2006 AASHTO BRIDGE COMMITTEE AGENDA ITEM: 1

SUBJECT: LRFD Bridge Design Specifications: Section 14, Various Articles

TECHNICAL COMMITTEE: T-2 Bearings

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

☒ **DESIGN SPEC** ☐ **CONSTRUCTION SPEC** ☐ **MOVABLE SPEC**
☐ **LRFR MANUAL** ☐ **OTHER**

☒ **US VERSION** ☐ **SI VERSION** ☐ **BOTH**

DATE PREPARED: 12/16/05
DATE REVISED:

AGENDA ITEM:

Item #1

Revise/add the following in Article 14.3 Notation:

a_{cr}	=	creep deflection divided by initial dead load deflection (dim.) (14.7.5.3.3)
G	=	shear modulus of the elastomer (ksi) (14.6.3.1) (C14.7.5.3.3)
h_{ri}	=	thickness of i^{th} elastomeric layer in elastomeric bearing (in.) (14.7.5.1) (14.7.5.3.3)
S	=	shape factor of thickest layer of an elastomeric bearing, shape factor of a layer in an elastomeric bearing (C14.6.3.2) (14.7.5.1) (C14.7.5.3.3)
δ_d	=	initial dead load compressive deflection of bearing (in.) (14.7.5.3.3)
δ_L	=	instantaneous live load compressive deflection of bearing (in.) (14.7.5.3.3)
δ_{lt}	=	long term dead load compressive deflection of bearing (in.) (14.7.5.3.3)
ε	=	compressive strain in an elastomer layer of a laminated bearing (C14.7.5.3.3)
ε_{di}	=	initial dead load compressive strain in i^{th} elastomer layer of a laminated bearing (14.7.5.3.3)
ε_{Li}	=	instantaneous live load compressive strain in i^{th} elastomer layer of a laminated bearing (14.7.5.3.3)
σ	=	compressive stress in an elastomer layer of a laminated bearing (ksi) (C14.7.5.3.3)

Item #2

Add the following two sentences to the end of the first paragraph of Article C14.6.3.1:

For example, the designer should take note that in cold temperatures which approach the appropriate minimum specified zone temperatures, the shear modulus, G, of an elastomer may be as much as four times that at 73°F. See Article 14.7.5.2 and the *AASHTO Materials Specification M 251* for more information.

Item #3

Revise Articles 14.7.5.2, C14.7.5.2, 14.7.5.3, and C14.7.5.3 as shown in the attached pages.

Item #4

Revise Articles 14.7.6.2, C14.7.6.2, 14.7.6.3, and C14.7.6.3 as shown in the attached pages.

OTHER AFFECTED ARTICLES:

None

BACKGROUND:

References to durometer hardness for elastomeric bearing design have been moved from Method B (14.7.5) to Method A (14.7.6) in the Design Specifications. This was done in conjunction with the re-inclusion of traditional testing methods for elastomeric bearings in the AASHTO Materials Specification M 251-06. Also, the testing criteria for elastomeric bearings in Section 18 of the Construction Specifications which conflicts with the new Materials Specification has been removed.

ANTICIPATED EFFECT ON BRIDGES:

Bearings for typical bridges should be more cost effective.

REFERENCES:

AASHTO Materials Specifications M 251-06, M251-04 and M 251-97

OTHER:

None

ATTACHMENT - 2006 AGENDA ITEM 1 - T-2

Item #3

14.7.5.2 Material Properties

The elastomer shall have a shear modulus between 0.080 and 0.175 ksi. ~~and a nominal hardness between 50 and 60 on the Shore A scale.~~ It shall conform to the requirements of Section 18.2 of the *AASHTO LRFD Bridge Construction Specifications* and the *AASHTO Materials Specification M 251*.

The shear modulus of the elastomer at 73°F shall be used as the basis for design. ~~If the elastomer is specified explicitly by its shear modulus, that value shall be used in design, and the o~~ Other properties, such as creep deflection, ~~shall~~ should be obtained from Table 14.7.6.2-1 ~~of Article 14.7.6.2~~ or from tests conducted using the *AASHTO Materials Specification M 251.* ~~If the material is specified by its hardness, the shear modulus shall be taken as the least favorable value from the range for that hardness given in Table 1. Intermediate values may be obtained by interpolation.~~

C14.7.5.2

Materials with a ~~nominal hardness greater than 60~~ shear modulus greater than 0.175 ksi are prohibited because they generally have a smaller elongation at break and greater stiffness and greater creep than their softer counterparts. This inferior performance is generally attributed to the larger amounts of filler present. Their fatigue behavior does not differ in a clearly discernible way from that of softer materials.

Shear modulus, *G*, is the most important material property for design, and it is, therefore, the primary ~~preferred~~ means of specifying the elastomer. Hardness has been widely used in the past, and is still permitted for Method A design, because the test for it is quick and simple. However, the results obtained from it are variable and correlate only loosely with shear modulus. ~~The ranges given in Table 1 represent the variations found in practice. Table 1 goes up to 70 hardness because the table is also referenced in the provisions for plain elastomeric pads. If the material is specified by hardness, a safe and presumably different estimate of *G* should be taken for each of the design calculations, depending on whether the parameter being calculated is conservatively estimated by over- or under-estimating the shear modulus. Specifying the material by hardness thus imposes a slight penalty in design.~~

~~Creep varies from one compound to another and is generally more prevalent in harder elastomers but is seldom a problem if high-quality materials are used. This is particularly true because the deflection limits are based on serviceability and are likely to be controlled by live load, rather than total load. The creep values given in Table 1 are representative of neoprene and are conservative for natural rubber.~~

~~Table 14.7.5.2-1 Shear Modulus, *G*.~~

	~~Hardness (Shore A)~~		
	~~50~~	~~60~~	~~70~~
~~Shear Modulus @ 73°F (ksi)~~	~~0.095–0.130~~	~~0.130–0.200~~	~~0.200–0.300~~
~~Creep deflection @ 25 years divided by instantaneous deflection~~	~~0.25~~	~~0.35~~	~~0.45~~

For the purposes of bearing design, all bridge sites shall be classified as being in temperature Zones A, B, C, D, or E for which design data are given in Table ~~2~~ 1. In the absence of more precise information, Figure 1 may be used as a guide in selecting the zone required for a given region.

Bearings shall be made from AASHTO low-

The zones are defined by their extreme low temperatures or the largest number of consecutive days when the temperature does not rise above 32°F, whichever gives the more severe condition.

Shear modulus increases as the elastomer cools, but the extent of stiffening depends on the elastomer temperature grades of elastomer as defined in Section 18 of the *AASHTO LRFD Bridge Construction Specifications* and the *AASHTO Materials Specification M 251*. The minimum grade of elastomer required for each low-temperature zone shall be taken as specified in Table ~~2~~ 1.

Any of the three design options listed below may be used:

- Specify the elastomer with the minimum low-temperature grade indicated in Table ~~2~~ 1 and determine the shear force transmitted by the bearing as specified in Article 14.6.3.1;

- Specify the elastomer with the minimum low-temperature grade for use when special force provisions are incorporated in the design and provide a low friction sliding surface, in which case the bridge shall be designed to withstand twice the design shear force specified in Article 14.6.3.1; or

- Specify the elastomer with the minimum low-temperature grade for use when special force provisions are incorporated in the design but do not provide a low friction sliding surface, in which case the components of the bridge shall be designed to resist four times the design shear force as specified in Article 14.6.3.1.

compound, time, and temperature. It is, therefore, important to specify a material with low-temperature properties that are appropriate for the bridge site. In order of preference, the low-temperature classification should be based on:

- The 50-year temperature history at the site,

- A statistical analysis of a shorter temperature history, or

- Figure 1.

Table ~~2~~ 1 gives the minimum elastomer grade to be used in each zone. A grade suitable for a lower-temperature may be specified by the Engineer, but improvements in low-temperature performance can often be obtained only at the cost of reductions in other properties. This low-temperature classification is intended to limit the force on the bridge substructure to 1.5 times the service limit state design force under extreme environmental conditions.

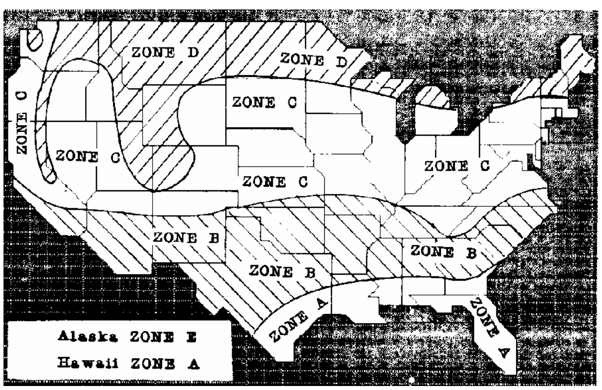


Figure 14.7.5.2-1 Temperature Zones.

Table 14.7.5.2-~~2~~ -1 Low-Temperature Zones and Minimum Grades of Elastomer.

Low-Temperature Zone	A	B	C	D	E
50-Year low temperature (°F)	0	−20	−30	−45	< −45
Maximum number of consecutive days when the temperature does not rise above 32°F	3	7	14	N/A	N/A
Minimum low-temperature elastomer grade	0	2	3	4	5
Minimum low-temperature elastomer grade when special force provisions are incorporated	0	0	2	3	5

14.7.5.3 Design Requirements

14.7.5.3.1 Scope

Bearings designed by the provisions herein shall be tested in accordance with the requirements for steel-reinforced elastomeric bearings as specified in Article 18.2~~.7~~ of the *AASHTO LRFD Bridge Construction Specifications* and the *AASHTO Materials Specification M 251.*

C14.7.5.3.1

Steel-reinforced bearings are designed to resist relatively high stresses. Their integrity depends on good quality control during manufacture, which can only be ensured by rigorous testing.

14.7.5.3.2 Compressive Stress

In any elastomeric bearing layer, the average compressive stress at the service limit state shall satisfy:

- For bearings subject to shear deformation:

$$\sigma_s \leq 1.66GS \leq 1.6\,\text{ksi} \quad (14.7.5.3.2\text{-}1)$$

$$\sigma_L \leq 0.66GS \quad (14.7.5.3.2\text{-}2)$$

- For bearings fixed against shear deformation:

$$\sigma_s \leq 2.00GS \leq 1.75\,\text{ksi} \quad (14.7.5.3.2\text{-}3)$$

$$\sigma_L \leq 1.00GS \quad (14.7.5.3.2\text{-}4)$$

where:

σ_s = service average compressive stress due to the total load (ksi)

σ_L = service average compressive stress due to live load (ksi)

G = shear modulus of elastomer (ksi)

S = shape factor of the thickest layer of the bearing

C14.7.5.3.2

These provisions limit the shear stress and strain in the elastomer. The relationship between the shear stress and the applied compressive load depends directly on shape factor, with higher shape factors leading to higher capacities. If movements are accommodated by shear deformations of the elastomer, they cause shear stresses in the elastomer. These add to the shear stresses caused by compressive load, so a lower load limit is specified.

The compressive limits, in terms of *GS*, were derived from static and fatigue tests correlated with theory (*Roeder and Stanton 1986; Roeder et al. 1990*). There was large scatter in the stress at which delamination started in different fatigue and static tests. The absolute limits of 1.6 and 1.75 ksi came from the static tests. Delamination is a service limit state, but it may lead to more serious structural problems. The specified stress limits provide a safety factor of approximately 1.5 against initial delamination. However, long-term loading was not investigated in the test program, although it is known to be more detrimental to the bond, so the real safety factor against initiation of debonding may be somewhat less than 1.5.

The compressive stress limits, in terms of *GS*, were derived from fatigue tests and are based on the observation that fatigue cracking in the experiments remained acceptably low if the maximum shear strain due to total dead and live load was kept below 3.0, and the maximum shear strain range for cyclic loading was kept below 1.5. The level of damage considered acceptable had to be selected arbitrarily; therefore, the limits are not absolute.

Increases in the load to simulate the effects of impact are not required. This is because the impact stresses are likely to be only a small proportion of the total load and because the stress limits are based on fatigue damage, the limits of which are not clearly defined.

Two limits are given, one for total load and one for live load, and the most restrictive one will control.

14.7.5.3.3 Compressive Deflection

Deflections of elastomeric bearings due to ~~total~~ dead load and to instantaneous live load alone shall be considered separately.

Instantaneous live load deflection shall be taken as:

C14.7.5.3.3

Limiting instantaneous live load deflections is important to ensure that deck joints and seals are not damaged. Furthermore, bearings that are too flexible in compression could cause a small step in the road surface at a deck joint when traffic passes from one girder to the

$$\delta_L = \sum \varepsilon_{Li} h_{ri} \qquad (14.7.5.3.3\text{-}1)$$

where:

ε_{Li} = instantaneous live load compressive strain in i^{th} elastomer layer of a laminated bearing

h_{ri} = thickness of i^{th} elastomeric layer in a laminated bearing (in.)

Initial dead load deflection shall be taken as:

$$\delta_d = \sum \varepsilon_{di} h_{ri} \qquad (14.7.5.3.3\text{-}2)$$

where:

ε_{di} = initial dead load compressive strain in i^{th} elastomer layer of a laminated bearing

h_{ri} = thickness of i^{th} elastomeric layer in a laminated bearing (in.)

Long-term dead load deflection, including the effects of creep, shall be taken as:

$$\delta_{lt} = \delta_d + a_{cr}\delta_d \qquad (14.7.5.3.3\text{-}3)$$

where:

a_{cr} = creep deflection divided by initial dead load deflection (dim.)

Values for ε_{Li} and ε_{di} shall be determined from test results or by analysis. ~~when considering long-term deflections. The effects of creep of the elastomer shall be added to the instantaneous deflection.~~ Creep effects should be determined from information relevant to the elastomeric compound used. If the engineer does not elect to obtain a value for the ratio, a_{cr}, from test results using Annex A2 of the *AASHTO Materials Specification M 251*, ~~In the absence of material-specific data,~~ the values given in Table 1 Article 14.7.~~5~~ 6.2 may be used.

other, giving rise to impact loading. A maximum relative live load deflection across a joint of 0.125 in. is suggested. Joints and seals that are sensitive to relative deflections may require limits that are tighter than this.

Long-term dead load deflections should be considered where joints and seals between sections of the bridge rest on bearings of different design and when estimating redistribution of forces in continuous bridges caused by settlement. ~~Provided high-quality materials are used, the effects of creep are unlikely to cause problems.~~

Laminated elastomeric bearings have a nonlinear load deflection curve in compression. In the absence of information specific to the particular elastomer to be used, Equation C1 or Figure C1 in Article C14.7.6.3.3 may be used as a guide for calculating dead and live load compressive strains for Equations 1 and 2.

$$\varepsilon = \frac{\sigma}{6GS^2} \qquad (C14.7.5.3.3\text{-}1)$$

where:

σ = instantaneous live load compressive stress or dead load compressive stress in an individual elastomer layer of a laminated bearing (ksi)

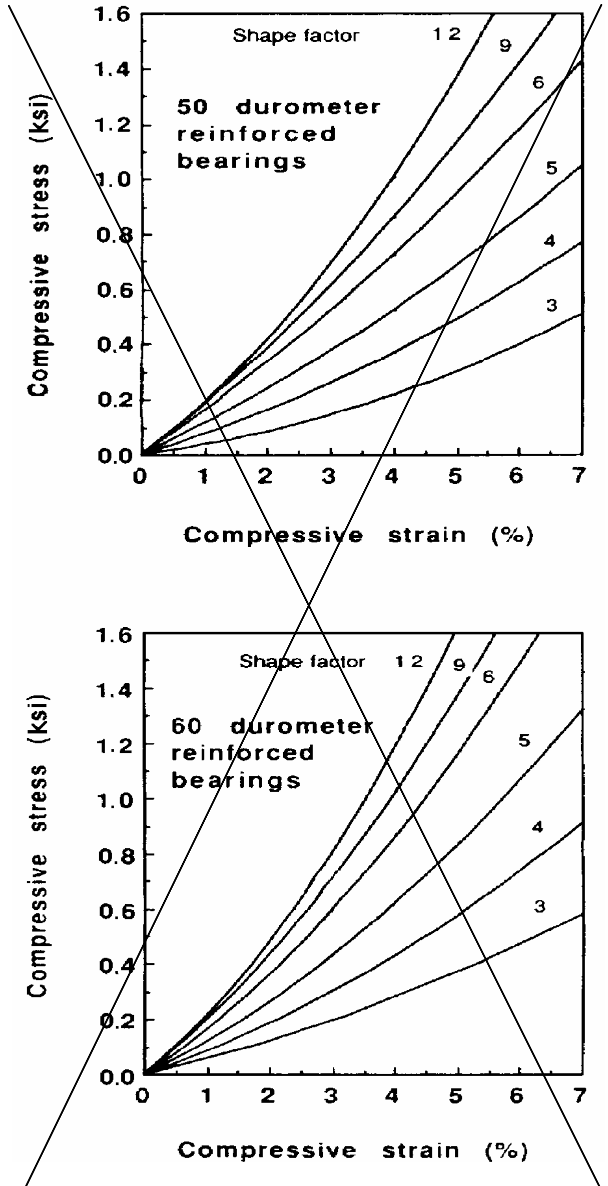
S = shape factor of an individual elastomer layer

G = shear modulus of the elastomer (ksi)

Equation C1 or Figure C1 in Article C14.7.6.3.3 may also be used as a guide for specifying an allowable value of compressive strain at the design dead plus live service compressive load when employing Section 8.8.1 of the *AASHTO Material Specification M 251*.

Guidance for specifying an allowable value for creep when Annex A2 of the *AASHTO Materials Specification M 251* is employed may be obtained from NCHRP Report 449 or from Table 1 in Article 14.7.6.2.

Reliable test data on total deflections are rare because of the difficulties in defining the true 0.0 for deflection. However, the change in deflection due to live load can be reliably predicted either by design aids based on test results or by using theoretically based equations (*Stanton and Roeder 1982*). In the latter case, it is important to include the effects of bulk compressibility of the elastomer, especially for high-shape factor bearings.



~~**Figure C14.7.5.3.3-1 Stress-Strain Curves.**~~

14.7.5.3.7 Reinforcement

The thickness of the steel reinforcement, h_s, shall satisfy ~~the provisions of Article 14.7.5.3.7 of the *AASHTO LRFD Bridge Construction Specifications*, and~~:

- At the service limit state:

$$h_s \geq \frac{3h_{max}\sigma_s}{F_y} \qquad (14.7.5.3.7\text{-}1)$$

- At the fatigue limit state:

$$\cancel{h_s \geq \frac{2.0h_{max}\sigma_L}{\Delta F_{TH}}}$$

C14.7.5.3.7

The reinforcement should sustain the tensile stresses induced by compression of the bearing. With the present load limitations, the minimum steel plate thickness practical for fabrication will usually provide adequate strength.

If the thickness of the steel reinforcement, h_s, is not specified by the engineer, the minimum thickness shall be .0625 in. according to Article 4.5 of the *AASHTO Materials Specification M 251.*

$$h_s \geq \frac{2h_{\max}\sigma_L}{\Delta F_{TH}} \qquad (14.7.5.3.7\text{-}2)$$

where:

ΔF_{TH} = constant amplitude fatigue threshold for Category A as specified in Article 6.6 (ksi)

h_{max} = thickness of thickest elastomeric layer in elastomeric bearing (in.)

σ_L = service average compressive stress due to live load (ksi)

σ_s = service average compressive stress due to total load (ksi)

F_y = yield strength of steel reinforcement (ksi)

If holes exist in the reinforcement, the minimum thickness shall be increased by a factor equal to twice the gross width divided by the net width.

Holes in the reinforcement cause stress concentrations. Their use should be discouraged. The required increase in steel thickness accounts for both the material removed and the stress concentrations around the hole.

Item #4

14.7.6.2 Material Properties

The elastomeric-type materials shall satisfy the requirements of Article 14.7.5.2~~,~~. In addition, hardness on the Shore A scale may be used as a basis for specification of bearing material. The shear modulus shall be between 0.080 ksi and 0.250 ksi, and the nominal hardness shall be between 50 and 70 on the Shore A scale and shall conform to the requirements of Section 18.2 of the *AASHTO LRFD Bridge Construction Specifications* and the *AASHTO Materials Specification M 251*. If the material is specified by its hardness, the shear modulus shall be taken as the least favorable value from the range for that hardness given in Table 1. Intermediate values may be obtained by interpolation. Other properties, such as creep deflection, are also given in Table 1. ~~This~~ There is an exception ~~shall not apply to~~ for steel-reinforced elastomeric bearings designed in accordance with the provisions of this section. The elastomer shall have a shear modulus between 0.080 and 0.175 ksi. and a nominal hardness between 50 and 60 on the Shore A scale.

The shear force on the structure induced by deformation of the elastomer in PEP, FGP and steel-reinforced elastomeric bearings shall be based on a *G* value not less than that of the elastomer at 73°F. Effects of relaxation shall be ignored.

The finished CDP shall have a nominal hardness between 85 and 95 on the Shore A scale. The cotton-duck reinforcement shall be either a two-ply cotton yarn or a single-ply 50-50 blend cotton-polyester. The fabric shall

C14.7.6.2

The elastomer requirements for PEP and FGP are the same as those required for steel-reinforced elastomeric bearings. The ranges given in Table 1 represent the variations found in practice. If the material is specified by hardness, a safe and presumably different estimate of *G* should be taken for each of the design calculations, depending on whether the parameter being calculated is conservatively estimated by over- or under-estimating the shear modulus. Specifying the material by hardness thus imposes a slight penalty in design. Creep varies from one compound to another and is generally more prevalent in harder elastomers or those with a higher shear modulus but is seldom a problem if high-quality materials are used. This is particularly true because the deflection limits are based on serviceability and are likely to be controlled by live load, rather than total load. The creep values given in Table 1 are representative of neoprene and are conservative for natural rubber.

CDP is made of elastomers with hardness and properties similar to that used for PEP and FGP. However, the closely spaced layers of duck fabric reduce the indentation and increase the hardness of the finished pad to have a minimum tensile strength of 150 lb./in. width when tested by the grab method. The fill shall be 40 ± 2 threads per inch, and the warp shall be 50 ± 1 threads per inch.

the 85 to 95 durometer range. The cotton-duck requirements are restated from the military specification because the reinforcement is essential to the good performance of these pads.

Table ~~14.7.5.2-1 Shear Modulus, *G*.~~ 14.7.6.2-1 Correlated Material Properties

	Hardness (Shore A)		
	50	60	70[1]
Shear Modulus @ 73°F (ksi)	0.095–0.130	0.130–0.200	0.200–0.300
Creep deflection @ 25 years divided by ~~instantaneous~~ initial deflection	0.25	0.35	0.45

[1] For PEP and FGP only.

14.7.6.3 Design Requirements

14.7.6.3.1 Scope

Steel-reinforced elastomeric bearings may be designed in accordance with this article, in which case they qualify for the test requirements appropriate for elastomeric pads. For this purpose, they shall be treated as FGP.

The provisions for FGP apply only to pads where the fiberglass is placed in double layers 0.125 in. apart.

The physical properties of neoprene and natural rubber used in these bearings shall conform to the *AASHTO Materials Specification M 251.* ~~following ASTM or AASHTO requirements, with modifications as noted:~~

~~Compound~~	~~ASTM Requirement~~	~~AASHTO Requirement~~
~~Neoprene~~	~~D 4014~~	~~M 251~~
~~Natural Rubber~~	~~D 4014~~	~~M 251~~

~~Modifications:~~

- ~~The Shore A Durometer hardness shall be 50 ± 10 points, and~~
- ~~Samples for compression set tests shall be prepared using a Type 2 die.~~

C14.7.6.3.1

The design methods for elastomeric pads are simpler and more conservative than those for steel-reinforced bearings, so the test methods are less stringent than those of Article 14.7.5. Steel-reinforced elastomeric bearings may be made eligible for these less stringent testing procedures by limiting the compressive stress as specified in Article 14.7.6.3.2.

The three types of pad, PEP, FGP, and CDP behave differently, so information relevant to the particular type of pad should be used for design. For example, in PEP, slip at the interface between the elastomer and the material on which it is seated or loaded is dependent on the friction coefficient, and this will be different for pads seated on concrete, steel, grout, epoxy, etc.

14.7.6.3.2 Compressive Stress

At the service limit state, the average compressive stress, σ_s, in any layer shall satisfy:

- For PEP:

$$\sigma_s \leq 0.80\,\text{ksi} \qquad (14.7.6.3.2\text{-}1)$$

- For FGP:

C14.7.6.3.2

In CDP, the pad stiffness and behavior is less sensitive to shape factor. The 1500 psi stress limit is approximately 15 percent of the maximum compressive load that can be consistently achieved with these pads. However, the average compressive strain at this service limit state stress limit is in the range of 0.08 to 0.15 in./in. These compressive strains are somewhat larger than those tolerated with steel-reinforced elastomeric bearings, and the strain limit provides a rational reason for limiting stress to this level. Larger compressive strains may result in

$$\sigma_s \leq 1.00GS \leq 0.80\,\text{ksi} \qquad (14.7.6.3.2\text{-}2)$$

- For CDP:

$$\sigma_s \leq 1.50\,\text{ksi} \qquad (14.7.6.3.2\text{-}3)$$

In FGP, the value of S used shall be that for the greatest distance between the midpoint of double reinforcement layers at the top and bottom of the elastomer layer.

For steel-reinforced elastomeric bearings designed in accordance with the provisions of this article:

$$\sigma_s \leq 1.0\,\text{ksi and } \sigma_s \leq 1.0GS \qquad (14.7.6.3.2\text{-}4)$$

where the value of S used shall be that for the thickest layer of the bearing.

These stress limits may be increased by 10 percent where shear deformation is prevented.

increased damage to the bridge and the bearing pad and reduced serviceability of the CDP.

The reduced stress limit for steel-reinforced elastomeric bearings designed in accordance with these provisions is invoked in order to allow these bearings to be eligible for the less stringent test requirements for elastomeric pads.

14.7.6.3.3 Compressive Deflection

In addition to the provisions of Article 14.7.5.3.3, the following shall also apply.

In lieu of using specific product data, the compressive deflection of a FGP should be taken as 1.5 times the deflection estimated for steel-reinforced bearings of the same shape factor in Article 14.7.5.3.3.

The maximum compressive deflection for CDP shall be computed based upon an average compressive strain of (σ_s/10000) in psi and in./in. units.

The initial compressive deflection of a PEP or in any layer of a steel-reinforced elastomeric bearing at the service limit state without impact shall not exceed $0.07h_{ri}$.

C14.7.6.3.3

The compressive deflection with PEP, FGP, and CDP will be larger and more variable than those of steel-reinforced elastomeric bearings. Appropriate data for these pad types may be used to estimate their deflections. In the absence of such data, the compressive deflection of a PEP and FGP may be estimated at 3 and 1.5 times, respectively, the deflection estimated for steel-reinforced bearings of the same shape factor in Article 14.7.5.3.3.

CDP is typically very stiff in compression. The shape factor may be computed, but it has a different meaning and less significance to the compressive deflection than it does for FGP and PEP (*Roeder et al. 2000*).

<u>Figure C1 provides design aids for determining the strain in an elastomer layer for steel reinforced bearings based upon durometer hardness and shape factor.</u>

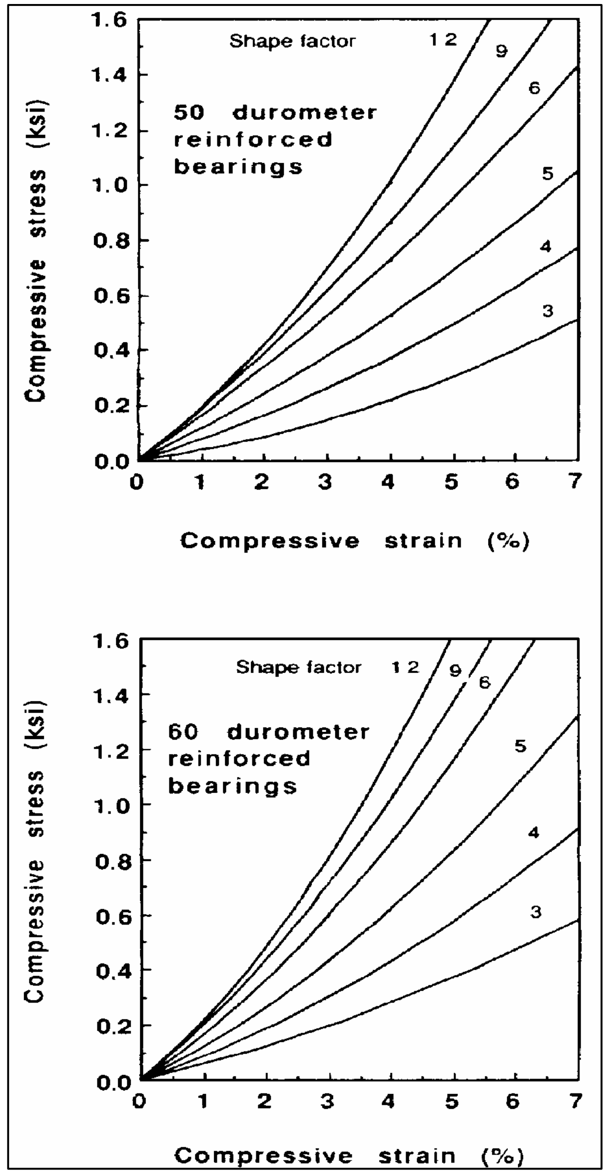


Figure C14.7.6.3.3-1 Stress-Strain Curves.

2006 AASHTO BRIDGE COMMITTEE AGENDA ITEM: 2

SUBJECT: LRFD Bridge Construction Specifications: Section 18, Articles 18.1 and 18.2

TECHNICAL COMMITTEE: T-2 Bearings / T-4 Construction

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

☐ **DESIGN SPEC** ☒ **CONSTRUCTION SPEC** ☐ **MOVABLE SPEC**
☐ **LRFR MANUAL** ☐ **OTHER**

☐ **US VERSION** ☐ **SI VERSION** ☒ **BOTH**

DATE PREPARED: 12/16/05
DATE REVISED:

AGENDA ITEM:

Revise Articles 18.1.2.2, C18.1.2.2, 18.1.4.2, C18.1.4.2, 18.2.3, C18.2.3, 18.2.4, C18.2.4, 18.2.5, C18.2.5, Appendix Table 18.1.4.2-1, Appendix Table 18.2.3.1-1, and Appendix Table 18.2.3.1-2 as shown in the attached pages.

OTHER AFFECTED ARTICLES:

None

BACKGROUND:

References to durometer hardness for elastomeric bearing design have been moved from Method B (14.7.5) to Method A (14.7.6) in the Design Specifications. This was done in conjunction with the re-inclusion of traditional testing methods for elastomeric bearings in the AASHTO Materials Specification M 251-06. Also, the testing criteria for elastomeric bearings in Section 18 of the Construction Specifications which conflicts with the new Materials Specification has been removed.

ANTICIPATED EFFECT ON BRIDGES:

Bearings for typical bridges should be more cost effective.

REFERENCES:

AASHTO Materials Specifications M 251-06, M251-04 and M 251-97

OTHER:

None

18.1.2.2 Steel Laminates

Steel laminates shall meet the requirements of the *AASHTO Materials Specification M 251*.

~~Unless otherwise specified in the contract documents, steel laminates used for reinforcement shall be made from rolled mild steel conforming to AASHTO M 270M/M 270 (ASTM A 709/A 709M), Grade 36 (Grade 250), ASTM A 1011/A 1011M, or equivalent. Laminates shall have a minimum nominal thickness of 16 gage (1.6 mm). Holes in plates for manufacturing purposes shall not be permitted unless they have been accounted for in the design and are in accordance with the contract documents.~~

~~C18.1.2.2~~

~~It is intended that a mild steel of well-defined ASTM Standard be used. However, no single suitable ASTM Specification is available which includes thickness both less than and greater than 0.125 in. (3.2 mm). For thin plate, SAE C1020 (*SAE Handbook*, 2004) is frequently used. This is a relatively ductile steel for cold-formed metal structures which has no specified yield strength but usually has a yield strength of approximately 33.0 ksi (230 MPa).~~

~~The minimum thickness of laminates is intended to ensure that the steel will not deform excessively during sandblasting or molding of the bearing. Since the minimum thickness of the steel is often governed by fabrication criteria, little is to be gained by using a higher-strength steel than necessary. Holes are not allowed in the steel plate unless they have been accounted for in design because the steel is in tension and would be weakened by holes. For large bearings, thickness greater than 16 gage (1.6 mm) may be needed for this purpose, and individual Manufacturers should be consulted. In many cases, use of the minimum thickness will ensure satisfaction of Article 14.7, “Special Design Provisions for Bearings,” of the AASHTO LRFD Bridge Design Specifications.~~

18.1.4.2 Fabrication Tolerances

Plain elastomeric pads and laminated bearings shall be built as specified in the contract documents within the tolerances of the *AASHTO Materials Specification M 251.*

Other fabrication tolerances are given in Table 18.1.4.2-1.

~~Plain elastomeric pads and laminated bearings shall be built as specified in the contract documents within the tolerances of Table 18.1.4.2-1. The classes of tolerances shall be as follows:~~

- ~~Class A = 0.001 × nominal dimensions~~
- ~~Class B = 0.002 × nominal dimensions~~
- ~~Class C = 0.005 × nominal dimensions~~

~~Load plate overall dimensions for flatness tolerance and surface finish shall apply only to surfaces in contact with the bearing.~~

~~C18.1.4.2~~

~~Some of the tolerances have been changed to relative values because an absolute tolerance, such as 0.0625 in. (1.5 mm), may be overly large for a small bearing and unrealistically small for a large bearing. Parallelism of the two faces of a single layer is controlled by the limitation of the thickness at any point.~~

~~Each bearing type has one or more tolerances which are particularly important. In bearings which depend on rocking or rolling surfaces, it is most important to ensure that the curvature of the curved surface is constant to within a fine tolerance. This is more important than the actual value of the radius of curvature. In nested roller bearings, it is also important that all the rollers have exactly the same radius of curvature to ensure that the load will be equally shared among them. In flat PTFE sliding surfaces, the surface finish of the mating surface, usually stainless steel, is particularly important. A #8 mirror (0.2 μm) finish or better is recommended in~~

~~all cases.~~

~~In bearings which depend on the sliding of one curved surface over another, such as curved PTFE sliding bearings, curved bronze sliding bearings, or pins and bushings which allow rotation, the difference in diameter of the two curved surfaces is the most important tolerance. The out-of-round or the variation in curvature of the curved surface is also important, and again the actual value of the radius of curvature is less important. If two parts of the bearing are made by different Fabricators, machining by fitting the two parts is not possible and it is necessary to machine each part to a specific radius within a very high accuracy. In the past, bearings made of components which are fabricated by different Manufacturers have given problems because of lack of a good fit. In pot bearings, the most important tolerances are those on the clearance between the pot and the piston and on the vertical clearance between the upper and lower parts of the bearing.~~

Table 18.1.4.2-1 Fabrication Tolerances.

