha = 0.55 \text{ for } \frac{S_u}{p_a} \le 1.5 \qquad (10.8.3.5.1b\text{-}2)$$

$$\alpha = 0.55 - 0.1\left(S_u/p_a - 1.5\right)$$
$$\text{for } 1.5 \le S_u/p_a \le 2.5 \qquad (10.8.3.5.1b\text{-}3)$$

where:

S_u = undrained shear strength (ksf)

α = adhesion factor (dim)

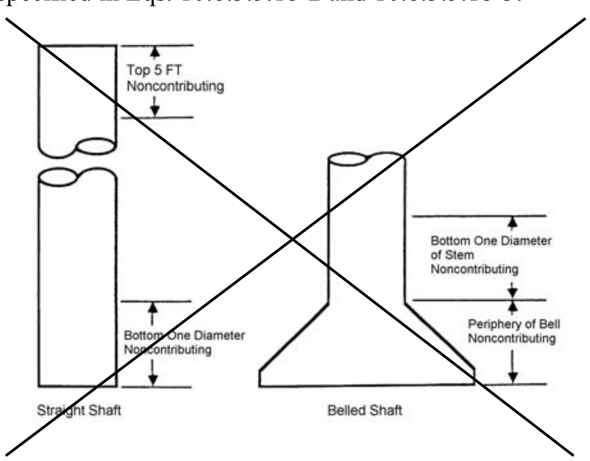
p_a = atmospheric pressure (= 2.12 ksf)

The following portions of a drilled shaft, illustrated in Figure 10.8.3.5.1b-1, should not be taken to contribute to the development of resistance through skin friction:

- At least the top 5.0 ft of any shaft;
- ~~For straight shafts, a bottom length of the shaft taken as the shaft diameter;~~
- Periphery of belled ends, if used; and
- ~~Distance above a belled end taken as equal to the shaft diameter.~~

When permanent casing is used, the side resistance shall be adjusted with consideration to the type and length of casing to be used, and how it is installed.

Values of α for contributing portions of shafts excavated dry in open or cased holes should be as specified in Eqs. 10.8.3.5.1b-2 and 10.8.3.5.1b-3.



material property or characteristic of the cohesive soil. The adhesion factor is usually related to S_u and is derived from the results of full-scale pile and drilled shaft load tests. Use of this approach presumes that the measured value of S_u is correct and that all shaft behavior resulting from construction and loading can be lumped into a single parameter. Neither presumption is strictly correct, but the approach is used due to its simplicity.

Steel casing will generally reduce the side resistance of a shaft. No specific data is available regarding the reduction in skin friction resulting from the use of permanent casing relative to concrete placed directly against the soil. Side resistance reduction factors for driven steel piles relative to concrete piles can vary from 50 to 75 percent, depending on whether the steel is clean or rusty, respectively (Potyondy, 1961). Greater reduction in the side resistance may be needed if oversized cutting shoes or splicing rings are used.

If open-ended pipe piles are driven full depth with an impact hammer before soil inside the pile is removed, and left as a permanent casing, driven pile static analysis methods may be used to estimate the side resistance as described in Article 10.7.3.8.6.

The upper 5.0 ft of the shaft is ignored in estimating R_n, to account for the effects of seasonal moisture changes, disturbance during construction, cyclic lateral loading, and low lateral stresses from freshly placed concrete. ~~The lower 1.0 diameter length above the shaft tip or top of enlarged base is ignored due to the development of tensile cracks in the soil near these regions of the shaft and a corresponding reduction in lateral stress and side resistance.~~

Bells or underreams constructed in stiff fissured clay often settle sufficiently to result in the formation of a gap above the bell that will eventually be filled by slumping soil. Slumping will tend to loosen the soil immediately above the bell and decrease the side resistance along the lower portion of the shaft.

The value of α is often considered to vary as a function of S_u. Values of α for drilled shafts are recommended as shown in Eqs. 10.8.3.5.1b-2 and 10.8.3.5.1b-3, based on the results of back-analyzed, full-scale load tests. ~~This recommendation is based on eliminating the upper 5.0 ft and lower 1.0 diameter of the shaft length during back-analysis of load test results.~~ The load tests were conducted in insensitive cohesive soils. Therefore, if shafts are constructed in sensitive clays, values of α may be different than those obtained from Eqs. 10.8.3.5.1b-2 and 10.8.3.5.1b-3. Other values of α may be used if based on the results of load tests.

The depth of 5.0 ft at the top of the shaft may need to be increased if the drilled shaft is installed in expansive clay, if scour deeper than 5.0 ft is anticipated, if there is substantial groundline deflection from lateral loading, or if there are other long-term loads or construction factors that could affect shaft resistance.

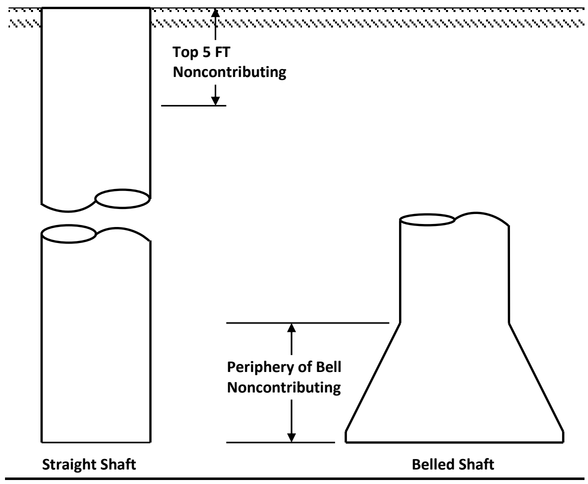


Figure 10.8.3.5.1b-1—Explanation of Portions of Drilled Shafts Not Considered in Computing Side Resistance (~~O'Neill and Reese, 1999~~ Brown et al., 2010)

~~A reduction in the effective length of the shaft contributing to side resistance has been attributed to horizontal stress relief in the region of the shaft tip, arising from development of outward radial stresses at the toe during mobilization of tip resistance. The influence of this effect may extend for a distance of 1*B* above the tip (O'Neill and Reese, 1999). The effectiveness of enlarged bases is limited when *L*/*D* is greater than 25.0 due to the lack of load transfer to the tip of the shaft.~~

The values of α obtained from Eqs. 10.8.3.5.1b-2 and 10.8.3.5.1b-3 are considered applicable for both compression and uplift loading.

10.8.3.5.1c—Tip Resistance

For axially loaded shafts in cohesive soil, the nominal unit tip resistance, q_p, by the total stress method as provided in ~~O'Neill and Reese (1999)~~ Brown et al. (2010) shall be taken as:

$$q_p = N_c S_u \leq 80.0 \ \underline{ksf} \qquad (10.8.3.5.1c\text{-}1)$$

in which:

$$N_c = 6\left[1 + 0.2\left(\frac{Z}{D}\right)\right] \leq 9 \qquad (10.8.3.5.1c\text{-}2)$$

where:

D = diameter of drilled shaft (ft)

Z = penetration of shaft (ft)

S_u = undrained shear strength (ksf)

The value of S_u should be determined from the results of in-situ and/or laboratory testing of undisturbed samples obtained within a depth of 2.0 diameters below the tip of the shaft. If the soil within 2.0 diameters of the tip has S_u <0.50 ksf, the value of N_c should be multiplied by 0.67.

C10.8.3.5.1c

These equations are for total stress analysis. For effective stress methods for shafts in clay, see ~~O'Neill and Reese (1999)~~ Brown et al. (2010).

The limiting value of 80.0 ksf for q_p is not a theoretical limit but a limit based on the largest measured values. A higher limiting value may be used if based on the results of a load test, or previous successful experience in similar soils.

10.8.3.5.2—Estimation of Drilled Shaft Resistance in Cohesionless Soils

10.8.3.5.2a—General

Shafts in cohesionless soils should be designed by

C10.8.3.5.2a

The factored resistance should be determined in effective stress methods for drained loading conditions or by empirical methods based on in-situ test results.

consideration of available experience with similar conditions.

~~Although many field load tests have been performed on drilled shafts in clays, very few have been performed on drilled shafts in sands.~~ The shear strength of cohesionless soils can be characterized by an angle of internal friction, ϕ_f, or empirically related to its *SPT* blow count, *N*. Methods of estimating shaft resistance and end bearing are presented below. Judgment and experience should always be considered.

10.8.3.5.2b—Side Resistance

~~The nominal axial resistance of drilled shafts in cohesionless soils by the β-method shall be taken as~~The side resistance for shafts in cohesionless soils shall be determined using the β method, take as:

$$\cancel{q_s = \beta\sigma'_v \leq 4.0 \text{ for } 0.25 \leq \beta \leq 1.2} \quad \text{~~(10.8.3.5.2b-1)~~}$$

~~in which, for sandy soils:~~

- ~~for $N_{60} \geq 15$:~~

$$\cancel{\beta = 1.5 - 0.135\sqrt{z}} \quad \text{~~(10.8.3.5.2b-2)~~}$$

- ~~for $N_{60} < 15$:~~

$$\cancel{\beta = \frac{N_{60}}{15}(1.5 - 0.135\sqrt{z})} \quad \text{~~(10.8.3.5.2b-3)~~}$$

~~where:~~

~~σ'_v = vertical effective stress at soil layer mid-depth (ksf)~~

~~β = load transfer coefficient (dim)~~

~~z = depth below ground, at soil layer mid-depth (ft)~~

~~N_{60} = average *SPT* blow count (corrected only for hammer efficiency) in the design zone under consideration (blows/ft)~~

~~Higher values may be used if verified by load tests.~~

~~For gravelly sands and gravels, Eq. 10.8.3.5.2b-4 should be used for computing β where $N_{60} \geq 15$. If $N_{60} < 15$, Eq. 10.8.3.5.2b-3 should be used.~~

$$\cancel{\beta = 2.0 - 0.06(z)^{0.75}} \quad \text{~~(10.8.3.5.2b-4)~~}$$

$$q_s = \beta\, \sigma'_v \quad (10.8.3.5.2b\text{-}1)$$

in which:

C10.8.3.5.2b

~~O'Neill and Reese (1999) provide additional discussion of computation of shaft side resistance and recommend allowing β to increase to 1.8 in gravels and gravelly sands, however, they recommend limiting the unit side resistance to 4.0 ksf in all soils.~~

~~O'Neill and Reese (1999) proposed a method for uncemented soils that uses a different approach in that the shaft resistance is independent of the soil friction angle or the *SPT* blow count. According to their findings, the friction angle approaches a common value due to high shearing strains in the sand caused by stress relief during drilling.~~

~~The detailed development of Eq. 10.8.3.5.2b-4 is provided in O'Neill and Reese (1999).~~

The method described herein is based on axial load tests on drilled shafts as presented by Chen and Kulhawy (2002) and updated by Kulhawy and Chen (2007). This method provides a rational approach for relating unit side resistance to N-values and to the state

$$\beta = \left(1 - \sin \varphi'_f\right)\left(\frac{\sigma'_p}{\sigma'_v}\right)^{\sin \varphi'_f} \tan \varphi'_f \qquad (10.8.3.5.2b\text{-}2)$$

where:

β = load transfer coefficient (dim)

φ'_f = friction angle of cohesionless soil layer (°)

σ'_p = effective vertical preconsolidation stress

σ'_v = vertical effective stress at soil layer mid-depth

The correlation for effective soil friction angle for use in the above equations shall be taken as:

$$\varphi'_f = 27.5 + 9.2 \log\left[\left(N_1\right)_{60}\right] \qquad (10.8.3.5.2b\text{-}3)$$

where:

$(N_1)_{60}$ = SPT N-value corrected for effective overburden stress

The preconsolidation stress in Eq. 10.8.3.5.2b-2 should be approximated through correlation to SPT N-values. For sands:

$$\frac{\sigma'_p}{p_a} = 0.47\left(N_{60}\right)^m \qquad (10.8.3.5.2b\text{-}4)$$

where:

m = 0.6 for clean quartzitic sands

m = 0.8 for silty sand to sandy silts

p_a = atmospheric pressure (same units as σ'_p, 2.12 ksf or 14.7 psi)

For gravelly soils:

$$\frac{\sigma'_p}{p_a} = 0.15\left(N_{60}\right) \qquad (10.8.3.5.2b\text{-}5)$$

of effective stress acting at the soil-shaft interface. This approach replaces the previously used depth-dependent β-method developed by O'Neill and Reese (1999), which does not account for variations in N-value or effective stress on the calculated value of β. Further discussion, including the detailed development of Eq. 10.8.3.5.2b-2, is provided in (Brown et al. 2010).

When permanent casing is used, the side resistance shall be adjusted with consideration to the type and length of casing to be used, and how it is installed.

Steel casing will generally reduce the side resistance of a shaft. No specific data is available regarding the reduction in skin friction resulting from the use of permanent casing relative concrete placed directly against the soil. Side resistance reduction factors for driven steel piles relative to concrete piles can vary from 50 to 75 percent, depending on whether the steel is clean or rusty, respectively (Potyondy, 1961). Casing reduction factors of 0.6 to 0.75 are commonly used. Greater reduction in the side resistance may be needed if oversized cutting shoes or splicing rings are used.

If open-ended pipe piles are driven full depth with an impact hammer before soil inside the pile is removed, and left as a permanent casing, driven pile static analysis methods may be used to estimate the side resistance as described in Article 10.7.3.8.6.

10.8.3.5.2c—Tip Resistance

The nominal tip resistance, q_p, in ksf, for drilled shafts in cohesionless soils by the ~~O'Neill and Reese (1999)~~ method described in Brown et al. (2010) shall be taken as:

$$\cancel{\text{for } N_{60} \leq 50,\ q_p = 1.2N_{60}} \quad (10.8.3.5.2c\text{-}1)$$

$$\text{If } N_{60} \leq 50, \text{ then } q_p = 1.2N_{60}$$

where:

N_{60} = average *SPT* blow count (corrected only for hammer efficiency) in the design zone under consideration (blows/ft)

The value of q_p in Eq. 10.8.3.5.2c-1 should be limited to 60 ksf, unless greater values can be justified though load test data.

~~Cohesionless soils with *SPT-N*~~$\cancel{_{60}}$ ~~blow counts greater than 50 shall be treated as intermediate geomaterial (IGM) and the tip resistance, in ksf, taken as:~~

$$\cancel{q_p = 0.59\left[N_{60}\left(\frac{p_a}{\sigma'_v}\right)\right]^{0.8}\sigma'_v} \quad \cancel{(10.8.3.5.2c\text{-}2)}$$

~~where:~~

~~p_a = atmospheric pressure (= 2.12 ksf)~~

~~σ'_v = vertical effective stress at the tip elevation of the shaft (ksf)~~

~~N_{60} should be limited to 100 in Eq. 10.8.3.5.2c-2 if higher values are measured.~~

C10.8.3.5.2c

~~O'Neill and Reese (1999)~~ Brown et al. (2010) provide additional discussion regarding the computation of nominal tip resistance and on tip resistance in specific geologic environments.

~~See O'Neill and Reese (1999) for background on IGMs.~~

10.8.3.5.3—Shafts in Strong Soil Overlying Weaker Compressible Soil - ***NO CHANGES – NOT SHOWN***

10.8.3.5.4—Estimation of Drilled Shaft Resistance in Rock

10.8.3.5.4a—General

Drilled shafts in rock subject to compressive loading shall be designed to support factored loads in:

- Side-wall shear comprising skin friction on the wall of the rock socket; or
- End bearing on the material below the tip of the drilled shaft; or
- A combination of both.

The difference in the deformation required to mobilize skin friction in soil and rock versus what is required to mobilize end bearing shall be considered when estimating axial compressive resistance of shafts embedded in rock. Where end bearing in rock is used as part of the axial compressive resistance in the design, the contribution of skin friction in the rock shall be reduced to account for the loss of skin friction that occurs once the shear deformation along the shaft sides is greater than the peak rock shear deformation, i.e., once the rock shear strength begins to drop to a residual value.

C10.8.3.5.4a

Methods presented in this Article to calculate drilled shaft axial resistance require an estimate of the uniaxial compressive strength of rock core. Unless the rock is massive, the strength of the rock mass is most frequently controlled by the discontinuities, including orientation, length, and roughness, and the behavior of the material that may be present within the discontinuity, e.g., gouge or infilling. The methods presented are semi-empirical and are based on load test data and site-specific correlations between measured resistance and rock core strength.

Design based on side-wall shear alone should be considered for cases in which the base of the drilled hole cannot be cleaned and inspected or where it is determined that large movements of the shaft would be required to mobilize resistance in end bearing.

Design based on end-bearing alone should be considered where sound bedrock underlies low strength overburden materials, including highly weathered rock. In these cases, however, it may still be necessary to socket the shaft into rock to provide lateral stability.

Where the shaft is drilled some depth into sound rock, a combination of sidewall shear and end bearing can be assumed (Kulhawy and Goodman, 1980).

If the rock is degradable, use of special construction procedures, larger socket dimensions, or reduced socket resistance should be considered.

Factors that should be considered when making an engineering judgment to neglect any component of resistance (side or base) are discussed in Article 10.8.3.5.4d. In most cases, both side and base resistances should be included in limit state evaluation of rock-socketed shafts.

For drilled shafts installed in karstic formations, exploratory borings should be advanced at each drilled shaft location to identify potential cavities. Layers of compressible weak rock along the length of a rock socket and within approximately three socket diameters or more below the base of a drilled shaft may reduce the resistance of the shaft.

For rock that is stronger than concrete, the concrete shear strength will control the available side friction, and the strong rock will have a higher stiffness, allowing significant end bearing to be mobilized before the side wall shear strength reaches its peak value. Note that concrete typically reaches its peak shear strength at about 250 to 400 microstrain (for a 10-ft long rock socket, this is approximately 0.5 in. of deformation at the top of the rock socket). If strains or deformations greater than the value at the peak shear stress are anticipated to mobilize the desired end bearing in the rock, a residual value for the skin friction can still be used. Article 10.8.3.5.4d provides procedures for computing a residual value of the skin friction based on the properties of the rock and shaft.

10.8.3.5.4b—Side Resistance

~~For drilled shafts socketed into rock, shaft resistance, in ksf, may be taken as (Horvath and Kenney, 1979):~~

$$\cancel{q_s = 0.65\alpha_E p_a \left(q_u / p_a\right)^{0.5} < 7.8 p_a \left(f_c' / p_a\right)^{0.5}} \quad \text{~~(10.8.3.5.4b-1)~~}$$

~~where:~~

~~q_u = uniaxial compressive strength of rock (ksf)~~

~~p_a = atmospheric pressure (= 2.12 ksf)~~

~~α_E = reduction factor to account for jointing in rock as provided in Table 10.8.3.5.4b-1~~

~~f'_c = concrete compressive strength (ksi)~~

~~**Table 10.8.3.5.4b-1—Estimation of α_E (O'Neill and Reese, 1999)**~~

~~E_m/E_i~~	~~α_E~~
~~1.0~~	~~1.0~~
~~0.5~~	~~0.8~~
~~0.3~~	~~0.7~~
~~0.1~~	~~0.55~~
~~0.05~~	~~0.45~~

For drilled shafts socketed into rock, unit side resistance, q_s in ksf, shall be taken as (Kulhawy et al., 2005):

$$\frac{q_S}{p_a} = C\sqrt{\frac{q_u}{p_a}} \quad (10.8.3.5.4b\text{-}1)$$

where:

C10.8.3.5.4b

~~Eq. 10.8.3.5.4b-1 applies to the case where the side of the rock socket is considered to be smooth or where the rock is drilled using a drilling slurry. Significant additional shaft resistance may be achieved if the borehole is specified to be artificially roughened by grooving. Methods to account for increased shaft resistance due to borehole roughness are provided in Section 11 of O'Neill and Reese (1999).~~

~~Eq. 10.8.3.5.4b-1 should only be used for intact rock. When the rock is highly jointed, the calculated q_s should be reduced to arrive at a final value for design. The procedure is as follows:~~

~~Step 1. Evaluate the ratio of rock mass modulus to intact rock modulus, i.e., E_m/E_i, using Table C10.4.6.5-1.~~

~~Step 2. Evaluate the reduction factor, α_E, using Table 10.8.3.5.4b-1.~~

~~Step 3. Calculate q_s according to Eq. 10.8.3.5.4b-1.~~

Eq. 10.8.3.5.4b-1 is based on regression analysis of load test data as reported by Kulhawy et al. (2005) and includes data from pervious studies by Horvath and Kenney (1979), Rowe and Armitage (1987), Kulhawy and Phoon (1993), and others. The recommended value of the regression coefficient C = 1.0 is applicable to "normal" rock sockets, defined as sockets constructed with conventional equipment and resulting in nominally clean sidewalls without resorting to special procedures or artificial roughening. Rock that is prone to smearing

p_a = atmospheric pressure taken as 2.12 ksf

C = regression coefficient taken as 1.0 for normal conditions

q_u = uniaxial compressive strength of rock (ksf)

If the uniaxial compressive strength of rock forming the sidewall of the socket exceeds the drilled shaft concrete compressive strength, the value of concrete compressive strength (f'_c) shall be substituted for q_u in Eq. 10.8.3.5.4b-1.

For fractured rock that caves and cannot be drilled without some type of artificial support, the unit side resistance shall be taken as:

$$\frac{q_s}{p_a} = 0.65\alpha_E\sqrt{\frac{q_u}{p_a}} \qquad (10.8.3.5.4b\text{-}2)$$

The joint modification factor, α_E is given in Table 10.8.3.5.4b-1 based on RQD and visual inspection of joint surfaces.

Table 10.8.3.5.4b-1—Estimation of α_E (O'Neill and Reese, 1999)

RQD (%)	Joint Modification Factor, α_E	
	Closed joints	Open or gouge-filled joints
100	1.00	0.85
70	0.85	0.55
50	0.60	0.55
30	0.50	0.50
20	0.45	0.45

or rapid deterioration upon exposure to atmospheric conditions, water, or slurry are outside the "normal" range and may require additional measures to insure reliable side resistance. Rocks exhibiting this type of behavior include clay shales and other argillaceous rocks. Rock that cannot support construction of an unsupported socket without caving is also outside the "normal" and will likely exhibit lower side resistance than given by Eq. 10.8.3.5.4b-1 with C = 1.0. For additional guidance on assessing the magnitude of C, see Brown, et al. (2010).

Shafts are sometimes constructed by supporting the hole with temporary casing or by grouting the rock ahead of the excavation. When using these construction methods, disturbance of the sidewall results in lower unit side resistances. Based on O'Neill and Reese (1999) and as discussed in Brown et al. (2010), the reduction in side resistance can be related empirically to the RQD and joint conditions.

10.8.3.5.4c—Tip Resistance

End-bearing for drilled shafts in rock may be taken as follows:

- If the rock below the base of the drilled shaft to a depth of 2.0*B* is either intact or tightly jointed, i.e., no compressible material or gouge-filled seams, and the depth of the socket is greater than 1.5*B* ~~(O'Neill and Reese, 1999)~~:

C10.8.3.5.4c

If end bearing in the rock is to be relied upon, and wet construction methods are used, bottom clean-out procedures such as airlifts should be specified to ensure removal of loose material before concrete placement.

The use of Eq. 10.8.3.5.4c-1 also requires that there are no solution cavities or voids below the base of the drilled shaft.

$$q_p = 2.5q_u \qquad (10.8.3.5.4c\text{-}1)$$

- If the rock below the base of the shaft to a depth of 2.0*B* is jointed, the joints have random orientation, and the condition of the joints can be evaluated as:

$$\cancel{q_p = \left[\sqrt{s} + \sqrt{(m\sqrt{s} + s)}\right] q_u \qquad (10.8.3.5.4c\text{-}2)}$$

~~where:~~

~~s, m = fractured rock mass parameters and are specified in Table 10.4.6.4-4~~

~~q_u = unconfined compressive strength of rock (ksf)~~

$$q_p = A + q_u\left[m_b\left(\frac{A}{q_u}\right) + s\right]^{a} \qquad (10.8.3.5.4c\text{-}2)$$

In which:

$$A = \sigma'_{vb} + q_u\left[m_b\frac{(\sigma'_{v,b})}{q_u} + s\right]^{a} \qquad (10.8.3.5.4c\text{-}3)$$

where:

σ'_{vb} = vertical effective stress at the socket bearing elevation (tip elevation)

s, *a*, and $\imath_b$ = Hoek-Brown strength parameters for the fractured rock mass determined from GSI (see Article 10.4.6.4)

q_u = uniaxial compressive strength of intact rock

Eq. 10.8.3.5.4c-1 should be used as an upper-bound limit to base resistance calculated by Eq. 10.8.2.5.4c-2, unless local experience or load tests can be used to validate higher values.

For further information see ~~O'Neill and Reese (1999)~~Brown et al. (2010).

~~Eq. 10.8.3.5.4c-2 is a lower bound solution for bearing resistance for a drilled shaft bearing on or socketed in a fractured rock mass. This method is appropriate for rock with joints that are not necessarily oriented preferentially and the joints may be open, closed, or filled with weathered material. Load testing will likely indicate higher tip resistance than that calculated using Eq. 10.8.3.5.4c-2. Resistance factors for this method have not been developed and must therefore be estimated by the designer.~~ Bearing capacity theory provides a framework for evaluation of base resistance for cases where the bearing rock can be characterized by its GSI. Eq. 10.8.3.5.4c-2 (Turner and Ramey, 2010) is a lower bound solution for bearing resistance of a drilled shaft bearing on or socketed into a fractured rock mass. Fractured rock describes a rock mass intersected by multiple sets of intersecting joints such that the strength is controlled by the overall mass response and not by failure along pre-existing structural discontinuities. This generally applies to rock that can be characterized by the descriptive terms shown in Figure 10.4.6.4-1 (*e.g.*, "blocky", "disintegrated", etc.).

10.8.3.5.4d—Combined Side and Tip Resistance

Design methods that consider the difference in shaft movement required to mobilize skin friction in rock versus what is required to mobilize end bearing~~, such as the methodology provided by O'Neill and Reese (1999),~~ shall be used to estimate axial compressive resistance of shafts embedded in rock.

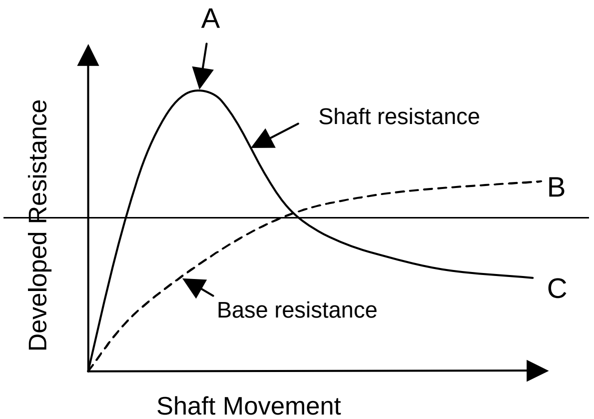
C10.8.3.5.4d

~~Typically, the axial compression load on a shaft socketed into rock is carried solely in shaft side resistance until a total shaft movement on the order of 0.4 in. occurs.~~

~~Designs which consider combined effects of side friction and end bearing of a drilled shaft in rock require that side friction resistance and end bearing resistance be evaluated at a common value of axial displacement, since maximum values of side friction and end bearing are not generally mobilized at the~~

~~same displacement.~~

~~Where combined side friction and end bearing in rock is considered, the designer needs to evaluate whether a significant reduction in side resistance will occur after the peak side resistance is mobilized. As indicated in Figure C10.8.3.5.4d-1, when the rock is brittle in shear, much shaft resistance will be lost as vertical movement increases to the value required to develop the full value of q_p. If the rock is ductile in shear, i.e., deflection softening does not occur, then the side resistance and end bearing resistance can be added together directly. If the rock is brittle, however, adding them directly may be unconservative. Load testing or laboratory shear strength testing, e.g., direct shear testing, may be used to evaluate whether the rock is brittle or ductile in shear.~~



~~**Figure C10.8.3.5.4d-1—Deflection Softening Behavior of Drilled Shafts under Compression Loading (after O'Neill and Reese, 1999).**~~

~~The method used to evaluate combined side friction and end bearing at the strength limit state requires the construction of a load-vertical deformation curve. To accomplish this, calculate the total load acting at the head of the drilled shaft, Q_{T1}, and vertical movement, w_{T1}, when the nominal shaft side resistance (Point *A* on Figure C10.8.3.5.4d-1) is mobilized. At this point, some end bearing is also mobilized. For detailed computational procedures for estimating shaft resistance in rock, considering the combination of side and tip resistance, see O'Neill and Reese (1999).~~

<u>A design decision to be addressed when using rock sockets is whether to neglect one or the other component of resistance (side or base). For example, design based on side resistance alone is sometimes assumed for cases in which the base of the drilled hole cannot be cleaned and inspected or where it is determined that large downward movement of the shaft would be required to mobilize tip resistance. However, before making a decision to omit tip resistance, careful consideration should be given to applying available methods of quality construction and inspection that can provide confidence in tip resistance. Quality construction practices can</u>

result in adequate clean-out at the base of rock sockets, including those constructed by wet methods. In many cases, the cost of quality control and assurance is offset by the economies achieved in socket design by including tip resistance. Load testing provides a means to verify tip resistance in rock.

Reasons cited for neglecting side resistance of rock sockets include (1) the possibility of strain-softening behavior of the sidewall interface (2) the possibility of degradation of material at the borehole wall in argillaceous rocks, and (3) uncertainty regarding the roughness of the sidewall. Brittle behavior along the sidewall, in which side resistance exhibits a significant decrease beyond its peak value, is not commonly observed in load tests on rock sockets. If there is reason to believe strain softening will occur, laboratory direct shear tests of the rock-concrete interface can be used to evaluate the load-deformation behavior and account for it in design. These cases would also be strong candidates for conducting field load tests. Investigating the sidewall shear behavior through laboratory or field testing is generally more cost-effective than neglecting side resistance in the design. Application of quality control and assurance through inspection is also necessary to confirm that sidewall conditions in production shafts are of the same quality as laboratory or field test conditions.

Materials that are prone to degradation at the exposed surface of the borehole and are prone to a "smooth" sidewall generally are argillaceous sedimentary rocks such as shale, claystone, and siltstone. Degradation occurs due to expansion, opening of cracks and fissures combined with groundwater seepage, and by exposure to air and/or water used for drilling. Hassan and O'Neill (1997) note that this behavior is most prevalent in cohesive IGM's and that in the most severe cases degradation results in a smear zone at the interface. Smearing may reduce load transfer significantly. As reported by Abu-Hejleh et al. (2003), both smearing and smooth sidewall conditions can be prevented in cohesive IGM's by using roughening tools during the final pass with the rock auger or by grooving tools. Careful inspection prior to concrete placement is required to confirm roughness of the sidewalls. Only when these measures cannot be confirmed would there be cause for neglecting side resistance in design.

Analytical tools for evaluating the load transfer behavior of rock socketed shafts are given in Turner (2006) and Brown et al. (2010).

10.8.3.5.5—Estimation of Drilled Shaft Resistance in Intermediate Geomaterials (IGMs)

For detailed base and side resistance estimation procedures for shafts in cohesive IGMs, the procedures provided by ~~O'Neill and Reese (1999)~~ Brown et al. (2010) should be used.

C10.8.3.5.5

See Article 10.8.2.2.3 for a definition of an IGM.

~~For convenience, since a common situation is to tip the shaft in a cohesionless IGM, the equation for tip resistance in a cohesionless IGM is provided in Article C10.8.3.5.2c.~~

10.8.3.5.6—Shaft Load Test - ***NO CHANGES – NOT SHOWN***

10.8.3.6—Shaft Group Resistance - ***NO CHANGES – NOT SHOWN***

10.8.3.7—Uplift Resistance - ***NO CHANGES – NOT SHOWN***

10.8.3.8—Nominal Horizontal Resistance of Shaft and Shaft Groups - ***NO CHANGES – NOT SHOWN***

10.8.3.9—Shaft Structural Resistance - ***NO CHANGES – NOT SHOWN***

10.8.4—Extreme Event Limit State

The provisions of Article 10.5.5.3 and 10.7.4 shall apply.

C10.8.4

See commentary to Articles 10.5.5.3 and 10.7.4.

10.9—MICROPILES – ***NO CHANGES – NOT SHOWN***

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APPENDIX A10—SEISMIC ANALYSIS AND DESIGN OF FOUNDATIONS – *NO CHANGES – NOT SHOWN*

2013 AASHTO BRIDGE COMMITTEE AGENDA ITEM: 28

SUBJECT: LRFD Bridge Design Specifications: Section 11, Article C11.6.2.3

TECHNICAL COMMITTEE: T-15 Substructures and Retaining Walls

☒ **REVISION**	☐ **ADDITION**	☐ **NEW DOCUMENT**
☒ **DESIGN SPEC**	☐ **CONSTRUCTION SPEC**	☐ **MOVABLE SPEC**
☐ **MANUAL FOR BRIDGE EVALUATION**	☐ **SEISMIC GUIDE SPEC**	☐ **BRIDGE ELEMENT INSP GUIDE**
	☐ **OTHER**	

DATE PREPARED: 12/31/12
DATE REVISED:

AGENDA ITEM:

Revise the last sentence of the 5th paragraph in Article C11.6.2.3 as follows:

With regard to selection of a resistance factor for evaluation of overall stability of walls, examples of structural elements supported by a wall that may justify the use of the 0.65 resistance factor include a bridge or pipe arch foundation, a building foundation, a pipeline, a critical utility, or another retaining wall. If the structural element is located beyond the failure surface for external stability behind the wall illustrated conceptually in Figure11.10.2-1, or if the wall does not support a structural element, a resistance factor of 0.75 may be used.

OTHER AFFECTED ARTICLES:

None

BACKGROUND:

The current language is not clear as written and could be interpreted to assume that the wall itself is a structural element on a slope requiring the use of the 0.65 resistance factor for all walls. This was not the intent of the specification.

ANTICIPATED EFFECT ON BRIDGES:

None

REFERENCES:

None

OTHER:

None

2013 AASHTO BRIDGE COMMITTEE AGENDA ITEM: 29

SUBJECT: LRFD Bridge Design Specifications: Section 11, Articles 11.10.2.3.1 & 11.10.6.2.1

TECHNICAL COMMITTEE: T-15 Substructures and Retaining Walls

☒ **REVISION** ☐ **ADDITION** ☐ **NEW DOCUMENT**

☒ **DESIGN SPEC** ☐ **CONSTRUCTION SPEC** ☐ **MOVABLE SPEC**
☐ **MANUAL FOR BRIDGE EVALUATION** ☐ **SEISMIC GUIDE SPEC** ☐ **BRIDGE ELEMENT INSP GUIDE**
☐ **OTHER**

DATE PREPARED: 12/31/12
DATE REVISED: 3/22/13

AGENDA ITEM:

<u>Item #1</u>

In Article 11.10.2.3.1, revise the 4th paragraph as follows:

For segmental concrete facing blocks, facing stability calculations shall include an evaluation of the maximum vertical spacing between reinforcement layers, the maximum allowable facing height above the uppermost reinforcement layer, inter-unit shear capacity, and resistance of the facing to bulging. The maximum spacing between reinforcement layers shall be limited to twice the width, W_u illustrated in Figure 11.10.6.4.4b-1, of the segmental concrete facing block unit or 2.7 ft, whichever is less, <u>subject to the limitations provided in Article 11.10.6.2.1</u>. The maximum facing height up to the wall surface grade above the uppermost reinforcement layer shall be limited to $1.5W_u$ illustrated in Figure 11.10.6.4.4b-1 or 24.0 in., whichever is less, provided that the facing above the uppermost reinforcement layer is demonstrated to be stable against a toppling failure through detailed calculations. The maximum depth of facing below the lowest reinforcement layer shall be limited to the width, W_u, of the proposed segmental concrete facing block unit.

<u>Item #2</u>

In Article 11.10.6.2.1, revise the second to last paragraph as follows:

A vertical spacing, S_v, greater than 2.7 ft should not be used without full scale wall data (e.g., reinforcement loads and strains, and overall deflections) that support the acceptability of larger vertical spacing<u>, except for MSE wall systems with facing units equal to or greater than 2.7 ft high with a minimum facing unit width, W_u equal to or greater than the facing unit height. For these larger facing units the maximum spacing, S_v, shall not exceed the width of the facing unit, W_u, or 3.3 ft, whichever is less</u>.

OTHER AFFECTED ARTICLES:

None

BACKGROUND:

The maximum vertical spacing, S_v, of 2.7 ft was originally based on the typical maximum spacing used in steel reinforced concrete panel faced MSE walls for which there has been a substantial amount of performance experience. However, in recent years, there have been numerous MSE walls constructed around the world with gabion basket facings in which the vertical reinforcement spacing is 40 inches (i.e., equal to the depth of the gabion) that have performed well. In addition to production walls with large vertical reinforcement spacing that have demonstrated excellent performance, both instrumented walls and numerical modeling have demonstrated that the reinforcement loads continue to vary linearly as a function of S_v for S_v values of 3.3 ft or more. Results from instrumented walls that demonstrate this are summarized in Allen, et al. (2003) and Bathurst, et al. (2008).

ANTICIPATED EFFECT ON BRIDGES:

None

REFERENCES:

Allen, T.M., Bathurst, R.J., Holtz, R.D., Walters, D.L. and Lee, W.F., 2003. "A New Working Stress Method for Prediction of Reinforcement Loads in Geosynthetic Walls", *Canadian Geotechnical Journal*, Vol. 40, No. 5, pp. 976-994.

Bathurst, R. J., Miyata, Y., Nernheim, A., and Allen, T. M., 2008, "Refinement of K-stiffness Method for geosynthetic reinforced soil walls," *Geosynthetics International*, Vol. 15, No. 4, pp. 269-295.

OTHER:

None

2013 AASHTO BRIDGE COMMITTEE AGENDA ITEM: 30

SUBJECT: LRFD Bridge Design Specifications: Section 11, Articles 11.10.5.3 & C11.10.5.3

TECHNICAL COMMITTEE: T-15 Substructures and Retaining Walls

☒ **REVISION**	☐ **ADDITION**	☐ **NEW DOCUMENT**
☒ **DESIGN SPEC**	☐ **CONSTRUCTION SPEC**	☐ **MOVABLE SPEC**
☐ **MANUAL FOR BRIDGE EVALUATION**	☐ **SEISMIC GUIDE SPEC**	☐ **BRIDGE ELEMENT INSP GUIDE**
	☐ **OTHER**	

DATE PREPARED: 12/31/12
DATE REVISED:

AGENDA ITEM:

Item #1

Revise Article 11.10.5.3 as follows:

11.10.5.3—Sliding

The provisions of Article 10.6.3.4 shall apply.

Sliding stability of MSE walls shall be evaluated at the base of the bottom of the wall facing, and, as a minimum, at the interface between the soil and reinforcement for the lowest reinforcement layer. The coefficient of sliding friction at the base of the reinforced soil mass shall be determined using the friction angle of the foundation soil, ϕ_f, or reinforced fill soil, ϕ_r.

For discontinuous reinforcements, e.g., strips, the angle of sliding friction shall be taken as the lesser of ϕ_r of the reinforced fill and ϕ_f of the foundation soil. For continuous reinforcements, e.g., grids and sheets, the angle of sliding friction shall be taken as the lesser of ϕ_r, ϕ_f, and ρ, where ρ is the soil-reinforcement interface friction angle. In the absence of specific data, a maximum friction angle, ϕ_f, of 30 degrees, ϕ_r, of 34 degrees and a maximum soil-reinforcement interface angle, ρ, of 2/3 ϕ_f or 2/3ϕ_r ~~may~~should be used.

If the lowest reinforcement layer is above the bottom of the wall facing, to check sliding at the base of the wall, the friction angle of the foundation soil, ϕ_f, or reinforced fill soil, ϕ_r, whichever is less, shall be used to assess sliding resistance. To check sliding resistance at the lowest reinforcement layer in this case, since the reinforcement is fully within the reinforced fill, the interface friction angle, ρ, should be based on the friction angle for the reinforced fill, ϕ_r.

Item #2

Revise Article C11.10.5.3 as follows:

C11.10.5.3

For relatively thick facing elements, it may be desirable to include the facing dimensions and weight in sliding and overturning calculations, i.e., use *B* in lieu of *L* as shown in Figure 11.10.5.2-1.

Testing of the foundation soil or the backfill soil should be considered so that less conservative friction angle values than the default minimums could be used to estimate sliding resistance. Test results that could be used for this purpose include laboratory soil shear strength or interface shear testing, or in-situ testing combined with correlations to soil shear strength.

OTHER AFFECTED ARTICLES:

None

BACKGROUND:

Article 11.10.5.3, as currently written, does not clearly differentiate between the cases of continuous versus discontinuous soil reinforcement for the selection of interface friction parameters, nor does it differentiate how to handle sliding on the lowest reinforcement level versus sliding at the base of the wall. The proposed revisions clarify these issues.

ANTICIPATED EFFECT ON BRIDGES:

None

REFERENCES:

None

OTHER:

None

2013 AASHTO BRIDGE COMMITTEE AGENDA ITEM: 31

SUBJECT: LRFD Bridge Design Specifications: Section 11, Articles 11.10.6.2.1 & 11.10.6.3.2

TECHNICAL COMMITTEE: T-15 Substructures and Retaining Walls

☒ **REVISION** ☒ **ADDITION** ☐ **NEW DOCUMENT**

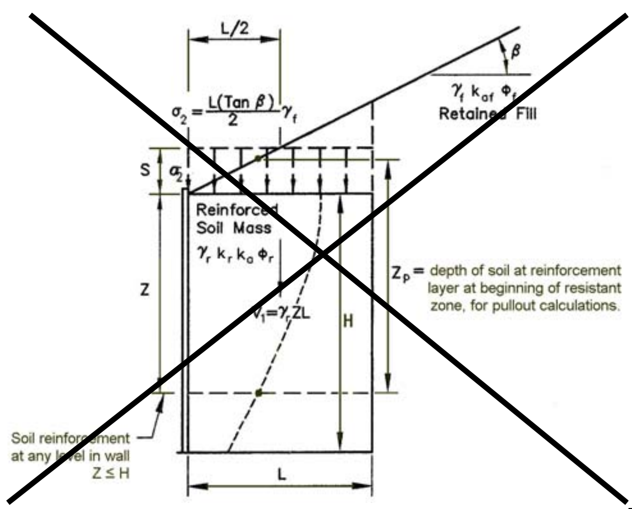
☒ **DESIGN SPEC** ☐ **CONSTRUCTION SPEC** ☐ **MOVABLE SPEC**
☐ **MANUAL FOR BRIDGE EVALUATION** ☐ **SEISMIC GUIDE SPEC** ☐ **BRIDGE ELEMENT INSP GUIDE**
☐ **OTHER**

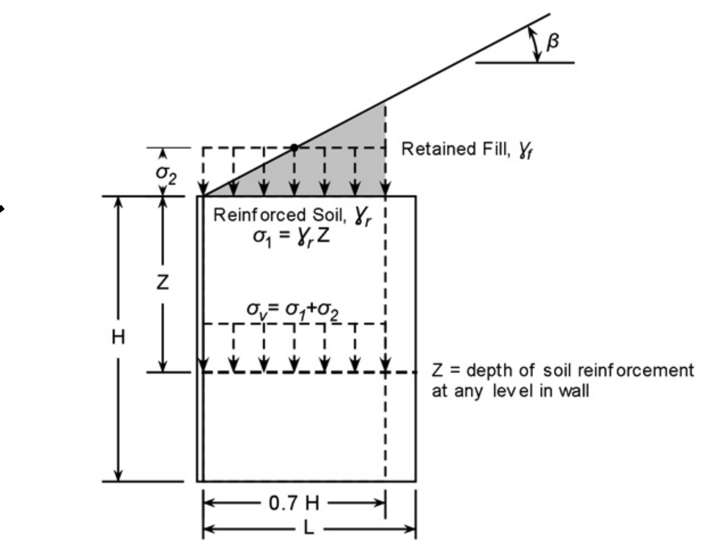
DATE PREPARED: 12/31/12
DATE REVISED:

AGENDA ITEM:

Item #1

In Article 11.10.6.2.1, replace Figure 11.10.6.2.1-2 with revised figure below that shows the 0.7H limitation of sloping surcharge used by the Simplified Method and removes items not related to calculation of vertical stress for internal stability analysis, moving pullout related items to a new figure in Article 11.10.6.3.2.





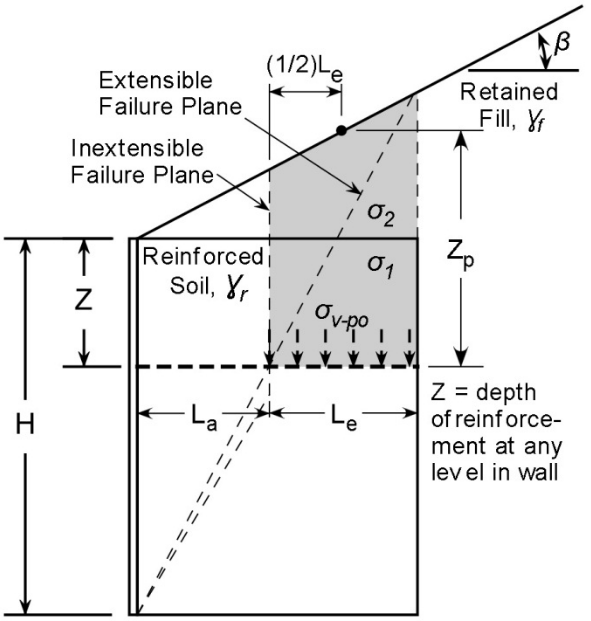
Maximum Vertical Stress:
$\sigma_1 = \gamma_r Z$
σ_2 = Average overburden pressure over 0.7H
σ_2 = (1/2) 0.7H tanβ γ_f for sloping backfill, average for broken back
$\sigma_v = \gamma_r Z + (1/2)\ 0.7H \tan\beta\ \gamma_f$

Figure 11.10.6.2.1-2—Calculation of Vertical Stress for Sloping Backslope Condition for Internal Stability Analysis

Item #2

In Article 11.10.6.3.2, add the following text and new Figure 11.10.6.3.2-1 after the list of variables for Eq. 11.10.6.3.2-1:

The vertical stress, σ_v, used to calculate pullout resistance shall be determined as shown in Figure 11.10.6.2.1-2 for the horizontal backslope condition and in Figure 11.10.6.3.2-1 for the sloping backslope condition.



Nominal Vertical Confining Pressure:

$\sigma_1 = \gamma_r Z$

$\sigma_2 = \gamma_f (Z_p - Z)$ for sloping backfill

$\sigma_{v\text{-}po} = \gamma_r Z + \gamma_f (Z_p - Z)$

$Z_p = Z + (L_a + (1/2)L_e) \tan\beta$ for sloping backfill

Figure 11.10.6.3.2-1 - Vertical confining pressure and Zp depth in resistant zone beneath sloping backfill

Then renumber the existing Figure 11.10.6.3.2-1 to Figure 11.10.6.3.2-2, and change the references to this existing figure in both the specifications and the commentary from Figure 11.10.6.3.2-1 to Figure 11.10.6.3.2-2.

OTHER AFFECTED ARTICLES:

None

BACKGROUND:

Article 11.10.6.2.1 currently does not provide limits for calculating equivalent sloping surcharge on MSE wall structure when reinforcement lengths get longer than 0.7H due to external stability considerations. FHWA-NHI-10-024 (Berg, et al., 2009) addresses this situation and limits the equivalent surcharge to the zone over 0.7H. The existing figure also attempts to show how to calculate the vertical stress needed for pullout resistance, overly complicating the figure, which has resulted in some confusion. Furthermore, the determination of Z_p in the figure is not consistent with the FHWA manual FHWA-NHI-10-024 (Berg, et al., 2009). The proposed figure shows the determination of Z_p that is consistent with that FHWA manual.

ANTICIPATED EFFECT ON BRIDGES:

None

REFERENCES:

Berg, R. R., Christopher, B. R., and Samtani, N. C., 2009, *Design of Mechanically Stabilized Earth Walls and Reinforced Slopes*, No. FHWA-NHI-10-024 Vol I and NHI-10-025 Vol II, Federal Highway Administration, 306 pp (Vol I) and 378 pp (Vol II).

