- For all other cases, including slab-type bridges (excluding top slabs of box culverts) where the span exceeds 15.0 ft, all of the load specified in Article 3.6.1.2 shall be applied.

Where the refined methods are used to analyze decks, force effects shall be determined on the following basis:

- Where the slab spans primarily in the transverse direction, only the axles of the design truck of Article 3.6.1.2.2 or design tandem of Article 3.6.1.2.3 shall be applied to the deck slab.
- Where the slab spans primarily in the longitudinal direction (including slab-type bridges), all of the loads specified in Article 3.6.1.2 shall be applied.

Wheel loads shall be assumed to be equal within an axle unit, and amplification of the wheel loads due to centrifugal and braking forces need not be considered for the design of decks.

It is theoretically possible that an extreme force effect could result from a 32.0-kip axle in one lane and a 50.0-kip tandem in a second lane, but such sophistication is not warranted in practical design.

3.6.1.3.4—Deck Overhang Load

For the design of deck overhangs with a cantilever, not exceeding 6.0 ft from the centerline of the exterior girder to the face of a structurally continuous concrete railing, the outside row of wheel loads may be replaced with a uniformly distributed line load of 1.0 klf intensity, located 1.0 ft from the face of the railing.

Horizontal loads on the overhang resulting from vehicle collision with barriers shall be in accordance with the provisions of Section 13.

C3.6.1.3.4

Structurally continuous barriers have been observed to be effective in distributing wheel loads in the overhang. Implicit in this provision is the assumption that the 25.0-kip half weight of a design tandem is distributed over a longitudinal length of 25.0 ft, and that there is a cross beam or other appropriate component at the end of the bridge supporting the barrier which is designed for the half tandem weight. This provision does not apply if the barrier is not structurally continuous.

3.6.1.4—Fatigue Load

3.6.1.4.1—Magnitude and Configuration

1 The fatigue load shall be one design truck or axles thereof specified in Article 3.6.1.2.2, but with a constant spacing of 30.0 ft between the 32.0-kip axles.

The dynamic load allowance specified in Article 3.6.2 shall be applied to the fatigue load. → 0.15

For the design of orthotropic decks and wearing surfaces on orthotropic decks, the loading pattern as shown in Figure 3.6.1.4.1-1 shall be used.

C3.6.1.4.1

For orthotropic steel decks, the governing 16.0-kip wheel loads should be modeled in more detail as two closely spaced 8.0-kip wheels 4.0 ft apart to more accurately reflect a modern tractor-trailer with tandem rear axles. Further, these wheel loads should be distributed over the specified contact area (20.0 in. wide × 10.0 in. long for rear axles and 10.0 in. square for front axles), which better approximates actual pressures applied from a dual tire unit (Kulicki and Mertz, 2006; Nowak, 2008). Note that the smaller 10.0 in. × 10.0 in. front wheels can be the controlling load for fatigue design of many orthotropic deck details.

This loading should be positioned both longitudinally and transversely on the bridge deck, ignoring the striped lanes, to create the worst stress or deflection, as applicable.

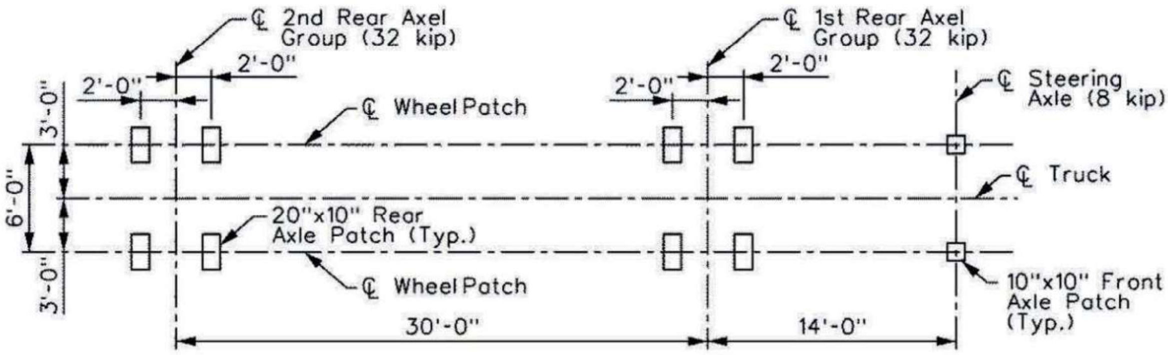


Figure 3.6.1.4.1-1—Refined Design Truck Footprint for Fatigue Design of Orthotropic Decks

3.6.1.4.2—Frequency

The frequency of the fatigue load shall be taken as the single-lane average daily truck traffic ($ADTT_{SL}$). This frequency shall be applied to all components of the bridge, even to those located under lanes that carry a lesser number of trucks.

In the absence of better information, the single-lane average daily truck traffic shall be taken as:

② $$ADTT_{SL} = p \times ADTT \quad (3.6.1.4.2\text{-}1)$$

where:

$ADTT$ = the number of trucks per day in one direction averaged over the design life

$ADTT_{SL}$ = the number of trucks per day in a single-lane averaged over the design life

p = fraction of traffic in a single lane, taken as specified in Table 3.6.1.4.2-1

C3.6.1.4.2

Since the fatigue and fracture limit state is defined in terms of accumulated stress-range cycles, specification of load alone is not adequate. Load should be specified along with the frequency of load occurrence.

For the purposes of this Article, a truck is defined as any vehicle with more than either two axles or four wheels.

The single-lane *ADTT* is that for the traffic lane in which the majority of the truck traffic crosses the bridge. On a typical bridge with no nearby entrance/exit ramps, the shoulder lane carries most of the truck traffic. The frequency of the fatigue load for a single lane is assumed to apply to all lanes since future traffic patterns on the bridge are uncertain.

Consultation with traffic engineers regarding any directionality of truck traffic may lead to the conclusion that one direction carries more than one-half of the bidirectional *ADTT*. If such data is not available from traffic engineers, designing for 55 percent of the bidirectional *ADTT* is suggested.

The value of $ADTT_{SL}$ is best determined in consultation with traffic engineers. However, traffic growth data is usually not predicted for the design life of the bridge, taken as 75 yr in these Specifications unless specified otherwise by the Owner. Techniques exist to extrapolate available data such as curve fitting growth rate vs. time and using extreme value distributions, but some judgment is required. Research has shown that the average daily traffic (*ADT*), including all vehicles, i.e., cars and trucks, is physically limited to about 20,000 vehicles per lane per day under normal conditions. This limiting value of traffic should be considered when estimating the *ADTT*. The *ADTT* can be determined by multiplying the *ADT* by the fraction of trucks in the traffic. In lieu of site-specific fraction of truck traffic data, the values of Table C3.6.1.4.2-1 may be applied for routine bridges.

③ **Table 3.6.1.4.2-1—Fraction of Truck Traffic in a Single Lane, *p***

Number of Lanes Available to Trucks	*p*
1	1.00
2	0.85
3 or more	0.80

Table C3.6.1.4.2-1—Fraction of Trucks in Traffic

Class of Highway	Fraction of Trucks in Traffic
Rural Interstate	0.20
Urban Interstate	0.15
Other Rural	0.15
Other Urban	0.10

3.6.1.4.3—Load Distribution for Fatigue

3.6.1.4.3a—Refined Methods

Where the bridge is analyzed by any refined method, as specified in Article 4.6.3, a single design truck shall be positioned transversely and longitudinally to maximize stress range at the detail under consideration, regardless of the position of traffic or design lanes on the deck.

C3.6.1.4.3a

If it were assured that the traffic lanes would remain as they are indicated at the opening of the bridge throughout its entire service life, it would be more appropriate to place the truck at the center of the traffic lane that produces maximum stress range in the detail under consideration. But because future traffic patterns on the bridge are uncertain and in the interest of minimizing the number of calculations required of the Designer, the position of the truck is made independent of the location of both the traffic lanes and the design lanes.

3.6.1.4.3b—Approximate Methods

Where the bridge is analyzed by approximate load distribution, as specified in Article 4.6.2, the distribution factor for one traffic lane shall be used.

3.6.1.5—Rail Transit Load

Where a bridge also carries rail-transit vehicles, the Owner shall specify the transit load characteristics and the expected interaction between transit and highway traffic.