Item		Thickness Tolerance, in.	Dimension Tolerance, in.	Flatness or Out-of-Round Tolerance, in.	Surface Finish, μin.(rms)
Metal Rocker & Roller Bearings		—			
	Single Roller: Diameter	—	–0.063, +0.063	-0.001, +0.001	63
	Nested Roller: Diameter	—	–0.002, +0.002	–0.001, +0.001	63
	Rockers: Diameter	—	–0.125, +0.125	–0.001, +0.001	125
	Pins: Diameter	—	–0.005, +0.000	–0.002, +0.002	32
	Bushings: Diameter	—	–0.000, +0.005	–0.002, +0.002	32
Pot Bearings					
	Overall Dimensions	–0.000, +0.250	–0.000, +0.125	—	—
	Pot Depth (Inside)	—	–0.000, +0.025	—	—
	Pot Wall: Thickness and Average Inside Diameter	–0.000, +0.125	–0.003, +0.003	–0.001, +0.001	32
	Pot Base: Top and Bottom Surfaces	–0.000, +0.025	—	Class C	63
	Piston: Rim	–0.000, +0.063	–0.003, +0.003	–0.001, +0.001	32
	Piston: Top and Bottom Surfaces	–0.000, +0.025	—	Class C	63
	Elastomeric Disc (Unstressed)	–0.000, +0.125	–0.000, +0.063	—	—
Disc Bearings					
	Overall Dimensions	–0.000, +0.250	–0.000, +0.125	—	—
	Shear-Restricting Element	—	–0.000, +0.005	Class A	32
	Other Machined Parts	–0.000, +0.063	–0.000, +0.063	Class B	63
	Urethane Disc	–0.000, +0.063	–0.000, +0.125	Class B	63
Flat PTFE Sliding Bearings					
	PTFE	–0.000, +0.063	–0.000, +0.030	Class A	—
	Stainless Steel	–0.000, +0.063	–0.000, +0.125	Class A	#8 Mirror
Flat Bronze and Copper Alloy Sliding Bearings					
	Sliding Surfaces	–0.000, +0.125	–0.000, +0.125	Class A	32
Curved PTFE Sliding Bearings					
	Convex Radius	—	–0.010, +0.000	–0.002, +0.002	#8 Mirror
	Concave Radius	—	–0.000, +0.010	–0.002, +0.002	125
Curved Bronze and Copper Alloy Sliding Bearings					
	Convex Radius	—	–0.010, +0.000	–0.002, +0.002	32
	Concave Radius	—	–0.000, +0.010	–0.002, +0.002	32
~~Steel-Reinforced Elastomeric Bearings~~					
	~~Overall Dimensions~~	~~–0.000, +0.250~~	~~–0.000, +0.250~~	—	—
	~~Internal Rubber Layers~~	~~–0.125, +0.125 and ±0.20*design~~	—	—	—
	~~Cover~~	~~–0.000, +0.125~~	—	—	—
	~~Parallelism: Top and Bottom Surfaces~~	~~±0.005 rad~~	—	—	—
	~~Parallelism: Sides~~	—	~~±0.020 rad~~	—	—
~~Elastomeric Pads~~					
	~~Overall Dimensions~~	~~–0.000, +0.125~~	~~–0.000, +0.250~~	—	—
Guides					
	Contact Surface	—	–0.000, +0.125	Class A	32
	Distance between Guides	—	–0.000, +0.030	—	—
	Parallelism of Guides	—	±0.005 rad	—	—
Load Plates					
	Overall Dimensions	–0.063, +0.063	–0.250, +0.250	Class A	125
	Bevel Slope	±0.002 rad	—	—	—

18.2 ELASTOMERIC BEARINGS

18.2.1 Scope

Elastomeric bearings as herein defined shall include unreinforced pads (consisting of elastomer only) and reinforced bearings with steel or fabric laminates.

18.2.2 General Requirements

Bearings shall be furnished with the dimensions, material properties, elastomer grade, and type of laminates specified in the contract documents. The design load shall be shown in the contract documents and testing shall be performed accordingly. Unless otherwise specified in the contract documents, bearings shall be Grade 3, 60-durometer elastomer, and steel reinforced, and shall be subjected to the load-testing requirements specified herein.

18.2.3 Materials

~~18.2.3.1 Properties of the Elastomer~~

The raw elastomer shall be either virgin Neoprene (polychloroprene) or virgin natural rubber (polyisoprene). The elastomer compound shall be classified as being of low-temperature Grade 0, 2, 3, 4, or 5. The grades and other material properties are defined in the *AASHTO LRFD Bridge Design Specifications Section 14* and the *AASHTO Materials Specification M 251.* ~~by the testing requirements in Tables 18.2.3.1-1 and 18.2.3.1-2.~~ A higher grade of elastomer may be substituted for a lower one.

~~C18.2.3.1~~ C18.2.3

At present, only natural rubber (polyisoprene) and Neoprene (polychloroprene) are permitted. This is because both have an extensive history of satisfactory use. In addition, much more field experience exists with these two materials than with any other and almost all of it is satisfactory.

The low-temperature grading system addresses the problem of stiffening of the elastomer at low temperatures. Special compounding and curing are needed to avoid the problem, but they increase cost and, in extreme cases, may adversely affect some other properties. These adverse effects can be minimized by choosing a grade of elastomer appropriate for the conditions prevailing at the site. The grades follow the approach of AASHTO M 251 ~~or ASTM D 4014~~, with some stringent low-temperature test criteria for higher grades.

~~Tables 18.2.3.1-1 and 18.2.3.1-2 outline the required properties of the elastomer. The standards are sometimes different for Neoprene and natural rubber, which appears inconsistent because in some ways the requirements resemble a performance specification. However, the present state of knowledge is inadequate to precisely quantify those material properties needed to assure good bearing behavior so the tests are intended to ensure good quality material. Natural rubber and Neoprene have different strengths and weaknesses so different tests are indeed appropriate. Generally, natural rubber creeps less, suffers less low-temperature stiffening, and has a better elongation at break but Neoprene has better chemical, ozone, and aging resistance.~~

~~The previous low-temperature brittleness test has~~

~~been augmented by two other tests: the Clash-Berg test for low-temperature stiffness (ASTM D 1043) and a test for low-temperature crystallization stiffening (the quad shear test conducted at low temperature). All three tests are required for elastomers of Grade 3 and above. Previously, the brittleness test at –40°F (–40°C) was required for all elastomers, including those to be used in the southern tier states; yet, no test was required for thermal or crystallization stiffening, even in the northern tier states or Alaska.~~

~~The brittleness test essentially detects glass transition but gives no indication of stiffening. The Clash-Berg test is introduced to detect instantaneous low-temperature stiffening. It is quick to perform and requires only a modest investment in special equipment. Crystallization stiffening is both time- and temperature-dependent but constitutes a significant portion of the total low-temperature stiffening of many elastomers. Detecting it is, therefore, important and is done by the long-duration shear stiffness test. In addition to the quad shear apparatus, this test requires a freezer that surrounds the apparatus. Because of the nature of the crystallization, the test may take up to 28 days; therefore, it is not required for every lot of bearings.~~

~~Hardness is maintained as a material property because it is widely used in rubber technology and is easy to measure. However, measurements are sensitive to the method used, and hardness generally gives only rough indication of the mechanical properties, particularly at low temperatures. The shear modulus is a much more useful property, but is more time-consuming to measure.~~

~~Harder elastomers have a greater shear stiffness and thus exert larger pier forces due to thermal expansion than materials of low hardness, unless the plan area of the bearing is reduced proportionately. This could cause the bearing to be rather slender, possibly leading to instability problems. Further, 70-durometer material generally creeps more than its softer counterparts. Thus, when larger compressive stiffness is required, it is recommended that reinforced bearings of softer elastomer with thinner layers and higher shape factor be used.~~

~~The elastomer compound shall meet the minimum requirements of Tables 18.2.3.1-1 and 18.2.3.1-2, except as otherwise specified by the Engineer. Test requirements may be interpolated for intermediate hardness. If the material is specified by its shear modulus, its measured shear modulus shall lie within 15 percent of the specified value. A consistent value of hardness shall also be supplied for the purpose of defining limits for the tests in Tables 18.2.3.1-1 and 18.2.3.1-2. If the hardness is specified, the measured shear modulus must fall within the range of Table 14.7.5.2-1 of the AASHTO LRFD Bridge Design Specifications. When test specimens are cut from the finished product, the physical properties shall be~~

~~Table 14.7.5.2-1, Shear Modulus, is located in "Material Properties" in the AASHTO LRFD Bridge Design Specifications, 2004.~~

~~In Tables 18.2.3.1-1 and 18.2.3.1-2, ASTM D 1043 refers to the "modulus of rigidity," while AASHTO M 251 or ASTM D 4014 refers to "shear modulus stiffness." The word "stiffness" is used herein to cover both terms.~~

~~permitted to vary from those specified in Tables 18.2.3.1-1 and 18.2.3.1-2 by ten percent. All material tests shall be carried out at 73°F ± 4°F (23°C ± 2°C), unless otherwise noted. Shear modulus tests shall be carried out using the apparatus and procedure described in Annex A of ASTM D 4014.~~

~~Table 18.2.3.1-1 Polychloroprene (Neoprene) Quality-Control Tests.~~

~~PHYSICAL PROPERTIES~~				
~~D 2240~~	~~Hardness (Shore A Durometer)~~	~~50 ± 5~~	~~60 ± 5~~	~~70 ± 5~~
~~D 412~~	~~Tensile Strength, minimum ksi~~	~~2.25~~	~~2.25)~~	~~2.25~~
	~~Ultimate Elongation, minimum %~~	~~400~~	~~350~~	~~300~~
~~HEAT RESISTANCE~~				
~~D 573, 70 h at 212°F (100°C)~~	~~Change in Durometer Hardness, maximum points~~	~~15~~	~~15~~	~~15~~
	~~Change in Tensile Strength, maximum %~~	~~–15~~	~~–15~~	~~–15~~
	~~Change in Ultimate Elongation, maximum %~~	~~–40~~	~~–40~~	~~–40~~
~~COMPRESSION SET~~				
~~D 395, Method B~~	~~22 h at 212°F, maximum %~~	~~35~~	~~35~~	~~35~~
~~OZONE~~				
~~D 1149~~	~~100 pphm Ozone in Air by Volume, 20% Strain 100° F ± 2° F 100 h Mounting Procedure D 518, Procedure A~~	~~No Cracks~~	~~No Cracks~~	~~No Cracks~~
~~LOW-TEMPERATURE BRITTLENESS~~				
~~D 746, Procedure B~~	~~Grades 0 and 2—No Test Required~~			
	~~Grade 3—Brittleness at –40°F~~	~~No Failure~~	~~No Failure~~	~~No Failure~~
	~~Grade 4—Brittleness at –55°F~~	~~No Failure~~	~~No Failure~~	~~No Failure~~
	~~Grade 5—Brittleness at –70°F~~	~~No Failure~~	~~No Failure~~	~~No Failure~~
~~INSTANTANEOUS THERMAL STIFFENING~~				
~~D 1043~~	~~Grades 0 and 2—Tested at –25°F~~	~~Stiffness at test temperature shall not exceed four times the stiffness measured at 73°F~~		
	~~Grade 3—Tested at –40°F~~			
	~~Grade 4—Tested at –50°F~~			
	~~Grade 5—Tested at –65°F~~			
~~LOW-TEMPERATURE CRYSTALLIZATION~~				
~~Quad Shear Test as Described in Annex A of ASTM D 4014~~	~~Grade 0—No Test Required~~	~~Stiffness at test time and temperature shall not exceed four times the stiffness measured 73°F with no time delay. The stiffness shall be measured with a quad shear test rig in an enclosed freezer unit. The test specimens shall be taken from a randomly selected bearing. A ±25% strain cycle shall be used and a complete cycle of strain shall be applied with a period of 100 s. The first 0.75 cycle of strain shall be discarded and the stiffness shall be determined by the slope of the force deflection curve for the next 0.50 cycle of loading.~~		
	~~Grade 2—7 days at 0°F~~			
	~~Grade 3—14 days at –15°F~~			
	~~Grade 4—21 days at –35°F~~			
	~~Grade 5—28 days at –35°F~~			

~~**Table 18.2.3.1-2 Polyisoprene (Natural Rubber) Quality-Control Tests.**~~

~~PHYSICAL PROPERTIES~~				
~~D 2240~~	~~Hardness (Shore A Durometer)~~	~~50 ± 5~~	~~60 ± 5~~	~~70 ± 5~~
~~D 412~~	~~Tensile Strength, minimum ksi~~	~~2.25~~	~~2.25~~	~~2.25~~
	~~Ultimate Elongation, minimum %~~	~~450~~	~~400~~	~~300~~
~~HEAT RESISTANCE~~				
~~D 573 70 h at 158°F~~	~~Change in Durometer Hardness, maximum points~~	~~10~~	~~10~~	~~10~~
	~~Change in Tensile Strength, maximum %~~	~~–25~~	~~–25~~	~~–25~~
	~~Change in Ultimate Elongation, maximum %~~	~~–25~~	~~–25~~	~~–25~~
~~COMPRESSION SET~~				
~~D 395 Method B~~	~~22 h at 158°F, maximum %~~	~~25~~	~~25~~	~~25~~
~~OZONE~~				
~~D 1149~~	~~25 pphm Ozone in Air by Volume, 20% Strain 100°F ± 2°F 48 h Mounting Procedure, D 518, Procedure A~~	~~No Cracks~~	~~No Cracks~~	~~No Cracks~~
~~LOW-TEMPERATURE BRITTLENESS~~				
~~D 746 Procedure B~~	~~Grades 0 and 2—No Test Required~~			
	~~Grade 3—Brittleness at –40°F~~	~~No Failure~~	~~No Failure~~	~~No Failure~~
	~~Grade 4—Brittleness at –55° F~~	~~No Failure~~	~~No Failure~~	~~No Failure~~
	~~Grade 5—Brittleness at –70° F~~	~~No Failure~~	~~No Failure~~	~~No Failure~~
~~INSTANTANEOUS THERMAL STIFFENING~~				
~~D 1043~~	~~Grades 0 and 2—Tested at –25°F~~	~~Stiffness at test temperature shall not exceed four times the stiffness measured at 73°F~~		
	~~Grade 3—Tested at –40°F~~			
	~~Grade 4—Tested at –50°F~~			
	~~Grade 5—Tested at –65°F~~			
~~LOW-TEMPERATURE CRYSTALLIZATION~~				
~~Quad Shear Test as Described in Annex A of ASTM D 4014~~	~~Grade 0—No Test Required~~	~~Stiffness at test time and temperature shall not exceed four times the stiffness measured 73°F with no time delay. The stiffness shall be measured with a quad shear test rig in an enclosed freezer unit. The test specimens shall be taken from a randomly selected bearing. A ±25% strain cycle shall be used, and a complete cycle of strain shall be applied with a period of 100 s. The first 0.75 cycle of strain shall be discarded and the stiffness shall be determined by the slope of the force deflection curve for the next 0.25 cycle of loading.~~		
	~~Grade 2—7 days at 0°F~~			
	~~Grade 3—14 days at –15°F~~			
	~~Grade 4—21 days at –35°F~~			
	~~Grade 5—28 days at –35°F~~			

~~**18.2.3.2 Fabric Reinforcement**~~

~~Fabric reinforcement shall be woven from 100 percent continuous glass fibers of "E" type yarn. The minimum thread count in either direction shall be 25 threads per in. (one thread per mm). The fabric shall have either a crowfoot or an 8 Hardness Satin weave. Each ply of fabric shall have a minimum breaking strength of 0.800 kip/in. (140 N/mm) of width in each thread direction. Unless otherwise specified in the contract documents, holes shall not be permitted in the fabric.~~

~~**C18.2.3.2**~~

~~Fiberglass is the only fabric proven to perform adequately as reinforcement and only one grade is currently permitted. Polyester has proved too flexible, and both it and cotton are not strong enough. The strength of the reinforcement governs the compressive strength of the bearing when minimum amounts are used; therefore, if stronger fabric with acceptable bond properties is developed, the stress limits of Article 14.7.5, "Steel Reinforced Elastomeric Bearings—Method B," of the AASHTO LRFD Bridge Design Specifications, 2004 may be reconsidered. However, thorough testing over a wide range of loading conditions, including fatigue, will be needed prior to acceptance.~~

~~**18.2.3.3 Bond**~~

~~The vulcanized bond between fabric and reinforcement shall have a minimum peel strength of 30.0 lb/in. (5.3 N/mm). Steel laminated bearings shall develop a minimum peel strength of 40.0 lb/in. (7.0 N/mm). Peel strength tests shall be performed by ASTM D 429, Method B.~~

~~**C18.2.3.3**~~

~~Adequate bond is essential if the installation is to be effective. It is particularly important at the edges of the bearing.~~

18.2.4 Fabrication

Bearings shall meet the fabrication requirements of the *AASHTO Materials Specification M 251.*

~~**18.2.4.1 Requirements for All Elastomeric Bearings**~~

~~Bearings that are designed as a single unit must be built as a single unit.~~

~~Steel laminates must first be thoroughly sandblasted and cleaned and then protected against contamination until fabrication is complete.~~

~~Flash tolerance, finish, and appearance shall meet the requirements of the latest edition of the Rubber Handbook, published by the Rubber Manufacturers Association, Inc., RMA F3 and RMA T.063 for molded bearings and RMA F2 for extruded bearings.~~

~~**C18.2.4.1**~~

~~The shape factor, bearing stiffness and strength, and general behavior under load will be different if a bearing is built in sections.~~

~~The provisions for cleaning and protecting laminates is needed in order to achieve good bond.~~

~~Edge cover is primarily needed to prevent corrosion of the reinforcement and ozone attack of the bond. However, it also decreases the probability of delamination by reducing the stress concentrations at the exposed outer surface.~~

~~In the past, bonding during vulcanization has been the most successful method of attaching the laminates and is required for bonding of internal laminates. Practical difficulties, however, may arise in hot-bonding of external plates, thus, hot-bonding is strongly recommended for them but not required.~~

~~**18.2.4.2 Steel Laminated Elastomeric Bearings**~~

~~Bearings with steel laminates shall be cast as a unit in a mold and shall be bonded and vulcanized under heat and pressure. The mold finish shall conform to standard shop practice. The internal steel laminates shall be sand-blasted and cleaned of all surface coatings, rust, mill scale, and dirt before bonding and shall be free of sharp edges and burrs. External load plates shall be protected from rusting by the Manufacturer and should be hot-bonded to the bearing during vulcanization.~~

~~**C18.2.4.2**~~

~~External load plates are often referred to as sole plates.~~

~~In the past, bonding during vulcanizing has been the most successful method of attaching the laminates and is required for bonding of internal laminates. However, practical difficulties may arise in hot-bonding of external load plates; thus, hot-bonding is strongly recommended for them but not required.~~

~~**18.2.4.3 Fabric Reinforced Elastomeric Bearings**~~

~~Fabric reinforced bearings shall be vulcanized in large sheets and cut to size. Cutting shall be performed in such a way as to avoid heating the materials and shall produce a smooth finish with no separation of the fabric from the elastomer. Fabric reinforcement shall be at least single-ply for the top and bottom reinforcement layers and double-ply for internal reinforcement layers. Fabric shall be free of folds and ripples and shall be parallel to the top and bottom surfaces.~~

~~**18.2.4.4 Plain Elastomeric Pads**~~

~~Plain pads may be molded, extruded, or vulcanized in large sheets and cut to size. Cutting shall not heat the material and shall produce a smooth finish.~~

18.2.5 Testing

~~**18.2.5.1 Scope**~~

Materials for elastomeric bearings and the finished bearings themselves shall be subjected to the tests described ~~herein. Material tests shall be in accordance with either Table 18.2.3.1-1 or Table 18.2.3.1-2, as appropriate~~ in the *AASHTO Material Specification M 251*.

~~C18.2.5.1~~ C18.2.5

Testing requirements fall into two main categories: material quality-control tests and load tests on the finished bearings to detect poor fabrication.

Complete bearings may be tested and this is most easily done using two identical bearings on top of one another with a shear load plate between them. However, in bearings with more than two or three layers, bending and buckling effects may reduce the shear stiffness of the complete bearing below the value GA/h_{rt} given by the simple shear model. It is important to distinguish between unacceptable material and failure to analyze the rather complicated behavior with sufficient accuracy.

~~**18.2.5.2 Frequency of Testing**~~

~~The ambient-temperature tests on the elastomer specified in Article 18.2.5.3 shall be conducted for the materials used in each lot of bearings. In lieu of performing a shear modulus test for each batch of material, the Manufacturer may elect to provide certificates from tests performed on identical formulations within the preceding year, unless otherwise specified by the Engineer. Test certificates from the supplier shall be provided for each lot of reinforcement.~~

~~The three low-temperature tests on the elastomer specified in Article 18.2.5.4 shall be conducted on the material used in each lot of bearings for Grades 3, 4, and 5 material and the instantaneous thermal stiffening test shall be conducted on material of Grades 0 and 2. For Grade 3 material, in lieu of the low-temperature crystallization test, the Manufacturer may choose to provide certificates from low-temperature crystallization tests performed on identical material within the last year, unless otherwise specified by the Engineer. Low-temperature brittleness and crystallization tests shall not be required for Grades 0 and 2 materials, unless especially requested by the Engineer.~~

~~Every finished bearing shall be visually inspected in accordance with the provisions of Article 18.2.5.5.~~

~~Every steel reinforced bearing shall be subjected to the short-term load test specified in Article 18.2.5.6.~~

~~From each lot of bearings designed under the AASHTO LRFD Bridge Design Specifications, 2004, a random sample shall be subjected to the long-term load test specified in Article 18.2.5.7. The sample shall consist of at least one bearing chosen randomly from each size and material batch and shall comprise at least ten percent of the lot. If one bearing of the sample fails, all the bearings of that lot shall be rejected, unless the Manufacturer elects to test each bearing of the lot at the Manufacturer's expense. In lieu of this procedure, the Engineer may require every bearing of the lot to be tested.~~

~~The Engineer may require shear stiffness tests on material from a random sample of the finished bearings in accordance with the provisions of Article 18.2.5.8.~~

~~18.2.5.3 Ambient-Temperature Tests on the Elastomer~~

~~The elastomer used shall satisfy or exceed the criteria specified in the appropriate Table 18.2.3.1-1 or 18.2.3.1-2. The bond to the reinforcement, if any, shall also satisfy the provisions of Article 18.2.3.3, "Bond." The shear modulus of the material shall be tested at 73°F (23°C) using the apparatus and procedure described in Annex A of ASTM D 4014. It shall fall within 15 percent of the value specified in the contract documents. If no shear modulus is specified, the range of hardness shall conform to Article 14.7.5.2 of *the* AASHTO LRFD Bridge Design Specifications.~~

~~C18.2.5.3~~

~~Ambient temperature for elastomer tests refers to Article 14.7.5.2, "Material Properties" of the AASHTO LRFD Bridge Design Specifications, 2004.~~

~~The material tests at ambient temperature have been retained from previous editions of the Specification. They are quick and easy to do and are to be performed on each batch of material.~~

~~18.2.5.4 Low-Temperature Tests on the Elastomer~~

~~Low-temperature tests shall be performed in accordance with the requirements of Tables 18.2.3.1-1 and 18.2.3.1-2 and the compound shall satisfy all criteria for its grade.~~

~~C18.2.5.4~~

~~Three low-temperature material tests are required for elastomer Grades 3 to 5, and the temperature at which the tests are conducted are now different for each grade. One test is required for Grades 0 and 2 materials. The low-temperature crystallization test can be time-consuming so it may be done annually for each material instead of for each batch. The low-temperature brittleness and instantaneous stiffening tests are quick and easy so they are to be performed on every batch of Grade 3 to Grade 5 material in order to provide some ongoing control of low-temperature behavior.~~

~~18.2.5.5 Visual Inspection of the Finished Bearing~~

~~Each finished bearing shall be inspected for compliance with dimensional tolerances and for overall quality of manufacture. In steel reinforced bearings, the edges of the steel shall be protected everywhere from corrosion.~~

~~18.2.5.6 Short-Duration Compression Tests on Bearings~~

~~The bearing shall be loaded in compression to 150 percent of its rated service load. If a rotational element exists, a tapered plate shall be introduced in the load train so that the bearing sustains the load at the maximum simultaneous design rotation. The load shall be held for 5 min, removed, then reapplied for a second period of 5 min. The bearing shall be examined visually while under the second loading. If the load drops below the required value during either application, the test shall be restarted from the beginning.~~

~~The bearing shall be rejected if:~~

- ~~the bulging pattern suggests laminate parallelism,~~
- ~~a layer thickness is outside the specified tolerances,~~
- ~~a poor laminate bond exists, or~~
- ~~three or more separate surface cracks greater than 0.08 in. (2 mm) wide and 0.08 in. (2 mm) deep exists.~~

~~C18.2.5.6~~

~~The bulging pattern provides a means of checking gross defects in fabrication. This proof-load test is only an approximate indicator of bearing quality and it may both allow a few low-quality bearings into service as well as cause the rejection of a small number of bearings that would have performed adequately. However, the latter is a small price for detecting most major fabrication defects. It is important because only surface hardness and external dimensions can be checked with any ease once the bearing has been delivered.~~

~~18.2.5.7 Long-Duration Compression Tests on Bearings~~

~~The long-term compression test shall be conducted as specified in Article 18.2.5.6, except that the second load shall be maintained for 15 h. The bearing shall be visually examined at the end of the test while still under load. If any patterns or cracks specified in Article 18.2.5.6 occur, the bearing shall be rejected.~~

~~C18.2.5.7~~

~~Delamination is the most common defect and the 15-h compression test is more likely to show it than is the 5-min test. Because the 15-h test is more time-consuming, it may be done on a random sample of the bearing lot, but the press production time that it uses may be minimized if it is conducted overnight. Bearings made from Grade 4 or Grade 5 elastomers are to be subjected to the same test because achieving the necessary low-temperature properties requires special compounding which could place other properties such as bond at risk if it is not done properly. The 15-h load test may also be used to resolve differences arising from the failure of a bearing to pass a lower level test.~~

~~18.2.5.8 Shear Modulus Tests on Material From Bearings~~

~~The shear modulus of a material in the finished bearing shall be evaluated by testing a specimen cut from it using the apparatus and procedure described in Annex A of ASTM D 4014, amended where necessary in Table 18.2.3.1-1 or 18.2.3.1-2 or, at the discretion of the Engineer, a comparable nondestructive stiffness test may be conducted on a pair of finished bearings. The shear modulus shall fall within 15 percent of the specified value. If no shear modulus is specified in the contract documents, the range for hardness shall conform to Article 14.7.5.2 of the AASHTO LRFD Bridge Design Specifications. If the test is conducted on finished bearings, the material shear modulus shall be computed from the measured shear stiffness of the bearings, taking due account of the influence on shear stiffness of bearing geometry and compressive load.~~

~~C18.2.5.8~~

~~The shear test provides a check on the material properties from the body of the bearing. Specially molded samples, such as those used in the material quality-control tests, are much smaller than the finished bearing and so may require different curing times and temperatures. Specimens cut from the finished bearing provide a comparison with the material quality-control samples.~~

~~Article 14.7.5.2 is located in “Material Properties” in the AASHTO LRFD Bridge Design Specifications, 2004.~~

APPENDIX SECTION 18

Table 18.1.4.2-1 Fabrication Tolerances.

Item		Thickness Tolerance, mm	Dimension Tolerance, mm	Flatness or Out-of-Round Tolerance, mm	Surface Finish, µm (rms)
Metal Rocker and Roller Bearings		—			
	Single Roller: Diameter	—	–1.600, +1.600	–0.025, +0.025	1.6
	Nested Roller: Diameter	—	–0.050, +0.050	–0.025, +0.025	1.6
	Rockers: Diameter	—	–3.175, +3.175	–0.025, +0.025	3.2
	Pins: Diameter	—	–0.120, +0.000	–0.050, +0.050	0.8
	Bushings: Diameter	—	–0.000, +0.120	–0.050, +0.050	0.8
Pot Bearings					
	Overall Dimensions	–0.000, +6.350	–0.000, +3.175	—	—
	Pot Depth (Inside)	—	–0.000, +0.635	—	—
	Pot Wall: Thickness and Average Inside Diameter	–0.000, +3.175	–0.075, +0.075	–0.025, +0.025	0.8
	Pot Base: Top and Bottom Surfaces	–0.000, +0.635	—	Class C	1.6
	Piston: Rim	–0.000, +1.600	–0.075, +0.075	–0.025, +0.025	0.8
	Piston: Top and Bottom Surfaces	–0.000, +0.635	—	Class C	1.6
	Elastomeric Disc (Unstressed)	–0.000, +3.175	–0.000, +1.600	—	—
Disc Bearings					
	Overall Dimensions	–0.000, +6.350	–0.000, +3.175	—	—
	Shear-Restricting Element	—	–0.000, +0.125	Class A	0.8
	Other Machined Parts	–0.000, +1.600	–0.000, +1.600	Class B	1.6
	Urethane disc	–0.000, +1.600	–0.000, +3.175	Class B	1.6
Flat PTFE Sliding Bearings					
	PTFE	–0.000, +1.600	–0.000, +0.760	Class A	—
	Stainless steel	–0.000, +1.600	–0.000, +3.175	Class A	0.2
Flat Bronze and Copper Alloy Sliding Bearings					
	Sliding surfaces	–0.000, +3.175	–0.000, +3.175	Class A	0.8
Curved PTFE Sliding Bearings					
	Convex radius	—	–0.255, +0.000	–0.050,+0.050	0.2
	Concave radius	—	–0.000, +0.255	–0.050,+0.050	3.2
Curved Bronze and Copper Alloy Sliding Bearings					
	Convex radius	—	–0.255, +0.000	–0.050,+0.050	0.8
	Concave radius	—	–0.000, +0.255	–0.050,+0.050	0.8
~~Steel-reinforced Elastomeric Bearings~~					
	~~Overall dimensions~~	~~–0.000, +6.350~~	~~–0.000, +6.350~~	—	—
	~~Internal rubber layers~~	~~–3.175, +3.175~~	—	—	—
	~~Cover~~	~~–0.000, +3.175~~	—	—	—
	~~Parallelism: top and bot. surfaces~~	~~±0.005 rad~~	—	—	—
	~~Parallelism: sides~~	—	~~±0.020 rad~~	—	—
~~Elastomeric Pads~~					
	~~Overall dimensions~~	~~–0.000, +3.175~~	~~–0.000, +6.350~~	—	—
Guides					
	Contact surface	—	–0.000, +3.175	Class A	0.8
	Distance between guides	—	–0.000, +0.760	—	—
	Parallelism of guides	—	±0.005 rad	—	—
Load Plates					
	Overall dimensions	–1.600, +1.600	–6.350, +6.350	Class A	3.2
	Bevel slope	±0.002 rad	—	—	—

~~**Table 18.2.3.1-1 Polychloroprene (Neoprene) Quality-Control Tests.**~~

~~PHYSICAL PROPERTIES~~				
~~D 2240~~	~~Hardness (Shore A Durometer)~~	~~50 ± 5~~	~~60 ± 5~~	~~70 ± 5~~
~~D 412~~	~~Tensile Strength, minimum MPa~~	~~15.5~~	~~15.5~~	~~15.5~~
	~~Ultimate Elongation, minimum %~~	~~400~~	~~350~~	~~300~~
~~HEAT RESISTANCE~~				
~~D 573, 70 h at 100°C~~	~~Change in Durometer Hardness, maximum points~~	~~15~~	~~15~~	~~15~~
	~~Change in Tensile Strength, maximum %~~	~~–15~~	~~–15~~	~~–15~~
	~~Change in Ultimate Elongation, maximum %~~	~~–40~~	~~–40~~	~~–40~~
~~COMPRESSION SET~~				
~~D 395, Method B~~	~~22 h at 100°C, maximum %~~	~~35~~	~~35~~	~~35~~
~~OZONE~~				
~~D 1149~~	~~100 pphm ozone in air by volume, 20% strain 38°C ± 1°C 100 h mounting procedure D 518, Procedure A~~	~~No Cracks~~	~~No Cracks~~	~~No Cracks~~
~~LOW-TEMPERATURE BRITTLENESS~~				
~~D 746, Procedure B~~	~~Grades 0 and 2—No Test Required~~			
	~~Grade 3—Brittleness at –40°C~~	~~No Failure~~	~~No Failure~~	~~No Failure~~
	~~Grade 4—Brittleness at –48°C~~	~~No Failure~~	~~No Failure~~	~~No Failure~~
	~~Grade 5—Brittleness at –57°C~~	~~No Failure~~	~~No Failure~~	~~No Failure~~
~~INSTANTANEOUS THERMAL STIFFENING~~				
~~D 1043~~	~~Grades 0 and 2—Tested at –32°C~~	~~Stiffness at test temperature shall not exceed four times the stiffness measured at 23°C~~		
	~~Grade 3—Tested at –40°C~~			
	~~Grade 4—Tested at –46°C~~			
	~~Grade 5—Tested at –54°C~~			
~~LOW-TEMPERATURE CRYSTALLIZATION~~				
~~Quad Shear Test as Described in Annex A of ASTM D 4014~~	~~Grade 0—No Test Required~~	~~Stiffness at test time and temperature shall not exceed four times the stiffness measured 23°C with no time delay. The stiffness shall be measured with a quad shear test rig in an enclosed freezer unit. The test specimens shall be taken from a randomly selected bearing. A ±25% strain cycle shall be used, and a complete cycle of strain shall be applied with a period of 100 s. The first 0.75 cycle of strain shall be discarded and the stiffness shall be determined by the slope of the force deflection curve for the next 0.50 cycle of loading.~~		
	~~Grade 2—7 days at –18°C~~			
	~~Grade 3—14 days at –26°C~~			
	~~Grade 4—21 days at –37°C~~			
	~~Grade 5—28 days at –37°C~~			

~~Table 18.2.3.1-2 Polyisoprene (Natural Rubber) Quality Control Tests.~~

~~PHYSICAL PROPERTIES~~				
~~D 2240~~	~~Hardness (Shore A Durometer)~~	~~50 ± 5~~	~~60 ± 5~~	~~70 ± 5~~
~~D 412~~	~~Tensile Strength, minimum MPa~~	~~15.5~~	~~15.5~~	~~15.5~~
	~~Ultimate Elongation, minimum %~~	~~450~~	~~400~~	~~300~~
~~HEAT RESISTANCE~~				
~~D 573 70 h at –70°C~~	~~Change in Durometer Hardness, maximum points~~	~~10~~	~~10~~	~~10~~
	~~Change in Tensile Strength, maximum %~~	~~–25~~	~~–25~~	~~–25~~
	~~Change in Ultimate Elongation, maximum %~~	~~–25~~	~~–25~~	~~–25~~
~~COMPRESSION SET~~				
~~D 395 Method B~~	~~22 h at 70°C, maximum %~~	~~25~~	~~25~~	~~25~~
~~OZONE~~				
~~D 1149~~	~~25 pphm ozone in air by volume, 20% strain 38°C ±1°C 48 h mounting procedure D 518, Procedure A~~	~~No Cracks~~	~~No Cracks~~	~~No Cracks~~
~~LOW-TEMPERATURE BRITTLENESS~~				
~~D 746 Procedure B~~	~~Grades 0 and 2—No Test Required~~			
	~~Grade 3—Brittleness at –40°C~~	~~No Failure~~	~~No Failure~~	~~No Failure~~
	~~Grade 4—Brittleness at –48°C~~	~~No Failure~~	~~No Failure~~	~~No Failure~~
	~~Grade 5—Brittleness at –57°C~~	~~No Failure~~	~~No Failure~~	~~No Failure~~
~~INSTANTANEOUS THERMAL STIFFENING~~				
~~D 1043~~	~~Grades 0 and 2—Tested at –32°C~~	~~Stiffness at test temperature shall not exceed four times the stiffness measured at 23°C~~		
	~~Grade 3—Tested at –40°C~~			
	~~Grade 4—Tested at –46°C~~			
	~~Grade 5—Tested at –54°C~~			
~~LOW-TEMPERATURE CRYSTALLIZATION~~				
~~Quad Shear Test as described in Annex A of ASTM D 4014~~	~~Grade 0—No Test Required~~	~~Stiffness at test time and temperature shall not exceed four times the stiffness measured 23°C with no time delay. The stiffness shall be measured with a quad shear test rig in an enclosed freezer unit. The test specimens shall be taken from a randomly selected bearing. A ±25% strain cycle shall be used, and a complete cycle of strain shall be applied with a period of 100 s. The first 0.75-cycle of strain shall be discarded and the stiffness shall be determined by the slope of the force deflection curve for the next 0.25-cycle of loading.~~		

2006 AASHTO BRIDGE COMMITTEE AGENDA ITEM: 3 (REVISION 3)

SUBJECT: AASHTO LRFD Bridge Construction Specifications: Section 26, Various Articles

TECHNICAL COMMITTEE: T-4 Construction / T-13 Culverts

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

☐ **DESIGN SPEC** ☒ **CONSTRUCTION SPEC** ☐ **MOVABLE SPEC**
☐ **LRFR MANUAL** ☐ **OTHER**

☐ **US VERSION** ☐ **SI VERSION** ☒ **BOTH**

DATE PREPARED: 1/9/06
DATE REVISED: 5/25/06

AGENDA ITEM:

SECTION 26 - METAL CULVERTS

Item #1

Revise Article 26.2 as follows:

26.2 WORKING DRAWINGS

Where specified or requested by the Engineer, the Contractor shall provide Manufacturer's installation instructions or working drawings and substantiating calculations in sufficient detail to permit a structural review. ~~Whenever specified in the contract documents, the Contractor shall Manufacturer's assembly instructions or working drawings and substantiating calculations in sufficient detail to permit a structural review of the structural design.~~ The working drawings shall be submitted in advance of construction to allow for their review, revision, and approval without delay to the work.

