2019 Interim Revisions to the LRFD Specifications for Structural Supports for Highway Signs, Luminaires, and Traffic Signals

American Association of State Highway and Transportation Officials
AASHTO

First Edition 2015
Publication Code LRFDLTS-1-I3
ISBN 978-1-56051-719-1

AASHTO

American Association of State Highway and Transportation Officials
444 North Capitol Street, NW, Suite 249
Washington, DC 20001
202-624-5800 phone/202-624-5806 fax
www.transportation.org

ISBN: 978-1-56051-719-1

Pub Code: LRFDLTS-1-I3-OL

2019 INTERIM REVISIONS INSTRUCTIONS AND INFORMATION

General

AASHTO has issued proposed interim revisions to the *LRFD Structural Supports for Highway Signs, Luminaires, and Traffic Signals*, First Edition (2015). This packet contains the revised pages. They are designed to replace the corresponding pages in the book.

Affected Articles

Underlined text indicates revisions that were approved in 2018 by the AASHTO Committee on Bridges and Structures. ~~Strikethrough text~~ indicates any deletions that were likewise approved by the Committee. A list of affected articles is included below.

All interim pages are displayed on a yellow background to make the changes stand out when inserted in the first edition binder. They also have a page header displaying the interim publication year. Please note that these pages may also contain nontechnical (i.e., editorial) changes made by AASHTO publications staff; any changes of this type will not be marked in any way so as not to distract the reader from the technical changes.

2019 Changed Articles

SECTION 5: STEEL DESIGN
5.6.6.1
5.6.7

SECTION 11: FATIGUE DESIGN
11.9.2

THIS PAGE LEFT BLANK INTENTIONALLY

Stiffeners having a transition radius shall not be used.

Alternative stiffener geometries, stiffener spacing, and weld termination angles on the tube shall be approved by the Owner based upon evaluation, analysis, testing, or acceptable field performance, singly or in combination.

thin stiffeners can reduce distortion of the tube but fail to sufficiently reduce the stress at the fillet-weld and can cause fatigue cracking through the throat of the stiffener-to-transverse plate weld.

A ratio of stiffener thickness to tube thickness of 1.25 provides an optimum solution with equal likelihood of fatigue cracking at the stiffener termination and at the tube-to-transverse-plate weld.

Decreasing the ratio of the stiffener height to stiffener spacing reduces protection to the fillet-weld. An optimum solution is obtained when the stiffener height is about 1.6 times the stiffener spacing.

Reducing the termination angle of the stiffener on the tube wall improves the fatigue performance of stiffened connections. Using a stiffener termination angle of 15 degrees ensures that the stiffener sections are fully effective in sharing load.

Stiffeners with a transition radius at the termination on the tube wall are fabrication intensive and are expected to be costlier than a tapered alternative. To avoid exposure of the lack of fusion at the weld root in fillet welds and partial-penetration groove welds, a stiffener termination with a transition radius must be groove welded, which requires non-destructive inspection in the vicinity of weld termination. It is difficult to grind the weld toe without inadvertently thinning the tube at the transition.

The stiffened groove-welded tube-to-transverse-plate connection is unlikely to be cost-effective and is excluded from this specification.

Figures of stiffeners are illustrated in Table 11.9.3.1-1. (Detail 6.2 and 6.3)

5.6.5—Backing Ring

For full-penetration groove-welded tube-to-transverse-plate connections, the thickness of the backing ring shall not exceed $^1/_4$ in. The height of the backing ring, when welded to the tube at the top prior to performing ultrasonic inspection of the groove weld, shall be as given by Eq. 5.6.5-1, rounded to the nearest integer:

$$H = 2t(\tan\theta) + R \qquad (5.6.5\text{-}1)$$

where:

H = height of backing ring at a groove-welded tube-to-transverse-plate connection (in.),

R = root gap at a groove-welded tube-to-transverse-plate connection (in.),

θ = angle of the sound beam for ultrasonic inspection of groove welds (degrees), and

t = the tube wall thickness (in.).

C5.6.5

In full-penetration groove-welded tube-to-transverse-plate connections with the backing ring welded to the plate and the tube wall, fatigue cracking can occur both at the groove-weld toe and the backing ring top weld toe on the tube wall. Depending on the diameter and thickness of the tube, and the height and thickness of the backing ring, the backing ring can participate in transferring forces from the tube to the transverse plate and can introduce variability in the fatigue performance of the connection. Providing a 2 in. × $^1/_4$ in. backing ring limits this participation to a reasonable level in typical support structures.

However, when the backing ring is welded to the tube at the top, this weld interferes with the ultrasonic inspection of the groove weld by allowing the sound wave to travel from the outside of the shaft through the weld into the backing ring. The sound wave then gets trapped in the backing ring and does not reach the groove weld. For a successful inspection, the weld at the top of the backing ring should be above the centerline of the probe. According to AWS D1.1, the ultrasonic beam should bounce at least once to the area of inspection, For tube-to-transverse-plate connection employing an external collar, the tube thickness for the above equation shall include the thickness of the collar and the tube.