OTHER:

None

2013 AASHTO BRIDGE COMMITTEE AGENDA ITEM: 32

SUBJECT: LRFD Bridge Design Specifications: Section 11, Article 11.10.6.4.2a

TECHNICAL COMMITTEE: T-15 Substructures and Retaining Walls

☒ **REVISION** ☐ **ADDITION** ☐ **NEW DOCUMENT**

☒ **DESIGN SPEC** ☐ **CONSTRUCTION SPEC** ☐ **MOVABLE SPEC**
☐ **MANUAL FOR BRIDGE EVALUATION** ☐ **SEISMIC GUIDE SPEC** ☐ **BRIDGE ELEMENT INSP GUIDE**
☐ **OTHER**

DATE PREPARED: 2/14/13
DATE REVISED: 4/17/13

AGENDA ITEM:

In Article 11.10.6.4.2a, add the following under the 2nd bullet:

- Loss of carbon steel = 0.47 mil./yr. after zinc depletion

<u>These metal loss rates shall be considered applicable to hot-dip galvanized steel. The carbon steel loss rate shall not be used for black steel, or for steel coated by any other material or process.</u>

OTHER AFFECTED ARTICLES:

None

BACKGROUND:

The current specification does not explicitly state that the loss rates apply only to galvanized steel. As a result, some designers mistakenly use these loss rates when designing with black (never galvanized) steel. The added text makes it clear that the loss rates discussed in this section are only applicable to galvanized steel and do not apply to black steel or to steel coated by any other material or process.

ANTICIPATED EFFECT ON BRIDGES:

None

REFERENCES:

None

OTHER:

None

2013 AASHTO BRIDGE COMMITTEE AGENDA ITEM: 33

SUBJECT: LRFD Bridge Design Specifications: Section 11, Article 11.10.6.4.2a

TECHNICAL COMMITTEE: T-15 Substructures and Retaining Walls

☒ **REVISION**	☐ **ADDITION**	☐ **NEW DOCUMENT**
☒ **DESIGN SPEC**	☐ **CONSTRUCTION SPEC**	☐ **MOVABLE SPEC**
☐ **MANUAL FOR BRIDGE EVALUATION**	☐ **SEISMIC GUIDE SPEC**	☐ **BRIDGE ELEMENT INSP GUIDE**
	☐ **OTHER**	

DATE PREPARED: 12/31/12
DATE REVISED:

AGENDA ITEM:

Revise the last paragraph in Article 11.10.6.4.2a as follows:

Galvanized coatings shall be a minimum of 2 oz~~.~~/ft^2 or 3.4 mils~~.~~ in thickness, applied in conformance to AASHTO ~~M 111M/M 111~~M111/M111M (ASTM A123/A~~-~~123M) ~~for strip-type reinforcements or ASTM A641 for bar mat or grid-type reinforcement~~.

OTHER AFFECTED ARTICLES:

None

BACKGROUND:

Carbon steel wire is used in the fabrication of welded wire grids and bar mats for use as MSE wall reinforcements. Galvanization occurs after fabrication. ASTM A641, *Standard Specification for Zinc–Coated (Galvanized) Carbon Steel Wire*, does not apply because the welded wire grids or bar mats are fabricated from black wire, not from galvanized wire. Galvanization of fabricated products should be in accordance with ASTM A123, *Standard Specification for Zinc (Hot-Dip Galvanized) Coatings on Iron and Steel Products*.

ANTICIPATED EFFECT ON BRIDGES:

None

REFERENCES:

ASTM A641, *Standard Specification for Zinc–Coated (Galvanized) Carbon Steel Wire*
ASTM A123, *Standard Specification for Zinc (Hot-Dip Galvanized) Coatings on Iron and Steel Products*

OTHER:

None

2013 AASHTO BRIDGE COMMITTEE AGENDA ITEM: 34

SUBJECT: LRFD Bridge Design Specifications: Section 11, Article C11.10.6.4.2a

TECHNICAL COMMITTEE: T-15 Substructures and Retaining Walls

☒ **REVISION** ☐ **ADDITION** ☐ **NEW DOCUMENT**

☒ **DESIGN SPEC** ☐ **CONSTRUCTION SPEC** ☐ **MOVABLE SPEC**
☐ **MANUAL FOR BRIDGE EVALUATION** ☐ **SEISMIC GUIDE SPEC** ☐ **BRIDGE ELEMENT INSP GUIDE**
☐ **OTHER**

DATE PREPARED: 12/31/12
DATE REVISED:

AGENDA ITEM:

Revise the 4th paragraph of Article C11.10.6.4.2a as follows:

Recommended test methods for soil chemical property determination include AASHTO T 289 ~~I~~ for pH, AASHTO T 288 ~~I~~ for resistivity, AASHTO T 291 ~~I~~ for chlorides and AASHTO T 290 ~~I~~ for sulfates. <u>AASHTO T 288 measures resistivity of a soil at various moisture contents and reports the minimum obtained resistivity. Note 6 of Method T 288-12 (Note 5 in previous editions of the test method), which describes taking the soil specimen to a slurry state, is not applicable because backfill in MSE structures cannot exist in a slurry state. Therefore, T 288 should only be taken to a water content high enough to achieve 100 percent saturation, and Note 6 should not be used.</u>

OTHER AFFECTED ARTICLES:

None

BACKGROUND:

The “I” designation after each test method identified in this paragraph is outdated, as these test procedures are no longer interim procedures. Therefore, the “I” designation has been crossed out.

Report FHWA-NHI-09-087 (*Corrosion/Degradation of Soil Reinforcements for Mechanically Stabilized Earth Walls and Reinforced Soil Slopes,* Elias, et al., 2009) states: "Resistivity should be determined under the most adverse condition (saturated state) in order to obtain a comparable resistivity that is independent of seasonal and other variations in soil-moisture content. AASHTO has adopted Method T-288 for measuring resistivity after review and analysis of a number of available methodologies. This laboratory test measures resistivity of a soil at various moisture contents up to saturation and reports the minimum obtained resistivity."

The proposed additional commentary must be added to Article C11.10.6.4.2a to obtain meaningful results from resistivity testing of MSE wall backfill. In some soils the minimum resistivity is obtained when the soil is in a slurry condition. For such soils, Note 6 of Method T 288 says to remove the soil slurry from the resistivity test box, add water, pour the slurry water back into the test box, then add only as much soil as may be needed to fill the box. This procedure is repeated as many times as needed to measure minimum resistivity, potentially resulting in a slurry. Within a MSE wall, it is not possible for the backfill to achieve a slurry state, as the wall would collapse long before such a state was achieved, even if it was possible to increase the backfill water content such that the backfill becomes a slurry. Therefore, Note 6 should not be followed. The resistivity test should end with the resistivity measured at 100% saturation to provide results applicable to MSE walls, as stated in NHI manual.

ANTICIPATED EFFECT ON BRIDGES:

None

REFERENCES:

Elias, et al., 2009, *Corrosion/Degradation of Soil Reinforcements for Mechanically Stabilized Earth Walls and Reinforced Soil Slopes*, Publication No. FHWA-NHI-09-087, Washington, DC.

OTHER:

None

2013 AASHTO BRIDGE COMMITTEE AGENDA ITEM: 35

SUBJECT: LRFD Bridge Design Specifications: Section 11, Article C11.10.11

TECHNICAL COMMITTEE: T-15 Substructures and Retaining Walls

☐ **REVISION** ☒ **ADDITION** ☐ **NEW DOCUMENT**

☒ **DESIGN SPEC** ☐ **CONSTRUCTION SPEC** ☐ **MOVABLE SPEC**
☐ **MANUAL FOR BRIDGE EVALUATION** ☐ **SEISMIC GUIDE SPEC** ☐ **BRIDGE ELEMENT INSP GUIDE**
☐ **OTHER**

DATE PREPARED: 12/31/12
DATE REVISED: 4/12/13

AGENDA ITEM:

Add the following at the beginning of Article C11.10.11:

For analysis of the spread footing on top of the reinforced soil zone, the following values of bearing resistance of the reinforced soil zone may be used (Berg, et al., 2009):

- For service limit state, bearing resistance = 4 ksf to limit the vertical movement to less than approximately 0.5 in. within the reinforced soil mass
- For strength limit state, factored bearing resistance = 7 ksf

OTHER AFFECTED ARTICLES:

None

BACKGROUND:

Design tools that can be used to predict vertical deformations of footings supported on MSE walls are not available, and there is no guidance related to the bearing resistance values that can be safely used to provide good performance. The recommended values are from the FHWA MSE wall manual and are consistent with the current successful practice.

ANTICIPATED EFFECT ON BRIDGES:

None

REFERENCES:

Berg, R. R., Christopher, B. R., and Samtani, N. C., 2009, *Design of Mechanically Stabilized Earth Walls and Reinforced Slopes*, No. FHWA-NHI-10-024 Vol I and NHI-10-025 Vol II, Federal Highway Administration, 306 pp (Vol I) and 378 pp (Vol II).

OTHER:

None

2013 AASHTO BRIDGE COMMITTEE AGENDA ITEM: 36

SUBJECT: LRFD Bridge Design Specifications: Section 11, Article C11.10.11 & Figure 11.10.10.1-2

TECHNICAL COMMITTEE: T-15 Substructures and Retaining Walls

☒ **REVISION**	☐ **ADDITION**	☐ **NEW DOCUMENT**
☒ **DESIGN SPEC**	☐ **CONSTRUCTION SPEC**	☐ **MOVABLE SPEC**
☐ **MANUAL FOR BRIDGE EVALUATION**	☐ **SEISMIC GUIDE SPEC**	☐ **BRIDGE ELEMENT INSP GUIDE**
	☐ **OTHER**	

DATE PREPARED: 12/31/12
DATE REVISED:

AGENDA ITEM:

Item #1

In Article C11.10.11, revise the 1st existing paragraph as follows.

The minimum length of reinforcement, based on experience, has been ~~the greater of 22.0 ft or~~ 0.6 (H + d) + 6.5 ft. The length of reinforcement should be constant throughout the height to limit differential settlements across the reinforced zone. Differential settlements could overstress the reinforcements.

Item #2

Revise Figure 11.10.10.1-2 as follows:

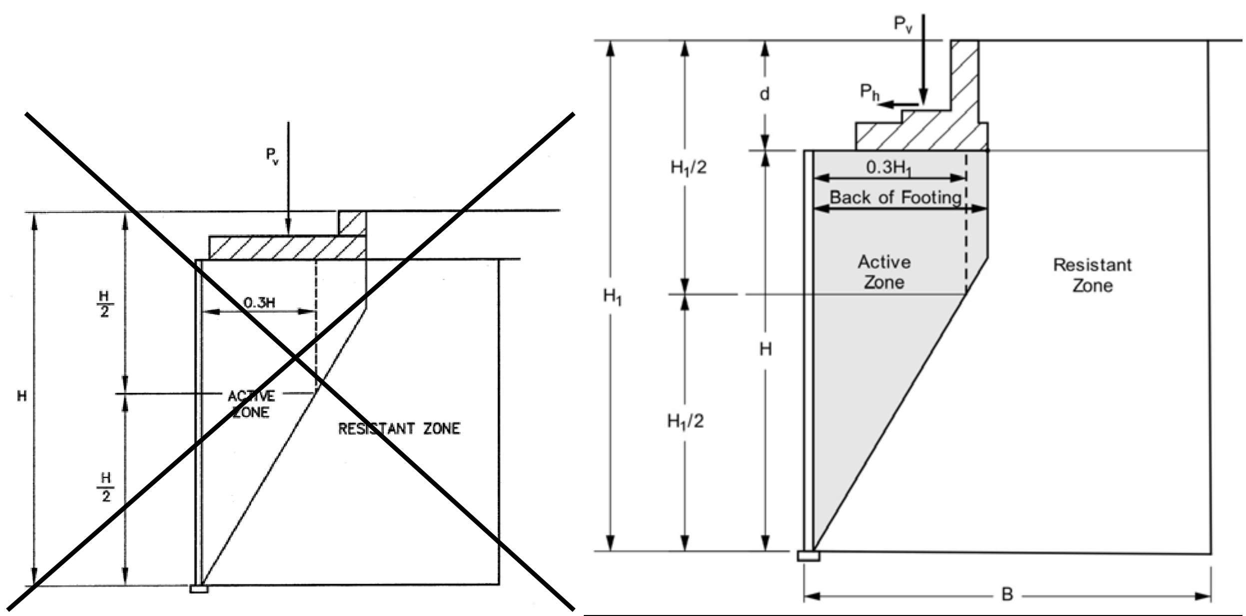


Figure 11.10.10.1-2—Location of Maximum Tensile Force Line in Case of Large Surcharge Slabs (Inextensible Reinforcements)

OTHER AFFECTED ARTICLES:

None

BACKGROUND:

The minimum 22 ft reinforcement length requirement is not a technical requirement but has remained in this article because of tradition, and furthermore for relatively short abutment walls can result in excessive reinforcement length. There is no technical need for this minimum requirement since the normal design practice addresses the reinforcement length required through standard analysis and design, including:

- Sliding, bearing, eccentricity,
- Global and compound stability,
- Checking for the length required to be stable against pullout failure, and
- Making sure that the minimum reinforcement length is equal to or greater than the experience based minimum length of 0.6 (H + d) + 6.5 ft.

Internal and external stability requirements including evaluation of global and compound stability are adequate to assess the minimum reinforcement length required for an MSE abutment.

Regarding Figure 11.10.10.1-2, H and D are not correctly defined in this figure, which has caused some confusion on how to apply the 0.6 (H + d) + 6.5 ft criterion. The revised figure corrects this problem.

ANTICIPATED EFFECT ON BRIDGES:

None

REFERENCES:

None

OTHER:

None

2013 AASHTO BRIDGE COMMITTEE AGENDA ITEM: 37

SUBJECT: LRFD Bridge Design Specifications: Section 11, Article 11.11.1

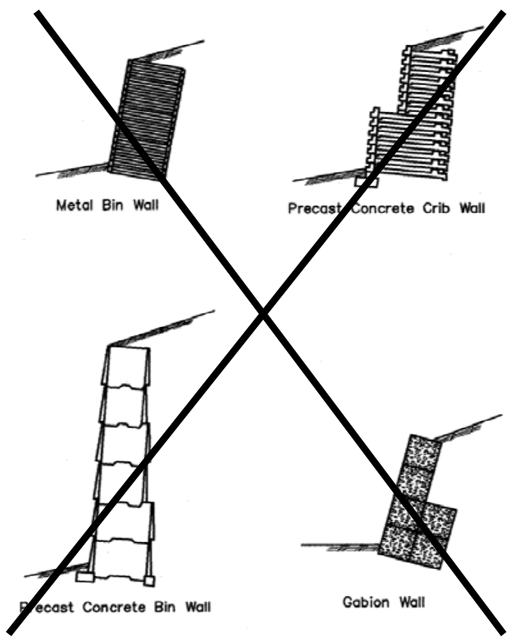
TECHNICAL COMMITTEE: T-15 Substructures and Retaining Walls

☒ **REVISION**	☐ **ADDITION**	☐ **NEW DOCUMENT**
☒ **DESIGN SPEC**	☐ **CONSTRUCTION SPEC**	☐ **MOVABLE SPEC**
☐ **MANUAL FOR BRIDGE EVALUATION**	☐ **SEISMIC GUIDE SPEC**	☐ **BRIDGE ELEMENT INSP GUIDE**
	☐ **OTHER**	

DATE PREPARED: 12/31/12
DATE REVISED:

AGENDA ITEM:

In Article 11.11.1, update Figure C11.11.1-1 to include other prefabricated modular gravity wall system illustrations currently in use and included in the C11.11.1 text, as shown:



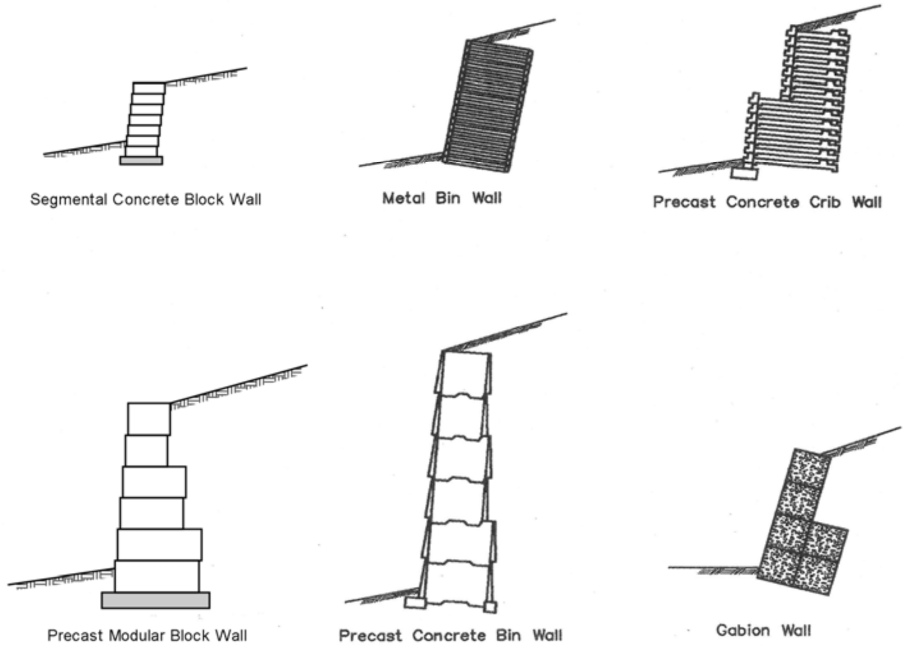


Figure C11.11.1-1—Typical Prefabricated Modular Gravity Walls

OTHER AFFECTED ARTICLES:

None

BACKGROUND:

Prefabricated modular wall figure is out of date and does not show gravity wall systems currently being used and referenced in the article text. Figure should be updated as shown to be complete.

ANTICIPATED EFFECT ON BRIDGES:

None

REFERENCES:

None

OTHER:

None

2013 AASHTO BRIDGE COMMITTEE AGENDA ITEM: 38

SUBJECT: D1.5 Bridge Welding Code: Section 2, Figure 2.4

TECHNICAL COMMITTEE: T-17 Welding

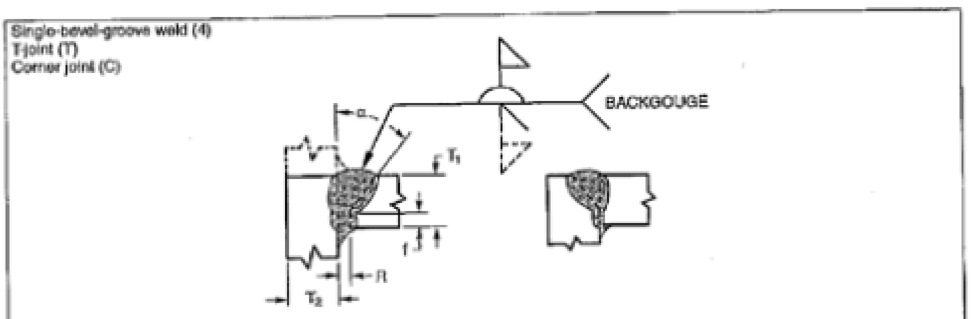
☒ **REVISION** ☐ **ADDITION** ☐ **NEW DOCUMENT**

☐ **DESIGN SPEC** ☐ **CONSTRUCTION SPEC** ☐ **MOVABLE SPEC**
☐ **MANUAL FOR BRIDGE EVALUATION** ☐ **SEISMIC GUIDE SPEC** ☐ **BRIDGE ELEMENT INSP GUIDE**
☒ **OTHER Bridge Welding Code**

DATE PREPARED: 1/14/13
DATE REVISED:

AGENDA ITEM:

Change Figure 2.4 as follows (two figures, English and Metric)

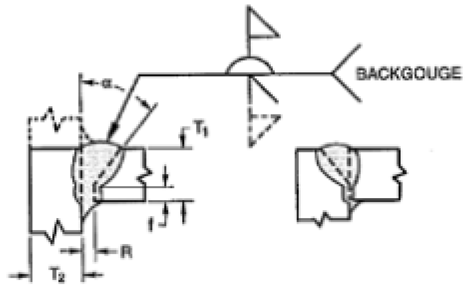


Welding Process	Joint Designation	Base Metal Thickness (U = unlimited) T1	T2	Groove Preparation: Root Opening Root Face Groove Angle	Tolerances: As Detailed (see 2.12.1)	As Fit-Up (see 3.3.4)	Allowed Welding Positions	Gas Shielding for FCAW	Notes
SMAW	TC-U4b	U	U	R = 0 to 1/8 f = 0 to 1/8 α = 45°	+1/16, −0 +1/16, −0 +10°, −0°	+1/16, −1/8 Not limited 10°, −5°	All	—	a, c, f, i
GMAW FCAW	TC-U4b-GF	U	U				All	Not required	c, f, i
SAW	TC-U4b-S	U	U	R = 0 f = ~~1/8 max~~ α = 60°	±0 +0, −1/8 +10°, −0°	+1/4, −0 ±1/16 10°, −5°	F	—	c, f, i

Figure 2.4 (Continued)—Details of Welded Joints for CJP Groove Welds (see 2.12.1) (Dimensions in Inches)

¼" max

Single-bevel-groove weld (4)
T-joint (T)
Corner joint (C)



Welding Process	Joint Designation	Base Metal Thickness (U = unlimited)		Groove Preparation			Allowed Welding Positions	Gas Shielding for FCAW	Notes
				Root Opening Root Face Groove Angle	Tolerances				
		T_1	T_2		As Detailed (see 2.12.1)	As Fit-Up (see 3.3.4)			
SMAW	TC-U4b	U	U	R = 0 to 3 f = 0 to 3 α = 45°	+2, −0 +2, −0 +10°, −0°	+2, −3 Not limited 10°, −5°	All	—	a, c, f, l
GMAW FCAW	TC-U4b-GF	U	U				All	Not required	c, f, l
SAW	TC-U4b-S	U	U	R = 0 f = ~~3 max.~~ α = 60°	±0 +0, −3 +10°, −0°	+6, −0 ±2 10°, −5°	F	—	c, f, l

Figure 2.4 (Continued)—Details of Welded Joints for CJP Groove Welds (see 2.12.1) (Dimensions in Millimeters)

6 max.

OTHER AFFECTED ARTICLES:

None

BACKGROUND:

D1.1 and D1.5 have different minimum landing (f) for these joints when using SAW. D1.1 allows a maximum ¼” landing, but D1.5 only allows 1/8”. The allowance for more landing will provide more flexibility for protecting against blow-through and for balancing passes and distortion.

ANTICIPATED EFFECT ON BRIDGES:

None

REFERENCES:

None

OTHER:

None

2013 AASHTO BRIDGE COMMITTEE AGENDA ITEM: 39

SUBJECT: D1.5 Bridge Welding Code: Section 3, Articles C3.13.1 & C3.13.2.1

TECHNICAL COMMITTEE: T-17 Welding

☒ **REVISION** ☒ **ADDITION** ☐ **NEW DOCUMENT**

☐ **DESIGN SPEC** ☐ **CONSTRUCTION SPEC** ☐ **MOVABLE SPEC**
☐ **MANUAL FOR BRIDGE EVALUATION** ☐ **SEISMIC GUIDE SPEC** ☐ **BRIDGE ELEMENT INSP GUIDE**
☒ **OTHER Bridge Welding Code**

DATE PREPARED: 1/14/13
DATE REVISED:

AGENDA ITEM:

Item #1

Add the following paragraph to the end of Article C3.13.1:

Steel backing acts as a part of the weld and may, to a limited extent, influence the chemistry and mechanical properties of the weld. AASHTO M270M/M270 (ASTM A709/A709M) Grade 250 [36] steel bars may be used as backing for almost all welds. Toughness is not specified for backing not larger than 10 mm by 30 mm [3/8 in by 1-1/4 in] because small backing should not affect the fracture characteristics of the member.

Item #2

Delete Article C3.13.2.1

OTHER AFFECTED ARTICLES:

None

BACKGROUND:

Existing Article C-3.13.2.1 is inconsistent with itself and the code. In the 2nd sentence it says that Gr. 36 bars may be used as backing for almost all welds, but it does not say for backing that is to be removed for almost all welds. Then in the 3rd sentence it says that backing that stays in place has to meet the requirements of the weaker of the materials being joined, which is inconsistent with the previous sentence and with 3.13.1(1), which gives no removal-dependent restriction on backing material selection. Finally, the existing penultimate sentence is not accurate; it states that toughness is not specified for backing when it should say that toughness is not specified for backing within this size restriction.

ANTICIPATED EFFECT ON BRIDGES:

None

REFERENCES:

None

OTHER:

None

2013 AASHTO BRIDGE COMMITTEE AGENDA ITEM: 40

SUBJECT: The Manual for Bridge Evaluation: Section 1, Various Articles (T18-1)

TECHNICAL COMMITTEE: T-18 Bridge Manangement, Evaluation and Rehabilitation

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☒ **REVISION** ☒ **ADDITION** ☐ **NEW DOCUMENT**

☐ **DESIGN SPEC** ☐ **CONSTRUCTION SPEC** ☐ **MOVABLE SPEC**
☒ **MANUAL FOR BRIDGE EVALUATION** ☐ **SEISMIC GUIDE SPEC** ☐ **BRIDGE ELEMENT INSP GUIDE**
☐ **OTHER**

DATE PREPARED: 1/8/13
DATE REVISED:

AGENDA ITEM:

Item #1

Revise Section 1: Introduction Table of Contents as follows:

Item #2

Revise Article 1.1—PURPOSE as follows:

~~This Manual serves as a standard and provides uniformity in the procedures and policies for determining the physical condition, maintenance needs, and load capacity of the nation's highway bridges.~~ The purpose of the Manual for Bridge Evaluation (MBE) is to serve as a resource for use in developing specific policy and procedures for the inspection and evaluation of existing highway bridges. The MBE also includes the nationally recognized guidance for the load rating of highway bridges.

The National Bridge Inspection Standards (NBIS) as found in the Code of Federal Regulations (23 CFR 650 Subpart C) defines the regulations for the inspection and evaluation of the nation's bridges.

The MBE is incorporated by reference in the CFR (23 CFR 650 Subpart C) to be used along with other reference documents such as the American Association of State Highway and Transportation Officials (AASHTO) Guide Manual for Bridge Element Inspection, the Federal Highway Administration's (FHWA) Bridge Inspector's Reference Manual (commonly known as the BIRM), and the latest National Bridge Inventory (NBI) coding guidance document for the inspection and evaluation of the nation's bridges.

The National Bridge Inspection Standards have evolved and been improved over the years since their creation in the early 1970's.

The MBE has also evolved and been revised and improved to reflect best practices determined by research, State Departments of Transportation and others. In the future as improved practices and research are developed, the MBE will reflect those improvements.

Throughout this Manual there are subsections titled in part " Provisions to support the NBIS Requirements". These subsections were developed to provide specific guidance and best practices which are considered to be required under the regulations.

Item #3

Delete Article C1.1

Item #4

Revise Article 1.2—SCOPE as follows:

~~This Manual has been developed to assist Bridge Owners by establishing inspection procedures and evaluation practices that meet the National Bridge Inspection Standards (NBIS). The Manual has been divided into eight Sections, with each Section representing a distinct phase of an overall bridge inspection and evaluation program.~~

~~Section 1 contains introductory and background information on the maintenance inspection of bridges as well as definitions of general interest terms. Key components of a comprehensive bridge file are defined in Section 2. The record of each bridge in the file provides the foundation against which changes in physical condition can be measured. Changes in condition are determined by field inspections. A bridge management system is an effective tool in allocating limited resources to bridge related activities. An overview of bridge management systems is included in Section 3. The types and frequency of field inspections are discussed in Section 4, as are specific inspection techniques and requirements. Conditions at a bridge site or the absence of information from original construction may warrant more elaborate material tests, and various testing methods are discussed in Section 5. Section 6 discusses the load rating of bridges and includes the Load and Resistance Factor method, the Load Factor method and the Allowable Stress method. No preference is placed on any rating method. The evaluation of existing bridges for fatigue is discussed in Section 7. Field load testing is a means of supplementing analytical procedures in determining the live-load capacity of a bridge and for improving the confidence in the assumptions used in modeling the bridge. Load test procedures are described in Section 8.~~

~~The successful application of this Manual is directly related to the organizational structure established by the Bridge Owner. Such a structure should be both effective and responsive so that the unique characteristics and special problems of individual bridges are considered in developing an appropriate inspection plan and load capacity determination.~~ <u>The Manual has been divided into eight Sections, with each Section representing a distinct phase of an overall bridge inspection and evaluation program.</u>

- <u>Section 1 contains the purpose, scope, applicability, inspection and evaluation quality measures, and definition of general interest terms.</u>
- <u>Section 2 contains the provisions for proper documentation to be included in a bridge file. The bridge file associated with each bridge provides the foundation against which changes in physical condition can be compared</u>
- <u>Section 3 provides an overview of bridge management systems and their key elements.</u>
- <u>Section 4 contains the types and frequency of field inspections, as well as specific inspection techniques and procedures.</u>
- <u>Section 5 contains various inspection and evaluation testing methods. Conditions at a bridge site or the absence of information from original construction may warrant more elaborate material tests to determine properties for evaluation.</u>
- <u>Section 6 is the nationally recognized specification for the load rating of bridges and includes the Load and Resistance Factor method, the Load Factor method and the Allowable Stress method.</u>
- <u>Section 7 contains the provisions for the evaluation of existing bridges for fatigue.</u>
  <u>Section 8 contains the field performed load test procedures. Field load testing is a means of supplementing analytical procedures in determining the live-load capacity of a bridge and for improving the confidence in the assumptions</u>

<u>The successful application of this Manual is directly related to the organizational structure established by the Bridge Inspection Program Manager. Such a structure should be both effective and responsive so that the unique characteristics and special problems of individual bridges are considered in developing an appropriate inspection plan and load capacity determination.</u>

Item #5

Delete Article C1.2

Item #6

Revise Article 1.3—APPLICABILITY as follows:

~~The provisions of this Manual apply to all highway structures which qualify as bridges in accordance with the AASHTO definition for a bridge (see Article 1.5). These provisions may be applied to smaller structures which do not qualify as bridges.~~ The provisions of this Manual apply to all highway structures which qualify as bridges in accordance with the AASHTO definition for a bridge (see Article 1.5). These provisions may be applied to smaller structures which do not qualify as bridges at the discretion of the Bridge Inspection Program Manager.

Federal regulations entitled the *National Bridge Inspection Standards* (NBIS) have been promulgated which establish minimum requirements for inspection programs and minimum qualifications for bridge inspection personnel. The NBIS apply to all highway bridges on public roads which are more than 20 ft in length.

Where conflicts or inconsistencies exist between this manual and the federal requirements specified in the NBIS, the FHWA coding guidance, or the Bridge Inspectors Reference Manual (BIRM), the NBIS, BIRM and FHWA coding guidance shall govern.

Item #7

Delete Article C1.3

Item #8

Revise Article 1.4—QUALITY MEASURES as follows:

Retitle the article Section 1.4~~—QUALITY MEASURES~~QUALITY

~~To maintain the accuracy and consistency of inspections and load ratings, Bridge Owners should implement appropriate quality control and quality assurance measures. Typical quality control procedures include the use of checklists to ensure uniformity and completeness, the review of reports and computations by a person other than the originating individual, and the periodic field review of inspection teams and their work. Quality assurance measures include the overall review of the inspection and rating program to ascertain that the results meet or exceed the standards established by the Owner.~~ To maintain the accuracy and consistency of inspections and load ratings, bridge inspection programs need to have appropriate quality control (QC) and quality assurance (QA) measures in place. Quality control procedures are intended to maintain the quality of the bridge inspections, bridge data, scour evaluations, and load ratings, and are usually performed continuously within the bridge inspection teams or units performing these functions. Quality control procedures can vary depending on the structural and scour conditions of a bridge with increased level of review commensurate with increased deterioration of bridge conditions. Quality assurance procedures are used to verify the adequacy of the quality control procedures to meet or exceed the standards established by the Bridge Inspection Program Manager. Quality assurance procedures are usually performed independent of the bridge inspection and load rating teams on a sample of their work.

Item #9

Delete Article C1.4

Item #10

Add new Article 1.4.1—PROVISIONS TO SUPPORT THE NBIS REQUIREMENTS as follows:

1.4.1—PROVISIONS TO SUPPORT THE NBIS REQUIREMENTS

A quality control and quality assurance (QC/QA) program is to include periodic field review of inspection teams, periodic bridge inspection refresher training for program managers and team leaders, QC/QA measures for inventory data, and independent review of inspection reports and computations. The bridge inspection Program Manager is responsible to develop a QC/QA program that generally conforms to the provisions of Section 1.4. Specific details are to be determined by the Bridge Inspection Program Manager.

Item #11

Add new Article 1.4.2—QC/QA PROCEDURES as follows:

1.4.2—QC/QA PROCEDURES

Typical quality procedures may include the use of checklists to ensure uniformity and completeness, the review of reports and computations by a person other than the originating individual, and the periodic field review of inspection teams and their work. The documented quality control plan may include:

- Defined quality control roles and responsibilities;
- Qualifications for the Program Manager, bridge inspection personnel, and load rating personnel, including:
    - Education
    - Certification or registration
    - Training;
    - Years and type of experience; and
- Procedures for review and validation of inspection reports and data;
- Procedures for documenting important bridge inspection information
- Procedures for review validation of load rating and scour calculations and data; and
- Procedures for identification and resolution of data issues, including errors, omissions, compatibility between items, changes, or any combination thereof.

Quality assurance measures include the overall review of the inspection and rating program to ascertain that the results meet or exceed the standards established by the Bridge Inspection Program Manager. The documented quality assurance plan may include:

- Defined quality assurance roles and responsibilities;
- Frequency parameters for review of districts or units and bridges;
- Procedures and sampling parameters for selecting bridges to conduct independent review and check of results, including:
    - Condition rating of elements or change in condition rating
    - Load rating and scour evaluations
    - Posting status
    - Deficiency status
    - Critical findings and the status of any follow-up action, and
    - Location of bridge

Quality assurance measures provide a validation that QC practices are resulting in accurate and thorough inspections, complete bridge files, accurate and complete load ratings and scour evaluations; and qualified inspectors and load raters. Results from QA reviews are used by the Bridge Inspection Program Manager to maintain the quality of the program and make improvements where needed.

Item #12

In Article 1.5—DEFINITIONS AND TERMINOLOGY add the following:

Scour Critical Bridge—A scour critical bridge is one whose foundation(s) has been determined to be unstable for the predicted scour conditions.

OTHER AFFECTED ARTICLES:

None

BACKGROUND:

The AASHTO Manual for Bridge Evaluation (MBE) and its predecessors have provided bridge owners with guidance on the practice of the inspection and load rating of bridges. In 2004 the AASHTO Manual for Bridge Evaluation was incorporated into the Code of Federal Regulations (CFR) in regards to the National Bridge Inspection Standards (NBIS) by reference.

In 2010 the FHWA developed quality assurance criteria to ensure States compliance with the CFR, referred to as the “23 Metrics”.

The proposed changes to the Manual for Bridge Evaluation in this ballot item are an effort to clarify for bridge owners and the FHWA which bridge inspection and load rating practices information contained within the MBE is applicable in the CFR.

ANTICIPATED EFFECT ON BRIDGES:

The changes should improve the understanding of the practice of bridge inspection and load rating.

REFERENCES:

None

OTHER:

None

2013 AASHTO BRIDGE COMMITTEE AGENDA ITEM: 41

SUBJECT: The Manual for Bridge Evaluation: Section 1, Article 1.6; Section 6, Articles C6A.4 & C6A.6; Part B, Articles C6B.5.2.1& C6B.5.3.1; and Appendix L6B (T18-3)

TECHNICAL COMMITTEE: T-18 Bridge Management, Evaluation and Rehabilitation/ T-14 Steel

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☒ **REVISION** ☒ **ADDITION** ☐ **NEW DOCUMENT**

☐ **DESIGN SPEC** ☐ **CONSTRUCTION SPEC** ☐ **MOVABLE SPEC**
☒ **MANUAL FOR BRIDGE EVALUATION** ☐ **SEISMIC GUIDE SPEC** ☐ **COASTAL GUIDE SPEC**
☐ **OTHER**

DATE PREPARED: 1/11/13
DATE REVISED: 4/15/13

AGENDA ITEM:

Make the revisions to the indicated articles of the Manual for Bridge Evaluation shown in Attachment A.

OTHER AFFECTED ARTICLES:

None

BACKGROUND:

Following the I-35W Bridge collapse investigation, the National Transportation Safety Board (NTSB) made five recommendations to the Federal Highway Administration (FHWA) and AASHTO. One of these recommendations was to require bridge owners to include main truss member gusset plates as part of the load rating process for these bridges.

To assist the states with this process, FHWA issued a Guidance document in February 2009. This document required, at a minimum, for main truss member gusset plates to be evaluated for five limit states using either the Load Factor Rating (LFR) or Load and Resistance Factor Rating (LRFR) philosophies.

The Guidance document was based on existing provisions in the AASHTO LRFD Bridge Design Specifications and the older AASHTO Standard Specifications for Highway Bridges along with engineering judgment. The FHWA Guidance document was thought to yield conservative gusset plate ratings. As States began to evaluate their inventory with the Guidance document, a need for more direction on some checks was identified, while some facets of other checks were thought to be too conservative. This was the case particularly for the shear reduction factor (Ω) associated with the shear yielding check, and the K-factor selection for use in the column analogy compressive buckling resistance check.

To address these concerns, FHWA initiated a research project collaboratively with the AASHTO-sponsored National Cooperative Highway Research Program (NCHRP) to evaluate the shear, tensile and compressive resistance of gusset plates at the strength limit state (NCHRP Project 12-84). The project tested 12 full-scale experimental gusset plate connections, and used finite element analysis to explore a variety of geometric parameters that could not be experimentally investigated. The outcome of this project resulted in these proposed revisions to the AASHTO Manual of Bridge Evaluation. A companion item proposes similar revisions to the AASHTO LRFD Bridge Design Specifications based on the results of this research. It should be noted that a decision was made to ensure that the LRFR and LFR gusset plate rating specifications in the MBE are reasonably self-sufficient and do not refer back to the LRFD Bridge Design Specifications and the Standard Specifications to a significant extent for determining the factored resistance of the gusset plate and its connections.

A second companion item proposes an example for inclusion in the MBE Appendix A: Illustrative Examples that will illustrate LRFR and LFR of main truss member gusset plates according to these proposed revisions. It is envisioned that the 2009 FHWA Guidance document for the load rating of these gusset plates will not be maintained in the future, and that the proposed provisions contained herein would supersede the Guidance document.

Differences in the specified resistance factors for main truss member gusset plates shown in this ballot item and the companion item recommending changes to the AASHTO LRFD Bridge Design Specifications will be noted. There was vigorous debate between the NCHRP Project 12-84 research team and the Project Panel, along with the members of the T-18 and T-14 Technical Committees, regarding this issue. The primary issue of contention came down to the reliability indices and whether or not connections should be expected to have a higher level of reliability than members. The existing AASHTO LRFD Bridge Design Specifications calibrated the specification to provide a reliability index of 3.5 for member design. The research team contended that many of the primary failure modes of gusset plates do not give any warning of impending failure and should be calibrated to a higher level of reliability, as is currently done in the AISC LRFD Specification for Structural Steel Buildings. Therefore, the initial calibration performed by the research team established resistance factors for main truss member gusset plates that provided a reliability index of 4.5. The NCHRP Panel performed some spot checks using these more conservative resistance factors and found many instances of bridges that would have to be posted based on these initial recommendations. As a result, an agreement was made between the research team and the Panel to accept the higher reliability index of 4.5 for design because in design, plate thicknesses can be more easily increased. However, in rating, it is obviously not as convenient to increase plate thicknesses; thus, there was consensus to use the existing reliability index of 3.5 for rating (at the Inventory level). This was the primary driver behind some of the differences in the resistance factors that are observed between the two specifications. Additionally, the NCHRP project was not able to collect sufficient data for all possible gusset-plate modes of failure to justify a difference in some of the factors that are provided in the two specifications. Furthermore, the resistance factors reduce as the unfactored dead-to-live (DL/LL) ratio increases. The resistance factors were selected at different DL/LL ratios in the two specifications to again, reflect the inherent differences between design and load rating. For design, resistance factors were established at a DL/LL of 6.0 meaning the design will most always be conservative, and gusset plates for trusses with DL/LL ratios less than 6.0 will be overdesigned; a result that is more easily tolerated in design. To carry this same philosophy over to the load rating specification would discount the ratings of more lightly loaded trusses. Therefore, the panel was quite adamant that resistance factors be established at lower DL/LL ratios. Thus, for load rating, the specified resistance factors were established at a DL/LL of 1.0. However to ensure that the gusset plates in more heavily loaded trusses will still be appropriately rated, an additional reduction factor must be applied in the load rating to compensate for DL/LL ratios in excess of 1.0.

ANTICIPATED EFFECT ON BRIDGES:

Assuming that most bridge owners have rated their gusset plate inventory using the existing FHWA Guidance document, the following summarizes some of the more important differences between the proposed MBE LRFR provisions and the LRFR provisions provided in the FHWA Guidance document:

1. In rating for shear yielding, the Ω factor is 0.88 and ϕ_{vy}=1.00 in the proposed provisions for a total reduction factor of 0.88 applied to the average shear stress. In the FHWA Guidance document, the Engineer had the ability to choose Ω=0.74 or Ω=1.00 in conjunction with a ϕ_{vy}=0.95 for a total reduction factor of 0.70 or 0.95 applied to the average shear stress. Therefore, if Ω=0.74 was assumed originally, the proposed specifications will result in an ~25% increase in the rating for shear yielding. If Ω=1.00 was assumed originally, the proposed specifications will result in an ~8% decrease in the rating for shear yielding. No changes are made to the rating procedures for shear rupture in the proposed provisions.
2. In the calculation of the rating for compression resistance, higher or lower ratings will be obtained using the proposed provisions over those obtained using the FHWA Guidance document depending on the assumptions that were made when rating with the FHWA Guidance document. The FHWA Guidance document recommended using an equivalent column length, which was the average of three different lengths along the Whitmore plane, referred to as L_{avg}. This length and an assumed column length factor were used to calculate a column slenderness parameter, λ_{avg}, which in turn was used to calculate the critical buckling stress of the idealized column. The new rating provisions will certainly produce less

favorable ratings over the FHWA Guidance document when $\lambda_{avg} < \sim 1.0$. This is because the partial plane shear yield criterion that is instituted in the proposed provisions will control for these very compact gusset plates, and this criterion was not checked in the FHWA Guidance document. On the contrary, if $\lambda_{avg} > \sim 1.5$ the new provisions will produce more favorable ratings over the FHWA Guidance document because of the new effective column length factor of 0.5, which is much lower than the K-factor that was likely employed when using the FHWA Guidance document. The ratings with the new provisions and the FHWA Guidance document are expected to be similar when $1.0 < \lambda_{avg} < 1.5$. No changes are made to the rating procedures for tension yielding and net section fracture on the Whitmore plane.

3. The new proposed rating specifications use a resistance factor of 1.00 for block shear rupture, whereas in the FHWA Guidance document (and AASHTO LRFD Bridge Design Specifications), this factor is 0.80. The factor of 0.80 was found to provide a reliability index of 4.5 whereas the decision was made to use a reliability index of 3.5 in the MBE for Inventory level assessments; therefore, the resistance factor was increased accordingly. This should result in a 25% increase in block shear rupture ratings over those obtained using the FHWA Guidance document.
4. The FHWA Guidance document recommended using a Whitmore section analysis in the rating of tension and compression chord splices. The real stress patterns in the analysis models did not correlate well with this assumption and this method is no longer recommended. Therefore, if this particular check controlled using the FHWA Guidance document, it will no longer apply under the proposed provisions. A new chord splice check is introduced within the proposed specifications that better accounts for the variability of gusset plate geometries versus the Whitmore section approach. The effect of this new approach on the load rating for these splices is difficult to ascertain as its effect will be specific to each joint, which typically has a unique geometry.
5. Overall, the proposed MBE rating specifications reflect a better understanding of gusset plate behavior than the provisions provided in the 2009 FHWA Guidance document and should result in a more uniform reliability of gusset plate ratings.

NCHRP Project 12-84 primarily focused on the development of an LRFR approach to gusset plate rating. This required the derivation of resistance factors to provide a target reliability index of 3.5 for Inventory level assessments. The translation of these resistance factors to an LFR philosophy is difficult because the live-load models are different (HS20 versus HL93), and the project did not perform a comprehensive live-load study for both short and long span trusses. As a result, the LRFR resistance factors cannot merely be carried over to LFR. If a nominal resistance equation utilizes a reduction factor specific to that resistance behavior (for instance, the reduction factor Ω for shear yielding), that factor was carried over, but most of the resistance factors were made unity for LFR. If no better information could be derived from the research, the same resistance factor published in the FHWA Guidance document was repeated in the proposed specifications. As a result, some resistance factors will be different between LRFR and LFR in the proposed provisions. The following summarizes some of the more important differences between the proposed MBE LFR provisions and the LFR provisions provided in the FHWA Guidance document:

6. In the rating for shear yielding, the Ω factor is 0.88. In the existing FHWA Guidance document, the Engineer had the ability to choose Ω=0.74 or Ω=1.00. Therefore if Ω=0.74 was assumed originally, the proposed specifications will result in an ~19% increase in the rating for shear yielding. If Ω=1.00 was assumed originally, the proposed specifications will result in an ~12% decrease in the rating for shear yielding. No changes are made to the rating procedures for shear rupture in the proposed provisions.
7. In the calculation of the rating for compression resistance, higher or lower ratings will be obtained using the proposed provisions over those obtained using the FHWA Guidance document depending on the assumptions that were made when rating with the FHWA Guidance document. The FHWA Guidance document recommended using an equivalent column length, which was the average of three different lengths along the Whitmore plane, referred to as L_{avg}. This length and an assumed column length factor were used to calculate a column slenderness parameter, λ_{avg}, which in turn was used to calculate the critical buckling stress of the idealized column. The new rating provisions will certainly produce less favorable ratings over the FHWA Guidance document when $\lambda_{avg} < \sim 1.0$. This is because the partial plane shear yield criterion that is instituted in the proposed provisions will control for these very compact gusset plates, and this criterion was not checked in the FHWA Guidance document. On the contrary, if $\lambda_{avg} > \sim 1.5$ the new provisions will produce more favorable ratings over the FHWA Guidance document because of the new effective column length factor of 0.5, which is much lower than the K-factor that was likely

employed when using the FHWA Guidance document. The ratings with the new provisions and the FHWA Guidance document are expected to be similar when $1.0 < \lambda_{avg} < 1.5$. No changes are made to the rating procedures for tension yielding and net section fracture on the Whitmore plane.

8. For block shear rupture, the resistance equation was updated to reflect the revision made to this equation in the Fifth Edition AASHTO LRFD Bridge Design Specification. A resistance factor of 0.85 is specified for the block shear rupture check, which is the same as the factor specified in the FHWA Guidance document. Thus, it is unlikely there will be a significant difference between the block shear rupture rating determined using the proposed LFR specification and the FHWA Guidance document.
9. The FHWA Guidance document recommended using a Whitmore section analysis in the rating of tension and compression chord splices. The real stress patterns in the analysis models did not correlate well with this assumption and this method is no longer recommended. Therefore, if this particular check controlled using the FHWA Guidance document, it will no longer apply under the proposed provisions. A new chord splice check is introduced within the proposed specifications that better accounts for the variability of gusset plate geometries versus the Whitmore section approach. The effect of this new approach on the load rating for these splices is difficult to ascertain as its effect will be specific to each joint, which typically has a unique geometry.

REFERENCES:

FHWA. 2009. Load Rating Guidance and Examples For Bolted and Riveted Gusset Plates In Truss Bridges, FHWA-IF-09-014, U.S. Department of Transportation, Washington, DC.

See also the revised MBE Article 1.6 in Attachment A.

OTHER:

None

ATTACHMENT A – 2013 AGENDA ITEM 41 - T-18/T-14

Make the following revisions to Articles 1.6, 6A.4.2.4, 6A.6, C6B.5.2.1, C6B.5.3.1 & Appendix L6B of the Manual for Bridge Evaluation:

1.6—REFERENCES

Add the following references:

Brown, J. D., D. J. Lubitz, Y. C. Cekov, and K. H. Frank. 2007. *Evaluation of Influence of Hole Making Upon the Performance of Structural Steel Plates and Connections*, Report No. FHWA/TX-07/0-4624-1. University of Texas at Austin, Austin, TX.

Hafner, A., O. T. Turan, and T. Schumacher. 2012. "Experimental Tests of Truss Bridge Gusset Plates Connections with Sway-Buckling Response," *Journal of Bridge Engineering*, American Society of Civil Engineers, Reston, VA (accepted for publication).

Kulak, G. L., J. W. Fisher, and J. H. A. Struik. 1987. *Guide to Design Criteria for Bolted and Riveted Joints*, Second Edition. John Wiley and Sons, Inc. New York, NY.

NCHRP. 2013. *Guidelines for the Load and Resistance Factor Design and Rating of Welded, Riveted and Bolted Gusset-Plate Connections for Steel Bridges*, NCHRP Report 7XX, Transportation Research Board, National Research Council, Washington D.C (to be published).

Sheikh-Ibrahim, F. I. 2002. "Design Method for the Bolts in Bearing-Type Connections with Fillers," *AISC Engineering Journal*, American Institute of Steel Construction, Chicago, IL, Vol. 39, No. 4, pp. 189-195.