C3.6.1.5

If rail transit is designed to occupy an exclusive lane, transit loads should be included in the design, but the bridge should not have less strength than if it had been designed as a highway bridge of the same width.

If the rail transit is supposed to mix with regular highway traffic, the Owner should specify or approve an appropriate combination of transit and highway loads for the design.

Transit load characteristics may include:

- loads,
- load distribution,
- load frequency,
- dynamic allowance, and
- dimensional requirements.

3.6.1.6—Pedestrian Loads

A pedestrian load of 0.075 ksf shall be applied to all sidewalks wider than 2.0 ft and considered simultaneously with the vehicular design live load in the vehicle lane. Where vehicles can mount the sidewalk, sidewalk pedestrian load shall not be considered concurrently. If a sidewalk may be removed in the future, the vehicular live loads shall be applied at 1.0 ft from edge-of-deck for design of the overhang, and 2.0 ft from edge-of-deck for design of all other components. The pedestrian load shall not be considered to act concurrently with vehicles.

C3.6.1.6

See the provisions of Article C3.6.1.1.2 for applying the pedestrian loads in combination with the vehicular live load.

compression—compression and a fatigue crack will not propogate beyond a heat-affected zone.

Where force effects in cross-frames or diaphragms are computed from a refined analysis, it is desirable to check any fatigue-sensitive details on these members that are subjected to a net applied tensile stress. In such cases, the effect of positioning the fatigue truck in two different transverse positions located directly over the adjacent connected girders, or directly over the adjacent connected girder webs in the case of a box section, usually creates the largest range of stress or torque in these bracing members. There is an extremely low probability of the truck being located in these two critical relative transverse positions over millions of cycles. Also, field observation has not indicated a significant problem with the details on these members caused by load-induced fatigue or fatigue due to cross-section distortion. Therefore, it is recommended that the fatigue truck be positioned to determine the maximum range of stress or torque, as applicable, in these members as specified in Article 3.6.1.4.3a, with the truck confined to one critical transverse position per each longitudinal position throughout the length of the bridge in the analysis.

6.6.1.2.2—Design Criteria

For load-induced fatigue considerations, each detail shall satisfy:

4

$$\gamma(\Delta f) \leq (\Delta F)_n \qquad (6.6.1.2.2\text{-}1)$$

where:

γ = load factor specified in Table 3.4.1-1 for the fatigue load combination

(Δf) = force effect, live load stress range due to the passage of the fatigue load as specified in Article 3.6.1.4 (ksi)

$(\Delta F)_n$ = nominal fatigue resistance as specified in Article 6.6.1.2.5 (ksi)

C6.6.1.2.2

Eq. 6.6.1.2.2-1 may be developed by rewriting Eq. 1.3.2.1-1 in terms of fatigue load and resistance parameters:

$$\eta\gamma(\Delta f) \leq \phi(\Delta F)_n \qquad (C6.6.1.2.2\text{-}1)$$

but for the fatigue limit state,

$\eta = 1.0$
$\phi = 1.0$

6.6.1.2.3—Detail Categories

Components and details shall be designed to satisfy the requirements of their respective detail categories summarized in Table 6.6.1.2.3-1. Where bolt holes are depicted in Table 6.6.1.2.3-1, their fabrication shall conform to the provisions of Article 11.4.8.5 of the *AASHTO LRFD Bridge Construction Specifications.* Where permitted for use, unless specific information is available to the contrary, bolt holes in cross-frame, diaphragm, and lateral bracing members and their connection plates shall be assumed for design to be punched full size.

Except as specified herein for components and details on fracture critical members, where the projected 75-year single lane Average Daily Truck Traffic $(ADTT)_{SL}$ is less than or equal to the applicable value specified in Table 6.6.1.2.3-2 for the Detail Category under consideration, the

C6.6.1.2.3

Components and details susceptible to load-induced fatigue cracking have been grouped into eight categories, called detail categories, by fatigue resistance.

Experience indicates that in the design process the fatigue considerations for Detail Categories A through B′ rarely, if ever, govern. Nevertheless, Detail Categories A through B′ have been included in Table 6.6.1.2.3-1 for completeness. Investigation of components and details with a fatigue resistance based on Detail Categories A through B′ may be appropriate in unusual design cases.

Table 6.6.1.2.3-1 illustrates many common details found in bridge construction and identifies potential crack initiation points for each detail. In Table 6.6.1.2.3-1, "Longitudinal" signifies that the direction of applied stress is parallel to the longitudinal axis of the detail. "Transverse" signifies that the direction of applied stress is

Fatigue II load combination specified in Table 3.4.1-1 may be used in combination with the nominal fatigue resistance for finite life specified in Article 6.6.1.2.5. Otherwise, the Fatigue I load combination shall be used in combination with the nominal fatigue resistance for infinite life specified in Article 6.6.1.2.5. The single-lane Average Daily Truck Traffic $(ADTT)_{SL}$ shall be computed as specified in Article 3.6.1.4.2.

For components and details on fracture-critical members, the Fatigue I load combination specified in Table 3.4.1-1 should be used in combination with the nominal fatigue resistance for infinite life specified in Article 6.6.1.2.5.

Orthotropic deck components and details shall be designed to satisfy the requirements of their respective detail categories summarized in Table 6.6.1.2.3-1 for the chosen design level shown in the table and as specified in Article 9.8.3.4.

perpendicular to the longitudinal axis of the detail.

Category F for allowable shear stress range on the throat of a fillet weld has been eliminated from Table 6.6.1.2.3-1. When fillet welds are properly sized for strength considerations, Category F should not govern. Fatigue will be governed by cracking in the base metal at the weld toe and not by shear on the throat of the weld. Research on end-bolted cover plates is discussed in Wattar et al. (1985).

Where the design stress range calculated using the Fatigue I load combination is less than $(\Delta F)_{TH}$, the detail will theoretically provide infinite life. Except for Categories E and E′, for higher traffic volumes, the design will most often be governed by the infinite life check. Table 6.6.1.2.3-2 shows for each detail category the values of $(ADTT)_{SL}$ above which the infinite life check governs, assuming a 75-year design life and one stress range cycle per truck.

The values in the second column of Table 6.6.1.2.3-2 were computed as follows:

$$75-year(ADTT)_{SL}=\frac{A}{\left[\dfrac{0.80(\Delta F)_{TH}}{1.75}\right]^{3}(365)(75)(n)} \quad \text{(C6.6.1.2.3-1)}$$

using the values for A and $(\Delta F)_{TH}$ specified in Tables 6.6.1.2.5-1 and 6.6.1.2.5-3, respectively, a fatigue design life of 75 years and a number of stress range cycles per truck passage, n, equal to one. These values were rounded up to the nearest five trucks per day. That is, the indicated values were determined by equating infinite life and finite life resistances with due regard to the difference in load factors used with the Fatigue I and Fatigue II load combinations. For other values of n, the values in Table 6.6.1.2.3-2 should be modified by dividing by the appropriate value of n taken from Table 6.6.1.2.5-2.

For other values of the fatigue design life, the values in Table 6.6.1.2.3-2 should be modified by multiplying the values by the ratio of 75 divided by the fatigue life sought in years.

The procedures for load-induced fatigue are followed for orthotropic deck design. Although the local structural stress range for certain fatigue details can be caused by distortion of the deck plate, ribs, and floorbeams, research has demonstrated that load-induced fatigue analysis produces a reliable assessment of fatigue performance.

Considering the increased γ_{LL} and cycles per truck passage (n) in orthotropic decks, the 75-year $ADTT_{SL}$ equivalent to infinite life (trucks per day) results in 870 for deck plate details and 4,350 for all other details, based on Category C. Thus, finite life design may produce more economical designs on lower-volume roadways.