When the top weld of the backing ring is made after the ultrasonic inspection of the groove weld, or when the backing ring is not welded at the top, the height of the backing ring shall not exceed 2 in.

which creates a full "V" signal. From experience, a shallow beam angle such as 70 degrees produces the best results. Thus, for thicker tubes with a 45-degree bevel and a root gap, the probe placement gets higher and therefore the backing ring needs to be taller.

The backing ring heights for different tube thicknesses are tabulated in the following table for a root gap of $^{1}/_{4}$ in. and an angle of ultrasonic beam of 70°.

Table C5.6.5-1—Required Backing Ring Height

t, in.	H, in.
$t \leq 0.3125$	2
$0.3125 < t \leq 0.50$	3
$0.50 < t \leq 0.6875$	4

This requirement for backing ring height is not applicable if the top weld of the backing ring is made after the ultrasonic inspection of the groove weld or if the backing ring is not welded at the top. In such cases, a maximum 2-in.-high backing ring will be sufficient. Also refer to Section 14 for additional recommendations regarding welding the backing ring to the tube.

When welded to the tube, the backing ring provides a redundant load path when the tube-to-transverse-plate groove weld develops fatigue cracking.

5.6.6—Holes and Cutouts

5.6.6.1—Unreinforced and Reinforced Holes and Cutouts

Unreinforced and reinforced holes and cutouts shall be detailed as shown in Figure 5.6.6.1-1, Figure 5.6.6.1-2, and Figure 5.6.6.1-3. ~~The width of opening in the cross sectional plane of the tube shall not be greater than 40 percent of the tube diameter *D* at that section for structures that are designed according to Section 11. The corners of the opening shall be rounded to radius as shown. In the figures below, double-headed arrows indicate termination is beyond the view illustrated.~~

For structures that are designed according to Section 11, the width of opening in the cross-sectional plane of the tube shall be as shown in Table 5.6.6.1-1, where D is the tube diameter at the section.

- ~~55 percent of the tube diameter *D* at that section for *D* not exceeding 30.0 in.~~
- ~~40 percent of the tube diameter D at that section for *D* exceeding 30.0 in.~~

C5.6.6.1

In laboratory fatigue tests (Roy et al., 2011), fatigue cracking from unreinforced hand holes in sign/signal support structure specimens initiated from the edge of hand hole at the point of maximum stress concentration. The hand holes in the test specimens were located in the plane of the mast-arm but on the away face to produce the most critical stress condition in the hand hole detail for fatigue.

It is recommended that in sign/signal support structures the hand holes and other holes and cutouts be located in a region of low stress. Since the fatigue stress cycles in sign/signal support structures are imparted primarily due to wind-induced galloping oscillations in the plane containing the arm, it is recommended that the hand holes be located on the side at 90 degrees to that containing the cantilever arm (Figure C5.6.6.1-1). The hand hole may be located on either side.

Table 5.6.6.1-1–Maximum Opening for Unreinforced and Reinforced Cutouts

Structure Type	D	Maximum Opening
Sign/Signal Support Structures	All	$0.40 \times D$
Pole-Type High-Level Luminaire Support Structures	Up to 30 in.	$0.55 \times D$
	Greater than 30 in.	$0.40 \times D$

The corners of the opening shall be rounded to a radius as shown. In the following figures, double-headed arrows indicate termination is beyond the view illustrated.