Yamamoto, et al. 1998. "Buckling Strengths of Gusseted Truss Joints," *Journal of Structural Engineering*, American Society of Civil Engineers, Reston, VA, Vol. 114.

Yura, J. A., K. H. Frank, and D. Polyzois. 1987. *High-Strength Bolts for Bridges*, PMFSEL Report No. 87-3. University of Texas, Austin, TX, May 1987.

Yura, J. A., M. A. Hansen, and K.H. Frank. 1982. "Bolted Splice Connections with Undeveloped Fillers," *Journal of the Structural Division*. American Society of Civil Engineers, New York, NY, Vol. 108, No. ST12, December, pp. 2837-2849.

6A.4—LOAD-RATING PROCEDURES

6A.4.2.4—System Factor: φ_s

System factors are multipliers applied to the nominal resistance to reflect the level of redundancy of the complete superstructure system. Bridges that are less redundant will have their factored member capacities reduced, and, accordingly, will have lower ratings.

System factors that correspond to the load factor modifiers in the *AASHTO LRFD Bridge Design Specifications* should be used. The system factors in Table 6A.4.2.4-1 are more conservative than the LRFD design values and may be used at the discretion of the evaluator until they are modified in the *AASHTO LRFD Bridge Design Specifications*.

The system factor for riveted and bolted gusset plates and their connections for all force effects shall be taken as 0.90.

C6A.4.2.4

Structural members of a bridge do not behave independently, but interact with other members to form one structural system. Bridge redundancy is the capability of a bridge structural system to carry loads after damage to or the failure of one or more of its members. Internal redundancy and structural redundancy that exists as a result of continuity are neglected when classifying a member as nonredundant.

6A.6—STEEL STRUCTURES

6A.6.3—Resistance Factors

Except as specified herein, resistance factors, φ, for steel members, for the strength limit state, shall be taken as specified in LRFD Design Article 6.5.4.2.

For load rating of main truss member gusset plates, the resistance factors shall be taken as follows:

- For gusset plate compression $\varphi_{cg} = 0.95$
- For gusset plate chord splices $\varphi_{cs} = 0.85$
- For gusset plate shear yielding $\varphi_{vy} = 1.00$
- For gusset plate block shear rupture $\varphi_{bs} = 1.00$
- For gusset plate shear fracture $\varphi_{vu} = 0.80$
- For tension, fracture in net section $\varphi_{u} = 0.80$
- For tension, yielding in gross section $\varphi_{y} = 0.95$
- For A325 and A490 bolts in shear $\varphi_{s} = 0.80$
- For A307 bolts in shear $\varphi_{s} = 0.75$
- For fasteners bearing on material $\varphi_{bb} = 0.80$

C6A.6.3

For service limit states, φ = 1.0.

Users of this specification and the Bridge Design Specification will note some differences in the specified resistance factors for main truss member gusset plates. The differences are due to the fact that a higher acceptable level of reliability can be tolerated more readily in design than in rating. In addition, the determination of the resistance factors in the two specifications was based on different dead-to-live load ratios in order to provide more lenient factors for use in rating. The resistance factors are based on the findings from NCHRP Project 12-84 (NCHRP, 2013), which did not obtain sufficient data for all possible gusset-plate modes of failure to justify a difference in some of the factors that are provided in the two specifications.

6A.6.5—Effects of Deterioration on Load Rating

A deteriorated structure may behave differently than the structure as originally designed and different failure modes may govern its load capacity. Corrosion is the major cause of deterioration in steel bridges. Effects of corrosion include section loss, unintended fixities, movements and pressures, and reduced fatigue resistance.

C6A.6.5

Tension Members with Section Losses Due to Corrosion

Corrosion loss of metals can be uniform and evenly distributed or it can be localized. Uniform reduction in the cross-sectional area of a tension member causes a proportional reduction in the capacity of the member. Since localized corrosion results in irregular localized reductions in area, a simplified approach to evaluating the effects of localized corrosion is to consider the yielding of the reduced net area as the governing limit state. Due to their self-stabilizing nature, stress concentrations and eccentricities induced by asymmetrical deterioration may be neglected when estimating the tension strength of members with moderate deterioration.

For eyebars and pin plates, the critical section is located at the pin hole normal to the applied stress. In evaluating eyebars with significant section loss in the head, the yielding of the reduced net section in the head should be checked as it may be a governing limit state.

Deterioration of lacing bars and batten plates in built-up tension members may affect the load sharing among the main tension elements at service loads. At ultimate load, yielding will result in load redistribution among the tension elements and the effect on capacity is less significant.

Compression Members with Section Losses Due to Corrosion

Uniform Corrosion

Local Effects—The susceptibility of members with reduced plate thickness to local buckling should be evaluated with respect to the limiting width/thickness ratios specified in LRFD Design Article 6.9.4.2. If these values are exceeded, AISC *LRFD Manual of Steel Construction* may be used to evaluate the local residual compressive capacity.

Overall Effects—Most compression members encountered in bridges are in the intermediate length range and have a box-shape or H-shape cross section. Moderate uniform corrosion of these sections has very little effect on the radius of gyration. The reduction of compressive resistance for short and intermediate length members, for moderate deterioration, is proportional to the reduction in cross-sectional area.

Localized Corrosion

Deterioration at the ends of fixed-end compression members may result in a change in the end restraint conditions and reduce its buckling strength. Localized corrosion along the member can cause changes in the moment of inertia. Asymmetric deterioration can induce load eccentricities. The effects of eccentricities can be estimated using the eccentricity ratio ec/r^2, where e is the load eccentricity in the member caused by localized section loss, c is the distance from the neutral axis to the extreme fiber in compression of the original section, and r is the radius of gyration of the original section. Effects of eccentricity may be neglected for eccentricity ratios under 0.25.

Built-Up Members with Deteriorated Lacing Bars/Batten Plates

The main function of lacing bars and batten plates is to resist the shear forces that result from buckling of the member about an axis perpendicular to the open web. They also provide lateral bracing for the main components of the built-up member. Localized buckling of a main component can result because of loss of lateral bracing from the deterioration of the lacing bars. The slenderness ratio of each component shape between connectors and the ~~nomina~~ nominal compressive resistance of built-up members should be evaluated as specified in LRFD Design Article 6.9.4.3.

Corrosion of lacing bars and batten plates reduces the shear resistance of the built-up member and, therefore, a reduction in its overall buckling strength may result. Approximate analytical solutions for the buckling resistance of built-up members with deteriorated lacing and batten plates can be formulated using a reduced effective modulus of elasticity of the member, given in NCHRP Report 333. It has been determined that moderate deterioration of up to about 25 percent loss of the original cross-section of lacing bars and batten plates has very little effect on the overall member capacity, as long as the resistance to local failure is satisfactory.

Flexural Members with Section Losses Due to Corrosion

Uniform Corrosion

The reduction in bending resistance of laterally supported beams with stiff webs will be proportional to the reduction in section modulus of the corroded cross-section compared to the original cross-section. Either the elastic or plastic section modulus shall be used, as appropriate. Local and overall beam stability may be affected by corrosion losses in the compression flange.

The reduction in web thickness will reduce shear resistance and bearing capacity due to both section loss and web buckling. When evaluating the effects of web losses, failure modes due to buckling and out-of-plane movement that did not control their original design may govern. The loss in shear resistance and bearing capacity is linear up to the point ~~there~~ where buckling occurs.

Localized Corrosion

Small web holes due to localized losses not near a bearing or concentrated load may be neglected. All other web holes should be analytically investigated to assess their effect.

A conservative approach to the evaluation of tension and compression flanges with highly localized losses is to assume the flange is an independent member loaded in tension or compression. When the beam is evaluated with respect to its plastic moment capacity, the plastic section modulus for the deteriorated beam may be used for both localized and uniform losses.

Main Truss Member Gusset Plates

The resistance of gusset plates may be reduced if section loss due to corrosion is present at certain locations coinciding with the failure planes assumed in applying the resistance equations specified in Article 6A.6.12.6.

For evaluating the tension resistance, only the section loss that intersects the Whitmore section must be accounted for when calculating the resistance. The section loss may be smeared uniformly over the entire Whitmore section.

For evaluating the shear resistance, the use of the remaining area across a failure plane is sufficient for determining the resistance regardless of whether or not multi-layered gusset plates are present or the corrosion is localized, is asymmetric about the connection work point, or affects only one gusset plate.

For evaluating the compressive resistance, the actual area remaining in the partial shear plane defined in Article 6A.6.12.6.6 is to be considered. When evaluating the compressive resistance according to Article 6A.6.12.6.7, an equivalent plate thickness should be defined for the Whitmore section based on a projection upon the Whitmore section of all cross-sectional area loss occurring between the Whitmore section and the adjoining members in the direction of the member, as shown in Figure C6A.6.5-1. In this case, a smeared uniform plate thickness must be derived for L_{total} considering the isolated section loss occurring over $L_{corrosion}$. These methods were found to be conservative as reported in NCHRP (2013).

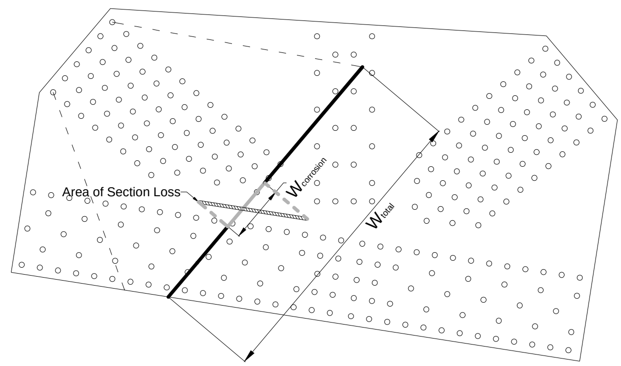

Figure C6A.6.5-1—Section-Loss Band Projected Upon the Whitmore Section to Determine an Equivalent Average Plate Thickness for the Compressive Resistance Evaluation

6A.6.12.1—General

External connections of nonredundant members shall be evaluated during a load rating analysis in situations where the evaluator has reason to believe that their capacity may govern the load rating of the entire bridge. Evaluation of critical connections shall be performed in accordance with the provisions of these articles.

C6A.6.12.1

External connections are connections that transfer calculated load effects at support points of a member. Nonredundant members are members without alternate load paths whose failure is expected to cause the collapse of the bridge.

It is common practice to assume that connections and splices are of equal or greater capacity than the members they adjoin. With the introduction of more accurate evaluation procedures to identify and use increased member load capacities, it becomes increasingly important to also closely scrutinize the capacity of connections and splices to ensure that they do not govern the load rating.

~~Specifically, truss gusset plate connection analysis has been summarized in *FHWA Gusset Guidance Load Rating Guidance and Examples for Bolted and Riveted Gusset Plates in Truss Bridges*, FHWA-IF-09-014, February 2009. A good deal of engineering judgment is required to apply this guidance as connection geometry is variable and to account for effects of measurable corrosion if present. Other references as follows may also be helpful in order to use the guidance:~~

~~Cheng, J. J.R. and G. Y. Grondin. 2001. *Design and Behavior of Gusset Plate Connections.*~~

~~Galambos, T. V. 1998. *Guide to Stability Design Criteria for Metal Structures*, Fifth Edition. John Wiley and Sons, New York, NY.~~

~~Yamamoto, et al. 1998. “Buckling Strengths of Gusseted Truss Joints,” *ASCE Journal of Structural Engineering*,~~

~~Vol. 114.~~

~~Analysis of gusset connections of truss bridges should be preceded by a field investigation of gusset plates at all truss joints. Field inspections of gusset plates need to focus on corrosion, distortion, and connections. Section losses can occur along gusset plate areas that trap debris or hold water, usually along the top of the bottom chord. Distortion in the gusset plate can be from original construction, or can be caused by overstressing of the plate due to overloads, inadequate thickness/bracing, forces associated with pack rust between plates, or traffic impact. Gusset plate member connections should be inspected closely according to the provisions of Article 4.8.3.10.~~

6A.6.12.6—Gusset Plates

Main truss member gusset plates shall be load rated for shear, compression, and/or tension, as applicable, occurring in the vicinity of each connected member. Except as specified herein, a load rating analysis of main truss member gusset plates and their connections shall be conducted according to the provisions of Articles 6A.6.12.6.1 through 6A.6.12.6.9. Alternatively, a load rating analysis may be performed according to the provisions of Article 6A.6.12.6.11. The system factor, φ_s, for riveted and bolted gusset plates specified in Article 6A.4.2.4 shall be applied to the load rating of the gusset plates and their connections. The load rating provisions specified herein may be used for the evaluation of gusset plates and their connections for design loads, legal loads or permit loads, and shall utilize the appropriate live load factors provided in these Specifications for the load rating of primary members.

C6A.6.12.6

A load rating analysis of the main truss gusset connections of truss bridges should be preceded by a field investigation of the gusset plates at all truss joints. Field inspections of the gusset plates need to focus on corrosion, distortion, and their connections. Section losses can occur along gusset plate areas that trap debris or hold water, usually along the top of the bottom chord. Distortion in the gusset plate can occur during the original construction, or can be caused by overstressing of the plate due to overloads, inadequate thickness/bracing, forces associated with the development of pack rust between the plates, or traffic impact. Gusset plate member connections should be inspected closely according to the provisions of Article 4.8.3.10. Effects of deterioration on the resistance of the gusset plate should be accounted for as discussed in Article C6A.6.5.

The resistance equations provided herein were developed and calibrated to a target reliability index of 3.5 at the Strength I Inventory level at an unfactored dead-to-live load ratio, DL/LL, of 1.0. For larger values of the unfactored DL/LL, calculated resistances for application in the load rating analysis at the strength limit state are to be reduced as specified in Article 6A.6.12.6.1. In situations where the unfactored DL/LL is less than 1.0, an increase in the calculated resistances could be justified by backward interpolation according to the provisions of Article 6A.6.12.6.1, although the anticipated gains would be marginal.

The provisions provided in this article are intended for the load rating of double gusset-plate connections used in trusses that may each be made from multiple layers of plates. The validity of the requirements for application to single gusset-plate connections has not been verified.

These provisions are based on the findings from NCHRP Project 12-84 (NCHRP, 2013), and supersede the 2009 FHWA Guidelines for gusset-plate load ratings. Example calculations illustrating the application of the resistance equations contained herein for the load rating of a gusset-plate connection by LRFR are provided in

NCHRP (2013) and in Appendix A.

6A.6.12.6.1–Resistance Reduction for DL/LL Ratio

If the unfactored dead-to-live load ratio, DL/LL, as determined by the member forces on the gusset plate connection is greater than 1.0, the resistances determined in Articles 6A.6.12.6.2 through 6A.6.12.6.11 for application in the load rating analysis at the strength limit state shall be reduced as specified herein. The resistance reduction shall decrease linearly from 1.00 to 0.90 as DL/LL increases from 1.0 to 6.0. The resistance reduction shall not be taken as less than 0.90.

The resistance reduction shall not be applied in the load rating analysis at the service limit state.

C6A.6.12.6.1

To maintain a constant reliability index, the required resistance factor decreases as the unfactored dead-to-live ratio, DL/LL, increases. Since resistance factors were developed and calibrated for an unfactored DL/LL of 1.0, the resistance reduction specified herein for application in the load rating analysis at the strength limit state accounts for the necessary decrease in the resistance factor for an unfactored DL/LL greater than 1.0.

6A.6.12.6.2—Fastener Shear Resistance

The factored shear resistance of rivets, $\varphi_s F_{uv}$, at the strength limit state shall be determined as specified in Article 6A.6.12.5.1.

The factored shear resistance, φR_n, of a high-strength bolt (ASTM A325 or ASTM A490) or an ASTM A307 bolt (Grade A or B) at the strength limit state in joints whose length between extreme bolts measured parallel to the line of action of the force is less than 50.0 in. shall be taken as:

- Where threads are excluded from the shear plane:

$$\varphi R_n = \varphi_s 0.48 A_b F_{ub} N_s \quad \text{(6A.6.12.6.2-1)}$$

- Where threads are included in the shear plane:

$$\varphi R_n = \varphi_s 0.38 A_b F_{ub} N_s \quad \text{(6A.6.12.6.2-2)}$$

where:

φ_s = resistance factor bolts in shear specified in Article 6A.6.3

A_b = area of the bolt corresponding to the nominal diameter (in.^{2})

F_{ub} = specified minimum tensile strength of the bolt specified in Table 6A.6.12.6.2-1 (ksi)

N_s = number of shear planes per bolt

The factored shear resistance of a bolt in connections greater than 50.0 in. in length shall be taken as 0.80 times the value given by Eq. 6A.6.12.6.2-1 or 6A.6.12.6.2-2.

For ASTM A307 bolts, shear design shall be based on Eq. 6A.6.12.6.2-2. When the grip length of an ASTM A307 bolt exceeds 5.0 diameters, the factored resistance shall be lowered one percent for each 1/16 in. of grip in excess of 5.0 diameters.

C6A.6.12.6.2

The nominal resistance of a high-strength bolt in shear, R_n, is based upon the observation that the shear strength of a single high-strength bolt is about 0.60 times the tensile strength of that bolt (Kulak et al., 1987). However, in shear connections with more than two bolts in the line of force, deformation of the connected material causes a nonuniform bolt shear force distribution so that the resistance of the connection in terms of the average bolt resistance decreases as the joint length increases. Rather than provide a function that reflects this decrease in average bolt resistance with joint length, a single reduction factor of 0.80 was applied to the 0.60 multiplier. This accommodates bolts in joints up to 50.0 in. in length without seriously affecting the economy of very short joints. The nominal shear resistance of bolts in joints longer than 50.0 in. must be further reduced by an additional 20 percent. Studies have shown that the allowable stress factor of safety against shear failure ranges from 3.3 for compact, i.e., short, joints to approximately 2.0 for joints with an overall length in excess of 50.0 in. It is of interest to note that the longest and often the most important joints had the lowest factor, indicating that a factor of safety of 2.0 has proven satisfactory in service (Kulak et al., 1987).

The average value of the nominal resistance for bolts with threads in the shear plane has been determined by a series of tests to be 0.833 ($0.6F_{ub}$), with a standard deviation of 0.03 (Yura et al., 1987). A value of about 0.80 was selected for the formula based upon the area corresponding to the nominal body area of the bolt.

The shear resistance of bolts is not affected by pretension in the bolts, provided that the connected material is in contact at the faying surfaces.

The threaded length of an ASTM A307 bolt is not as predictable as that of a high-strength bolt. The requirement to use Eq. 6A.6.12.6.2-2 reflects that uncertainty.

ASTM A307 bolts with a long grip tend to bend, thus reducing their resistance.

Table 6A.6.12.6.2-1–Specified Minimum Tensile Strength of Bolts

	F_u (ksi)
A307 Grade A or B	60
A325 for diameters 0.5 through 1.0 in.	120
A325 for diameters greater than 1.0	105
A490	150

When bolts carrying loads pass through undeveloped fillers 0.25 in. or more in thickness in axially loaded connections, the factored shear resistance of the bolt shall be reduced by the following factor:

$$R = \left[\frac{(1+\gamma)}{(1+2\gamma)}\right] \quad (6A.6.12.6.2\text{-}3)$$

where:

γ = A_f/A_p

A_f = sum of the area of the fillers on both sides of the connected plate (in.^{2})

A_p = smaller of either the connected plate area or the sum of the splice plate areas on both sides of the connected plate (in.^{2}). For chord splices, when considering the gusset plate(s), only the portion of the gusset plate(s) that overlaps the connected plate shall be considered in the calculation of the splice plate areas.

Fillers must be secured by means of additional bolts so that the fillers are, in effect, an integral part of a shear-connected component at the strength limit state. The integral connection results in well-defined shear planes and no reduction in the factored shear resistance of the bolts. For undeveloped fillers 0.25 in. or more in thickness, the reduction factor given by Eq. 6A.6.12.6.2-3 is to be applied to the factored resistance of the bolts in shear. This factor compensates for the reduction in the nominal shear resistance of a bolt caused by bending in the bolt. The reduction factor is only to be applied on the side of the connection with the fillers. The factor was developed mathematically (Sheikh-Ibrahim, 2002), and verified by comparison to the results from an experimental program on axially loaded bolted splice connections with undeveloped fillers (Yura et al., 1982). Alternatively, if fillers are extended beyond the connected parts and connected with enough bolts to develop the force in the fillers, the fillers may be considered developed.

For slip-critical high-strength bolted connections, the factored slip resistance of a bolt need not be adjusted for the effect of the fillers. The resistance to slip between the fillers and either connected part is comparable to that which would exist between the connected parts if the fillers were not present.

6A.6.12.6.3—Bolt Slip Resistance

The nominal slip resistance of a high-strength bolt in a slip-critical connection at the service limit state shall be taken as:

$$R_n = K_h K_s N_s P_t \quad (6A.6.12.6.3\text{-}1)$$

where:

N_s = number of slip planes per bolt

P_t = minimum required bolt tension specified in Table 6A.6.12.6.3-1 (kips)

K_h = hole size factor taken as 1.0 for standard holes, or as specified in LRFD Design Table 6.13.2.8-

C6A.6.12.6.3

Extensive data developed through research has been statistically analyzed to provide improved information on slip probability of high-strength bolted connections in which the bolts have been preloaded to the requirements of Table 6A.6.12.6.3-1. Two principal variables, bolt pretension and coefficient of friction, i.e., the surface condition factor of the faying surfaces, were found to have the greatest effect on the slip resistance of connections.

Hole size factors less than 1.0 are provided in LRFD Design Table 6.13.2.8-2 for bolts in oversize and slotted holes because of their effects on the induced tension in bolts using any of the specified installation methods. In the case of bolts in long-slotted holes, even though the slip load is the same for bolts loaded transverse or parallel to the axis of the slot, the values for bolts loaded

2 for oversize or slotted holes

K_s = surface condition factor specified in Table 6A.6.12.6.3-2

Table 6A.6.12.6.3-1—Minimum Required Bolt Tension

Bolt Diameter, in.	Required Tension – P_t (kip)	
	A325	A490
5/8	19	24
¾	28	35
7/8	39	49
1	51	64
1-1/8	56	80
1-1/4	71	102
1-3/8	85	121
1-1/2	103	148

Table 6A.6.12.6.3-2—Values of K_s

For Class A surface conditions	0.33
For Class B surface conditions	0.50
For Class C surface conditions	0.33

The following descriptions of surface condition shall apply to Table 6A.6.12.6.3-2:

- Class A Surface: unpainted clean mill scale, and blast-cleaned surfaces with Class A coatings,
- Class B Surface: unpainted blast-cleaned surfaces and blast-cleaned surfaces with Class B coatings, and
- Class C Surface: hot-dip galvanized surfaces roughened by wire brushing after galvanizing.

parallel to the axis have been further reduced, based upon judgment, because of the greater consequences of slip.

The minimum bolt tension values given in Table 6A.6.12.6.3-1 are equal to 70 percent of the minimum tensile strength of the bolts. The same percentage of the tensile strength has been traditionally used for the required tension of the bolts.

Further information on the surface condition factors provided in Table 6A.6.12.6.3-2 may be found in LRFD Design Article C6.13.2.8.

6A.6.12.6.4—Bearing Resistance at Fastener Holes

The effective bearing area of a fastener shall be taken as its diameter multiplied by the thickness of the gusset plate on which it bears.

For standard holes, oversize holes, short-slotted holes, and long-slotted holes parallel to the applied bearing force, the factored resistance of interior and end fastener holes at the strength limit state, φR_n, shall be taken as:

- With fasteners spaced at a clear distance between holes not less than 2.0d and with a clear end distance not less than 2.0d:

$$\varphi R_n = \varphi_{bb} 2.4 dtF_u \qquad \text{(6A.6.12.6.4-1)}$$

- If either the clear distance between holes is less than 2.0d, or the clear end distance is less than 2.0d:

$$\varphi R_n = \varphi_{bb} 1.2 L_c tF_u \qquad \text{(6A.6.12.6.4-2)}$$

C6A.6.12.6.4

The term fastener in this article is meant to encompass both rivets and high-strength bolts.

Bearing stress produced by a fastener pressing against the side of the hole in a connected part is important only as an index to behavior of the connected part. Thus, the same bearing resistance applies regardless of fastener shear strength or the presence or absence of threads in the bearing area. The critical value can be derived from the case of a single fastener at the end of a tension member.

It has been shown that a connected plate will not fail by tearing through the free edge of the material if the distance L, measured parallel to the line of applied force from a single fastener to the free edge of the member toward which the force is directed, is not less than the diameter of the fastener multiplied by the ratio of the bearing stress to the tensile strength of the connected part (Kulak et al., 1987).

The criterion for nominal bearing strength is:

where:

ϕ_{bb} = resistance factor for fasteners bearing on material specified in Article 6A.6.3

d = nominal diameter of the fastener (in.)

t = thickness of the connected material (in.)

F_u = tensile strength of the connected material (ksi)

L_c = clear distance between holes or between the hole and the end of the member in the direction of the applied bearing force (in.)

$$\frac{L}{d} \geq \frac{r_n}{F_u} \qquad \text{(C6A.6.12.6.4-1)}$$

where:

r_n = nominal bearing pressure (ksi)

F_u = specified minimum tensile strength of the connected part (ksi)

In these Specifications, the nominal bearing resistance of an interior hole is based on the clear distance between the hole and the adjacent hole in the direction of the bearing force. The nominal bearing resistance of an end hole is based on the clear distance between the hole and the end of the member. The nominal bearing resistance of the connected member may be taken as the sum of the resistances of the individual holes.

Holes may be spaced at clear distances less than the specified values, as long as the lower value specified by Eq. 6A.6.12.6.4-2 is used for the nominal bearing resistance.

For determining the factored bearing resistance of long-slotted holes loaded perpendicular to the applied bearing force, refer to LRFD Design Article 6.13.2.9.

6A.6.12.6.5—Multi-Layered Gusset and Splice Plates

Where multi-layered gusset and splice plates are used, the resistances of the individual plates may be added together when determining the factored resistances specified in Articles 6A.6.12.6.6 through 6A.6.12.6.9 provided that enough fasteners are present to develop the force in the layered gusset and splice plates.

C6A.6.12.6.5

Kulak et al. (1987) contains additional guidance on determining the number of fasteners required to develop the force in layered gusset and splice plates.

6A.6.12.6.6—Gusset Plate Shear Resistance

The factored shear resistance, V_r, of gusset plates at the strength limit state shall be taken as the smaller value based on shear yielding or shear rupture.

For shear yielding, the factored shear resistance shall be taken as:

$$V_r = \varphi_{vy} 0.58 F_y A_{vg} \Omega \qquad \text{(6A.6.12.6.6-1)}$$

where:

Ω = shear reduction factor for gusset plates taken as 0.88

φ_{vy} = resistance factor for gusset plate shear yielding specified in Article 6A.6.3

A_{vg} = gross area of the shear plane (in.^{2})

F_y = specified minimum yield strength of the gusset

C6A.6.12.6.6

The Ω shear reduction factor is used only in the evaluation of truss gusset plates for shear yielding. This factor accounts for the nonlinear distribution of shear stresses that form along a failure plane as compared to an idealized plastic shear stress distribution. The nonlinearity primarily develops due to shear loads not being uniformly distributed on the plane and also due to strain hardening and stability effects. The Ω factor was developed using shear yield data generated in NCHRP Project 12-84 (NCHRP, 2013). On average, Ω was 1.02 for a variety of gusset-plate geometries; however, the data were scattered due to proportioning of load between members, and variations in plate thickness and joint configuration. The specified Ω and resistance factors have been calibrated to account for shear plane length-to-thickness ratios varying from 85 to 325.

Failure of a full width shear plane requires relative mobilization between two zones of the plate, typically along chords. Mobilization cannot occur when a shear plane passes through a continuous member; for instance, plate (ksi)

For shear rupture, the factored shear resistance shall be taken as:

$$V_r = \varphi_{vu} 0.58 F_u A_{vn} \quad (6A.6.12.6.6\text{-}2)$$

where:

φ_{vu} = resistance factor for gusset plate shear rupture specified in Article 6A.6.3

A_{vn} = net area of the shear plane (in.^{2})

F_u = specified minimum tensile strength of the gusset plate (ksi)

Shear shall be checked on relevant partial and full failure plane widths. Partial shear planes shall only be checked around compression members and only Eq. 6A.6.12.6.6-1 shall apply to partial shear planes. The partial shear plane length shall be taken along adjoining member fastener lines between plate edges and other fastener lines. The following partial shear planes, as applicable, shall be evaluated to determine which shear plane controls:

- The plane that parallels the chamfered end of the compression member, as shown in Figure 6A.6.12.6.6-1;
- The plane on the side of the compression member that has the smaller framing angle between the member and the other adjoining members, as shown in Figure 6A.6.12.6.6 -2; and
- The plane with the least cross-sectional shear area if the member end is not chamfered and the framing angle is equal on both sides of the compression member.

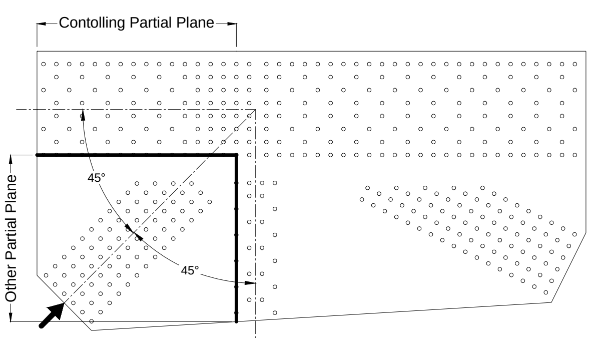

Figure 6A.6.12.6.6-1—Example of a Controlling Partial Shear Plane that Parallels the Chamfered End of the Compression Member Since that Member Frames in at an Angle of 45 Degrees to Both the Chord and the Vertical

a plane passing through a continuous chord member that would require shearing of the member itself.

Research has shown that the buckling of connections with tightly spaced members is correlated with shear yielding around the compression members. This is important because the buckling criteria used in Article 6A.6.12.6.7 would overestimate the compressive buckling resistance of these types of connections. Once a plane yields in shear, the reduction in the plate modulus reduces the out-of-plane stiffness such that the stability of the plate is affected. Generally, truss verticals and chord members are not subject to the partial plane shear yielding check because there is no adjoining member fastener line that can yield in shear and cause the compression member to become unstable. For example, the two compression members shown in Figure C6A.6.12.6.6-1 would not be subject to a partial plane shear check.

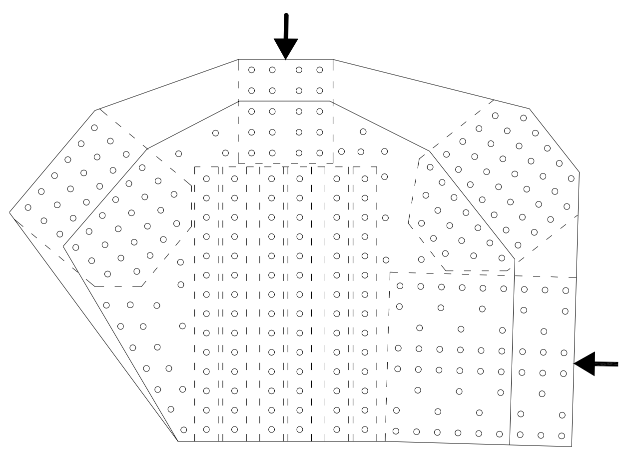

Figure C6A.6.12.6.6-1—Example Showing Truss Vertical and Chord Members in Compression that Do Not Require a Partial Shear Plane Check

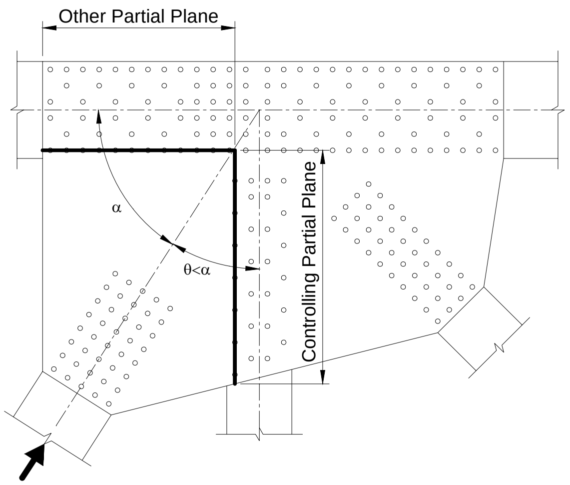

Figure 6A.6.12.6.6-2—Example of a Controlling Partial Shear Plane on the Side of a Compression Member without a Chamfered End that has the Smaller Framing Angle between that Member and the Other Adjoining Members (i.e. θ < α)

6A.6.12.6.7—Gusset Plate Compressive Resistance

The factored compressive resistance, P_r, of gusset plates at the strength limit state shall be taken as:

$$P_r = \varphi_{cg} P_n \quad (6A.6.12.6.7\text{-}1)$$

in which:

P_n = nominal compressive resistance of a Whitmore section determined from Eq. 6A.6.12.6.7-2 or 6A.6.12.6.7-3, as applicable (kips):

- If $\frac{P_e}{P_o} \geq 0.44$, then:

$$P_n = \left[0.658^{\left(\frac{P_o}{P_e}\right)} \right] P_o \quad (6A.6.12.6.7\text{-}2)$$

- If $\frac{P_e}{P_o} < 0.44$, then:

$$P_n = 0.877 P_e \quad (6A.6.12.6.7\text{-}3)$$

P_e = elastic critical buckling resistance (kips)

$$= \frac{3.29E}{\left(\frac{L_{mid}}{t_g}\right)^2} A_g \quad (6A.6.12.6.7\text{-}4)$$

C6A.6.12.6.7

Gusset plate zones in the vicinity of compression members are to be load rated for plate stability at the strength limit state. Experimental testing and finite element simulations performed as part of NCHRP Project 12-84 (NCHRP, 2013) and by others (Yamamoto et al., 1988; Hafner et al., 2012) have found that truss gusset plates subject to compression always buckle in a sidesway mode in which the end of the compression member framing into the gusset plate moves out-of-plane. The buckling resistance is dependent upon the chamfering of the member, the framing angles of the members entering the gusset, and the standoff distance of the compression member relative to the surrounding members; i.e. the distance, L_{mid}. An example connection showing a typical chamfered member end and member framing angle is provided in Figure C6A.6.12.6.7-1. The research found that the compressive resistance of gusset plates with large L_{mid} distances was reasonably predicted using modified column buckling equations and Whitmore section analysis. When the members were heavily chamfered reducing the L_{mid} distance, the buckling of the plate was initiated by shear yielding on the partial shear plane adjoining the compression member causing a destabilizing effect, as discussed in Article C6A.6.12.6.6.

Eq. 6A.6.12.6.7-4 is derived by substituting plate properties into column buckling formulas along with an effective length factor of 0.5 that was found to be relevant for a wide variety of gusset-plate geometries (NCHRP, 2013).

where:

φ_{cg} = resistance factor for gusset plate compression specified in Article 6A.6.3

A_g = gross cross-sectional area of the Whitmore section determined based on 30 degree dispersion angles, as shown in Figure 6A.6.12.6.7-1 (in.^{2}). The Whitmore section shall not be reduced if the section intersects adjoining member bolt lines.

E = modulus of elasticity (ksi).

F_y = specified minimum yield strength (ksi)

L_{mid} = distance from the middle of the Whitmore section to the nearest member fastener line in the direction of the member, as shown in Figure 6A.6.12.6.7-1 (in.).

P_o = equivalent nominal yield resistance = F_yA_g (kips)

t_g = gusset plate thickness (in.)

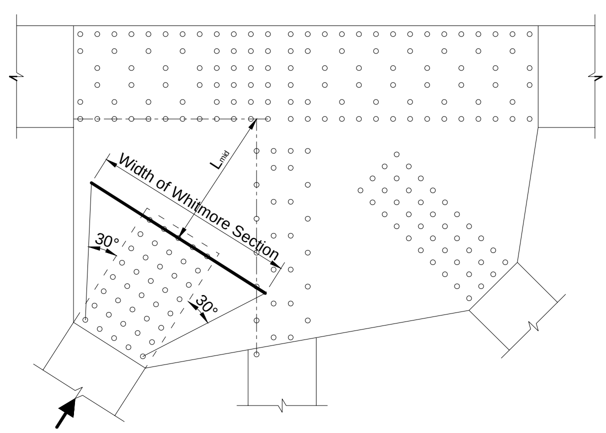

Figure 6A.6.12.6.7-1—Example Connection Showing the Whitmore Section for a Compression Member Derived From 30 Degree Dispersion Angles and the Distance L_{mid}

The provisions of this article shall not be applied to compression chord splices.

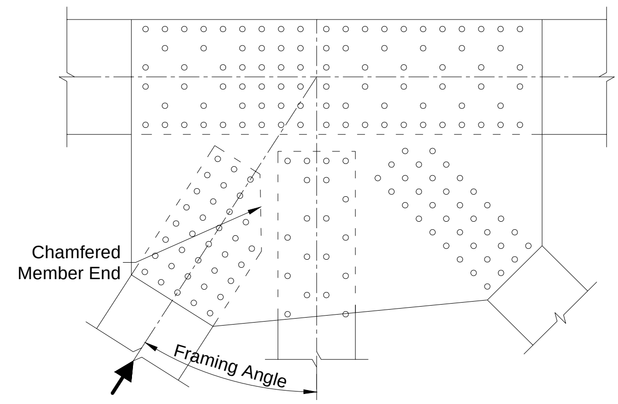

Figure C6A.6.12.6.7-1–Example Connection Showing a Typical Chamfered Member End and Member Framing Angle

6A.6.12.6.8—Gusset Plate Tensile Resistance

The factored tensile resistance, P_r, of gusset plates at the strength limit state shall be taken as the smallest factored resistance in tension based on block shear rupture, yielding on the Whitmore section, and net section fracture on the Whitmore section.

The factored block shear rupture resistance shall be taken as:

C6A.6.12.6.8

A conservative model has been adopted to predict the block shear rupture resistance in which the resistance to rupture along the shear plane is added to the resistance to rupture on the tensile plane. Block shear is a rupture or tearing phenomenon and not a yielding phenomenon. However, gross yielding along the shear plane can occur when tearing on the tensile plane commences if $0.58F_uA_{vn}$ exceeds $0.58F_yA_{vg}$. Therefore, Eq. 6A.6.12.6.8-1 limits the term $0.58F_uA_{vn}$ to not exceed $0.58F_yA_{vg}$. Eq. 6A.6.12.6.8-1 is consistent with the philosophy for

$$P_r = \varphi_{bs} R_p (0.58 F_u A_{vn} + F_u A_{tn}) \leq \varphi_{bs} R_p (0.58 F_y A_{vg} + F_u A_{tn}) \quad (6A.6.12.6.8\text{-}1)$$

where:

φ_{bs} = resistance factor for gusset plate block shear rupture specified in Article 6A.6.3

A_{vg} = gross area along the plane resisting shear stress (in.^{2})

A_{vn} = net area along the plane resisting shear stress (in.^{2})

A_{tn} = net area along the plane resisting tension stress (in.^{2})

F_y = specified minimum yield strength of the connected material (ksi)

F_u = specified minimum tensile strength of the connected material (ksi)

R_p = reduction factor for holes taken equal to 0.90 for bolt holes punched full size and 1.0 for bolt holes drilled full size or subpunched and reamed to size

The factored resistances for yielding on the Whitmore section and net section fracture on the Whitmore section shall be determined from Eqs. 6A.6.12.6.8-2 and 6A.6.12.6.8-3, respectively.

$$P_r = \varphi_y F_y A_g \quad (6A.6.12.6.8\text{-}2)$$

$$P_r = \varphi_u F_u A_n R_p U \quad (6A.6.12.6.8\text{-}3)$$

where:

φ_y = resistance factor for yielding of tension members specified in Article 6A.6.3

φ_u = resistance factor for fracture of tension members specified in Article 6A.6.3

A_g = gross cross-sectional area of the effective Whitmore section determined based on 30 degree dispersion angles, as shown in Figure 6A.6.12.6.8-1 (in.^{2}). The Whitmore section shall not be reduced if the section intersects adjoining member bolt lines.

A_n = net cross-sectional area of the effective Whitmore section determined based on 30 degree dispersion angles, as shown in Figure 6A.6.12.6.8-1 (in.^{2}). The Whitmore section shall not be reduced if the section intersects adjoining member bolt lines.

tension members where the gross area is used for yielding and the net area is used for rupture.

The reduction factor, R_p, conservatively accounts for the reduced rupture resistance in the vicinity of holes that are punched full size (Brown et al., 2007). No reduction in the net section fracture resistance is required for holes that are drilled full size or subpunched and reamed to size.

The net area, A_n, is the product of the plate thickness and its smallest net width. The width of each standard hole is to be taken as the nominal diameter of the hole. The width of oversize and slotted holes, where permitted, is to be taken as the nominal diameter or width of the hole. The net width is to be determined for each chain of holes extending across the member or element along any transverse, diagonal, or zigzag line.

The net width for each chain is to be determined by subtracting from the width of the element the sum of the widths of all holes in the chain and adding the quantity $s^2/4g$ for each space between consecutive holes in the chain, where:

s = pitch of any two consecutive holes (in.)

g = gage of the same two holes (in.)

F_u = specified minimum tensile strength of the gusset plate (ksi)

F_y = specified minimum yield strength of the gusset plate (ksi)

R_p = reduction factor for holes taken equal to 0.90 for bolt holes punched full size and 1.0 for bolt holes drilled full size or subpunched and reamed to size

U = reduction factor to account for shear lag; taken as 1.0 for gusset plates

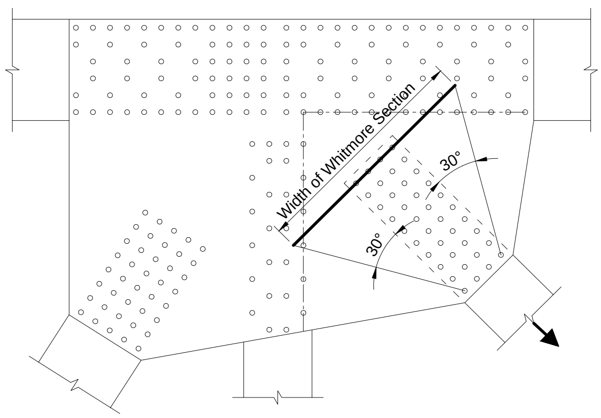

Figure 6A.6.12.6.8-1—Example Connection Showing the Whitmore Section for a Tension Member Derived from 30 Degree Dispersion Angles

The provisions of this article shall not be applied to tension chord splices.

6A.6.12.6.9—Chord Splices

Gusset plates that splice two chord sections together shall be checked using a section analysis considering the relative eccentricities between all plates crossing the splice and the loads on the spliced plane.

For compression chord splices, the factored compressive resistance, P_r, of the spliced section at the strength limit state shall be taken as:

$$P_r = \varphi_{cs} F_{cr} \left(\frac{S_g A_g}{S_g + e_p A_g} \right) \qquad (6A.6.12.6.9\text{-}1)$$

in which:

F_{cr} = stress in the spliced section at the limit of usable resistance (ksi). F_{cr} shall be taken as the specified minimum yield strength of the gusset plate when the following equation is satisfied:

C6A.6.12.6.9

This Article is only intended to cover the load rating of chord splices that occur within the gusset plates. For gusset plates also serving the role of a chord splice, the forces from all members framing into the connection must be considered. The chord splice forces are the resolved axial forces acting on each side of the spliced section, as illustrated in Figure C6A.6.12.6.9-1. The chord splice should be investigated for the larger of the two resolved forces on either side of the splice.

$$\frac{KL_{splice}\sqrt{12}}{t_g} < 25 \qquad \text{(6A.6.12.6.9-2)}$$

where:

φ_{cs} = resistance factor for gusset plate chord splices specified in Article 6A.6.3

A_g = gross area of all plates in the cross-section intersecting the spliced plane (in.^{2})

e_p = distance between the centroid of the cross-section and the resultant force perpendicular to the spliced plane (in.)

K = effective column length factor taken as 0.50 for chord splices

L_{splice} = center-to-center distance between the first lines of fasteners in the adjoining chords as shown in Figure 6A.6.12.6.9-1 (in.)

S_g = gross section modulus of all plates in the cross-section intersecting the spliced plane (in.^{3})

t_g = gusset plate thickness (in.)

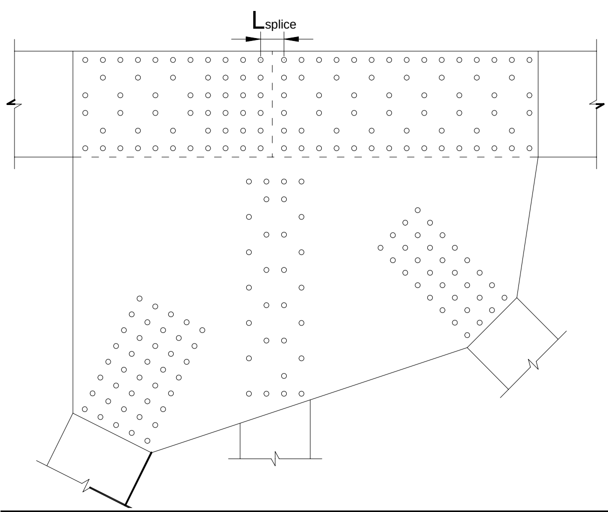

Figure 6A.6.12.6.9-1—Example Connection Showing the Chord Splice Parameter, L_{splice}

For tension chord splices, the factored tensile resistance, P_r, of the spliced section at the strength limit state shall be taken as the lesser of the values given by

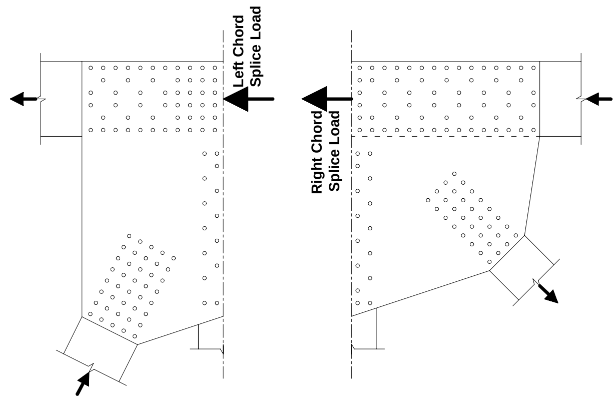

Figure C6A.6.12.6.9-1–Example Connection Showing the Resolution of the Member Forces into Forces Acting on Each Side of a Chord Splice

The resistance equations in this article assume the gusset and splice plates behave as one combined spliced section to resist the applied axial load and eccentric bending that occurs due to the fact that the resultant forces on the section are offset from the centroid of the combined section, as illustrated in Figure C6A.6.12.6.9-2. The combined spliced section is treated as a beam and the factored resistance at the strength limit state is determined assuming the stress in the combined section at the limit of usable resistance is equal to the specified minimum yield strength of the gusset plate if the slenderness limit for the spliced section given by Eq. 6A.6.12.6.9-2 is met, which will typically be the case. If not, the Engineer will need to derive a reduced value of F_{cr} to account for possible elastic buckling of the gusset plate within the splice.

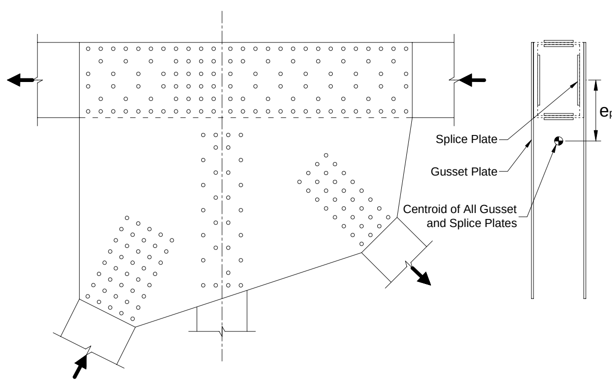

Figure C6A.6.12.6.9-2–Illustration of the Combined Spliced Section at a Chord Splice

The Whitmore section check specified in Article 6A.6.12.6.7 is not considered applicable for the load rating of a compression chord splice.

The yielding and net section fracture checks on the Whitmore section specified in Article 6A.6.12.6.8 are not considered applicable for the load rating of a tension

Eqs. 6A.6.12.6.9-3 and 6A.6.12.6.9-4.

$$P_r = \varphi_{cs} F_y \left(\frac{S_g A_g}{S_g + e_p A_g} \right) \quad (6A.6.12.6.9\text{-}3)$$

$$P_r = \varphi_{cs} F_u \left(\frac{S_n A_n}{S_n + e_p A_n} \right) \quad (6A.6.12.6.9\text{-}4)$$

where:

φ_{cs} = resistance factor for gusset plate chord splices specified in Article 6A.6.3

A_g = gross area of all plates in the cross-section intersecting the spliced plane (in.^{2})

A_n = net area of all plates in the cross-section intersecting the spliced plane (in.^{2})

e_p = distance between the centroid of the cross-section and the resultant force perpendicular to the spliced plane (in.)