Table 6.6.1.2.3-1—Detail Categories for Load-Induced Fatigue

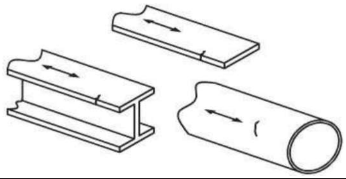
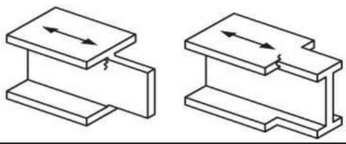
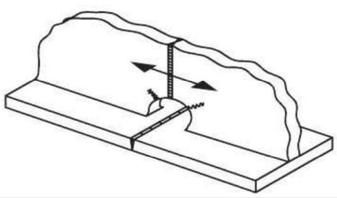
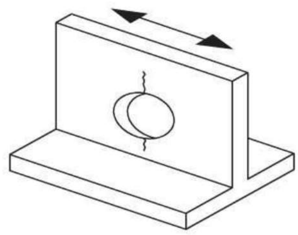
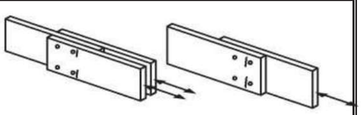
Description	Category	Constant A (ksi)^3	Threshold $(\Delta F)_{TH}$ ksi	Potential Crack Initiation Point	Illustrative Examples
Section 1—Plain Material away from Any Welding					
1.1 Base metal, except noncoated weathering steel, with rolled or cleaned surfaces. Flame-cut edges with surface roughness value of 1,000 μ-in. or less, but without re-entrant corners.	A	250×10^8	24	Away from all welds or structural connections	
1.2 Noncoated weathering steel base metal with rolled or cleaned surfaces designed and detailed in accordance with FHWA (1989). Flame-cut edges with surface roughness value of 1,000 μ-in. or less, but without re-entrant corners.	B	120×10^8	16	Away from all welds or structural connections	
1.3 Member with re-entrant corners at copes, cuts, block-outs or other geometrical discontinuities made to the requirements of AASHTO/AWS D1.5, except weld access holes.	C	44×10^8	10	At any external edge	
1.4 Rolled cross sections with weld access holes made to the requirements of AASHTO/AWS D1.5, Article 3.2.4.	C	44×10^8	10	In the base metal at the re-entrant corner of the weld access hole	
1.5 Open holes in members (Brown et al., 2007).	D	22×10^8	7	In the net section originating at the side of the hole	
Section 2—Connected Material in Mechanically Fastened Joints					
2.1 Base metal at the gross section of high-strength bolted joints designed as slip-critical connections with pretensioned high-strength bolts installed in holes drilled full size or subpunched and reamed to size—e.g., bolted flange and web splices and bolted stiffeners. (Note: see Condition 2.3 for bolt holes punched full size; see Condition 2.5 for bolted angle or tee section member connections to gusset or connection plates.)	B	120×10^8	16	Through the gross section near the hole	

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Table 6.6.1.2.3-1 (continued)—Detail Categories for Load-Induced Fatigue

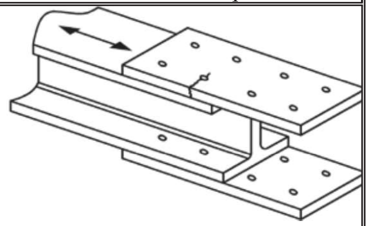
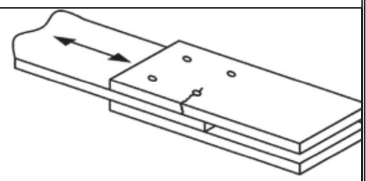
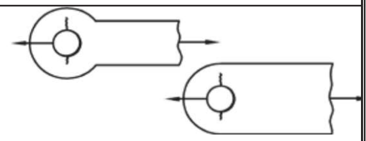
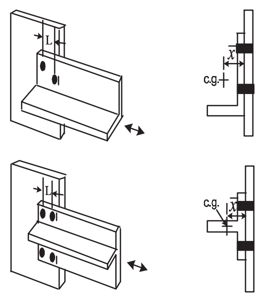
Description	Category	Constant A $(ksi)^3$	Threshold $(\Delta F)_{TH}$ ksi	Potential Crack Initiation Point	Illustrative Examples
2.2 Base metal at the net section of high-strength bolted joints designed as bearing-type connections but fabricated and installed to all requirements for slip-critical connections with pretensioned high-strength bolts installed in holes drilled full size or subpunched and reamed to size. (Note: see Condition 2.3 for bolt holes punched full size; see Condition 2.5 for bolted angle or tee section member connections to gusset or connection plates.)	B	120×10^8	16	In the net section originating at the side of the hole	
2.3 Base metal at the net or gross section 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).	D	22×10^8	7	In the net section originating at the side of the hole or through the gross section near the hole, as applicable	
2.4 Base metal at the net section of eyebar heads or pin plates (Note: for base metal in the shank of eyebars or through the gross section of pin plates, see Condition 1.1 or 1.2, as applicable.)	E	11×10^8	4.5	In the net section originating at the side of the hole	
2.5 Base metal in angle or tee section members connected to a gusset or connection plate with high-strength bolted slip-critical connections. The fatigue stress range shall be calculated on the effective net area of the member, $A_e = UA_g$, in which U=(1- $\bar{x}$ /L) and where A_g is the gross area of the member. $\bar{x}$ is the distance from the centroid of the member to the surface of the gusset or connection plate and L is the out-to-out distance between the bolts in the connection parallel to the line of force. The effect of the moment due to the eccentricities in the connection shall be ignored in computing the stress range (McDonald and Frank, 2009). The fatigue category shall be taken as that specified for Condition 2.1. For all other types of bolted connections, replace A_g with the net area of the member, A_n, in computing the effective net area according to the preceding equation and use the appropriate fatigue category for that connection type specified for Condition 2.2 or 2.3, as applicable.	See applicable Category above	See applicable Constant above	See applicable Threshold above	Through the gross section near the hole, or in the net section originating at the side of the hole, as applicable	

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Table 6.6.1.2.3-1 (continued)—Detail Categories for Load-Induced Fatigue

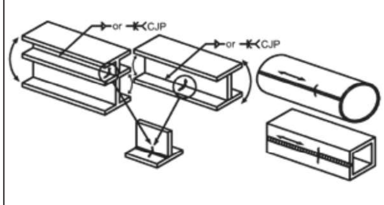
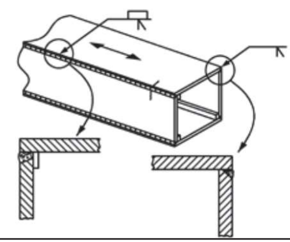
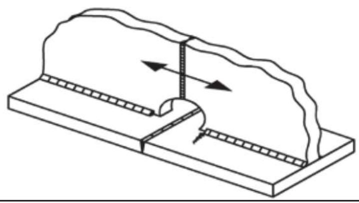
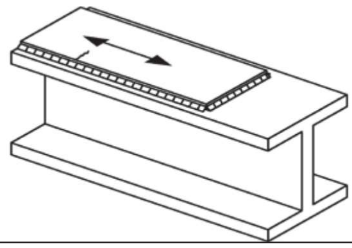
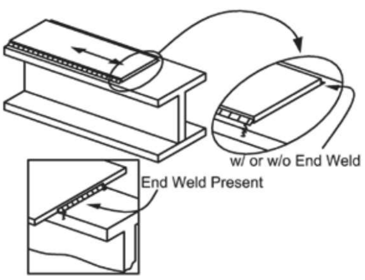
Description	Category	Constant A (ksi)^3	Threshold $(\Delta F)_{TH}$ ksi	Potential Crack Initiation Point	Illustrative Examples
Section 3—Welded Joints Joining Components of Built-up Members					
3.1 Base metal and weld metal in members without attachments built up of plates or shapes connected by continuous longitudinal complete joint penetration groove welds back-gouged and welded from the second side, or by continuous fillet welds parallel to the direction of applied stress.	B	120×10^8	16	From surface or internal discontinuities in the weld away from the end of the weld	
3.2 Base metal and weld metal in members without attachments built up of plates or shapes connected by continuous longitudinal complete joint penetration groove welds with backing bars not removed, or by continuous partial joint penetration groove welds parallel to the direction of applied stress.	B′	61×10^8	12	From surface or internal discontinuities in the weld, including weld attaching backing bars	
3.3 Base metal and weld metal at the termination of longitudinal welds at weld access holes made to the requirements of AASHTO/AWS D1.5, Article 3.2.4 in built-up members. (Note: does not include the flange butt splice).	D	22×10^8	7	From the weld termination into the web or flange	
3.4 Base metal and weld metal in partial length welded cover plates connected by continuous fillet welds parallel to the direction of applied stress.	B	120×10^8	16	From surface or internal discontinuities in the weld away from the end of the weld	
3.5 Base metal at the termination of partial length welded cover plates having square or tapered ends that are narrower than the flange, with or without welds across the ends, or cover plates that are wider than the flange with welds across the ends:				In the flange at the toe of the end weld or in the flange at the termination of the longitudinal weld or in the edge of the flange with wide cover plates	
Flange thickness ≤ 0.8 in.	E	11×10^8	4.5		
Flange thickness > 0.8 in.	E′	3.9×10^8	2.6		