The Contractor shall not start the construction of any metal culvert for which working drawings are required until the drawings have been approved by the Engineer. Such approval will not relieve the Contractor of responsibility for results obtained by use of these drawings or any other contractual responsibilities.

Item #2

Move text from old Article 26.5.7, create new Article 26.5.7.1 under 26.5.7 heading, and modify as follows:

26.5.7 Inspection Requirements for CMP

26.5.7.1 Visual Inspection

CMP shall be inspected after placement in the trench, as required during backfilling, and after completion of installation to ensure that final installation conditions allow the pipe to perform as designed. Installation of bedding and backfill materials, as well as their placement and compaction, shall be determined to meet the requirements of this section.

During the initial phase of the installation process, inspection shall concentrate on detecting improper practice and poor workmanship. Errors in line and grade, as well as any improper assembly or backfill techniques, shall be corrected prior to placing significant backfill or trench fill. Coupling bands shall be properly indexed with the corrugation and tightened, and bell/spigot joints shall be properly seated to prevent the infiltration of soil fines. Where gaskets are used, they shall not bulge or hang into the pipe and, if visible, should appear uniformly oriented around the pipe.

Racking or denting of the pipe shall be taken to indicate improper backfill placement~~, which shall be corrected~~. At the contractor's expense, pipe ~~Wall~~ sections damaged during installation shall be evaluated ~~and then repaired~~ by ~~an~~ a Professional Engineer and when directed, that ~~or the~~ section of the pipe shall be repaired or replaced.

Coated pipes shall be inspected to ensure the coating has no cracks, scratches, or locations of peeling. Coatings shall be repaired in accordance with material specification requirements.

Final internal inspections shall be conducted on all buried CMP installations to evaluate issues that may affect long term performance. Final inspections shall be conducted no sooner than 30 days after completion of installation and final fill.

~~Shallow cover installations shall be checked to ensure the minimum cover level is provided.~~

The inspection will verify that bedding, backfill, and compaction requirements were followed during installation. The pipe shall be checked for alignment, joint separation, cracking at bolt holes, localized distortions, bulging, flattening, or racking. ~~Minimum or near minimum~~ Shallow cover installations shall be checked to ensure the minimum cover level is provided and inspected prior to and immediately after vehicular load is applied.

Item #3

Renumber old C26.5.7 commentary to C26.5.7.1 and modify as follows:

C26.5.7.1

See Article 2.0, "Inspection," of *Highway Drainage Guidelines, Volume XIV: Culvert Inspection, Material Selection, and Rehabilitation*, 2000.

Inspections at the appropriate times during installation allow corrections to be made in assembly and backfill practices. The timing and number of visual inspections depend on the significance of the structure and its cover depth. Construction inspection during early stages of the project will allow the contractor to evaluate and, if necessary, modify construction and quality control practices.

Deeply buried structures perform more closely to their full, allowable strength level. Where the depth of cover will be significant, it is especially important to detect any problems before the pipe is buried to a depth where repair will be difficult or expensive.

Soil consolidation continues with time after installation of the pipe. While 30- days will not encompass the time frame for complete consolidation of the soil surrounding the pipe, it is intended to give sufficient time to observe some of the effects that this consolidation will have. However, occasionally pavement is placed over the pipe sooner than 30 days. While the 30-day time limit should be maintained, a brief inspection of the pipe prior to paving over it, may be prudent to ensure that good construction practices are being applied.

It is recommended that inspection personnel not enter culverts less than 24.0 in. (600 mm) in diameter. Internal inspection of culverts in this size range is best conducted using video cameras. Culverts should only be entered by inspection personnel trained in working within confined spaces and using procedures in full compliance with applicable Local, State, and Federal OSHA regulations.

~~CMP installations can be properly inspected by visual observations. Larger sizes allow for direct access to any problem areas. Because of the strain capacity of the materials and the stability of the profiles involved, uniform shape change is not critical as long as local flattening or squaring does not occur, and, as the backfill consolidates, the pipe dimensions remain stable. This is evidenced when round metal pipes are formed into pipe-arch and ellipse shapes. In these instances, the dimensions are significantly altered with little effect on load carrying capacity.~~

Racking, or loss of symmetry is structurally important in larger pipes because a flattened area is formed on one side of the crown as the top centerline is racked to the opposite side. Differential shape changes at the joint, or joint separation may allow exfiltration or infiltration resulting in erosion of the backfill material.

Slight peaking of the cross-sectional shape should be taken as indicative of achieving or exceeding minimum compaction requirements. ~~Generally, an initial change in vertical and/or horizontal dimensions, where the shape remains smooth, concentric, and constant over time should not be taken as structurally significant. However, if these dimensions do not remain constant, the affected length should be replaced.~~

Item #4

Add new Article 26.5.7.2

26.5.7.2 Installation Deflection

The pipe shall be evaluated to determine whether the internal diameter of the barrel has been reduced more than the limits set forth in this article when measured not less than 30 days following completion of installation.

Because of their broad diameter tolerances, metal pipes 24 in. (600 mm) in diameter and smaller typically are not deflection tested. A visual inspection should be performed to check for denting or other damage using a video camera or other means. If deflection testing is required by the owner or the visual inspection indicates excessive deflection, a device approved by the Engineer that can physically verify the dimensions of the pipe and is not limited by poor lighting, waterflow, pipe length, or other limiting conditions of the installed environment shall be used. If deflection testing is performed, deflection for metal pipes 24 in. (600 mm) in diameter and smaller shall not exceed 7.5 percent of the nominal diameter of the pipe plus a manufacturing tolerance as determined to be appropriate by the owner.

Pipes larger than 24 in. (600 mm) may be entered and deflection levels measured directly. In lieu of direct measurements, a calibrated video camera or any other device approved by the Engineer that can physically verify the dimensions of the pipe and is not limited by poor lighting, waterflow, pipe length, or other limiting conditions of the installed environment may be used.

In all installations of pipes larger than 24 in. in diameter, at least 10 percent of the total number of pipe runs representing at least 10 percent of the total ~~job~~ pipe footage on the project shall be randomly selected by the Engineer and inspected for deflection. Also, as determined by the 100 percent visual inspection in Article 26.5.7.1, all areas in which deflection can be visually detected shall be inspected for deflection.

Where direct measurements are made, a measurement shall be taken once every 10 ft. (3000 mm) for the length of the pipe, and a minimum of four measurements per pipe installation is required.

Pipes larger than 24 in. (600 mm) in diameter should be evaluated by direct measurement. Deflection shall be determined by comparing span and rise measurements with the nominal pipe diameter. Vertical deflection, as a percent, shall be expressed as: 100((rise/diameter) – 1.0). Similarly, horizontal deflection shall be expressed as: 100((span/diameter) – 1).

For all round pipes larger than 24 in. (600 mm) in diameter, including round and single radius arch structural plate, deflections exceeding 7.5 percent the nominal diameter of the pipe plus the manufacturing tolerance of either 1 percent the nominal diameter or 0.5 in. (13 mm), whichever is greater, shall be considered as indicative of poor backfill materials, poor workmanship or both. These pipes shall require remediation or replacement. Passing deflection criterion shall not eliminate the need to evaluate associated denting, racking or other shape damage.

For pipe arches, deflections resulting in a decrease in rise or increase in span exceeding 7.5 percent shall be considered indicative of poor backfill materials, poor workmanship, or both. These pipes shall require remediation or replacement. Passing deflection criterion shall not eliminate the need to evaluate associated denting, racking or other shape damage.

Structural plate structures should be inspected by direct measurement. They shall be assembled in accordance with the shape tolerances of Article 26.4.3. Immediately after backfilling, the structure shall be measured to check for any immediate deflections that occurred during the backfilling operation. After 30 days, the structure shall be measured again to check for any additional deflection. All deflection measurements shall be based on design dimensions. For multiple radius structures such as ellipses, pipe-arches, and low profile and high profile arches, the crown (top) radius shall not increase by more than 10 percent of the design radius as calculated from the measured middle ordinate off a suitable length straight edge, If the top radius exceeds the design value by more than 10 percent or if the structure is racked or unsymmetrical by more than 2 percent, it shall require remediation. The degree of racking or loss of symmetry shall be determined by dropping a plumb line from the actual top centerline of the installed structure and measuring the half spans that exist on each side of the plumb line to the maximum span line. For a symmetrical structure, these measurements at each individual cross-section should be equal. The degree of racking or loss of symmetry shall be expressed as a percent: 100 ((half span A – half span B)/span) < 2.

Item #5

Add new commentary for new Article 26.5.7.2.

C26.5.7.2

Ten percent of each pipe installation shall be defined as 10 percent of the number of pipe runs, and not less than 10 percent of the total length of installed pipe on the project. The requirement of deflection testing 10 percent of each pipe installation is intended to serve as a minimum and does not limit owners from more stringent requirements.

There are many appropriate methods for measuring deflection, including video inspection equipment and direct measurement. ~~Mandrels have historically not been used for corrugated metal pipes.~~ Whichever method is used for deflection measurement, a minimum of 10 percent of the total length of installed pipe shall be tested, in addition to any areas that were identified in the visual inspection as having deflection.

The deflection limits provided are similar to the deflection criteria for other flexible pipes in these Specifications. These limits do not necessarily reflect the capability of the pipe, but were chosen as limits at which the installation indicates poor workmanship that needs to be corrected to prevent future maintenance problems. ~~Corrugated metal pipes have performed well structurally when deflected as much as 20 percent as long as their shape is stable, concentric, without flattened areas, and the radius changes are smooth.~~ To prevent owners from having to measure every single pipe to establish base dimensions, deflection measurements shall be based on nominal pipe dimensions. Manufacturing tolerances per AASHTO M 36 for individual products were added to a base deflection limit of 7.5 percent to arrive at the limits as defined in this article.

Due to the broad diameter tolerances on small diameter metal pipes, it is difficult to perform deflection testing as the pass/fail criterion are often inaccurate. However they do need to be checked for denting and other damage. A thorough visual inspection for these pipes is recommended rather than deflection testing. If owners choose to deflection test small diameter pipes, they should keep in mind that the tolerances for round metal pipe are ± 1 percent or ± 0.5 in, (13mm), whichever is greater. Manufacturing processes use these tolerances and the diameter often varies within the pipe. Especially in smaller pipes, this is significant to any perceived deflection.. A –0.5 in. (13 mm) tolerance in a 12-in. (300 mm) pipe itself amounts to 4.2 percent of the diameter. This manufacturing tolerance, if it is not taken into account, can result in the acceptance of poorly installed pipe or the rejection of well installed pipe. Alternatively, owners who choose to deflection test pipes 24.0 in. (600 mm) and less in diameter can require a manufacturer's certification of the mean diameter, and deflection test based on that data.

According to the tolerance limits established in AASHTO M 36 and AASHTO M 196, the tolerance for the rise in a pipe arch can vary greatly to the positive, but is zero for negative tolerance. Additionally, the tolerance for span in a pipe arch is zero to the positive and can vary greatly to the negative. As such, the threshold criterion for deflection has been set at a 7.5 percent decrease for rise and at a 7.5 percent increase for span. This eliminates the large tolerances for pipe arches as a factor in checking for deflection.

For structural plate structures, a 10 percent increase in crown radius does not indicate a 10 percent change in rise. Depending on the shape, related rise deflections are more typically 5 percent or less. Since there are nearly an infinite number of possible design shapes for structural plate, the dimension change limits are compared to base dimensions shown on the working drawings for that particular structure. Measurements should be taken immediately after installation and backfilling, as well as after 30 days, so that corrective measures can be taken if necessary before additional construction over the structure is completed.

OTHER AFFECTED ARTICLES:

None

BACKGROUND:

T-13 Culverts coordinated specification changes to Section 26 of the AASHTO LRFD Bridge Construction Specifications. Section 26 revisions were developed with input from the AASHTO Flexible Culvert Liaison Committee. These revisions were prepared in response to the request of the Subcommittee to develop deflection criteria for metal culverts at the 2005 AASHTO Bridge Meeting. Deflection limits for other culvert materials were adopted at the 2005 Meeting.

ANTICIPATED EFFECT ON BRIDGES:

Improved quality of product by inspection. Defines installation expectations

REFERENCES:

None

OTHER:

None

2006 AASHTO BRIDGE COMMITTEE AGENDA ITEM: 4 (REVISION 1)

SUBJECT: LRFD Bridge Construction Specifications: Section 30, Article 30.5.6.2

TECHNICAL COMMITTEE: T-4 Construction / T-13 Culverts

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

☐ **DESIGN SPEC** ☒ **CONSTRUCTION SPEC** ☐ **MOVABLE SPEC**
☐ **LRFR MANUAL** ☐ **OTHER**

☐ **US VERSION** ☐ **SI VERSION** ☒ **BOTH**

DATE PREPARED: 12-22-05
DATE REVISED: 5-24-06

AGENDA ITEM:

SECTION 30 – THERMOPLASTIC PIPE

Revise Article 30.5.6.2

30.5.6.2 Installation Deflection

The pipe shall be evaluated to determine whether the internal diameter of the barrel has been reduced more than 5% when measured not less than 30 days following completion of installation.

Pipes shall be checked for deflection using a mandrel or any other device approved by the Engineer that can physically verify the dimensions of the pipe and is not limited by poor lighting, waterflow, pipe length, or other limiting conditions of the installed environment. Pipes larger than 24 in. (600 mm) may be entered and deflection levels measured directly.

In all pipe installations, at least 10 percent of the total number of pipe runs representing at least 10 percent of the total ~~project~~ pipe footage on the project shall be randomly selected by the engineer and inspected for deflection. Also, as determined by the 100% visual inspection in Section 30.5.6.1, all areas in which deflection can be visually detected shall be inspected for deflection.

Where direct measurements are made, a measurement shall be taken once every 10 feet (3000 mm) for the length of the pipe, and a minimum of 4 measurements per pipe installation is required.

If a mandrel is used for the deflection test, it shall be a nine (or greater odd number) arm mandrel, and shall be sized and inspected by the Engineer prior to testing. A properly sized proving ring shall be used to check or test the mandrel for accuracy. The mandrel shall be pulled through the pipe by hand with a rope or cable. Where applicable, pulleys may be incorporated into the system to change the direction of pull so that inspection personnel need not physically enter the pipe or manhole.~~with a force not greater than 1000 lb (4.5 kN).~~

For locations where pipe deflection exceeds 5% of the inside diameter, an evaluation shall be conducted by the Contractor utilizing a Professional Engineer and submitted to the Engineer for review and approval considering the severity of the deflection, structural integrity, environmental conditions and the design service life of the pipe. Pipe remediation or replacement shall be required for locations where the evaluation finds that the deflection could be problematic. For locations where pipe deflection exceeds 7.5% of the inside diameter, remediation or replacement of the pipe is required.

OTHER AFFECTED ARTICLES:

None

BACKGROUND:

Section 30 was modified and approved at the 2005 AASHTO Bridge Meeting. Subsequent to passage of that revision, concerns were raised by industry regarding the use of force to pull a mandrel through a pipe. A mandrel manufacturer discouraged using force due to potential damage to the mandrel. The input provided has been incorporated into this revision.

ANTICIPATED EFFECT ON BRIDGES:

Better quality of product by inspection.

REFERENCES:

None

OTHER:

None

2006 AASHTO BRIDGE COMMITTEE AGENDA ITEM: 6A (NEW)

SUBJECT: LRFD Bridge Design Specifications: Section 13, Articles 13.9.2 and 13.9.3

TECHNICAL COMMITTEE: T-7 Railings

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

☒ **DESIGN SPEC** ☐ **CONSTRUCTION SPEC** ☐ **MOVABLE SPEC**
☐ **LRFR MANUAL** ☐ **OTHER**

☒ **US VERSION** ☐ **SI VERSION** ☐ **BOTH**

DATE PREPARED: 5/23/06
DATE REVISED: 5/23/06

AGENDA ITEM:

Item #1

In Article 13.9.2, revise the first paragraph as follows:

The height of a bicycle railing shall not be less than ~~54.0~~ 42.0 in., measured from the top of the riding surface.

Item #2

Add the following paragraphs as new commentary to Article 13.9.2:

Railings, fences or barriers on either side of a shared use path on a structure, or along bicycle lane, shared use path or signed shared roadway located on a highway bridge should be a minimum of 42.0 in. high. The 42.0 in. minimum height is in accordance with the *AASHTO Guide for the Development of Bicycle Facilities*, 3rd Edition, (*1999*).

On such a bridge or bridge approach where high-speed high-angle impact with a railing, fence or barrier are more likely to occur (such as short radius curves with restricted sight distance or at the end of a long descending grade) or in locations with site-specific safety concerns, a railing, fence or barrier height above the minimum should be considered.

Item #3

In Article 13.9.3, revise Figure 1 as follows:

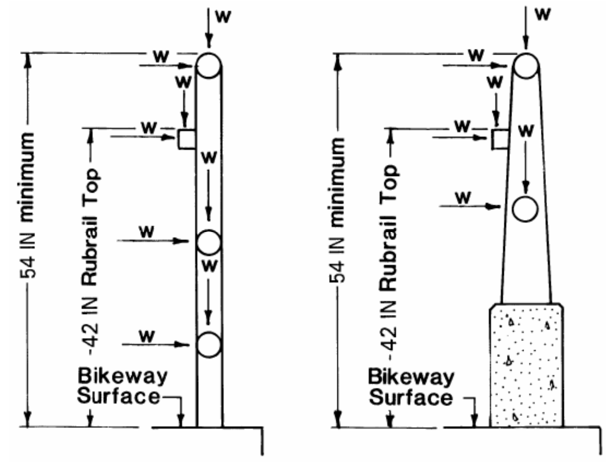


Figure 13.9.3-1 Bicycle Railing Loads—To be used on the outer edge of a bikeway when highway traffic is separated from bicycle traffic by a traffic railing. Railing shape illustrative only.

OTHER AFFECTED ARTICLES:

None

BACKGROUND:

None

ANTICIPATED EFFECT ON BRIDGES:

None

REFERENCES:

None

OTHER:

None

2006 AASHTO BRIDGE COMMITTEE AGENDA ITEM: 7 (REVISION 1)

SUBJECT: SI to English conversion of the AASHTO LRFD Movable Highway Bridge Design Specifications with other enhancements and revisions to Article 5.4

TECHNICAL COMMITTEE: T-8 Movable Bridges

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

☐ **DESIGN SPEC** ☐ **CONSTRUCTION SPEC** ☒ **MOVABLE SPEC**
☐ **LRFR MANUAL** ☐ **OTHER**

☒ **US VERSION** ☐ **SI VERSION** ☐ **BOTH**

DATE PREPARED: 1/13/06
DATE REVISED: 5/25/06

AGENDA ITEM:

Item #1

For the proposed new English document, see Attachment A.

Item #2

For the pending editorial corrections, see Attachment B.

Item #3

In addition:

- Minor revisions to the wording in Articles 5.4.1, 5.4.2, 5.4.3, and 5.4.4 is proposed to add flexibility to the design as pertaining to the effects of inertia.

- The term dim will be added to the Notations of each Section and the Definition and Notions will be expanded upon for each Section.

- It is also proposed that an index to the Specifications be developed under the oversight of T-8 and included in this new document.

Item #4

Delete Appendix A

OTHER AFFECTED ARTICLES:

None

BACKGROUND:

This proposed new document will be the 2nd Edition of the LRFD Movable Specifications incorporating 2002 interim changes as well as a conversion to English units and a Word Perfect to Word software conversion. This 2nd

Edition includes the SI Version of equations, tables and figures as Appendix A. Development of an index is proposed to supplement the document along with other enhancements. Changes to Article 5.4 will add flexibility to the Specifications and allow for more conservative design if the owner so desires.

ANTICIPATED EFFECT ON BRIDGES:

None

REFERENCES:

AASHTO LRFD Movable Highway Bridge Design Specifications – First Edition 2000 including 2002 interims

OTHER:

None

SECTION 1 - TABLE OF CONTENTS - GENERAL PROVISIONS

SPECIFICATIONS

1.1 GENERAL CRITERIA

These Specifications cover the design of movable highway bridges.

The requirements for fixed span bridges, as given in the latest edition of the AASHTO LRFD Bridge Design Specifications, shall apply to movable bridges, except as otherwise provided herein. The concepts of structural safety through redundancy and ductility and of protection against scour and collision are emphasized in those specifications.

These Specifications are not intended to supplant proper training or the exercise of judgment by the Designer, and state only the minimum requirements necessary to provide for public safety. The Owner or the Designer may require the sophistication of design or the quality of materials and construction to be higher than the minimum requirements.

The design provisions of these Specifications employ the Load and Resistance Factor Design (LRFD) methodology to the extent practical at the time of writing. The factors have been developed from the theory of reliability based on current statistical knowledge of loads and structural performance.

The parts of the specification dealing with workmanship and erection are intended to be guidelines for the designer, to include as required in the specifications written for the specific project.

The structural, mechanical, hydraulic, and electrical design will be furnished by the Owner, unless it is expressly stated in the invitation for bids that such designs, or specified portions of them, are to be furnished by the Contractor.

The operating time shall be stated in the contract documents and all gates, warning devices and operating equipment and control systems shall be designed to meet or exceed the criterion for time of opening.

The contracts shall state:

- the time of opening,
- assumed time of vessel passage, and
- the time of operation for both the main and auxiliary power.

The contract documents shall make clear the division of responsibility between the Owner and the Contractor for designing, furnishing and erecting or installing all components of the structure, and shall fully specify or describe all of such components which are the responsibility of, or affect the work of, the Contractor.

COMMENTARY

C1.1

The commentary is not intended to provide a complete historical background concerning the development of these or previous Specifications, nor is it intended to provide a detailed summary of the studies and research data reviewed in formulating the provisions of the Specifications. However, references to some of the research data are provided for those who wish to study the background material in depth.

The term "notional" is often used in these specifications to indicate an idealization of a physical phenomenon, as in "notional load" or "notional resistance." Use of this term strengthens the separation of an engineer's "notion" or perception of the physical world in the context of design from the physical reality itself.

The term "shall" denotes a requirement for compliance with these Specifications.

The term "should" indicates a strong preference for a given criterion.

The term "may" indicates a criterion that is usable, but other local and suitably documented, verified, and approved criterion may also be used in a manner consistent with the LRFD approach to bridge design.

In the context of this article exceeding the design criteria means performing the intended function in less time than stated.

The Contractor shall furnish and erect the structure ready for operation and to receive traffic, except for such components as are specified to be furnished or installed by the Owner.

1.2 ABBREVIATIONS, DEFINITIONS AND COMPONENT CLASSIFICATIONS

Bascule - Bridge type which rotates in the vertical plane and one end is counterbalanced by the other on the principle of a seesaw or by weights. Rotation is about a horizontal axis which may or may not be fixed.

Swing - Bridge type which a portion of the span rotates about a vertical axis.

Vertical Lift - Bridge type which movable span is moved up/down while remaining horizontal.

The following abbreviations are used herein:

AASHTO - American Association of State Highway and Transportation Officials

ABMA - American Bearing Manufacturers Association

AGMA - American Gear Manufacturers Association

AISI - American Iron and Steel Institute

ANSI - American National Standards Institute

AREMA - American Railway Engineering and Maintenance-of-Way Association

ASM - American Society for Materials

ASME - American Society of Mechanical Engineers

ASTM - American Society for Testing and Materials

AWS - American Welding Society

CAD - Computer Aided Design (Drafting)

EEIPS - Double Extra Improved Plow Steel

EIPS - Extra Improved Plow Steel

EP - Extreme Pressure

FEA - Finite Element Analysis

IEEE - Institute of Electrical and Electronics Engineers, Inc.

IPCEA - Insulated Power Cable Engineers Association

IPS - Improved Plow Steel

ISO - International Organization for Standardization

IWRC - Independent Wire Rope Core

MUTCD - Manual on Uniform Traffic Control Devices for Streets and Highways

NCHRP - National Cooperative Highway Research Program

NEC - National Electrical Code

NEMA - National Electrical Manufacturers Association

NESC - National Electrical Safety Code

NFPA - National Fluid Power Association; National Fire Protection Association

NLGI - National Lubricating Grease Institute

PTFE - Polytetrafluoroethylene

SAE - Society of Automotive Engineers

SI - System International for Units

UL - Underwriter's Laboratories

WRTB - Wire Rope Technical Board

The parts of the bridge will be classified as follows and may be paid for as indicated below:

Communication Facilities - Radio, telephone and any other communication facilities, including wiring therefor.

Concrete - Concrete or mortar used in counterweights, including concrete balance blocks, and concrete in pockets of column bases and similar places. No deductions shall be made for enclosed reinforcing steel, scrap metal, or steel punchings.

Counterweight Sheaves - Cast or welded sheaves together with their shafts, bearings and their connecting bolts.

Electrical Equipment - High-voltage equipment and transformers as specified, the control panels and control console with their attachments, and electrical parts beyond (whether on or off the moving span), such as motors, tachometers and encoders, generators, gear motors, controllers, programmable logic controllers and computers, resistors, electric brakes, solenoids, circuit breakers, fuses, relays, contactors, switches, electric indicators, synchronizing and leveling equipment, limit switches, meters, trolley wheels and contact shoes, service and indicating lights, navigation lights and signals, traffic gates and barriers, traffic signs, electric heaters, conductors, wiring, submarine and other cables, boxes, ducts and conduits and their fittings, as specified for the operation of the moving span and accessories, and the lighting and climate control of the houses.

Elevators - The complete elevator system, including cars, gates, motors, and other operating machinery; guide rails and shoes, counterweights, buffers, hoisting cables, governors or other speed control devices and wiring.

Engines, Tanks, Pumps - Internal combustion engines with tanks, compressors, starters, pumps, indicators, whistles, air compressor tanks, and interrelated piping to and including clutch shaft, but not clutch for delivery of power, and piping not including valve for delivery of air and not to include such engines used as prime movers for standby electric power.

Houses for Machinery and Operators - All parts of such houses, except steel framing and plating if any; also all furniture, heating/ventilating/air conditioning, cranes and hoists, fire extinguishers, supplies, and similar items, as specified in the invitation to bid.

Hydraulic Equipment - All hydraulic equipment, including hydraulic fluid and portable filtration units used during reservoir filling, necessary to provide the operating system specified.

Machinery -

Air buffers
Axles
Bars
Bearings
Bells
Brakes (unless part of electrical equipment)
Bridge locks
Cables and wires for push-pull devices
Capstans
Center-pivot stands
Centering devices
Clutches
Clutch operators
Couplings
Cranks
Deflector castings and Plates
Disks
Eccentrics
Gear covers and guards
Gears
Hooks
Indicators
Levers
Racks
Rollers
Roller guides
Roller treads
Screws
Shafts
Sheaves (except counterweight sheaves)
Shims
Shock absorbers
Speed reducers
Spools
Toggles
Wedges
Wedge bases
Wheels
Whistles
Winding drums
Worm gearings
Wrenches

Pins about whose axes the connecting members rotate.

Equalizing devices and other fastenings for wire ropes (except sockets).

Bolts attaching machinery parts to each other and to their supports.

Similar parts which require machine shopwork and which are not included in any other class. Pumps, tanks, cylinders, piping valves of hydraulic systems.

Machinery parts attached to structural parts shall be separately weighed before attachment thereto. Lubricants and hydraulic fluids will not be included in the weight measured for payment.

Metal in Counterweights - Cast iron or other metal used as counterweights; also scrap metal or steel punchings used to increase the unit weight of counterweight concrete.

Reinforcing Steel - All reinforcing bars and mesh for concrete.

Removals - Any and all parts of an existing structure required to be removed.

Roadway and Sidewalk Floor - Roadway deck and footwalks at roadway level complete in place.

Structural Carbon Steel and Special Structural Steel - In addition to the moving span, any parts of rolled, forged, or cast steel which can be fabricated by the common shop methods such as punching, reaming, drilling, boring, shearing, planing, bending, welding, etc., usual for stationary structures, except structural steel parts that function as machinery parts, which shall be classified under the appropriate machinery items.

Rim girders in swing bridges, segmental girders in rolling bascule bridges and the girders on which they roll, parts supporting the machinery, machinery housing, except gear cases, counterweight frames, counterweight trusses, counterweight boxes, operating struts, rope attachment brackets or hangers, towers, steel framing and plates in houses for machinery and operators, handrails, stairways and ladders shall be classified as structural steel.

Members composed wholly of carbon steel shall be classified as structural carbon steel.

Members composed wholly of high-strength steel, or other special structural steel shall be classified as such. Members or parts containing two or more classes of structural steel shall be classified and paid for according to the computed ratios of weights of the several classes of steel contained therein.

Timber - Any lumber not allocated to another class by the foregoing definitions, together with nails, bolts and other fastenings. Measurement of lumber shall be based on nominal sizes for the lengths in place.

Time of Opening - The total elapsed time from the signal to stop traffic to the signal to release the stopped traffic.

Time of Operation - Normal length of time for opening the bridge, after all span locks are released and any lifts or wedges are withdrawn.

Time of Vessel Passage - The time specified by the Owner for the bridge to remain in the open position to allow for vessels passing through the channel.

Tread Plates and Castings - Tread plates or castings for segmental girders and track girders for rolling-lift bridges, together with their shims and connecting bolts.

Trunnions and Bearings - Trunnions for moving leaves and counterweights of bascule bridges together with their bearings, supporting pedestals and their bolts.

Wire Ropes and Sockets - Wire ropes and their sockets, shims and attachments, together with the socket pins.