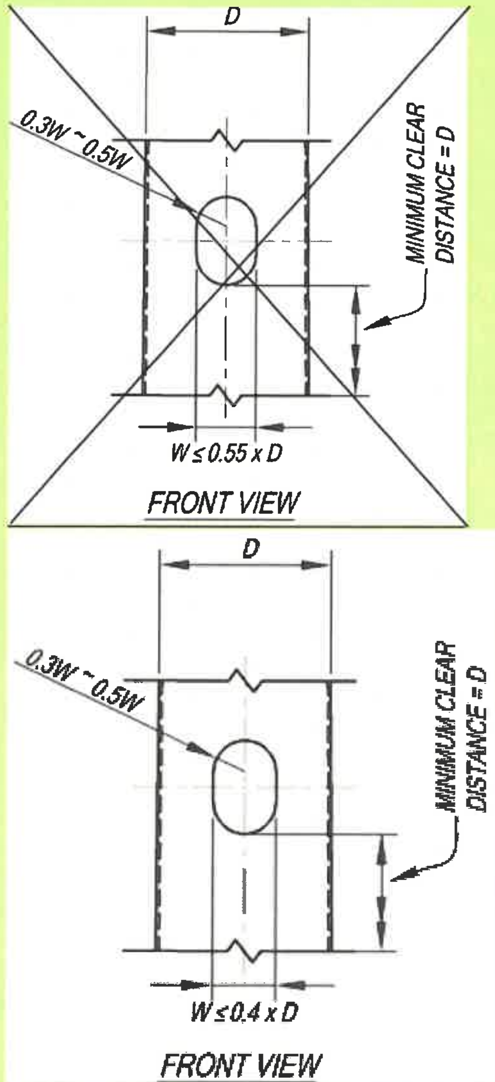


Figure 5.6.6.1-1—Details of Unreinforced Holes and Cutouts

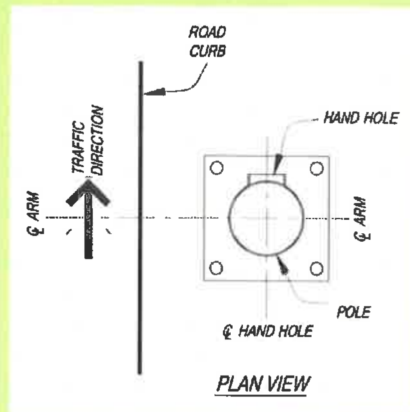
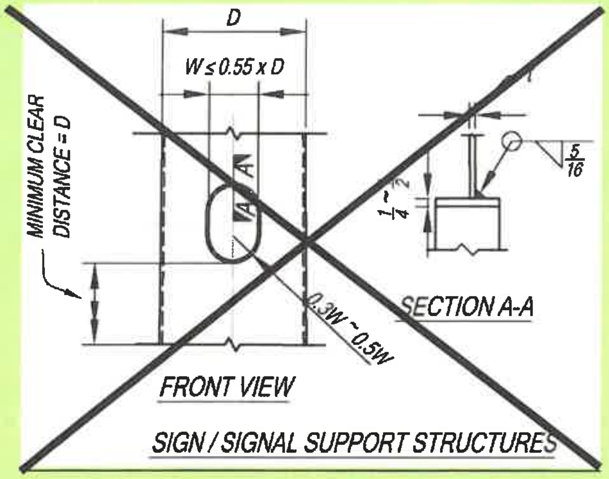


Figure C5.6.6.1-1—Recommended Orientation of a Hand Hole



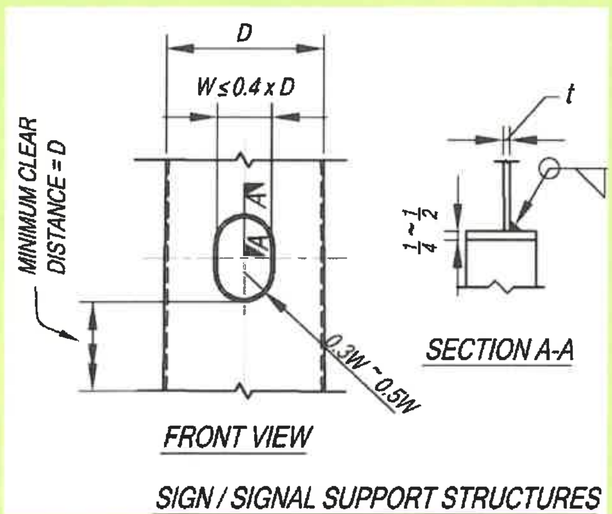
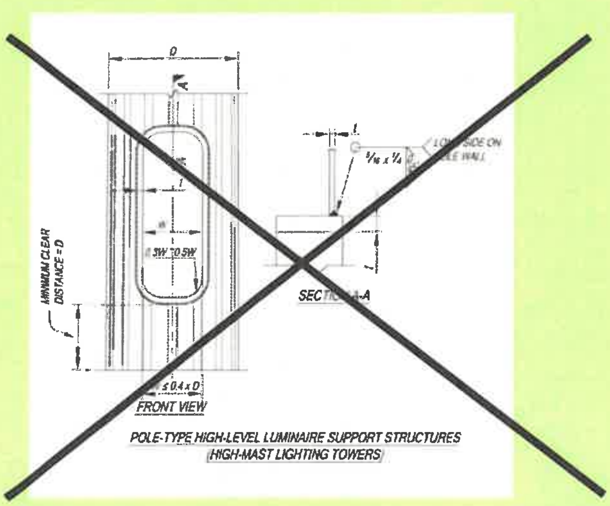