F_y = specified minimum yield strength of the gusset plate (ksi)

F_u = specified minimum tensile strength of the gusset plate (ksi)

S_g = gross section modulus of all plates in the cross-section intersecting the spliced plane (in.^{3})

S_n = net section modulus of all plates in the cross-section intersecting the spliced plane (in.^{3})

Tension chord splice members shall also be checked for block shear rupture as specified in Article 6A.6.12.6.8.

chord splice.

6A.6.12.6.10—Edge Slenderness

Gusset plates shall not be load rated on the basis of edge slenderness.

C6A.6.12.6.10

NCHRP Project 12-84 (NCHRP, 2013) found no direct correlation between the buckling resistance of the gusset plate and the free edge slenderness. In addition, merely adding stiffeners to just the free edges will not provide any appreciable increase in the compressive resistance of the plate. However, properly stiffening the free edges, as discussed below, could suppress plate buckling.

Since gusset plate buckling was always observed to occur in a sway mode, either a diaphragm must be added between the two gussets, preferably also connected to the chord, to stiffen against sway, or else stiffening elements must be placed along the free edges such that their full out-of-plane yield moment resistance can be developed at the planes that would bend if sway occurs. These requirements do not apply if the free edges are merely being stiffened without relying on an increase in buckling resistance. In this case, there are no criteria specified for sizing of the edge stiffeners, but the traditional practice of using angles with leg thicknesses of 0.50 in. has generally proven adequate to reduce deformations of the free edges during fabrication, erection, and service.

The effect of proper edge stiffening on the compressive resistance of the gusset plate was examined experimentally and analytically in NCHRP Project 12-84 (NCHRP, 2013). The increase in compressive resistance was highly dependent upon the configuration of the connection and was found to vary from 6 percent to 45 percent. Generally, connections using chamfered members that allowed for very closely spaced member arrangements experienced little increase in compressive resistance. Connections that had large spans of free plate between the compression members and the surrounding members experienced the largest increase in compressive resistance. That is, properly stiffened free edges tend to suppress buckling as predicted by the Whitmore section analysis specified in Article 6A.6.12.6.7 in gusset plates with large L_{mid} distances. However, proper edge stiffening will likely not suppress the buckling resulting from partial plane shear yielding in cases with small L_{mid} distances. Therefore, in such cases, the resistance calculated according to the provisions of Article 6A.6.12.6.6 would be considered to represent the upper bound of compressive buckling resistance with properly stiffened free edges, unless a refined simulation analysis indicates otherwise. A refined simulation analysis, which is permitted according to the provisions of Article 6A.6.12.6.11, may be used to better quantify the increase in compressive resistance offered by properly stiffened free edges.

6A.6.12.6.11—Refined Analysis

A refined simulation analysis using the finite element method may be employed to determine the nominal resistance of a gusset-plate connection at the strength limit state in lieu of satisfying the requirements specified in Articles 6A.6.12.6.6 through 6A.6.12.6.9. The nominal resistance obtained from the refined simulation analysis shall be multiplied by 0.90 in order to obtain the factored resistance of the connection.

C6A.6.12.6.11

A refined simulation analysis does not consider the variability of material properties and fabrication tolerances assumed in the AASHTO LRFR calibration. As a result, to be consistent with the philosophy of the AASHTO LRFR specifications, the 0.90 reduction factor was developed as a partial φ factor accounting for these two issues. This value assumes the simulation analysis is accurate enough such that there is no variation in the professional factor and was calibrated to provide a target reliability index of 3.5. The reduction factor specified in Article 6A.6.12.6.1 is also to be considered.

The necessary fidelity of the model is dependent upon the failure mode under investigation. For instance, simple planar shell finite element models of single gusset plates have been successfully used to identify the nominal shear resistance of gusset-plate connections. These models included nonlinear material properties with strain hardening, and member loads were applied as surface tractions at fastener locations. However, additional modeling effort is required to predict the nominal compressive buckling resistance of a gusset plate.

Considering the following list of model attributes, NCHRP Project 12-84 researchers were able to attain model predictions within 9% of experimental values for a 3-dimensional two-panel truss system isolated out of an entire bridge where the connection of interest was located in the center between two panels (NCHRP, 2013). Model symmetry was not used because the sway buckling mode would not be captured. The following list, which is not considered exhaustive, summarizes other important attributes of the preceding model:

- The gusset plate, splice plates, and the members for a distance of two member depths away from the gusset-plate edge were modeled with shell elements. The truss was represented with beam elements at all other locations;

- The shell elements were able to capture nonlinear geometric and material effects. Nonlinear material properties considered strain hardening;

- Each fastener was represented with a line element with deformable, nonlinear material properties;

- The mesh contained initial imperfections on all compression members with a maximum out-of-plane magnitude limited by the smaller of: 1) the longest free edge length divided by 150; 2) 0.1 times the gap between the end of the compression member and the next adjoining member; or 3) 100% of the gusset-plate thickness;

- The model was proportionally loaded until failure. Typically, buckling can be identified when the analysis no longer converges to a solution. Shear failures are more difficult to identify, but typically occur when the plate exhibits load/displacement softening or when a strain threshold is exceeded after which the analysis predictions become unrealistic.

PART B—ALLOWABLE STRESS RATING AND LOAD FACTOR RATING

6B.5.2—Allowable Stress Method

In the Allowable Stress method, the capacity of a member is based on the rating level evaluated: Inventory level-Allowable Stress, or Operating level-Allowable Stress. The properties to be used for determining the allowable stress capacity for different materials follow. For convenience, the tables provide, where appropriate, the Inventory, Operating, and yield stress values. Allowable stress and strength formulas should be those provided herein or those contained in the AASHTO Standard Specifications. When situations arise that are not covered by these specifications, then rational strength of material formulae should be used consistent with data and plans verified in the field investigation. Deviations from the AASHTO Standard Specifications should be fully documented.

When the bridge materials or construction are unknown, the allowable stresses should be fixed by the Engineer, based on field investigations and/or material testing conducted in accordance with Section 5, and should be substituted for the basic stresses given herein.

6B.5.2.1—Structural Steel

The allowable unit stresses used for determining safe load capacity depend on the type of steel used in the structural members. When nonspecification metals are encountered, coupon testing may be used to determine a nominal yield point. When information on specifications of the steel is not available, allowable stresses should be taken from the applicable "Date Built" column of Tables 6B.5.2.1-1 and 6B.5.2.1-2.

Table 6B.5.2.1-1 gives allowable Inventory stresses and Table 6B.5.2.1-2 gives the allowable Operating stresses for structural steel. The nominal yield stress, F_y, is also shown in Tables 6B.5.2.1-1 and 6B.5.2.1-2. Tables 6B.5.2.1-3 and 6B.5.2.1-4 give the allowable Inventory and Operating Stresses for bolts and rivets. For compression members, the effective length, KL, may be determined in accordance with the AASHTO Standard Specifications or taken as follows:

KL = 75 percent of the total length of a column having riveted end connections

= 87.5 percent of the total length of a column having pinned end connections

The modulus of elasticity, E, for steel should be 29,000,000 lb/in.^{2}

If the investigation of shear and stiffener spacing is desirable, such investigation may be based on the AASHTO Standard Specifications.

C6B.5.2.1

When nonspecification materials are encountered, standard coupon testing procedures may be used to establish the nominal yield point. To provide a 95 percent confidence limit, the nominal yield point would typically be the mean coupon test value minus 1.65 standard deviations.

Mechanical properties of eyebars, high-strength eyebars, forged eyebars, and cables vary depending on manufacturer and year of construction. In the absence of material tests, the Engineer should carefully investigate the material properties using manufacturer's data and compilations of older steel properties before establishing the yield and allowable stresses to be used in load rating the bridge.

The formulas for the allowable bending stress in partially supported or unsupported compression flanges of beams and girders, given in Tables 6B.5.2.1-1 and 6B.5.2.1-2 are the corresponding formula based on given in Table 10.32.1A of the Allowable Stress Design portion of the AASHTO Standard Specifications. The equation in Table 6B.5.2.1-1 is to be used for an Inventory Rating and the equation in Table 6B.5.2.1-2 is to be used for an Operating Rating.

The previously used formulas are inelastic parabolic formulas which treat the lateral torsional buckling of a beam as flexural buckling of the compression flange. This is a very conservative approach for beams with short unbraced lengths. The flexural capacity is reduced for any unbraced length greater than zero. This does not reflect the true behavior of a beam. A beam may reach M_p with unbraced lengths much greater than zero. In addition, the formula neglects the St. Venant torsional stiffness of the cross-sections. This is a significant contribution to the latera1 torsional buckling resistance of rolled shapes, particularly older "I" shapes. The previous formulas must also be limited to the values of l/b listed. This limit is the slenderness ratio when the estimated buckling stress is equal to half the yield strength or 0.275 F_y in terms of an allowable stress. Many floor stringers will have unbraced lengths beyond this limit. If the formulas are used beyond these limits, negative values of the allowable stress can result.

The new formulas have no upper limit which allows the determination of allowable stresses for all unbraced lengths. In addition, the influence of the moment gradient upon buckling capacity is considered using the modifier C_b in the new formulas.

The specification formulas are based on the exact formulations of the lateral torsional buckling of beams. They are currently used in the AISC LRFD Specifications and other specifications throughout the world. They are also being used to design and rate steel bridges by the Load Factor method. Figures 6B.5.2.1-1 and 6B.5.2.1-2 given below show a comparison between the specification formulas and the previous specification formulas for two sections. Figure 6B.5.2.1-1 compares results for a $W18 \times 46$ rolled section. The new specification gives a much higher capacity than the previous specification. The difference is due to the inclusion of the St. Venant torsional stiffness, J, in the proposed specification. Figure 6B.5.2.1-2 shows a similar comparison for a plate-girder section. The section, labeled section 3, has 1.5 × 16 in. flanges and a $^{5}/_{16} \times 94$ in. web. The previous specification equation gives higher values than the new specification for large unbraced lengths. The previous specification is unconservative in this range. Both graphs show that, for small unsupported lengths, the new specification gives higher allowable stress values. The higher values result from the fact that there is an immediate reduction in capacity versus unsupported length in the previous specification.

Tables 6B.5.2.1-3 and 6B.5.2.1-4 contain the allowable inventory and operating stresses for low-carbon steel bolts, rivets, and high-strength bolts. For high-strength bolts (Table 6B.5.2.1-4), the values for inventory rating correspond to the Allowable Stress design values in the AASHTO Standard Specifications (Tables 10.32.3B and 10.32.3C). The values for the operating rating correspond to the inventory rating values multiplied by the ratio 0.75/0.55. The corresponding values for low-carbon steel bolts (ASTM A307) in Table 6B.5.2.1-3 are based on the values given in Table 10.32.3A of the Standard Specifications.

~~Guidance on the treatment of gusset plates can be found in Article C6A.6.12.1.~~

Specifications and guidance for determining the capacity of gusset plates can be found in Appendix L6B – Formulas for the Capacity, *C*, of Typical Bridge Components Based on the Load Factor Method. Allowable Inventory and Operating stresses for fasteners used in gusset plates can be found in Tables 6B.5.2.1-3 and 6B.5.2.1-4.

Guidance on considering the effects of deterioration on load rating of steel structures can be found in Article C6A.6.5.

6B.5.3—Load Factor Method

Nominal capacity of structural steel, reinforced concrete and prestressed concrete should be the same as specified in the load factor sections of the AASHTO Standard Specifications. Nominal strength calculations should take into consideration the observable effects of deterioration, such as loss of concrete or steel-sectional area, loss of composite action or corrosion.

Allowable fatigue strength should be checked based on the AASHTO Standard Specifications. Special structural or operational conditions and policies of the Bridge Owner may also influence the determination of fatigue strength.

C6B.5.3

Nominal capacities for members in the proposed guidelines are based on AASHTO's Standard Specifications contained in the load factor section. This resistance depends on both the current dimensions of the section and the nominal material strength.

Different methods for considering the observable effects of deterioration were studied. The most reliable method available still appears to be a reduction in the nominal resistance based on measured or estimated losses in cross-sectional area and/or material strengths.

At the present time, load factor methods for determining the capacity of timber and masonry structural elements are not available.

6B.5.3.1—Structural Steel

The yield stresses used for determining ratings should depend on the type of steel used in the structural members. When nonspecification metals are encountered, coupon testing may be used to determine yield characteristics. The nominal yield value should be substituted in strength formulas and is typically taken as the mean test value minus 1.65 standard deviations. When specifications of the steel are not available, yield strengths should be taken from the applicable "date built" column of Tables 6B.5.2.1-1 to 6B.5.2.1-4.

The capacity of structural steel members should be based on the load factor requirements stated in the AASHTO Standard Specifications. The capacity, *C*, for typical steel bridge members is summarized in Appendix L6B. For beams, the overload limitations of Article 10.57 of the AASHTO Standard Specifications should also be considered.

Except as specified in Appendix L6B.6.2.1, ~~T~~the Operating rating for welds, bolts, and rivets should be determined using the maximum strengths from Table 10.56A in the AASHTO Standard Specifications.

The Operating rating for friction joint fasteners (ASTM A325 bolts) should be determined using a stress of 21 ksi. A_1 and A_2 should be taken as 1.0 in the basic rating equation.

C6B.5.3.1

Guidance on considering the effects of deterioration on load rating of steel structures can be found in Article C6A.6.5.

~~Guidance on the treatment of gusset plates can be found in Article C6A.6.12.1.~~ Specifications and guidance for determining the capacity of gusset plates can be found in Appendix L6B – Formulas for the Capacity, *C*, of Typical Bridge Components Based on the Load Factor Method.

APPENDIX L6B—FORMULAS FOR THE CAPACITY, *C*, OF TYPICAL BRIDGE COMPONENTS BASED ON THE LOAD FACTOR METHOD

Add the following paragraphs to Appendix L6B – Formulas for the Capacity, *C*, of Typical Bridge Components Based on the Load Factor Method − at the end of Section L6B.2:

L6B.2.6—Gusset Plates

Main truss member gusset plates shall be load rated for shear, compression, and/or tension occurring in the vicinity of each connected member. The following sections below outline the necessary checks for performing a Load Factor rating for these gusset plates and their connections, which are based on the research performed under NCHRP Project 12-84 (NCHRP, 2013) that only considered LRFR. These provisions supersede the 2009 FHWA Guidelines for gusset-plate load ratings. All of the resistance factors used in this section have not been rigorously determined considering the base HS-20 live load model used for Load Factor rating. Additional reductions are not required in LFR based on the DL/LL ratio. Future research looking into the live load variability for truss systems may justify the use of lower resistance factors. An example gusset plate rating by LFR can be found in Appendix A.

L6B.2.6.1–Fasteners

Fasteners in bolted and riveted gusset plate connections shall be evaluated to prevent fastener shear and plate bearing failures.

The shear capacity of one fastener shall be taken as:

$$C = (\phi F)mA$$

where:

ϕF = shear capacity per fastener area of one fastener specified in Table L6B.2.6.1-1(ksi)

m = number of shear planes

A = cross-sectional area of one fastener (in.^{2}). For rivets, use the undriven diameter to calculate the area.

When bolts carrying loads pass through undeveloped fillers 0.25 in. or more in thickness in axially loaded connections, the bolt shear capacity shall be reduced by:

$$R = \left[\frac{(1+\gamma)}{(1+2\gamma)}\right]$$

where:

γ = A_f/A_p

A_f = sum of the area of the fillers on both sides of the connected plate (in.^{2})

A_p = smaller of either the connected plate area or the sum of the splice plate areas on both sides of the connected plate (in.^{2}). For chord splices, when considering the gusset plate(s), only the portion of the gusset plate(s) that overlaps the connected plate shall be considered in the calculation of the splice plate areas.

Alternatively, if fillers are extended beyond the connected parts and connected with enough bolts to develop the force in the fillers, the fillers may be considered developed. For rivets, the Undeveloped Filler Plate Reduction Factor, R_3, shall be considered as specified in Article 6A.6.12.5.1.

Table L6B.2.6.1-1—Shear Capacity Per Fastener Area

	ϕF (ksi) [a]
A307	18
A325 - threads included in shear plane	35
A325 – threads excluded from shear plane	43
A490 - threads included in shear plane	43
A490 – threads excluded from shear plane	53
Rivets	See Table 6A.6.12.5.1-1
[a] – Tabulated values shall be reduced by 20 percent in bearing-type connections whose length between extreme fasteners in each of the spliced parts measured parallel to the line of axial force exceeds 50 inches.	

The bearing capacity of the connected material at standard, oversize, short-slotted holes or long-slotted holes parallel to the applied force shall be taken as:

$$C = 0.9L_c tF_u \leq 1.8dtF_u \quad (10\text{-}166b)$$

where:

d = nominal diameter of the fastener (in.)

t = thickness of the gusset plate (in.)

F_u = specified minimum tensile strength of the gusset plate given in Table 10.2A (ksi)

L_c = clear distance between the holes or between the hole and the edge of the material in the direction of the applied bearing force (in.)

For determining the bearing capacity of long-slotted holes loaded perpendicular to the applied force, refer to Article 10.56.1.3.

The Operating rating for friction joint high-strength bolts should be determined according to the provisions of Article 6B.5.3.1.

L6B.2.6.2–Multi-Layered Gusset and Splice Plates

Where multi-layered gusset and splice plates are used, the resistances of the individual plates may be added together in determining the overall resistance provided that enough fasteners are present to develop the force in the layered gusset and splice plates.

L6B.2.6.3–Gusset Plate Shear Resistance

Gusset plates shall be load rated for shear yielding and shear rupture on relevant partial and full shear failure plane widths.

Yielding

$$C = \varphi_{vy}(0.58)F_y A_g \Omega$$

Rupture

$C = \varphi_{vu}(0.58)F_uA_n$

where:

φ_{vy} = resistance factor for gusset plate shear yielding taken as 1.00

φ_{vu} = resistance factor for gusset plate shear rupture taken as 0.85

Ω = shear reduction factor for gusset plates taken as 0.88

A_g = gross area of the plate resisting shear (in.^{2})

A_n = net area of the plate resisting shear (in.^{2})

F_y = specified minimum yield strength of the gusset plate given in AASHTO Table 10.2A (ksi)

F_u = specified minimum tensile strength of the gusset plate given in AASHTO Table 10.2A (ksi)

Partial shear planes shall only be checked around compression members and only shear yielding on partial shear planes shall be checked. The partial shear plane length shall be taken along adjoining member fastener lines between plate edges and other fastener lines. The following partial shear planes, as applicable, shall be evaluated to determine which shear plane controls:

- The plane that parallels the chamfered end of the compression member, as shown in Figure L6B.2.6.3-1;
- The plane on the side of the compression member that has the smaller framing angle between the that member and the other adjoining members, as shown in Figure L6B.2.6.3-2; and
- The plane with the least cross-sectional shear area if the member end is not chamfered and the framing angle is equal on both sides of the compression member.

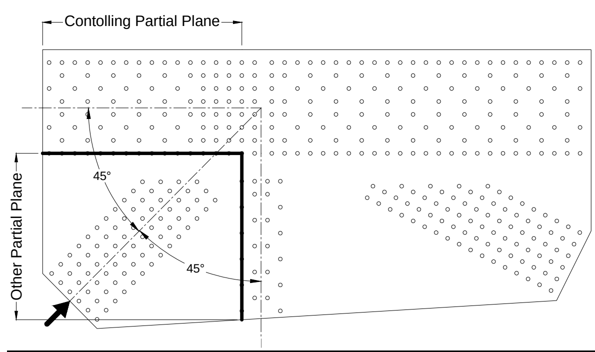

Figure L6B.2.6.3-1–Example of a Controlling Partial Shear Plane that Parallels the Chamfered End of the Compression Member Since that Member Frames in at an Angle of 45 Degrees to Both the Chord and the Vertical

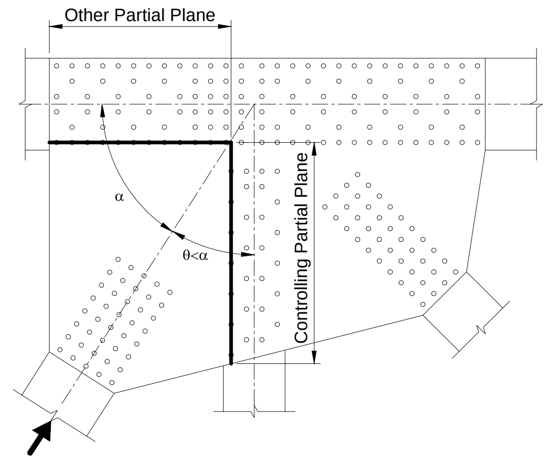

Figure L6B.2.6.3-2–Example of a Controlling Partial Shear Plane on the Side of a Compression Member Without a Chamfered End that has the Smaller Framing Angle between that Member and the Other Adjoining Members (i.e. θ < α)

L6B.2.6.4–Gusset Plate Compressive Resistance

Gusset plate zones in the vicinity of compression members shall be load rated for plate stability. The compressive capacity may be taken as the compressive capacity of a Whitmore section. The buckling capacity is dependent upon the chamfering of the member, the framing angles of the members entering the gusset, and the standoff distance of the compression member relative to the surrounding members; i.e. the distance, L_c. An example connection showing a typical chamfered member end and member framing angle is provided in Figure C6A.6.12.6.7-1.

The provisions of this article shall not be applied for the load rating of compression chord splices.

The compressive capacity of a Whitmore section shall be taken as:

$C = \varphi_{cg}\,(0.85)A_sF_{cr}$

in which:

- If $\frac{KL_c\sqrt{12}}{t} \le \sqrt{\frac{2\pi^2 E}{F_y}}$, then:

$$F_{cr} = F_y\left[1 - \frac{F_y}{4\pi^2 E}12\left(\frac{KL_c}{t}\right)^2\right]$$

- If $\frac{KL_c\sqrt{12}}{t} > \sqrt{\frac{2\pi^2 E}{F_y}}$, then:

$$F_{cr} = \frac{\pi^2 E}{12\left(\frac{KL_c}{t}\right)^2}$$

where:

φ_{cg} = resistance factor for gusset plate compression taken as 1.00

A_s = gross cross-sectional area of the Whitmore section determined based on 30 degree dispersion angles, as shown in Figure L6B.2.6.4-1 (in.^{2}). The Whitmore section shall not be reduced if the section intersects adjoining member bolt lines.

E = modulus of elasticity of gusset plate (ksi)

F_y = specified minimum yield strength of the gusset plate (ksi)

K = effective length factor in the plane of buckling taken as 0.50 for gusset plates

L_c = distance from the middle of the Whitmore section to the nearest member fastener line in the direction of the member, as shown in Figure L6B.2.6.4-1 (in.)

t = gusset plate thickness (in.)

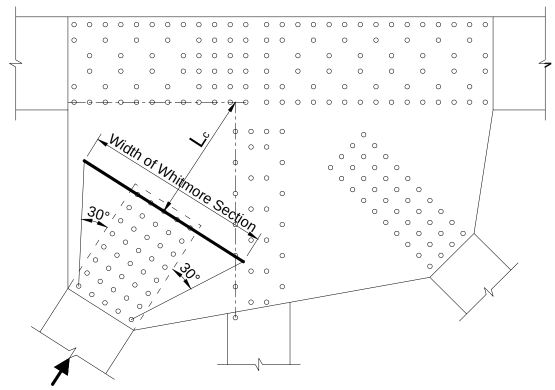

Figure L6B.2.6.4-1–Example Connection Showing the Whitmore Section for a Compression Member Derived from 30 Degree Dispersion Angles and the Distance L_c

L6B.2.6.5–Gusset Plate Tensile Resistance

Gusset plate zones in the vicinity of tension members shall be rated for yielding on the effective area of the Whitmore section and for block shear rupture.

The Whitmore section check shall not be applied for the load rating of tension chord splices.

Yielding

$C = \varphi_y F_y A_e$

in which:

A_e = effective cross-sectional area of the Whitmore section determined based on 30 degree dispersion angles, as shown in Figure L6B.2.6.5-1 (in.^{2}). The Whitmore section shall not be reduced if the section intersects adjoining member bolt lines.

$$= A_n + \beta A_g \leq A_g \quad (10\text{-}4w)$$

where:

φ_y = resistance factor for yielding of tension members taken as 1.00

A_n = net section of the member (in.^{2})

β = 0.0 for AASHTO M270 Grade 100/100W steels, or when holes exceed 1-1/4 inch in diameter

= 0.15 for all other steels and when holes are less than or equal to 1-1/4 inch in diameter

F_y = yield strength of the plate specified in AASHTO Table 10.2A (ksi)

Block Shear Rupture

The block shear rupture capacity shall be taken as:

$$C = \varphi_{bs} R_p (0.58 F_u A_{vn} + F_u A_{tn}) \le \phi_{bs} R_p (0.58 F_y A_{vg} + F_u A_{tn})$$

where:

φ_{bs} = resistance factor for block shear rupture taken as 0.85

A_{vg} = gross area along the plane resisting shear stress (in.^{2})

A_{vn} = net area along the plane resisting shear stress (in.^{2})

A_{tn} = net area along the plane resisting tension stress (in.^{2})

F_y = yield strength of the plate specified in AASHTO Table 10.2A (ksi)

F_u = tensile strength of the plate specified in AASHTO Table 10.2A (ksi)

R_p = reduction factor for holes taken equal to 0.90 for bolt holes punched full size and 1.0 for bolt holes drilled full size or subpunched and reamed to size

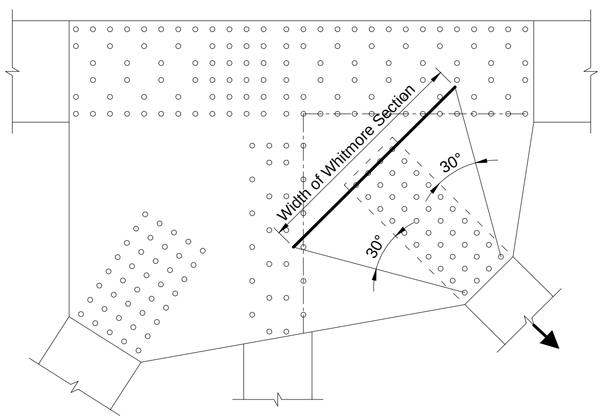

Figure L6B.2.6.5-1–Example Connection Showing the Whitmore Section for a Tension Member Derived from 30 Degree Dispersion Angles

L6B.2.6.6−Chord Splices

Gusset plates that splice two chord sections together shall be checked using a section analysis considering the relative eccentricities between all plates crossing the splice and the loads on the spliced plane. This Article is only intended to cover the load rating of chord splices that occur within the gusset plates. For gusset plates also serving the role of a chord splice, the forces from all members framing into the connection must be considered. The chord splice forces are the resolved axial forces acting on each side of the spliced section, as illustrated in Figure C6A.6.12.6.9-1. The chord splice should be investigated for the larger of the two resolved forces on either side of the splice.

The capacity equations in this article assume the gusset and splice plates behave as one combined spliced section to resist the applied axial load and eccentric bending that occurs due to the fact that the resultant forces on the section are offset from the centroid of the combined section, as illustrated in Figure C6A.6.12.6.9-2 (substitute e for e_p in the figure). The combined spliced section is treated as a beam and the capacity is determined assuming the stress in the combined section at the limit of usable capacity is equal to the specified minimum yield strength of the gusset plate if the slenderness limit for the spliced section given below is met, which will typically be the case.

The application of the Whitmore section check specified in Article L6B.2.6.4 is not considered applicable for the load rating of a compression chord splice.

For compression chord splices, the compressive capacity of the spliced section shall be taken as:

$$C = \phi_{cs} F_{cr} \left(\frac{S_g A_g}{S_g + eA_g} \right)$$

in which:

F_{cr} = stress in the spliced section at the usable limit of capacity (ksi). F_{cr} shall be taken as the specified minimum yield strength of the gusset plate when the following equation is satisfied:

$$\frac{KL_{splice}\sqrt{12}}{t_g} < 25$$

(Note: if the preceding equation is not satisfied, the Engineer will need to derive a reduced value of F_{cr} to account for possible elastic buckling of the gusset plate within the splice.)

where:

φ_{cs} = resistance factor for gusset plate chord splices taken as 1.00

A_g = gross area of all plates in the cross-section intersecting the spliced plane (in.^{2})

e = distance between the centroid of the cross-section and the resultant force perpendicular to the spliced plane (in.)

K = effective column length factor taken as 0.50 for chord splices

L_{splice}= center-to-center distance between the first lines of fasteners in the adjoining chords as shown in Figure L6B.2.6.6-1 (in.)

S_g = gross section modulus of all plates in the cross-section intersecting the spliced plane (in.^{3})

t_g = gusset plate thickness (in.)

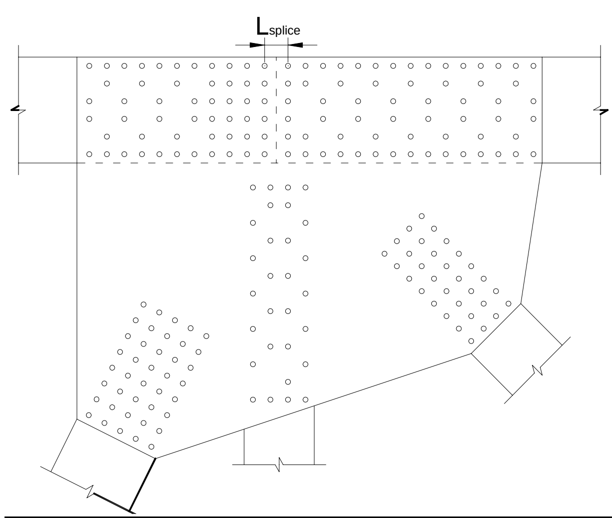

Figure L6B.2.6.6-1−Example Connection Showing Chord Splice Parameter, L_{splice}

For tension chord splices, the tensile capacity of the spliced section shall be taken as the lesser of the values given by the following equations:

$$C = \phi_{cs} F_y \left(\frac{S_g A_g}{S_g + eA_g} \right)$$

$$C = \phi_{cs} F_u \left(\frac{S_n A_n}{S_n + eA_n} \right)$$

where:

φ_{cs} = resistance factor for gusset plate chord splices taken as 1.00

A_g = gross area of all plates in the cross-section intersecting the spliced plane (in.^{2})

A_n = net area of all plates in the cross-section intersecting the spliced plane (in.^{2})

e = distance between the centroid of the cross-section and the resultant force perpendicular to the spliced plane (in.)

F_y = specified minimum yield strength of the gusset plate (ksi)

F_u = specified minimum tensile strength of the gusset plate (ksi)

S_g = gross section modulus of all plates in the cross-section intersecting the spliced plane (in.^{3})

S_n = net section modulus of all plates in the cross-section intersecting the spliced plane (in.^{3})

The yielding check on the effective area of the Whitmore section specified in Article L6B.2.6.5 is not considered applicable for the load rating of a tension chord splice; however, tension chord splice members shall be checked for block shear rupture as specified in Article L6B.2.6.5.

L6B.2.6.7−Edge Slenderness

Gusset plates should not be load rated based on any edge slenderness criteria. Refer to Article C6A.6.12.6.10 for additional guidance.

L6B.2.6.8–Refined Analysis

A refined simulation analysis using the finite element method may be employed to determine the nominal resistance of a gusset-plate connection at the strength limit state in lieu of satisfying the requirements specified in Articles L6B.2.6.3 through L6B.2.6.6. Refer to Article C6A.6.12.6.11 for a list of some suggested model attributes, which is not considered exhaustive.

2013 AASHTO BRIDGE COMMITTEE AGENDA ITEM: 42

SUBJECT: The Manual for Bridge Evaluation: Section 2, Various Articles (T18-2)

TECHNICAL COMMITTEE: T-18 Bridge Management, Evaluation and Rehabilitation

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☒ **REVISION** ☒ **ADDITION** ☐ **NEW DOCUMENT**

☐ **DESIGN SPEC** ☐ **CONSTRUCTION SPEC** ☐ **MOVABLE SPEC**
☒ **MANUAL FOR BRIDGE EVALUATION** ☐ **SEISMIC GUIDE SPEC** ☐ **BRIDGE ELEMENT INSP GUIDE**
☐ **OTHER**

DATE PREPARED: 1/8/13
DATE REVISED:

AGENDA ITEM:

Item #1

Revise Section 2: BRIDGE FILES(RECORDS) TABLE OF CONTENTS as follows:

SECTION 2: ~~BRIDGE FILES(RECORDS)~~BRIDGE FILES AND DOCUMENTATION
TABLE OF CONTENTS

Item #2

Insert new Article 2.0 as follows:

2.0—INTRODUCTION

Maintaining bridges safe for public travel is of utmost importance to transportation officials. In order to ensure public safety, management of in-service highway bridges requires the collection and maintenance of accurate, up-to-date, and comprehensive information for each bridge. The information includes data that does not typically change over the life of a bridge, data that is updated by field inspections, and data that is derived or calculated to assess specific attributes such as scour, vulnerability to extreme events, and safe load-carrying capacity.

Data that does not typically change includes information such as the length and width of the structure, and the structure type. Data that is updated as a result of field inspections includes condition information on the structural components and elements, clearances and changes to dead load, and the identification of conditions that introduce a potential risk to the bridge such as debris accumulation around substructure units and scour or erosion.

Information is organized in a file for each bridge. The bridge file includes a description of the characteristics and conditions of the structure, calculations for determining scour vulnerability (if over water), a determination of the load carrying capacity including computations substantiating reduced load limits, and details of any damage and alterations or repairs to the structure.

The information in a bridge file provides a cumulative history of the structure that is useful to review prior to conducting a bridge inspection, rating, or evaluation.

The information in a bridge file may exist electronically, on paper, or in external documents that are appropriately referenced within each bridge file or manual. The external documents may apply to multiple bridges and exist in various locations.

The bridge file provides information that directly relates to requirements of the National Bridge Inspection Standards (Article 2.1) and other supplemental information that bridge owners may find useful in managing bridges (Article 2.2).

Item #3

Replace Article 2.1 with the following:

2.1—PROVISIONS TO SUPPORT THE NBIS REQUIREMENTS

The Inspection Program Manager is responsible to maintain an inventory of all bridges that are subject to the NBIS, in accordance with coding guidance issued by the Federal Highway Administration. Certain data is defined by the FHWA that must be collected and retained. This data is typically retained within a computer database. Summaries of the inventory and evaluation data may be included in bridge files. The data accurately captures specific, key information to include bridge identification, location, structural attributes, dimensions, clearances, condition, and load carrying capacity.

A bridge file is to be prepared as required by the NBIS and described in this AASHTO Manual for Bridge Evaluation. Specifically, maintain reports on the results of bridge inspections and note any actions taken to address the findings of the inspections. Maintain relevant maintenance and inspection data and use it to assist with the assessment of current bridge conditions. Additionally, document observations and measurements needed to determine the physical and functional condition of the bridge. Identify any changes from initial or previously recorded conditions, and other actions needed ensure that the structure continues to satisfy present structural service requirements.

The format used to document the information in bridge files may vary significantly. If the information is not available in a consolidated inspection report, look for it elsewhere in the inspection file or as referenced to another location. Bridge owners may make generalized reference to where digital information is located in their own manuals or other files rather than individually in each bridge file.

Include the following specific bridge information. The level of documentation will vary depending on the complexity and condition of a structure; however, always include the following information that is common to all bridges.

Item #4

Insert the following new articles:

2.1.1—General File Information

- A General Plan and Elevation drawing or a sketch depicting the layout of the bridge, if available.
- A clear and understandable approach to labeling members and elements that enables an assessment of the inspection process and completeness.
- Inspection and inventory data as defined by the FHWA for reporting to the National Bridge Inventory. This data is often referred to as Structure Inventory and Appraisal information, or SI&A.
- Photographs showing a top view of the roadway across the bridge, a side elevation view, and an under view of the main or typical span superstructure configuration. Photographs necessary to show major defects, posting restrictions, and other important features should be included.
- A history of any structural damage.

2.1.2—Field Inspection Information

Inspection reports provide a chronological record of the date and type of all inspections performed on the bridge. Document each inspection conducted for a bridge and retain in the bridge file. Document the observations and findings from each inspection. Identify the Team Leader responsible for the inspection and report, and the date the inspection was conducted. Observations include a description of conditions of bridge members and the identification of factors that require further review, close monitoring, or additional attention during inspections:

- Brief narrative descriptions of general and component/element conditions with detail to justify a condition rating of 5 or less for NBI items and Condition State 3 or more for elements.
- Sketches or photographs, as appropriate, of elements or members showing typical and deteriorated conditions.
- Sketches documenting remaining section of components with sufficient detail to facilitate determination of the

load capacity. These sketches may be part of the load rating documentation.

- Notations of actions taken to address previous inspection findings.

2.1.3—Critical Findings and Actions Taken

Provide a detailed description and photographs of the specific critical finding(s) sufficient to document safety or structural concerns. Identify appropriate immediate actions or follow-up inspections. Include a record of the actions taken to resolve or monitor the critical finding(s).

2.1.4—Waterway Information

Provide channel cross-sections or sketches, soundings and stream profiles as needed to provide adequate information on the stability of the waterway and allow for adequate assessment of the risk to the structure. Update channel information periodically and when otherwise necessary. Perform a historical comparison to determine the extent of any scour, channel shifting, degradation, or aggradation of the channel. Determine and document a frequency for obtaining and updating these measurements, depending on an assessment of the bridge and stream characteristics. Consider the potential for lateral migration of the stream channel or head-cutting in determining the extent of channel documentation. A single cross section or sketch at one face may be appropriate for historically stable channels and embankments. Evaluate the need for obtaining cross sections for pipes and box culverts and structures with small drainage areas on a case-by-case basis. The BIRM provides additional information on inspecting channels.

2.1.5—Significant Correspondence

Provide correspondence and agreements regarding inspection responsibility, ownership, maintenance responsibilities with other agencies, or other issues that have an impact on the ability to ensure that thorough and timely inspections are completed.

2.1.6—Other Inspection Procedures or Requirements

Provide procedures for specific types of inspections (fracture critical, underwater and complex bridges). Address those items that need to be communicated to an inspection team leader to ensure a successful bridge inspection.

Provide procedures to document special access needs, inspection equipment, structural details, inspection methods and any special qualifications required of inspecting personnel. Overall inspection procedures may exist in a bridge inspection manual which address common aspects of these more complex inspections; however, if there are items unique to that structure that are not covered in the overall inspection procedures, provide written procedures specific to that fracture critical, underwater, and complex bridge inspection.

Document the following items, either in bridge specific inspection procedures, or by referring to general inspection procedures:

1. Identify each of the fracture critical, complex features and underwater members, and any elements that need special attention during those inspections, preferably on plan sheets, drawings or sketches.
2. Describe the inspection method(s) special access needs (under bridge inspection truck, climbing, etc. special inspection equipment (NDE) and frequency to be used for the elements.
3. Provide information regarding proximity necessary to details, such as “arm’s length” or “hands-on”.
4. Provide special qualifications required of inspection personnel by the Program Manager, if any. Section 4 of this manual provides additional information on inspection plans for bridges.

2.1.7—Load Rating Documentation

Provide dated load rating results along with the identification of the analyst to determine the safe load carrying capacity of the bridge and, where necessary, the load limits for posting. Include the load rating results which clearly identify the loads and methodology used in the analysis, a general statement of the results of the analysis, identification of members that were found to control the load rating, and any other modifying factors that were assumed in the analysis. Include updates to the calculations as needed to reflect changes in the condition of structural members, changes to the structural configuration, strength of members, or dead load, and changes to the legal live load that may alter the load rating result. . If calculations cannot be provided due to a lack of information (missing plans, unknown materials, etc.), provide documentation for justification of determined load rating. If a field load test is used to establish the load carrying capacity of the bridge, provide reports that describe and document the testing process and results.

2.1.8—Posting Documentation

Provide a summary of posting recommendations and actions taken for the bridge and date of posting.

2.1.9—Scour Assessment

Document the assessment conducted to determine the scour vulnerability of the bridge. Provide a clear reference to an alternate location if documentation is available outside of the inspection file.

2.1.10—Scour Plan of Action

For scour critical bridges, provide a copy of a plan of action, or a clear reference to the plan of action if the documentation is available in a location other than the inspection file. The plan of action is used to monitor known and potential deficiencies and address or monitor critical scour related findings.

Item #5

Delete Article C2.1

Item #6

Delete Article 2.2 and subarticles and replace with the following:

2.2—SUPPLEMENTAL DOCUMENTATION IN BRIDGE FILE

2.2.1—General

Retain the following supplemental documentation in the bridge file at the discretion of the Bridge Inspection Program Manager and subject to availability, especially for older bridges. Additional information not listed in this section may be included in the file and contain information bridge owners believe is necessary to operate their Bridge Management Systems. Some of this information may be referenced in the bridge file or references provided to outside source locations.

2.2.2—Plans and Drawings

Include one set of final drawings showing the "as-built" condition of the bridge. Construction shop drawings may be included.

2.2.3—Construction Documentation

Include construction documentation of relevant as-built information regarding the structure, including reference to material certifications and tests performed during construction activities such as pile driving, concrete placement, and prestressing operations.

Retain all pertinent certificates for the type, grade, and quality of materials incorporated in the construction of the bridge, such as steel mill certificates, concrete delivery slips, and other Manufacturers' certifications, in accordance with applicable policies and the appropriate statute of limitations.

Reference in the file any reports of nondestructive and laboratory tests of materials incorporated in the bridge during construction.

Retain one complete copy or reference to the special technical specifications under which the bridge was built. Reference the edition and date of the general technical specification in the bridge file.

2.2.4—Original Design Documentation

The original design calculations and documentation for the bridge should be retained. The AASHTO Bridge Design Specifications version and any project specific assumptions and criteria should be recorded.

2.2.5—Unique Considerations

Provide information as necessary on other items that may need to be addressed depending on each unique situation. If appropriate, include special coordination procedures prior to inspection (Coast Guard, security, operations personnel, etc.), safety concerns (rattlesnakes, bats, etc.), and optimum periods of the year to inspect the bridge (lake draw down, canal dry time, snow, ice, bird nesting seasons, etc.).

Identify special access needs or equipment necessary to gain the access required to inspect the features (traveler system, climbing, etc.).

Emphasize and highlight special structural details or situations, such as fatigue-prone details, pins and hangers in nonredundant systems, cathodic protection, and weathering or brittle steels,

Document any unusual environmental conditions that may have an effect on the structure, such as salt spray and industrial gases.

2.2.6—Utilities and Ancillary Attachments

Include information on utilities or ancillary attachments that are attached to the structure or that affect access to portions of the bridge. Note a utility in the immediate area, though not fastened to the bridge, e.g., a sewer line crossing the right-of-way and buried in the channel beneath the bridge. The type of connection should be noted.

2.2.7—Maintenance and Repair History

Include a chronological record documenting the maintenance and repairs to the bridge since its initial construction. Include details such as significant dates, description of project, contractor, cost, contract number.

Document the surface protective coatings used, including surface preparation, application methods, dry-film thickness and types of paint, concrete and timber sealants, and other protective membranes. This information may be in the referenced original design file.

2.2.8—Additional Waterway Documentation

When available for structures over waterways, include a chronological history of major hydraulic events, high-water marks, and scour activity in the bridge file. Channel profiles, mean high water levels, debris accumulation, and storm surge data are useful information for effectively managing bridges over waterways.

2.2.9—Traffic Data

When the information is available, include the frequency and type of vehicles using the bridge and their historical variations in the bridge file. Vehicle weight data, such as weigh-in-motion (WIM) data for a bridge can aid in the determination of bridge specific load factors when refining LRFR load capacity calculations.

Item #7

Delete Article C2.2

Item #8

Delete Article 2.3, subarticles and commentary

Item #9

Delete Article 2.4, subarticles and commentary

Item #10

Delete Article 2.5 and subarticles

Item #11

Delete Article 2.6

OTHER AFFECTED ARTICLES:

None

BACKGROUND:

The AASHTO Manual for Bridge Evaluation (MBE) and its predecessors have provided bridge owners with guidance on the practice of the inspection and load rating of bridges. In 2004 the AASHTO Manual for Bridge Evaluation was incorporated into the Code of Federal Regulations (CFR) in regards to the National Bridge Inspection Standards (NBIS) by reference.

In 2010 the FHWA developed quality assurance criteria to ensure States compliance with the CFR, referred to as the “23 Metrics”.

The proposed changes to the Manual for Bridge Evaluation in this ballot item are an effort to clarify for bridge owners and the FHWA which bridge inspection and load rating practices information contained within the MBE is applicable in the CFR.

ANTICIPATED EFFECT ON BRIDGES:

The changes should improve the understanding of the practice of bridge inspection and load rating.

REFERENCES:

None

OTHER:

None

2013 AASHTO BRIDGE COMMITTEE AGENDA ITEM: 43

SUBJECT: The Manual for Bridge Evaluation: Section 6, Article 6A.4.1 (T18-6)

TECHNICAL COMMITTEE: T-18 Bridge Management, Evaluation and Rehabilitation

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☐ **REVISION** ☒ **ADDITION** ☐ **NEW DOCUMENT**

☐ **DESIGN SPEC** ☐ **CONSTRUCTION SPEC** ☐ **MOVABLE SPEC**
☒ **MANUAL FOR BRIDGE EVALUATION** ☐ **SEISMIC GUIDE SPEC** ☐ **BRIDGE ELEMENT INSP GUIDE**
☐ **OTHER**

DATE PREPARED: 11/9/12
DATE REVISED: 4/15/13

AGENDA ITEM:

Item #1

Add the following paragraph to the end of Article 6A.4.1—Introduction:

The live load factors given in Articles 6A.4.3.2, 6A.4.4.2 and 6A.4.5.4 are established assuming that demands based on the two more lane loaded distribution factor will always govern over the demands based on single lane distribution factor for design loads, legal loads, and routine permits. However, in the unusual case where the single lane distribution factor is greater than the two-lane distribution factor, the higher distribution factor shall be used.

The multiple presence factor listed in the LRFD Specification does not apply for permit trucks. Therefore, the multiple presence factor should not be considered when establishing the load demands for routine permit trucks, when the single lane distribution governs.

Item #2

Add the following paragraph to the end of Article C6A.4.1:

Calibration of LRFR load factors was done using the simplified live load distribution factors of the LRFD Specification. During this calibration, it was assumed that the demands based on two or more lane loaded scenario will govern over the demands based on the single lane loaded case for multi-lane bridges for legal and routine permit trucks. However, there may be a few instances where the single lane distribution factor (based on the simplified equations given in the LRFD Specification, or based on the Lever rule method, or based on the rigid cross-section distribution of exterior girder) may exceed the two-lane distribution factor. In these situations, to be conservative, use of the higher live load distribution factor with the load factors in the MBE is recommended.

The multiple presence factor of 1.2 incorporated within the single-lane distribution factor equation accounts for the presence of a vehicle heavier than the design or rating truck in the lane. In other words MPF listed within the LRFD specification is applicable for design loadings (HL93) and legal trucks only, but not for permit trucks. As a result, the MPF for permit trucks was not considered during the calibration of permit loads for the LRFR Specification. The MPF should not be considered when establishing the load demand of either routine or special permit truck. It is important to note that when comparing the simplified live load distribution factors to establish the demand of routine permit trucks, MPF of 1.2 included within the simplified single lane distribution factor equation should be removed.

MBE Article 6A.4.5.4.2c addresses the use of refined analysis for permits and provides an adjustment to the load factors when using refined method of analysis (such as 3-D analysis) methods. Again, it is not necessary to incorporate the MPF for permit trucks during evaluation.

OTHER AFFECTED ARTICLES:

None

BACKGROUND:

LRFD Specification Article 3.6.1.1 addresses how the demand should be established for the HL93 loading. It states that "Unless specified otherwise herein, the extreme live load force effect shall be determined by considering each possible combination of number of loaded lanes multiplied by a corresponding multiple presence factor (MPF) to account for the probability of simultaneous lane occupation by the full HL93 design live load. In lieu of site specific data, the values in Table 3.6.1.1.2-1: …." In essence, it states that when establishing the HL93 demand for design, the largest demand either from the single lane loaded case or from the two or more lane loaded case should be considered.

The LRFR Specification addresses not only the HL93 loading, but also legal and permit loading. During LRFR ratings a few issues on MPF use were raised, which require some clarifications.

Calibration of LRFR load factors for legal and routine permit trucks were done utilizing the demands established using simplified live load distribution factor equations given in the LRFD Specification. In the load factor calibration, it was assumed that a side by side truck configuration will always produce higher load demands. As a result, the two lane live load distribution factor was used in the calibration process. However, there may be a few instances where the single lane distribution factor (based on the simplified equations given in the LRFD Specification, or based on the Lever rule method, or based on the rigid cross-section distribution of exterior girder) may exceed the two-lane distribution factor. In these situations, it would be conservative to use the higher live load distribution factor with the load factors in the MBE.