SPECIFICATIONS

1.3 DESIGN PHILOSOPHY

1.3.1 General

Bridges shall be designed for specified limit states to achieve the objectives of constructibility, safety, and serviceability, with due regard to issues of inspectibility, economy, and aesthetics, as specified in Article 2.5 of the AASHTO LRFD Bridge Design Specifications.

Regardless of the type of analysis used, Equation 1.3.2.1-1 shall be satisfied for all specified force effects and combinations thereof.

1.3.2 Limit States

1.3.2.1 GENERAL

Each component and connection shall satisfy Equation 1 for each limit state, unless otherwise specified. For service and extreme event limit states, resistance factors shall be taken as 1.0, except for bolts, for which the provisions of Article 6.5.5 of the AASHTO LRFD Bridge Design

COMMENTARY

C1.3.1

The design of bridge machinery in the United States is based on allowable working stress design, therefore, this specification follows the accepted industry design practice in this regard. As of this writing (1998), reliability-based design at the strength limit state is not possible given the dearth of necessary data.

The resistance of components and connections is determined, in many cases, on the basis of inelastic behavior, although the force effects are determined by using elastic analysis. This inconsistency is common to most current bridge specifications as a result of incomplete knowledge of inelastic structural action.

C1.3.2.1

Equation 1 is the basis of LRFD methodology. Ductility, redundancy, and operational importance are significant aspects affecting the margin of structural safety of bridges. Whereas the first two directly relate to physical strength, the last concerns the consequences of the bridge being out of service. The grouping of these aspects on the

SPECIFICATIONS

Specifications shall apply. All limit states shall be considered of equal importance.

$$\Sigma \eta_i Y_i Q_i \leq \Phi R_n = R_r \qquad (1.3.2.1\text{-}1)$$

in which:

For loads for which a maximum value of Y_i is appropriate:

$$\eta_i = \eta_D \eta_R \eta_I \geq 0.95 \qquad (1.3.2.1\text{-}2)$$

For loads for which a minimum value of Y_i is appropriate:

$$\eta_i = \frac{1}{\eta_D \eta_R \eta_I} £ 1.0 \qquad (1.3.2.1\text{-}3)$$

where:

Y_i = load factor: a statistically based multiplier applied to force effects

Φ = resistance factor

η_i = load modifier: a factor relating to ductility, redundancy, and operational importance

η_D = a factor relating to ductility, as specified in Article 1.3.3

η_R = a factor relating to redundancy as specified in Article 1.3.3

η_I = a factor relating to operational importance as specified in Article 1.3.3

Q_i = force effect

R_n = nominal resistance

R_r = factored resistance: ΦR_n

Application of Equation 1 to structural design shall be as specified in Section 3 of the AASHTO LRFD Bridge Design Specifications and Section 2 herein.

Application of Equation 1 to machinery design may take different forms for the various limit states under consideration, such as:

- For the service limit state, the load factors, Yi, will be taken as 1.0 unless specified otherwise and the resistance factor, Φ, will usually be taken equal to 1.0. Resistance should be based on allowable stresses, i.e., a measure of resistance divided by a factor of safety.

COMMENTARY

load side of Equation 1 is, therefore, arbitrary. However, it constitutes a first effort at codification. In the absence of more precise information, each effect, except that for fatigue and fracture, is estimated as ±5 percent, accumulated geometrically, a clearly subjective approach. With time, improved quantification of ductility, redundancy, and operational importance, and their interaction and system synergy, may be attained, possibly leading to a rearrangement of Equation 1, in which these effects may appear on either side of the equation or on both sides.

In the case of structural design, some of the limit states may be based on non-linear response of cross-sections. In the case of operating machinery, where even small amounts of misalignment from inelastic behavior may result in an inability of gears, especially high speed gears, to mesh properly, or may result in high local stresses in gears, shafts, and operating rods, such inelastic behavior must be avoided, even at the overload and extreme event limit states. This being the case, the resistance, R_n, will be based on the allowable stresses typical in machinery design, which have a relatively high factor of safety, and a resistance factor, Φ, which may be on the order of 2.25 to 3.6, depending on the individual provisions. Nonetheless, even when the resistance factor appears to be relatively high, the intent of the specification is that the operating machinery remain below the yield point even under relatively extraordinary events. This approach is significantly different than the approach taken in the structural design which allows significant damage to occur, including permanent deformations, as a result of the loads anticipated under the strength and extreme event limit states. Certain provisions of these specifications will require the designer to reassess the amount of structural damage which will be tolerated under these limit states so as to control the alignment of the movable bridge.

SPECIFICATIONS

- In the overload limit state, applicable only to operating equipment, the load factors, Y_i, may be taken as 1.0 or greater, and the resistance factor, Φ, will typically be greater than 1.0. Nominal resistance will be taken as specified for the service limit state.

- In the extreme event limit state, the load factors are taken as 1.0 or greater and resistance factors will be taken greater than 1.0 and typically greater than as specified for the overload limit state. Nominal resistance should be taken as specified for the service limit state.

1.3.2.2 SERVICE LIMIT STATE

For both the structural system and the operating equipment, the service limit state shall be taken as restrictions on stress, deformation, and crack width under regular service conditions, as appropriate, and as specified in Sections 6, and 7, and in the AASHTO LRFD Bridge Design Specifications for mechanical design, hydraulic design and structural design, respectively.

1.3.2.3 FATIGUE AND FRACTURE LIMIT STATE

The fatigue limit state shall be taken as restrictions on stress range as a result of:

- a single design truck occurring at the number of expected stress range cycles, and

- in the case of the operating equipment, the anticipated effects of bridge operation, possibly including the first and last live load cycle.

- structural loads resulting from bridge opening, possibly including the first and last live load cycle.

The structural fracture limit state shall be taken as a set of material toughness requirements of the AASHTO Material Specifications.

1.3.2.4 OVERLOAD LIMIT STATE

The overload limit state, applicable only to the operating equipment and associated structural supports, shall be taken to provide some relief from the requirements of the stress limitation specified for the service limit state for loadings which are generally:

- more frequent than the higher loads considered under the extreme event limit state,

COMMENTARY

C1.3.2.2

The service limit state provides certain experience-related provisions that cannot always be derived solely from strength or statistical considerations.

C1.3.2.3

The fatigue limit state is intended to limit crack growth under repetitive loads to prevent fracture during the design life of the bridge.

SPECIFICATIONS

- much less frequent than the loads considered under the fatigue limit state, and
- higher than the routine loads considered under the service limit state.

1.3.2.5 STRENGTH LIMIT STATE

The strength limit state shall apply only to structural components and shall be taken to ensure that strength and stability, both local and global, are provided to resist the specified statistically significant load combinations that a bridge is expected to experience in its design life.

1.3.2.6 EXTREME EVENT LIMIT STATES

The extreme event limit state shall be taken to ensure:

- the structural survival of a bridge during a major earthquake or flood, or when collided by a vessel, vehicle, or ice flow, possibly under scoured conditions, and
- variable operability requirements for the bridge under seismic events specified in Section 3.

1.3.3 Factors for Ductility, Redundancy and Operation

For the structural system, the load multipliers, η_i, η_d, and η_r shall be taken as specified in Articles 1.3.3, 1.3.4 and 1.3.5 of the AASHTO LRFD Bridge Design Specifications:

- Consideration should be given to selection of an importance factor, η_I, of 1.05 due to the operational importance of movable bridges and the potential for disruption to highway and marine traffic.
- Consideration should be given to the selection of a redundancy factor, η_r, equal to 1.05 for movable bridges which are non-redundant, as is the usual case.
- Consideration of a ductility factor, η_d, equal to 1.0 where detailings conform to the requirements of the AASHTO LRFD Bridge Design Specifications, and these specifications.

COMMENTARY

C1.3.2.5

Extensive distress and structural damage may occur under strength limit state, but overall structural integrity is expected to be maintained. This limit state does not apply to operating equipment because extensive distress could lead to inoperability for extended periods of time. The statistical data to calibrate this limit state to allow inelastic behavior in the case of the operating response is not yet available, and the extent of tolerable inelastic behavior which can be tolerated in machinery, if any, has not yet been established.

C1.3.2.6

Extreme event limit states are considered to be unique occurrences whose return period may be significantly greater than the design life of the bridge.

SPECIFICATIONS

COMMENTARY

In the case of the operating system, η_I, η_d, and η_r, should normally be taken as 1.0.

1.4 DESIGN OF BRIDGE SYSTEMS

1.4.1 Structural Design

The requirements for fixed span bridges, as given in the latest edition of the AASHTO LRFD Bridge Design Specifications, shall apply to movable bridges except as otherwise provided in Section 2.

The structural design will be furnished by the Owner, unless it is expressly stated in the invitation for bids that such designs, or specified portions of them, are to be furnished by the Contractor.

1.4.2 Machinery Design

The mechanical and hydraulic machinery design will be furnished by the Owner, unless it is expressly stated in the invitation for bids that such designs, or specified portions of them, are to be furnished by the Contractor.

Machinery design shall comply with the provisions of Sections 5, 6 and 7.

1.4.3 Electrical Design

The electrical design will be furnished by the Owner, unless it is expressly stated in the invitation for bids that such designs, or specified portions of them, are to be furnished by the Contractor. The provisions of Section 8 of these specifications shall apply to electrical design.

Where feasible, movable bridges should be operated by electric power. Electric power operated bridges shall be equipped with an auxiliary source of power when specified. This auxiliary power source may be a second electric service originating from a separate utility substation, an internal combustion engine, an engine-generator set, manual power, or other prime mover, as specified by the Engineer.

Any engine shall be equipped with suitable mufflers to control noise.

When electric motors are used, emergency motors and their associated motor control equipment shall be provided when specified by the Engineer, separate and independent of those used in conjunction with normal operation.

When emergency operation of the movable span is by a prime mover other than an electric motor, standby electric power may be limited to that necessary for the operation of navigation and other warning signals.

SPECIFICATIONS

1.4.4 Safety Design

1.4.4.1 GENERAL

Warning signs, hazard identification beacons, traffic signals, signal bells and gongs, gates and barriers, and other safety devices shall be provided for the protection of pedestrian and vehicular traffic. These shall be designed to be operative prior to the opening of the movable span and until the span has again been completely closed. The devices shall conform to the requirements for "Traffic Control at Movable Bridges," in the "Manual on Uniform Traffic Control Devices for Streets and Highways" (MUTCD).

1.4.4.2 CLEARANCES

Vertical and horizontal clearances of movable bridges, in the closed and open positions, shall be subject to the approval of regulatory authorities having jurisdiction over waterway navigation.

1.4.4.3 PROTECTION FROM WATERWAY TRAFFIC

The superstructure and substructure of movable bridges adjacent to a navigable channel shall be protected against damage from waterway traffic by suitable fenders, dolphins, or other protective devices. The provisions of Section 4 shall apply.

1.4.4.4 TRAFFIC GATES AND BARRIERS

Two types of gates should be provided on each approach roadway to a movable span bridge: a warning gate and a physical barrier, i.e., resistance gate. They should extend across the full width of undivided roadways. If a median barrier is provided, warning gates should extend across all roadways, while barriers may extend across only the roadways approaching the draw. Markings shall be provided on the gates and shall be in accordance with the MUTCD. Red signal lights shall be mounted on the gates and interconnected to operate with the traffic signals, and any time the gates are less than fully opened. The contract documents shall specify that the gates be furnished by the Contractor as complete units with operating motors and limit switches. Energy absorbing systems may be utilized for barrier gates.

Gates shall be specified to operate reliably at the maximum wind velocities for which the movable bridge is considered operational. In ice-prone regions, gates shall be specified to operate reliably with the maximum ice build-up on gate arms, without adjustment to the counterweight.

COMMENTARY

C1.4.4.4

Problems have been experienced with wind-induced deflections of gate arms preventing unlatching of the resistance gate arm ends. Problems have also been experienced with gates with long arms failing to operate with the unbalance due to ice accumulation.

SPECIFICATIONS

Provision for manual operation in case of power failure shall be required on all gates. Interlocking with the span mechanisms should be provided to prevent bridge openings before gate closure. The gates should operate independently using separate switches on the control console. Group raise switches may be specified, in addition to the individual controls, to allow raising all gates simultaneously. The switches, contactors, relays, etc., shall be furnished as part of the bridge control panel.

Consideration should be given to location of traffic gates off approach spans exposed to vessel collision to ensure the safety of motorists waiting for the bridge to close.

Momentary switches without seal-in contacts should be provided to permit the gate closure to stop upon release of the operating switch. Opening should be continuous upon and after momentary closure of the switch controls.

1.4.4.5 PEDESTRIAN GATES

Where a pedestrian walkway is provided, either a separate gate shall be provided to block access to the span walkway, or the traffic gates may be designed to also serve this purpose.

1.4.4.6 WARNING LIGHTS, ALARMS, AND TRAFFIC SIGNALS

1.4.4.6.1 Traffic Signals and Bells

Traffic signals shall be provided on all movable span bridges, other than manually-operated bridges, and shall be in accordance with the requirements in the MUTCD. Warning bells or gongs should be provided to supplement the traffic signals. For the manually-operated spans, standard stop signs supplemented by red flags or lights may be used.

Bells and gongs should be operated in conjunction with the gate closing and should be interconnected with the gate mechanism. An interlock shall be provided to break the bell and gong circuit when the last device prior to opening the span has been operated. This will usually occur when the span locks or end wedges are withdrawn.

1.4.4.6.2 Audible Navigation Signals, Navigation Lights, Aviation Lights

Navigation lights, aviation obstruction lights, beacons and other signals or markings required by regulatory authorities shall be furnished and provided with a suitable means of access to such lights and signals.

All navigation and other light units on the movable span and on fenders shall be capable of withstanding

COMMENTARY

In 1972, a freighter hit the approach spans of the Sidney Lanier Bridge in Georgia causing fatalities in the cars that were waiting on the approach spans for the lift span to close.

C1.4.4.6.1

Signals installed at movable bridges are a specific application of the broader discipline of traffic control signals. The Federal Manual on Uniform Traffic Control Devices recognizes both the standard three color signal, and the multiple red signals in vertical array for movable bridges. However, the standard three color signals are preferred from a traffic safety standpoint, since the non-standard signals are more likely to generate motorist confusion from lack of the intermediate yellow light warning of the imminent red stop signal. Use of the non-standard signals should be restricted to roadways with posted speeds of <u>25 mph</u> or less and low average daily traffic.

SPECIFICATIONS

shocks and rough treatment, and shall be completely weatherproof.

Audible navigation signals, such as air whistles, air horns, sirens, or other devices, shall be provided, as specified. Audible navigation signals shall conform to the requirements of all legally constituted bodies, agencies, or authorities having jurisdiction over the waterway, and shall be installed to their satisfaction and approval.

1.4.4.7 STAIRWAYS AND WALKWAYS

Stairways, platforms, and walks with railings and toe plates shall be provided to give safe access to the operator's house, machinery, trunnions, counterweights, lights, bridge seats, and all points requiring lubrication or other servicing. Unless specified otherwise, all access appurtenances shall be of metal construction. Ladders may be installed only where stairways are not feasible, and shall be provided with safety cages where required by codes. In vertical lift bridges, ladders, stairs, or walks shall be installed to give access to the moving span in any position from either tower. Hand railings shall be made of galvanized corrosion-resistant steel, or other rust-resistant metal pipe not less than 1¾ in. outside diameter, or of structural shapes. Stairways and ladders shall be made of steel or aluminum.

1.4.4.8 ROTATING AND MOVING MACHINERY

The contract documents shall require all moving machinery parts to be painted Federal Safety orange. This shall include shafts, couplings, sides of open gears, brake wheels, crank arms, and other moving parts as applicable. The working surfaces of these parts shall not be painted.

1.5 BALANCE AND COUNTERWEIGHTS

1.5.1 General

The Contractor shall be made responsible for the balance of the movable spans throughout the entire contract period, unless otherwise specified, and shall make such adjustments as are necessary to balance the span as specified. As near practical, the counterweights shall be sufficient to balance the movable span and its attachments in any position, except that there shall be small positive dead load reactions, specified by the designer, at the supports when the bridge is seated. Auxiliary counterweights or balance chains shall be provided for vertical lift bridges with vertical movements exceeding 80 ft.

Provision shall be made for unbalanced conditions in the design of the machinery and the power equipment.

COMMENTARY

C1.5.1

Theories on the best balance condition vary among owners. The designer should verify the owner's preference. In the absence of a stated preference, the following guidelines may be considered:

- If a bascule bridge has a roadway break behind the trunnion, i.e. toward the counterweight, the span imbalance should be greater than live loading cases that would tend to cause uplift, but shall not be less than the general cases below.

- For all bascule bridges, the movable leaves should be balanced such that the center of gravity with the span fully seated is located towards the channel at an angle

SPECIFICATIONS

A swing span having unequal lengths or unbalanced dead loads shall be balanced by counterweighting.

1.5.2 Counterweight Details

Provision shall also be made for independent supports for the counterweights of vertical lift bridges for the purpose of replacing counterweight ropes.

Counterweights usually shall be of concrete, supported by a steel frame. Boxes shall be rigidly braced and stiffened to prevent warping or bulging. All surfaces of the boxes in contact with the concrete shall be provided with open holes about 700 $in.^2$ to each 10 $ft.^2$ of surface to permit escape of water from the box as the concrete cures, or otherwise a low-slump concrete shall be used with any excess water drawn off as the concrete is placed. In the design of counterweight attachments, details which may produce fatigue effects due to vibration of the structure shall be avoided.

Concrete counterweights not enclosed in steel boxes shall be adequately reinforced.

Counterweights shall be made so as to be adjusted for variations in the weight of the span and in the unit weight of the concrete. Pockets shall be provided in the counterweights to house the balance blocks necessary to compensate for not less than 3.5 percent underrun and 5 percent overrun in the weight of the span. Each completed counterweight shall contain not less than one percent of its weight in balance blocks, arranged so as to be readily removable for future adjustment. Additional blocks for

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no greater than 20 degrees above or below a horizontal line passing through the trunnion.

- For single leaf bascules, the equivalent downward reaction at the toe should be 1,000 lb. per bascule girder with the leaf fully seated.
- For double leaf bascules, the equivalent downward reaction at the toe should be 1,500 lb. per bascule girder with the leaf fully seated.
- Bascule bridges operated hydraulically, especially with cylinders, should be balanced such that they are span heavy in all positions. The amount of imbalance must be carefully considered by the designer. The imbalance may range from the values specified herein to a non-counterweighted design. There have been several single leaf bascule spans constructed without counterweights.
- Vertical lift bridges should have a downward reaction of no less than 1,000 lb. per corner with the span fully seated. This value should increase with the size and mass of the vertical lift up to a maximum of 2,700 lb. per corner.

future adjustment in the amount of 0.5 percent of the weight of the counterweight shall also be provided and shall be stored at the site as specified by the Engineer. All balance blocks shall be firmly held in place so that they will not move during the operation of the bridge. Balance blocks shall be provided with recessed handles and shall weigh not more than 100 lb. each. Balance blocks shall be furnished only as necessary to meet the specified requirements for future adjustment and to secure the required balance of the span and the counterweights.

Pockets in counterweights shall be provided with drain holes not less than 2 in. in diameter. The pockets should be covered where exposed to the weather. The cover shall be weatherproof. The pockets should provide for both vertical and horizontal load adjustment.

1.5.3 Counterweight Concrete

Unless otherwise specified, concrete shall be made with Type II cement, and shall be proportioned as directed by the Engineer, with not more than 5½ gal of water per sack, taken as 43 kg of cement. Where heavy concrete is required for counterweights, the coarse aggregate shall be trap rock, magnetic iron ore, or other heavy material, or the concrete may consist of steel punchings or scrap metal, and mortar composed of 1 part cement and 2 parts of fine aggregate. The maximum weight of heavy concrete shall be 315 pcf and preferably not more than 275 pcf. Heavy concrete shall be placed in layers and consolidated with vibrators or tampers. Methods of mixing and placing shall be such as to give close control of the unit weight of the concrete and uniformity of unit weight throughout the mass. Steel ingots or billets shall preferably be used in counterweights when required to obtain specified counterweight density. They shall be distributed uniformly throughout the counterweight, shall be individually supported within the counterweight and shall be clean and free of oil or grease and not galvanized or coated with other materials.

For ascertaining the weight of the concrete, test blocks, having a volume of not less than 4 $ft.^3$ for ordinary concrete, 1 $ft.^3$ for heavy concrete and 1 $ft.^3$ for the mortar for heavy concrete, shall be cast at least 30 days before concreting is begun. Two test blocks of each kind shall be provided with one weighed immediately after casting and the other after it has seasoned. If samples show a tendency to swell, they shall be rejected.

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1.5.4 Counterweight Pits and Pit Pumps

If practical, counterweight pits shall be drained by gravity. Otherwise, provision shall be made for pumping water out and for cleaning of the counterweight pit. A sump shall be placed in the floor of the pit so that water will collect in it. The pump shall preferably be placed in the sump. Sump pumps should be automatically controlled with level switches for on and off, and shall be specified as trash pumps to pass small debris without clogging. Consideration should be given to environmental containments in pit effluent.

1.5.5 Diversion of Drainage

Provision shall be made for preventing excessive roadway drainage from entering the counterweight pit.

1.6 MACHINERY AND OPERATOR'S HOUSES

<u>See section 1.4.4 for additional safety design considerations.</u> See Section 8 for additional electrical requirements for machinery and operator's houses.

1.6.1 Machinery House

A suitable house or houses shall be provided for the protection of the machinery wherever practical. Houses shall be large enough for easy access to all machinery, and shall be fireproof and weatherproof. Houses or rooms containing electric control equipment shall have thermal insulation and climate control as specified. Windows, when provided, shall be glazed with shatterproof glass. Openings shall be large enough to admit passage of the largest unit of machinery.

The floor shall be concrete, steel, or other fireproof material, as specified. It shall have a nonslip surface. Floors in rooms containing electrical equipment such as motor control centers or control cabinets shall be covered with vinyl, rubber tile, or other electrically insulating materials as specified on areas surrounding such electrical equipment.

Consideration shall be given for a hand-operated overhead traveling crane or hoist, of sufficient capacity for handling the heaviest piece of machinery, to be specified in the machinery house.

Adequate heating and ventilation provisions shall be made for times when maintenance personnel are performing maintenance tasks in the house during the climatic extremes for the location.

Where a machinery house is not feasible or practical, exposed or semi-exposed machinery decks may be utilized for mounting weatherproof machinery.

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1.6.2 Operator's House and Machinery House Access Clearance

Access to the Operator's house and drive machinery shall be maintained at all stages of opening or closing the movable span.

If the operator is not located in the machinery house, a separate operator's house shall be provided. The type of construction shall be the same as that specified for the machinery house, except that for hand-operated bridges with the house located off the bridge structure, fully fireproof construction will not be required. Consideration shall be given to the use of bulletproof windows and doors.

The operator's house shall be located, and ample windows provided, so as to afford a clear view of operations on the bridge, including roadway, sidewalks and bike paths, and on the waterway.

Provision shall be made for the climate control apparatus to be installed in the operator's house by the Owner or by the Contractor, as may be specified.

Floors in rooms containing electrical equipment, such as motor control centers, control cabinets, or control console shall be covered with vinyl, rubber tile, or other electrically insulating materials as specified on areas surrounding such electrical equipment.

1.7 SPECIAL REQUIREMENTS FOR CONTRACTOR-SUPPLIED INFORMATION AND EQUIPMENT

1.7.1 Drawings and Diagrams

1.7.1.1 DRAWINGS

The contract documents shall specify that the Contractor shall furnish complete erection, shop detail and working plans of the structural and machinery parts, and electrical equipment. The Contractor shall make an assembly drawing and detail drawings of the machinery, including the electrical schematic wiring diagrams and/or conduit diagrams needed for operation of the complete mechanical system. These drawings shall be so complete that the machinery parts may be duplicated without reference to patterns, other drawings, or individual shop practice. Approval by the Engineer of working drawings shall not relieve the Contractor of responsibility for errors in dimensions, tolerances, or workmanship.

Where hydraulic power is specified, the Contractor shall furnish hydraulic control circuit and piping diagrams, hydraulic power unit layouts, and all assembly and detail drawings, including the electrical schematic wiring diagrams and conduit diagrams that are needed for the complete hydraulic system. The drawings shall be so complete that the hydraulic components can be replaced

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C1.6.2

For Swing or lift bridges, the Owner may locate the control house or provide an auxiliary control station off the movable span to assure safe access at all times.

For Swing bridges, the Owner should consider providing a boat or other safe means of emergency access to the swing span from the shore.

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without reference to original equipment, and shall conform to NFPA Hydraulic Standards.

The Contractor shall furnish the Engineer with the number of copies of working drawings as specified in the contract documents. Tracings or transparencies of shop detail drawings will be furnished unless otherwise specified by the Engineer.

1.7.1.2 WIRING DIAGRAMS, OPERATOR'S INSTRUCTIONS, ELECTRICAL AND MECHANICAL DATA BOOKLETS AND LUBRICATION CHARTS

The contract documents shall specify that the Contractor furnish the number of bound copies of a booklet containing descriptive leaflets and drawings covering all items of the electrical equipment, as specified in the contract documents. These booklets should contain:

- catalogue numbers;
- printed or type written statements prepared by the manufacturers of the equipment covering the proper methods of adjusting, lubricating, and otherwise maintaining each item;
- speed-torque-current curves for the span-operating motors for each point of speed control;
- a concise statement of the necessary operating functions in proper sequence;
- a detailed description of the functions of each item in connection with the various operating steps;
- reduced, reproduced copies of all wiring and conduit diagrams and drawings of the control console and switchboards; and
- a list of spare parts furnished.

The booklet shall contain a table of contents and shall designate each wire and item of equipment by the numbers on the wiring diagrams.

The Contractor shall also be required to furnish the specified number of copies of a similar booklet for the mechanical equipment, which shall include lubricating charts showing the locations of all lubricating fittings and other points of lubrication. Framed, sealed copies of the lubrication charts shall be mounted in appropriate places on the bridge.

The Contractor shall be required to provide the specified number of bound copies of a similar booklet for

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electronic control equipment, including the program operating instructions.

The Contractor shall be required to furnish the specified number of bound copies of a similar booklet for the hydraulic equipment conforming to the requirements of NFPA Hydraulic Standards.

1.7.2 Tools, Maintenance and Training

The Contractor shall furnish two sets of wrenches to fit heads and nuts of all bolts for the machinery, together with a suitable work bench, machinist's vise, and wall racks for the storage of equipment and spare parts.

Hydraulic hose crimping equipment and other special hydraulic equipment tools shall be furnished when hydraulic systems are installed, along with portable pressure gauges.

As a minimum, the Contractor shall provide two video cassettes covering all mechanical and electrical aspects of bridge operation for operator and maintenance training.

1.8 DEFECTS AND WARRANTEES

The following provision may be specified in the contract document where allowable under local law.

If any defects due to faulty workmanship or erection, or defective material, in any mechanical or electrical equipment or due to any design prepared by the Contractor are found within one year after the date of acceptance, the Contractor shall remedy such defects at his own expense. If necessary, the Owner may remedy such defects at the expense of the Contractor.

1.9 ACCESS FOR MAINTENANCE

Suitable access to all mechanical, hydraulic, and electrical components shall be considered during design for future repair, maintenance, or replacement of movable bridge equipment.

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C1.7.2

Some owners prefer that the Contractor provide technical manuals and/or on-site training of operations and maintenance personnel.

SECTION 3 - TABLE OF CONTENTS - SEISMIC DESIGN

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3.1 SCOPE

Seismic loads and movements should be considered in the structural members, and for the mechanical, electrical, and hydraulic equipment system design of movable bridges, in the open, closed, and intermediate positions. The provisions of the AASHTO LRFD Bridge Design Specifications shall apply, except as supplemented herein.

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C3.1

Movable bridges are relatively complex and irregular structures comprised of fixed and moving structural units, drive machinery and other operating systems that include structural, mechanical, electrical and hydraulic components. In addition to special structural and operating features applicable to all movable bridges, each movable bridge type has its unique seismic characteristics. Also, the seismic behavior of movable bridges varies depending on their position during an earthquake.

There is very little data available on the seismic performance of movable bridges. A review of bridge damage reports during past earthquakes has not found any reports of seismic damage to highway movable bridges. The only reported seismic damage identified occurred on a railroad vertical lift bridge during the Loma Prieta Earthquake as described in Appendix A. Other case studies of seismic behavior of movable bridges are summarized in Appendix B.

3.2 DEFINITIONS

Design Level Earthquake - The earthquake criteria specified in Article 3.10 of the AASHTO LRFD Bridge Design Specifications.

Operational Level Earthquake - Earthquake level corresponding to a low to moderate seismic event used to ensure that a bridge remains operational, i.e., movable.

3.3 PERFORMANCE CRITERIA

Because of the complex interaction of the various systems of a movable bridge, the owner and the designer should establish seismic performance goals consistent with the importance of the bridge using the guidelines below and in the following sections:

- bridges that need to remain functional immediately after a seismic event should be regarded as “critical”,
- as a minimum, bridges that can only be closed to traffic for a limited period of time for repairs after a design level event should be regarded as “essential”,
- bridges that can be closed to marine and highway traffic for an extended period of time without significant safety or economic impacts may be considered “other” bridges.

Performance criteria for substructure design should include limits on displacements and inelastic deformation based on the displacement and deformation tolerances of the superstructure.

Electrical and hydraulic conduits, hoses and piping

C3.3

Movable bridges are sensitive to distortions and misalignments. In order to remain functional after a seismic event, structural vibrations, displacements and deformations need to be limited. Critical or essential bridges shall be designed and detailed to reduce or minimize inelastic deformations during the design earthquake.

Substructure vibrations and inelastic deformations can adversely affect the operation and survival capacity of the superstructure. For example, substructures that are too flexible may amplify the response of bridge spans with counterweights above the roadway that have relatively large natural periods.

The designations “critical”, “essential” and “other” are related to response modification factors and minimum analysis methods in Articles 3.10.7 and 4.7.4 of the AASHTO LRFD Bridge Design Specifications, respectively. Further guidance on response modification factors is provided in Article 3.4.4 herein.

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should be supported and connected so as to resist earthquake effects, and to accommodate bridge movements during the seismic event.

3.4 SEISMIC LOADS, LIMIT STATES AND RESPONSE MODIFICATION FACTORS

3.4.1 Design Loads

As specified herein, seismic design may be based on either:

- a single event as specified in the AASHTO LRFD Bridge Design Specifications, designated the Design Earthquake, or
- two level events involving use of both the Design Earthquake and a lower level event corresponding to a shorter return period acceptable to the owner, designated the Operating Earthquake.

The return period for the operational level earthquake should be determined based on the importance of maintaining the bridge operational with respect to vehicular and marine traffic. As an alternative to defining the operational level earthquake, half of the demand values associated with the design level earthquake may be used.

When the movable spans are in one position, either open or closed, over 90 percent of the time, one half of the seismic loads due to the design level earthquake may be used as the design forces for the other position. Seismic loads resulting from the operational level earthquake should not be reduced. During bridge operation, seismic effects on operating components may be ignored.

For bascule type bridges, seismic loads should include vertical accelerations.

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C3.4.1

Designing a movable bridge to remain operational after a design level earthquake may not always be practical or necessary. However, movable bridges that may cause serious navigation and/or highway traffic interruptions need to be operational even after a low to moderate earthquake. Therefore, an operational level earthquake is needed to ensure that components related to the operation of the bridge do not suffer damage that would cause the bridge to be out of service during lower level ground motions.

The use of an operational level earthquake to minimize damage during low to moderate earthquakes has gained more and more acceptance in seismic evaluation and design projects. The design criteria for the Badger Avenue vertical lift bridge in Los Angeles, California, for example, included a lower operational level earthquake defined as an event with an intensity having a 50 percent probability of exceedance in 50 years - see Appendix B. The peak rock acceleration associated with this event is about half of the peak rock acceleration of an event with an intensity having a 10 percent probability of exceedance in 50 years. In the central and eastern United States the ratio between the accelerations associated with a 10 percent and 50 percent probability of exceedance in 50 years events is significantly lower. Nevertheless, a ratio of about two between the acceleration coefficients corresponding to the higher and the lower level events has also been used on a seismic project in Rhode Island (Sahakian, 1994).

When a movable bridge is kept in one position most of the time the risk that a seismic event will occur when the bridge is in the other position is low. The reduced exposure to earthquakes for a given bridge position may be taken into account by using lower seismic design forces for that position. The recommendation to use one half of the seismic loads corresponding to the design level earthquake is relatively conservative, and is intended for new construction only. It aims to account for possible changes in operation practices during the life of the bridge, identify vulnerabilities in a particular bridge position and lead to the use of cost-effective ways for improving overall seismic behavior. For example, providing lateral support to the ends of swing spans and to the counterweight of trunnion

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bascules when in the open position, can significantly reduce the loadings of center pivots and trunnions.

3.4.2 Limit States

The Design Earthquake and the Operating Earthquake shall be considered at the extreme event limit state. For structural components, including structural supports for other systems, resistance shall be determined as specified in the AASHTO LRFD Bridge Design Specifications. For other components, resistance corresponding to the extreme event limit state shall be used if provided, otherwise, the overload limit state resistance should be used.

3.4.3 Application

3.4.3.1 CRITICAL AND ESSENTIAL BRIDGES

Design of superstructure structural components as defined in Article 3.4.2 shall be based on the Design Earthquake.

Design of the substructure shall be based on the Design Earthquake.

Design of the operating system, including locking and lateral restraint mechanisms shall be based on either:

- a strategy of protecting the operating components, in both the open and closed positions, with a combination of brakes, hydraulic actuators locking and lateral restraint devices designed and detailed to resist the inertial forces developed by the Design Earthquake, or

- if practical braking, hydraulic actuators, locking and lateral restraint devices cannot be designed to resist the Design Earthquake, the operating components of "critical" bridges should be designed to resist the Design Earthquake in both the open and closed position, and the operating systems of "essential" bridges may be designed for the Operating Earthquake.

3.4.3.2 OTHER BRIDGES

All components of the operating system of bridges designated as "other bridges" may be designed for the Operating Earthquake.