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Table 6.6.1.2.3-1 (continued)—Detail Categories for Load-Induced Fatigue

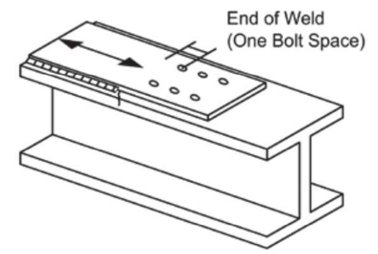
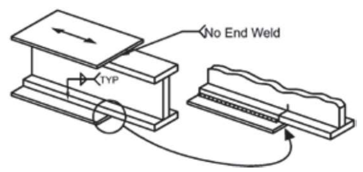
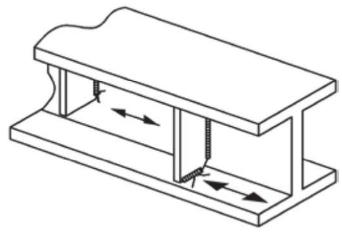
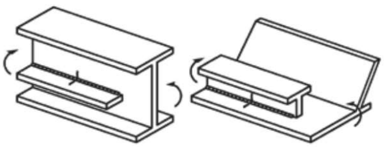
Description	Category	Constant A (ksi)^3	Threshold $(\Delta F)_{TH}$ ksi	Potential Crack Initiation Point	Illustrative Examples
Section 3—Welded Joints Joining Components of Built-Up Members (continued)					
3.6 Base metal at the termination of partial length welded cover plates with slip-critical bolted end connections satisfying the requirements of Article 6.10.12.2.3.	B	120×10^8	16	In the flange at the termination of the longitudinal weld	
3.7 Base metal at the termination of partial length welded cover plates that are wider than the flange and without welds across the ends.	E′	3.9×10^8	2.6	In the edge of the flange at the end of the cover plate weld	
Section 4—Welded Stiffener Connections					
4.1 Base metal at the toe of transverse stiffener-to-flange fillet welds and transverse stiffener-to-web fillet welds. (Note: includes similar welds on bearing stiffeners and connection plates). Base metal adjacent to bearing stiffener-to-flange fillet welds or groove welds.	C′	44×10^8	12	Initiating from the geometrical discontinuity at the toe of the fillet weld extending into the base metal	
4.2 Base metal and weld metal in longitudinal web or longitudinal box-flange stiffeners connected by continuous fillet welds parallel to the direction of applied stress.	B	120×10^8	16	From the surface or internal discontinuities in the weld away from the end of the weld	

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Table 6.6.1.2.3-1 (continued)—Detail Categories for Load-Induced Fatigue

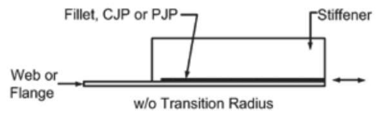
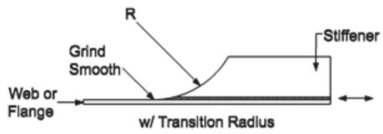
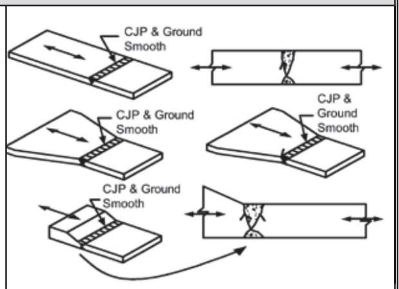
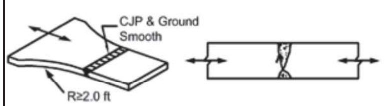
Description	Category	Constant A (ksi)^3	Threshold $(\Delta F)_{TH}$ ksi	Potential Crack Initiation Point	Illustrative Examples
Section 4—Welded Stiffener Connections (continued)					
4.3 Base metal at the termination of longitudinal stiffener-to-web or longitudinal stiffener-to-box flange welds:					
With the stiffener attached by welds and with no transition radius provided at the termination:				In the primary member at the end of the weld at the weld toe	
Stiffener thickness < 1.0 in.	E	11×10^8	4.5		
Stiffener thickness ≥ 1.0 in.	E′	3.9×10^8	2.6		
With the stiffener attached by welds and with a transition radius R provided at the termination with the weld termination ground smooth:				In the primary member near the point of tangency of the radius	
$R \geq 24$ in.	B	120×10^8	16		
24 in. > $R \geq 6$ in.	C	44×10^8	10		
6 in. > $R \geq 2$ in.	D	22×10^8	7		
2 in. > R	E	11×10^8	4.5		
Section 5—Welded Joints Transverse to the Direction of Primary Stress					
5.1 Base metal and weld metal in or adjacent to complete joint penetration groove welded butt splices, with weld soundness established by NDT and with welds ground smooth and flush parallel to the direction of stress. Transitions in thickness or width shall be made on a slope no greater than 1:2.5 (see also Figure 6.13.6.2-1).				From internal discontinuities in the filler metal or along the fusion boundary or at the start of the transition	
$F_y < 100$ ksi	B	120×10^8	16		
$F_y \geq 100$ ksi	B′	61×10^8	12		
5.2 Base metal and weld metal in or adjacent to complete joint penetration groove welded butt splices, with weld soundness established by NDT and with welds ground parallel to the direction of stress at transitions in width made on a radius of not less than 2 ft with the point of tangency at the end of the groove weld (see also Figure 6.13.6.2-1).	B	120×10^8	16	From internal discontinuities in the filler metal or discontinuities along the fusion boundary	

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Table 6.6.1.2.3-1 (continued)—Detail Categories for Load-Induced Fatigue

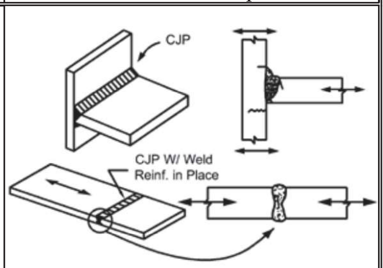
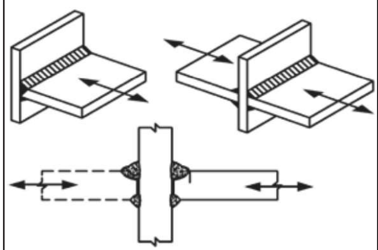
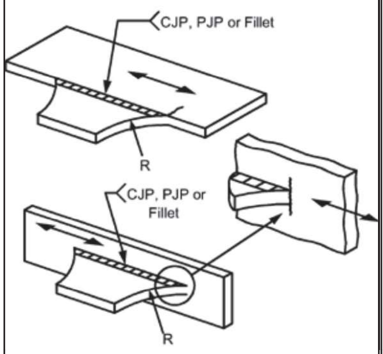
Description	Category	Constant A $(ksi)^3$	Threshold $(\Delta F)_{TH}$ ksi	Potential Crack Initiation Point	Illustrative Examples
5.3 Base metal and weld metal in or adjacent to the toe of complete joint penetration groove welded T or corner joints, or in complete joint penetration groove welded butt splices, with or without transitions in thickness having slopes no greater than 1:2.5 when weld reinforcement is not removed. (Note: cracking in the flange of the "T" may occur due to out-of-plane bending stresses induced by the stem).	C	44×10^8	10	From the surface discontinuity at the toe of the weld extending into the base metal or along the fusion boundary	
5.4 Base metal and weld metal at details where loaded discontinuous plate elements are connected with a pair of fillet welds or partial joint penetration groove welds on opposite sides of the plate normal to the direction of primary stress.	C as adjusted in Eq. 6.6.1.2.5-4	44×10^8	10	Initiating from the geometrical discontinuity at the toe of the weld extending into the base metal or initiating at the weld root subject to tension extending up and then out through the weld	
Section 6—Transversely Loaded Welded Attachments					
6.1 Base metal in a longitudinally loaded component at a transversely loaded detail (e.g. a lateral connection plate) attached by a weld parallel to the direction of primary stress and incorporating a transition radius R: With the weld termination ground smooth:				Near point of tangency of the radius at the edge of the longitudinally loaded component or at the toe of the weld at the weld termination if not ground smooth	
$R \geq 24$ in.	B	120×10^8	16		
24 in. $> R \geq 6$ in.	C	44×10^8	10		
6 in. $> R \geq 2$ in.	D	22×10^8	7		
2 in. $> R$	E	11×10^8	4.5		
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.)	E	11×10^8	4.5		