The MPF of 1.2 incorporated within the LRFD single-lane distribution factor accounts for the presence of a heavier non-permit truck that exceeds the legal weight in a single lane. Therefore, the multiple presence factor listed in the LRFD Specification does not apply for permit trucks. The MPF for permit trucks was not considered during the calibration of loads for the LRFR Specification.

This revision clarifies the proper use of the governing distribution factor for multi-lane bridges for legal and routine permit truck ratings, particularly for cases where the single lane distribution factor may govern over the two lane distribution. It recommends the use of the governing distribution factor even though the LRFR live load factor calibration was based on a two lane distribution factor. Furthermore, this clarifies that the MPF is not applicable for permit trucks.

ANTICIPATED EFFECT ON BRIDGES:

Eliminates potential confusion and promotes more consistency in load ratings as the distribution factor has a major influence on load rating results.

REFERENCES:

None

OTHER:

None

2013 AASHTO BRIDGE COMMITTEE AGENDA ITEM: 44

SUBJECT: The Manual for Bridge Evaluation: Section 6, Various Articles (T18-8)

TECHNICAL COMMITTEE: T-18 Bridge Management, Evaluation and Rehabilitation

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☒ **REVISION**	☒ **ADDITION**	☐ **NEW DOCUMENT**
☐ **DESIGN SPEC**	☐ **CONSTRUCTION SPEC**	☐ **MOVABLE SPEC**
☒ **MANUAL FOR BRIDGE EVALUATION**	☐ **SEISMIC GUIDE SPEC**	☐ **BRIDGE ELEMENT INSP GUIDE**
	☐ **OTHER**	

DATE PREPARED: 11/9/12
DATE REVISED: 4/15/13

AGENDA ITEM:

Item #1

Revise Article 6A.4.4.2.3a as follows:

Generalized live load factors for the Strength I limit state are specified in Table 6A.4.4.2.3a-1 for routine commercial traffic on structures other than buried structures. If, in the Engineer's judgment, an increase in the live load factor is warranted due to conditions or situations not accounted for in this Manual when determining the safe legal load, the Engineer may increase the factors in Table 6A.4.4.2.3a-1, not to exceed the value of the factor multiplied by 1.3.

Item #2

Revise Article 6A.4.4.2.3b as follows:

Generalized live load factors for the Strength I limit state are given in Table 6A.4.4.2.3b-1 for the NRL rating load and posting loads for specialized hauling vehicles satisfying Formula B specified in Article 6A.8.2~~.~~ on structures other than buried structures. If in the Engineer's judgment, an increase in the live load factor is warranted due to conditions or situations not accounted for in this Manual when determining the safe legal load, the Engineer may increase the factors in Table 6A.4.4.2.3-1, not to exceed the value of the factor multiplied by 1.3.

Item #3

Revise Article 6A.4.5.4.2 as follows:

Table 6A.4.5.4.2a-1 specifies live load factors for permit load rating that are calibrated to provide a uniform and acceptable level of reliability~~.~~ on structures other than buried structures. Load factors are defined based on the permit type, loading condition, and site traffic data.

Permit load factors given in Table 6A.4.5.4.2a-1 for the Strength II limit state are intended for spans having a rating factor greater than 1.0 when evaluated for AASHTO legal loads. Permit load factors are not intended for use in load-rating bridges for legal loads except as allowed under Article 6A.2.3.1.

Item #4

In Article 6A.4.5.4.2c, revise the last paragraph as follows:

When special permits mixed with traffic are evaluated using a refined analysis, a live load factor γ_L =1.0 shall be applied on the permit truck while a γ_L =1.10 shall be applied on the governing AASHTO or state legal truck placed in the adjacent lane.

Item #5

Revise Table 6A.4.5.4.2a-1as follows:

Table 6A.4.5.2a-1 Permit Load Factors: γ_L.

Permit Type	Frequency	Loading Condition	DF^a	ADTT (one direction)	Load Factor by Permit Weight Ratio[b]		
					GVW / AL < 2.0 (kip/ft)	2.0 < GVW / AL < 3.0 (kip/ft)	GVW / AL > 3.0 (kip/ft)
Routine or Annual Routine or Annual	Unlimited Crossings	Mix with traffic (other vehicles may be on the bridge)	Two or more lanes	>5000	1.4	1.35	1.30
				=1000	1.35	1.25	1.20
				<100	1.30	1.20	1.15
	Unlimited Crossings (Reinforced Concrete Box Culverts)[c]	Mix with traffic (other vehicles may be on the bridge)	One lane	All ADTTs	1.40		
					All Weights		
Special or Limited Crossing	Single-Trip	Escorted with no other vehicles on the bridge	One lane	N/A	1.10		
	Single-Trip	Mix with traffic (other vehicles may be on the bridge)	One lane	ALL ADTTs	1.20		
	Multiple-Trips (less than 100 crossings	Mix with traffic (other vehicles may be on the bridge)	One lane	ALL ADTTs	1.40		

Notes:

[a] *DF* = LRFD-distribution factor. When one-lane distribution factor is used, the built-in multiple presence factor should be divided out.

[b] Permit Weight Ratio = GVW/AL;. GVW = Gross Vehicle Weight; AL = Front axle to rear axle length; Use only axles on the bridge.

[c] Refer to Article 6A.5.12

Item #6

In Article 6A.5.12.10.3, revise the 2nd bullet by moving the sentence from the commentary to the specification as follows:

- *Legal Load and Permit Load*—Only the single-lane loaded condition needs to be checked for legal load and permit load ratings, even when the culvert carries multiple lanes. The 1.2 single-lane multiple presence factor should not be applied to this loading.

  A single legal load factor of 2.00 shall be specified for all traffic volumes. For routine as well as special permits, utilize the load factors provided in Table 6A.4.5.4.2a-1, without applying the multiple presence factor.

Item #7

In Article C6A.5.12.10.3, delete the last paragraph.

Item #8

Add the following to the 2nd paragraph in Article 6A.4.2.4:

System factors that correspond to the load factor modifiers in the *AASHTO LRFD Bridge Design Specifications* should be used. The system factors in Table 6A.4.2.4-1 are more conservative than the LRFD design values and may be used at the discretion of the evaluator until they are modified in the *AASHTO LRFD Bridge Design Specification*~~.~~, however, when rating nonredundant superstructures for legal loads using the generalized load factors given in Article 6A.4.4.2.3, the system factors from Table A 6A.4.2.4-1 shall be used to maintain an adequate level of system safety. The system factor for riveted and bolted gusset plates for all force effects shall be taken as 0.90.

OTHER AFFECTED ARTICLES:

None

BACKGROUND:

Load rating provisions for concrete box culverts were adopted by AASHTO in 2011, which provide specially calibrated live load factors for culvert ratings for design loads, legal loads and permit loads. In the past Section 6 only provided live load factors for rating bridge structures. These revisions will guide the user to the appropriate section for load rating of culverts and prevent any misapplication of load factors for LRFR ratings.

The 2012 Interims provides guidance on the use of refined analysis for permit evaluation. Item #4 clarifies that a state legal load may be used as the adjacent vehicle if it is the governing vehicle for the state, in place of the AASHTO truck. Item # 5 expands Table 6A.4.5.2a-1 to cover routine permit load factors for box culverts. Item #8 requires the use of system factors when using the reduced generalized legal load factors adopted in 2012. This change is proposed to provide consistency with Article *C6A.4.4.2.3*.

ANTICIPATED EFFECT ON BRIDGES:

The revisions will eliminate potential confusion that may exist in selecting live load factors for permits following the addition of culvert rating provisions into Section 6 in 2011.

REFERENCES:

None

OTHER:

None

2013 AASHTO BRIDGE COMMITTEE AGENDA ITEM: 45

SUBJECT: The Manual for Bridge Evaluation: Section 6, Articles 6A.6.3 and C6A.6.3 (T18-9)

TECHNICAL COMMITTEE: T-18 Bridge Management, Evaluation and Rehabilitation/ T-14 Steel

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☒ **REVISION** ☒ **ADDITION** ☐ **NEW DOCUMENT**

☐ **DESIGN SPEC** ☐ **CONSTRUCTION SPEC** ☐ **MOVABLE SPEC**
☒ **MANUAL FOR BRIDGE EVALUATION** ☐ **SEISMIC GUIDE SPEC** ☐ **COASTAL GUIDE SPEC**
☐ **OTHER**

DATE PREPARED: 9/26/12
DATE REVISED:

AGENDA ITEM:

Item #1

Revise Article 6A.6.3 as follows:

Except as specified herein, ~~R~~resistance factors, φ, for steel members, for the strength limit state, shall be taken as specified in LRFD Design Article 6.5.4.2.

If the year of construction is prior to 1991, the resistance factor for axial compression for steel, φ_c, shall be taken as 0.90 for built-up compression members.

Item #2

Add the following paragraph to the end of Article C6A.6.3:

The resistance factor, φ_c, for built-up members subject to axial compression is reduced from 0.95 to 0.90 if the year of construction is prior to 1991 to appropriately reflect the fact that steel built-up compression members may have been fabricated from universal mill plate. Such columns are contained in the data band of lowest strength reflected by SSRC Column Category 3P. Since only one column curve based on SSRC Column Category 2P is used for all columns, earlier versions of the LRFD Design Specification specified a lower φ_c of 0.90 to reflect the larger data spread and coefficient of variation found in the data for all column categories. Since the production of universal mill plates was discontinued around 1990, and changes in steelmaking practice since that time have resulted in materials of higher quality and much better defined properties, φ_c was raised from 0.90 to 0.95 in the LRFD Design Specification (2013).

OTHER AFFECTED ARTICLES:

None

BACKGROUND:

In the original research on the probability-based strength of steel columns (Bjorhovde, 1972, 1978, 1988), three column curves were recommended. The three column curves were the approximate means of bands of strength curves for columns of similar manufacture based on extensive analyses and confirmed by full-scale tests. Hot-formed and cold-formed heat treated HSS columns fell into the data band of highest strength (SSRC Column Category 1P), while built-up wide-flange columns made from universal mill plates were included in the data band of lowest strength (SSRC Column Category 3P). The largest group of data, however, clustered around SSRC Column Category 2P. Thus, it was decided to use only one column curve, SSRC Column Category 2P, for all column types. The AASHTO LRFD Specification followed suit. The use of only one column curve results in a larger data spread and thus a larger coefficient of variation, and so a resistance factor $\varphi_c = 0.85$ was adopted in the original AISC LRFD Specification for the column equations to achieve a level of reliability comparable to that of beams. Resistance factors in the AASHTO LRFD Specification are typically set at a level that is 0.05 higher than those in the AISC LRFD Specification; thus, φ_c was set to 0.90 in the original AASHTO LRFD Specification.

Since that time, significant additional analyses and tests, as well as changes in practice, have demonstrated that an increase in φ_c from 0.85 to 0.90 in the AISC LRFD Specification was warranted, indeed even somewhat conservative (Bjorhovde, 1988). Significant changes in industry practice since that time have included the following: (1) built-up shapes are no longer manufactured from universal mill plates; (2) the most commonly used structural steel is now ASTM A 709 Grade 50 or 50W, with a specified minimum yield stress of 50 ksi; and (3) changes in steelmaking practice have resulted in materials of higher quality and much better defined properties. The level and variability of the yield stress thus have led to a reduced coefficient of variation for the relevant material properties (Bartlett et al., 2003). As a result, for consistency, a separate 2013 Agenda Item has been developed to raise φ_c for steel members (or components) subject to axial compression from 0.90 to 0.95 in the AASHTO LRFD Specification.

This particular Agenda Item proposes that the lower value of $\varphi_c = 0.90$ conservatively be maintained for load rating of steel built-up members subject to axial compression if the year of construction is prior to 1991 in case the members were fabricated from universal mill plate. The production of universal mill plate was completely discontinued around 1990.

ANTICIPATED EFFECT ON BRIDGES:

Slightly lower ratings will be obtained for steel built-up compression members if the year of construction was prior to 1991.

REFERENCES:

Bartlett, R.M., Dexter, R.J., Graeser, M.D., Jelinek, J.J., Schmidt, B.J. and Galambos, T.V. (2003), "Updating Standard Shape Material Properties Database for Design and Reliability," *Engineering Journal,* AISC, Vol. 40, No. 1, pp. 2–14.

Bjorhovde, R. (1972), "Deterministic and Probabilistic Approaches to the Strength of Steel Columns," Ph.D. Dissertation, Lehigh University, Bethlehem, PA, May.

Bjorhovde, R. (1978), "The Safety of Steel Columns," *Journal of the Structural Division,* ASCE, Vol. 104, No. ST9, September, pp. 1371–1387.

Bjorhovde, R. (1988), "Columns: From Theory to Practice," *Engineering Journal*, AISC, Vol. 25, No. 1, 1st Quarter, pp. 21–34.

Ziemian, R.D. (ed.) (2010), *Guide to Stability Design Criteria for Metal Structures*, 6th Ed., John Wiley & Sons, Inc., Hoboken, NJ.

OTHER:

None

2013 AASHTO BRIDGE COMMITTEE AGENDA ITEM: 46

SUBJECT: The Manual for Bridge Evaluation: APPENDIX A: Illustrative Examples, Table of Contents & Example A11 (T18-4)

TECHNICAL COMMITTEE: T-18 Bridge Management, Evaluation and Rehabilitation

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☐ **REVISION** ☒ **ADDITION** ☐ **NEW DOCUMENT**

☐ **DESIGN SPEC** ☐ **CONSTRUCTION SPEC** ☐ **MOVABLE SPEC**
☒ **MANUAL FOR BRIDGE EVALUATION** ☐ **SEISMIC GUIDE SPEC** ☐ **BRIDGE ELEMENT INSP GUIDE**
☐ **OTHER**

DATE PREPARED: 1/8/13
DATE REVISED: 4/15/13

AGENDA ITEM:

Item #1

Add to the following to the MBE Appendix A: Illustrative Examples, Table of Contents:

A11	Single span 420.0 ft.	Through Truss	Gusset Plates	Design	Strength I	LRFR & LFR	A -xx

Item #2

Add the following Example A11 to the MBE Appendix A, as shown in Attachment B, if Ballot Item T18-3 containing gusset plate load rating provisions is adopted for inclusion in Section 6 of the MBE by the SCOBS.

OTHER AFFECTED ARTICLES:

None

BACKGROUND:

The example is proposed to be included in the MBE Appendix A: It illustrates the load rating of gusset plates and the application of these proposed revisions to Section 6 of the MBE. It is envisioned that the FHWA guidance for load rating of gusset plates will not be maintained in the future.

ANTICIPATED EFFECT ON BRIDGES:

The revisions contribute to a better understanding of gusset plate behavior and capacity and should result in a higher reliability for safety of truss bridges.

REFERENCES:

None

OTHER:

None

A11 —THROUGH TRUSS BRIDGE: GUSSET PLATE RATING

These gusset plate checks for design loads illustrate the application of the LRFR and LFR specifications (ballot item T18-3), using the L2 joint of the truss depicted in Figure A11.1-1. Sections A11.1 through A11.9 provide an LRFR gusset plate rating example, while Sections A11.10 through A11.18 provide an LFR gusset plate rating example.

A11.1 – Bridge and Member Data

The truss bridge on I-94 was constructed in 1991. All members and plates were made with AASHTO M223 Gr. 50 material; all fasteners are 7/8" diameter A325 fully-tensioned bolts in standard holes that were sub-punched and reamed to size. For the purposes of this design example, a center-to-center distance of 3 inches is assumed for all bolts. The truss is striped for four lanes of traffic and carries an ADTT of 10,000/day. There is no wearing surface on the deck (i.e. DW=0). The superstructure has a current NBI Condition Rating of 7.

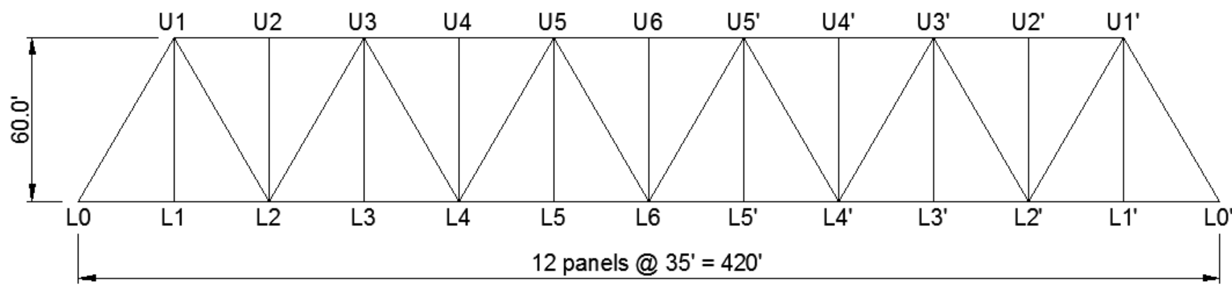

Figure A11.1-1: Truss Elevation

Detailing of the joint is shown in Figure A11.1-2. The loads shown are envelope loads developed using HS20-44 & Alternate loading. Impact loading was considered to be 28.5% of the live load.

For the purposes of this example, this LRFR rating will assume these loads were from HL-93 loading considering 33% impact, though a separate structural analysis would have to be performed to truly understand the differences in live load effects.

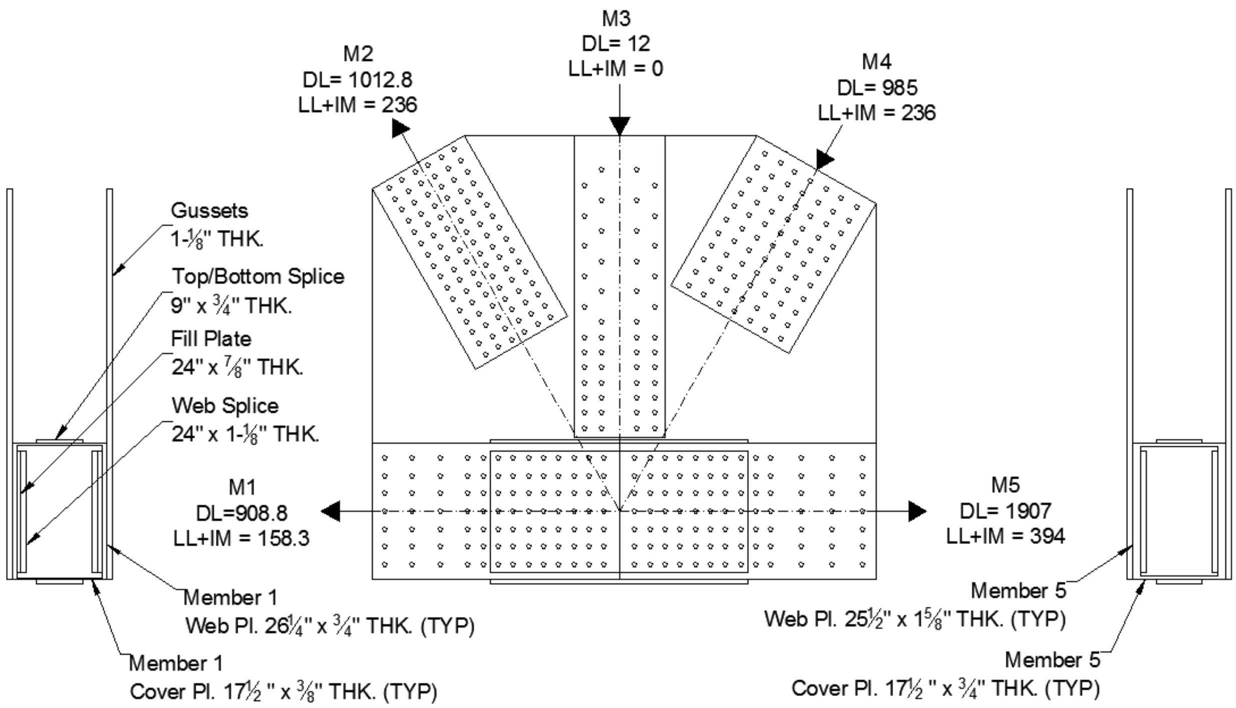

Figure A11.1-2: Panel Point L2 Details

The various evaluation factors used throughout the example are shown in Table A11.1-1.

	Strength (Table 6A.4.2.2-1)	Service (Table 6A.4.2.2-1)
Inventory Live Load Factor (γ_{LL_INV})	1.75	1.3
Operating Live Load Factor (γ_{LL_OPR})	1.35	1.00
Dead Load Factor (γ_{DL})	1.25	1.00
System Factor (φ_s) (Table 6A.4.2.4-1)	0.90	0.90
Condition Factor (φ_c) (Table 6A.4.2.3-1)	1.00	1.00

Table A11.1-1: LRFR Load Factors for Gusset Plates

In addition, detailed inspection reports of the L2 joint noted the following areas of discrete section loss on only one of the two gusset plates. The contour image in Figure A11.1-3 shows the pattern of section loss, again, only on one of two gusset plates. This section loss will be used in the relevant resistance checks.

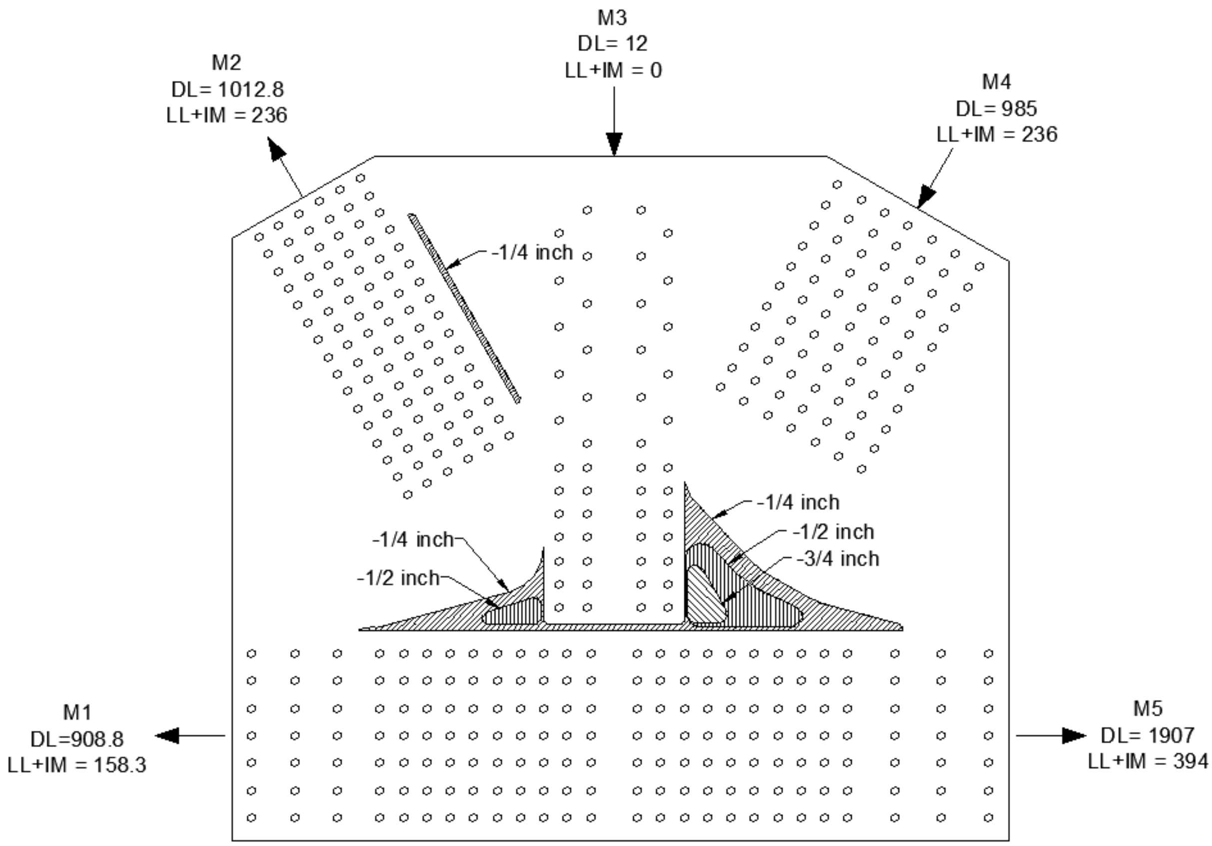

Figure A11.1-3: Losses to One Gusset Plates at L2

The following rating checks will be performed:

1) Check tension resistance of M1
2) Check the chord splice resistance between M1 and M5
3) Check tension resistance of M2
4) Check compression resistance of M3
5) Check compression resistance of M4
6) Check the horizontal shear resistance of the gusset plate
7) Check the vertical shear resistance of the gusset plate

A11.2 -- Check the Tension Resistance of M1

Only block shear resistance of the gusset plate, shear resistance of the bolts, and bearing resistance of the gusset must be checked. The checks for M5 are very similar in nature and will not be shown. Additionally, the bolt shear and bearing resistance checks will only be demonstrated for the gusset plate at M1, these checks would normally be performed at each fastener pattern and for the splice plates too for a complete rating.

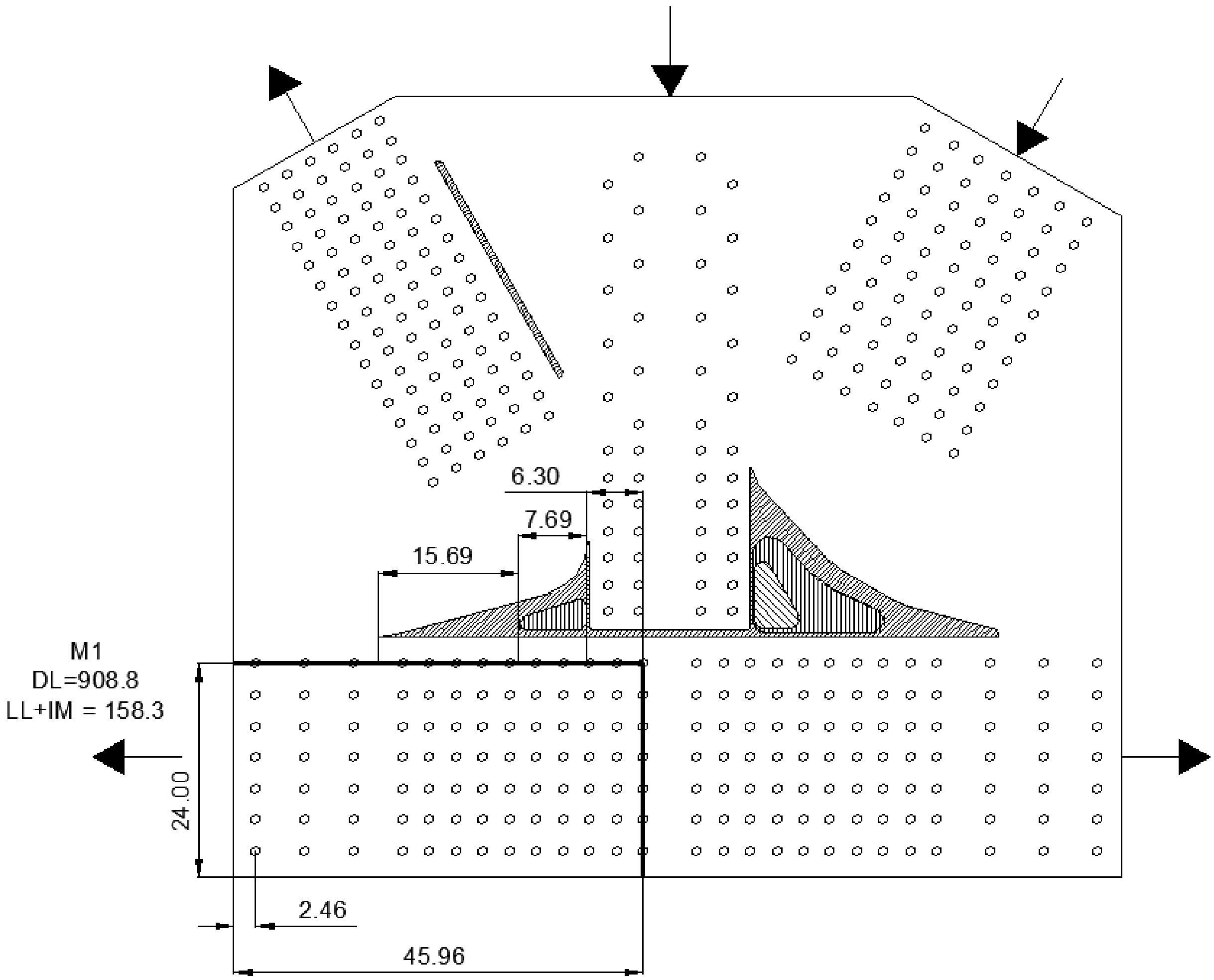

Figure A11.2-1: Block Shear Failure Plane for Member M1 at L2

First determine the shear resistance of the fasteners using Article 6A.6.12.6.2. Because web splice plates are used, some fasteners will be in double shear, but conservatively assume just single shear. The connection is not greater than 50 inches in length, therefore a connection length reduction factor does not apply. Since the web splice plates have fill plates greater than 0.25 inches, the bolt shear must be reduced by the fill plate factor.

$$R_n = \varphi_{sb} 0.48 A_b F_{ub} N_s R_{fill}$$ Eqn. 6A.6.12.6.2-1

$$R_{fill} = \left[\frac{1+\gamma}{1+2\gamma}\right]$$ Eqn. 6A.6.12.6.2-3

$$\gamma = \left[\frac{A_{fill}}{A_p}\right]$$

In this case, A_p is determined as the lesser of area of the lower chord web plate (¾”x26¼”) and the sum of the areas of the web splice plate (1⅛”x24”) and gusset plate (1⅛”x87.8”). Given the large area of the gusset plate, it is clear that the area of the member web plate will control the calculation of A_p.

$$\gamma = \frac{(24)(0.875)}{(26.25)(0.75)} = 1.067$$

$$R_{fill} = \left[\frac{1+\gamma}{1+2\gamma}\right] = \left[\frac{1+1.067}{1+(2)(1.067)}\right] = 0.66$$

$$R_r = 0.80(0.48)\left(\frac{\pi(0.875^2)}{4}\right)120(1)(0.66) = 18.3\,\text{kip/bolt}$$

Check the bearing resistance of the gusset plate. All holes are at distances greater than 2 bolt diameters from each other and the edges, therefore only one bearing resistance equation needs to be used.

$$R_r = \phi_{bb} 2.4 dtF_u$$

$$R_r = 0.80(2.4)\left(\frac{7}{8}\right)(1.125)(65) = 122.8\,\text{kip/bolt}$$ Eqn. 6A.6.12.6.4-1

Note that the chord web plate thickness is ¾”, and the bearing resistance of the web plate will govern over the bearing resistance of the gusset plate. Therefore, a similar bearing resistance computation for the web plate should be performed as part of the member rating. However, as this example is for gusset plates alone, the bearing resistance computation for the chord member will not be shown herein.

The resistance is the lesser of the fastener shear resistance and plate bearing resistance. In this case, the fastener resistance controls. For this member, there are 182 bolts. Conservatively, the fill plate reduced bolt shear resistance is assumed over all the bolts, despite the fill plate not covering the entire bolt pattern. The total resistance is:

$$R_r = 182 * 18.3\,\text{kip/bolt} = 3330\,\text{kips}$$

Next determine the resistance due to block shear (see Figure A11.2-1). First calculate out the shear and tensile areas.

$$A_{tg} = 2 * 24.00 * 1.125 = 54.00 in.^2$$

$$A_{tn} = 54.00 - 13\left(\frac{7}{8}+\frac{1}{8}\right)1.125 = 39.38 in.^2$$

<u>Gross shear plane of the gusset plate without corrosion damage</u>

$$A_{vg1} = 45.96 * 1.125 = 51.71 in.^2$$

<u>Gross shear plane of the gusset plate with corrosion damage</u>

$$A_{vg2} = (45.96)(1.125) - (6.30+15.69)(0.25) - (7.69)(0.50) = 42.36 in.^2$$

<u>Net shear plane</u>

$$A_{vn} = (51.71+42.36) - 25\left(\frac{7}{8}+\frac{1}{8}\right)1.125 = 65.95 in.^2$$

Calculate the DL/LL ratio for this limit state to account for further reduction according to Article 6A.6.12.6.1.

$$\frac{P_{1DL}}{P_{1LL}} = \frac{908.8}{158.3} = 5.74 \text{ which is greater than } 1.0 \text{ therefore the additional reduction is :}$$

$$\mathrm{R}_{\mathrm{DL\_LL}} = 1 - 0.1\left(\frac{5.74 - 1.0}{5}\right) = 0.90$$ Article 6A.6.12.6.1

Calculate the block shear resistance of each gusset plate.

$$\text{is } F_u A_{vn} \le F_y \left(A_{vg1} + A_{vg2}\right)?$$
$$65(65.95) \le 50(51.71 + 42.36) \rightarrow 4287 \le 4703, \text{ yes, therefore}$$
$$P_{bs1} = \phi_{bs} R_p \left(0.58 F_u A_{vn} + F_u A_{tn}\right)$$ Eqn. 6A.6.12.6.8-1
$$P_{bs1} = (1.00)(1.0)\left[(0.58)(65)(65.95) + (65)(39.38)\right] = 5046$$

Since the resistance from the bolts is less than the block shear resistance, then the capacity is controlled by the bolt resistance, C=3330 kips.

Calculate the rating factors.

$$RF_{LRFR} = \frac{\phi_s \phi_c C(R_{DL\_LL}) - \gamma_{DL} P_{1DL}}{\gamma_{LL} P_{1LL}}$$ Eqn. 6A.4.2.1-1
$$RF_{LRFRinv} = \frac{0.90(1.00)(3330)(0.90) - (1.25)(908.8)}{(1.75)(158.3)} = 5.63$$
$$RF_{LRFRopr} = RF_{LRFRinv} \frac{\gamma_{LLinv}}{\gamma_{LLopr}} = 5.63 \frac{1.75}{1.35} = 7.30$$

The gusset plate was also originally designed to be slip-critical with the members. Therefore rating factors must also be calculated for the slip resistance. Slip resistance is checked with the Service II load combination.

The slip resistance of the bolt group is calculated as:

$$R_r = R_n = K_h K_s N_s P_t (\#ofbolts)$$ Eqn. 6A.6.12.6.3-1

For standard holes, $K_h = 1.0$ Article 6A.6.12.6.3

A Class B surface preparation was specified, $K_s = 0.50$ Table 6A.6.12.6.3-2

Conservatively, one shear plane is assumed for all bolts, $N_s = 1$

$P_t = 39$ kips Table 6A.6.12.6.3-1

$$R_r = 1.0(0.50)(1)(39)(182) = 3549 \text{ kips}$$

Per Article 6A.6.12.6.3, slip only needs to be calculated at the Operating level under Service II loads.

$$RF_{LRFRopr} = \frac{\phi_s \phi_c C - \gamma_{DL} P_{1DL}}{\gamma_{LL} P_{1LL}} = \frac{0.90(1.00)(3549) - (1.00)(908.8)}{(1.00)(158.3)} = 14.44$$ Eqn. 6A.4.2.1-1

Note that in the above equation, the dead-to-live load ratio reduction factor is not applied to the capacity per Article 6A.6.12.6.1 since it is a service limit-state.

A11.3 -- Check the Chord Splice Resistance

Since the gusset plate is also performing as a tension chord splice, the resistance of the spliced section must be evaluated according to the provisions of Article 6A.6.12.6.9. To begin, the net and gross section modulus need to be calculated for the composite section of gusset and splice plates (See Figure A11.3-1).

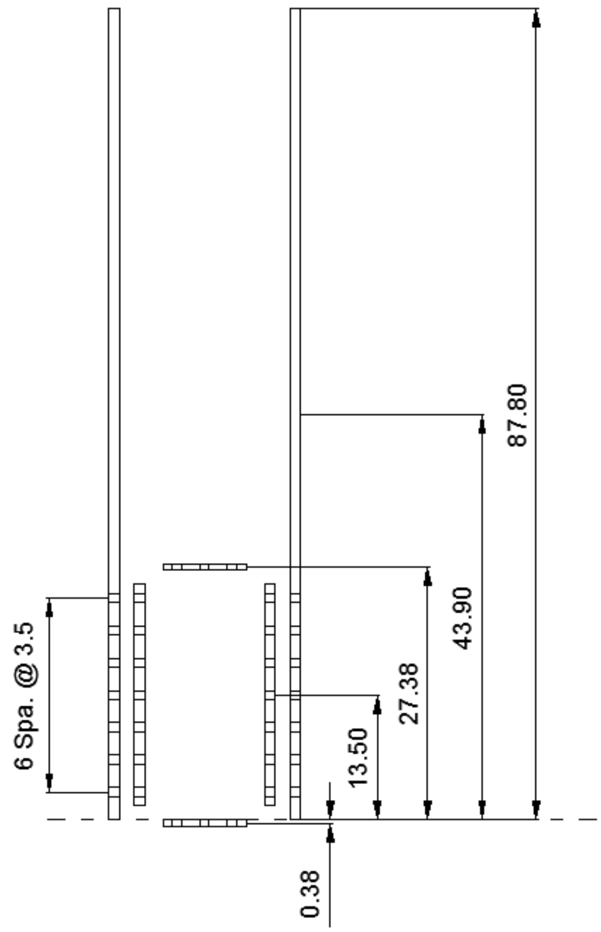

Figure A11.3-1: Section of Gusset and Splice Plates at L2

Start by calculating the gross and net section properties of the combined section of gusset and web splice plates. For brevity, the calculations will not be shown. In this case they were calculated using a drawing program; alternatively, the calculations can be completed using a spreadsheet or long-hand equations.

The gross and net section areas are:

$A_g = 265.1\,\text{in.}^2$

$A_n = 229.0\,\text{in.}^2$

The heights of the gross and net section centroids, using the bottom of the gusset as the datum, are:

$h_g = 36.2\,\text{in.}$

$h_n = 39.7\,\text{in.}$

Next calculate the gross and net moments of inertia.

$$I_g = 178592.7 \text{ in.}^4$$
$$I_n = 154792.4 \text{ in.}^4$$

The maximum bending stress will be at the bottom of gusset plate. Therefore, calculate the gross and net section moduli relative to the bottom of the gusset.

$$S_g = \frac{178592.7}{36.2} = 4933.5 \text{ in.}^3$$
$$S_n = \frac{154792.4}{39.7} = 3899.1 \text{ in.}^3$$

Since the chord splice section intersects the workpoint of the joint, the force resultant of the chord is at the center of the chord depth, so the gross and net section eccentricities are:

$$e_{p\_gross} = 36.2 - \frac{27}{2} = 22.7 \text{ in.}$$
$$e_{p\_net} = 39.7 - \frac{27}{2} = 26.2 \text{ in.}$$

The resistance of the chord splice can be calculated as the minimum of Eqns. 6A.6.12.6.9-3 and 6A.6.12.6.9-4.

$$P_{r\_gross} = \phi_{cs} F_y \left(\frac{S_g A_g}{S_g + e_{p\_gross} A_g} \right) \quad \text{Eqn. 6A.6.12.6.9-3}$$
$$P_{r\_gross} = 0.85(50)\left(\frac{(4933.5)(265.1)}{4933.5 + (22.7)(265.1)} \right) = 5076 \text{ kips}$$
$$P_{r\_net} = \phi_{cs} F_u \left(\frac{S_n A_n}{S_n + e_{p\_net} A_n} \right) \quad \text{Eqn. 6A.6.12.6.9-4}$$
$$P_{r\_net} = 0.85(65)\left(\frac{(3899.1)(229.0)}{3899.1 + (26.2)(229.0)} \right) = 4984 \text{ kip.}$$

The lesser resistance controls; in this case, it is controlled by fracture on the net section, C=4984 kips.

Next determine the factored load in the chord splice. Look at the loads on both sides of the splice and use the more severe.

Left Side
$$1.25\left(908.8 + (1012.8)(\cos(60°))\right) + 1.75\left(158.3 + (236)(\cos(60°))\right) = 2252 \text{ kips}$$
Right Side
$$1.25\left(1907 - (985)(\cos(60°))\right) + 1.75\left(394 - (236)(\cos(60°))\right) = 2251 \text{ kips}$$

Since the loads are nearly the same on each side of the splice, determine the most severe DL/LL ratio reduction.

$$\frac{P_{DL\_left}}{P_{LL\_left}} = \frac{908.8 + (1012.8)(\cos(60°))}{158.3 + (236)(\cos(60°))} = 5.12$$
$$\frac{P_{DL\_right}}{P_{LL\_right}} = \frac{1907 - (985)(\cos(60°))}{394 - (236)(\cos(60°))} = 5.12$$

Since the dead load-to-live load ratio reduction factor is the same for both sides, use the controlling loads from the left side of the splice.

$$\mathrm{R}_{\mathrm{DL\_LL}} = 1 - 0.1\left(\frac{5.12 - 1.0}{5}\right) = 0.92$$

Article 6A.6.12.6.1

Calculate the LRFR rating factors.

$$RF_{LRFR} = \frac{\phi_s \phi_c C(R_{DL\_LL}) - \gamma_{DL} P_{DL}}{\gamma_{LL} P_{LL}}$$

Eqn. 6A.4.2.1-1

$$RF_{LRFRinv} = \frac{0.90(1.00)(4984)(0.92) - (1.25)(908.8 + (1012.8)(\cos(60°)))}{(1.75)(158.3 + (236)(\cos(60°))} = 4.87$$

$$RF_{LRFRopr} = RF_{LRFRinv} \frac{\gamma_{LLinv}}{\gamma_{LLopr}} = 4.87 \frac{1.75}{1.35} = 6.32$$

A11.4 -- Check Tension Resistance of M2

The tension capacity will be governed by the lesser of the gross yield and net fracture on the Whitmore section (see Figure A11.4-1), and block shear (See Figure A11.4-2). Consider the section loss in the one gusset plate.

Define the gross and net section areas on the Whitmore section.

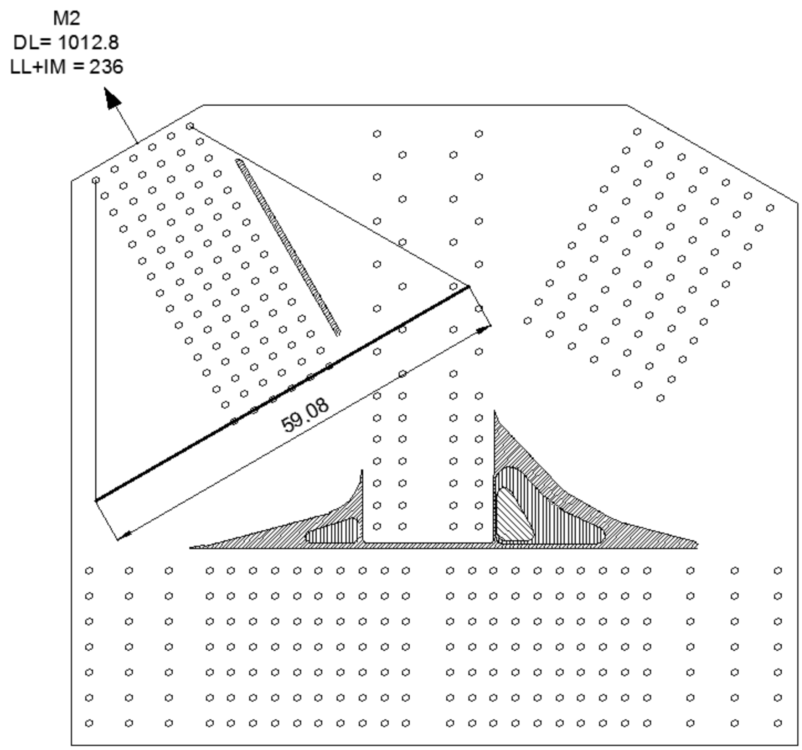

Figure A11.4-1: Member M2 Whitmore Section at L2

$$A_g = 2*59.08*1.125 = 132.93 in.^2$$

$$A_n = 132.93 - 12\left(\frac{7}{8}+\frac{1}{8}\right)1.125 = 119.43 in.^2$$

** There are no reductions due to corrosion for the Whitmore tension resistance because no section loss intersects the Whitmore section. **

Define the gross and net sections for block shear check.

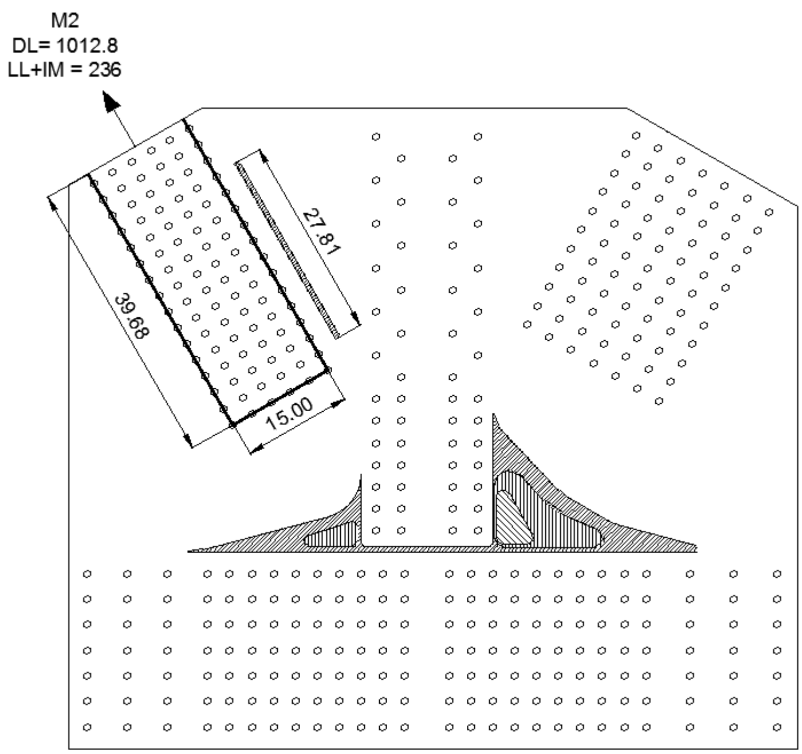

Figure A11.4-2: Member M2 Block Shear Plane at L2

$$A_{tg} = 2*15.00*1.125 = 33.75 in.^2$$

$$A_{tn} = 33.75 - 10\left(\frac{7}{8}+\frac{1}{8}\right)1.125 = 22.5 in.^2$$

Gross shear plane of the gusset plate without corrosion damage

$$A_{vg1} = 2*39.68*1.125 = 89.28 in.^2$$

Gross shear plane of the gusset plate with corrosion damage

$$A_{vg2} = 39.68*1.125 + 27.81(1.125-0.25) + (39.68-27.81)1.125 = 82.33 in.^2$$

Net shear plane

$$A_{vn} = (2*89.28) - 62\left(\frac{7}{8}+\frac{1}{8}\right)1.125 = 108.81 in.^2$$

Calculate the LRFR rating factors.

Calculate the DL/LL ratio for this limit state to account for further reduction according to Article 6A.6.12.6.1.

$$\frac{P_{2DL}}{P_{2LL}} = \frac{1012.8}{236} = 4.29 \text{ which is greater than } 1.0 \text{ therefore the additional reduction is :}$$

$$R_{DL\_LL} = 1 - 0.1\left(\frac{4.29 - 1.0}{5}\right) = 0.93$$ Article 6A.6.12.6.1

Calculate the yield on gross and fracture on net capacities on the Whitmore section.

$$P_y = \phi_y F_y A_g = 0.95 * 50 * 132.93 = 6314$$ Eqn. 6A.6.12.6.8-2

$$P_r = \phi_u F_u A_n R_p U = 0.80 * 65 * 119.43 * 1.0 * 1.0 = 6210$$ Eqn. 6A.6.12.6.8-3

Calculate the block shear resistance of each gusset plate.

$$\text{is } F_u A_{vn} \le F_y \left(A_{vg1} + A_{vg2}\right)?$$ Eqn. 6A.6.12.6.8-1

$$65(108.81) \le 50(89.28 + 82.33) \rightarrow 7073 \le 8580, \text{ yes; therefore:}$$

$$P_{bs1} = \phi_{bs} R_p \left(0.58 F_u A_{vn} + F_u A_{tn}\right) = 1.00 * 1.0\left[(0.58 * 65 * 108.81) + (65 * 22.5)\right] = 5564.6$$

The overall tension capacity, C, is the lesser of the Whitmore yield on gross, Whitmore fracture on net, and block shear; in this case it's controlled by block shear, so C=5564.6 kips.