3.4.4 Response Modification Factors

Response Modification Factors, R-factors, should be taken as follows for the design level earthquake:

- R-factors for steel superstructure components should

C3.4.4

The relatively low R-factors for the superstructure member are intended to minimize distortions and misalignments. In addition to being critical vertical load carrying components, trunnion joints, shafts, counterweight

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be taken as 1.0 for main structural members and connections for mechanical and hydraulic components, 0.8 for critical connections, including trunnion joints, shafts, counterweight sheaves and counterweight supports, and 1.2 for secondary members.

- The R-factors for substructure design may be taken as specified in Article 3.10.7 of the AASHTO LRFD Bridge Design Specifications. However, consideration should be given to lowering R-factors to reduce deformations. The use of plastic hinging should also be carefully considered with respect to misalignment.

R-factors greater than 1.0 should not be used for the operational level earthquake.

3.5 SEISMIC ANALYSIS

3.5.1 General

Minimum analysis level for movable bridges in Seismic Zones 2, 3 and 4 should be multimode elastic analysis using a three-dimensional model. The fully open, closed and several open positions should be analyzed.

Modeling assumption should consider the unique structural, operational, and seismic load distribution characteristics of each bridge type, as specified in Article 3.5.2.

The analysis should identify clear load paths that transmit the inertia loads from the counterweights and the moving spans to the support structures and foundations through main members, cross frames and lateral bracing systems and connections, for design and detailing purposes.

3.5.2 Seismic Load Distribution

3.5.2.1 LOAD PATH

A viable load path shall be established to transmit the inertia loads from the moving spans, the fixed spans and the counterweights, as applicable, to the foundation.

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sheaves and counterweight support joints could be subjected to large and unpredictable load concentrations.

Consideration of reduced R-factors for substructure is based on the operation, movement and alignment tolerances of the superstructure. Similarly, plastic hinging should be avoided wherever practical.

C3.5.1

Movable bridges are articulated structures that are usually irregular and asymmetrical in geometry and support conditions. The counterweights are large concentrated masses that can cause significant overturning moments and torsional effects. There is a large concentration of loads at joints between moving units, such as the trunnions in bascule bridges, that are subjected to bending, shear and torsional loading from the moving units. The seismic characteristics of movable bridges vary with their position. Analysis for several open positions can provide a better understanding of the bridge response and identify vulnerable locations.

C3.5.2.1

Each bridge type has its unique seismic characteristics. Knowledge of the interaction between moving units, and of the behavior of connections, end support conditions and other specific details is necessary in order to make correct modeling assumptions. Good understanding of the seismic load distribution characteristics and of the location of vulnerable components is also needed for proper analysis assumptions.

The bridge types specifically addressed herein include bascule, vertical lift and swing type bridges. The bascule bridge type is further divided into trunnion, rolling leaf and

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3.5.2.2 TRUNNION BASCULE BRIDGE

Analysis for the open position and during operation should consider the following:

- When the leaf is in the open position its whole weight is only supported by the trunnions.
- Rotation of the span is resisted by the rack pinion and the drive machinery.
- Movement in the longitudinal direction is resisted by the trunnions resulting in in-plane loading of the girder webs.
- Trunnions are supported by trunnion bearings connected either directly or through a trunnion beam to the pier. Some resistance to larger longitudinal movements is also provided at the contact surface between the rack pinion and the curved rack on the leaf.
- Movement in the transverse direction is also resisted by the trunnions, but it results in out-of-plane loading of the girder webs.
- The lateral loads on the trunnions and the trunnion bearings are resisted by the piers.
- Due to the asymmetrical nature of the leaf and the location of the trunnion bearings on the pier, the lateral seismic forces can result in both shear and torsion effects.

Analysis for the closed position should consider the following:

- When the leaf is in the closed position its front end is restrained by span locks and centering devices, but these elements usually offer only limited lateral resistance and almost no longitudinal resistance.
- When tail locks are also used, the stability of the leaf is improved. Movement in the longitudinal direction is mainly resisted by the trunnions.

The lateral loads within the leaf should be resisted by the lateral bracing system of the girders which transfers

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heel trunnion bascules. Although the heel trunnion bascule bridge type is not commonly used for new bridge construction, it has also been included, for bridge evaluation purposes.

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lateral seismic loads from the moving span and counterweight to the trunnions.

3.5.2.3 ROLLING LEAF BASCULE BRIDGES

Analysis for the open position and during operation should consider the following:

- When the leaf is in the open position it is supported only at its contact with the track girder. Rotation of the span is resisted by the drive pinion and the drive machinery reacting against the rack frame.
- When span locks are provided for the open position, the span locks also resist span rotation.
- Movement in the longitudinal direction is resisted by the pintles on the track girder. Some resistance to larger longitudinal movements is also provided at the contact surface between the pinion teeth and the fixed rack.
- Movement in the transverse direction is resisted by the pintles on the track girders, and the lateral loads on the segmental girders are resisted by the piers.
- The loading of the bascule piers depends on the position of the span.

Analysis for the closed position should consider the following:

- When the movable span is in the closed position its front end is restrained by span locks and centering devices, but these usually offer limited lateral resistance and almost no longitudinal resistance.
- Movement in the longitudinal direction is mainly resisted by the pintles on the track girder.

The lateral loads within the moving span are resisted by the lateral bracing system between girders or trusses which transfers the inertia loads from the span, counterweight and machinery to the segmental girders.

3.5.2.4 HEEL TRUNNION BASCULE BRIDGES

3.5.2.4.1 Bascule Span Response

Analysis for the open position and during operation should consider the following:

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- When the bascule span is in the open position its entire weight is supported by the main trunnions. Rotation of the span is resisted by the racks mounted on the operating struts, the pinions and the drive machinery. The reaction is transferred to the fixed truss span.

- Movement in the longitudinal direction is resisted mainly by the main trunnions. Some resistance is provided by the operating struts at the contact points between the pinions and the racks on the struts.

- Movement of the bascule span in the transverse direction is resisted by the main trunnions. Some lateral resistance is also provided by the bracing members of the counterweight links, reacting against the counterweight frame and the tower span.

Analysis for the closed position should consider the following:

- When the bascule span is in the closed position its front end is restrained by span locks and centering devices, but these offer only limited lateral resistance and almost no longitudinal resistance.

- Movement in the longitudinal direction is mainly resisted by the main trunnions. Some resistance to larger longitudinal movements may also occur at the contact surface between the rack on the operating strut and the pinion teeth.

3.5.2.4.2 Counterweight Frame Response

Analysis of the counterweight frame should consider the following:

- The counterweight frame is supported by the counterweight trunnions. It is balanced by the reaction in the counterweight links on one side and by the weight of the counterweights on the other.

- All lateral loads are resisted by the counterweight trunnions which are supported by the tower span. The members supporting and bracing the counterweights can be subjected to relatively high loads.

3.5.2.4.3 Tower Span Response

Analysis of the tower should consider the following:

- The tower span supports the counterweight frame and the drive machinery. The main lateral loading of the

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tower span comes from the counterweight frame at the counterweight trunnion connection. The concentrated loads applied at the top of the tower span can be quite high and cause significant shear and overturning moments.

- The tower frame usually offers reasonable resistance in the longitudinal direction, but is relatively flexible in the transverse direction.
- To provide the necessary vertical clearance to vehicular traffic, lateral loads are resisted through frame action that results in bending of the tower legs.

3.5.2.5 VERTICAL LIFT BRIDGES

3.5.2.5.1 Lift Span Response

Analysis for the open position and during operation should consider the following:

- Lift towers are subject to the lateral reactions from both the lift span and the counterweights. These loadings can induce high shear, torsion and bending moments on the towers.
- The lift span is balanced by the counterweights through the rope and the balance chain system.
- Unbalanced vertical seismic effects are resisted by sheaves, trunnions and the drive machinery. Movements of the lift span in the horizontal direction are restrained by the tower guides.
- Horizontal movements of the counterweights are also restrained by guides mounted along the tower.
- Tower loading is resisted at its base connection through anchor bolts which are subject to both shear and tension forces.

Analysis for the closed position should consider the following:

- When the lift span is in the closed position, its ends are restrained by the span locks, centering devices and the span guides which are tapered providing for a tighter support. Movement in the longitudinal direction is resisted at only one end of the span.
- Lateral bracing system within the lift span transfers the lateral loads from the lift span to the towers.

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3.5.2.5.2 Lift Tower Response

Analysis of the lift tower should consider the following:

- The lift towers support the weight of the whole bridge, the lift span, the counterweights, the rope system and the trunnions, and the drive machinery. They also resist the longitudinal and lateral loadings of the lift span and the counterweights.
- In the longitudinal direction the inertia of the whole lift span is usually resisted by only one tower.
- When the lift span is up the loads from the span are applied at the top of the tower and the loads from the counterweight are applied at the bottom of the tower.
- Lift towers are subject to concentrated lateral reactions from both the lift span and the counterweights at varying elevations.
- Loadings induce shear, torsion and bending moments on the towers.
- Tower loading is resisted at its base connection by anchor bolts which are subject to both shear and tension forces.
- The tower frame acts like a truss in the longitudinal direction offering reasonable resistance, but it is relatively flexible in the transverse direction due to bending of the lower tower legs, which are not braced in order to provide the necessary vertical clearance for vehicular traffic.

3.5.2.6 SWING BRIDGES

Analysis for the open position and during operation should consider the following:

- In the open position the weight of the swing span is supported on the center bearing in center bearing type bridges and on tapered rollers in rim bearing type bridges.
- Rotation of the span is resisted by the drive pinions reacting against the curved rack mounted on the pier.
- Horizontal span movements are resisted by the center bearing in center bearing bridges and by the rim bearing assembly and the small center bearing in rim bearing bridges.
- Tilting of the span is resisted by the balance wheel

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assembly in center bearing bridges and by the rim bearing assembly in rim bearing bridges.

- If span locks for the open position are provided, part of the torsional, shear and overturning loads could be resisted by the span locks, improving the stability of the span.

Analysis for the closed position should consider the following:

- When the span is closed the wedges participate in resisting vertical loads but offer little horizontal resistance.
- Centering latches can improve the shear capacity of the ends of the span and increase its stability. However, most centering latch designs provide very little shear resistance.

The horizontal loading of the swing span is transferred through its lateral bracing system.

3.6 DESIGN AND DETAILING GUIDELINES

The design and detailing of movable bridges should account for the inherent seismic vulnerabilities of each bridge type, as described in the commentary.

Due to the articulated nature of movable bridges and the inherent vulnerability of connections and contact surfaces between moving parts, special care is needed in design and detailing. The following design and detailing issues should be considered:

- Add redundancy to the structure and the operating systems.
- Use details that are not very sensitive to misalignments.
- Reduce susceptibility of steel members to local and overall buckling.
- Connections between moving parts should have adequate load transfer and movement capacity.
- Provide members directly supporting the counterweight and their connections with sufficient capacity to resist large in-plane and out-of-plane concentrated loads.
- Provide adequate lateral bracing systems for the

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As a minimum, design needs to consider the following seismic vulnerabilities:

TRUNNION BASCULE BRIDGES:

- High concentration of loads due to the inertia of the whole leaf at the trunnions make the trunnion assemblies and their connections to the girders and the pier very vulnerable. In addition, the girders are vulnerable due to out-of-plane loading of the web at the location of the trunnion connection.
- Seismic forces trying to rotate the span when open may break the pinion teeth, or damage reducer gears, couplings, motor, housings and mounting anchor bolts.
- When the bridge is in the closed position the span locks and centering devices, and their connections to the span have relatively low shear capacity, and could fail early offering relatively little additional resistance to the leaf.
- Counterweight boxes add significant mass to the tail end of the moving span resulting in large inertia forces in its supporting members and connections, especially during lateral loading.
- Asymmetrical characteristics of the leaf and its

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moving span and the counterweight support frame.

- Include counterweight stops and lateral restraining features if possible, for both the open and closed position.
- Detail structural, mechanical and electrical components to allow for movements between adjacent structural units.
- Use self aligning bearings.
- Provide span locks for both the open and closed position with adequate capacity to resist seismic loads.
- Design machinery, electrical and hydraulic components so that they could be easily replaced.
- Include suitable means of access for structural and mechanical inspections.
- Reinforce trunnion bascule girder webs at trunnion connections.
- Design the connection of the trunnion bearings to the supporting pier in trunnion bascule bridges with adequate capacity to resist large shear, moment and torsional loadings.
- Provide adequate lateral support to the segmental and the track girders of rolling lift bascules.
- Provide tower leg members at the base of the tower in vertical lift bridges with adequate axial and transverse bending capacity to resist large overturning moments.
- Design the connection of the tower legs and base to the supporting pier in vertical lift bridges to have adequate capacity.
- Provide jacking locations for realigning the towers of vertical lift bridges.
- Add catch ring blocks at the pivot pier in swing bridges.

COMMENTARY

supports can result in torsional loading of the counterweight which can further increase the forces in the supporting members.

ROLLING LEAF BASCULE BRIDGES:

- Counterweights add a large concentrated mass at the tail end of the moving span and cause an abrupt change in stiffness along the span.
- Members and connections supporting the counterweight could be subjected to significant forces due to large inertia loads and torsional effects caused by the asymmetrical nature of the moving span and its supports.
- If the bridge is narrow it could be susceptible to overturning in the transverse direction.
- On through truss bascules the counterweight is located above the roadway on top of the far end of the segmental girders. Due to the vertical clearance requirements for traffic the center of gravity of the counterweight can be quite high, resulting in large overturning forces. In addition, the transverse capacity of the span at the location of the counterweight is relatively low. Lateral loading at this location can result in high out-of-plane bending of the connections of the counterweight to the segmental girders. Failure of these connections can lead to significant damage including the drop of the counterweight.
- Excluding friction, the pintles on the tread plate of the track girder are the only elements that resist the lateral loads from the moving span. These loads could be quite high due to the combined effect of the shear and torsion forces on the moving span. The pintles react against the track girders resulting in transverse loading of their top flange. The track girders and their lateral bracing system may not have adequate capacity for large out-of-plane loadings.
- Drive pinions and machinery could also be damaged by seismic forces trying to rotate the span when open, especially if there are no span locks for the open position. The drive pinions react against the rack frames which may not have adequate capacity for high seismic loads.
- The span locks and centering devices are normally designed for relatively low nonseismic shear loads and could be damaged providing limited resistance. These devices should be designed for seismic loads as well.

SPECIFICATIONS

COMMENTARY

HEEL TRUNNION BASCULE BRIDGES:

- The high concentration of loads due to the inertia of the bascule span at the main trunnions make the main trunnion assemblies and their connections to the base of the tower span very vulnerable.

- The gusset plate connection of the bascule span to the trunnions has relatively low resistance to transverse loads which induce out of plane bending in the gussets above the trunnions.

- The counterweight trunnions are also vulnerable because of the high concentration of loads due to the inertia of the counterweight.

- Seismic forces rotating the span when open may damage the rack on the operating struts, the drive pinions and machinery. When the bridge is in the closed position the span locks and centering devices which are normally designed for relatively low shear loads are also vulnerable, offering relatively little lateral resistance.

- Large inertia loads may damage the members supporting and bracing the counterweights and their end connections. In addition, the members linking the counterweight frame to the bascule span and their pinned end connections are also vulnerable. These are critical elements since their failure could result in the drop of the bascule span and the counterweights.

- Tower span, which supports the counterweights, resists lateral loads through frame action, offering relatively little resistance in the transverse direction. Also, the high location of the center of gravity of the counterweights when the bridge is closed can result in high overturning moments at the tower base. The tower span legs and the tower base connections are subject to both bending and axial loads.

VERTICAL LIFT BRIDGES:

- Vertical lift bridges have relatively good symmetry and regularity characteristics and usually do not have critical locations with very high and unpredictable load concentrations such as the trunnions in bascule bridges.

- Lift bridges have better redundancy characteristics than other movable bridges.

SPECIFICATIONS

COMMENTARY

- Before engaging the tower guides, the lift span and the counterweights act as pendulums and their loading is minimal.

- Even after contact with the tower is made the freedom of limited movement may have a beneficial effect on the seismic response.

- Towers and their base connection support the whole weight of the bridge and resist the lateral and longitudinal reactions of the lift span and the counterweights.

- Lateral reactions are applied to only one side of the tower resulting in torsional loads in the tower and pier.

- The seismic reactions at the tower base could be quite large since the towers need to be tall in order to provide the vertical clearance required for navigation.

- When the lift span is in the up position and the reaction of the whole span is applied to one tower in the longitudinal direction, the loading of the tower could be quite severe. However, during larger movements both towers will probably be engaged. Alternatively larger movements can be minimized and both towers still engaged if details include spring-loaded, shock-absorbing span guide rollers that effectively connect the two towers through the span lift by holding the rollers against the towers.

- Loadings of the towers can induce high shear, torsion and bending moments on the tower base connection that could exceed the capacity of the anchor bolts.

- Drive machinery is usually not as susceptible to seismic loads as in other movable bridges because it is only subjected to unbalanced vertical loads due to seismic loading.

- The lift span and the counterweights are horizontally restrained by the tower guides.

SWING BRIDGES:

- Center bearing bridges in the open position without end supports are usually the most vulnerable component as they have very little resistance to shear and overturning.

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COMMENTARY

- Rim bearing bridges have better lateral resistance than center bearing bridges, but they are also quite vulnerable without end supports.

- In the closed position, the stability of the span is improved by wedges and centering latches which would need to be designed for seismic loads.

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APPENDIX A3: MOVABLE BRIDGE DAMAGE DURING PAST EARTHQUAKES

There is very little data available on the seismic performance of movable bridges. A review of bridge damage reports during past earthquakes has not found any reports of seismic damage to highway movable bridges.

The only reported seismic damage identified so far, occurred on a railroad vertical lift bridge during the Loma Prieta Earthquake (October 17, 1989). A postseismic inspection for the Southern Pacific Railroad (currently Union Pacific Railroad) found minor damage at Bridge No. 33.31, Martinez, CA. The bridge is located 25 miles northeast of San Francisco at Suisun Bay. It is a 5,603 foot long double track structure, with a vertical lift span. The bridge was built in 1929. The earthquake caused severe bending of the counterweight guide rails (2- L 4 x 4 x 9/16). At the time of the earthquake the bridge was in a lowered position (counterweights raised). The guide rails were damaged only at the lower counterweight guide castings. The damage was not found for several days until a bridge tender reported problems with rough operations. As a result of this finding, another Southern Pacific vertical lift bridge, the Napa River Bridge, which was designed based on the seismic criteria of the 1970's, was also inspected. There, the span is kept in a raised position. No damage was found in the span guides or the counterweight guides.

There were three other railroad bascule bridges in the area affected by the Loma Prieta Earthquake, but there were no reports of damage.

APPENDIX B3: SEISMIC DESIGN AND EVALUATION CASE STUDIES

B3.1 TRUNNION BASCULE BRIDGES

B3.1.1 Double-Leaf Trunnion Bridge

A seismic evaluation of a double-leaf trunnion bridge with a total span length of about 130 feet between center lines of trunnions and 28-foot roadway is reported in (Lai, 1994). The bridge was evaluated for an earthquake with a 0.3g ground acceleration. The multimode spectral analysis performed identified flexural rotation of the leaf about the trunnion axis as the first mode of vibration with a period of about 2.2 seconds, and torsion of the leaf as the following three modes of vibration. A maximum elastic moment in the trunnion of 83 percent of the maximum moment due to self weight and an unbalanced moment of the span of 31 percent of the maximum unbalanced dead load moment were calculated. It was recommended that the design of the trunnions and the design of the rack pinion and the drive machinery include such additional loads. Due to the high mass participation obtained in the vertical direction, it was also recommended to include vertical ground motion in the seismic analysis of trunnion type bascule bridges.

B3.1.2 Ballard Bridge, Seattle, Washington

The seismic evaluation of the Ballard Bridge in Seattle, Washington is reported in (Korpi, 1994). The Ballard Bridge is a double-leaf truss trunnion bascule with each movable leaf being 101 feet. The bridge carries four lanes of traffic and has sidewalks on both sides of the roadway. The spacing of the bascule trusses is 44 feet. The trunnion bearings are supported by a trunnion girder that connects the movable leaves to the concrete trunnion columns. The bridge was evaluated in the closed position in which it is most of the time. To increase resistance in the longitudinal direction, it was recommended to reinforce the ends of the trunnion girder and to add counterweight restrainers. The restrainer consisted of large plates bolted to the bottom of the counterweight box and to the pier walls. The restrainers were designed to engage when the bridge closes. It was also recommended to strengthen the lateral bracing of the counterweight and leaf.

B3.1.3 University Bridge, Seattle, Washington

The seismic evaluation of the University Bridge in Seattle, Washington is reported in (Lem, 1996). The University Bridge is a 291-foot-long double-leaf truss trunnion bascule bridge. It carries four lanes of traffic and has sidewalks along both sides. Typical out-to-out width of the bridge is 75 feet. Each bascule leaf is 143 feet long from midspan to the back of the counterweight. The counterweight is housed inside a lightly reinforced concrete bascule pier under a 45-foot span. The bridge had no span locks tying the leaves together. The seismic loading used was based on the AASHTO Standard Specifications for Highway Bridges using an Acceleration coefficient of 0.3 and a Soil Coefficient of 1.2. The seismic analysis included response spectrum analyses of global and local models for the closed position only. Based on the results of the analysis it was recommended to reinforce the leaves by providing an additional lateral bracing system, strengthen the connection of the trunnion bearings to the supporting pier and strengthen the pier walls.

B3.2 ROLLING LIFT BASCULE BRIDGES

B3.2.1 Seismic Evaluation Case Study

A seismic evaluation of a double-leaf rolling lift type bascule bridge that provides a horizontal channel clearance of 150 feet and has a 28-foot roadway is reported in (Lai, 1994). Elastic response spectral analyses were conducted for the bridge at the closed position, the ready to open position, and at the 40 degree open position. The multimode spectral analysis performed identified the flexural mode of the span about a horizontal axis as the first mode of vibration with a period of about 1.7 seconds in the lowered position and about 2.1 seconds at 40 degrees open. Torsion of the span was identified as the following three modes of vibration. The bridge dynamic characteristics were found to change at different opening positions, which is different from what was observed for a trunnion type bridge. The reason for the change in dynamic characteristics is the shifting of the location of support as the bridge rolls along the track girder. The results of the analysis indicated that seismic loads can cause a significant increase in the unbalanced moment of the span. At the 40 degree open position, an unbalanced moment due to seismic loads of 70 percent of the unbalanced moment due to dead load was calculated for an earthquake with a 0.3g ground acceleration. Based on these results it is recommended that the additional unbalanced moment be taken into account in the design process. Due to the high mass participation obtained in the vertical direction, it is also recommended to include vertical ground motion for the seismic analysis of rolling lift type bascule bridges.

B3.3 HEEL TRUNNION BASCULE BRIDGES

B3.3.1 Third Street Bascule Bridge, San Francisco, California

A seismic evaluation of the Third Street Bascule Bridge in San Francisco, California which is a single leaf heel trunnion bascule is reported in (Schamber, 1997). The length of the bascule span is 142 feet between the center lines of the main trunnion and the front end live load bearing. The distance center-to-center between trusses is 53.25 feet and the total width of the bridge is approximately 81 feet. Four 2-inch diameter anchor bolts were used to connect the tower base to the top of each pier. Response spectrum analyses were performed for the closed and the open positions. The first natural periods obtained for the closed position were 1.90 seconds for the counterweight frame and tower span for the transverse direction, and below 0.72 seconds for the longitudinal direction. The first natural periods obtained for the open position were 1.76 seconds for the transverse direction and 2.39 seconds for the longitudinal direction. The analyses found the anchor bolts at the tower bases to be most vulnerable. Other critical components included the counterweight trunnions and the main trunnions in the closed position, and the link pins, pinion and racks on the operating struts, the counterweight trunnions and the main trunnions in the open position.

B3.3.2 Steamboat Slough Bridge, Rio Vista, California

The seismic evaluation of the Steamboat Slough Bridge located between Rio Vista and Sacramento, California is reported in (Mathur, 1996). The Steamboat Slough Bridge is a double-leaf heel trunnion bascule. A multi mode spectral analysis was performed for the standard Caltrans ARS Spectrum corresponding to a Horizontal Bedrock Acceleration of 0.2g. This spectrum has a peak acceleration of 0.7g at a period of 0.5 seconds. The fundamental period obtained was about 1 second. It was found that the seismic demands on members that support the counterweight are high, exceeding their capacity, and that the seismic demands on members of the bascule span are small relative to their capacity. The critical loadings of the main legs of the tower span included large shear demands in their weak plane and combined compression and bending. Also, the riveted connection at the counterweight link near its connection to the link pins was found not to have the capacity to transfer the tensile forces in the member.

B3.3.3 Badger Avenue Bridge, Los Angeles, California

A seismic evaluation of the Badger Avenue Bridge in Los Angeles, California, a double leaf heel trunnion bascule, was made before its replacement with a vertical lift bridge (Syed, 1996). The bridge carried both railroad and highway traffic, and the total length of the movable spans between the center lines of the main trunnions was 219 feet. It was evaluated for a two-level probabilistic based ground motion criteria. The criteria included a lower operational level earthquake (OLE) defined as an event with an intensity having a 50 percent probability of exceedance in 50 years, and an upper, contingency level earthquake (CLE), defined as an event with an intensity having a 10 percent probability of exceedance in 50 years. The performance criteria for the OLE was that the structure remain within the elastic range with little or no damage. For load demands due to the CLE, the bridge was expected to be operational with minimal down time for repairs. Stress levels in truss and tower members were allowed to reach up to 90 percent F_y. Few secondary members were allowed to go into the plastic range with repairable damage. Peak rock horizontal acceleration magnitudes of 0.2g and 0.4g were used for the OLE and CLE, respectively. A multi mode spectral analysis was performed. It was found that the bascule span and the counterweight truss members were most vulnerable in the open position. Under both OLE and CLE events, significant overstress was found to occur in the tower span truss and some bracing members, all bottom lateral braces of the bascule and the counterweight trusses, and several top lateral members of these trusses. Particularly susceptible were the members forming the link between the bascule and the counterweight trusses which are critical components for the stability of the bridge. The large forces in the operating struts obtained under both OLE and CLE were larger than what the existing drive machinery, and even a new drive system, was able to handle.

B3.4 VERTICAL LIFT BRIDGES

B3.4.1 Badger Avenue Vertical Lift Bridge, Los Angeles, California

The seismic design of the vertical lift replacement bridge of the Badger Avenue Bridge in Los Angeles, California, is reported in (Syed, 1996). The length of the lift span is 220 feet and the towers are 230 feet tall. The distance between the center lines of the front tower columns and the rear tower columns was set at 50 feet to match the exiting foundations. The bridge was designed for a two-level probabilistic based ground motion criteria. The criteria included a lower operational level earthquake (OLE) defined as an event with an intensity having a 50 percent probability of exceedance in 50 years, and an upper, contingency level earthquake (CLE), defined as an event with an intensity having a 10 percent probability of exceedance in 50 years. The performance criteria for the OLE was that the structure remain within the elastic range with little or no damage. For load demands due to a CLE, the bridge was expected to be operational with minimal down time for repairs. Stress levels in truss and tower members were allowed to reach up to 90 percent Fy. A few secondary members were allowed to go into the plastic range with repairable damage. Peak rock horizontal acceleration magnitudes of 0.2g and 0.4g were used for the OLE and CLE. A multi mode spectral method of analysis was used. The maximum horizontal displacement responses obtained at the top of the tower were about 12 inches for both the up and down positions of the lift span.

B3.4.2 Sacramento River Bridge, Rio Vista, California

The seismic evaluation and retrofit of the Sacramento River Bridge at Rio Vista, California, which is a vertical lift bridge, is reported in (Abbas, 1996). The lift span is 310 feet long and the towers rise approximately 173 feet above roadway deck. The roadway is 26 feet wide carrying two lanes of traffic. Typical tower column is connected to the pier through eight 2-1/4 inch diameter bolts, embedded approximately 8 feet in the pier. Guide rollers on the towers restrain the lift span against longitudinal and transverse movement. In the down position, the lift span rests on fixed bearings at one end, and on sliding bearings at the other end. The bridge was investigated for a "no collapse" criteria in a maximum credible earthquake of magnitude 6.75, with a PGA of 0.5g. The structure was evaluated in the lift span down position only, due to the very low probability of occurrence of a major earthquake estimated for the lift span up position. The seismic evaluation consisted of a force and displacement based demand-to-capacity ratio analysis. Global force and displacement demands on the structure were computed using linear elastic dynamic response spectrum analysis. Tower frames were also analyzed using static nonlinear lateral and dynamic nonlinear time history analyses. The time periods of the dominant modes were 1.1 seconds for the longitudinal response of the towers with the counterweight on top, and 1.15 seconds for the transverse response of the towers with the counterweight on top. The transverse response of the towers indicated frame action at the base of the towers. Capacity evaluations were conducted in accordance with the provisions of the AASHTO Standard Specifications for Highway Bridges. The static nonlinear analysis of the as-built structure showed flexural yielding of anchor bolt bearing plates followed by yielding of the anchor bolts, showing that the towers are likely to rock during a major earthquake producing large uplifts and excessive lateral displacements. It was found that increasing the connection capacity can force a soft story mechanism at the base of the towers in the transverse direction. To ensure controlled rocking response of the towers without a soft story mechanism, it was recommended to strengthen the tower columns and to add passive energy dissipators at the tower base.

SECTION 2 - TABLE OF CONTENTS - STRUCTURAL DESIGN

SPECIFICATIONS

COMMENTARY

2.1 SCOPE

2.1.1 Specifications

The structural design of movable bridges shall conform to the requirements of the latest edition of the AASHTO LRFD Bridge Design Specifications (AASHTO, 2004) including all applicable interim changes, except as modified or supplemented herein.

2.1.2 Bridge Types

2.1.2.1 GENERAL

Movable bridges shall be of the following types, unless otherwise specified by the Engineer:

- bascule span bridges,
- swing span bridges, or
- vertical lift bridges.

2.1.2.2 CONTRACT DOCUMENTS

The contract documents shall specify, or the Engineer shall determine the following:

- the type of movable bridge,
- for bascule span bridges, the type of bascule,
- for swing span bridges, the type of center. Swing span bridges should be the center bearing type, unless there is a compelling reason otherwise.
- for vertical lift bridges, the type of tower, location of the prime mover(s) and provisions for keeping the moving span level, and
- the system of emergency operation, if any, and the standby power system, if any.

2.1.2.3 PROHIBITED STRUCTURE TYPES

Pin-connected trusses shall not be used for movable spans.

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2.2 DEFINITIONS

Center Bearing Swing Bridge – Movable bridge which opens by rotating about a vertical axis, and whose span weight is supported on a thrust bearing, positioned at the movable span's center of gravity and rotation.

Design Life - Period of time upon which the statistical derivation of transient loads is based, 75 years.

Double-Leaf Bascule Bridge – Movable bridge which opens by rotating two cantilever spans, each about their own horizontal axis, and where the radius of both span's travel are tangent at the center of the channel.

Dynamic Load Allowance - An increase in the applied static force effects to account for the dynamic interaction between the bridge and moving loads.

Fatigue Limit States - Limit states relating to the initiation and/or propagation of cracks due to a repeated variation of normal stress with a tensile component.

Fixed Bridge - A bridge with a fixed vehicular or navigational clearance.

Force Effects - A deformation, stress or stress resultant, i.e., thrust, shear, torque or moment, caused by applied loads, imposed deformations or volumetric changes.

Heel-Trunnion Bascule Bridge – Movable bridge which opens by rotating a single cantilever span about a horizontal axis, The counterweight is mounted on a frame (not in the plane of the movable span) and counters the weight of the span through linkages.

Limit State - A condition beyond which the bridge or component ceases to fulfill the function for which it was designed.

Load Factor - A factor accounting primarily for the variability of loads, the lack of accuracy in analysis, and the probability of simultaneous occurrence of different loads, but also related to the statistics of the resistance through the calibration process.

Movable Bridge - A bridge with a variable vehicular or navigational clearance.

Rim Bearing Swing Bridge - Movable bridge which opens by rotating about a vertical axis, and whose span weight is supported by a ring of rollers located at a radius of usually one-half the width of the movable span from the center of rotation.

Rim Girder - A structural girder that distributes the weight of the rim bearing swing bridge's movable span to the rollers.

Rolling-Lift Bascule Bridge - Movable bridge which opens by rotating a single cantilever span about a moving horizontal axis. The counterweight and movable span are attached to a vertically curved girder that rotates the center of rotation away from the channel.

Segmental Girders - The vertical curved girder which supports the weight of a rolling-lift bascule bridge, and rotates with the movable span as the center of rotation moves away from the channel.

Service Life - The period of time that the bridge is expected to be in operation.

Single-Leaf Bascule Bridge - Movable bridge which opens by rotating a single cantilever span about a horizontal axis.

Strength Limit States - Limit states relating to strength and stability.

Stress Range - The algebraic difference between extreme stresses resulting from the passage of a load and/or the opening of a span.

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Track Girders - The horizontal flat girder that is fixed, and which supports and aligns the movement of the segmental girder of a rolling-lift bascule bridge.

Transfer Span - A bridge structure which is used for loading or off loading vehicles, pedestrians or cargo from a floating vessel in tidal or nontidal conditions under the direction of a qualified attendant. A transfer span is connected on the shore end to a pier, trestle, wharf or dock and the offshore end is directly supported or resting on a floating vessel or supported from a fixed or floating structure with a short apron to transition from bridge to vessel.

Trunnion Bascule Bridge - Movable bridge which opens by rotating a single cantilever span about a horizontal axis. The counterweight is located behind the movable span and counters the weight of the span.

Vertical Lift Bridge - Movable bridge which opens by vertically raising one movable span.