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Table 6.6.1.2.3-1 (continued)—Detail Categories for Load-Induced Fatigue

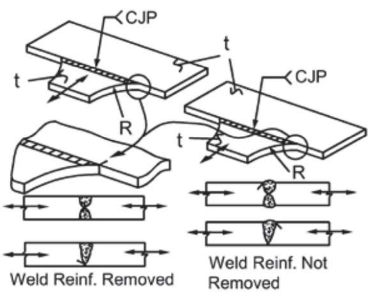
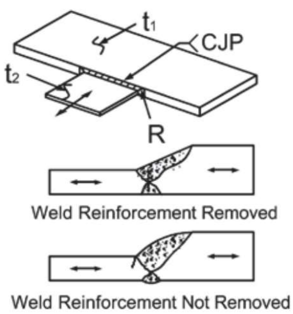
Description	Category	Constant A (ksi)^3	Threshold $(\Delta F)_{TH}$ ksi	Potential Crack Initiation Point	Illustrative Examples
Section 6—Transversely Loaded Welded Attachments (continued)					
6.2 Base metal in a transversely loaded detail (e.g. a lateral connection plate) attached to a longitudinally loaded component of equal thickness by a complete joint penetration groove weld parallel to the direction of primary stress and incorporating a transition radius R, with weld soundness established by NDT and with the weld termination ground smooth: With the weld reinforcement removed:					
$R \geq 24$ in.	B	120×10^8	16	Near points of tangency of the radius or in the weld or at the fusion boundary of the longitudinally loaded component or the transversely loaded attachment	
24 in. $> R \geq 6$ in.	C	44×10^8	10		
6 in. $> R \geq 2$ in.	D	22×10^8	7		
2 in. $> R$	E	11×10^8	4.5		
With the weld reinforcement not removed:				At the toe of the weld either along the edge of the longitudinally loaded component or the transversely loaded attachment	
$R \geq 24$ in.	C	44×10^8	10		
24 in. $> R \geq 6$ in.	C	44×10^8	10		
6 in. $> R \geq 2$ in.	D	22×10^8	7		
2 in. $> R$	E	11×10^8	4.5		
(Note: Condition 6.1 shall also be checked.)					
6.3 Base metal in a transversely loaded detail (e.g. a lateral connection plate) attached to a longitudinally loaded component of unequal thickness by a complete joint penetration groove weld parallel to the direction of primary stress and incorporating a weld transition radius R, with weld soundness established by NDT and with the weld termination ground smooth:				At the toe of the weld along the edge of the thinner plate In the weld termination of small radius weld transitions At the toe of the weld along the edge of the thinner plate	
With the weld reinforcement removed:					
R $\geq$ 2 in.	D	22×10^8	7		
$R <$ 2 in.	E	11×10^8	4.5		
For any weld transition radius with the weld reinforcement not removed.	E	11×10^8	4.5		
(Note: Condition 6.1 shall also be checked.)					

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Table 6.6.1.2.3-1 (continued)—Detail Categories for Load-Induced Fatigue

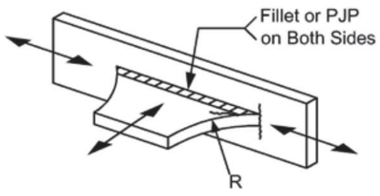
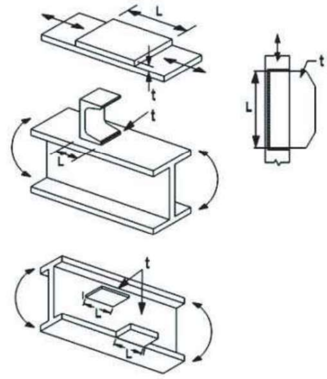
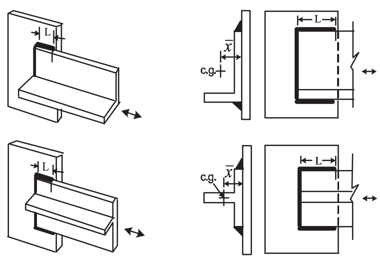
Description	Category	Constant A (ksi)^3	Threshold $(\Delta F)_{TH}$ ksi	Potential Crack Initiation Point	Illustrative Examples
Section 6—Transversely Loaded Welded Attachments (continued)					
6.4 Base metal in a transversely loaded detail (e.g. a lateral connection plate) attached to a longitudinally loaded component by a fillet weld or a partial joint penetration groove weld, with the weld parallel to the direction of primary stress (Note: Condition 6.1 shall also be checked.)	See Condition 5.4				
Section 7—Longitudinally Loaded Welded Attachments					
7.1 Base metal in a longitudinally loaded component at a detail with a length L in the direction of the primary stress and a thickness t attached by groove or fillet welds parallel or transverse to the direction of primary stress where the detail incorporates no transition radius:				In the primary member at the end of the weld at the weld toe	
$L < 2$ in.	C	44×10^8	10		
2 in. $\leq L \leq 12t$ or 4 in	D	22×10^8	7		
$L > 12t$ or 4 in.					
$t < 1.0$ in.	E	11×10^8	4.5		
$t \geq 1.0$ in.	E′	3.9×10^8	2.6		
(Note: see Condition 7.2 for welded angle or tee section member connections to gusset or connection plates.)					
7.2 Base metal in angle or tee section members connected to a gusset or connection plate by longitudinal fillet welds along both sides of the connected element of the member cross-section, and with or without backside welds. The fatigue stress range shall be calculated on the effective net area of the member, $A_e = UA_g$, in which $U = (1- \bar{x}/L)$ and where A_g is the gross area of the member. $\bar{x}$ is the distance from the centroid of the member to the surface of the gusset or connection plate and L is the maximum length of the longitudinal welds. The effect of the moment due to the eccentricities in the connection shall be ignored in computing the stress range (McDonald and Frank, 2009).	E′	3.9×10^8	2.6	Toe of fillet welds in connected element	

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Table 6.6.1.2.3-1 (continued)—Detail Categories for Load-Induced Fatigue