Figure 5.6.6.1-2—Details of Reinforced Holes and Cutouts



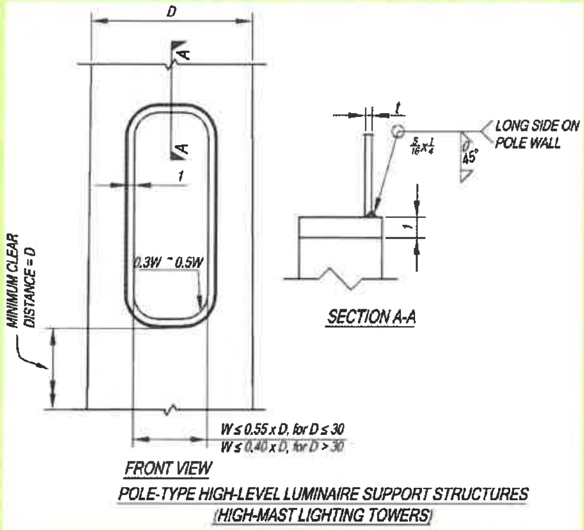


Figure 5.6.6.1-3—Details of Reinforced Holes and Cutouts for High-Mast Poles

In service, fatigue cracks at reinforced hand holes have been reported from the toe of the hand hole frame-to-pole (reinforcement-to-tube) weld in high level lighting support structures. In laboratory fatigue tests (Roy et al., 2011), fatigue cracking from hand hole details in sign/signal support structure specimens initiated only from the lack of fusion at the root of the hand hole frame-to-pole (reinforcement-to-tube) fillet-weld. Because of limited access, the hand hole frames in sign and signal structures can be welded only from the outside, increasing the possibility of lack of fusion defects at the weld root.

The hand holes in the test specimens were located in the plane of the mast-arm but on the away face such as to produce the most critical stress condition in the hand hole detail for fatigue. Since the fatigue stress cycles in sign/signal support structures are imparted primarily due to wind-induced galloping oscillations in the plane containing the arm, it is recommended that the hand holes be located on the side at 90° to that containing the arm. In high level lighting support structure specimens, the hand hole details did not develop any fatigue cracking (Roy et al., 2011).

Alternative geometries shall be approved by the Owner based upon evaluation, analysis, testing, or acceptable field performance, singly or in combination. Location of cutouts and appurtenances shall be approved by the Owner based on sound engineering practices.

5.6.7—Mast-Arm-to-Pole Connections

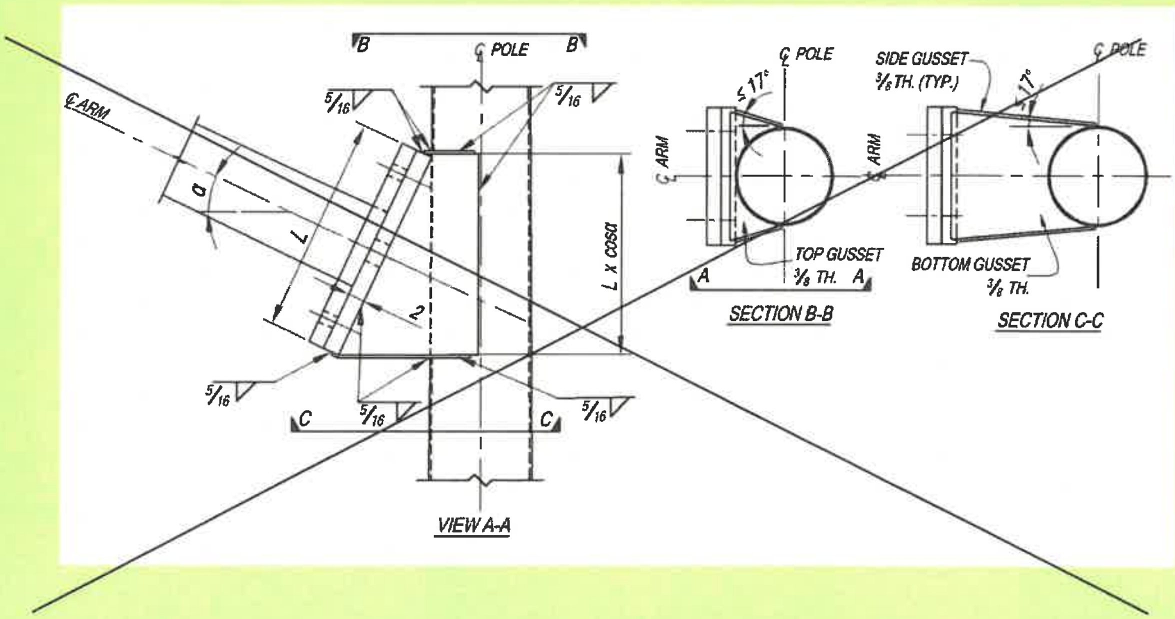
Mast-arm-to-pole connections employing fillet-welded gusseted box or ring-stiffened box have been shown to be effective and fatigue resistant. Connections validated with testing ~~may be used and~~ are encouraged. Fillet-welded gusseted box connections shall be limited to a *D/t* ratio of 50.