2.3 NOTATION

D	=	diameter of the segment (in.) (2.5.1.1.3)
DAD	=	operating impact specified in Article 2.4.1.2.2 applied to the maximum dead load effect in all members that are in motion, and the load effect on a stationary member caused by the moving dead load (DIM) (2.4.1.2.2)
D_{cw}	=	dead load, bridge closed, counterweight supported for repairs (lb.) (2.4.2.1)
D_o	=	dead load, bridge open in any position or closed with ends just touching (lb.) (2.4.2.1)
F_y	=	specified minimum yield strength of the material in tension (psi) (2.5.1.1.3)
L_c	=	live load on a swing bridge including dynamic load allowance, IM, bridge closed with ends just touching, with bridge considered as a continuous structure, reactions at both ends to be positive (lb.) (2.4.2.1)
L_{cw}	=	live load including dynamic load allowance, IM, bridge closed, counterweight supported for repairs (lb.) (2.4.2.1)
L_s	=	live load on a swing bridge including dynamic load allowance, IM, on one arm as a simple span, bridge closed with ends just touching (lb.) (2.4.2.1)
M_o	=	maximum forces on structural parts, including parts of the structure that support the counterweight assembly, caused by the operation of machinery, increased for machinery impact, DAM, as specified in Article 2.4.1.2.3 (lb. or lb.-in.) (2.4.2.1)
P_{LB}	=	allowable load for line bearing between segmental girder and track girder (lb./in.) (2.5.1.1.3)
W_o	=	wind load, bridge open in any position, or closed with ends just touching (lb.) (2.4.2.1)

2.4 LOADS, LOAD FACTORS AND COMBINATIONS

2.4.1 General Provisions and Limit States

The loads specified in this article are applicable to all types of movable bridges. Loads, load factors and load combinations specific to a particular type of movable bridge shall be taken as specified in Articles 2.4.2. The provisions for bascule span bridges shall also apply to rolling lift bridges.

2.4.1.1 LIVE LOAD AND DEAD LOAD

The dead load and live load shall be as specified in Section 3 of the AASHTO LRFD Bridge Design Specifications under the designations DC, DW, EL, EH ES, EV, LL, IM, BR, CE, PL and LS. The D_o and D_{cw} dead loads shall be applied as appropriate to the condition of the

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bridge at the time these dead loads are considered and are defined as follows:

D_o = the sum of DC and DW appropriate when the bridge is open in any position or closed with the ends just touching

D_{cw} = the sum of DC and DW appropriate when the bridge is closed with the counterweight supported for repairs

2.4.1.2 DYNAMIC LOAD ALLOWANCE

2.4.1.2.1 Live Load Dynamic Load Allowance (IM)

The dynamic load allowance for live load shall be as specified in the current AASHTO LRFD Bridge Design Specifications.

2.4.1.2.2 Dead Load Dynamic Load Allowance (DAD)

Structural parts in which the force effect varies with the movement of the span, or in parts which move or support moving parts shall be designed for a load taken as 20 percent of D_o to allow for dynamic load allowance or vibratory effect. This dynamic load allowance shall not be combined with live load.

2.4.1.2.3 Force Effects Due to Operation of Machinery (DAM)

Structural components supporting forces caused by machinery during operation of the span shall be designed for the calculated machinery forces, increased 100 percent as a dynamic load allowance.

The machinery forces transferred to the structural system shall be taken as those specified for the service limit state for machinery design in Articles 5.7 and 7.4, augmented by the dynamic load allowance specified above, and factored for the strength limit state structural design as specified in Table 2.4.2.3-1.

COMMENTARY

C2.4.1.2.2

An example of situations in which dead load force effect varies with the movement of the span is a bascule bridge. Examples of parts which move or support moving parts include swing span trusses, vertical lift bridge trusses and towers, and supports for bascule trunnions.

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2.4.1.2.4 End Floorbeams

The end floorbeams of the moving span shall be proportioned for full factored live load plus twice the normal dynamic load allowance specified in Article 3.6.2 of the AASHTO LRFD Bridge Design Specifications.

2.4.1.3 WIND LOADS

2.4.1.3.1 General

In proportioning members and determining the stability of swing, bascule, and vertical lift spans, and their towers, wind loads shall be assumed acting either transversely, longitudinally or diagonally as specified in Article 3.8.1.2.2 of the AASHTO LRFD Bridge Design Specifications.

Exposed areas for transverse wind loads on the spans shall be determined as provided in the current AASHTO LRFD Bridge Design Specifications, supplemented and modified as follows:

- Exposed areas for longitudinal wind load on the spans shall be taken as one-half those for transverse wind, except for bascule spans when open where they shall be modified as specified herein for loads acting normal to the floor.
- Exposed areas for transverse and longitudinal wind loads on houses and counterweights shall be their vertical projections.
- Exposed areas for transverse and longitudinal wind loads on towers and their bracing shall be the vertical projections of all columns and bracing not shielded by the counterweights and houses.
- Exposed areas of open-deck grid floor bridges subjected to ice and to wind acting normal to the floor shall be taken as 85 percent of the area of a quadrilateral using actual width and whose length is that of the span.
- For bridges with solid floors, or with sidewalks, the actual exposed floor surface shall be used.

2.4.1.3.2 Movable Span Closed

The structure shall be designed as a fixed span in accordance with the AASHTO LRFD Bridge Design Specifications.

COMMENTARY

C2.4.1.3.1

In past editions of these specifications, only longitudinal, transverse, and 45 degree diagonal wind were considered.

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2.4.1.3.3 Movable Span Open

The provisions of the AASHTO LRFD Bridge Design Specifications shall apply and shall be augmented as specified herein.

Force effects from loads specified below shall be evaluated at the strength limit state using Load Combination Strength III specified in Table 3.4.1-1 of the AASHTO LRFD Specifications and Table 2.4.2.3-1 herein.

When the movable span is normally left in the open position, the open position shall be investigated for the full wind pressures specified in Article 3.8 of the AASHTO LRFD Bridge Design Specifications, combined with dead load and other applicable loads in the load combinations which include wind. For swing bridges, provision shall also be made for 50 psf on one arm and 35 psf on the other arm. Wind loading shall be applied to both arms and shall be considered as blowing in a single direction even when wind pressure varies between arms.

When the movable span is normally left in the closed position, the open position shall also be investigated for a load combination 60 percent of the wind pressures, specified in Article 3.8 of the AASHTO LRFD Bridge Design Specifications applied to the structure combined with dead load and impact on dead load, as specified in Articles 2.4.1.1 and 2.4.1.2.2, and any other applicable loads. For swing bridges, provision shall also be made for 30 psf on one arm and 20 psf on the other arm. Wind loading shall be applied to both arms and shall be considered as blowing in a single direction even when wind pressure varies between arms.

2.4.1.4 SEISMIC LOADS

Seismic loads, limit states and load combinations shall be as specified in the AASHTO LRFD Bridge Design Specifications and Section 3 of these specifications.

2.4.1.5 FATIGUE LIMIT STATE

The fatigue provisions of the AASHTO LRFD Bridge Design Specifications shall apply as augmented herein.

In addition to the fixed-span bridge fatigue design check of the AASHTO LRFD Bridge Design Specifications, which is based upon the number of trucks to cross the bridge during its 75-year design life, fatigue shall be checked for movable bridges based upon the number of times the bridge is opened. The stress range arising from the operation of the span from fully closed to full open position, and return, including the effect of wind and the passage of the last truck before opening and the first truck after closing, shall be less than the allowable stress range specified in Article 6.6.1 of the AASHTO LRFD Bridge Design Specifications, based

COMMENTARY

C2.4.1.5

The stress range arising from the operation of the span is primarily due to the change in the dead load stresses from opening and closing the bridge. If the bridge is kept open for a significant amount of time and thus subject to potentially significant wind, this wind load can contribute to this stress range also. Finally, the last truck before opening and the first truck after closing will also contribute to the stress range.

Since the number of openings of even the most frequently opened movable bridges will typically be much less than the number of trucks to cross the bridge during the 75-year design life, the stress range due to opening the bridge needs to be much greater than that due to the passage of the fatigue truck for this fatigue check to govern.

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upon the estimated number of cycles taken as the number of times the bridge is expected to be opened in its 75-year design life.

For this case, the value of "N" in Equation 6.6.1.2.5-2 from the AASHTO LRFD Bridge Design Specifications should be replaced with "N_0", the number of operational open and close cycles for which,

$$N_0 = (365)(75)\, n_0\, (ADO)$$

where:

n_0 = number of stress range cycles per operation, and estimated by the structural designer with respect to the bridge type

ADO = estimated average daily number of openings

2.4.1.6 SETTLEMENT

Special precaution shall be taken to control settlement or other movements of the piers to not exceed criteria established for the particular bridge. Where piles are used, consideration shall be given to the use of batter piles to resist torsional and horizontal forces produced by movement of the span.

2.4.1.7 VESSEL COLLISION AND PIER PROTECTION

Vessel collision loads and limit states shall be taken as specified in the AASHTO LRFD Bridge Design Specifications and Section 4 of these specifications.

2.4.1.8 HYDRAULIC CYLINDER STRUCTURAL CONNECTIONS

Force effects on structural connections to cylinders shall be based on a cylinder pressure as specified in Article 7.4.3. These connections and their supporting members shall be designed for the overload limit state, specified in Article 1.3.2.1.

2.4.1.9 STRENGTH AND RIGIDITY OF STRUCTURAL MACHINERY SUPPORTS

Structural components directly supporting machinery design loads shall also be designed for the overload limit state specified in Article 1.3.2.4. The resistance factor, φ, for use with structural, not mechanical, design under this limit state shall be taken as 0.75.

The overload limit state shall be taken as specified in Articles 5.7 and 7.4.

COMMENTARY

The weight of the last and first trucks before and after the operation of the span, respectively, is indeterminate. It is conservative to consider the trucks to be the maximum fatigue truck, or a 110,000 lb. truck. If this is too conservative (i.e. the fatigue check does not work), the 55,000 lb. fatigue truck may be considered appropriate if the number of openings of the bridge is a significant number and truly random.

C2.4.1.8

See Article 7.6.2.

C2.4.1.9

Forces from operating equipment, for the overload limit state, must be carried by the supporting structure. Service limit state design for these overloads would be too conservative, while strength limit state design would be unconservative. The resistance factor, φ = 0.75, provides the same reliability for the structure as is provided for the machinery.

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In the design of structural parts subject to force effects from machinery or from forces applied for moving or stopping the span, consideration shall be given to providing adequate stiffness and rigidity consistent with the operating tolerance of applicable machinery components and the avoidance of resonance. Deflections shall be investigated sufficiently to insure that they will not interfere with proper machinery operation.

2.4.1.10 ICE ACCRETION

Except as modified herein, the provisions of Article 3.9.6 of the AASHTO LRFD Bridge Design Specifications shall apply to the superstructure design.

The effect of ice loads on structural supports for machinery shall be accounted for in machinery loads transmitted to machinery structural support systems through the power requirements specified in Article 5.4.

2.4.2 Bridge-type Specific Provisions

2.4.2.1 MOVABLE BRIDGE LOAD COMBINATIONS

The load combinations identified in Table 2.4.2.3-1 and comprised of loads defined in Article 2.3 shall be considered in the design of movable bridges.

2.4.2.2 APPLICATION OF FIXED BRIDGE LOAD COMBINATIONS

The load combinations, specified in Table 3.4.1-1 of the AASHTO LRFD Bridge Design Specifications for all applicable limit states shall apply to movable bridges, as follows, using the resistance factors specified therein:

- Where a bascule span is not in operating mode and the counterweight is not temporarily supported for repairs.
- Where a swing span is in the closed position, the bridge ends are lifted to give the positive reaction specified in the contract documents, factored as a dead load of component, DC, as specified in Table 3.4.1-2 of the AASHTO LRFD Bridge Design Specifications.
- When a vertical lift bridge is not in operating mode and the counterweight is not temporarily supported for repairs.

The additional strength limit state load combinations specified in Article 2.4.2.3 shall also apply.

COMMENTARY

Machinery support requirements for structural rigidity can be quite demanding in comparison to normal tolerated deflections. Design of structural machinery supports should be closely coordinated with the machinery design, for both static and dynamic machinery conditions.

C2.4.1.10

See Commentary C5.4.1.

C2.4.2.2

The positive reaction at the ends of a swing span is typically equal to 150 percent of the maximum negative reaction due to live load and dynamic load allowance. Table 3.4.1-2 of the AASHTO LRFD Bridge Design Specifications contains maximum load factors and minimum load factors. In this context, the minimum load factor is seen as addressing wear of the locking mechanism.

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2.4.2.3 MOVABLE BRIDGE-SPECIFIC LOAD COMBINATIONS - STRENGTH LIMIT STATE

The following movable bridge-specific load combinations, defined as follows and as specified in Table 1, shall be considered as appropriate. Definitions of loads and load factors shall be as defined in Articles 2.3 and 2.4.1:

Load combinations for bascule and vertical lift bridge structures:

- **Strength BV-I** - Load combination relating to structure in the open or closed position and dynamic effects of operating machinery.
- **Strength BV-II** - Load combination relating to structure in any open position, dynamic effects of operating machinery, and wind.
- **Strength BV-III** - Load combination relating to structure in the closed position, with live load and the counterweight independently supported.

Load combinations for swing bridge structures:

- **Strength S-I** - Load combination relating to structure in any open or closed position and dynamic effects of operating machinery.
- **Strength S-II** - Load combination relating to live load on a simple span configuration.
- **Strength S-III** - Load combination relating to live load on a continuous span configuration.
- **Strength S-IV** - Load combination relating to structure in the open position, dynamic effects of operating machinery, and wind.
- **Strength S-V** - Load combination relating to live load on a simple span configuration and wind.
- **Strength S-VI** - Load combination relating to live load on a continuous span configuration and wind.

COMMENTARY

C2.4.2.3

Table 1 contains additional load cases to be investigated beyond those required in Table 3.4.1-1 of the AASHTO LRFD Bridge Design Specifications. These additional cases apply to the structural design of the movable bridge. The table contains a set of design cases for bascule and vertical bridges and another set for swing bridges.

With respect to the load combinations required for bascule and vertical lift bridges, Load Case BV-I deals with the structure in the open position, including the dynamic effects resulting from the acceleration of the span for stopping or starting. This case has load factors similar to the strength design case for dead load components of fixed bridge cases as required in Table 3.4.1-1 of the AASHTO LRFD Bridge Design Specifications. Load Case BV-II deals with dead load and dynamic effects in combination with the wind loads specified in these specifications for movable bridges in the open condition. Load Case BV-III deals with the situation where the bridge is in the closed position and the counterweight is supported independent to the bridge as may be appropriate for certain maintenance or rehabilitation conditions, and the structure is available for traffic. Since this is a temporary condition, the load factors required are similar to those applicable to permit loads in Load Combination Strength II of Table 3.4.1-1 of the AASHTO LRFD Bridge Design Specifications.

In the case of swing bridges, Load Case S-I is similar to BV-I, requiring the analysis of the same loads using the same load factors as required for bascule and vertical lift bridges. Consideration of Load Cases S-I through S-VI will show that a pattern of factors appears in the table relating to whether the structure supports dead load only, live load on one portion of the swing bridge treated as a simple span, or live load on the full bridge treated as a continuous structure. There is a further separation of these load cases, depending on whether or not wind is applied in combination with the live load. It can be seen that the pattern of factors is similar to comparable load cases in Table 3.4.1-1 of the fixed bridge specifications. Swing bridges must be designed with adequate end lift in the bridge closed position so that no live load condition, as specified in Table 1 herein or in Table 3.4.1-1 of the AASHTO LRFD Bridge Design Specifications, results in a negative reaction, i.e., uplift at the live load supports. Consideration of two special live load conditions is required to account for the possibility that the ends of the bridge might touch the supports, but the end wedges, or equivalent devices, do not lift the ends of the bridge properly before traffic is permitted on the bridge. Under such a situation, the component force effects from the permanent loads would be different from the intended, i.e., wedged condition.

Table 2.4.2.3-1 - Movable Bridge-Specific Load Combinations

LOAD COMBINATION	D_O	D_{CW}	L_S	L_C	L_{CW}	W_O	DAD	M_O
Bascule and Vertical Lift Bridges								
Strength BV-I	1.55	0	NA	NA	0	0	1.55	1.55
Strength BV-II	1.25	0	NA	NA	0	1.25	1.25	1.25
Strength BV-III	0	1.25	NA	NA	1.40	0	0	0
Swing Bridges								
Strength S-I	1.55	NA	0	0	NA	0	1.55	1.55
Strength S-II	1.35	NA	1.75	0	NA	0	0	0
Strength S-III	1.35	NA	0	1.75	NA	0	0	0
Strength S-IV	1.25	NA	0	0	NA	1.25	1.25	1.25
Strength S-V	1.25	NA	1.40	0	NA	1.25	0	0
Strength S-VI	1.25	NA	0	1.40	NA	1.25	0	0

2.5 MOVABLE BRIDGE DESIGN FEATURES AND REQUIREMENTS

2.5.1 Movable Bridge Specific Design Features and Requirements

2.5.1.1 BASCULE SPAN BRIDGES

2.5.1.1.1 Types

Whenever practical, bascule spans should be of the trunnion or rolling lift type.

2.5.1.1.2 Support Conditions

The forward ends of the single-leaf bascules shall be supported. The forward ends of double-leaf bascules shall be provided with effective locking mechanisms designed to transmit the shear necessary to produce equal deflections of the main girders or trusses under unbalanced live loads. Ordinarily, the center locks shall not be designed to carry moment.

2.5.1.1.3 Segmental and Track Girders

The flanges of the segmental and track girders of rolling lift bridges shall be symmetrical about the central planes of the webs. The central planes of the webs of the segmental

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girders shall coincide with the corresponding central planes of the webs of the track girders. The treads attached to the segmental girders and track girders shall be steel castings or rolled steel plates and shall not be considered as part of the flanges of these girders.

The allowable load for line bearing P_{LB} (lb./in.) between treads of segments having a diameter of 10 ft. or more shall not exceed:

$$P_{LB} = (12{,}000 + 80D)\left(\frac{F_y - 13{,}000}{20{,}000}\right) \quad (2.5.1.1.3\text{-}1)$$

where:

D = diameter of the segment (in.)

F_y = specified minimum yield strength of the material in tension (psi)

The thickness t (in.) of sole plates and of the flanges of flange-and-web castings shall not be less than t = 3.00 + 0.004 D. Tread plates may be flange-and-web castings. The edge thickness of the rolling flange shall not be less than 3 in., and the flange thickness at any horizontal section of the web of the casting shall be such that the unit bearing on the web of the casting shall not exceed one-half of the yield strength of the material in tension, with the length of the bearing taken as twice the depth from the rolling face to the plane under consideration.

The effective length of the line bearing for each web shall not exceed the thickness of the web of the segmental or track girder, including the effective thickness of the side plates, plus 1.6 times the least depth of the tread. The edge of the web shall be machined so as to bear continuously upon the tread. The thickness of the web shall be such that the quotient obtained by dividing the load by the area of a portion of the edge of the web whose length equals twice the thickness of the tread, shall not exceed one-half of the minimum yield strength of the material in tension.

Flange angles shall not be considered as transmitting any load from the web to the treads. The bearing value of side plates shall not exceed the resistance of those fasteners or welds connecting them to the web, which are included between diverging lines, in the plane of the web, that intersect at the line contact between the treads and track. These lines make an angle whose tangent is 0.8 with the normal to the rolling surface. The load, as used in this article, shall be the weight of the structure, no addition being made for rolling impact.

Solid tread plates on segmental girders shall have a radius slightly smaller than the segmental girders in order to facilitate the securing of tight contact with the girders throughout their length when drawn up with the attaching bolts.

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When not otherwise stipulated in the contract documents, all tread plates shall be made as long as practical. When tread plates are made in segments, the faces of the tread plates at the joints between the segments shall be in planes at right angles to the rolling surface and preferably at an angle of 45 to 60 degrees with the longitudinal centerline of the tread plate.

The portions of the segmental and track girders which are in contact when the bridge is closed shall be designed for the sum of the dead load, the live load and impact stresses. Under this loading, the allowable line loading shall be 150 percent of that given in the equation shown above in this article.

The segmental and track girders shall be reinforced with stiffeners and diaphragms.

2.5.1.1.4 Floors and Floor Fastenings

Floors shall be adequately fastened to prevent displacement during the opening and closing of the span.

2.5.1.2 SWING SPAN BRIDGES

2.5.1.2.1 Center Bearing

Center bearing swing bridges shall be so designed that the entire weight of the movable span is carried on a center pivot.

2.5.1.2.2 Rim Bearing

The load on the rim girder of a rim bearing or combined rim and center bearing swing bridge shall be distributed equally among the bearing points. The bearing points shall be spaced equally around the rim girder.

Rigid struts shall connect the rim girder to a center pivot which is firmly anchored to the pier. Each strut shall be attached to the rim girder at each bearing point, and at intermediate points when required. No fewer than eight struts shall be used in any case.

The lower track shall be strong enough to distribute the load on the rollers uniformly over the masonry.

2.5.1.2.3 Combined Bearing

In a combined rim and center bearing swing bridge, a definite portion of the load, not less than 15 percent, shall be carried to the center by radial girders attached rigidly to the center and to the rim.

COMMENTARY

C2.5.1.2.1

When the span is moving, it shall be stabilized against unbalance or overturning wind-induced moments by balance wheels which are adjustable for height. When the span is stationary in the closed position, it shall be stabilized by center wedges. See Section 6.8.2.

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2.5.1.2.4 Rim Girders

The rim girder shall be so designed that the load will be properly distributed over the rollers. For calculating force effects in the girder, the loads shall be assumed to be distributed equally to all the rollers. The span length shall be taken as the arc length between the adjacent bearing points of the rim girder. This part of the girder shall be considered fixed at both ends. The girder shall be designed in accordance with the applicable flexural provisions in Section 6 of the AASHTO LRFD Bridge Design Specifications.

Rim girders shall be provided with stiffeners and with fillers on both sides of the web at points of concentrated loading. These stiffeners shall fit close against both flanges. The distance between adjacent intermediate stiffeners shall not exceed 24 in. On rim girders exceeding 5 ft. in depth, alternate intermediate stiffeners may extend only one-half of the depth of the girder, unless required to be of full depth to stiffen the web. The thickness of plate stiffeners or the outstanding legs of stiffener angles shall not be less than 1/8 of their width. The tread plate for the rollers shall be securely fastened to the rim girder and shall be 2 to 3 in. thick, depending on the weight of the bridge. Rim girder flange angles shall not be smaller than 6 x 4 x ¾ in. For welded construction, the flanges shall be a minimum of 1 in. thick.

2.5.1.2.5 Shear Over Center

In swing bridges having a center truss panel, this panel shall be so designed that shear will not be carried past the center. The web members of such panels shall be strong enough, however, to make the bridge secure against longitudinal wind pressure when it is open.

2.5.1.2.6 Reaction Due to Temperature

Provision shall be made for an end reaction due to a temperature differential between the top and bottom chords of 20°F for truss spans and 15°F for girder spans.

Provision shall also be made for longitudinal dimensional changes and transverse deflections at the ends of swing spans due to temperature changes.

2.5.1.3 VERTICAL LIFT BRIDGES

2.5.1.3.1 Tower and Tower Spans

When a tower is supported in whole or in part by an adjacent fixed span, any live load without impact which can be placed on the supporting span beyond the warning gates shall be considered in the design of the supporting span.

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The lateral bracing of towers shall be calculated by using 2.5 percent of the total compression in the columns in addition to the specified wind loads.

2.5.1.3.2 Anchorage for Cantilevered Floors

When the design provides for live load on any portion of the floor outside of a main truss or girder, suitable provision shall be made for anchorage to resist the resulting negative reaction of the opposite truss or girder.

2.5.1.3.3 Wind on Vertical Lift Spans

For wind loads, the vertical lift span shall be designed as a simple fixed span. The wind loading condition shall be as defined in Article 2.4.1.3.

SECTION 4 - TABLE OF CONTENTS - VESSEL COLLISION CONSIDERATIONS

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4.1 SCOPE

Vessel collision criteria should be considered in the design of movable bridges. The provisions of the AASHTO LRFD Bridge Design Specifications shall apply, except as supplemented herein.

COMMENTARY

C4.1

Vessel collisions with movable bridges are relatively frequent. In most cases the collision accidents result in damage to the bridge protection system. However, there have been many instances when the damage to the bridge was severe resulting in casualties.

The causes of vessel collision with movable bridges may be grouped into three categories:

- Human Error; negligence, misjudgment or miscommunication on the part of the vessel operator or the bridge tender,

- Mechanical or Electrical Failure; vessel suffers engine or steering failure and becomes aberrant, bridge machinery fails stuck in closed position, and

- Environmental Conditions; bridge spans are relatively small and sensitive to drifting of vessels due to wind currents. Reduced visibility can make the vessel operator not see that the bridge is closed. Also, the bridge tender may not be aware of the approaching vessel.

A review of more serious vessel collisions with movable bridges since 1970 indicated that nine out of the ten accidents involved the bridge superstructure. The accidents reviewed are summarized in the Appendix. In six of these accidents the vessel hit the side spans, while the movable spans were only hit in three cases. In one case the bridge was opening, in another case the bridge was stuck in the down position and in the third case the bridge was in the open position.

The causes of the accidents were human error in five cases, mechanical failure of the vessel in one case, mechanical failure of the bridge in one case, and unknown in two cases.

4.2 DEFINITIONS

Design Vessels - Vessels that comply with requirements of the AASHTO LRFD Bridge Design Specifications.

Geometric Probability - Conditional probability that a vessel will hit a bridge pier or span given that it has lost control, i.e., it is aberrant, in the vicinity of the bridge.

Operating Vessel - A vessel smaller than the Design Vessels specified in the AASHTO LRFD Bridge Design Specifications used to minimize frequent damage to movable bridges.

Protect - The term protection as used herein is related to a given vessel collision scenario and structure performance or acceptable damage level. It does not refer to full protection against any vessel collision.

Vessel Aberrancy - Probability that a vessel is out of control in the vicinity of a bridge.

4.3 PERFORMANCE CRITERIA

Because of the complex interaction of the various systems of a movable bridge, the owner and the designer should establish vessel collision performance goals consistent with the importance of the bridge using the guidelines below. The performance criteria should include both structural and operating requirements.

For the purpose of selecting maximum annual frequency of collapse, as specified in Article 3.14.5 of the AASHTO LRFD Bridge Design Specifications, bridges that need to be operational after a vessel impact should be included in the "critical" bridge category, and all other bridges should be considered "regular".

The function and the performance requirements for bridge protection systems should be clearly defined in relation to the performance requirements for the bridge itself.

C4.3

The design objective for fixed bridges is to minimize the risk of catastrophic failure of a bridge component due to collision with a large vessel in a cost-effective manner and at the same time reduce the risk of vessel damage and environmental pollution. Damage to structural members may be accepted provided that it does not result in bridge collapse, undue bridge service interruptions and traffic hazards. The collision impact forces used represent a probabilistically based, worst case, head-on collision, with the Design Vessel moving in a forward direction at a relatively high velocity. Thus, the design of fixed bridges is concerned, mainly, with the rare event of a major impact, whereas collisions with smaller vessels are considered relatively insignificant.

Movable bridges are low level waterway crossings that need to accommodate both marine and vehicular traffic. They are quite sensitive to movements, distortions and vibrations, and due to tight tolerances between moving parts, even minor vessel collisions can cause misalignments and interfere with the operation of the bridge. Vessel collisions with movable bridges can damage the bridge structure, the drive machinery, and other operating systems such as balance, interlocking and controls. Structural and some mechanical repairs can take relatively long periods of time. Since movable bridges serve both marine and vehicular traffic, breakdown in their operation can result in navigation and/or highway traffic interruptions with serious economic and societal consequences.

4.4 DESIGN VESSELS, LOADS AND LIMIT STATES

In addition to the Design Vessels required by the AASHTO LRFD Bridge Design Specifications, collision with a smaller vessel, or vessels, designated an Operating Vessel, defined in Article 4.2, may also be considered to:

- minimize damage from routine marine traffic,
- ensure that the bridge remains operational, and
- proportion fender system so that it is not severely damaged after minor collisions.

The selection of the Operating Vessel type and size should consider the vessel traffic make-up and size

C4.4

Movable bridges have a relatively high exposure to collisions with vessels that are smaller than the vessels of the AASHTO design scenario. This is due, primarily, to operational aspects related to the opening and closing of movable bridges, as well as to their limited horizontal and vertical clearances. While such collisions may not have catastrophic consequences, they could often cause damage to the bridge, and interrupt the traffic of motorists and/or vessels. In such cases, the possibility of minor but frequent vessel collision damage also needs to be addressed in design.

Including an Operating Vessel impact scenario in the design criteria can reduce the consequences of low-level impacts. Also, it can help with the design of the bridge structural and operating systems and provide a rational

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distribution, the existing hazards to navigation and the importance classification of the bridge.

The type, size and speed of the Operating Vessel may be specified by the owner or selected based on the following minimum criteria:

- the annual number of passages of vessels larger than the Operating Vessel is under 50 percent of the total vessel passages per year,
- the vessel speed is representative of typical transit conditions.

The force effects resulting from Design Vessels shall be investigated at the extreme event limit state.

The extreme event load combinations shall be utilized with the Operating Vessel collision. For the purpose of this design condition, the section capacity and resistance factors applicable to the strength limit state shall be used, and reliance of mechanism behavior shall be avoided wherever practical.

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design for guide fender systems. The type and size of the Operating Vessel needs to reflect the characteristics of the most typical vessels expected to pass through the bridge. Such expanded design criteria based on both a large Design Vessel and an Operating Vessel is especially important for movable bridges, whose operation could be hindered by low-level impacts.

4.5 INITIAL PLANNING

4.5.1 General

Vessel collision aspects should be considered as early as possible in the planning process for a new bridge, as they may significantly affect the total cost of the bridge. Decisions related to the bridge type, location and layout, and the use of bridge protection systems should take into account the waterway geometry, the layout of the navigation channel and the characteristics of the vessel traffic.

C4.5.1

Some of the design and detailing guidelines in Section 4.11 should also be considered as initial planning for the bridge planning process.

4.5.2 Site Selection

The provisions of Article 2.3.2.2.5 of the AASHTO LRFD Bridge Design Specifications shall apply.

4.5.3 Selection of Bridge Type, Configuration and Layout

The selection of the bridge type and configuration should consider the characteristics of the waterway and the vessel traffic, especially the requirements for horizontal and vertical clearances. The layout of the bridge should be established such that the number of piers and spans exposed to vessel collision are minimized to the extent practical and consistent with other design objectives. Finding the optimum bridge type, layout and protection system should be done through an iterative process, which

C4.5.3

The horizontal clearance of the navigation span can have a significant impact on the risk of vessel collision with the main piers. Due to structural and operational limitations, most movable bridges have relatively narrow spans in relation to the vessel traffic. Analysis of past collision accidents has shown that channel spans that are narrow relative to the vessel width and/or the width of the channel are particularly vulnerable to vessel collision.

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considers the costs involved in risk reduction, including political and social aspects.

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The vertical clearance of the navigation span is usually based on the highest vessel that uses the waterway in a ballasted condition and during periods of high water level. The vertical clearance requirements need to consider site specific data on current and projected vessels, and they should be coordinated with the U.S. Coast Guard.

4.5.4 Approach Spans

The initial planning of the bridge layout should also consider the vulnerability of the approach spans to vessel collision. To the extent practical, the number of approach piers should be minimized and the superstructure of the approach spans should be high enough to clear the majority of the vessel traffic, if possible. Special attention should be given to the design of the approach spans exposed to vessel impacts. Both local and overall span capacity criteria should be considered.

C4.5.4

Review of historical vessel collision data for all bridge types has shown that the approach spans were damaged in over 60 percent of the total number of accidents. The movable bridge accident review included in Appendix A shows a similar trend, i.e., six out of the ten accidents described involved the side spans.

4.5.5 Protection Systems

Bridge protection alternatives should be considered during the initial planning phase as the cost of bridge protection systems can be a significant portion of the total bridge cost.

The use of a pier protection system should be weighted against the design of the pier to withstand collision loads.

Bridge protection systems include fender systems, dolphins, protective islands, or other structures designed to redirect, reduce, withstand, or absorb the impact force and energy.

C4.5.5

Bridge protection systems may be used to:

- stop the vessel before reaching a bridge pier,
- reduce the vessel impact on the pier,
- prevent head-on collisions by redirecting the vessel,
- prevent vessels from reaching vulnerable substructure and superstructure elements,
- help guide vessels through the navigation channel,
- prevent sparks.

A pier protection system may be made either independent or integral with the bridge pier.

4.6 COLLISION RISK ANALYSIS

The vessel collision risk model described in the current AASHTO LRFD Bridge Design Specifications provisions shall apply, except that the unique characteristics of movable bridges and the special navigation conditions should be considered, especially when estimating the probability of vessel aberrancy and the geometric probability.

C4.6

The currently used risk models have mainly been developed in relation to fixed bridges. The risk factors associated with the operation of vessels approaching movable bridges are somewhat different from those related to the wider fixed span bridges.

The navigation of vessels through a movable bridge is a difficult process. Vessels have limited horizontal and vertical clearance to maneuver and are required to operate at reduced speeds at which they are less maneuverable and more susceptible to strong winds and currents. The size of vessels has been steadily increasing and this trend

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will most likely continue in the future, resulting in even less available clearance. Vessels passing through movable bridges are also likely to become aberrant during or even after crossing the span. In one case, for example, a barge tow going upstream has passed through the bridge, but due to its low speed, cross currents have turned it back hitting the side spans.

In addition to the normal difficulties of navigating through limited horizontal and vertical clearances, an important aspect that affects the safety of movable bridges is the communication between the vessel operator and the bridge tender. Miscommunication between the vessel and the bridge operator has been the cause of many accidents in the past.

The risk of vessel collision with movable bridges is particularly high during periods of low visibility and other adverse weather conditions, which can also interfere with the communication between the bridge tender and the vessel operator.

The relatively high likelihood of accidents near the navigation channel, should be considered when evaluating the geometric probability. A review of past accident data with movable bridges may be made to verify the applicability of the geometric probability formulation for fixed bridges.

In general, further work is needed to better adapt the currently used risk models to the special conditions that occur during vessel passage through movable bridges.

4.7 VESSEL IMPACT LOADS

Vessel collision loads should be determined as specified in the current AASHTO LRFD Bridge Design Specifications for a given vessel type, size and speed.