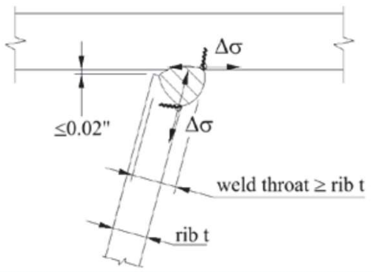
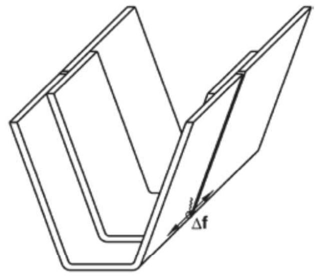
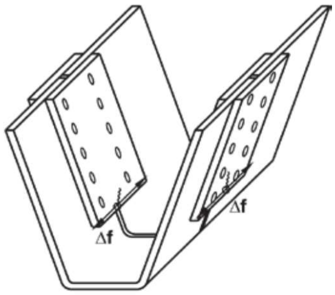
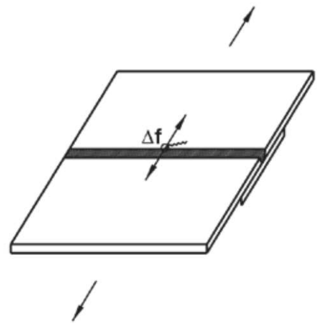
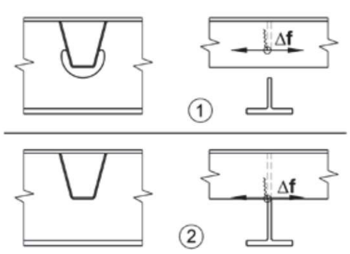
Description	Category	Constant A $(\text{ksi})^3$	Threshold $(\Delta F)_{TH}$ ksi	Potential Crack Initiation Point	Illustrative Examples
Section 8—Orthotropic Deck Details					
8.1 Rib to Deck Weld—One-sided (60% min) penetration weld with root gap ≤ 0.02 in. prior to welding. Weld throat ≥ rib wall thickness. Allowable Design Level 1, 2, or 3	C	44×10^8	10	See Figure	
8.2 Rib Splice (Welded)—Single groove butt weld with permanent backing bar left in place. Weld gap > rib wall thickness Allowable Design Level 1, 2, or 3	D	22×10^8	7	See Figure	
8.3 Rib Splice (Bolted)—Base metal at gross section of high strength slip critical connection Allowable Design Level 1, 2, or 3	B	120×10^8	16	See Figure	
8.4 Deck Plate Splice (in Plane)—Transverse or Longitudinal single groove butt splice with permanent backing bar left in place Allowable Design Level 1, 2, or 3	D	22×10^8	7	See Figure	
8.5 Rib to FB Weld (Rib)—Rib wall at rib to FB weld (fillet or CJP) Allowable Design Level 1, 2, or 3	C	44×10^8	10	See Figure	

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Table 6.6.1.2.3-1 (continued)—Detail Categories for Load-Induced Fatigue

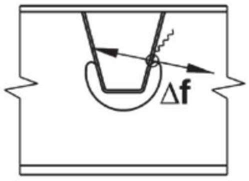
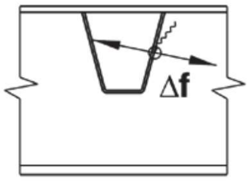
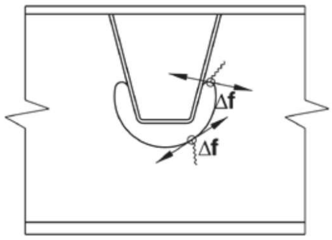
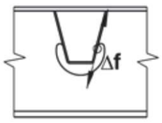
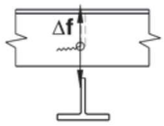
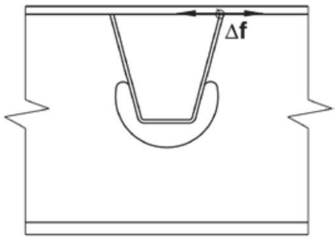
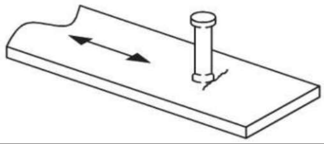
Description	Category	Constant A (ksi)^3	Threshold $(\Delta F)_{TH}$ ksi	Potential Crack Initiation Point	Illustrative Examples
8.6 Rib to FB Weld (FB Web)—FB web at rib to FB weld (fillet, PJP, or CJP) Allowable Design Level 1 or 3	C (see Note 1)	44×10^8	10	See Figure	
8.7 FB Cutout—Base metal at edge with "smooth" flame cut finish as per AWS D1.5 Allowable Design Level 1 or 3	A	250×10^8	24	See Figure	
8.8 Rib Wall at Cutout—Rib wall at rib to FB weld (fillet, PJP, or CJP) Allowable Design Level 1 or 3	C	44×10^8	10	See Figure	
8.9 Rib to Deck Plate at FB Allowable Design Level 1 or 3	C	44×10^8	10	See Figure	
Note 1: Where stresses are dominated by in-plane component at fillet or PJP welds, Eq. 6.6.1.2.5-4 shall be considered. In this case, Δf should be calculated at the mid-thickness and the extrapolation procedure as per Article 9.8.3.4.3 need not be applied.					
Section 9—Miscellaneous					
9.1 Base metal at stud-type shear connectors attached by fillet or automatic stud welding	C	44×10^8	10	At the toe of the weld in the base metal	

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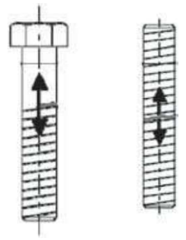
Table 6.6.1.2.3-1 (continued)—Detail Categories for Load-Induced Fatigue

Description	Category	Constant A (ksi^3)	Threshold $(\Delta F)_{TH}$ ksi	Potential Crack Initiation Point	Illustrative Examples
Section 9—Miscellaneous (continued)					
9.2 Nonpretensioned high-strength bolts, common bolts, threaded anchor rods, and hanger rods with cut, ground, or rolled threads. Use the stress range acting on the tensile stress area due to live load plus prying action when applicable.				At the root of the threads extending into the tensile stress area	
(Fatigue II) Finite Life	E′	3.9×10^8	N/A		
(Fatigue I) Infinite Life	D	N/A	7		

⑤

Table 6.6.1.2.3-2—75-year $(ADTT)_{SL}$ Equivalent to Infinite Life

Detail Category	75-year $(ADTT)_{SL}$ Equivalent to Infinite Life (trucks per day)
A	690
B	1120
B′	1350
C	1680
C′	975
D	2450
E	4615
E′	8485

6.6.1.2.4—Detailing to Reduce Constraint

Welded structures shall be detailed to avoid conditions that create highly constrained joints and crack-like geometric discontinuities that are susceptible to constraint-induced fracture, as summarized in Tables 6.6.1.2.4-1 and 6.6.1.2.4-2. If a gap is specified between the weld toes at the joint under consideration, the gap shall not be less than 0.5 in.

C6.6.1.2.4

The objective of this Article is to provide recommended detailing guidelines for common joints to avoid details susceptible to brittle fracture.

The form of brittle fracture being addressed has been termed "constraint-induced fracture" and can occur without any perceptible fatigue crack growth and, more importantly, without any warning. This type of failure was documented during the Hoan Bridge failure investigation by Wright, Kaufmann, and Fisher (2003) and Kaufmann, Connor, and Fisher (2004). Criteria have been developed to identify bridges and details susceptible to this failure mode as discussed in Mahmoud, Connor, and Fisher (2005).

Attached elements parallel to the primary stress are sometimes interrupted when intersecting a full-depth transverse member. In regions subject to a net tensile stress under Strength Load Combination I, these elements are less susceptible to fracture and fatigue if the attachment parallel to the primary stress is continuous and the transverse attachment is discontinuous as shown in Tables 6.6.1.2.4-1 and 6.6.1.2.4-2. If a gap is specified between the weld toes at the joint under consideration, the gap must not be less than the specified 0.5-in. minimum; larger gaps are acceptable. If a gap is not specified, since the continuous longitudinal stiffener or lateral connection plate is typically welded to the web before the discontinuous vertical stiffener, the cope or snipe in the vertical stiffener should be reduced so that it just clears the longitudinal weld. The welds may either be stopped short of free edges as shown in Tables 6.6.1.2.4-1 and 6.6.1.2.4-2 or wrapped for sealing as specified in Article 6.13.3.7. A longitudinal stiffener or lateral connection plate may be discontinuous at the intersection, but only if the intersection is subject to a net compressive stress under Strength Load Combination I and the longitudinal stiffener or lateral connection plate is attached to the continuous vertical web stiffeners as shown in Tables 6.6.1.2.4-1 and 6.6.1.2.4-2; such a detail is recommended at intersections with bearing stiffeners.