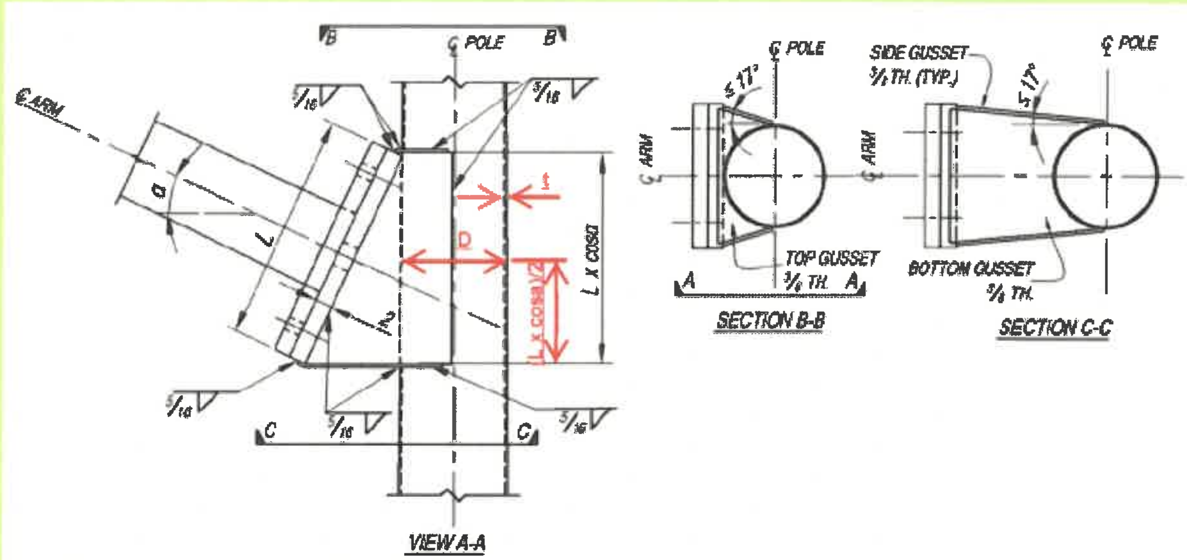
C5.6.7

Fillet-welded gusseted boxes or ring-stiffened boxes at the mast-arm-to-pole connections tested in the laboratory in full size specimens (Roy et al., 2011) did not develop any fatigue cracking under both in-plane and out-of-plane loading. These connections were tested at various load levels and in some specimens were subjected to in excess of 40 million stress cycles. In all specimens, fatigue cracking occurred in other critical details in the structure, such as the tube-to-transverse-plate welds in the mast-arm and/or the pole, and/or hand holes.

In-service fatigue cracking of ~~these connections~~ fillet-welded gusseted box connections on larger diameter poles has been reported. Fatigue testing has shown the advantage of ring stiffeners that completely encircle a pole relative to a built-up box connection. Fillet-welded gusseted box connections performed well in fatigue testing where *D/t* was limited to 50 or less (Roy et al., 2011). For tapered tubes, *D* shall be measured at mid-height of the connection. For built-up box connections, it is recommended that the width of the box be at least the same as the diameter of the column (i.e., the sides of the box are tangent to the sides of the column).

~~Ring-stiffened box connections are more fabrication intensive and should be employed in geographic regions where support structures are expected to experience significant wind induced oscillations. In other regions, gusseted box connections are expected to provide satisfactory performance.~~ See Figures C5.6.7-1 and C5.6.7-2 for details.





Fillet-welded gusseted box connections shall be limited to a D/t ratio of 50.