4.8 BRIDGE ANALYSIS CONSIDERATIONS

Bridge design should consider the possibility of collision with the moving span in either closed or partially open positions. Modeling assumptions should include the unique structural, operational, and load distribution characteristics of each movable bridge type. Elastic analysis should be used for the Operating Vessel case where such is specified by the owner. Nonlinear behavior may be included in analysis for the Design Vessels applicable to the extreme event limit state.

A clear load path from the location of the vessel impact to the bridge foundation should be established and the components and connections within such load path should be adequately designed and detailed. The design of individual bridge components should be based on the

C4.8

Designing for vessel collision is commonly based on equivalent static loads that include global forces for checking overall capacity and local forces for checking local strength of bridge components.

The contribution of the superstructure to the transfer of loads to adjacent substructure units depends on the capacity of the connection of the superstructure to substructure and the relative stiffness of the substructure at the location of the impact. The modeling of pile foundations could vary from the simple assumption of a point of fixity to nonlinear soil-structure interaction models, depending on the limit state considered and the sensitivity of the response to the soil conditions. Lateral load capacity analysis methods for pile groups that include nonlinear

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appropriate criteria specified in the AASHTO LRFD Bridge Design Specifications.

COMMENTARY

behavior are recommended in (Brown 1992) and the features of a finite element analysis computer program developed for bridge piers composed of pier columns and cap supported on a pile cap and nonlinear piles and soil are presented in (Hoit 1996). Transient foundation uplift or rocking involving separation from the subsoil of an end bearing foundation pile group or the contact area of a foundation footing may be allowed under Design Vessel impact loading provided that sufficient consideration is given to the structural stability of the substructure.

4.9 BRIDGE PROTECTION SYSTEMS

Substructure and superstructure bridge elements exposed to vessel collision but not designed for collision loads should be equipped with an adequate protection system. The protection system should be able to resist collision loads with the operating vessel without significant damage, and major collisions with the Design Vessel through plastic deformations, if necessary. The protection system should prevent vessel rake, flare or overhang from reaching vulnerable structure components, even when it is in a fully deformed position.

The superstructure of the moving span on bascule bridges should be outside the design clearance envelope when they are in an open position. The protection system along the sides of the navigation channel should prevent contact between the vessel and the span.

Pivot piers of swing bridges shall be protected by fenders that extend up and down the stream from the pivot pier over a length at least equal to the length of the swing span. The fender should be somewhat wider than either the pier or the superstructure to provide adequate protection to both. The ends of swing spans when open should be protected by dolphins or other structures strong enough to resist vessel impact.

C4.9

The channel piers are usually protected by fender systems, but the fenders normally used can be easily damaged, and offer little protection against major vessel collisions. Adequately sized and designed dolphins placed in front of the channel piers offer more effective resistance to high energy collisions. However, there are no consistent analysis and design procedures for bridge protection systems, especially for structures such as dolphins, which are expected to undergo large plastic deformations during a collision.

To be effective, the location and geometry of the bridge protection system should be such as to prevent the bow flair or overhang of vessels from reaching the superstructure. In several collisions in the past the vessel was able to impact vertical lift towers and bascule spans in the open position before even engaging the fender.

The side spans over the waterway are commonly not protected by fenders, however, they are usually very susceptible to vessel collision.

4.10 BRIDGE OPERATOR'S HOUSE

The operator's house should be located away from vessel reach and so as to afford a clear view of operations on the highway, and on both sides of the waterway.

4.11 DESIGN AND DETAILING GUIDELINES

To the degree practical, the design and detailing of movable bridges should account for the high risk and inherent vulnerabilities of movable bridges.

Due to the articulated nature of movable bridges and large impact loads involved in vessel collisions, special

C4.11

The end support conditions of the moving spans and counterweights have a significant effect on the overall bridge resistance to lateral loads, but current lateral load design requirements for the components supporting and

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care is needed in design and detailing. The following characteristics should be considered in design for vessel collision:

- use details that are not very sensitive to misalignments.
- add redundancy to the structure and the operating systems.
- provide structural support systems that can prevent span loss, such as catch ring blocks at the pivot pier in swing bridges.
- consider structural systems that allow for easy repair, jacking or lifting operations.
- include structural systems that could allow removal of the moving span for off-site repairs.
- prevent instability failure modes in critical members by reducing susceptibility to local and overall buckling.
- provide adequate capacity to connections in general and connections between moving parts in particular.
- the lateral bracing system for the moving spans and the fixed spans exposed to vessel collision should have adequate capacity.
- detail structural, mechanical and electrical components to allow for structural deformations and movements between adjacent structural units.
- consider providing span locks with adequate capacity for both the open and closed position.
- design machinery, electrical and hydraulic components so that they can be easily replaced.
- include suitable means of access for structural and mechanical postevent inspections.
- provide for clear communication means between vessel operator and bridge tender.
- noise in the Operator's house from the machinery should be minimized.
- electrical power cables, including submarine cables, should be positioned and supported so as to be fully protected from damage by impact from marine traffic.

COMMENTARY

restraining the moving spans and counterweights are relatively low.

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- mechanical, hydraulic, or electrical systems should not be located on walls susceptible to vessel impact.

- precautions should be taken to locate and/or protect drive systems, such as hydraulic systems, drive gearing, motors and electrical power and control systems from possible damage due to direct or indirect impact damage from marine vessels.

COMMENTARY

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APPENDIX A4

A4.1 LIST OF VESSEL COLLISIONS WITH MOVABLE BRIDGES

The following is a list of the more serious vessel collisions with movable bridges that have occurred since 1970. The number of collisions that have resulted in minor damage to the bridge or the fender system is significantly larger.

1972 Sidney Lanier Bridge, Brunswick River, Georgia, USA (A/III).

Bridge: Vertical Lift
Vessel: 13,000 DWT freighter. SS African Neptune.
Accident: The bow of the ship hit the superstructure of side spans.
Damage: Three side spans collapsed, causing ten fatalities in cars waiting for the lift span to close.
Cause: Miscommunication between the helmsman and the ship pilot. The ship was making a turn before approaching the bridge.
Lit.: US National Transportation Safety Board and US Coast Guard: "SS African Neptune collision with the Sidney Lanier Bridge", Marine Accident Report, 1973.

1974 Welland Canal, Ontario, Canada. (Unknown/III).

Bridge: Vertical Lift
Vessel: Ore carrier (670 feet long).
Accident: The ship hit the lift span while opening.
Damage: The lift span fell in the canal and the tows were damaged.
Cause: Unknown
Lit.: Engineering News Record, August 29, 1974 and September 12, 1974.

1977 Benjamin Harrison Memorial Bridge, James River, Hopewell, Virginia (B/II).

Bridge: Vertical Lift
Vessel: 25,000 DWT tanker in ballast. SS Amrine Floridian.
Accident: The stem of the ship destroyed a side pier about 400 feet from the navigation span centerline.
Damage: The collapse of a side span led to the collapse of the steel tower on one side of the lift span leaving one end of the lift span hanging from the remaining tower.
Cause: Electrical fault in steering gear.
Lit.: US National Transportation Safety Board: "US Tank Ship SS Marine Floridian, Collision with Benjamin Harrison Memorial Bridge"' Marine Accident Report, 1978.

1977 Tingstad Bridge, Gothenburgs Harbour, Sweden (A/III).

Bridge: Steel truss swing bridge with a 187 feet long swing span allowing two navigation channel openings of 52 feet each.
Vessel: 1,600 DWT tank ship.
Accident: The ship hit a side span of the bridge.
Damage: The superstructure of the side span and the bridge abutment were seriously damaged.
Cause: Pilot error.
Lit.: Olnhausen (1983).

1978 Berwick Bay Bridge, Atchafalaya River, Morgan City, Louisiana, USA (A/III).

Bridge: Vertical Lift
Vessel: Tugboat pushing four barges.
Accident: The lead barge hit the side span.
Damage: One 232 feet side span collapsed.
Cause: Very difficult navigation conditions (bend in channel plus high current). The tow was underpowered for these conditions.
Lit.: US National Transportation Safety Board and US Coast Guard: "Collision of M/V Stud with the Southern Pacific Bridge...", Marine Accident Report, 1980.
Comment: There were 534 accidents at this location.

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1979 Bridge over Gota River near Kungalv (Jordfallet), Sweden (B/III).

Bridge: Double-leaf bascule bridge with a clear navigation channel width of 140 feet between fenders. The bascule piers are 66 feet wide reinforced concrete piers.

Vessel: 3,000 DWT ship.

Accident: The ship hit the superstructure of one bascule, which could not be opened because of an electrical failure which left one leaf down. The ship which had a width of only 40 feet could have passed through the remaining channel of 70 feet, but it hit the leaf with its deckhouse.

Damage: The front corner of one bascule leaf was seriously damaged. A 30 feet by 30 feet corner of the deck was bent downward at an angle of about 70 degrees. In spite of the very strong impact, the machinery and the counterweight arm of the bascule leaf that was hit were not damaged.

Cause: Electrical failure on the bridge.

Lit.: Olnhausen (1983).

Comment: The same bridge was hit again in 1981 by a 480 DWT ship, which missed the channel and struck one of the bascule piers about 26 feet behind the fender. The ship caused a triangular hole about 10 feet wide in the 1.5 feet thick concrete wall just above the water level. The ship did not reach the tail end of the bascule span.

1982 Hannibal Railroad Bridge, Mississippi River, Hannibal, Missouri, USA (A/III).

Bridge: Swing Span

Vessel: Tugboat pushing 15 barges.

Accident: Barges struck abutment while passing the swing span, lost control and tugboat swung into approach span.

Damage: One approach span collapsed.

Cause: Careless navigation.

Lit.: Engineering News Record, May 13, 1982.

1985 Vertical Lift Bridge over the Beauharnois Canal, St. Lawrence Seaway, 25 miles west of Montreal, Canada (Unknown/III).

Bridge: Vertical Lift. Protected by three rock-filled sheet-pile dolphins in front of each tower on each side of the channel. The dolphins are 40 feet in diameter and placed diagonally to direct ships towards the lift span opening. The river crossing is about 3,000 feet long carrying a two-lane roadway and a single rail track.

Vessel: Indian freighter Jalagodavari (530 feet long).

Accident: Freighter passed through between two protection dolphins and through the side span. The vertical lift span was raised at the time.

Damage: One side span (60 feet long) collapsed and two or four vehicles dropped into the river. The pier supporting the collapsed span and partially supporting the south lift tower was demolished. The lift tower did not collapse but had to be shored. No fatalities.

Cause: Unknown.

Lit.: Engineering News Record, December 5, 1985.

1994 106th Street Bridge, Calumet River, Chicago, Illinois, USA (Unknown/III).

Bridge: Trunnion bascule bridge, built in 1929.

Vessel: Freighter.

Accident: The northeast corner of the east leaf of the bridge was struck by a freighter while in the open position.

Damage: The impact shoved the 1,800-ton leaf 1 inch to the north and rotated its northern edge 9 inches to the east. There, the 120 foot long section became stuck at an 80 degree angle, 10 degrees past its normal position. Lateral bracing on the underside was also damaged and the two truss chords crimped inward, shortening the distance between them by 4 inches. Eight 2.25-inch anchor bolts were sheared off. The damage required replacement of 42 tons of steel. The dislodged bascule did not fall off its trunnion supports. Final alignment was a special problem during bridge repairs.

Cause: Unknown.

Lit.: Engineering News Record, February 28, 1994.

<u>1996 Million Dollar Bridge over the Fore River, Portland, Maine (A/III).</u>

Bridge: Double-leaf bascule with a navigation span width of 96 feet. Protected by conventional guide pile fender systems in front of the channel piers.
Vessel: Loaded tanker ship (560 feet long and 85 feet wide).
Accident: The tanker hit the pile fender system.
Damage: A large portion of the fender was destroyed and the flare of the ship's bow caused significant damage to one of the bascule leaves. Also, 170,000 gallons of fuel oil were spilled in the river due to a hole ripped in the vessel hull by a step in the footing under the water.
Cause: Pilot error.
Lit.: Engineering News Record, 1996.

SECTION 5 - TABLE OF CONTENTS - MECHANICAL DESIGN LOADS AND POWER REQUIREMENTS

SPECIFICATIONS COMMENTARY

5.1 SCOPE

The provisions in this section apply to design loads and power requirements for operation of movable bridges. The loads established herein must be resisted by span operating machinery as described in the following sections.

5.2 DEFINITIONS

Acceleration Torque - Torque produced by prime mover at any time between the initial start condition and full load speed. This will be a variable as the torque value will vary with the speed.

Actual Speeds - Velocity at which machinery will move or rotate under the actual load or resistance, which is dependent upon the speed vs. torque characteristics of the prime mover or the power limiting settings of a hydraulic pump.

Breakdown Torque - (Applies only to AC induction-type motors) The maximum torque the motor will develop without an abrupt drop in speed.

Controlled - (Applies only to AC motors) An AC drive that has an effective and proven mode of torque limiting at any motor speed taken as 0 RPM through full-load speed.

Encoders - Device used to translate shaft angular position into digital electronic signals to be used as feedback signals for position and speed controls.

Full Load Speed - Rotational speed of motor at which it produces rated horsepower.

Full Load Torque - Torque produced by motor at full load speed.

Peak Torque - The maximum torque a motor can develop at any speed (including stalled condition).

Prime Mover - The source of rotational energy.

Stall Torque - The motor torque available at the stall condition immediately following cessation of motor shaft rotation.

Starting Torque - The motor torque available, prior to rotation of motor shaft, to initiate movement of the span.

Synchronous Speed - (Applies only to AC induction-type motors) Theoretical speed of motor with no load or frictional losses (rotational speed of magnetic field).

Uncontrolled - Any prime mover that does not have an effective and proven method of torque limiting at any motor speed.

5.3 NOTATION

AT	=	acceleration torque (lb.-in.) (5.7.1)
BDT	=	breakdown torque (lb.-in.) (5.7.1)
d,D	=	diameter (in.) (5.8.3)
FLT	=	full load torque (lb.-in.) (5.4.1)
OL_A	=	acceleration overload factor (DIM) (5.4.1)
OL_{cv}	=	constant velocity overload factor (DIM) (5.4.1)
OL_s	=	starting overload factor (DIM) (5.4.1)
P_m	=	motor size, power (hp) (5.4.1)

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PT = peak torque (lb.-in.) (5.7.1)
ST = starting torque (lb.-in.) (5.7.1)
T_A = maximum torque required for acceleration (lb.-in.) (5.4.1)
T_{cv} = maximum torque required for constant velocity (lb.-in.) (5.4.1)
T_s = maximum torque required for starting (lb.-in.) (5.4.1)
η = efficiency (decimal) (DIM) (5.8.4.2.2)

5.4 SIZING PRIME MOVER FOR SPAN OPERATION

5.4.1 General

The prime mover shall be of sufficient size to provide an excess torque, including allowances for prime mover overloads, to accelerate the span sufficiently to meet or exceed the requirements for time of opening for the bridge as specified in Section 1.

Power requirements shall be determined for loading conditions specified herein by bridge type. Sizing of the prime mover shall utilize the allowable overloads as a percentage of rated full load torque specified in Table 1:

Table 5.4.1-1 - Allowable Prime Mover Torque Overloads

Motor Type	(OL_s) Starting	(OL_A) Accelerating	(OL_{cv}) Constant Velocity
AC Induction	1.25	1.5	1.0
DC	1.25	1.5	1.0
IC Engine (4 cylinders or more)	0.75	0.8	1.0
IC Engine (less than 4 cylinders)	0.67	0.73	1.0
Hydraulic	See Article 7.5.11		

The actual speeds produced by the electric motors or other prime movers under load shall be used for calculating torque and in proportioning the bridge operating machinery, rather than the notional or synchronous speeds.

The minimum required full load torque shall be the larger of:

$$T_{min} = \frac{T_s}{OL_s} \qquad (5.4.1\text{-}1)$$

C5.4.1

Several of the loading conditions involve consideration of ice accretion loads. Where freezing is a rare occurrence, some judgment should be applied to reduce this requirement, thus resulting in a reduction in the size of the prime mover and machinery.

The allowable overloads in Table 1 represent common overload capacities for various classes of prime movers. The speed-torque curve for each prime mover should be investigated by the designer.

For instance, AC wound rotor motors are typically capable of very high stall and accelerating torques, but IC engines tend to have less available torque, i.e., lower efficiencies during stalled and accelerating conditions.

The actual speed and torque of an AC induction motor is always somewhat less than the notional speed, or synchronous speed. Synchronous speed is the apparent rotational velocity, expressed in RPM, of the magnetic field in the motor stator. AC induction motors only develop their rated torque at speeds typically 2-4 percent less than synchronous speed. This is the speed which must be used for calculating the motor's rated full-load torque. For example, a typical 1200 RPM motor may develop its rated power output at 1170 RPM, 2.5 percent below synchronous

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$$T_{min} = \frac{T_A}{OL_A} \quad (5.4.1\text{-}2)$$

$$T_{min} = \frac{T_{cv}}{OL_{cv}} \quad (5.4.1\text{-}3)$$

where:

T_s = maximum torque required for starting (lb.-in.)

T_A = maximum torque required for acceleration (lb.-in.)

T_{cv} = maximum torque required for constant velocity (lb.-in.)

OL_s = starting overload factor specified in Table 1 (DIM)

OL_A = acceleration overload factor specified in Table 1 (DIM)

OL_{cv} = constant velocity overload factor specified in Table 1 (DIM)

When determining the maximum starting torque, T_s, and maximum constant velocity torque, T_{cv}, every possible bridge position shall be considered for both directions of travel.

When determining the maximum accelerating torque, T_A, the maximum acceleration time shall not be taken greater than ten seconds. Loading shall be equal to that of T_{cv}, but shall include inertial loads from the span and machinery.

Inertial loading of the span shall include the span and counterweight.

After determining T_{min}, the prime mover shall be selected at the lowest standard power, P_m (hp), rating such that:

- FLT $\geq T_{min}$, and

- $$FLT = \frac{63{,}000 P_m}{n} \quad (5.4.1\text{-}4)$$

where:

n = full-load speed (RPM)

P_m = standard motor size, power (hp)

Where consideration of ice accretion loads specified in various Articles herein is appropriate, the exposed area of

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speed. An 1800 RPM motor may develop rated power output at 1775 RPM, only 1.4 percent below synchronous speed.

The constant velocity torque (T_{cv}) will vary according to the position of the movable span. The movable span can be stopped at any position by the operator, therefore, every span position must be considered for T_s.

Machinery inertia includes all mechanical items and the motor rotor, if an electrical drive is used, which can be a substantial load.

At the time of this writing, 1998, most motor manufacturers in this Country are manufacturing motors to NEMA Standard MG 1. The motor frame dimensions and power are expressed in inches and horsepower, respectively. (Motor manufacturers reportedly will, upon special request, provide nameplates in metric units, by soft conversion of the power rating using a conversion factor of 0.75 kW/HP.

In some cases, metric motors can be acquired based on the European IEC standards. Some dimensional incompatibility should be anticipated where metric motors would be mounted on English-dimensioned machinery, such as integral-type gear motors and C-face or flange-mounted pumps or gear reducers.)

Ice accretion is considered in determining the power requirement. Power requirements lead to design loads for the structural supports for the machinery system via the

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the deck shall be taken as specified for wind loads in Article 2.4.1.3.1.

5.4.2 Bascule Spans

The maximum bridge starting torque, T_s, shall be determined using the friction coefficients for starting and neglecting inertia.

The following load conditions shall be used to size the prime mover:

- **Maximum Starting Torque (T_s)** - Shall be determined for span operation against static frictional resistance, unbalanced conditions specified in Article 1.5, a wind load of 10 psf on any vertical projection, and an ice loading of 2.5 psf on the area specified in Article 2.4.1.3.1.

- **Maximum Constant Velocity Torque (T_{cv})** - Shall be determined for span operation against dynamic frictional resistances, unbalanced conditions specified in Article 1.5, and a wind load of 2.5 psf acting normal to the floor on the area specified in Article 2.4.1.3.1.

5.4.3 Swing Spans

The maximum bridge starting torque shall be determined using the friction coefficients for starting and neglecting inertia.

The following load conditions shall be used to size the prime mover:

- **Maximum Starting Torque (T_s)** - Shall be determined for span operation against static frictional resistances, a wind load of 10 psf on any vertical projection of the open bridge, and an ice loading of 2.5 psf on the area specified in Article 2.4.1.3.1. Provision shall

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machinery loads specified in Section 2 of these Specifications.

There are no general requirements for ice accretion loads on superstructures in the AASHTO LRFD Highway Bridge Design Specifications. Rather, ice accretion is treated as a site specific load in Article 3.9.6 therein. For most sites, it was considered that significant ice accretion on the superstructure would reduce the live load and dynamic load amplification due to the resulting poor driving conditions, i.e. there would be an offset in the loads. Thus ice accretion is generally not considered in the design of fixed bridge superstructures, and it follows that the same reasoning applies to the movable bridges when they are functioning as a fixed bridge.

Since ice accretion would not necessarily be inconsistent with the need to open the bridge, ice loads are applied, where appropriate, when determining power requirements and the structural ramifications thereof.

C5.4.2

Bascule types using operation struts, control arm linkages, etc., must include their effect on imbalance which may significantly effect maximum span torque values and angles of occurrence.

T_s is usually maximum with the span near fully open.

T_{cv} is usually maximum with the span near fully closed.

C5.4.3

Ice loading will add to the dead weight of the swing span and, therefore, the frictional torque will increase.

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also be made for a wind load of 10 psf on one arm and 5 psf on the other arm.

- **Maximum Constant Velocity Torque (T_{cv})** - Shall be determined for span operation against dynamic frictional resistances and a wind load of 2.5 psf acting horizontally against the vertical projection of one arm. For unequal arm swing spans the vertical projection of the longer arm shall be used. Under this loading, span operation shall occur in the normal time for operation.

Wind loading shall be applied to both arms and shall be considered as blowing in a single direction even when wind pressure varies between arms.

5.4.4 Vertical Lift Spans

The following load conditions shall be used to size the prime mover:

- **Maximum Starting Torque (T_s)** - Shall be determined for span operation against static frictional resistances, rope bending, unbalanced conditions specified in Article 1.5, a wind load of 2.5 psf on the area specified in Article 2.4.1.3.1 acting normal to the floor, and an ice loading of 2.5 psf on the area specified in Article 2.4.1.3.1.

- **Maximum Constant Velocity Torque (T_{cv})** - Shall be determined for span operation against dynamic frictional resistances, rope bending, unbalanced conditions specified in Article 1.5, and a wind load of 2.5 psf on the area specified in Article 2.4.1.3.1 acting normal to the floor. This wind loading shall be considered to include frictional resistances from span and counterweight guides caused by horizontal wind on the moving span.

5.5 HOLDING REQUIREMENTS

Machinery for holding the span against the loads and under the conditions specified herein may be proportioned for the overload limit state.

When bascule or swing spans are normally left in the open position, the span can be held in the fully open position against the wind loads specified in Article 2.4.1.3.1 by either:

- proportioning the machinery alone; or

- proportioning separate holding or locking devices, such that when combined with the holding capacity of the machinery, are capable of holding the span in the fully open position.

5.5.1 Bascule Spans

Where the span is normally left in the closed position, the machinery shall also be proportioned to hold the span in the fully open position against a wind load of 20 psf on any vertical projection of the open bridge.

5.5.2 Swing Spans

Where the span is normally left in the closed position, the machinery shall also be proportioned to hold the span in the fully open position against a wind load of 20 psf on one arm and 25 psf on the other arm, on any vertical projection of the open bridge. The wind direction shall be the same on both arms.

5.5.3 Vertical Lift Spans

Where a vertical lift span is normally left in the open position, resistance to satisfy wind loads specified in Article 2.4.1.3 shall be provided by separate holding or locking devices combined with the holding capacity of the machinery.

5.6 SIZING BRAKES

5.6.1 General

Determination of the required capacity of the brakes, both for holding the span against the wind pressure specified in Article 2.4.1.3 and/or stopping the span when in motion shall be based on:

- Use of 40 percent of the friction coefficients for motion specified in Article 5.8.2.
- Disregarding rope stiffness, solid roller friction, and machinery efficiency.

When sizing brakes, the inertial forces of both the span and machinery must be resisted.

Bridges which are solely manually-operated may be provided with only one set of brakes; one brake unit should be proportioned for stopping the span and shall be considered equivalent to a motor brake while the other brake unit should be proportioned to assist in dynamic braking for emergency stopping or to assist in static braking or "parking" the span in any position and shall be considered equivalent to a machinery brake.

C5.6.1

The motor rotor and motor brake wheels are often significant contributors to machinery inertia forces.

This does not apply for an emergency manual drive.

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Power-operated bridges shall be provided with two sets of brakes:

- One set, designated as the motor brakes, and consisting of one or two brake units, shall be on the shaft of an electrical motor, or if the prime mover is other than an electric motor, as near the shaft of the prime mover as practical.
- The other set, designated as the machinery brakes, and consisting of two brake units, shall be as near the operating ropes, pinion and ring gears, or pinion and racks as practical.

Brakes for manually or power operated bridges shall have the capacities as specified in Articles 5.6.2 through 5.6.4.

5.6.2 Bascule Spans

The motor brake(s) shall have sufficient capacity to stop the span in a maximum of 10 seconds when the span is moving at a speed conforming to the normal time for opening under the influence of the greatest unbalanced loads specified in Article 5.4.2 for T_{cv} plus an ice load of 2.5 psf on the area specified in Article 2.4.1.3.1.

The machinery brakes for bascule bridges shall be such that the combined motor and machinery brakes will have sufficient capacity to stop the span in 10 seconds when it is moving at speed conforming to the normal time for opening speed under the influence of the greatest unbalanced loads specified in Article 5.4.2 for T_s, and to hold the span against the wind pressure specified in Article 5.5.1.

5.6.3 Swing Spans

The motor brake(s) shall have sufficient capacity to stop the span in a maximum of 10 seconds when the span is moving at speed conforming to the normal time for opening under the influence of the greatest unbalanced loads specified in Article 5.4.3 for T_{cv}.

The machinery brakes for swing bridges shall be such that the combined motor and machinery brakes will have sufficient capacity to stop the span in 10 seconds when it is moving at speed conforming to the normal time for opening under the influence of the greatest unbalanced loads specified in Article 5.4.3 for T_s, and to hold the span against the wind pressure specified in Article 5.5.2.

SPECIFICATIONS

COMMENTARY

5.6.4 Vertical Lift Spans

The motor brake(s) shall have sufficient capacity to stop the span in a maximum of 10 seconds when the span is moving at speed conforming to the normal time for opening, but under the influence of the greatest unbalanced loads specified in Article 5.4.4 for T_{cv} plus an ice load of 2.5 psf on the area specified in Article 2.4.1.3.

The machinery brakes for vertical-lift bridges shall have a capacity, as measured at the shafts of the motor brakes, equal to half that of the motor brakes.

5.7 MACHINERY DESIGN CRITERIA

5.7.1 General

Where machinery of the usual manufactured types are specified, the contract documents shall specify testing requirements to document that such machinery satisfies the project requirements as determined by the Engineer.

The type of prime mover and its performance criteria shall be specified in the contract documents.

Machinery for moving the span shall be designed for the minimum percentages of full-load rated torque of the prime mover and the limit states specified in Table 1. The service limit state shall satisfy the normal time of operation. The resistances shall be taken as specified in Sections 6 and 7, and the resistance factors as specified in Section 6.

C5.7.1

The usual manufactured types of machinery include, electric motors, generators, internal combustion engines, hydraulic pumps, hydraulic motors, enclosed speed reducers, hydraulic cylinders, air compressors, etc.

Time of operation is not typically specified for the requirements listed as the overload limit state in Table 1.

Table 5.7.1-1 - Machinery Design Prime Mover Loads

Prime Mover	Service Limit State	Overload Limit State Stress
A.C. (Uncontrolled)	1.5 FLT	Greater of 1.5 ST or 1.5 BDT
A.C. (Controlled)	1.5 FLT	Greater of 1.0 ST or 1.5 AT
D.C. (Controlled)	1.5 FLT	3.0 FLT
Hydraulic	Refer to Hydraulic Section - Article 7.4	Refer to Hydraulic Section - Article 7.4
I.C. Engines	1.5 FLT	1.0 PT @ Full Throttle
Manual Operation	See provisions of Article 5.7.2.1	

Where controlled A.C. Drivers are specified, design for breakdown torque shall not be considered.

The provisions of Article 5.7.3 shall apply when determining resistance to braking and holding loads. Hydraulic system and component efficiencies shall be taken as specified in Article 7.6.3.

An A.C. drive shall be considered "controlled" if it has an effective and proven mode of torque limiting at any motor speed taken as 0 RPM through full load speed. Commonly used current limiting may not produce effective torque limiting at all speeds.

SPECIFICATIONS

5.7.2 Engine-Generator Drives

The provisions of Article 6.9 shall apply.

5.7.2.1 AUXILIARY DRIVES

Auxiliary drives shall be considered prime movers and machinery loading shall be as specified in Article 5.7.1 as applied to the auxiliary drive.

Auxiliary Electric Motor Drive shall satisfy the requirements of Section 8.

Auxiliary Hydraulic Motor/HPU Drive shall satisfy the requirements of Section 7.

Where the bridge or parts thereof are to be operated by hand, the number of people and the time of operation shall be calculated based on the following assumptions:

- The force which a person can exert continuously on a capstan lever is 40 lb. while walking at a speed of 160 fpm.
- The force which a person can exert continuously on a crank at a radius of 15 in. is 30 lb. with rotation at 15 RPM.
- For calculating the strength of the machinery parts, the force one person applies to a capstan lever shall be assumed as 150 lb., and to a crank as 50 lb. Under these forces, the allowable unit stresses may be increased 50 percent.

5.7.3 Braking

Provisions shall be made in the design of the machinery, including operating ropes where applicable, as follows:

- For the stresses caused by the brakes when only one set of brakes is provided, or for the stresses caused by either the motor brakes or the machinery brakes when two sets of mechanical brakes are provided, resisted at the service limit state.
- For the stresses caused by the combined action of mechanical motor and machinery brakes, or by the combined action of controlled electrical or hydraulic motor dynamic or regenerative braking and machinery emergency brakes if controls are arranged to permit simultaneous braking, or by the machinery emergency brakes alone if controlled electrical or hydraulic motor braking by the bridge drive controls is not permitted simultaneously with machinery brake operation,

COMMENTARY

C5.7.2.1

Machinery design for auxiliary drives will be a separate consideration in addition to normal span drive loading.

resisted at the overload limit state using a modified resistance factor of $\varphi = 1.5$.

The full effect of machinery friction shall be included when determining the effect of the brakes on the machinery design while stopping the span.

5.8 MACHINERY EFFICIENCIES AND LOSSES

5.8.1 General

In calculating the efficiency loads to be overcome by the machinery, the forces or moments shall be reduced to the total torque acting on or about the:

- centerline of the trunnion of a fixed trunnion bascule,
- instant center of roll of a rolling bascule,
- vertical centerline of rotation of a swing, or
- centerline of the counterweight sheaves of a vertical lift.

In the absence of verifiable data on equipment specified for the project, the data provided in Articles 5.8.2 through 5.8.4 may be used.

5.8.2 Friction Factors

Friction factors for plain radial bearings, plain thrust bearings, and rolling bearings and rollers may be taken as specified in Table 1. For manually-operated bridges, the coefficients for motion specified in Table 1 shall be increased by 25 percent.

SPECIFICATIONS COMMENTARY

Table 5.8.2-1 - Friction Factors for Typical Bearings

Bearing Type	For Starting	For Motion
Plain Radial Type:		
For Trunnion or Journal Friction		
• Sleeve bearings, one or more complete rotations	0.14	0.09
• Sleeve bearings, less than one complete rotation	0.18	0.12
For linear sliding bearings	0.18	0.12
Plain Thrust Bearings:		
For friction on center disks	0.15	0.10
For collar friction at ends of conical rollers	0.15	0.10
Rolling Element Bearings and Rollers:		
Rolling element bearings	0.004	0.003
For rolling friction of bridges having rollers with flanges, or build-up segmental girders	0.009	0.006
For rolling friction of solid cast rollers without flanges	$0.063/\sqrt{D}$	$0.063/\sqrt{D}$
where:		
D = diameter of roller (in.)	$0.0113/\sqrt{D}$	$0.0113/\sqrt{D}$

5.8.3 Wire Rope Bending Losses

For 180 degrees bending of wire ropes, for each sheave, the coefficient of direct tension in rope for starting and motion is 0.3 d/D.

where:

d = diameter of rope (in.)

D = diameter of sheave (in.)

5.8.4 Efficiency Factors for Gearing

5.8.4.1 OPEN SPUR GEARING

The efficiency of any pair of open spur gears, bearing friction not included, may be taken as 0.96.

SPECIFICATIONS

5.8.4.2 ENCLOSED REDUCERS

5.8.4.2.1 Parallel Spur and Helical, or Bevel

A preliminary estimate of enclosed speed reducer efficiency may be taken as:

Single Reduction:	0.98
Double Reduction:	0.96
Triple Reduction:	0.94
Quadruple Reduction:	0.92

5.8.4.2.2 Worm Gear

A preliminary estimate of enclosed single reduction worm gearing efficiency, η, with rolling element bearings may be taken as:

$$\eta \approx 1.15 - 0.25 (\log_{10} u) \qquad (5.8.4.2.2\text{-}1)$$

where:

u = worm gear ratio ≥ 10

COMMENTARY

C5.8.4.2.1

For efficiency of enclosed speed reducers or increasers, use the manufacturer's published value or the minimum efficiency as recommended by the latest AGMA standards. The given values are for estimates only.

Gear oil churning and special shaft seals may lower the efficiency values given.

Rolling element bearing friction is included in the factors specified.

C5.8.4.2.2

Refer to AGMA standards for a more accurate equation.

The approximate equation presented here was derived from information from manufacturers' catalogues.

Worm driving and rolling element bearing friction are included.