Calculate the rating factors.

$$RF_{LRFR} = \frac{\phi_s \phi_c C(R_{DL\_LL}) - \gamma_{DL} P_{2DL}}{\gamma_{LL} P_{2LL}}$$ Eqn. 6A.4.2.1-1

$$RF_{LRFRinv} = \frac{0.90 * 1.00 * 5564.6 * 0.93 - 1.25 * 1012.8}{1.75 * 236} = 8.21$$

$$RF_{LRFRopr} = RF_{LRFRinv} \frac{\gamma_{LLinv}}{\gamma_{LLopr}} = 8.21 \frac{1.75}{1.35} = 10.64$$

A11.5 -- Check the Compression Resistance of M3

It is clear that this member will not control with only 12 kips of dead load, and since there is no live load, the rating factor cannot be calculated. For illustration, only the capacity calculations will be shown.

For the Whitmore buckling check, project the worst case section loss onto the Whitmore section (See Figure A11.5-1) in the direction of the member to calculate the equivalent gusset plate thickness.

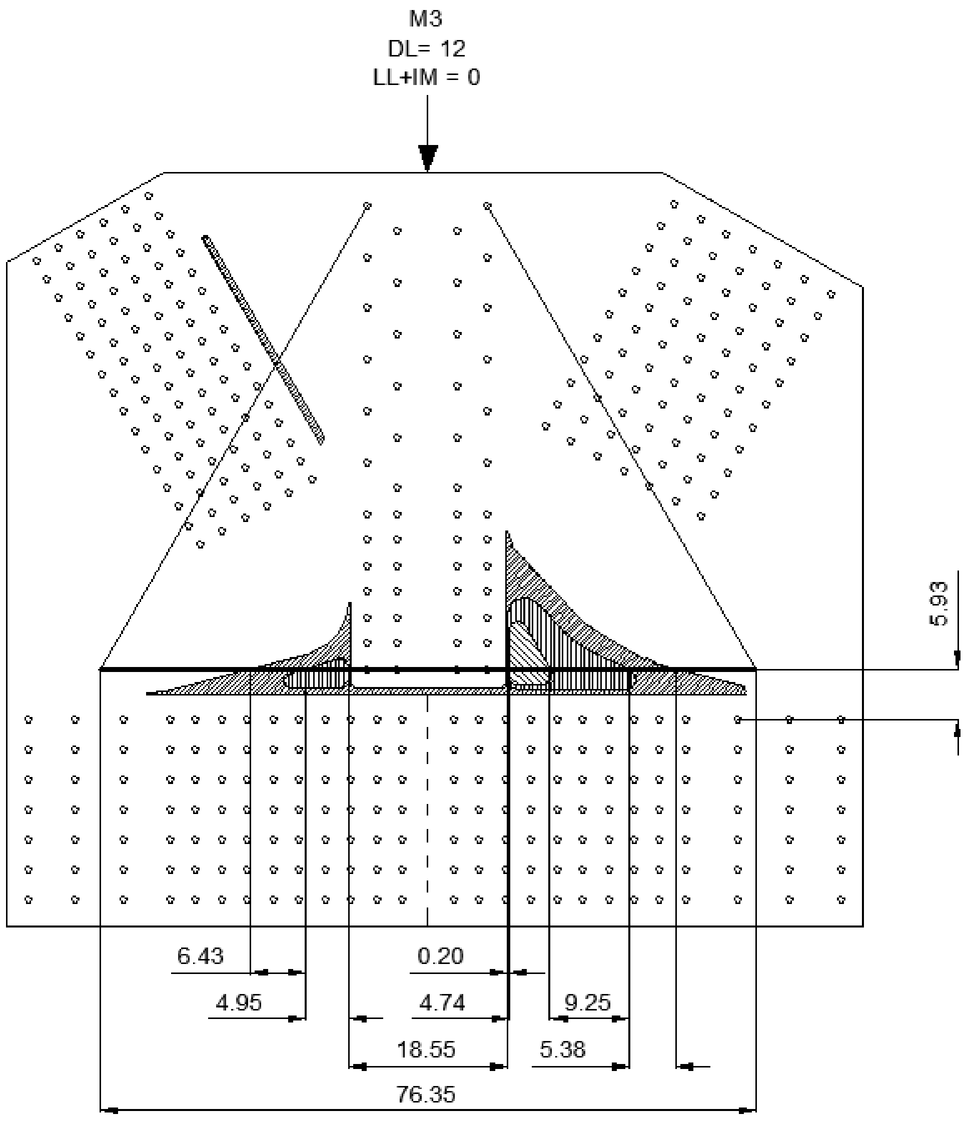

Figure A11.5-1: Member M3 Whitmore Section at L2

$$t_{eq\_whit} = \frac{30.36(1.125-0.25)+14.40(1.125-0.5)+4.74(1.125-0.75)+1.125(76.35-30.36-14.40-4.74)}{76.35}$$

$$t_{eq\_whit} = 0.88$$

This member has no live load, so it has no extra dead-to-live load reduction.

Calculate the factored Whitmore buckling strength per gusset plate because only one of the plates has section loss.

For the gusset plate with section loss

$$P_{e1} = \frac{3.29E}{\left(\frac{L_{mid}}{t_{eq\_whit}}\right)^2} A_g = \frac{3.29(29000)}{\left(\frac{5.93}{0.88}\right)^2}(76.35*0.88) = 141928$$ Eqn. 6A.6.12.6.7-4

$$P_{o1} = F_y A_g = 50(76.35*0.88) = 3377.4$$ Article 6A.6.12.6.7

$$\frac{P_{e1}}{P_{o1}} = \frac{141928}{3377.4} = 42.0 \text{ is greater than 0.44; therefore :}$$

$$\text{Whit}_1 = 0.658^{P_{o1}/P_{e1}} P_{o1} = 0.658^{3377.4/141928} 3377.4 = 3343.9$$ Eqn. 6A.6.12.6.7-2

For the gusset plate without section loss

$$P_{e2} = \frac{3.29E}{\left(\frac{L_{mid}}{t_g}\right)^2} A_g = \frac{3.29(29000)}{\left(\frac{5.93}{1.125}\right)^2}(76.35*1.125) = 294952$$ Eqn. 6A.6.12.6.7-4

$$P_{o2} = F_y A_g = 50(76.35*1.125) = 4294.7$$ Article 6A.6.12.6.7

$$\frac{P_{e2}}{P_{o2}} = \frac{294952}{4294.7} = 68.68 \text{ is greater than 0.44; therefore :}$$

$$\text{Whit}_2 = 0.658^{P_{o2}/P_{e2}} P_{o2} = 0.658^{4294.7/294952} 4294.7 = 4268.6$$ Eqn. 6A.6.12.6.7-2

Therefore the total Whitmore buckling capacity is the sum of the two individual gusset plates:

$$Whit = \phi_{cg}\left(Whit_1 + Whit_2\right) = 0.95*\left(3343.9 + 4268.6\right) = 7231.9$$ Eqn. 6A.6.12.6.7-1

This vertical member does not have any admissible partial shear planes that would control the compression resistance. Therefore, the capacity, C, is the Whitmore buckling strength.

A11.6 -- Check the Compression Resistance of M4

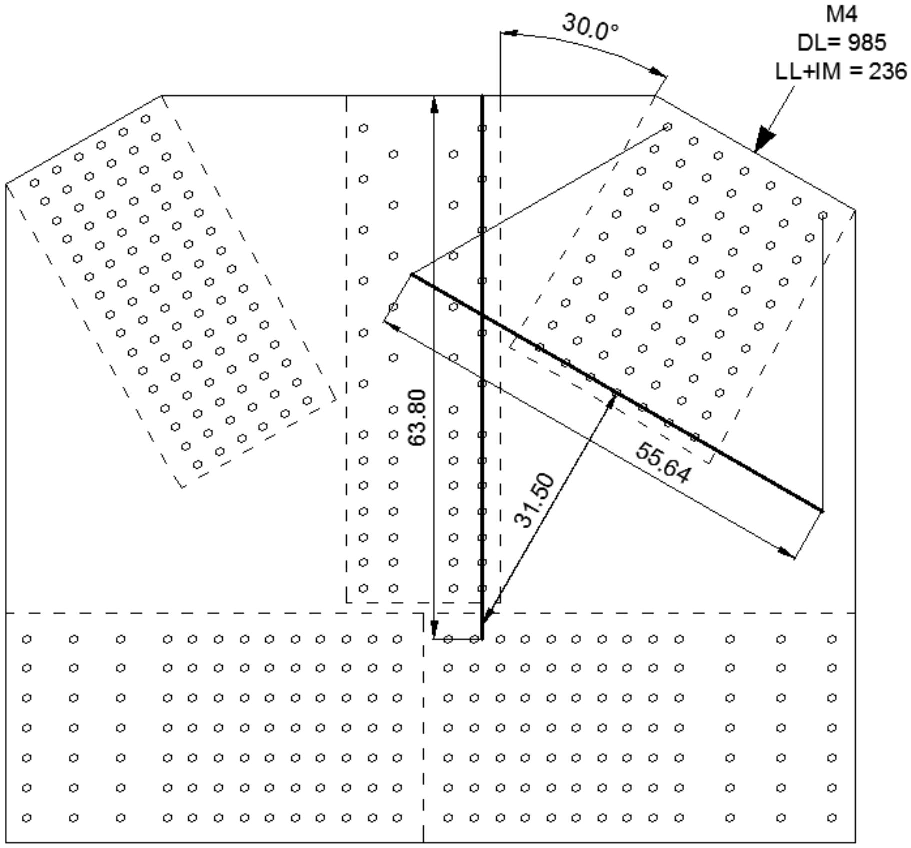

Figure A11.6-1: Member M4 Compression Checks at L2

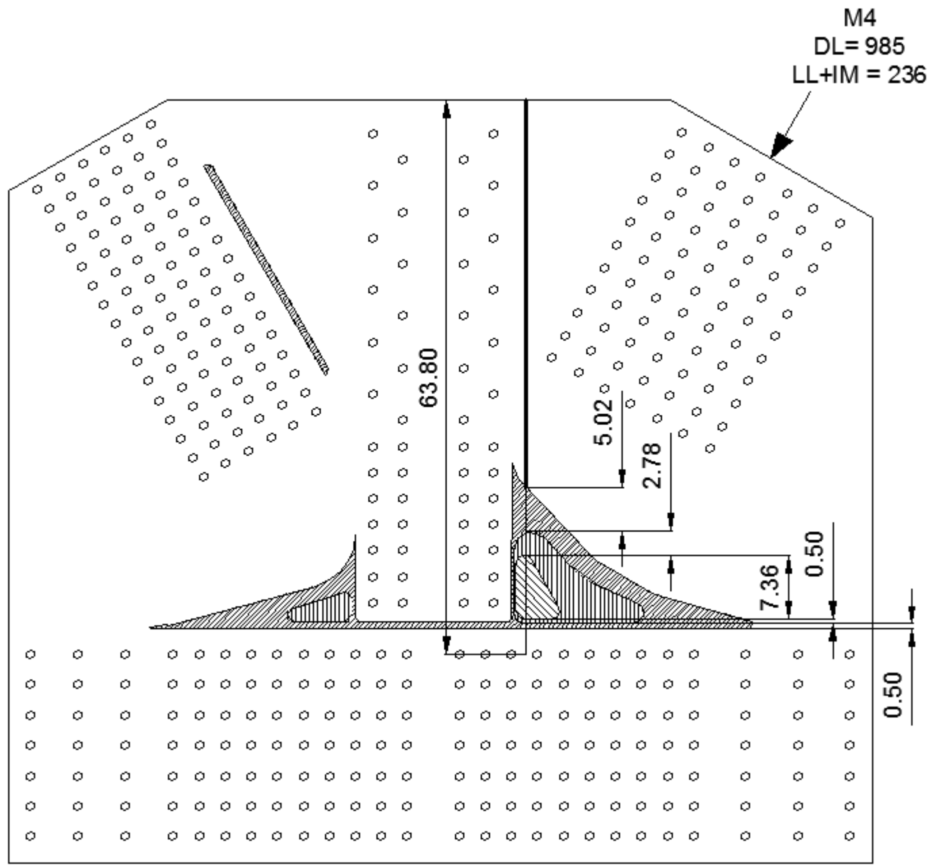

Figure A11.6-2: Member M4 Partial Shear Plane at L2

Calculate the equivalent gusset plate thickness on the partial shear plane for the gusset plate with section loss (see Figure A11.6-2).

$$t_{eq\_shr} = \frac{5.52(1.125-0.25)+3.28(1.125-0.5)+7.36(1.125-0.75)+1.125(63.80-5.52-3.28-7.36)}{63.80}$$

$$t_{eq\_shr} = 0.99$$

For the Whitmore buckling check, project the worst case section loss onto the Whitmore section in the direction of the member to calculate the equivalent gusset plate thickness (see Figure A11.6-3).

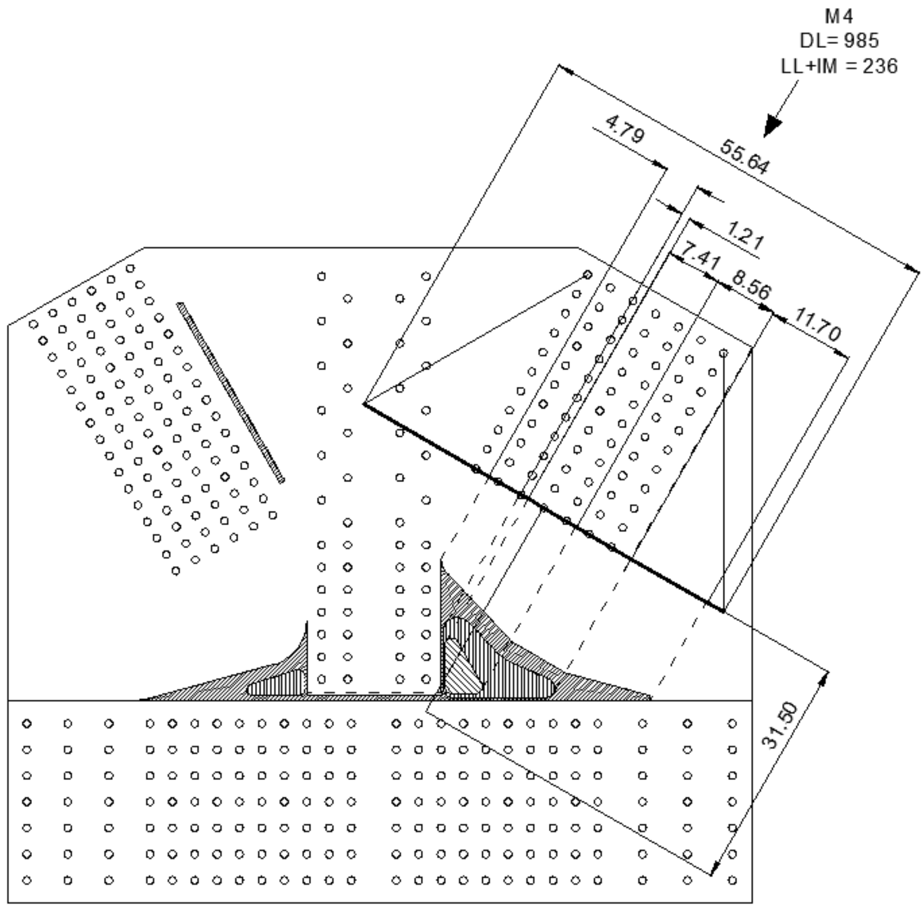

Figure A11.6-3: Member M4 Whitmore Section at L2

$$t_{eq\_whit} = \frac{16.49(1.125-0.25)+9.77(1.125-0.5)+7.41(1.125-0.75)+1.125(55.64-16.49-9.77-7.41)}{55.64}$$

$$t_{eq\_whit} = 0.86$$

For LRFR, calculate the DL/LL ratio reduction according to Article 6A.6.12.6.1.

$$\frac{P_{4DL}}{P_{4LL}} = \frac{985}{236} = 4.17 \text{ which is greater than } 1.0\text{; therefore, the additional reduction is :}$$

$$\mathrm{R}_{\mathrm{DL\_LL}} = 1 - 0.1\left(\frac{4.17-1.0}{5}\right) = 0.94$$

Article 6A.6.12.6.1

Calculate the factored Whitmore buckling strength per gusset plate, because only one of the plates has section loss.

<u>For the gusset plate with section loss</u>

$$P_{e1} = \frac{3.29E}{\left(L_{mid}/t_{eq\_whit}\right)^2} A_g = \frac{3.29(29000)}{\left(31.50/0.86\right)^2}(55.64*0.86) = 3441.4$$

Eqn. 6A.6.12.6.7-4

$$P_{o1} = F_y A_g = 50(55.64*0.86) = 2401.5$$

Article 6A.6.12.6.7

$$\frac{P_{e1}}{P_{o1}} = \frac{3441.4}{2401.5} = 1.43 \text{ is greater than 0.44; therefore:}$$

$$\text{Whit}_1 = 0.658^{P_{o1}/P_{e1}} P_{o1} = 0.658^{2401.5/3441.4} 2401.5 = 1793.2$$ Eqn. 6A.6.12.6.7-2

For the gusset plate without section loss

$$P_{e2} = \frac{3.29E}{\left(L_{mid}/t_g\right)^2} A_g = \frac{3.29(29000)}{\left(31.50/1.125\right)^2}(55.64*1.125) = 7617.6$$ Eqn. 6A.6.12.6.7-4

$$P_{o2} = F_y A_g = 50(55.64*1.125) = 3129.8$$ Article 6A.6.12.6.7

$$\frac{P_{e2}}{P_{o2}} = \frac{7617.6}{3129.8} = 2.43 \text{ is greater than 0.44; therefore:}$$

$$\text{Whit}_2 = 0.658^{P_{o2}/P_{e2}} P_{o2} = 0.658^{3129.8/7617.64} 3129.8 = 2635.3$$ Eqn. 6A.6.12.6.7-2

Therefore, the total Whitmore buckling capacity is the sum of the two individual gusset plates.

$$Whit = \phi_{cg}\left(Whit_1 + Whit_2\right) = 0.95*\left(1793.2 + 2635.3\right) = 4207.1$$ Eqn. 6A.6.12.6.7-1

Since this is a compression member, the partial plane shear yield capacity also needs to be calculated.

For the gusset plate with section loss

$$PS_1 = \frac{\Omega\left(0.58F_y\right)A_{shear}}{\cos(\theta)} = \frac{0.88(0.58*50)(63.80*0.99)}{\cos(30)} = 1863.4$$ Eqn. 6A.6.12.6.6-1

For the gusset plate without section loss

$$PS_2 = \frac{\Omega\left(0.58F_y\right)A_{shear}}{\cos(\theta)} = \frac{0.88(0.58*50)(63.80*1.125)}{\cos(30)} = 2115.1$$ Eqn. 6A.6.12.6.6-1

Therefore, the total partial plane shear capacity is the sum of the two individual gusset plates.

$$PS = \phi_{vg}\left(PS_1 + PS_2\right) = 1.00*\left(1863.4 + 2115.1\right) = 3978.5$$

The overall buckling capacity, C, is the lesser of the Whitmore buckling strength and that from the partial plane shear yield. In this case, partial plane shear yielding controls, and C=3978.5 kips.

Calculate the rating factors.

$$RF_{LRFR} = \frac{\phi_s \phi_c C(R_{DL\_LL}) - \gamma_{DL} P_{4DL}}{\gamma_{LL} P_{4LL}}$$ Eqn. 6A.4.2.1-1

$$RF_{LRFRinv} = \frac{0.9*1.00*3978.5*0.94 - 1.25*985}{1.75*236} = 5.17$$

$$RF_{LRFRopr} = RF_{LRFRinv}\frac{\gamma_{LLinv}}{\gamma_{LLopr}} = 5.17\frac{1.75}{1.35} = 6.70$$

A11.7 -- Check Horizontal Shear Capacity of the Gusset Plate

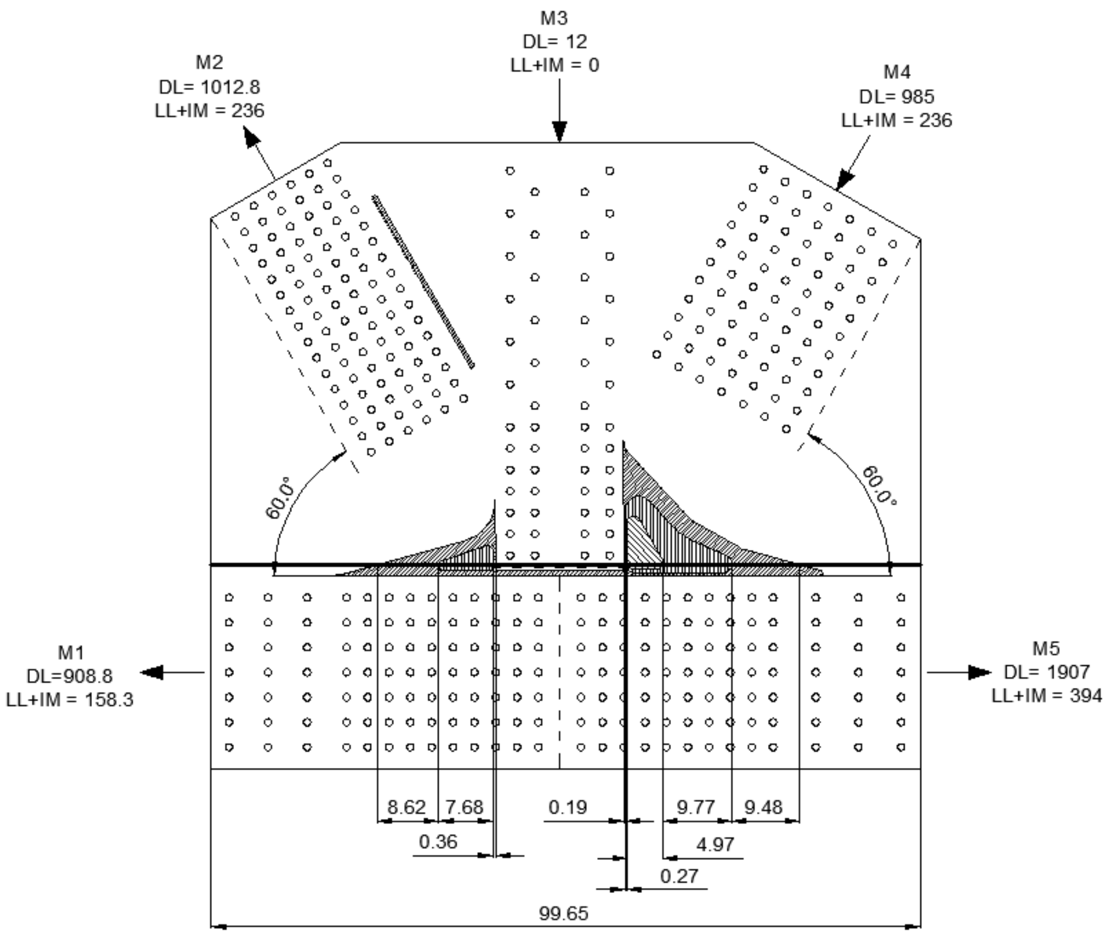

Figure A11.7-1: L2 Horizontal Shear Plane

First calculate the horizontal shear loads acting on the horizontal shear plane (see Figure A11.7-1). Note that the loads represented on the joint are based on envelope loads, not coincident loads. This will result in a conservative rating, which could be refined based on analysis of coincident shear loads.

$$P_{HDL} = P_{2HDL} + P_{4HDL} = (1012.8)(\cos 60°) + (985)(\cos 60°) = 998.9 \text{ kips}$$
$$P_{HLL} = P_{2HLL} + P_{4HLL} = (236)(\cos 60°) + (236)(\cos 60°) = 236 \text{ kips}$$

Calculate the DL/LL ratio for this limit state to account for further reduction according to MBE Article 6A.6.12.6.1.

$$\frac{P_{HDL}}{P_{HLL}} = \frac{998.9}{236} = 4.23 \text{ which is greater than 1.0; therefore, the additional reduction is:}$$

$$\mathrm{R}_{\mathrm{DL\_LL}} = 1 - 0.1\left(\frac{4.23 - 1.0}{5}\right) = 0.94$$

Article 6A.6.12.6.1

Calculate the equivalent gusset plate thickness on the horizontal shear plane for the gusset plate with section loss.

$$t_{eq\_shr} = \frac{99.65(1.125) - (8.62 + 0.36 + 0.19 + 9.48)(0.25) - (7.68 + 0.27 + 9.77)(0.5) - 4.97(0.75)}{99.65}$$

$$t_{eq\_shr} = 0.95$$

Calculate the resistance to gross section shear yielding.

For the gusset plate with section loss

$$V_{ny1} = \phi_{vg}(0.58F_y)A_g\Omega = 1.00(0.58*50)(99.65*0.95)(0.88) = 2416$$ Eqn. 6A.6.12.6.6-1

For the gusset plate without section loss

$$V_{ny2} = \phi_{vg}(0.58F_y)A_g\Omega = 1.00(0.58*50)(99.65*1.125)(0.88) = 2861$$ Eqn. 6A.6.12.6.6-1

Therefore the total resistance to shear yielding is:

$$V_{ny} = V_{ny1} + V_{ny2} = 2416 + 2861 = 5277$$

Calculate the resistance to shear fracture. Note that the section loss in the one gusset plate does not affect the net section.

$$V_{nu} = \phi_{vu}(0.58F_u)A_{vn} = 0.80(0.58*65)\left(2*99.65 - 52\left(\frac{7}{8} + \frac{1}{8}\right)\right)1.125 = 4998$$ Eqn. 6A.6.12.6.6-2

The capacity is determined by the lesser of the shear yield and shear fracture capacities. In this case, the shear fracture of the net section governs, and C=4998 kips.

Calculate the LRFR rating factors.

$$RF_{LRFR} = \frac{\phi_s\phi_c C(R_{DL\_LL}) - \gamma_{DL}P_{HDL}}{\gamma_{LL}P_{HLL}}$$ Eqn. 6A.4.2.1-1

$$RF_{LRFRinv} = \frac{0.9*1.00*4998*0.94 - 1.25*998.9}{1.75*236} = 7.21$$

$$RF_{LRFRopr} = RF_{LRFRinv}\frac{\gamma_{LLinv}}{\gamma_{LLopr}} = 7.21\frac{1.75}{1.35} = 9.35$$

A11.8 -- Check Vertical Shear Capacity of the Gusset Plate

This check will not control, because a valid shear plane cannot develop through the chord member.

A11.9 -- Summary of LRFR Load Rating Factors

	Inventory	Operating
Member 1 (bolt shear and slip resistance control)	5.63	7.30
Member 2 (block shear controls)	8.21	10.64
Member 3	Rating factors were not calculated	
Member 4 (partial plane shear controls)	5.17	6.70
Member 5	Rating factors were not demonstrated; procedures are similar to those for Member 1.	
Chord Splice (net fracture controls)	4.87	6.32
Horizontal Shear (fracture controls)	7.21	9.35
Vertical Shear	This is not a relevant failure mode for this joint	

Table A11.9-1: Gusse Plate L2 LRFR Summary

Note that the summary of rating factors presented in the above table considers solely the checks performed in this rating example. For a complete rating, the rating engineer must also consider the capacity of the bolts to ensure that the bolts do not govern the ratings for members 2 and 4 and for the chord splice.

A11.10 – Load Factor Rating of Gusset Plates

These gusset plate checks for design loads illustrate the application of the LFR specifications, using the L2 joint of the truss depicted in Figure A11.1-1. See Figure A11.1-2 for detailing of Joint L2. For overall bridge and member data, including a description of live loading, see Section A11.1. For section losses to the gusset plates at L2 considered in this rating, see Section A11.1 and Figure A11.1-3.

The loads shown are envelope loads developed using HS20-44 & Alternate loading. Impact loading was considered to be 28.5% of the live load.

The various evaluation factors used throughout the example are shown in Table A11.10-1.

	Strength (Article 6B.4.3)	Service (Article 6B.5.3.1)
Inventory Live Load Factor (γ_{LL_INV})	2.17	N/A
Operating Live Load Factor (γ_{LL_OPR})	1.30	1.00
Dead Load Factor (γ_{DL})	1.30	1.00

Table A11.10-1 LFR Load Factors for Gusset Plates

The following rating checks will be performed:

1) Check tension resistance of M1
2) Check the chord splice resistance between M1 and M5
3) Check tension resistance of M2
4) Check compression resistance of M3
5) Check compression resistance of M4
6) Check the horizontal shear resistance of the gusset plate
7) Check the vertical shear resistance of the gusset plate

A11.11 -- Check the Tension Resistance of M1

Only block shear resistance of the gusset plate, shear resistance of the bolts, and bearing resistance of the gusset must be checked. The checks for M5 are very similar in nature and will not be shown. Additionally, the bolt shear and bearing resistance checks will only be demonstrated for the gusset plate at M1, these checks would normally be performed at each fastener pattern and for the splice plates too for a complete rating.

First determine the shear resistance of the fasteners. Because web splice plates are used, some fasteners will be in double shear, but conservatively assume just single shear. The connection is not greater than 50 inches in length, therefore a connection length reduction factor does not apply. Since the web splice plates have fill plates greater than 0.25 inches, the bolt shear must be reduced by the fill plate factor.

$$\phi R = (\phi F)mAR_{fill}$$

Article L6B.2.6.1

$$\gamma = \left[\frac{A_{fill}}{A_p}\right]$$

In this case, A_p is determined as the lesser of area of the lower chord web plate (¾"x26¼") and the sum of the areas of the web splice plate (1⅛"x24") and gusset plate (1⅛"x87.8"). Given the large area of the gusset plate, it is clear that the area of the member web plate will control the calculation of A_p.

$$\gamma = \frac{(24)(0.875)}{(26.25)(0.75)} = 1.067$$

$$R_{fill} = \left[\frac{1+\gamma}{1+2\gamma}\right] = \left[\frac{1+1.067}{1+(2)(1.067)}\right] = 0.66$$

Article L6B.2.6.1

$$\phi R = (43)(1)\left(\frac{\pi(0.875^2)}{4}\right)(0.66) = 17.1\,\text{kip/bolt}$$

Check the bearing resistance of the gusset plate. Since all holes are centered a minimum of 3 inches from each other, assume a clear edge distance of 2.46 inches controls for all bolts.

$$\phi R = 0.9L_c tF_u \leq 1.8dtF_u$$

Standard Specification Eqn. 10-166b

$$\phi R = 0.9(2.46)(1.125)(65) \leq 1.8(0.875)(1.125)(65) \rightarrow 161.9 \leq 115.2$$

Since the inequality does not hold true, the bearing resistance is capped at 115.2 kips/bolt. This is much larger than the shear resistance of the fastener; therefore, the shear resistance controls the capacity.

$$\phi R = 182 * 17.1\,\text{kip/bolt} = 3112\,\text{kips}$$

Note that the chord web plate thickness is ¾", and the bearing resistance of the web plate will govern over the bearing resistance of the gusset plate. Therefore, a similar bearing resistance computation for the web plate should be performed as part of the member rating. However, as this example is for gusset plates alone, the bearing resistance computation for the chord member will not be shown herein.

Next determine the resistance due to block shear. See Section A11.2 for the gusset plate shear and tensile areas to consider for block shear.

Calculate the block shear resistance of each gusset plate (see Figure A11.2-1).

is $F_u A_{vn} \le F_y\left(A_{vg1} + A_{vg2}\right)$?

$65(65.95) \le 50(51.71 + 42.36) \rightarrow 4287 \le 4703$, yes, therefore — Article L6B.2.6.5

$$C = \phi_{bs} R_p\left(0.58 F_u A_{vn} + F_u A_{tn}\right)$$

$$C = (0.85)(1.0)\left[(0.58)(65)(65.95) + (65)(39.38)\right] = 4289$$

Since the resistance from the bolts is less than the block shear resistance, the capacity is controlled by the bolt resistance, C=3112 kips.

Calculate the rating factors.

$$RF_{LFR} = \frac{C - \gamma_{DL} P_{1DL}}{\gamma_{LL} P_{1LL}}$$ Eqn. 6B.4.1-1

$$RF_{LFRinv} = \frac{3112 - 1.30 * 908.8}{2.17 * 158.3} = 5.62$$

$$RF_{LFRopr} = RF_{LFRinv} \frac{\gamma_{LLinv}}{\gamma_{LLopr}} = 5.62 \frac{2.17}{1.30} = 9.38$$

The gusset plate was also originally designed to be slip-critical with the members; therefore, rating factors must also be calculated for the slip resistance. Slip resistance is checked with the Service II load combination.

The slip resistance per bolt is calculated as:

$$R_n = (\phi F_s) A_b (\#ofbolts) = 21\left(\frac{\pi(0.875^2)}{4}\right)(182) = 2298$$ Standard Specification Eqn. 10-172a

φF_s is taken as 21 ksi according to Article 6B.5.3.1.

Per Article 6B.5.3.1, slip only needs to be calculated at the Operating level under Service II loads.

$$RF_{LFRopr} = \frac{C - \gamma_{DL} P_{1DL}}{\gamma_{LL} P_{1LL}} = \frac{2298 - (1.00)(908.8)}{(1.00)(158.3)} = 8.78$$ Eqn. 6B.4.1-1

A11.12 -- Check the Chord Splice Resistance

Since the gusset plate is also performing as a tension chord splice, the resistance of the spliced section must be evaluated according to the provisions of Article L6B.2.6.6. For the net and gross section properties for the composite section of gusset and splice plates, see Section A11.3 and Figure A11.3-1.

The resistance of the chord splice can be calculated according to Section L6B.2.6.6.

Verify: $\dfrac{Kl\sqrt{12}}{t_g} < 25$.

$$\frac{(0.50)(6.0)\sqrt{12}}{(1.125)} = 9.24 < 25$$

$$C_{gross} = \phi_{cs} F_{cr} \left(\frac{S_g A_g}{S_g + e_{p\_gross} A_g} \right)$$

$$C_{gross} = 1.00(50)\left(\frac{(4933.5)(265.1)}{4933.5 + (22.7)(265.1)} \right) = 5971 \text{ kips}$$

$$C_{net} = \phi_{cs} F_u \left(\frac{S_n A_n}{S_n + e_{p\_net} A_n} \right)$$

$$C_{net} = 1.00(65)\left(\frac{(3899.1)(229.0)}{3899.1 + (26.2)(229.0)} \right) = 5863 \text{ kip}$$

The lesser resistance controls; in this case, it is controlled by fracture on the net section, C=5863 kips.

Next determine the factored load in the chord splice. Look at the loads on both sides of the splice and use the more severe.

Left Side

$$1.3\big(908.8 + (1012.8)(\cos(60°))\big) + 2.17\big(158.3 + (236)(\cos(60°))\big) = 2439 \text{ kips}$$

Right Side

$$1.3\big(1907 - (985)(\cos(60°))\big) + 2.17\big(394 - (236)(\cos(60°))\big) = 2438 \text{ kips}$$

Use the controlling loads from the left side of the splice.

Calculate the LFR rating factors per Equation 6B.4.1-1.

$$RF_{LFRinv} = \frac{C - \gamma_{DL} P_{DL}}{\gamma_{LL} P_{LL}} = \frac{5863 - 1.30(908.8 + (1012.8)(\cos(60°)))}{2.17(158.3 + (236)(\cos(60°)))} = 6.71$$

$$RF_{LFRopr} = RF_{LFRinv} \frac{\gamma_{LLinv}}{\gamma_{LLopr}} = 6.71 \frac{2.17}{1.30} = 11.20$$

A11.13 -- Check Tension Resistance of M2

The tension capacity will be governed by the lesser of the gross yield and net fracture on the Whitmore section (see Figure A11.4-1), and block shear (see Figure A11.4-2). Consider the section loss in the one gusset plate.

Calculate the effective area for calculating capacity of the Whitmore section in tension.

$A_e = A_n + \beta A_g \leq A_g$ Standard Specification Eqn. 10-4w

$A_e = 119.43 + (0.15)(132.93) \leq 132.93 = 139.37 \leq 132.93$ is not true, so $A_e = 132.93$

$P_n = \phi F_y A_e = 1.00 * 50 * 132.93 = 6646$ Article L6B.2.6.5

For the gross and net sections for block shear check, see Section A11.4. Calculate the block shear resistance of each gusset plate.

is $F_u A_{vn} \leq F_y \left(A_{vg1} + A_{vg2}\right)$?

$65(108.81) \leq 50(89.28 + 82.33) \rightarrow 7073 \leq 8580$, yes; therefore: Article L6B.2.6.5

$C = \phi_{bs} R_p \left(0.58 F_u A_{vn} + F_u A_{tn}\right)$

$C = (0.85)(1.0)\left[(0.58)(65)(108.81) + (65)(22.5)\right] = 4729.9$

The tension capacity is controlled by the minimum capacity based on the Whitmore yield and block shear. In this case, block shear controls, so C=4729.9 kips.

Calculate the rating factors.

$$RF_{LFR} = \frac{C - \gamma_{DL} P_{2DL}}{\gamma_{LL} P_{2LL}} = \frac{4729.9 - 1.30 * 1012.8}{2.17 * 236} = 6.66$$ Eqn. 6B.4.1-1

$$RF_{LFRopr} = RF_{LFRinv} \frac{\gamma_{LLinv}}{\gamma_{LLopr}} = 6.66 \frac{2.17}{1.30} = 11.12$$

A11.14 -- Check the Compression Resistance of M3

It is clear that this member will not control with only 12 kips of dead load, and since there is no live load, the rating factor cannot be calculated. For illustration, only the capacity calculations will be shown.

For the Whitmore buckling check, project the worst case section loss onto the Whitmore section (See Figure A11.5-1) in the direction of the member to calculate the equivalent gusset plate thickness.

Calculate the factored Whitmore buckling strength per gusset plate because only one of the plates has section loss. For the equivalent gusset plate thickness for the plate with section loss, see Section A11.5.

For the gusset plate with section loss

$K = 0.5$ Article L6B.2.6.4

$$r_{s1} = \frac{t_{eq\_whit}}{\sqrt{12}} = \frac{0.88}{\sqrt{12}} = 0.25$$

$$\text{is } \frac{KL_{mid}}{r_{s1}} \le \sqrt{\frac{2\pi^2 E}{F_y}} \rightarrow \left(\frac{0.5*5.93}{0.25}\right) \le \sqrt{\frac{2\pi^2(29000)}{50}} \rightarrow 11.67 \le 107?$$ Article L6B.2.6.4

Yes. Therefore:

$$F_{cr1} = F_y\left[1 - \frac{F_y}{4\pi^2 E}\left(\frac{KL_{mid}}{r_{s1}}\right)^2\right] = 50\left[1 - \frac{50}{4\pi^2(29000)}\left(11.67^2\right)\right] = 49.7$$ Article L6B.2.6.4

$$Whit_1 = 0.85 F_{cr} A_g = 0.85*49.7(76.35*0.88) = 2838.5$$ Article L6B.2.6.4

For the gusset plate without section loss

$K = 0.5$ Article L6B.2.6.4

$$r_{s2} = \frac{t_g}{\sqrt{12}} = \frac{1.125}{\sqrt{12}} = 0.32$$

$$\text{is } \frac{KL_{mid}}{r_{s2}} \le \sqrt{\frac{2\pi^2 E}{F_y}} \rightarrow \left(\frac{0.5*5.93}{0.32}\right) \le \sqrt{\frac{2\pi^2(29000)}{50}} \rightarrow 9.13 \le 107?$$ Article L6B.2.6.4

Yes. Therefore:

$$F_{cr2} = F_y\left[1 - \frac{F_y}{4\pi^2 E}\left(\frac{KL_{mid}}{r_{s2}}\right)^2\right] = 50\left[1 - \frac{50}{4\pi^2(29000)}\left(9.13^2\right)\right] = 49.8$$ Article L6B.2.6.4

$$Whit_2 = 0.85 F_{cr} A_g = 0.85*49.8(76.35*1.125) = 3637.2$$ Article L6B.2.6.4

Therefore, the total Whitmore buckling capacity is the sum of the two individual gusset plates.

$$Whit = \phi_{cg}\left(Whit_1 + Whit_2\right) = 1.00*\left(2838.5 + 3637.2\right) = 6475.7$$ Article L6B.2.6.4

This vertical member does not have any admissible partial shear planes that would control the compression resistance. Therefore, the capacity, C, is the Whitmore buckling strength.

A11.15 -- Check the Compression Resistance of M4

For the Whitmore buckling check, project the worst case section loss onto the Whitmore section in the direction of the member to calculate the equivalent gusset plate thickness. For the equivalent gusset plate thickness for the gusset plate with section loss, see Section A11.6 and Figure A11.6-3. Calculate the factored Whitmore buckling strength per gusset plate, because only one of the plates has section loss.

<u>For the gusset plate with section loss</u>

$K = 0.5$ Article L6B.2.6.4

$$r_{s1} = \frac{t_{eq\_whit}}{\sqrt{12}} = \frac{0.86}{\sqrt{12}} = 0.25$$

$$\text{is } \frac{KL_{mid}}{r_{s1}} \leq \sqrt{\frac{2\pi^2 E}{F_y}} \rightarrow \left(\frac{0.5*31.5}{0.25}\right) \leq \sqrt{\frac{2\pi^2(29000)}{50}} \rightarrow 63.2 \leq 107?$$ Article L6B.2.6.4

Yes. Therefore:

$$F_{cr1} = F_y\left[1 - \frac{F_y}{4\pi^2 E}\left(\frac{KL_{mid}}{r_{s1}}\right)^2\right] = 50\left[1 - \frac{50}{4\pi^2(29000)}\left(63.2^2\right)\right] = 41.3$$ Article L6B.2.6.4

$$Whit_1 = 0.85F_{cr}A_g = 0.85*41.3(55.64*0.86) = 1685.2$$ Article L6B.2.6.4

<u>For the gusset plate without section loss</u>

$K = 0.5$ Article L6B.2.6.4

$$r_{s2} = \frac{t_g}{\sqrt{12}} = \frac{1.125}{\sqrt{12}} = 0.32$$

$$\text{is } \frac{KL_{mid}}{r_{s2}} \leq \sqrt{\frac{2\pi^2 E}{F_y}} \rightarrow \left(\frac{0.5*31.5}{0.32}\right) \leq \sqrt{\frac{2\pi^2(29000)}{50}} \rightarrow 48.5 \leq 107?$$ Article L6B.2.6.4

Yes. Therefore:

$$F_{cr2} = F_y\left[1 - \frac{F_y}{4\pi^2 E}\left(\frac{KL_{mid}}{r_{s2}}\right)^2\right] = 50\left[1 - \frac{50}{4\pi^2(29000)}\left(48.5^2\right)\right] = 44.9$$ Article L6B.2.6.4

$$Whit_2 = 0.85F_{cr}A_g = 0.85*44.9(55.64*1.125) = 2387.0$$ Article L6B.2.6.4

Therefore, the total Whitmore buckling capacity is the sum of the two individual gusset plates.

$$Whit = \phi_{cg}\left(Whit_1 + Whit_2\right) = 1.00*\left(1685.2 + 2387.0\right) = 4072.2$$ Article L6B.2.6.4

Since this is a compression member, the partial plane shear yield capacity also needs to be calculated. For the equivalent gusset plate thickness on the partial shear plane for the gusset plate with section loss, see Section A11.6 and Figure All.6-2.

<u>For the gusset plate with section loss</u>

$$PS_1 = \frac{\Omega\left(0.58F_y\right)A_{shear}}{\cos(\theta)} = \frac{0.88\left(0.58*50\right)\left(63.80*0.99\right)}{\cos(30)} = 1863.4$$ Article L6B.2.6.3

For the gusset plate without section loss

$$PS_2 = \frac{\Omega(0.58F_y)A_{shear}}{\cos(\theta)} = \frac{0.88(0.58*50)(63.80*1.125)}{\cos(30)} = 2115.1$$ Article L6B.2.6.3

Therefore, the total partial plane shear capacity is the sum of the two individual gusset plates.

$$PS = \phi_{vy}(PS_1 + PS_2) = 1.00*(1863.4 + 2115.1) = 3978.5$$

The overall buckling capacity, C, is the lesser of the Whitmore buckling strength and that from the partial plane shear yield. In this case, partial plane shear yielding controls, and C=3978.5 kips.

Calculate the rating factors.

$$RF_{LFRinv} = \frac{C - \gamma_{DL}P_{4DL}}{\gamma_{LL}P_{4LL}} = \frac{3978.5 - 1.30*985}{2.17*236} = 5.27$$ Eqn. 6B.4.1-1

$$RF_{LFRopr} = RF_{LFRinv}\frac{\gamma_{LLinv}}{\gamma_{LLopr}} = 5.27\frac{2.17}{1.30} = 8.79$$

A11.16 -- Check Horizontal Shear Capacity of the Gusset Plate

First calculate the horizontal shear loads acting on the horizontal shear plan (see Figure A11.7-1). Note that the loads represented on the joint are based on envelope loads, not coincident loads. This will result in a conservative rating, which could be refined based on analysis of coincident shear loads.

$$P_{HDL} = P_{2HDL} + P_{4HDL} = (1012.8)(\cos 60°) + (985)(\cos 60°) = 998.9 \text{ kips}$$
$$P_{HLL} = P_{2HLL} + P_{4HLL} = (236)(\cos 60°) + (236)(\cos 60°) = 236 \text{ kips}$$

For the equivalent gusset plate thickness on the horizontal shear plane for the gusset plate with section loss, see Section A11.7.

Calculate the resistance to gross section shear yielding.

<u>For the gusset plate with section loss</u>

$$V_{ny1} = \phi_{vy}(0.58F_y)A_g\Omega = 1.00(0.58*50)(99.65*0.95)(0.88) = 2416$$ Article L6B.2.6.3

<u>For the gusset plate without section loss</u>

$$V_{ny2} = \phi_{vy}(0.58F_y)A_g\Omega = 1.00(0.58*50)(99.65*1.125)(0.88) = 2861$$ Article L6B.2.6.3

Therefore the total resistance to shear yielding is:

$$V_{ny} = V_{ny1} + V_{ny2} = 2416 + 2861 = 5277$$

Calculate the resistance to shear fracture. Note that the section loss in the one gusset plate does not affect the net section.

$$V_{nu} = \phi_{vu}(0.58F_u)A_{vn} = 0.85(0.58*65)\left(2*99.65 - 52\left(\frac{7}{8}+\frac{1}{8}\right)\right)1.125 = 5310$$ Article L6B.2.6.3

The capacity is determined by the lesser of the shear yield and shear fracture capacities. In this case, the shear yielding of the gross section governs, and C=5277 kips.

Calculate the rating factors.

$$RF_{LFRinv} = \frac{C - \gamma_{DL}P_{HDL}}{\gamma_{LL}P_{HLL}} = \frac{5277 - 1.30*998.9}{2.17*236} = 7.77$$ Eqn. 6B.4.1-1

$$RF_{LFRopr} = RF_{LFRinv}\frac{\gamma_{LLinv}}{\gamma_{LLopr}} = 7.77\frac{2.17}{1.30} = 12.97$$

A11.17 -- Check Vertical Shear Capacity of the Gusset Plate

This check will not control, because a valid shear plane cannot develop through the chord member.

A11.18 -- Summary of LFR Load Rating Factors

	Inventory	Operating
Member 1 (bolt shear and slip resistance control)	5.62	8.78
Member 2 (block shear controls)	6.66	11.12
Member 3	Rating factors were not calculated	
Member 4 (partial plane shear controls)	5.27	8.79
Member 5	Rating factors were not demonstrated; procedures are similar to those for Member 1.	
Chord Splice (net fracture controls)	6.71	11.20
Horizontal Shear (yielding controls)	7.77	12.97
Vertical Shear	This is not a relevant failure mode for this joint	

Table A11.18-1: Gusse Plate L2 LFR Summary

Note that the summary of rating factors presented in the above table considers solely the checks performed in this rating example. For a complete rating, the rating engineer must also consider the capacity of the bolts to ensure that the bolts do not govern the ratings for members 2 and 4 and for the chord splice.

2013 AASHTO BRIDGE COMMITTEE AGENDA ITEM: 47

SUBJECT: The Manual for Bridge Evaluation: APPENDIX A: Illustrative Examples, Table of Contents (T18-5)

TECHNICAL COMMITTEE: T-18 Bridge Management, Evaluation and Rehabilitation

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☐ **REVISION** ☒ **ADDITION** ☐ **NEW DOCUMENT**

☐ **DESIGN SPEC** ☐ **CONSTRUCTION SPEC** ☐ **MOVABLE SPEC**
☒ **MANUAL FOR BRIDGE EVALUATION** ☐ **SEISMIC GUIDE SPEC** ☐ **BRIDGE ELEMENT INSP GUIDE**
☐ **OTHER**

DATE PREPARED: 12/18/12
DATE REVISED:

AGENDA ITEM:

<u>Item #1</u>

Insert the following paragraph below Appendix A Table of Contents, Page A-i:

<u>The LRFR load rating examples included in Appendix A are carried over from past editions of the MBE and are based on the original LRFR live load factors. Reduced legal load factors and revised permit load factors were adopted in 2012 based on new research studies and recommendations. The reader should be cognizant of the changes resulting from the 2012 Interims to the MBE with regard to revised LRFR live load factors when using these load rating examples as a guide.</u>

<u>Item #2</u>

Insert the following note into the examples of Appendix A where there are references to Table 6A.4.4.2.3a-1, Table 6A.4.4.2.3b-1, or Table 6A.4.5.4.2a-1 in the right column of the illustrative examples:

<u>The reader should note that the live load factors used in this example are the original LRFR live load factors and do not reflect the new reduced legal load factors and permit load factors provided in the in the 2012 Interims to the MBE.</u>

References to Table 6A.4.4.2.3a-1, Table 6A.4.4.2.3b-1, or Table 6A.4.5.4.2a-1 appear in the following pages of Appendix A: A-18, A-30, A-34, A-35, A-63, A-64, A-65, A-116, A-126, A-157, A-161, A-188, A-223.