Figure C5.6.7-1—Details of Fillet-Welded Gusseted Box Connections

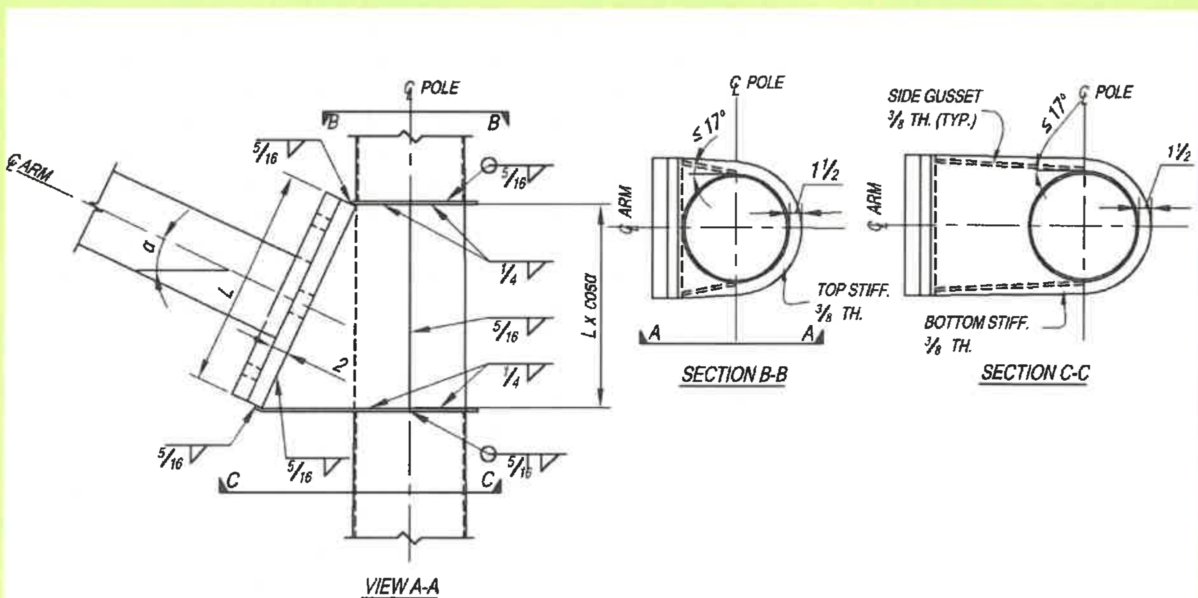


Figure C5.6.7-2—Details of Fillet-Welded Ring-Stiffened Box Connections

5.6.8—Slip Type Field Splice

The minimum length of any telescopic (i.e., slip type) field splices for all structures shall be 1.5 times the inside diameter of the exposed end of the female section.

C5.6.8

ASCE/SEI 48-11 (2011) for the design transmission poles provides a more rigorous approach that may be consulted for guidance.

THIS PAGE LEFT BLANK INTENTIONALLY

a complete set of all possible connection details; rather it is intended to include the most commonly used connection details in support structures. Any detail that is not listed in Table 11.9.3.1-1 may be classified based on alternate methodologies provided in Appendix C.

Appropriate details can improve the fatigue resistance of these structures, and can help in producing a cost-effective design by reducing the member size required for fatigue resistant details.

Stiffened and unstiffened tube-to-transverse plate connections, reinforced and unreinforced handholes, and anchor rods are the most fatigue critical details in the support structures. Most fatigue cracking in service and in laboratory tests under NCHRP Project 10-70 on full size specimens has occurred at these details. The details of specimens tested under NCHRP Project 10-70 are shown in Table C11.9.3.1-1.

11.9.2—Stress Range

Nominal stress range shall be used when fatigue design of connection details is carried out using Table 11.9.3.1-1 and shall be calculated at the site of potential fatigue cracking.

The detail categories in Table 11.9.3.1-1 were developed based on nominal stress to be calculated as discussed below:

- For unreinforced holes and cutouts in tubes, the nominal stress shall be calculated considering the net section property of the tube and magnified by a stress concentration factor ~~of 4.0, where the width of the opening is limited to 40 percent of the tube diameter.~~ as presented in Table 11.9.2-1, where *D* is the tube diameter at the mid-height of the opening.
- For reinforced holes and cutouts in tubes, the nominal stress for design against fatigue cracking at the toe of the reinforcement-to-tube weld shall be calculated considering the net section property of the tube and the reinforcement.
- For design against fatigue cracking from the root, the above nominal stress shall be magnified by a stress concentration factor ~~of 4.0, where the width of the opening is limited to 40 percentof the tube diam~~eter. as presented in Table 11.9.2-1, where *D* is the tube diameter at the mid-height of the opening.
- In full-penetration, groove-welded, tube-to-transverse plate connections, the nominal stress shall be calculated on the gross section of the tube at the groove-weld toe on the tube irrespective of a backing ring welded to the tube or not.
- For partial penetration, groove-welded, mast-arm-to-column pass-through connections, the nominal stress shall be calculated on the gross section of the column at the base of the connection.
- For fillet-welded tube-to-transverse plate connections (socket connections), nominal stress shall be calculated

C11.9.2

Nominal stress is a stress in a component that can be derived using simple strength of material calculations based on applied loading and nominal section properties. The nominal stress should be calculated considering gross geometric changes at the section, e.g., tapers, handholes, stiffeners, welded backing rings, etc., which locally magnify or decrease the nominal stress.

on the gross section of the tube at the fillet-weld toe on the tube.

- In stiffened tube-to-transverse plate connections, the nominal stress at the termination of the stiffener shall be calculated based on the gross section of the tube at a section through the toe of the wrap-around-weld on the tube.
- In stiffened tube-to-transverse plate connections, the nominal stress at the weld toe on the tube of the tube-to-transverse plate fillet-weld shall be calculated based on the gross section of only the tube at the section.
- In stiffened tube-to-transverse plate connections, the nominal stress at the stiffener-to-plate weld shall be calculated based on the gross section of the tube and the stiffeners at the section.

For computing nominal stress at the tube-to-transverse plate fillet-weld in a stiffened connection, only the gross section of the tube without the stiffeners should be considered. The fatigue resistance for these connections in Table 11.9.3.1-1 has been accordingly defined. The effect of the stiffeners is implicitly included in the computation of fatigue stress concentration factor in Eq. 11.9.3.1-4 in Table 11.9.3.1-2.