Table 6.6.1.2.4-1—Details to Avoid Conditions Susceptible to Constraint-Induced Fracture at the Intersection of Longitudinal Stiffeners and Vertical Stiffeners Welded to the Web

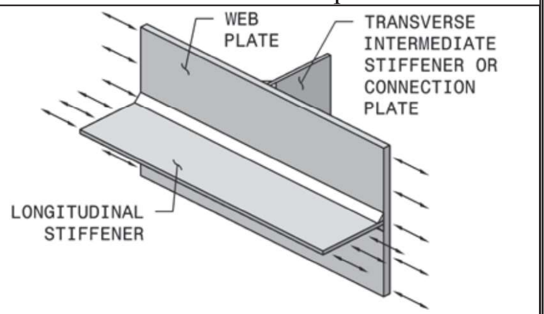
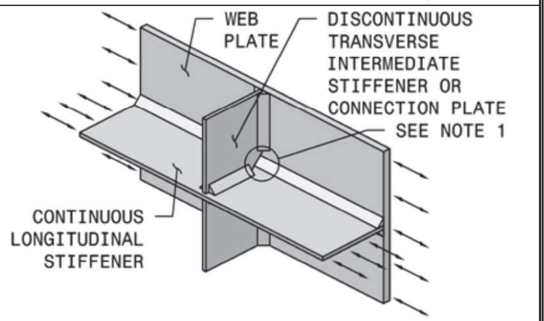
Description	Net Stress State at Location of Intersection (under Strength Load Combination I)	Illustrative Example
At locations where a longitudinal stiffener is on the opposite side of the web from a transverse intermediate stiffener or connection plate	Tension, Compression, or Reversal	
At locations where a longitudinal stiffener must intersect a transverse intermediate stiffener or a connection plate Note the longitudinal stiffener is continuous.	Tension, Compression, or Reversal	

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Table 6.6.1.2.4-1 (continued)—Details to Avoid Conditions Susceptible to Constraint-Induced Fracture at the Intersection of Longitudinal Stiffeners and Vertical Stiffeners Welded to the Web

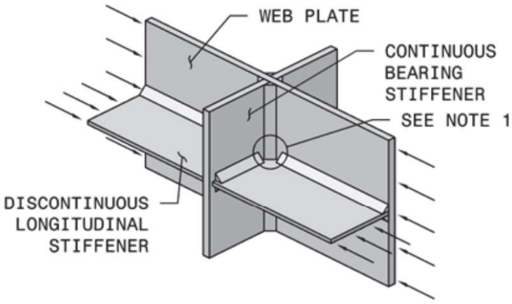
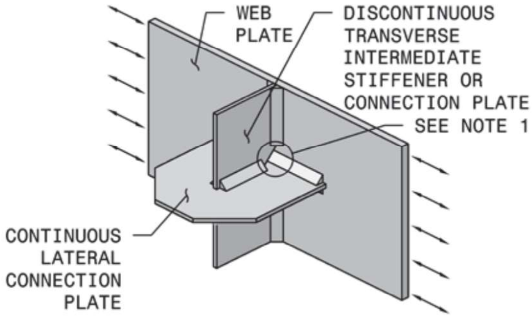
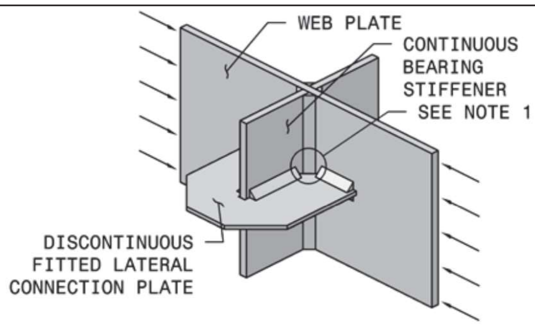
Description	Net Stress State at Location of Intersection (under Strength Load Combination I)	Illustrative Example
At locations where a longitudinal stiffener must intersect a bearing stiffener, or must intersect the outermost bearing stiffener if more than one pair of stiffeners is used Note the bearing stiffener is continuous. The same detail may be used at locations where a longitudinal stiffener must intersect a transverse intermediate stiffener or connection plate if the intersection is subject to net compression only.	Compression Only	
Note 1: If a gap is specified between the weld toes, the recommended minimum distance between the weld toes is 0.75 in., but shall not be less than 0.5 in. Larger gaps are also acceptable.		

Table 6.6.1.2.4-2—Details to Avoid Conditions Susceptible to Constraint-Induced Fracture at the Intersection of Lateral Connection Plates and Vertical Stiffeners Welded to the Web

Description	Net Stress State at Location of Intersection (under Strength Load Combination I)	Illustrative Example (Lateral bracing members not shown for clarity)
When it is not practical to attach a lateral connection plate to a flange, and the lateral connection plate must be placed on the same side of the web as a transverse intermediate stiffener or connection plate. See also the provisions of Article 6.6.1.3.2. Note the transverse intermediate stiffener or connection plate is discontinuous.	Tension, Compression, or Reversal	
At the intersection of a lateral connection plate with a bearing stiffener when it is not practical to attach the lateral connection plate to a flange. See also the provisions of Article 6.6.1.3.2. Note the bearing stiffener is continuous. The same detail may be used at the intersection of a lateral connection plate with a transverse intermediate stiffener or connection plate if the intersection is subject to net compression only.	Compression Only	
Note 1: If a gap is specified between the weld toes, the recommended minimum distance between the weld toes is 0.75 in., but shall not be less than 0.5 in. Larger gaps are also acceptable.		

6.6.1.2.5—Fatigue Resistance

Except as specified below, nominal fatigue resistance shall be taken as:

- For the Fatigue I load combination and infinite life:

$$(\Delta F)_n = (\Delta F)_{TH} \quad (6.6.1.2.5\text{-}1)$$

- For the Fatigue II load combination and finite life:

6

$$(\Delta F)_n = \left(\frac{A}{N}\right)^{\frac{1}{3}} \quad (6.6.1.2.5\text{-}2)$$

in which:

7

$$N = (365)(75)n(ADTT)_{SL} \quad (6.6.1.2.5\text{-}3)$$

where:

A	=	constant taken from Table 6.6.1.2.5-1 (ksi^3)
n	=	number of stress range cycles per truck passage taken from Table 6.6.1.2.5-2
$(ADTT)_{SL}$	=	single-lane *ADTT* as specified in Article 3.6.1.4
$(\Delta F)_{TH}$	=	constant-amplitude fatigue threshold taken from Table 6.6.1.2.5-3 (ksi)

C6.6.1.2.5

The requirement on higher-traffic-volume bridges that the maximum stress range experienced by a detail be less than the constant-amplitude fatigue threshold provides a theoretically infinite fatigue life. This requirement is reflected in Eq. 6.6.1.2.5-1. Values of n for longitudinal members have been revised based on the calibration reported in Kulicki et al., 2014.

The fatigue resistance above the constant amplitude fatigue threshold, in terms of cycles, is inversely proportional to the cube of the stress range, e.g., if the stress range is reduced by a factor of 2, the fatigue life increases by a factor of 2^3. This is reflected in Eq. 6.6.1.2.5-2. Orthotropic deck details that are connected to the deck plate (e.g., the rib-to-deck weld) are subjected to cycling from direct individual wheel loads. Thus, the passage of one design truck results in five fatigue load cycles as each axle produces one load cycle. The force effect (Δf) can be conservatively taken as the worst case from the five wheels or by application of Miner's Rule to determine the effective stress range from the group of wheels.

In the AASHTO 2002 *Standard Specifications*, the constant amplitude fatigue threshold is termed the allowable fatigue stress range for more than 2 million cycles on a redundant load path structure.

The fatigue design life has been considered to be 75 years in the overall development of the Specifications. If a fatigue design life other than 75 years is sought, a number other than 75 may be inserted in the equation for N.

Figure C6.6.1.2.5-1 is a graphical representation of the nominal fatigue resistance for Categories A through E′. 8

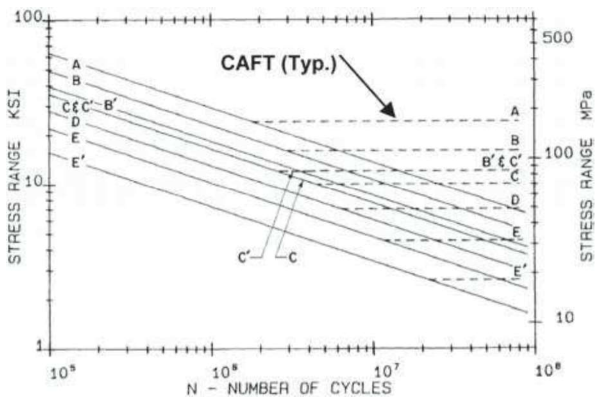


Figure C6.6.1.2.5-1—Stress Range Versus Number of Cycles

The nominal fatigue resistance for base metal and weld metal at details where loaded discontinuous plate elements are connected with a pair of fillet welds or partial joint penetration groove welds on opposite sides of the plate normal to the direction of primary stress, or where partial joint penetration groove welds are transversely loaded in tension, shall be taken as:

$$(\Delta F)_n = (\Delta F)_n^c \left(\frac{0.61 - 0.56\left(\frac{2a}{t_p}\right) + 0.68\left(\frac{w}{t_p}\right)}{t_p^{0.167}} \right) \leq (\Delta F)_n^c \tag{6.6.1.2.5-4}$$

where:

$(\Delta F)_n^c$ = nominal fatigue resistance for Detail Category C determined from Eqs. 6.6.1.2.5-1 or 6.6.1.2.5-2, as applicable (ksi)

$2a$ = length of the non-welded root face in the direction of the thickness of the loaded plate (in.) For fillet welded connections, the quantity $(2a/t_p)$ shall be taken equal to 1.0.

t_p = thickness of loaded plate (in.)

w = leg size of the reinforcement or contour fillet, if any, in the direction of the thickness of the loaded plate (in.)