OTHER AFFECTED ARTICLES:

None

BACKGROUND:

The 2012 Interims provide revised LRFR permit load factors and reduced generalized legal load factors. The LRFR load rating examples included in Appendix A are carried over from past editions of the MBE and are based on the original LRFR live load factors. Reduced legal load factors were adopted in 2012 based on the reliability index to live load factor comparison studies completed in NCHRP 12-78. The permit load factors were updated following the recommendations of NCHRP 20-07 Task 285. The examples in this appendix developed prior to the adoption of the 2012 Interims are intended to illustrate the application of the load rating provisions and as such continue to reflect the use of the original LRFR live load factors. The reader should be cognizant of the changes resulting from the 2012 Interims with regard to LRFR live load factors when using these examples as a guide.

ANTICIPATED EFFECT ON BRIDGES:

The proposed statement will alert the user regarding live load factor changes resulting from the 2012 Interims, to ensure that the examples are used correctly. It is important that the examples in Appendix A are consistent with the latest MBE provisions. The inclusion of this statement is intended to maintain that consistency.

REFERENCES:

2012 Interims to the MBE

OTHER:

None

2013 AASHTO BRIDGE COMMITTEE AGENDA ITEM: 48

SUBJECT: AASHTO Guide Manual for Bridge Element Inspection: Section 3 (T18-7)

TECHNICAL COMMITTEE: T-18 Bridge Management, Evaluation and Rehabilitation

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☒ **REVISION** ☐ **ADDITION** ☐ **NEW DOCUMENT**

☐ **DESIGN SPEC** ☐ **CONSTRUCTION SPEC** ☐ **MOVABLE SPEC**
☐ **MANUAL FOR BRIDGE EVALUATION** ☐ **SEISMIC GUIDE SPEC** ☒ **BRIDGE ELEMENT INSP GUIDE**
☐ **OTHER**

DATE PREPARED: 1/11/13
DATE REVISED: 4/15/13

AGENDA ITEM:

Revise the AASHTO Guide Manual for Bridge Element Inspection as shown in Attachment C **(Provided on CD)**.

OTHER AFFECTED ARTICLES:

This ballot item is proposed to replace the Guide Manual for Bridge Element Inspection and change the status of the Guide Manual into a full AASHTO Manual for Bridge Element Inspection. It is presented in pdf file format. Word files are available for the individual sections.

BACKGROUND:

In 2011 AASHTO created and published the AASHTO Guide Manual for Bridge Element Inspection a major improvement and replacement of the AASHTO Guide to Commonly Recognized Elements (CoRE) published in 1997.

This AASHTO Guide Manual for Bridge Element Inspection is built on the element level condition assessment methods developed in the AASHTO Guide for Commonly Recognized Structural Elements. Improvements were made to fully capture the condition of the elements by reconfiguring the element language to utilize multiple distress paths within the defined condition states.

The AASHTO Guide Manual for Bridge Element Inspection provided a comprehensive set of bridge elements that was designed to be flexible in nature to satisfy the needs of all agencies. The combined set of both National and Bridge Management elements captured the components necessary for an agency to manage all aspects of the bridge inventory utilizing the full capability of a Bridge Management System (BMS).

The guide manual provided the following benefits:

- Established a set of National Bridge Elements to be used as a minimum standard.
- Established a set of expanded Bridge Management Elements that can be used to capture additional condition data to fully utilize a Bridge Management System.
- Standardized the number of condition states at four.
- Changed the units for decks and slabs to area based units.
- Separated wearing surfaces from deck and slab elements.
- Separated protective coatings (paint) from steel elements.
- Supported flexibility for agencies to develop elements.

The goal of the guide manual was to capture the condition of bridges in a simple way that could be standardized across the nation while providing the flexibility to be adapted to both large and small agency settings. The guide manual was not intended to supplant proper training or the exercise of engineering judgment by the inspector or professional engineer.

Over the last year several agencies have been utilizing the AASHTO Guide Manual for Bridge Element Inspection for the inspection of their bridges, and FHWA has developed a course titled “Introduction to Element Level Bridge Inspection” that is based on the AASHTO Guide Manual for Bridge Element Inspection. AASHTO has also developed a data migrator to facilitate the conversion of CoRe elements into the new guide elements. T-18 has received questions, input, suggested improvements and feedback regarding the guide manual from these agencies, states, and the FHWA resulting from its use.

With the inclusion of bridge element inspection on the National Highway System in the Moving Ahead for Progress in the 21 Century Act (MAP-21), T-18 believes it is appropriate to propose the improvements summarized below and included in the ballot item and transition the guide manual to full manual status. The FHWA plans to reference this manual if approved as part of the element level data collection requirement of MAP-21.

Improvements included in the proposed transition to the AASHTO Manual for Bridge Element Inspection from the Guide Manual include:

1) New Format - The condition state table for each element has been consolidated. The result of this change is a cleaner and easier to read element description.
2) Generic labels have been added along with color coding for the condition state titles (good, fair, poor and severe).
3) Comments that had been received have been incorporated into the manual.
4) The defect flags available for any given element have been expanded to include all defined element defects. For example, a reinforced concrete deck element now has the ability to define a spalling and exposed reinforcing defect that was previously unavailable.
5) U.S. Customary Units are used throughout the manual.
6) A number of "other" material elements have been added to capture new materials and unusual materials (FRP, etc.)
7) Appendix B, C and D have been updated.
8) New elements were added for Prestressed Concrete Culverts and the "top flange" element has been split into a Prestressed Concrete and Reinforced Concrete element.

ANTICIPATED EFFECT ON BRIDGES:

The new AASHTO Manual for Bridge Element Inspection will result in better condition assessments that will allow local, State, and Federal agencies to more accurately report the condition of the bridge inventory in the United States. The impact of this improvement will be better decision making, better trade-off analysis and better representation of bridge needs.

The manual incorporates a number of suggested improvements from bridge inspectors, bridge management engineers and bridge owners to improve the ease of field measurement, condition assessment and presentation of the bridge element condition information.

REFERENCES:

AASHTO Guide Manual for Bridge Element Inspection, 1st Ed.

OTHER:

None

2013 AASHTO BRIDGE COMMITTEE

SUBJECT: LRFD Bridge Construction Specifications

Editorial revisions and additions to various articles of the AASHTO LRFD Bridge Construction Specifications

2013 EDITORIAL CHANGES – CONSTRUCTION

Location of Change	Current Text	Proposed Text
Article 11.4.12.1	Flanges of curved, welded girders may be cut to the radii specified in the contract documents or curved by applying heat as specified in the succeeding articles providing the radii is not less than allowed by Article 10.15.2, "Minimum Radius of Curvature," of the AASHTO *Standard Specifications for Highway Bridges*, 17th Edition, Design Specifications.	Flanges of curved, welded girders may be cut to the radii specified in the contract documents or curved by applying heat as specified in the succeeding articles ~~providing the radii is not less than allowed by Article 10.15.2, "Minimum Radius of Curvature," of the AASHTO *Standard Specifications for Highway Bridges*, 17th Edition, Design Specifications~~.

2013 AASHTO BRIDGE COMMITTEE

SUBJECT: LRFD Bridge Design Specifications

Editorial revisions and additions to various articles of the AASHTO LRFD Bridge Design Specifications

2013 EDITORIAL CHANGES – DESIGN

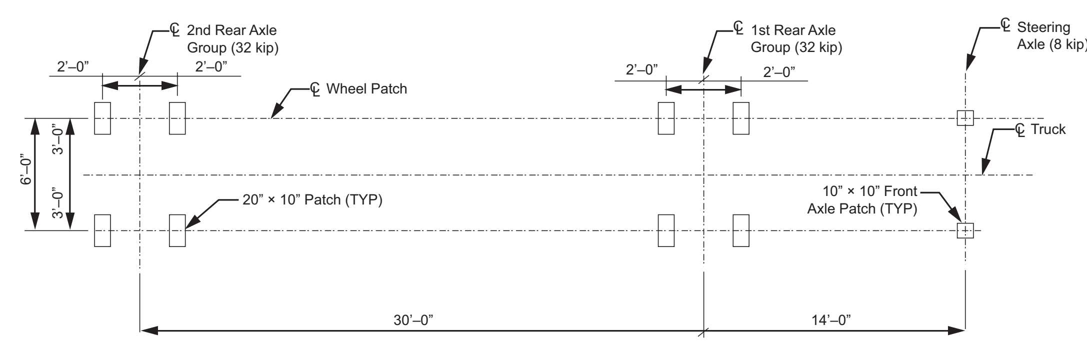
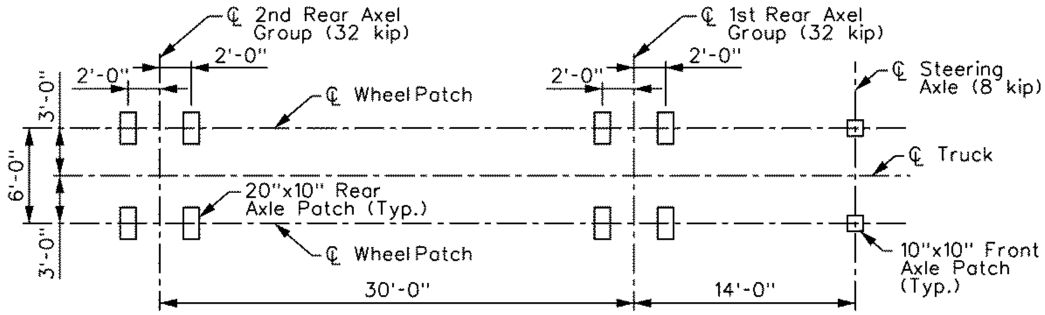
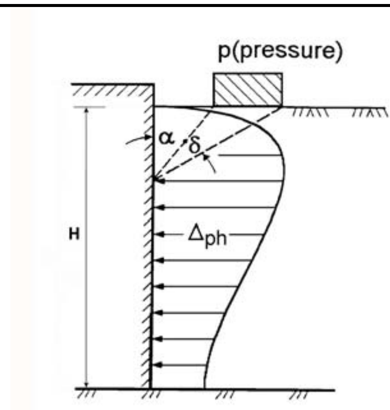
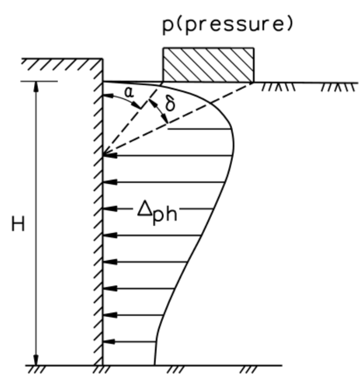
<table>
<tr><th>Location of Change</th><th>Current Text</th><th>Proposed Text</th></tr>
<tr><td>Section 2, Article C2.5.2.6.2, 4th paragraph</td><td>For a straight multibeam bridge, this is equivalent to saying that the distribution factor for deflection is equal to the number of lanes divided by the number of beams.</td><td>For a straight ~~multibeam~~ girder bridge, this is equivalent to saying that the distribution factor for deflection is equal to the number of lanes divided by the number of beams.</td></tr>
<tr><td>Section 2, Article 2.5.2.7.1, Title</td><td>Exterior Beams on Multibeam Bridges</td><td>Exterior Beams on ~~Multibeam~~ Girder System Bridges</td></tr>
<tr><td>Section 3, Article 3.3.1</td><td>δ = angle of truncated ice wedge (degrees); friction angle between fill and wall (degrees); angle between foundation wall and a line connecting the point on the wall under consideration and a point on the bottom corner of the footing furthest from the wall (rad) (C3.9.5) (3.11.5.3) (3.11.6.2)</td><td>δ = angle of truncated ice wedge (degrees); friction angle between fill and wall (degrees); angle between ~~foundation wall and a line connecting~~ the far and near corners of a footing measured from the point on the wall under consideration ~~and a point on the bottom corner of the footing furthest from the wall~~ (rad) (C3.9.5) (3.11.5.3) (3.11.6.2)</td></tr>
<tr><td>Section 3, Article 3.3.2</td><td>PS = secondary forces from post-tensioning</td><td>PS = secondary forces from post-tensioning for strength limit states; total prestress forces for service limit states</td></tr>
<tr><td>Section 3, Figure 3.6.1.4.1-1 & Title</td><td colspan="2">Current


Figure 3.6.1.4.1-1—Refined Design Truck Footprint for Fatigue Design</td></tr>
<tr><td></td><td colspan="2"></td></tr>
</table>

Location of Change	Current Text	Proposed Text
	Proposed  **Figure 3.6.1.4.1-1—Refined Design Truck Footprint for Fatigue Design of Orthotropic Decks**	
Section 3, Eq. 3.11.5.7.2b-2	$k_a = 1 - \frac{4S_u}{\gamma_s H} + 2\sqrt{2}\frac{d}{H}\left(\frac{1-5.14S_{ub}}{\gamma_s H}\right) \geq 0.22$	$k_a = 1 - \frac{4S_u}{\gamma_s H} + 2\sqrt{2}\frac{d}{H}\left(1-\frac{5.14S_{ub}}{\gamma_s H}\right) \geq 0.22$
Section 3, Figure 3.11.6.2.1-1	 **Figure 3.11.6.2-1—Horizontal Pressure on Wall Caused by a Uniformly Loaded Strip**	 **Figure 3.11.6.2-1—Horizontal Pressure on Wall Caused by a Uniformly Loaded Strip**
Section 4, Article 4.2	---	*Multibeam Decks*—Bridges with superstructure members consisting of adjacent precast sections with the top flange as a complete full-depth integral deck or a structural deck section placed as an overlay. Sections can be closed cell boxes or open stemmed.
Section 4, Article 4.3	g_m = multiple lane live load distribution factor (4.6.2.2.4)	g_m = multiple lane live load distribution factor ~~(4.6.2.2.4)~~ (4.6.2.2.5)
	g_1 = single lane live load distribution factor (4.6.2.2.4)	g_1 = single lane live load distribution factor ~~(4.6.2.2.4)~~ (4.6.2.2.5)
Section 4, Article C4.6.2.1.8, 3rd paragraph, 1st sentence	The reduction factor of 1.5 in the last sentence of Article 4.6.2.1.8 accounts for smaller dynamic load allowance (15 percent vs. 33 percent), smaller load factor (0.75 vs. 1.75) and no multiple presence (1.0 vs. 1.2) when considering the Fatigue I limit state.	The reduction factor of 1.5 in the last sentence of Article 4.6.2.1.8 accounts for smaller dynamic load allowance (15 percent vs. 33 percent), smaller load factor (~~0.75~~ 1.50 vs. 1.75) and no multiple presence (1.0 vs. 1.2) when considering the Fatigue I limit state.

Location of Change	Current Text	Proposed Text
Section 4, Article 4.6.2.2.1	---	*k* = factor used in calculation of distribution factor for multibeam bridges
Section 4, Article 4.6.2.2.1, 15th paragraph	Except as permitted by Article 2.5.2.7.1, regardless of the method of analysis used, i.e., approximate or refined, exterior girders of multibeam bridges shall not have less resistance than an interior beam.	Except as permitted by Article 2.5.2.7.1, regardless of the method of analysis used, i.e., approximate or refined, exterior girders of ~~multibeam~~ girder system bridges shall not have less resistance than an interior beam.
Section 4, Table 4.6.2.2.2b-1, Column 2, Row 7	f g if sufficiently connected to act as a unit	f, also g if sufficiently connected to act as a unit
	h g, i, j if connected only enough to prevent relative vertical displacement at the interface	h, also g, i, j if connected only enough to prevent relative vertical displacement at the interface
Section 4, Table 4.6.2.2.2d-1, Column 2, Row 8	h i, j if connected only enough to prevent relative vertical displacement at the interface	h, also i, j if connected only enough to prevent relative vertical displacement at the interface
Section 4, Table 4.6.2.2.2e-1, Column 1, Row 3	Concrete Deck on Concrete Spread Box Beams, Cast-in-Place Multicell Box Concrete Box Beams and Double T- Sections used in Multibeam Decks	Concrete Deck on Concrete Spread Box Beams, Cast-in-Place Multicell Box, Concrete ~~Box~~ Beams ~~and Double T- Sections~~ used in Multibeam Decks
Section 4, Table 4.6.2.2.2e-1, Column 2, Row 3	b, c, d, f, g	b, c, d, f, g, h, also i and j if sufficiently connected to prevent vertical displacement at the interface
Section 4, Table 4.6.2.2.3a-1, Column 2, Row 8	h i, j if connected only enough to prevent relative vertical displacement at the interface	h, also i, j if connected only enough to prevent relative vertical displacement at the interface
Section 4, Table 4.6.2.2.3b-1, Column 2, Row 8	h i, j if connected only enough to prevent relative	h, also i, j if connected only enough to prevent relative vertical displacement at the interface

Location of Change	Current Text	Proposed Text
	vertical displacement at the interface	
Section 4, Article C4.6.2.2.3c, 2nd paragraph, 1st sentence	The equal treatment of all beams in a multibeam bridge is conservative regarding positive reaction and shear.	The equal treatment of all beams in a ~~multibeam~~ deck bridge is conservative regarding positive reaction and shear.
Section 4, Article 4.6.2.2.5, Eq. No. 1	$G = G_p\left(\frac{g_1}{Z}\right) + G_D\left(g_m - \frac{g_1}{Z}\right)$ (4.6.2.2.4-1)	$G = G_p\left(\frac{g_1}{Z}\right) + G_D\left(g_m - \frac{g_1}{Z}\right)$ (~~4.6.2.2.4-1~~) (4.6.2.2.5-1)
Section 4, Article 4.6.2.7, Title	**Lateral Wind Load Distribution in Multibeam Bridges**	**Lateral Wind Load Distribution in ~~Multibeam~~ Girder System Bridges**
Section 5, Article 5.5.3.1, 2nd paragraph	In regions of compressive stress due to permanent loads and prestress in reinforced concrete components, fatigue shall be considered only if this compressive stress is less than the maximum tensile live load stress resulting from the Fatigue I load combination as specified in Table 3.4.1-1 in combination with the provisions of Article 3.6.1.4.	In regions of compressive stress due to unfactored permanent loads and prestress in reinforced concrete components, fatigue shall be considered only if this compressive stress is less than the maximum tensile live load stress resulting from the Fatigue I load combination as specified in Table 3.4.1-1 in combination with the provisions of Article 3.6.1.4.
Section 5, Article 5.5.3.1, 5th paragraph	For fully prestressed components in other than segmentally constructed bridges, the compressive stress due to the Fatigue I load combination and one-half the sum of effective prestress and permanent loads shall not exceed $0.40f'_c$ after losses.	For fully prestressed components in other than segmentally constructed bridges, the compressive stress due to the Fatigue I load combination and one-half the sum of the unfactored effective prestress and permanent loads shall not exceed $0.40f'_c$ after losses.
Section 5, Article 5.5.3.2, where list	f_{min} = minimum live-load stress resulting from the Fatigue I load combination, combined with the more severe stress from either the permanent loads or the permanent loads, shrinkage, and creep-induced external loads; positive if tension, negative if compression (ksi)	f_{min} = minimum live-load stress resulting from the Fatigue I load combination, combined with the more severe stress from either the unfactored permanent loads or the unfactored permanent loads, shrinkage, and creep-induced external loads; positive if tension, negative if compression (ksi)
Section 5, Eq. 5.8.4.2-1	$V_{ui} = \frac{V_{u1}}{b_{vi}d_v}$	$v_{ui} = \frac{V_{u1}}{b_{vi}d_v}$
Section 5, Article C5.8.4.2, notation	V_1 = the factored vertical… M_1 = the factored moment… Δl = unit length segment of girder	~~V_1~~ V_{u1} = the factored vertical… ~~M_1~~ M_{u1} = the factored moment… ~~Δl~~ $\Delta \ell$ = unit length segment of girder

Location of Change	Current Text	Proposed Text
Section 5, Article C5.10.4.3, 2nd & 3rd paragraphs	Resistance to in-plane forces in curved girders may be provided by increasing the concrete cover over the duct, by adding confinement tie reinforcement or by a combination thereof. It is not the purpose of this Article to encourage the use of curved tendons around re-entrant corners or voids. Where possible, this type of detail should be avoided.	In-plane force effects are due to a change in direction of the tendon within the plane of curvature. Resistance to in-plane forces in curved girders may be provided by increasing the concrete cover over the duct, by adding confinement tie reinforcement or by a combination thereof. Figure C5.10.4.3.1a-1 shows an in-plane deviation in the vertical curve, and Figure C5.10.4.3.1a-2 shows a potential in-plane deviation in the horizontal curve. ~~It is not the purpose of this article to encourage the use of curved tendons around re-entrant corners or voids. Where possible, this type of detail should be avoided.~~ Out-of-plane force effects are due to the spreading of the wires or strands within the duct. Out-of-plane force effects are shown in Figure C5.10.4.3.2-1 and can be affected by ducts stacked vertically or stacked with a horizontal offset.
Section 5, Article 5.10.5	Unless a vibration analysis indicates otherwise, the unsupported length of external tendons shall not exceed 25.0 ft.	Unless a vibration analysis indicates otherwise, the unsupported length of external tendons shall not exceed 25.0 ft. External tendon supports in curved concrete box girders shall be located far enough away from the web to prevent the free length of tendon from bearing on the web at locations away from the supports. When deviation saddles are required for this purpose, they shall be designed in accordance with Article 5.10.9.3.7.
Section 5, Article 5.13.2.2, 2nd paragraph	Intermediate diaphragms may be used between beams in curved systems or where necessary to provide torsional resistance and to support the deck at points of discontinuity or at right angle points of discontinuity or at angle points in girders.	~~Intermediate diaphragms may be used between beams in curved systems or where necessary to provide torsional resistance and to support the deck at points of discontinuity or at right angle points of discontinuity or at angle points in girders.~~
Article C6.5.5, 1st paragraph, last sentence	A special inspection of joints and connections, particularly in fracture critical members, should be performed as described in *The Manual for Bridge Evaluation* (2011) after a seismic event.	A special inspection of joints and connections, particularly in fracture critical members, should be performed as described in ~~*The Manual for Bridge Evaluation*~~ AASHTO (2011a) after a seismic event.
Table 6.6.1.2.3-1, pgs. 6-38 through 6-44, Table Heading	Threshold $(\Delta f)_{TH}$ ksi	Threshold $(\cancel{\Delta f}\Delta F)_{TH}$ ksi
Table 6.6.1.2.3-1 (all pages) and Table 6.6.1.2.5-1, Table Heading	(ksi^3)	$(ksi^{\cancel{3}})^{3}$

Location of Change	Current Text	Proposed Text
Table 6.6.1.2.3-1, Condition 6.1	For any transition radius with the weld termination not ground smooth (Note: Condition 6.2, 6.3 or 6.4, as applicable, shall also be checked.)	For any transition radius with the weld termination not ground smooth. **(Note: Condition 6.2, 6.3 or 6.4, as applicable, shall also be checked.)**
Table 6.6.1.2.3-1, Condition 6.3	For any weld transition radius with the weld reinforcement not removed (Note: Condition 6.1 shall also be checked.)	For any weld transition radius with the weld reinforcement not removed. **(Note: Condition 6.1 shall also be checked.)**
Table 6.6.1.2.3-1, Section 8 Row	Section 8—Miscellaneous	Section 8~~—Miscellaneous~~Orthotropic Deck Details
Article C6.10.8.2.3, last paragraph	To avoid a significant reduction in the lateral torsional buckling resistance, flange transitions can be located within 20 percent of the unbraced length from the brace point with the smaller moment, given that the lateral moment of inertia of the flange or flanges of the smaller section is equal to or larger than one-half of the corresponding value in the larger section.	~~To avoid a significant reduction in the lateral torsional buckling resistance, flange transitions can be located within 20 percent of the unbraced length from the brace point with the smaller moment, given that the lateral moment of inertia of the flange or flanges of the smaller section is equal to or larger than one-half of the corresponding value in the larger section.~~
Article C6.16.1, 1st sentence	These specifications are based on the recent work published by Itani et al. (2010), NCHRP (2002, 2006), MCEER/ATC (2003), Caltrans (2006), AASHTO's *Guide Specifications for LRFD Seismic Bridge Design* (2009), and AISC (2005 and 2005b).	These specifications are based on the recent work published by Itani et al. (2010), NCHRP (2002, 2006), MCEER/ATC (2003), Caltrans (2006), ~~AASHTO's *Guide Specifications for LRFD Seismic Bridge Design*~~ AASHTO (~~2009~~2011), and AISC (~~2005 and 2005b~~2010 and 2010b).
Article C6.16.1, 6th paragraph, last sentence	The designer may find information on this topic in AASHTO's *Guide Specifications for LRFD Seismic Bridge Design* (2009) and MCEER/ATC (2003) to complement information available elsewhere in the literature.	The ~~designer~~ Engineer may find information on this topic in ~~AASHTO's *Guide Specifications for LRFD Seismic Bridge Design*~~ AASHTO (~~2009~~2011) and MCEER/ATC (2003) to complement information available elsewhere in the literature.
Article C6.16.2, 7th sentence	To this end, the expected yield strength of various steel materials has been established through a survey of mill test reports and ratios of the expected to nominal yield strength, R_y, have been provided elsewhere (AISC, 2005b) and they are adopted herein.	To this end, the expected yield strength of various steel materials has been established through a survey of mill test reports and ratios of the expected to nominal yield strength, R_y, have been provided elsewhere (AISC, ~~2005~~2010b) and they are adopted herein.
Article C6.16.4.2, last paragraph, 2nd sentence	In lieu of experimental test data, the overstrength ratio for shear key resistance may be obtained from *Guide Specifications for LRFD Seismic Bridge Design* (2009).	In lieu of experimental test data, the overstrength ratio for shear key resistance may be obtained from the *Guide Specifications for LRFD Seismic Bridge Design* (~~2009~~2011).
Article 6.17—References	AISC. 2005. *Specification for Structural Steel Buildings*, ANSI/AISC 360-05. American Institute of Steel Construction, Chicago, IL. AISC. 2005a. *Steel Construction Manual*, 13th Edition. American Institute of Steel Construction, Chicago, IL.	AISC.~~2005~~2010. *Specification for Structural Steel Buildings*, ANSI/AISC 360-~~05~~10. American Institute of Steel Construction, Chicago, IL. AISC. ~~2005a~~2010a. *Steel Construction Manual*, ~~13th~~14th Edition. American Institute of Steel Construction, Chicago, IL. **Note: Change all existing references to AISC (2005) throughout Section 6 to AISC (2010). Change all existing references to AISC (2005a) throughout Section 6 to AISC (2010a).** ***Please do***

Location of Change	Current Text	Proposed Text
		not change any existing references to AISC in Section 6 other than AISC (2005) and AISC (2005a), except as noted herein.
	AISC. 1994. *Load and Resistance Factor Design Specifications for Structural Joints Using ASTM A325 or A490 Bolts*. American Institute of Steel Construction, Chicago, IL.	AISC. ~~1994~~2009. ~~*Load and Resistance Factor Design*~~ *Specification~~s~~ for Structural Joints Using ~~ASTM A325 or A490~~ High-Strength Bolts*. Research Council on Structural Connections, available from the American Institute of Steel Construction, Chicago, IL, December 31, 2009. **Note: Change existing reference to (AISC, 1994) in Article C6.13.2.8 to (AISC, 2009).**
	AISC. 2005b. *Seismic Provisions for Structural Steel Buildings*. American Institute of Steel Construction, Chicago, IL.	AISC. ~~2005~~2010b. *Seismic Provisions for Structural Steel Buildings*. American Institute of Steel Construction, Chicago, IL.
	AASHTO. 2010. *AASHTO LRFD Bridge Design Specifications,* PE Edition, LRFD-PE, American Association of State Highway and Transportation Officials, Washington, DC, Fifth Edition with 2010 Interim, U.S. customary units.	AASHTO. ~~2010~~2012. *AASHTO LRFD Bridge Design Specifications,* PE Edition, LRFD-PE, American Association of State Highway and Transportation Officials, Washington, DC, ~~Fifth Edition with 2010 Interim~~Sixth Edition, U.S. customary units. **Note: Change all existing references to AASHTO (2010) throughout Section 6 to AASHTO (2012).**
	AASHTO. 2011. *The Manual for Bridge Evaluation,* Second Edition, MBE-2-M. American Association of State Highway and Transportation Officials, Washington, DC.	AASHTO. 2011a. *The Manual for Bridge Evaluation,* Second Edition, MBE-2-M. American Association of State Highway and Transportation Officials, Washington, DC.
Section 9, Figure 9.8.3.4.4-1, Title	**Local Structural Stress**	**Local Structural Stress for Level 3 Design of Orthotropic Decks**
Section 9, Article C9.8.3.6.2, 1st paragraph, last sentence	Levels between 75 and 95 percent, with a target of 80 percent, are achievable and the lower bound of 70 percent is supported by research (Xiao, 2008).	Levels between 75 and 95 percent, with a target of 80 percent, are achievable and the lower bound of 70 percent is supported by research ~~(Xiao, 2008)~~ (Sim and Uang, 2007).
Section 9, Article 9.10—References	Xiao. 2008. *Effect of Fabrication Procedures and Weld Melt-Through on Fatigue Resistance of Orthotropic Steel Deck Welds, Final Report*, No. CA08-0607. Department of Structural Engineering, University of California, San Diego, CA.	~~Xiao. 2008. *Effect of Fabrication Procedures and Weld Melt-Through on Fatigue Resistance of Orthotropic Steel Deck Welds, Final Report*, No. CA08-0607. Department of Structural Engineering, University of California, San Diego, CA.~~
Section 10, Article 10.8.3.7.2, 1st paragraph	The uplift resistance of a single straight-sided drilled shaft should be estimated in a manner similar to that for determining side resistance for drilled shafts in compression, as specified in Article 10.8.3.3.	The uplift resistance of a single straight-sided drilled shaft should be estimated in a manner similar to that for determining side resistance for drilled shafts in compression, as specified in Article ~~10.8.3.3~~ 10.8.3.5.
Section 12, Table 12.6.6.3-1, Rows 10 and 12	| Type | Condition | Minimum Cover |	| ~~Type~~ | ~~Condition~~ | ~~Minimum Cover~~ |

Location of Change	Current Text	Proposed Text
Section 12, Table 12.6.6.3-1, Row 11	$B_c/8$ or $B'_c/8$, whichever is greater, $\geq$ 12.0 in.	$B_c/8$ or $B'_c/8$, whichever is greater, $\geq$ 12.0 in.
Index	lateral wind load distribution in multibeam bridges…..4-62	lateral wind load distribution in ~~multibeam~~ girder system bridges…..4-62
	Wind load multibeam bridges…..4-59	Wind load ~~multibeam~~ girder system bridges…..4-59

2013 AASHTO BRIDGE COMMITTEE

SUBJECT: Movable Highway Bridge Design Specifications

Editorial revision to the AASHTO LRFD Movable Highway Bridge Design Specifications

2013 EDITORIAL CHANGES – MOVABLE

Location of Change	Current Text	Proposed Text
Article 6.6.3.2, Eq. 4	$C_S = a\ (\sigma_{ut})^b$	$C_S = a\left(\dfrac{\sigma_{ut}}{1000}\right)^b$
Article 6.7.4.2, 2nd bullet	• Where less twist is desirable, as in shafts driving end-lifting devices…………(0.006)	• Where less twist is desirable, as in shafts driving end-lifting devices…………~~(0.006)~~ (0.00667)
Article 6.7.5.1, 5th & 6th paragraphs	For full depth spur gear teeth, the addendum shall be the inverse of the diametral pitch (equal to the tooth module), the dedendum shall be 1.250 divided by the diametral pitch (1.157 times the module), and the circular pitch shall be π divided by the diametral pitch (π times the tooth module). The face width of a spur gear should be not less than $8/P_d$, nor more than $14P_d$ (not less than 8, nor more than 14, times the tooth module).	For full depth spur gear teeth, the addendum shall be the inverse of the diametral pitch (equal to the tooth module), the dedendum shall be 1.250 divided by the diametral pitch ~~(1.157 times the module)~~, and the circular pitch shall be π divided by the diametral pitch ~~(π times the tooth module)~~. The face width of a spur gear should be not less than $8/P_d$, nor more than $\cancel{14P_d}$ $\underline{14/P_d}$ ~~(not less than 8, nor more than 14, times the tooth module)~~.
Article 6.7.5.1, 2nd bullet list	• for pinions other than motor pinions, transmitting power for moving the span……………………………3.14 in. • for motor pinions………………….4.19 in. • for main rack teeth………………..2.09 in	• for pinions other than motor pinions, transmitting power for moving the span…………………………3.14 ~~in.~~ in.$^{-1}$ • for motor pinions………………4.19 ~~in.~~ in.$^{-1}$ • for main rack teeth……………..2.09 ~~in.~~ in.$^{-1}$
Article 6.7.5.2.2, second to last bullet	• for 99 percent reliability	• 1.00 for 99 percent reliability
Article 6.7.5.2.4, where list	$_f$ = stress correction factor = 1 (dim)	$\underline{K}_f$ = stress correction factor = 1 (dim)
Article 6.7.7.2.4, 2nd where list	X_o = a static axial load factor (dim) Y_o = a static radial load factor (dim)	X_o = a static ~~axial~~ radial load factor (dim) Y_o = a static ~~radial~~ axial load factor (dim)
Table C6.7.10.1-1, Rows 12 & 13	$4\ ^1/_2$ | $5\ ^1/_2$ | $1\ ^1/_2 \times 1\ ^1/_2$ $5\ ^1/_2$ | $6\ ^1/_2$ | $32 \times 1\ ^1/_2$	$4\ ^1/_2$ | $5\ ^1/_2$ | $\cancel{1\ ^1/_2 \times 1\ ^1/_2}$ $\underline{1^1/_4 \times 1^1/_4}$ $5\ ^1/_2$ | $6\ ^1/_2$ | $\cancel{32 \times 1\ ^1/_2}$ $\underline{1^1/_2 \times 1^1/_2}$
Article C6.8.3.3.4, last paragraph	When determining P_o for counterweight ropes, only inertial loads are effective.	**Move this paragraph across from the "P_o" in the where list of the Specification** When determining P_o for counterweight ropes, only inertial loads are effective.
Article C6.8.3.3.6, last paragraph	E_R (psi) — Percent of Ultimate Load 10.8×10^6 (74 500) — 0-20 12×10^6 (83 000) — 21-65	E_R (psi) — Percent of Ultimate Load 10.8×10^6 ~~(74 500)~~ — 0-20 12×10^6 ~~(83 000)~~ — 21-65

Location of Change	Current Text	Proposed Text
Article 6.8.3.4.2, last paragraph, last sentence	The distance center-to-center of grooves shall be at least 6.5 mm more than the diameter of the rope.	The distance center-to-center of grooves shall be at least ~~6.5 mm~~ 0.25 in. more than the diameter of the rope.

American Association of State Highway and Transportation Officials Highway Subcommittee on Bridges and Structures

Chair

Gregg Fredrick, P.E.
Assistant Chief Engineer, Engineering and Planning
Wyoming Department of Transportation
5300 Bishop Boulevard
Cheyenne, WY 82009

Phone Number (307) 777-4484
Fax Number (307) 777-4163
E-mail Address: Gregg.fredrick@wyo.gov

Vice Chair

Bruce V. Johnson P.E.
State Bridge Engineer
Oregon Department of Transportation
4040 Fairview Industrial Drive, SE
Salem, OR 97302-1142

Phone Number (503) 986-3864
Fax Number (503) 986-3407
E-mail Address: bruce.v.johnson@odot.state.or.us

Secretary

M. Myint Lwin
Director, Office of Bridge Technology
Federal Highway Administration
Room E75-318
1200 New Jersey Avenue, S.E.
Washington, DC 20590

Phone Number (202) 366-4589
Fax Number (202) 366-3077
E-mail Address: myint.lwin@ dot.gov

Liaison

Kelley Rehm
Program Manager for Bridges and Structures
American Association of State Highway and Transportation Officials
602 Idlewood Drive
Mount Juliet, TN 37122

Phone Number (859) 433-9623
Fax Number (866) 301-1322
E-mail Address: krehm@aashto.org

State Member

Alabama AL

John F. "Buddy" Black P.E. (Primary Member)
Bridge Engineer
Alabama Department of Transportation
1409 Coliseum Boulevard
Montgomery, AL 36110-2060

Phone Number (334) 242-6004
Fax Number (334) 353-6502
E-mail Address: blackj@dot.state.al.us

Eric J. Christie P.E. (Member)
Assistant State Maintenance Engineer for Bridges
Alabama Department of Transportation
Maintenance Bureau, Room H-101
1409 Coliseum Boulevard
Montgomery, AL 36110-2060

Phone Number (334) 242-6281
Fax Number (334) 242-6378
E-mail Address: christiee@dot.state.al.us

William "Tim" Colquett P.E. (Member)
Assistant Bridge Engineer
Alabama Department of Transportation
1409 Coliseum Boulevard
Montgomery, AL 36110-2060

Phone Number (334) 242-6007
Fax Number (334) 353-6502
E-mail Address: colquettw@dot.state.al.us

Alaska AK

Richard A. Pratt P.E. (Primary Member)
Chief Bridge Engineer
Alaska Department of Transportation and Public Facilities
P.O. Box 112500
Juneau, AK 99811-2500

Phone Number (907) 465-8890
Fax Number (907) 465-6947
E-mail Address: richard.pratt@alaska.gov

Arizona AZ

Jean A. Nehme Ph.D., P.E. (Primary Member)
State Bridge Engineer
Arizona Department of Transportation
205 South 17th Avenue, Mail Drop 613E
Phoenix, AZ 85007-3213

Phone Number (602) 712-7481
Fax Number (602) 712-3056
E-mail Address: jnehme@azdot.gov

Arkansas AR

Carl Fuselier (Primary Member)
Division Head, Bridge
Arkansas State Highway and Transportation Department
State Highway Building
P.O. Box 2261, 10324 Interstate 30
Little Rock, AR 72203-2261

Phone Number (501) 569-2361
Fax Number (501) 569-2623
E-mail Address: Carl.Fuselier@arkansashighways.com

California CA

Barton J. Newton P.E. (Primary Member)
State Bridge Engineer
California Department of Transportation
Deputy Chief, Division of Engineering Services, Structure Policy and Innovation
1801 30th Street, MS 9-2/7J
P.O. Box 168041
Sacramento, CA 95816-8041

Phone Number (916) 227-8728
Fax Number (916) 227-8242
E-mail Address: barton_newton@dot.ca.gov

Susan Hida (Member)
Assistant State Bridge Engineer
California Department of Transportation
Division of Engineering Services, Structure Policy and Design
P.O. Box 168041, MS9-3/1H
Sacramento, CA 95816-8041

Phone Number (916) 227-8738
Fax Number (916) 227-9576
E-mail Address: susan_hida@dot.ca.gov

Michael Keever P.E. (Member)
Technical Specialist for Seismic Design and Research
California Department of Transportation
Division of Engineering Services, Structure, Policy and Innovation
1801 30th Street, MS 9-2/7J
P.O. Box 168041
Sacramento, CA 95816-8041

Phone Number (916) 227-8806
Fax Number (916) 227-8242
E-mail Address: Mike.Keever@dot.ca.gov

Colorado CO

Joshua Laipply (Primary Member)
State Bridge Engineer
Colorado Department of Transportation
4201 East Arkansas Avenue
Denver, CO 80222-3406

Phone Number (303) 757-9309
Fax Number (303) 757-9197
E-mail Address: Joshua.laipply@dot.state.co.us

Anthony DeVito (Member)
Region 1 Transportation Director
Colorado Department of Transportation
4201 East Arkansas Avenue
Denver, CO 80222-3406

Phone Number (303) 757-9459
Fax Number
E-mail Address: Anthony.devito@dot.state.co.us

Connecticut CT

Timothy D. Fields (Primary Member)
Transportation Supervising Engineer
Connecticut Department of Transportation
Bridge Consultant Design
P.O. Box 317546 / 2800 Berlin Turnpike
Newington, CT 06131-7546

Phone Number (860) 594-3217
Fax Number (860) 594-3373
E-mail Address: timothy.fields@ct.gov

Delaware DE

Barry Benton P.E. (Primary Member)
Assistant Director, Bridge
Delaware Department of Transportation
800 Bay Road, Route 113
P.O. Box 778
Dover, DE 19903-0778

Phone Number (302) 760-2311
Fax Number (302) 739-2217
E-mail Address: Barry.Benton@state.de.us

Jason Hastings P.E. (Member)
Bridge Design Resource Engineer
Delaware Department of Transportation
P.O. Box 778
Dover, DE 19903-0778

Phone Number (302) 760-2310
Fax Number (302) 739-2217
E-mail Address: Jason.Hastings@state.de.us

Douglass Robb P.E. (Member)
Bridge Design Engineer
Delaware Department of Transportation
800 Bay Road, Route 113
P.O. Box 778
Dover, DE 19903-0778

Phone Number (302) 760-2312
Fax Number (302) 739-2217
E-mail Address: Douglass.Robb@state.de.us

District of Columbia DC

Ronaldo T. "Nick" Nicholson P.E. (Primary Member)
Chief Engineer
District of Columbia Department of Transportation
55 M Street, S.E, 4th Floor.
Washington, DC 20003

Phone Number (202) 671-4691
Fax Number (202) 671-4710
E-mail Address: Ronaldo.Nicholson@dc.gov

L. Donald Cooney (Member)
Structural Engineer
District of Columbia Department of Transportation
Infrastructure Project Management Administration
55 M Street, S.E, 4th Floor.
Washington, DC 20003

Phone Number (202) 671-4681
Fax Number (202) 671-0655
E-mail Address: donald.cooney@dc.gov

Konjit "Connie" Eskender P.E. (Member)
Project Engineer
District of Columbia Department of Transportation
Infrastructure Project Management Administration
55 M Street, S.E, 4th Floor.
Washington, DC 20003

Phone Number (202) 671-4568
Fax Number (202) 741-5269
E-mail Address: konjit.eskender@dc.gov

Florida FL

Sam Fallaha P.E. (Primary Member)
Assistant State Structures Design Engineer
Florida Department of Transportation
Structures Design Office
605 Suwannee Street, MS 33
Tallahassee, FL 32399-0450

Phone Number (850) 414-4296
Fax Number (850) 414-4955
E-mail Address: sam.fallaha@dot.state.fl.us

Dennis Golabek (Member)
Assistant State Structures Design Engineer
Florida Department of Transportation
Structures Design Office
605 Suwannee Street, MS 33
Tallahassee, FL 32399-0450

Phone Number (850) 414-4296
Fax Number (850) 414-4955
E-mail Address: Dennis.Golabek@dot.state.fl.us

Jeffrey A. Pouliotte P.E., C.P.M. (Member)
State Structures Maintenance Engineer
Florida Department of Transportation
Office of Maintenance
605 Suwannee Street, MS 52
Tallahassee, FL 32399-0450

Phone Number (850) 410-5691
Fax Number (850) 410-5511
E-mail Address: jeffrey.pouliotte@dot.state.fl.us

Georgia GA

Paul V. Liles Jr., P.E. (Primary Member)
Assistant Director of Engineering
Georgia Department of Transportation
One Georgia Center, 24th Floor
600 West Peachtree Street, NW
Atlanta, GA 30308-3607

Phone Number (404) 631-1882
Fax Number (404) 631-1954
E-mail Address: pliles@dot.ga.gov

Bill DuVall, P.E. (Member)
Assistant State Bridge Engineer
Georgia Department of Transportation
One Georgia Center, 24th Floor
600 West Peachtree Street, NW
Atlanta, GA 30308-3607

Phone Number (404) 631-1883
Fax Number (404) 631-1954
E-mail Address: bduvall@dot.ga.gov

Ben Rabun., P.E. (Member)
State Bridge Engineer
Georgia Department of Transportation
One Georgia Center, 24th Floor
600 West Peachtree Street, NW
Atlanta, GA 30308-3607

Phone Number (404) 631-1882
Fax Number (404) 631-1954
E-mail Address: brabun@dot.ga.gov

Hawaii HI

Paul T. Santo (Primary Member)
Bridge Design Engineer
Hawaii Department of Transportation
601 Kamokila Boulevard, Suite 611
Kapolei, HI 96707-2037

Phone Number (808) 692-7611
Fax Number (808) 692-7617
E-mail Address: paul.santo@hawaii.gov

Idaho ID

Matthew M. Farrar (Primary Member)
State Bridge Engineer
Idaho Transportation Department
P.O. Box 7129
Boise, ID 83707-1129

Phone Number (208) 334-8538
Fax Number (208) 334-8256
E-mail Address: matt.farrar@itd.idaho.gov

Illinois IL

D. Carl Puzey (Primary Member)
Acting Chief of Bridges and Structures
Illinois Department of Transportation
Room 240
2300 S. Dirksen Parkway
Springfield, IL 62764-0002

Phone Number (217) 782-2124
Fax Number (217) 782-7540
E-mail Address: Carl.Puzey@illinois.gov

Tim Armbrecht P.E., S.E. (Member)
Acting Engineer of Structural Services
Illinois Department of Transportation
Room 222
2300 S. Dirksen Parkway
Springfield, IL 62764-0002

Phone Number (217) 782-6266
Fax Number (217) 782-7540
E-mail Address: Tim.Armbrecht@illinois.gov

Indiana IN

Anne M. Rearick (Primary Member)
Manager, Structural Services
Indiana Department of Transportation
Division of Production Management
100 N. Senate Avenue, Room N 642
Indianapolis, IN 46204-2273

Phone Number (317) 232-5152
Fax Number (317) 233-4929
E-mail Address: arearick@indot.in.gov

Iowa IA

Norman L. McDonald P.E. (Primary Member)
Director, Office of Bridges and Structures
Iowa Department of Transportation
800 Lincoln Way
Ames, IA 50010-6915

Phone Number (515) 239-1206
Fax Number (515) 239-1978
E-mail Address: norman.mcdonald@dot.iowa.gov

Kansas KS

Loren R. Risch P.E. (Primary Member)
Engineering Manager, State Bridge Office
Kansas Department of Transportation
Eisenhower State Office Building
700 SW Harrison
Topeka, KS 66603-3754

Phone Number (785) 296-3761
Fax Number (785) 752-8687
E-mail Address: loren@ksdot.org

James J. Brennan (Member)
Assistant Geotechnical Engineer
Kansas Department of Transportation
Materials and Research Center
2300 Van Buren Street
Topeka, KS 66611-1195

Phone Number (785) 291-3858
Fax Number (785) 296-2526
E-mail Address: brennan@ksdot.org

Kentucky KY

Mark Hite P.E. (Primary Member)
Director, Division of Structural Design
Kentucky Transportation Cabinet
200 Mero Street
Frankfort, KY 40622

Phone Number (502) 564-4560
Fax Number (502) 564-2581
E-mail Address: mark.hite@ky.gov

Marvin Wolfe P.E. (Member)
Division of Structural Design
Kentucky Transportation Cabinet
3rd Floor East
200 Mero Street
Frankfort, KY 40622

Phone Number (502) 564-4560
Fax Number (502) 564-2581
E-mail Address: Marvin.Wolfe@ky.gov

Louisiana LA

Paul Fossier, P.E. (Primary Member)
State Bridge Design Engineer
Louisiana Department of Transportation and Development
P.O. Box 94245
Baton Rouge, LA 70804-9245

Phone Number (225) 379-1302
Fax Number (225) 379-1786
E-mail Address: Paul.Fossier@la.gov

Arthur D'Andrea, P.E. (Member)
Assistant Bridge Engineer
Louisiana Department of Transportation and Development
P.O. Box 94245
Baton Rouge, LA 70804-9245

Phone Number (225) 379-1319
Fax Number (225) 379-1786
E-mail Address: Arthur.D'Andrea@la.gov

Zhengzheng “Jenny” Fu, P.E. (Member)
Assistant Bridge Design Administrator
Louisiana Department of Transportation and Development
P.O. Box 94245
Baton Rouge, LA 70804-9245

Phone Number (225) 379-1321
Fax Number (225) 379-1786
E-mail Address: zhengzheng.fu@la.gov

Maine ME

Wayne Frankhauser, Jr. P.E. (Primary Member)
Manager, Bridge Program
Maine Department of Transportation
Transportation Building
16 State House Station
Augusta, ME 04333-0016