For computing nominal stress at the stiffener-to-transverse plate weld, the gross section including the tube and the stiffeners at the section should be considered.

Table 11.9.2-1–Stress Concentration Factors for Unreinforced and Reinforced Hand Holes

Structure Type	Clear Opening	Stress Concentration Factor
Sign/Signal Support Structures	Up to $0.40 \times D$	4.0
Pole-Type High-Level Luminaire Support Structures	Up to $0.45 \times D$	4.0
	Greater than $0.45 \times D$ and up to $0.55 \times D$	5.7

11.9.3—Fatigue Resistance

Support structures shall be proportioned such that the wind load induced stress is below the CAFT providing infinite life. For infinite life, nominal fatigue resistance shall be taken as:

$$\gamma(\Delta f)_n = \phi(\Delta F)_{TH} \tag{11.9.3-1}$$

The remaining fatigue life of existing steel structures may be assessed based on a finite life. For finite life, nominal fatigue resistance shall be taken as:

$$\phi(\Delta F)_n = \phi\left(\frac{A}{N}\right)^{\frac{1}{3}} \tag{11.9.3-2}$$

where

$(\Delta F)_n$ = the nominal fatigue resistance as specified in Table 11.9.3.1-1
$(\Delta F)_{TH}$ = the CAFT; A is the finite life constant
N = the number of wind load induced stress cycles expected during the life time of the structures.

The values of $(\Delta F)_{TH}$ and A for steel structure details are specified in Table 11.9.3.1-1. The values for γ are specified in Table 3.4-1, and the value for ϕ is 1.0.

Aluminum structures shall be designed to provide infinite life. The value of $(\Delta F)_{TH}$ of aluminum structure details shall be determined by dividing the respective threshold values of steel with 2.6.

Fatigue resistance of typical fatigue-sensitive connection details in support structures for finite and infinite life designs

C11.9.3

When the wind load induced maximum stress range (determined as static load effects per Article 11.7) experienced by a component or a detail is less than the CAFT, the component or detail can be assumed to have a theoretically infinite fatigue life. Using Eq. 11.9.3-1 to establish $(\Delta F)_n$ in Eq. 11.5.1-1 should ensure infinite life performance.

In the finite life regime at stress ranges above the CAFT, the fatigue life is inversely proportional to the cube of the stress range. For example, if the stress range is reduced by a factor of 2, the fatigue life increases by a factor of $2^3 = 8$ This result is reflected in Eq. 11.9.3-2. When assessing the finite life of an existing structure, the number of wind load induced stress cycles expected during the life time of the structure should be estimated from analysis based on historical wind records or directly by field measurements on similar structures, as decided by the owner.

The constant A and the constant amplitude fatigue threshold (ΔF)TH for the detail classes specified in Table 11.9.3.1-1 are consistent with steel detail categories in LRFD Design. Figure C11.9.3-1 is a graphical representation of the nominal fatigue resistance for detail categories as per LRFD Design.

shall be determined from Table 11.9.3.1-1. The fatigue stress concentration factors as functions of connection geometry in tubular structures shall be determined as given in Article 11.9.3.1. The potential location of cracking in each detail is identified in the table. "Longitudinal" implies that the direction of applied stress is parallel to the longitudinal axis of the detail, and "transverse" implies that the direction of applied stress is perpendicular to the longitudinal axis of the detail.

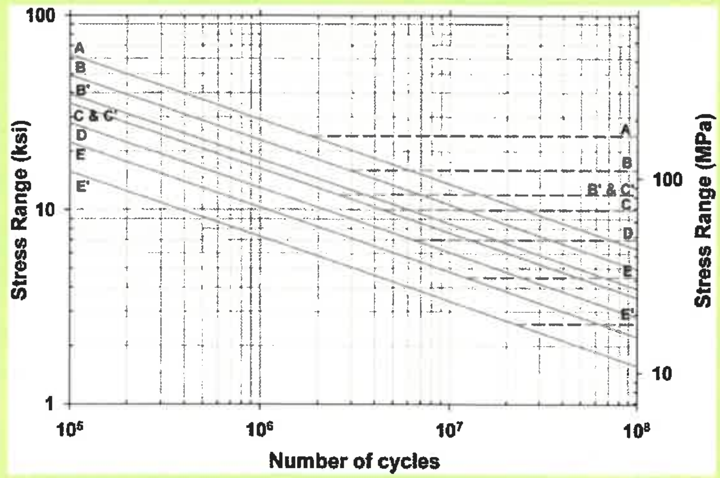


Figure C11.9.3-1—Stress Range vs. Number of Cycles

The fatigue resistance of support structures was established based on laboratory fatigue tests of full-scale cantilevered structures and substantiated by analytical studies. The resistance is based on elastic section analysis and nominal stresses on the cross-section. The resistance includes effects of residual stresses due to fabrication and anchor bolt pretension, which are not to be considered explicitly in the nominal stress computations.