Eq. 6.6.1.2.5-4 accounts for the potential of a crack initiating from the weld root and includes the effects of weld penetration. Therefore, Eq. 6.6.1.2.5-4 is also applicable to partial joint penetration groove welds, as shown in Figure C6.6.1.2.5-2, where such welds are transversely loaded in tension.

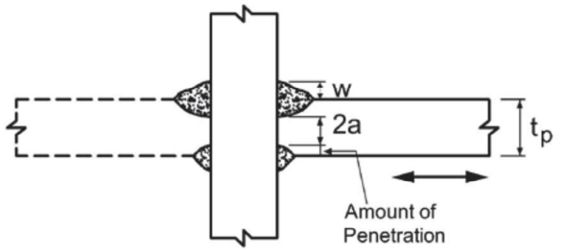


Figure C6.6.1.2.5-2—Loaded Discontinuous Plate Element Connected by a Pair of Partial Joint Penetration Groove Welds

The effect of any weld penetration may be conservatively ignored in the calculation of $(\Delta F)_n$ from Eq. 6.6.1.2.5-4 by taking the quantity $(2a/t_p)$ equal to 1.0. The nominal fatigue resistance based on the crack initiating from the weld root in Eq. 6.6.1.2.5-4 is limited to the nominal fatigue resistance for Detail Category C, which assumes crack initiation from the weld toe. Eq. 6.6.1.2.5-4 was developed for values of $(2a/t_p)$ ranging from 0.30 to 1.1, and values of (w/t_p) ranging from 0.30 to 1.0. For values of $(2a/t_p)$ less than 0.3 and/or (w/t_p) greater than 1.0, the nominal fatigue resistance is equal to the resistance for Detail Category C. The development of Eq. 6.6.1.2.5-4 is discussed in Frank and Fisher (1979).

The values of the detail category constant, A, and constant-amplitude fatigue threshold, $(\Delta F)_{TH}$, specified in Tables 6.6.1.2.5-1 and 6.6.1.2.5-3 for bolts subject to axial tension shall apply only to fully pretensioned high-strength bolts. Otherwise, Condition 9.2 in Table 6.6.1.2.3-1 shall apply.

In the AASHTO *Standard Specifications* (2002), allowable stress ranges are specified for both redundant and nonredundant members. The allowables specified for nonredundant members are arbitrarily reduced from those specified for redundant members due to the more severe consequences of failure of a nonredundant member. However, greater fracture toughness is also specified for nonredundant members. In combination, the reduction in allowable stress range and the greater fracture toughness constitute an unnecessary double penalty for nonredundant members. The requirement for greater fracture toughness has been maintained in these Specifications. Therefore, the allowable stress ranges represented by Eqs. 6.6.1.2.5-1 and 6.6.1.2.5-2 are applicable to both redundant and nonredundant members.

⑨

Table 6.6.1.2.5-1—Detail Category Constant, *A*

Detail Category	Constant, *A* times 10^8 (ksi^3)
A	250.0
B	120.0
B′	61.0
C	44.0
C′	44.0
D	22.0
E	11.0
E′	3.9
ASTM F3125, Grades A325 and F1852 Bolts in Axial Tension	17.1
ASTM F3125, Grades A490 and F2280 Bolts in Axial Tension	31.5

⑩

Table 6.6.1.2.5-2—Cycles per Truck Passage, *n*

Longitudinal Members	
Simple Span Girders	1.0
Continuous Girders: 1) near interior support 2) elsewhere	 1.5 1.0
Cantilever Girders	5.0
Orthotropic Deck Plate Connections Subjected to Wheel Load Cycling	5.0
Trusses	1.0
Transverse Members	
Spacing > 20.0 ft	1.0
Spacing ≤ 20.0 ft	2.0

For the purpose of determining the stress-range cycles per truck passage for continuous spans, a distance equal to one-tenth the span on each side of an interior support should be considered to be near the support.

The number of stress-range cycles per passage is taken as 5.0 for cantilever girders because this type of bridge is susceptible to large vibrations, which cause additional cycles after the truck has left the bridge (Moses et al., 1987; Schilling, 1990).

Orthotropic deck details that are connected to the deck plate (e.g., the rib-to-deck weld) are subjected to cycling from direct individual wheel loads. Thus, the passage of one design truck results in five fatigue load cycles as each axle produces one load cycle. The force effect (Δf) can be conservatively taken as the worst case from the five wheels or by application of Miner's Rule to determine the effective stress range from the group of wheels.

11 **Table 6.6.1.2.5-3—Constant-Amplitude Fatigue Thresholds**

Detail Category	Threshold (ksi)
A	24.0
B	16.0
B′	12.0
C	10.0
C′	12.0
D	7.0
E	4.5
E′	2.6
ASTM F3125, Grades A325 and F1852 Bolts in Axial Tension	31.0
ASTM F3125, Grades A490 and F2280 Bolts in Axial Tension	38.0

6.6.1.3—Distortion-Induced Fatigue

Load paths that are sufficient to transmit all intended and unintended forces shall be provided by connecting all transverse members to appropriate components comprising the cross-section of the longitudinal member. The load paths shall be provided by attaching the various components through either welding or bolting.

To control web buckling and elastic flexing of the web, the provision of Article 6.10.5.3 shall be satisfied.

C6.6.1.3

When proper detailing practices are not followed, fatigue cracking has been found to occur due to strains not normally computed in the design process. This type of fatigue cracking is called distortion-induced fatigue. Distortion-induced fatigue often occurs in the web near a flange at a welded connection plate for a cross-frame where a rigid load path has not been provided to adequately transmit the force in the transverse member from the web to the flange.

These rigid load paths are required to preclude the development of significant secondary stresses that could induce fatigue crack growth in either the longitudinal or the transverse member (Fisher et al., 1990).

6.6.1.3.1—Transverse Connection Plates

Except as specified herein, connection plates shall be welded or bolted to both the compression and tension flanges of the cross-section where:

- connecting diaphragms or cross-frames are attached to transverse connection plates or to transverse stiffeners functioning as connection plates,
- internal or external diaphragms or cross-frames are attached to transverse connection plates or to transverse stiffeners functioning as connection plates, and
- floorbeams or stringers are attached to transverse connection plates or to transverse stiffeners functioning as connection plates.

In the absence of better information, the welded or bolted connection should be designed to resist a 20.0-kip lateral load for straight, nonskewed bridges.

Where intermediate connecting diaphragms are used:

C6.6.1.3.1

These provisions appear in Article 10.20 of the AASHTO *Standard Specifications* "Diaphragms and Cross Frames" with no explanation as to the rationale for the requirements and no reference to distortion-induced fatigue.

These provisions apply to both diaphragms between longitudinal members and diaphragms internal to longitudinal members.

The 20.0-kip load represents a rule of thumb for straight, nonskewed bridges. For curved or skewed bridges, the diaphragm forces should be determined by analysis (Keating et al., 1990). It is noted that the stiffness of this connection is critical to help control relative displacement between the components. Hence, where possible, a welded connection is preferred as a bolted connection possessing sufficient stiffness may not be economical.

For box sections, webs are often joined to top flanges and cross-frame connection plates and transverse stiffeners are installed, and then these assemblies are attached to the common box flange. In order to weld the webs