Phone Number (207) 624-349
Fax Number (207) 624-3491
E-mail Address: Wayne.Frankhauser.Jr@maine.gov

Michael Wight (Member)
Senior Structural Designer
Maine Department of Transportation
Transportation Building
16 State House Station
Augusta, ME 04333-0016

Phone Number (207) 624-3435
Fax Number (207) 624-3491
E-mail Address: Michael.wight@maine.gov

Maryland MD

Earle S. Freedman (Primary Member)
Director, Office of Structures
Maryland Department of Transportation
State Highway Administration
707 N Calvert St MS C-203
Baltimore, MD 21202-3601

Phone Number (410) 545-8060
Fax Number (410) 209-5002
E-mail Address: efreedman@sha.state.md.us

Jeffrey L. Robert (Member)
Project Engineer—Office of Structures
Maryland Department of Transportation
State Highway Administration
707 N Calvert St MS C-203
Baltimore, MD 21202-3601

Phone Number (410) 545-8327
Fax Number (410) 209-5002
E-mail Address: jrobert@sha.state.md.us

Greg Roby (Member)
Office of Structures
Maryland Department of Transportation
State Highway Administration
707 N Calvert St MS C-203
Baltimore, MD 21202-3601

Phone Number (410) 545-8327
Fax Number (410) 209-5002
E-mail Address: groby@sha.state.md.us

Massachusetts MA

Alexander K. Bardow P.E. (Primary Member)
Director of Bridges and Structures
Massachusetts Department of Transportation
10 Park Plaza, Suite 6430
Boston, MA 02116-3973

Phone Number (617) 973-7571
Fax Number (617) 973-7554
E-mail Address: alexander.bardow@state.ma.us

Walter P. Heller P.E. (Member)
Acting District Highway Director, District 6
Massachusetts Department of Transportation
185 Kneeland Street, 9th Floor
Boston, MA 02111

Phone Number (617) 377-7151
Fax Number
E-mail Address: Walter.Heller@dot.state.ma.us

Michigan MI

David Juntunen (Primary Member)
Bridge Development Engineer
Michigan Department of Transportation
P.O. Box 30050
Lansing, MI 48909-7550

Phone Number (517) 335-2993
Fax Number (517) 335-2731
E-mail Address: JuntunenD@michigan.gov

Matthew Chynoweth (Member)
Bridge Field Services Engineer
Michigan Department of Transportation
P.O. Box 30050
Lansing, MI 48909-7550

Phone Number (517) 322-3322
Fax Number (517) 243-4302
E-mail Address: chynowethm@michigan.gov

Minnesota MN

Nancy Daubenberger (Primary Member)
Director, Office of Bridges
Minnesota Department of Transportation
3485 Hadley Avenue North
Oakdale, MN 55128-3307

Phone Number (651) 366-4504
Fax Number (651) 366-4497
E-mail Address: Nancy.Daubenberger@state.mn.us

Kevin Western (Member)
Bridge Design Engineer
Minnesota Department of Transportation
Office of Bridges and Structures
3485 Hadley Avenue North
Oakdale, MN 55128-3307

Phone Number (651) 366-4501
Fax Number (651) 747-2108
E-mail Address: kevin.western@state.mn.us

Arielle Ehrilich, PE (Member)
Acting State Bridge Design Engineer
Minnesota Department of Transportation
Office of Bridges and Structures
3485 Hadley Avenue North
Oakdale, MN 55128-3307

Phone Number (651) 366-4501
Fax Number (651) 747-2108
E-mail Address: Arielle.ehrlich@state.mn.us

Mississippi MS

Nick J. Altobelli P.E. (Primary Member)
Director of Structures
Mississippi Department of Transportation
P.O. Box 1850
Jackson, MS 39215-1850

Phone Number (601) 359-7200
Fax Number (601) 359-7070
E-mail Address: naltobelli@mdot.state.ms.us

Justin Walker P.E. (Member)
Deputy Director of Structures
Mississippi Department of Transportation
P.O. Box 1850
Jackson, MS 39215-1850

Phone Number (601) 359-7200
Fax Number (601) 359-7070
E-mail Address: jwalker@mdot.state.ms.us

Austin Banks. (Member)
Bridge Rating Engineer
Mississippi Department of Transportation
P.O. Box 1850
Jackson, MS 39215-1850

Phone Number (601) 359-7200
Fax Number (601) 359-7070
E-mail Address: abanks@mdot.state.ms.us

Missouri MO

Dennis Heckman (Primary Member)
State Bridge Engineer
Missouri Department of Transportation
PO Box 270
Jefferson City, MO 65102-0270

Phone Number (573) 751-4676
Fax Number (573) 526-5488
E-mail Address: dennis.heckman@modot.mo.gov

Scott B. Stotlemeyer (Member)
Assistant State Bridge Engineer
Missouri Department of Transportation
P.O. Box 270
Jefferson City, MO 65102-0270

Phone Number (573) 522-8752
Fax Number (573) 526-488
E-mail Address: Scott.Stotlemeyer@modot.mo.gov

Montana MT

Kent M. Barnes (Primary Member)
Bridge Design Engineer
Montana Department of Transportation
P.O. Box 201001
Helena, MT 59620-1001

Phone Number (406) 444-6260
Fax Number (406) 444-6155
E-mail Address: kbarnes@mt.gov

Nebraska NE

Mark J. Traynowicz (Primary Member)
Engineer, Bridge Division
Nebraska Department of Roads
P.O. Box 94759
Lincoln, NE 68509-4759

Phone Number (402) 479-4701
Fax Number (402) 479-4325
E-mail Address: Mark.Traynowicz@nebraska.gov

Mark Ahlman (Member)
Assistant Bridge Engineer—Design
Nebraska Department of Roads
P.O. Box 94759
Lincoln, NE 68509-4759

Phone Number (402) 479-3936
Fax Number (402) 479-3752
E-mail Address: Mark.Ahlman@nebraska.gov

Fouad Jaber (Member)
Assistant State Bridge Engineer
Nebraska Department of Roads
Bridge Division
P.O. Box 94759
Lincoln, NE 68509-4759

Phone Number (402) 479-3967
Fax Number (402) 479-3752
E-mail Address: Fouad.Jaber@nebraska.gov

Nevada NV

Mark P. Elicegui P.E. (Primary Member)
Chief Bridge Engineer
Nevada Department of Transportation
1263 South Stewart Street
Carson City, NV 89712-0002

Phone Number (775) 888-7542
Fax Number (775) 888-7405
E-mail Address: melicegui@dot.state.nv.us

Todd Stefonowicz (Member)
Assistant Chief Bridge Engineer
Nevada Department of Transportation
1263 South Stewart Street
Carson City, NV 89712-0002

Phone Number (775) 888-7550
Fax Number (775) 888-7405
E-mail Address: tstefonowicz@dot.state.nv.us

New Hampshire NH

Mark W. Richardson (Primary Member)
Administrator, Bureau of Bridge Design
New Hampshire Department of Transportation
P.O. Box 483
Concord, NH 03302-0483

Phone Number (603) 271-2731
Fax Number (603) 271-2759
E-mail Address: mrichardson@dot.state.nh.us

David L. Scott (Member)
In-House Design Chief, Bridge Design Bureau
New Hampshire Department of Transportation
P.O. Box 483
Concord, NH 03302-0483

Phone Number (603) 271-2731
Fax Number (603) 271-2759
E-mail Address: dscott@dot.state.nh.us

New Jersey NJ

Eli "Dave" Lambert III, P.E. (Primary Member)
State Highway Engineer
New Jersey Department of Transportation
P.O. Box 600
Trenton, NJ 08625-0600

Phone Number (609) 530-4235
Fax Number (609) 530-5557

E-mail Address: Dave.Lambert@dot.state.nj.us

New Mexico NM

Ray M. Trujillo P.E. (Primary Member)
State Bridge Engineer
New Mexico Department of Transportation
Bridge Bureau
Room 214
P.O. Box 1149
Santa Fe, NM 87504-1149

Phone Number (505) 827-5448
Fax Number (505) 827-5345

E-mail Address: raymond.trujillo@state.nm.us

Jeff C. Vigil P.E. (Member)
Bridge Management Engineer
New Mexico Department of Transportation
Bridge Bureau
Room 215A
P.O. Box 1149
Santa Fe, NM 87504-1149

Phone Number (505) 827-5457
Fax Number (505) 827-5339

E-mail Address: Jeff.Vigil@state.nm.us

New York NY

Richard Marchione P.E. (Primary Member)
Director, Office of Structures
New York State Department of Transportation
50 Wolf Road
Albany, NY 12232-2633

Phone Number (518) 457-4453
Fax Number (518) 485-7826

E-mail Address: rmarchione@dot.state.ny.us

Wahid Albert P.E. (Member)
Director, Structures Design Bureau
New York State Department of Transportation
Office of Structures, POD 4-3
50 Wolf Road
Albany, NY 12232-2633

Phone Number (518) 457-4453
Fax Number (518) 485-7826

E-mail Address: walbert@dot.state.ny.us

Donald F. Dwyer (Member)
Associate Soils Engineer
New York State Department of Transportation
50 Wolf Road, Mail Pod 3-1
Albany, NY 12232-2633

Phone Number (518) 457-4724
Fax Number (518) 457-0282

E-mail Address: ddwyer@dot.state.ny.us

North Carolina NC

Greg R. Perfetti (Primary Member)
State Structures Engineer
North Carolina Department of Transportation
1581 Mail Service Center
Raleigh, NC 27699-1581

Phone Number (919) 707-6400
Fax Number (919) 250-4082

E-mail Address: gperfetti@ncdot.gov

North Dakota ND

Terrence R. Udland (Primary Member)
Bridge Engineer
North Dakota Department of Transportation
608 E. Boulevard Avenue
Bismarck, ND 58505-0700

Phone Number (701) 328-1969
Fax Number (701) 328-0103
E-mail Address: tudland@nd.gov

Ohio OH

Timothy J. Keller P.E. (Primary Member)
Administrator, Office of Structural Engineering
Ohio Department of Transportation
1980 West Broad Street, 3rd Floor
Columbus, OH 43223-1102

Phone Number (614) 466-2463
Fax Number (614) 752-4824
E-mail Address: tim.keller@dot.state.oh.us

Jawdat Siddiqi (Member)
Assistant Administrator, Office of Structural Engineering
Ohio Department of Transportation
1980 West Broad Street
Columbus, OH 43223-1102

Phone Number (614) 728-2057
Fax Number (614) 752-4824
E-mail Address: jawdat.siddiqi@dot.state.oh.us

Oklahoma OK

Robert J. Rusch (Primary Member)
Division Engineer, Bridge Division
Oklahoma Department of Transportation
200 N.E. 21st Street
Oklahoma City, OK 73105-3299

Phone Number (405) 521-2606
Fax Number (405) 522-0134
E-mail Address: brusch@odot.org

Walter Peters P.E. (Member)
Assistant Bridge Engineer—Operations
Oklahoma Department of Transportation
200 N.E. 21st Street
Oklahoma City, OK 73105-3299

Phone Number (405) 521-2606
Fax Number (405) 522-0134
E-mail Address: wpeters@odot.org

John A. Schmiedel P.E. (Member)
Acting Assistant Bridge Engineer—Design
Oklahoma Department of Transportation
200 N.E. 21st Street
Oklahoma City, OK 73105-3299

Phone Number (405) 521-6488
Fax Number (405) 522-0134
E-mail Address: jschmiedel@odot.org

Oregon OR

Bruce V. Johnson P.E. (Vice Chair)
State Bridge Engineer
Oregon Department of Transportation
4040 Fairview Industrial Drive, SE
Salem, OR 97302-1142

Phone Number (503) 986-3864
Fax Number (503) 986-3407
E-mail Address: bruce.v.johnson@odot.state.or.us

Hormoz Seradj (Member)
Steel Standards Engineer
Oregon Department of Transportation
4040 Fairview Industrial Drive, SE
Salem, OR 97302-1142

Phone Number (503) 986-3346
Fax Number (503) 986-3407
E-mail Address: hormoz.seradj@odot.state.or.us

Pennsylvania PA

Thomas P. Macioce P.E. (Primary Member)
Bridge Engineer
Pennsylvania Department of Transportation
P.O. Box 2966
Harrisburg, PA 17105-2966

Phone Number (717) 787-2881
Fax Number (717) 346-0346
E-mail Address: tmacioce@pa.gov

Louis J. Ruzzi, P.E. (Member)
Engineering District 11 - Bridges
Pennsylvania Department of Transportation
Bridge Quality Assurance Division
45 Thomas Run Road
Bridgeville, PA 15017

Phone Number (412) 429-4893
Fax Number (412) 429-5085
E-mail Address: lruzzi@pa.gov

Puerto Rico PR

Vacant (Member)

Puerto Rico Department of Transportation and Public Works
Minillas Station
P.O. Box 41269
San Juan, PR 00940-1269

Phone Number
Fax Number
E-mail Address:

Rhode Island RI

David Fish P.E. (Primary Member)
Managing Engineer, Bridge Design
Rhode Island Department of Transportation
State Office Building
2 Capitol Hill
Providence, RI 02903-1124

Phone Number (401) 222-2053
Fax Number (401) 222-1271
E-mail Address: dfish@dot.ri.gov

South Carolina SC

Barry W. Bowers (Primary Member)
Structural Design Support Engineer
South Carolina Department of Transportation
P.O. Box 191
Columbia, SC 29202-0191

Phone Number (803) 737-4814
Fax Number (803) 737-0608
E-mail Address: bowersbw@scdot.org

Jeff Sizemore (Member)
Geotechnical Design Support Engineer
South Carolina Department of Transportation
P.O. Box 191
Columbia, SC 29202-0191

Phone Number (803) 737-1571
Fax Number (803) 737-0608
E-mail Address: sizemoreJC@dot.state.sc.us

South Dakota SD

Kevin Goeden P.E. (Primary Member)
Chief Bridge Engineer
South Dakota Department of Transportation
Becker Hanson Building
700 East Broadway Avenue
Pierre, SD 57501-2586

Phone Number (605) 773-3285
Fax Number (605) 773-2614
E-mail Address: kevin.goeden@state.sd.us

Tennessee

TN

Wayne J. Seger P.E. (Primary Member)
Director, Structures Division
Tennessee Department of Transportation
Suite 1100
700 James K. Polk Building Fifth and Deaderick
Nashville, TN 37243-0339

Phone Number (615) 741-3351
Fax Number (615) 532-7745
E-mail Address: Wayne.Seger@tn.gov

Henry Pate (Member)
Structural Design Office
Tennessee Department of Transportation
James K. Polk Building, Suite 1100
700 James K. Polk Building Fifth and Deaderick
Nashville, TN 37243-0339

Phone Number (615) 741-8295
Fax Number (615) 532-7745
E-mail Address: Henry.Pate@tn.gov

Texas

TX

Gregg A. Freeby P.E. (Primary Member)
Director, Bridge
Texas Department of Transportation
Dewitt C. Greer State Highway Building
125 East 11th Street
Austin, TX 78701-2483

Phone Number (512) 416-2183
Fax Number (512) 416-3144
E-mail Address: Gregg.Freeby@txdot.gov

Keith L. Ramsey P.E. (Member)
Field Operations Section Director, Bridge Division
Texas Department of Transportation
Dewitt C. Greer State Highway Building
125 East 11th Street
Austin, TX 78701-2483

Phone Number (512) 416-2250
Fax Number (512) 416-3144
E-mail Address: Keith.Ramsey@txdot.gov

Utah

UT

Carmen Swanwick (Primary Member)
Chief Structural Engineer
Utah Department of Transportation
P.O Box 148470
Salt Lake City, UT 84119-5998

Phone Number (801) 965-4981
Fax Number (801) 965-4187
E-mail Address: Cswanwick@utah.gov

Joshua Sletten (Member)
Bridge Management Engineer
Utah Department of Transportation
P.O. Box 148470
Salt Lake City, UT 84119-5998

Phone Number (801) 965-4879
Fax Number (801) 965-4187
E-mail Address: jsletten@utah.gov

Vermont

VT

Wayne B. Symonds (Primary Member)
Structures Design Engineer
Vermont Agency of Transportation
National Life Building, Drawer 33
Montpelier, VT 05633-5001

Phone Number (802) 828-0503
Fax Number
E-mail Address: Wayne.Symonds@state.vt.us

Virginia VA

Kendal "Ken" Walus P.E. (Primary Member)
State Structure and Bridge Engineer
Virginia Department of Transportation
1401 East Broad Street
Richmond, VA 23219-2052

Phone Number (804) 786-4575
Fax Number (804) 786-2988
E-mail Address: kendal.walus@vdot.virginia.gov

Prasad L. Nallapaneni P.E. (Member)
Senior Structural Engineer
Virginia Department of Transportation
1401 East Broad Street
Richmond, VA 23219-2052

Phone Number (804) 371-2770
Fax Number (804) 786-2988
E-mail Address: prasad.nallapaneni@vdot.virginia.gov

Washington WA

Tony M. Allen (Member)
Geotechnical Engineer
Washington State Department of Transportation
State Materials Laboratory
P.O. Box 47365
Olympia, WA 98504-7365

Phone Number (360) 709-5450
Fax Number (360) 709-5585
E-mail Address: allent@wsdot.wa.gov

Bijan Khaleghi (Member)
Bridge Engineer, Concrete Specialist
Washington State Department of Transportation
P.O. Box 47340
Olympia, WA 98504-7340

Phone Number (360) 705-7181
Fax Number (360) 705-6814
E-mail Address: khalegb@wsdot.wa.gov

West Virginia WV

Gregory Bailey (Primary Member)
Director, Engineering Division
West Virginia Department of Transportation
Building 5, Room A317
1900 Kanawha Boulevard East
Charleston, WV 25305-0440

Phone Number (304) 558-9722
Fax Number (304) 558-0605
E-mail Address: Gregory.L.Bailey@wv.gov

Wisconsin WI

Scot Becker (Primary Member)
Acting Director, Bureau of Structures
Wisconsin Department of Transportation
P.O. Box 7916
Madison, WI 53707-7916

Phone Number (608) 266-0075
Fax Number (608) 266-5166
E-mail Address: scot.becker@dot.wi.gov

Beth A. Cannestra (Member)
Director, Bureau of Project Development
Wisconsin Department of Transportation
P.O. Box 7916
Madison, WI 53707-7916

Phone Number (608) 266-0075
Fax Number (608) 266-5166
E-mail Address: beth.cannestra@dot.wi.gov

William C. Dreher P.E. (Member)
Chief Structures Design Engineer
Wisconsin Department of Transportation
P.O. Box 7916
Madison, WI 53707-7916

Phone Number (608) 266-8489
Fax Number
E-mail Address: William.Dreher@dot.wi.gov

Keith R. Fulton P.E. (Primary Member)
State Bridge Engineer
Wyoming Department of Transportation
5300 Bishop Boulevard
Cheyenne, WY 82009-3340

Phone Number (307) 777-4427
Fax Number (307) 777-4279
E-mail Address: Keith.Fulton@wyo.gov

Paul G. Cortez P.E. (Member)
Bridge Operations Engineer
Wyoming Department of Transportation
5300 Bishop Boulevard
Cheyenne, WY 82009-3340

Phone Number (307) 777-4442
Fax Number (307) 777-4279
E-mail Address: Paul.Cortez@wyo.gov

Michael E. Menghini P.E. (Member)
Assistant State Bridge Engineer
Wyoming Department of Transportation
5300 Bishop Boulevard
Cheyenne, WY 82009-3340

Phone Number (307) 777-4427
Fax Number (307) 777-4279
E-mail Address: Michael.Menghini@wyo.gov

U.S. DOT Member

FHWA — DC

M. Myint Lwin (Secretary)
Director, Office of Bridge Technology
Federal Highway Administration
Room E75-318
1200 New Jersey Avenue, S.E.
Washington, DC 20590

Phone Number (202) 366-4589
Fax Number (202) 366-3077
E-mail Address: myint.lwin@dot.gov

Raj Ailaney (Member)
Senior Bridge Engineer
Federal Highway Administration
Office of Bridge Technology,
1200 New Jersey Avenue, S.E.
Washington, DC 20590

Phone Number (202) 366-6749
Fax Number (202) 366-3077
E-mail Address: Rajkumar.Ailaney@dot.gov

AASHTO

AASHTO — DC

Kelley Rehm (Liaison)
Program Manager – Bridges and Structures
American Association of State Highway and Transportation Officials
602 Idlewood Drive
Mount Juliet, DC 37122

Phone Number (859) 433-9623
Fax Number (866) 301-1322
E-mail Address: krehm@aashto.org

Associate Member—Bridge, Port, and Toll

Golden Gate Bridge — CA

Kary H. Witt (Member)
Deputy General Manager, Bridge Division
Golden Gate Bridge, Highway, and Transportation District
Box 9000, Presidio Station
San Francisco, CA 94129-0601

Phone Number (415) 923-2240
Fax Number (415) 923-2011
E-mail Address: kwitt@goldengate.org

MDTA — MD

Dan Williams P.E. (Member)
Bridge Engineering Manager
Maryland Transportation Authority
300 Authority Drive
Dundalk, MD 21222

Phone Number (410) 537-7824
Fax Number
E-mail Address: dwilliams1@mdta.state.md.us

N.J. Turnpike Authority — NJ

Richard J. Raczynski P.E. (Member)
Chief Engineer
New Jersey Turnpike Authority
P.O. Box 5042
Woodbridge, NJ 07095-5042

Phone Number (732) 442-8600
Fax Number (732) 293-1200
E-mail Address: raczynski@turnpike.state.nj.us

N.Y. State Bridge Authority — NY

William J. Moreau P.E. (Member)
Chief Engineer
New York State Bridge Authority
Mid Hudson Bridge Plaza
P.O. Box 1010
Highland, NY 12528-8010

Phone Number (845) 691-4077
Fax Number (845) 691-7914
E-mail Address: bmoreau@nysba.state.ny.us

Penn. Turnpike Authority PA

James L. Stump P.E. (Member)
Bridge Engineer Manager
Pennsylvania Turnpike Commission
P.O. Box 67676
Harrisburg, PA 17106-7676

Phone Number (717) 939-9551
Fax Number (717) 986-9645
E-mail Address: jstump@paturnpike.com

Associate Member—Federal

U.S. Army Corps of Engineers MN

Phillip W. Sauser P.E. (Member)

U.S. Army Corps of Engineers
Department of the Army
CEMVP-EC-D 190 5th Street East, Suite 401
St. Paul, MN 55101-1636

Phone Number (651) 290-5722
Fax Number
E-mail Address: phillip.w.sauser@usace.army.mil

Christopher H. Westbrook P.E. (Member)
Structural Engineer, Bridge Safety Program Manager
U.S. Army Corps of Engineers
Department of the Army
Attn: CECW-CE
441 G Street, N.W.
Washington, MN 20314

Phone Number (202) 761-7584
Fax Number (202) 761-1960
E-mail Address: Christopher.H.Westbrook@usace.army.mil

USDA Forest Service UT

Tom Gillins P.E., S.E. (Member)
Structural Engineer
USDA Forest Service
324 25th Street
Ogden, UT 84401

Phone Number (801) 625-5236
Fax Number (303) 625-5229
E-mail Address: tgillins@fs.fed.us

Associate Member—International

Alberta AB

Lloyd Atkin P.E. (Member)
Director, Bridge Engineering and Water Management
Alberta Transportation
2nd Floor, Twin Atria Building
4999 - 98th Avenue
Edminton, AB T6B 2X3

Phone Number (780) 415-1080
Fax Number (780) 422-5426
E-mail Address: Lloyd.Atkin@gov.ab.ca

Korea

Eui-Joon Lee (Member)
Director of Construction Planning Team
Korea Expressway Corporation
Sujeong-gu, Seongnam-si
430, DaewangPangyoro (239-1, Geumto-dong)
Gyeonggi-do, 461-703

Phone Number (000) 000-0000
Fax Number
E-mail Address: lejlej@ex.co.kr

Sang-Soon Lee (Member)
Senior Manager of Structure Management Team
Korea Expressway Corporation
Sujeong-gu, Seongnam-si
430, DaewangPangyoro (239-1, Geumto-dong)
Gyeonggi-do, 461-703

Phone Number (000) 000-0000
Fax Number
E-mail Address: lssp@ex.co.kr

Ontario ON

VACANT

Ontario Ministry of Transportation
Garden City Tower, 2nd Floor
301 St. Paul Street
St. Catherines, ON L2R 7R4

Phone Number (905) 704-2341
Fax Number (905) 704-2060
E-mail Address:

Saskatchewan SK

Howard Yea (Member)
Director, Bridge Services
Saskatchewan Ministry of Highways and Infrastructure
1630 Park, 2nd Floor
Regina, SK S4P 3V7

Phone Number (306) 787-4830
Fax Number (306) 787-9777
E-mail Address: hyea@highways.gov.sk.ca

Other

Newfoundland NF

Peter Lester (Friend)
Chief Bridge Engineer
Newfoundland Dept. of Works, Services and Transportation
P.O. Box 8700
St. Johns, NF A1B 4J6

Phone Number (709) 729-3990
Fax Number (709) 729-0283
E-mail Address:

TRB DC

Waseem Dekelbab
Senior Program Officer
Transportation Research Board
TRB Mail Room
500 Fifth Street, N.W.
Washington, DC 20001

Phone Number (202) 334-1409
Fax Number (202) 334-2006
E-mail Address: wdekelbab@nas.edu

U.S. Coast Guard DC

Kamal Elnahal (Member)
Senior Structural Engineer
United States Coast Guard
2100 2nd Street, S.W
Washington, DC 20166-6511

Phone Number (202) 372-1511
Fax Number
E-mail Address: Kamal.A.Elnahal@uscg.mil

AASHTO Highway Subcommittee on Bridges and Structures
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	Member	Member Dept.	Region
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	Albert, Wahid	New York	Region I
	D'Andrea, Arthur	Louisiana	Region II
	Liles, Paul V.	Georgia	Region II
	Walus, Kendal "Ken"	Virginia	Region II
	Chynoweth, Matthew	Michigan	Region III
	Heckman, Dennis	Missouri	Region III
	Duwadi, Sheila	FHWA	Ex Officio
	Ernst, Steve	FHWA	Ex Officio
	Witt, Kary H.	Golden Gate Bridge	Ex Officio

T-2 Technical Committee for Bearings and Expansion Devices

	Member	Member Dept.	Region
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	Hite, Mark	Kentucky	Region II
	Perfetti, Greg R.	North Carolina	Region II
	Bowers, Barry W.	South Carolina	Region II
	Puzey, D. Carl	Illinois	Region III
	Stotlemeyer, Scott	Missouri	Region III
	Traynowicz, Mark J.	Nebraska	Region IV
	Seradj, Hormoz	Oregon	Region IV
	Sidhom, Samir	FHWA	Ex Officio

T-3 Technical Committee for Seismic Design

	Member	Member Dept.	Region
Chair	Pratt, Richard A.	Alaska	Region IV
Vice Chair	VACANT		
	Bardow, Alexander K.	Massachusetts	Region I
	Fish, David	Rhode Island	Region I
	Fuselier, Carl	Arkansas	Region II
	Liles, Paul V.	Georgia	Region II
	Bowers, Barry W.	South Carolina	Region II
	Puzey, D. Carl	Illinois	Region III
	Rearick, Anne M.	Indiana	Region III
	Heckman, Dennis	Missouri	Region III
	Keever, Michael	California	Region IV
	Barnes, Kent M.	Montana	Region IV
	Elicegui, Mark P.	Nevada	Region IV
	Johnson, Bruce V.	Oregon	Region IV
	Swanwick, Carmen	Utah	Region IV
	Manceaux, Derrell	FHWA	Ex Officio
	Yen, Phillip	FHWA	Ex Officio

T-4 Technical Committee for Construction

	Member	Member Dept.	Region
Chair	VACANT		
Vice Chair	Liles, Paul V.	Georgia	Region II
	Fields, Timothy D.	Connecticut	Region I
	Robb, Douglass	Delaware	Region I
	Sherlock, David B.	Maine	Region I
	Lambert, Eli "Dave"	New Jersey	Region I
	Symonds, Wayne B.	Vermont	Region I
	Seger, Wayne J.	Tennessee	Region II
	Chynoweth, Matthew	Michigan	Region III
	Dreher, William	Wisconsin	Region III
	Santo, Paul T.	Hawaii	Region IV
	Goeden, Kevin	South Dakota	Region IV
	Swanwick, Carmen	Utah	Region IV
	Ailaney, Raj	FHWA	Ex Officio
	Beerman, Benjamin	FHWA	Ex Officio
	Elnahal, Kamal	U.S.Coast Guard	Ex Officio

T-5 Technical Committee for Loads and Load Distribution

	Member	Member Dept.	Region
Chair	Hida, Susan	California	Region IV
Vice Chair	Bailey, Gregory	West Virginia	Region II
	Benton, Barry	Delaware	Region I
	Folsom, Jeffrey S.	Maine	Region I
	Robert, Jeffrey L.	Maryland	Region I
	Lambert, Eli "Dave"	New Jersey	Region I
	Macioce, Thomas P.	Pennsylvania	Region I
	Golabek, Dennis	Florida	Region II
	DuVall, Bill	Georgia	Region II
	Western, Kevin	Minnesota	Region III
	Ehrlich, Arielle	Minnesota	Region III
	Ahlman, Mark	Nebraska	Region III
	Becker, Scot	Wisconsin	Region III
	Dreher, William	Wisconsin	Region IIImail
	Elicegui, Mark P.	Nevada	Region IV
	Schmiedel, John A.	Oklahoma	Region IV
	Gao, Lubin	FHWA	Ex Officio
	Tharmabala, Bala	Ontario	Ex Officio

T-6 Technical Committee for Fiber Reinforced Polymer Composites

	Member	Member Dept.	Region
Chair	Liles, Paul V.	Georgia	Region II
Vice Chair	VACANT		
	Robert, Jeffrey L.	Maryland	Region I
	Albert, Wahid	New York	Region I
	Frankhauser, Wayne	Maine	Region I
	Fallaha, Sam	Florida	Region II
	Chynoweth, Matthew	Michigan	Region III
	Keller, Timothy J.	Ohio	Region III
	Keever, Michael	California	Region IV
	Triandafilou, Lou	FHWA	Ex Officio

T-7 Technical Committee for Guardrail and Bridge Rail

	Member	Member Dept.	Region
Chair	Keller, Timothy J.	Ohio	Region III
Vice Chair	Black, John F. "Buddy"	Alabama	Region II
	Robert, Jeffrey L.	Maryland	Region I
	Bardow, Alexander K.	Massachusetts	Region I
	Fossier, Paul	Louisiana	Region II
	Bowers, Barry W.	South Carolina	Region II
	Rearick, Anne M.	Indiana	Region III
	Barton, John	Texas	Region IV
	Goeden, Kevin	South Dakota	Region IV
	Fulton, Keith R.	Wyoming	Region IV
	Wong, Waider	FHWA	Ex Officio

T-8 Technical Committee for Moveable Bridges

	Member	Member Dept.	Region
Chair	Fossier, Paul	Louisiana	Region II
Vice Chair	Nallapaneni, Prasad L.	Virginia	Region II
	Richardson, Mark W.	New Hampshire	Region I
	Roby, Greg	Maryland	Region I
	Fallaha, Sam	Florida	Region II
	Juntunen, David	Michigan	Region III
	Dreher, William C.	Wisconsin	Region III
	Dubin, Earl	FHWA	Ex Officio
	Elnahal, Kamal	U.S.Coast Guard	Ex Officio

T-9 Technical Committee for Bridge Preservation

	Member	Member Dept.	Region
Chair	Johnson, Bruce V.	Oregon	Region IV
Vice Chair	Pouliotte, Jeffrey A.	Florida	Region II
	Frankhauser, Wayne	Maine	Region I
	Marchione, Richard	New York	Region I
	Colquett, William "Tim"	Alabama	Region II
	Rabun, Ben	Georgia	Region II
	Walus, Kendal "Ken"	Virginia	Region II
	McDonald, Norman L.	Iowa	Region III
	Nehme, Jean A.	Arizona	Region IV
	Newton, Barton J.	California	Region IV
	Vigil, Jeff C.	New Mexico	Region IV
	Rusch, Robert J.	Oklahoma	Region IV
	Ahmad, Anwar S.	FHWA	Ex Officio
	Virmani, Paul Y.	FHWA	Ex Officio
	Williams, Dan	MDTA	Ex-Officio

T-10 Technical Committee for Concrete Design

	Member	Member Dept.	Region
Chair	Risch, Loren R.	Kansas	Region III
Vice Chair	Farrar, Matthew M.	Idaho	Region IV
	Richardson, Mark W.	New Hampshire	Region I
	Macioce, Thomas P.	Pennsylvania	Region I
	Fallaha, Sam	Florida	Region II
	Fu, Jenny	Louisiana	Region II
	Seger, Wayne J.	Tennessee	Region II
	Daubenberger, Nancy	Minnesota	Region III
	Dreher, William	Wisconsin	Region III
	Hida, Susan	California	Region IV
	Jaber, Fouad	Nebraska	Region IV
	Johnson, Bruce V.	Oregon	Region IV
	Freeby, Gregg A.	Texas	Region IV
	Khaleghi, Bijan	Washington	Region IV
	Hartmann, Joseph L.	FHWA	Ex Officio
	Holt, Reggie	FHWA	Ex Officio

T-11 Technical Committee for Research

	Member	Member Dept.	Region
Chair	Walus, Kendal "Ken"	Virginia	Region II
Vice Chair	Daubenberger, Nancy	Minnesota	Region III
	Fallaha, Sam	Florida	Region II
	Perfetti, Greg R.	North Carolina	Region II
	Juntunen, David	Michigan	Region III
	Nehme, Jean A.	Arizona	Region IV
	Keever, Michael	California	Region IV
	Udland, Terrence R.	North Dakota	Region IV
	Rusch, Robert J.	Oklahoma	Region IV
	Khaleghi, Bijan	Washington	Region IV
	Friedland, Ian M.	FHWA	Ex Officio
	Sauser, Phillip W.	U.S. Army Corps of Engineers	Ex Officio

T-12 Technical Committee for Structural Supports for Signs, Luminaires and Traffic Signals

	Member	Member Dept.	Region
Chair	McDonald, Norman L.	Iowa	Region III
Vice Chair	VACANT		
	Benton, Barry	Delaware	Region I
	Fuselier, Carl	Arkansas	Region II
	Golabek, Dennis	Florida	Region II
	DuVall, Bill	Georgia	Region II
	Risch, Loren R.	Kansas	Region III
	Santo, Paul T.	Hawaii	Region IV
	Stefonowicz, Todd	Nevada	Region IV
	Menghini, Michael E.	Wyoming	Region IV
	Soden, Derek	FHWA	Ex Officio

T-13 Technical Committee for Culverts

	Member	Member Dept.	Region
Chair	Bailey, Gregory	West Virginia	Region II
Vice Chair	Western, Kevin	Minnesota	Region III
	Hastings, Jason	Delaware	Region I
	Macioce, Thomas P.	Pennsylvania	Region I
	Hite, Mark	Kentucky	Region II
	Wolfe, Marvin	Kentucky	Region II
	Brennan, James J.	Kansas	Region III
	Trujillo, Ray M.	New Mexico	Region IV
	Anderson, Scott	FHWA	Ex Officio

T-14 Technical Committee for Structural Steel Design

	Member	Member Dept.	Region
Chair	Perfetti, Greg R.	North Carolina	Region II
Vice Chair	McDonald, Norman L.	Iowa	Region III
	Eskender, Konjit "Connie"	District of Columbia	Region I
	Marchione, Richard	New York	Region I
	Macioce, Thomas P.	Pennsylvania	Region I
	Golabek, Dennis	Florida	Region II
	Pate, Henry	Tennessee	Region II
	Puzey, D. Carl	Illinois	Region III
	Farrar, Matthew M.	Idaho	Region IV
	Seradj, Hormoz	Oregon	Region IV
	Fulton, Keith R.	Wyoming	Region IV
	Kozy, Brian	FHWA	Ex Officio
	Nguyen, Khoa	FHWA	Ex Officio

T-15 Technical Committee for Substructures and Retaining Walls

	Member	Member Dept.	Region
Chair	Siddiqi, Jawdat	Ohio	Region III
Vice Chair	Allen, Tony M.	Washington	Region IV
	Nicholson, Ronaldo T. "Nick"	District of Columbia	Region I
	Scott, David L.	New Hampshire	Region I
	Fish, David	Rhode Island	Region I
	Black, John F. "Buddy"	Alabama	Region II
	Sizemore, Jeff	South Carolina	Region II
	Brennan, James J.	Kansas	Region III
	Pratt, Richard A.	Alaska	Region IV
	Nichols, Silas	FHWA	Ex Officio

T-16 Technical Committee for Timber Structures

	Member	Member Dept.	Region
Chair	Macioce, Thomas P.	Pennsylvania	Region I
Vice Chair	Bardow, Alexander K.	Massachusetts	Region I
	Symonds, Wayne B.	Vermont	Region I
	Black, John F. "Buddy"	Alabama	Region II
	Bailey, Gregory	West Virginia	Region II
	Pratt, Richard A.	Alaska	Region IV
	Duwadi, Sheila	FHWA	Ex Officio
	Gillins, Tom	USDA Forest Service	Ex Officio

T-17 Technical Committee for Welding

	Member	Member Dept.	Region
Chair	Bardow, Alexander K.	Massachusetts	Region I
Vice Chair	Liles, Paul V.	Georgia	Region II
	Symonds, Wayne B.	Vermont	Region I
	Walker, Justin	Mississippi	Region II
	Bailey, Gregory	West Virginia	Region II
	Hartmann, Joseph L.	FHWA	Ex Officio
	Sauser, Phillip W.	U.S. Army Corps of Engineers	Ex Officio

T-18 Technical Committee for Bridge Management, Evaluation and Rehabilitation

	Member	Member Dept.	Region
Chair	Farrar, Matthew M.	Idaho	Region IV
Vice Chair	Newton, Barton J.	California	Region IV
	Roby, Greg	Maryland	Region I
	Marchione, Richard	New York	Region I
	Macioce, Thomas P.	Pennsylvania	Region I
	Christie, Eric J.	Alabama	Region II
	Pouliotte, Jeffrey A.	Florida	Region II
	Rabun, Ben	Georgia	Region II
	D'Andrea, Arthur	Louisiana	Region II
	Banks, Austin	Mississippi	Region II
	Armbrecht, Tim	Illinois	Region III
	Juntunen, David	Michigan	Region III
	Barnes, Kent M.	Montana	Region IV
	Vigil, Jeff C.	New Mexico	Region IV
	Ramsey, Keith L.	Texas	Region IV
	Cortez, Paul G.	Wyoming	Region IV
	Everett, Thomas D.	FHWA	Ex Officio

T-19 Technical Committee for Software and Technology

	Member	Member Dept.	Region
Chair	Becker, Scot	Wisconsin	Region III
Vice Chair	Nehme, Jean A.	Arizona	Region IV
	Hastings, Jason	Delaware	Region I
	Christie, Eric J.	Alabama	Region II
	Hite, Mark	Kentucky	Region II
	Altobelli, Nick J.	Mississippi	Region II
	Trujillo, Ray M.	New Mexico	Region IV
	Rusch, Robert J.	Oklahoma	Region IV
	Sletten, Joshua	Utah	Region IV
	Saad, Thomas K.	FHWA	Ex Officio

T-20 Technical Committee for Tunnels

	Member	Member Dept.	Region
Chair	Ruzzi, Lou	Pennsylvania	Region I
Vice Chair	Nallapaneni, Prasad L.	Virginia	Region II
	Nicholson, Ronaldo T. "Nick"	District of Columbia	Region I
	Heller, Walter P.	Massachusetts	Region I
	Bardow, Alexander K.	Massachusetts	Region I
	Dwyer, Donald F.	New York	Region I
	Johnson, Bruce	Oregon	Region IV
	Khaleghi, Bijan	Washington	Region IV
	Rohena, Jesus	FHWA	Ex Officio
	Williams, Dan	MDTA	Ex-Officio

FHWA OFFICERS AND EX-OFFICIO TECHNICAL COMMITTEE LIAISONS

OFFICERS

M. Myint Lwin (Secretary) Director, Office of Bridge Technology Federal Highway Administration 1200 New Jersey Avenue, S.E. Room E75-318 Washington, DC 20590	Phone Fax Email	(202) 366-4589 (202) 366-3077 myint.lwin@dot.gov
Raj Ailaney (Assistant Secretary) Senior Bridge Engineer – Planning & Contracts Federal Highway Administration Office of Bridge Technology, HIBT-1 1200 New Jersey Avenue, S.E., Room E75-314 Washington, DC 20590	Phone Fax Email	(202) 366-6749 (202) 366-3077 Raj.Ailaney@dot.gov

TECHNICAL COMMITTEE LIAISONS

T-1, Bridge and Tunnel Security

Steve Ernst Senior Bridge Engineer - Safety and Security Federal Highway Administration Office of Bridge Technology, HIBT-1 1200 New Jersey Avenue, S.E. Washington, DC 20590	Phone Fax Email	(202) 366-4619 (202) 366-3077 steve.ernst@dot.gov
Sheila Rimal Duwadi Team Leader Hazards Mitigation Federal Highway Administration, HRDI-50 Turner-Fairbank Highway Research Center Office of Infrastructure R&D 6300 Georgetown Pike McLean, VA 22101	Phone Fax Email	(202) 493-3106 (202) 493-3442 sheila.duwadi@dot.gov

T-2, Bearings and Expansion Devices

Samir Sidhom Bridge Design Team Leader Federal Lands Bridge Office 12300 W Dakota Ave, Suite 210 Lakewood, CO 80228	Phone Fax Email	(720) 963-3399 (720) 963-3379 samir.sidhom@dot.gov

T-3, Seismic Design

Derrell Manceaux Senior Structural Engineer Federal Highway Administration FHWA Resource Center 12300 W Dakota Ave, Suite #340 Lakewood, CO 80228	Phone Fax Email	(720) 963-3205 (720) 963-3232 Derrell.Manceaux@dot.gov

FHWA OFFICERS AND EX-OFFICIO TECHNICAL COMMITTEE LIAISONS

Phillip Yen Principal Bridge Engineer Federal Highway Administration Office of Bridge Technology, HIBT-1 1200 New Jersey Avenue, S.E, Washington, DC 20590	Phone Fax Email	(202) 366-5604 (202) 366-3077 Wen-huei.yen@.dot.gov

T-4, Construction

Raj Ailaney Senior Bridge Engineer – Planning and Contracts Federal Highway Administration Office of Bridge Technology, HIBT-1 1200 New Jersey Avenue, S.E, Washington, DC 20590	Phone Fax Email	(202) 366-6749 (202) 366-3077 raj.ailaney@dot.gov
Benjamin Beerman Structural Engineer Federal Highway Administration – Resource Center 61 Forsyth St. SW, 17T26 Atlanta, Georgia, 30303	Phone Fax Email	(404) 562- 3930 Benjamin.Beerman@dot.gov

T-5, Loads and Load Distribution

Lubin Gao Load Rating Engineer Federal Highway Administration Office of Bridge Technology, HIBT-10 1200 New Jersey Avenue, S.E, Washington, DC 20590	Phone Fax Email	(202) 366-4604 (708) 283-3077 Lubin.gao@dot.gov

T-6, Fiber Reinforced Polymer Composites

Lou Triandafilou Team Leader Bridge and Foundation Engineering Federal Highway Administration, HRDI-40 Turner-Fairbank Highway Research Center Office of Infrastructure R&D 6300 Georgetown Pike McLean, VA 22101	Phone Fax Email	((202) 493-3059 ((202) 493-3442 lou.triandafilou@dot.gov

T-7, Guardrail and Bridge Rail

Waider Wong Structural Design Engineer 10 South Howard Street Suite 4000 Baltimore, MD 21201	Phone Fax Email	(410) 962-9252 (410) 962-3655 waider.wong@dot.gov

T-8, Movable Bridges

Earl Dubin Structural Engineer Federal Highway Administration Leo W. O'Brien Federal Building Room 719, Clinton Avenue & N. Pearl Street Albany, NY 12207	Phone Fax Email	(518) 431-4125x229 (518) 431-4121 earl.dubin@dot.gov

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T-9, Bridge Preservation

Paul Virmani Research Chemist Federal Highway Administration, HRDI-60 Turner-Fairbank Highway Research Center Office of Infrastructure R&D 6300 Georgetown Pike McLean, VA 22101	Phone Fax Email	(202) 493-3052 (202) 493-3442 paul.virmani@dot.gov
Anwar Ahmad Bridge Preservation Engineer Federal Highway Administration Office of Bridge Technology, HIBT-30 1200 New Jersey Avenue, S.E, Washington, DC 20590	Phone Fax Email	(202) 366-8501 (202) 366-3077 Anwar.ahmad@dot.gov

T-10, Concrete Design

Reggie Holt Senior Bridge Engineer - Concrete Federal Highway Administration Office of Bridge Technology, HIBT-10 1200 New Jersey Avenue, S.E. Washington, DC 20590	Phone Fax Email	(202) 366-4596 (202) 366-3077 Reggie.Holt@dot.gov
Joseph L. Hartmann Team Leader, Bridge and Tunnel Team Federal Highway Administration Office of Bridge Technology, HIBT-10 1200 New Jersey Avenue, S.E. Washington, DC 20590	Phone Fax Email	(202) 366-4599 (202) 366-3077 joey.hartmann@dot.gov

T-11, Research

Ian M. Friedland Assistant Director, Bridge and Structures R&D Federal Highway Administration Turner-Fairbank Highway Research Center Room F-219, HRDI-1 6300 Georgetown Pike Mclean, VA 22101-2296	Phone Fax Email	(202) 493-3023 (202) 493-3161 ian.friedland@ dot.gov

T-12, Structural Supports for Signs, Luminaires and Traffic Signals

Derek Soden Florida Division 545 John Knox Road Suite 200 Tallahassee, FL 32308	Phone Fax Email	(850) 553-2245 (850) 942-8308 Derek.soden@dot.gov

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T-13, Culverts

Scott Anderson Geotechnical & Hydraulic Tech. Service Team Leader 12300 West Dakota Avenue, Suite 340 Lakewood, CO 80228	Phone Fax Email	(720) 963-3244 (720) 963-3232 scott.anderson@dot.gov

T-14, Structural Steel Design

Brian Kozy Senior Bridge Engineer - Steel Federal Highway Administration Office of Bridge Technology, HIBT-10 1200 New Jersey Avenue, S.E. Washington, DC 20590	Phone Fax Email	(202) 493-0341 (202) 366-3077 Brian.kozy@dot.gov
Khoa Nguyen Bridge Design Team Leader Federal Highway Administration Western Federal Lands Highway Division 610 E. 5th Street Vancouver, WA 98661	Phone Fax Email	(360) 619-7700 (360) 619-7846 Khoa.nguyen@dot.gov

T-15, Substructures and Retaining Walls

Silas Nichols Senior Bridge Engineer - Geotechnical Federal Highway Administration Office of Bridge Technology, HIBT-20 1200 New Jersey Avenue, S.E. Washington, DC 20590	Phone Fax Email	(202) 366-1554 (202) 366-3077 Silas.Nichols@dot.gov

T-16, Timber Structures

Sheila Rimal Duwadi Team Leader Hazard Mitigation Federal Highway Administration, HRDI-50 Turner-Fairbank Highway Research Center Office of Infrastructure R&D 6300 Georgetown Pike McLean, VA 22101	Phone Fax Email	(202) 493-3106 (202) 493-3442 sheila.duwadi@dot.gov

T-17, Welding

Joseph L. Hartmann Team Leader, Bridge and Tunnel Team Federal Highway Administration Office of Bridge Technology, HIBT-10 1200 New Jersey Avenue, S.E. Washington, DC 20590	Phone Fax Email	(202) 366-4599 (202) 366-3077 joey.hartmann@dot.gov

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T-18, Bridge Management, Evaluation, and Rehabilitation

Thomas Everett	Phone	(202) 366-4675
Principal Bridge Engineer	Fax	(202) 366-3077
Federal Highway Administration	Email	thomas.everett@dot.gov
Office of Bridge Technology, HIBT-30		
1200 New Jersey Avenue, S.E..		
Washington, DC 20590		

T-19, Software and Technology

Thomas Saad	Phone	(708) 283-3521
Structural Design Engineer	Fax	(708) 283-3501
Federal Highway Administration	Email	Thomas.Saad@dot.gov
4749 W Lincoln Highway (RTE 30) Suite 600		
Matteson, IL 60443		

T-20, Tunnels

Jesus Rohena	Phone	(202) 366-4593
Senior Bridge Engineer - Tunnels	Fax	(202) 366-3077
Federal Highway Administration	Email	jesus.rohena@dot.gov
Office of Bridge Technology, HIBT-10		
1200 New Jersey Avenue, S.E.		
Washington, DC 20590		