Fatigue resistance of tube-to-transverse plate connections are classified in Table 11.9.3.1-1 in terms of separate fatigue stress concentration factors for finite and infinite life designs, which explicitly incorporate the effects of stress concentration due to the connection geometry and the weld toe notch. The effects of weld toe micro-discontinuities are implicitly considered in the experimental results for all connections. Research (Roy et al., 2011) shows that the infinite life fatigue resistance of connection details in support structures does not always correspond to their respective finite life detail categories in LRFD Design.

To assist designers, the details of full size support structure specimens that were tested in the laboratory under NCHRP Project 10-70 (Roy et al., 2011) are tabulated in Table C11.9.3.1-1 along with their fatigue resistance. Designers are encouraged to directly employ these details in service, wherever applicable, with nominal stress range calculated as per Article 11.9.2.

The fatigue resistance of handholes or cutouts is defined in terms of the magnified nominal stress as defined earlier.

Fatigue resistance of the fillet-welded T-, Y-, and K-tube-to-tube, angle-to-tube, and plate-to-tube connections was not established by laboratory testing. Fatigue resistance of these connections in Table 11.9.3.1-1 has been retained from the previous edition of the specification, which corresponds to the classification for cyclic punching shear stress in tubular members specified by the *AWS Structural Welding Code D1.1—Steel* based on research in the offshore industry on connections of thicker and larger diameter tubes. Stresses in tubular connections are strongly dependent on their geometric parameters and therefore, extrapolation of the fatigue design provisions from the AWS specification may not be consistent with the performance of the pass-through connections in service. Until further research can provide a better estimate of the fatigue resistance of these connections, they should be classified as indicated in Table 11.9.3.1-1.

Stool-type stiffened fillet-welded tube-to-transverse plate connections, similar to those in service in Iowa, were tested in the laboratory (Roy et al., 2011), but on thinner tubes (see Table C11.9.3.1-1). These stiffened connections employ a pair of rectangular vertical stiffeners welded to the tube wall and transverse plate and connected by a plate at the top. The top plate serves as an anchorage for the anchor rods, and is not welded to the tube. These connection details have performed extremely well in Iowa, where no cracking were observed during 40 years of service. In laboratory tests, however, these connections did not perform well. This detail may provide better fatigue performance in thicker and larger diameter tubes as was used for the structures in service. Until further research can provide a better estimate of the fatigue resistance of these stiffened connections, the fatigue performance of the welds terminating at the end of vertical stiffeners in the stool type stiffened tube-to-end plate connections should be classified as indicated in Table 11.9.3.1-1.

11.9.3.1—Stress Concentration Factors

For finite life evaluation of tubular connections, fatigue stress concentration factors in Table 11.9.3.1-1 shall be calculated as per equations given in Table 11.9.3.1-2.

For infinite life design of tubular connections, the fatigue stress concentration factor in Table 11.9.3.1-1 shall be calculated as:

$$K_I = \left[\left(1.76 + 1.83t_T\right) - 4.76 \times 0.22^{K_F}\right] K_F \qquad (11.9.3.1\text{-}1)$$

where K_F is calculated from Table 11.9.3.1-2 for the respective details.

The parameters used in the expressions for stress concentration factors are:

D_{BC} = diameter of circle through the fasteners in the transverse plate (for connections with two or more fastener circles, use the outer most circle diameter) (in.)
D_{OP} = diameter of concentric opening in the transverse plate (in.)
D_T = external diameter of a round tube or outer flat-to-flat distance of a multisided tube at top of transverse plate (in.)
h_{ST} = height of longitudinal attachment (stiffener) (in.)
N_B = number of fasteners in the transverse plate
N_s = number of sides

C11.9.3.1

Fatigue resistance of tubular connections in support structures depends on the relative stiffness of the components at a connection or the connection geometry. Geometric stresses arise from the need to maintain compatibility between the tubes and other components at the connections. This geometric stress concentration affects the fatigue resistance of the connections for both finite and infinite life performance. In addition, the resistance of the connections against any fatigue crack growth for infinite life is also affected by the local stress concentration related to local geometry of the weld. The effects of global and local geometric stress concentrations on the fatigue resistance of various connections in the support structures were determined experimentally and analytically under NCHRP Project 10-70 (Roy et al., 2011).

Traffic arm-to-pole connections often contain more than one bolt circle having two rows of 3 or 4 connecting bolts, as shown in the Figure C11.9.3.1-1. Table C11.9.3.1-1 provides the $\underline{K_F}$ equation for the tube-to-transverse plate connections that contain the bolt circle variable, D_{BC}. The bolt circle chosen influences the CAFT value. Finite element analysis shows that the internal bolts have little influence on the fatigue stresses in the tube. Therefore, the outer most bolt circle should be used in the $\underline{K_F}$ equation from Table C11.9.3.1-1, which will result in the more conservative CAFT value.