SECTION 6 - TABLE OF CONTENTS- MECHANICAL DESIGN

SPECIFICATIONS

6.1 SCOPE

The provisions in this section apply to the design of the machinery for moving, aligning, and locking a movable bridge span. The section addresses the requirements for bascule, swing, and vertical lift movable spans.

COMMENTARY

C6.1

6.2 DEFINITIONS

Addendum - Portion of gear tooth outside (greater than) the pitch radius.

AGMA - American Gear Manufacturer's Association

Allowable Static Design Stress - Permissible value of stress for calculations involving components subjected to static loading.

Average (mean) Stress - One-half of the sum of the maximum and minimum stress.

Backlash - The smallest amount of space between the faces of mating gears.

Bevel Gear - Type of gear commonly used when shafts intersect and utilizes the concept of rolling cones.

Brittle - Materials designed against ultimate strength for which failure means fracture; easily broken snapped or cracked.

Contact Stress Failure - Failure of gear teeth based on projected area of contact.

Crank Pins - Joint between linkages where stress alternates between application and release.

Cyclic Stress - Stress range which follows a pattern over and over.

Dedendum - Portion of gear tooth from the root to the pitch line.

Deflector Sheaves - Component used on span drive vertical lift bridges to guide operating ropes from the top chord (horizontal) to the tower attachments (vertical).

Diametral Pitch - Index of gear tooth sizes that is defined as the number of teeth divided by the pitch diameter (in.).

Ductile - Materials designed against yield strength and failure is visible before fracture.

Enclosed Gearing - Gear set of which all moving elements are included in a given frame and cover that is dust-proof and oil-tight.

Endurance Limit Strength - Stress level at which completely reversing cyclic stress (fatigue) causes failure in one million (1×10^6) cycles; the ability to withstand fatigue loads.

Fatigue Failure - Point at which cyclic loading causes fracture or permanent deformation.

Fatigue Limit State - Limit state relating to cyclic stress and crack propagation.

Fatigue Strength - Ability to withstand cyclic loading.

Helical - A gear with a cylindrical pitch surface and teeth that are at an angle to the axis.

Herringbone - A type of helical gearing where half of the teeth are right-handed and the other half are left-handed.

SPECIFICATIONS COMMENTARY

Idler Gear - A gear that has the same number of teeth as the mating gear and, therefore, introduces no change in shaft RPM, but changes the rotational direction.

L-10 Life - Basic rating life of a component for 90% reliability based on load and speed data, typically given as a number of revolutions.

Lay - The manner in which the wires in a strand or the strands in a rope are helically positioned.

Mechanical Shrink-fit Assembly - Mechanical connection where assembly is performed by heating or cooling one element relative to the other, and when an equilibrium temperature is reached, an interference fit is produced.

Minimum Yield Strength - The lowest value of stress a material shows a specified limiting deviation from the proportionality of stress to strain.

Module - Metric index of gear tooth sizes that is defined as pitch diameter (mm) divided by number of teeth.

Open Gearing - Gear set that is not sealed and may have moving elements exposed to the environment.

Pitting Resistance/Wear/Surface Durability - AGMA terms used for rating gearing from the aspect of contact surface stress.

Service Limit State - Limit state relating to stress, deformation and cracking applied to normal operating loads.

Sheave - A pulley or wheel having a grooved rim, typically used for wire ropes on vertical lift bridges.

Spur Gear Teeth -Teeth on the cylindrical pitch surface of a gear that are parallel to the axis.

Stress Range - Maximum stress minus the minimum stress.

Uniaxial Tensile Stress - Stress acting along only one axis.

Yield Failure/Intermittent Overload - Overload condition for which yield failure may occur in spur gear teeth experiencing less than 100 cycles in its design life.

6.3 NOTATION

6.3.1 General

A	=	constant cross-sectional area (in.^{2}); projected area (in.^{2}) (6.6.1) (6.6.2.5)
a	=	constant for finding surface roughness factor (DIM) (6.6.3.2)
$\sqrt{a}$	=	Neuber constant for finding q (notch sensitivity) (in.$^{0.5}$) (6.7.3.2)
B	=	exponent quantity (6.7.5.2.2)
b	=	exponent for finding surface roughness factor (DIM); pinion/gear face width (mm); key width (rectangular or square) (in.) (6.6.3.2) (6.7.10.1)
^oC	=	degrees Celsius (6.7.5.2.2)
C_D	=	endurance limit modifying factor based on diameter (DIM) (6.6.3.2)
C_f	=	gear design surface finish factor (DIM) (6.7.5.2.3)
C_H	=	gear design hardness ratio factor (DIM) (6.7.5.2.3)
C_M	=	endurance limit modifying factor for miscellaneous condition (DIM) (6.6.3.2)
C_{or}	=	basic static load rating, element bearing (lb.) (6.7.7.2.4)
C_p	=	gear design elastic coefficient (psi$^{-0.5}$)(6.7.5.2.3)
C_R, C_S, C_T	=	reliability, surface, temperature factors, respectively (DIM) (6.6.3.2)
C_r	=	basic dynamic load rating, rolling element bearing (lb.) (6.7.7.2.2)

d = pitch diameter of pinion (in.); diameter of the wire rope (in.) (6.7.5.2.1) (C6.8.3.3.4)
D = diameter for use in various sliding bearings (in.); rolling bearings (in.); shaft diameter (in.); tread diameter of sheave rope grooves (in.) (6.6.2.5)&(6.7.7.1.2) (6.6.2.6) (6.6.3.2) (6.7.4.3) (6.8.3.3.4)
d_w = diameter of outer wires in the wire rope (in.) (6.8.3.3.4)
E_R = tensile modulus of elasticity of the wire rope (psi) (C6.8.3.3.6)
E_w = tensile modulus of elasticity of the steel wire (psi) (6.8.3.3.4)
°F = degrees Fahrenheit (6.7.5.2.2)
F = pinion/gear face width (in.) (6.7.5.2.2)
F_{oxy} = factored large bearing load (lb.) (6.7.7.2.4)
F_{ua} = applied axial load (lb.) (6.7.7.2.2)
F_{ur} = radial load (lb.) (6.7.7.1.2)
G = gear for J and I tables (DIM) (C6.7.5.2.2)
g = acceleration due to gravity = 32.2 (ft./s.^{2}) (6.7.4.3)
H_B = Brinell hardness number (BHN) (DIM) (6.6.4.2)
H_{BP}, H_{BG} = Brinell hardness number for pinion/gear (DIM) (6.7.5.2.3)
h = key height (in.) (6.7.10.1)
h_t = total gear tooth height (in.) (6.7.5.2.2)
I = gear design tooth geometry factor - surface durability (DIM) (6.7.5.2.3)
J = gear design tooth geometry factor - fatigue (DIM) (6.7.5.2.2)
K_B = gear design rim factor (DIM) (6.7.5.2.2)
K_F = fatigue stress concentration factor (normal stress) (DIM) (6.7.3.2)
K_{FS} = fatigue stress concentration factor (shear stress) (DIM) (6.7.3.2)
K_f = gear design stress correction factor (DIM) (6.7.5.2.4)
K_m = gear design load distribution factor (DIM) (6.7.5.2.2)
K_{my} = gear design load distribution factor for overload (DIM) (6.7.5.2.4)
K_o = gear design overload factor (DIM) (6.7.5.2.2)
K_R = gear design reliability factor (DIM) (6.7.5.2.2)
K_S = gear design tooth size factor (DIM) (6.7.5.2.2)
K_t = theoretical stress concentration factor (normal stress) (DIM) (6.7.3.2)
K_T = gear design temperature factor (DIM) (6.7.5.2.2)
K_{ts} = theoretical stress concentration factor (shear stress) (DIM) (6.7.3.2)
K_v = gear design velocity (dynamic) factor (DIM) (6.7.5.2.2)
K_y = gear design yield strength factor (DIM) (6.7.5.2.4)
k = radius of gyration (in.) (6.6.1)
L = length of shaft between supports (in.) (6.7.4.2)
L_{act} = actual length (in.) (6.6.1)
L_{eff} = effective length (in.) (6.6.1)
M_a = bending moment amplitude (lb.-in.) (6.7.4.1)
m_B = gear design backup ratio (DIM) (6.7.5.2.2)
m_t = gear design tooth module transverse (6.7.5.1)
N = gear design number of load cycles (DIM) (6.7.5.2.2)
n = rotational speed (RPM); rotational speed of bearing inner race (RPM) (6.6.2.5) (6.7.7.2.2)
n_c = critical shaft rotational speed (RPM) (6.7.4.3)
n_p = pinion speed (RPM) (C6.7.5.2.1)
N_p = gear design number of pinion teeth (DIM) (6.7.5.2.2)
n_S = static design factor (DIM) (6.6.1)
P = power which the gear transmits (hp); pinion for J and I tables; direct load on wire rope (lb.) (C6.7.5.2.1) (C6.7.5.2.2) (6.8.3.3.4)
P_{ac} = allowable transmitted power for gear design surface durability (hp) (C6.7.5.2.1)
P_{at} = allowable transmitted power for gear design fatigue (hp) (C6.7.5.2.1)
P_d = diametral pitch (in.$^{-1}$) (6.7.5.1)
P_o = operating loads, e.g., the larger of starting or inertial loads (lb.) (6.8.3.3.4)
P_{or} = factored radial design resistance (lb.) (6.7.7.2.4)
P_r = equivalent dynamic radial load for rolling element bearings (lb.) (6.7.7.2.2)

P_{ut}	=	wire rope minimum ultimate breaking load (lb.) (6.8.3.3.6)
p	=	allowable static design resistance in compression; bearing design pressure (psi) (6.6.1) (6.7.7.1.2)
Q_v	=	gear quality number (DIM) (6.7.5.2.2)
q	=	fatigue design notch sensitivity factor (DIM) (6.7.3.2)
R_a	=	surface arithmetic average roughness (μin.) (6.6.3.2)
R_b	=	bearing resistance (lb.) (6.6.2.5)
R_R	=	line bearing resistance on rollers (lb./in.) (6.6.2.6)
r	=	notch/fillet radius or radius of a hole (in.) (6.7.3.2)
S_{at}	=	gear design allowable stress - fatigue (psi) (6.6.4.2)
S_{ac}	=	gear design allowable stress - surface durability (psi) (6.6.4.3)
S_{ay}	=	gear design allowable yield (psi) (6.6.4.4)
S_F	=	safety factor for bending strength (DIM) (6.7.5.2.2)
S_H	=	surface durability design factor (DIM) (6.7.5.2.3)
T	=	torque, torsional moment (lb.-in.) (6.7.5.2.1)
T_m	=	mean-steady torque (lb.-in.) (6.7.4.1)
t_R	=	rim thickness (in.)(6.7.5.2.2)
V	=	velocity; journal surface speed (fpm) (6.7.7.1.2)
v_t	=	gear design pitch line velocity (fpm) (6.7.5.2.2)
W	=	weight (lb.) (6.7.4.3)
W_{max}	=	maximum peak transmitting gear force (lb.) (6.7.5.2.4)
W_t	=	tangential transmitting gear force (lb.) (6.7.5.2.1)
W_{tac}	=	factored surface durability resistance of spur gear teeth (lb.) (6.7.5.2.3)
W_{tat}	=	factored flexural resistance of spur gear teeth (lb.) (6.7.5.2.2)
X, X_o	=	bearing design factors (radial) (DIM) (6.7.7.2.2), (6.7.7.2.4)
Y, Y_o	=	bearing design factors thrust (axial) (DIM) (6.7.7.2.2), (6.7.7.2.4)
Y_N	=	gear design life factor - fatigue (DIM) (6.7.5.2.2)
Z_N	=	gear design life factor (DIM) (6.7.5.2.3)
α	=	factor for allowable bearing resistance (lb./in.•RPM); factor for allowable line bearing resistance (lb./in.); endurance limit factor (DIM) (6.6.2.5), (6.6.2.6), (6.6.3.2)
σ	=	normal stress (psi) (6.7.2.3)
σ'_a,σ'_m	=	Von Mises stresses (psi) (6.7.3.3.2)
σ_a	=	amplitude stress - fluctuating (psi) (6.7.3.3.1)
σ_b	=	maximum wire rope bending stress (psi) (6.8.3.3.4)
σ_e	=	endurance limit (psi) (6.6.3.2)
σ_m	=	mean or average stress (psi (6.7.3.3.1)
σ_{max}, σ_{min}	=	maximum and minimum cyclic stress (psi) (6.7.2.2) (6.7.3.3.1)
σ_t	=	maximum total stress in wire rope (psi) (6.8.3.3.4)
σ_{ut}	=	ultimate tensile strength (psi) (6.6.3.2)
σ_y,σ_{yt},σ_{yc}	=	yield strength of material - min. (psi) (C6.6.1)
τ	=	shear stress (psi) (6.7.2.3)
τ_a,τ_m	=	amplitude and mean cyclic shear stresses (psi) (6.7.3.3.1)
τ_{max}	=	maximum shear stress resulting from applied loads (psi) (6.7.2.2)

6.4 GENERAL REQUIREMENTS

6.4.1 Machinery

6.4.1.1 LIMIT STATES AND RESISTANCE FACTORS

Unless otherwise stated, machinery design shall be based on the service and fatigue limit states using the loads and resistances specified herein.

C6.4.1.1

The design of bridge machinery in the United States is based on allowable working stress design, therefore, this section follows the accepted industry design practice. As

SPECIFICATIONS

These specifications require consideration of design equations to prevent fatigue failure of critical machine elements.

Where applicable, depending on the seismic design strategy chosen to comply with Article 3.4.3, some machinery may be required to resist seismic loads for which the extreme event limit state shall apply.

Unless specified otherwise, the resistance factors shall be applied to the general limit state Equation 1.3.2.1-1 shall be taken as:

- For the service limit state.................................φ = 1
- For the fatigue limit state...................................φ = 1
- For the overload limit state:
  Forged, drawn, rolled, wrought steel ......φ = 2.25
  Cast steel...φ = 3
- For the extreme event limit state:
  Forged, drawn, rolled, wrought steel ....φ = 2.7
  Cast steel..φ = 3.6

6.4.1.2 GENERAL

The machinery for the movable bridge shall be of simple design and substantial construction. The arrangement of parts shall permit easy installation, adjustment, and replacement of worn or defective parts and shall be accessible for inspection, cleaning, lubricating, and repairing. On any machinery with liquid or grease reservoirs, suitable petcocks and/or tube extensions at drains shall be provided to facilitate fluid changes.

6.4.1.3 LOCATION OF MACHINERY

The location of the machinery shall be selected with consideration for easy access for repair and maintenance, or future removal and replacement. Unless there are compelling reasons to the contrary, the machinery shall be located on the stationary part of the bridge.

6.4.1.4 SUPPORT AND ANCHORAGE

The attachment of machinery to its supports shall be adequate to hold the parts in place under all conditions of service. Where practical, each group of machinery shall be mounted on a self-contained steel frame, base, bedplate, or other sufficiently rigid structural steel support.

The provisions of Articles 6.7.14.1 and 6.7.14.2 shall apply.

COMMENTARY

of this writing (1998), reliability-based design at the strength limit state is not possible given the dearth of necessary data. See commentary to Article 1.3.2.1 for further discussion.

Historically, the design of machinery was done by primarily static analysis. Open spur gear design now follows current AGMA design procedures.

C6.4.1.4

A support shall be considered “sufficiently rigid” if its maximum deflection or distortion under allowable overloads does not cause misalignment of any component beyond design or rated misalignment capacity.

SPECIFICATIONS

6.4.2 Aligning and Locking of Movable Span

Movable bridges shall be equipped with span locks or other suitable mechanisms to accurately align the span ends to the approach roadway, both horizontally and vertically, and to secure the movable span in the closed position so that it cannot be displaced either horizontally or vertically under the action of traffic or other conditions of service. Effective end lifting and centering devices shall be used for swing bridges, and for bascule bridges centering devices may be used in conjunction with span locks. Span locks shall also be provided for vertical lift bridges where specified by the Owner.

Span locks shall be designed so that they cannot be engaged unless the movable portions of the span are within ½ in. of the proper position.

Where bascule, swing, or vertical lift bridges are normally, or seasonally left in the open position, span locks shall also be provided to hold the span in the fully open position.

6.4.3 Elevators

An elevator or elevators should be provided on any movable bridge.

When provided, elevators shall meet the requirements for passenger elevators of the ANSI/ASME Safety Code for Elevators and Escalators, ASME A17.3-1996, or latest revision, and applicable local codes.

The elevator cars shall be fully enclosed with solid sides and roof. They shall have a net floor area of not less than $12\ ft.^2$ and a capacity of not less than 1,200 lb.

On tower-drive vertical lift bridges, consideration shall be given to providing an elevator in each tower designed to carry personnel and hand carried maintenance equipment between the roadway level, if possible, or the operator's house and the machinery houses at the tops of the towers. Intermediate stops may be provided where specified by the Owner.

Elevators shall be power-operated, with single automatic control permitting the car to be called from a station at any landing and sent to any landing from the car station.

6.5 DESIGN LOADING REQUIREMENTS

The provisions of Section 5 shall apply.

COMMENTARY

C6.4.2

For a double leaf bascule, the two leaves must be aligned vertically within ½ in. relative to each other for the locks to be driven. For a vertical lift span, the ½ in. would again apply to the vertical distance.

Swing bridges must have their centering device properly engaged prior to engaging end lifts. When the end lifts also serve to center the swing span, the ½ in. would apply to the horizontal misalignment measured at the end lifts.

SPECIFICATIONS COMMENTARY

6.6 RESISTANCE OF MACHINERY PARTS

6.6.1 Resistance at the Service Limit State

C6.6.1

For commonly used materials, resistance values shall be computed using allowable static stresses in Table 1 which include the factors of safety and unsupported length provisions specified herein as applicable.

Table 6.6.1-1 - Allowable Static Stresses in psi

Material	AASHTO	ASTM	Tension	Compression	Fixed Bearing	Shear
Structural Steel	M 183	A 36	12,000	$12{,}000 - 55\,L_{eff}/k$	16,000	6,000
Forged Carbon Steel (except keys)	M 102	A 668 (CL D)	15,000	$15{,}000 - 65\,L_{eff}/k$	18,000	7,500
Forged Carbon Steel (keys)	M 102	A 668 (CL D)	-	-	15,000	7,500
Forged Alloy Steel	M 102	A 668 (CL G)	16,000	$16{,}000 - 70\,L_{eff}/k$	21,000	8,000
Cast Steel	M 103	A 27 (GR 485-250)	9,000	$10{,}000 - 45\,L_{eff}/k$	13,000	5,000
Cast Steel	-	A 148 (GR 620-415)	15,000	$15{,}000 - 65\,L_{eff}/k$	21,000	7,500
Bronze	M 107	B 22 ALLOY UNS C90500, C91100, C91300, C93700	7,000	7,000	-	-
Hot Rolled Steel Bar	M 255	A 675 (GR 515)	12,000	$12{,}000 - 55\,L_{eff}/k$	16,000	6,000

For materials not included in Table 1, resistance shall be determined by applying the remaining provisions of this article.

The minimum static design resistance at the service limit state shall be determined by applying the following factors of safety, n_S, to minimum tensile yield:

- Forged, drawn, rolled, wrought steel................ $n_S = 3$
- Cast steel .... $n_S = 4$

The static shear resistance shall be based upon one-half the allowable tensile design resistance.

SPECIFICATIONS

For slender members in compression, the allowable static design resistance in compression, p, shall be determined as:

$$p = \frac{A\sigma_y}{n_s}\left(1 - 4.6x10^{-3}\,\frac{L_{eff}}{k}\right) \qquad (6.6.1\text{-}1)$$

where:

A = cross-sectional area (in.^{2})

σ_y = minimum yield strength (psi)

L_{eff} = effective length, depending on end conditions (in.)

k = radius of gyration (in.)

In lieu of a more precise determination, the following approximate effective length may be used:

- Pinned-Pinned Ends Member...................... $L_{eff.} = L_{act.}$
- Pinned-Fixed Ends Member .... $L_{eff.} = 0.80\ L_{act.}$
- Fixed-Fixed Ends Member .... $L_{eff.} = 0.65\ L_{act.}$
- Fixed-Free Ends Member .... $L_{eff.} = 2.1\ L_{act.}$

where:

L_{act} = actual length, end to end between supports of the compression member

Fixed bearing resistance shall be determined based upon the minimum static design factor of safety, n_S, specified herein and applied to the minimum specified yield strength of the component material.

- Forged, drawn, rolled, wrought steel .... $n_s = 2.5$
- Cast steel... $n_s = 3$

6.6.2 Resistance of Components in Bearing at the Service Limit State

6.6.2.1 GENERAL

At the service limit state, the resistance of bearings other than roller bearings shall be designed for the lesser of allowable stresses from:

COMMENTARY

A slender column is one which may buckle at a nominal stress that is below yield.

It is noted that for steel and most other ductile materials, the yield strength in compression is the same as the yield strength in tension. That is, $\sigma_y = \sigma_{yt} = \sigma_{yc}$.

SPECIFICATIONS

- criteria to prevent permanent deformation as specified in Article 6.6.2.3 and 6.6.2.4, or
- criteria to prevent overheating and seizing as specified in Article 6.6.2.5.

6.6.2.2 BEARING ON COMPONENTS NOT SUBJECT TO MOTION

The provisions of Article 6.6.1 shall apply.

6.6.2.3 INTERMITTENT MOTION AND SLOW SPEEDS

For intermittent motion, and where speeds do not exceed 50 fpm, resistance shall be based on the following allowable bearing stresses, applied to the diametral projected area or net surface area for sliding surfaces:

- Pivots of swing bridges, hardened steel on AASHTO M 107 (ASTM B 22 Alloy UNS C91300) bronze disks .... 3,000 psi
- Pivots of swing bridges, hardened steel on AASHTO M 107 (ASTM B 22 Alloy UNS C91100) bronze disks .... 2,500 psi
- Trunnion bearings and counterweight sheave bearings, rolled or forged steel on AASHTO M 107 (ASTM B 22 Alloy UNS C91100) bronze:

  For loads while in motion .... 1,500 psi
  For loads while at rest .... 2,000 psi

- Shaft journals, rolled or forged steel on AASHTO M 107 (ASTM B 22 Alloy UNS C93700) bronze .... 1,000 psi
- Wedges, cast steel on cast steel or structural steel

  For loads while in motion .... 1,500 psi
  For loads while at rest .... 2,000 psi

- Acme screws which transmit motion, rolled or forged steel on AASHTO M 107 (ASTM B 22 Alloy UNS C90500) bronze .... 1,500 psi

For slowly rotating journals, as on trunnions, counterweight and deflector sheave bearings, and operating drum bearings, the bearing area used to determine resistance shall be taken as the gross projected bearing area, less the effective area of oil grooves.

COMMENTARY

C6.6.2.3

For materials other than those listed here, the maximum journal surface velocity (fpm) should not be exceeded.

Many materials used for plain journal bearings have a maximum allowable journal surface velocities in the range of 10 to 35 fpm. Refer to Bearing Manufacturer's catalogs. See also Table C6.7.7.1.2-1.

SPECIFICATIONS

For crank pins and similar joints with alternating application and release of stress, the bearing resistance determined as specified above may be doubled.

6.6.2.4 INTERMEDIATE SPEEDS

For intermediate motion at speeds exceeding 50 fpm, bearing resistance shall be determined using the gross projected bearing area and shall be based on the stresses specified below:

- Shaft journals, rolled or forged steel on AASHTO M 107 (ASTM B 22 Alloy UNS C93700) bronze .... 600 psi
- Thrust collars, rolled or forged steel on AASHTO M 107 (ASTM B 22 Alloy UNS C93700) bronze .... 200 psi

The provisions of Article 6.6.2.5 shall not be exceeded.

6.6.2.5 HEATING AND SEIZING

The maximum allowable bearing resistance, R_b (lb.), shall be taken as:

$$R_b = \alpha\left(\frac{A}{nD}\right) \quad (6.6.2.5\text{-}1)$$

where:

A = planar or projected area over which the load is acting (in.^{2})

n = rotational speed - revolutions per minute (RPM)

D = diameter of journal or step bearing, or mean diameter of collar or screw (in.)

α = a factor specified herein (lb./in.·RPM)

The factor α shall be taken as follows:

- For sleeve bearing shaft, journals rolled or forged steel on bronze... 250,000
- For step bearings, hardened steel on bronze... 60,000
- For thrust collars, rolled or forged steel on bronze... 50,000

COMMENTARY

C6.6.2.4

The maximum journal surface velocity (fpm) is different for various plain bearing materials. Most plain journal bearings are intended for slow speed applications only. Use rolling element bearings for most intermediate and high speeds, based on journal surface velocity.

C6.6.2.5

The purpose of this provision is to avoid heating and seizing on sleeve bearing shaft journals, step bearings for vertical shafts, thrust collars, and Acme thread power screws.

At high rotational speeds, rolling element bearings are strongly preferred. Use of plain bearings in this type of application can lead to excessive wear and shortened life.

SPECIFICATIONS

- For acme screws, rolled or forged steel on bronze 220,000

6.6.2.6 BEARING ON ROLLERS

The maximum allowable line bearing resistance, R_R per unit width (lb./in.) on rollers used in steel-steel interfaces shall be taken as:

$$R_R = \alpha\left(\frac{\sigma_y - 13{,}000}{20{,}000}\right) \qquad (6.6.2.6\text{-}1)$$

where:

σ_y = minimum yield strength of the weaker material (psi)

D = diameter of the roller (in.)

α = factor specified herein (lb./in.)

	Diameters Up to 25 in.	Diameters 25 to 125 in.
For rollers in motion:	400 D	2,000 $\sqrt{D}$
For rollers at rest:	600 D	3,000 $\sqrt{D}$

For manufactured trunnion and counterweight sheave roller bearings, the provisions of Article 6.7.7.2.4 shall apply.

6.6.3 Design for the Fatigue Limit State

6.6.3.1 GENERAL

The loads generated from operation at full load torque or normal operating pressure shall be used unless otherwise specified.

For steel parts other than spur gears subjected to cyclic stresses, when the expected number of load cycles is expected to be more than one million, design shall be based on the endurance limit specified herein.

COMMENTARY

C6.6.2.6

The guidelines in this article are typically used for special steel rollers designed for the following uses: swing span rim bearing rollers, swing span balance wheels (center bearing), end lift rollers or rockers, span guide rollers, counterweight guide rollers, etc.

Equation 1 is based on steel-steel interface only. Use of Equation 1 for other materials would be erroneous.

For steel and most other ductile materials, the yield strength in compression is the same as the yield strength in tension.

C6.6.3.1

From field measurements of several movable bridges, it has been found that the operating machinery constant velocity running loads usually range from 40 percent to 70 percent of full-load torque. Where the movable span is likely to be subject to frequent operation under heavy wind or ice loading, a load factor of up to 1.5 may be applied.

Where mechanical components are subjected to high numbers of cyclic stresses during the expected life of the part, designing statically may not be adequate, and a component may fail by fatigue.

SPECIFICATIONS

COMMENTARY

The endurance limit is referred to as the constant amplitude fatigue threshold in the specifications for fixed bridges.

Nonferrous metal such as aluminum or copper alloys are examples of materials which do not exhibit endurance limits.

6.6.3.2 ENDURANCE LIMIT

For wrought Carbon and Alloy steels, and for Stainless Steels, subjected to cyclic stresses, the endurance limit shall be taken as:

$$\sigma_e = \alpha\, \sigma_{ut}\ (C_D\ C_S\ C_R\ C_T\ C_M) \qquad (6.6.3.2\text{-}1)$$

where:

σ_{ut} = specified minimum ultimate tensile strength (psi)

α = factor depending on material (DIM) taken as:

- for wrought carbon and alloy steel and for Ferritic stainless steels..$\alpha = 0.50$
- for cast steels and for Austenitic stainless steels ....$\alpha = 0.40$

C_D = size factor (DIM) based on shaft diameter, D (in.) taken as:

- For $D \leq$ 0.3 in., $C_D = 1$ (6.6.3.2-2)
- For $D >$ 0.3 in., $C_D = (D / 7.6)^{-0.113}$ (6.6.3.2-3)

C_S = surface roughness factor (DIM) taken as:

$$C_S = a\ (\sigma_{ut})^b \qquad (6.6.3.2\text{-}4)$$

C_R = reliability factor (DIM)

- When using minimum specified ultimate tensile strength, $C_R = 1$.
- When using Typical Ultimate Strength properties for the value of σ_{ut}, C_R shall be taken as:

Reliability Percent	C_R
90.0	0.897
99.0	0.814
99.9	0.753

C6.6.3.2

The equation for the endurance limit σ_e is a function of many factors, as shown herein.

For steel mechanical components subjected to fewer than one million, but more than 10,000 cycles of stress during its expected life or for materials that have no true endurance limit, design shall be based on a finite life fatigue resistance.

The value of α specified for Austenitic stainless steel is based upon Deutschman, 1975.

Since the shaft diameter is not initially known in design, estimate D in the 4 to 8 in. range and C_D in the 0.7 to 0.75 range. Alternatively, the diameter may be estimated by doing a static failure analysis, and finding a diameter based on the maximum stress, the yield strength of the material and an appropriate design factor.

For components other than round (shafts), refer to Norton, 1998 for equivalent diameter equations.

These values of C_R are based on an assumed standard deviation in ultimate tensile strength of 8 percent. Typical or average values for the ultimate tensile strength, σ_{ut}, are often given in AISI and ASM tables of properties.

SPECIFICATIONS

C_T = temperature factor, usually taken as 1, except for very high or low temperatures (Juvinall, 1967) (DIM)

C_M = any miscellaneous factors applicable to the details of a particular design (DIM)

Table 6.6.3.2-1 - Variables for Determining C_s (Surface Roughness Factor) (Shigley, 1989)

Condition	a	b
For a ground surface	1.34	-0.085
For a cold finished or smooth machined surface with $R_a \leq$ 32 µin.	2.7	-0.265
For a hot rolled or rough machined with $R_a >$ 32 µin., or as a heat treated surface	14.4	-0.718
For an as cast or as forged surface	39.9	-0.995

R_a = surface roughness factor taken as an arithmetic mean (µin.)

6.6.4 Resistance of Open Spur Gearing Using Allowable Stresses

6.6.4.1 GENERAL

Three criteria shall be satisfied in the design of open spur gears:

- failure of the teeth at the fatigue limit state,
- surface durability through pitting and wear resistance, and
- resistance for overload conditions.

6.6.4.2 SPUR GEAR BENDING RESISTANCE AT THE FATIGUE LIMIT STATE (S_{at})

Resistance shall be based upon the allowable bending stress S_{at} (psi) used in the equations specified in Article 6.7.5.2.2.

When through hardened steel gear teeth having a Brinell hardness between 180 and 400 are subjected to one-way bending during any revolution and use AGMA Grade 1 material, then S_{at} may be taken as:

COMMENTARY

Miscellaneous factors are associated with welding, shot peening, plating, corrosion, or other manufacturing or environmental factors required for the specific design conditions. Shot peening will give a C_M value greater than 1, while the others will give values less than 1.

See National Standard ANSI/ASME B46.1.

C6.6.4.2

See AGMA 2001-C95 or latest revision for the definition of Grade 1 steel.

SPECIFICATIONS

$S_{at} = 77.3\ H_B + 12{,}800$ (6.6.4.2-1)

where:

H_B = Brinell hardness for the teeth (DIM)

For gears subjected to two-way bending during a rotation, such as idler gears, use 70 percent of this value for S_{at} given above.

6.6.4.3 ALLOWABLE SPUR GEAR CONTACT/ DURABILITY/WEAR STRESSES AT THE FATIGUE LIMIT STATE

Resistance shall be based upon the allowable contact stress S_{ac} (psi) used in the equations specified in Article 6.7.5.2.3.

For through hardened steel gear teeth having a Brinell hardness between 180 and 400 and use AGMA Grade 1 material, S_{ac} may be taken as:

$S_{ac} = 322\ H_B + 29{,}100$ (6.6.4.3-1)

6.6.4.4 ALLOWABLE SPUR GEAR YIELD STRESSES FOR INTERMITTENT OVERLOAD

Resistance applicable to the overload limit state shall be as specified in Article 6.7.5.2.4 using the values of S_{ay} specified herein.

For annealed or normalized gear teeth having a Brinell hardness between 150 and 240, S_{ay} shall be taken as:

$S_{ay} = 2\ H_B^2 - 300\ H_B + 31{,}000$ (6.6.4.4-1)

For quenched and tempered gear teeth having a Brinell hardness between 180 and 400, S_{ay} shall be taken as:

$S_{ay} = 482\ H_B - 32{,}800$ (6.6.4.4-2)

6.6.5 Wire Rope Allowable Stresses

Counterweight ropes shall be proportional such that:

- The total tensile stress from axial load and bending does not exceed 22.2 percent of the specified minimum ultimate tensile strength of the rope.
- The tension from the direct load only does not exceed 12.5 percent of the specified minimum ultimate tensile strength of the rope.

COMMENTARY

This could be extrapolated to an H_B value as low as 150.

AGMA defines an idler gear as any gear whose teeth see a stress reversal (two way reverse bending) every revolution. Operating drums on span drive vertical lift bridges may have idler gears, so that each hoisting drum rotates in the desired direction. Also a rack on a bascule span has teeth that see reverse bending every opening and closing cycle.

C6.6.4.3

See AGMA 2001-C95 or latest revision for the definition of Grade 1 steel.

This could be extrapolated to an H_B value as low as 150.

C6.6.4.4

These criteria have intended to prevent yielding by bending of teeth under overloads.

C6.6.5

Refer to Article 6.8.3.3.4 for wire rope stress equations.

For operating ropes, the respective corresponding limits shall be taken as 30 percent and 16.7 percent.

6.7 MECHANICAL MACHINERY DESIGN

6.7.1 General

Machinery shall be designed for loading from:

- the prime mover as specified in Article 5.7,
- holding as specified in Article 5.5, and
- braking as specified in Article 5.7.3.

6.7.2 Requirements for Design with Static Stresses

6.7.2.1 GENERAL

These provisions shall be taken to apply at the service limit state.

Stresses which are cyclic and changing shall not be treated as static unless the total number of stress cycles during the life of the part are less than about 10,000 cycles, or the range of stress is small in comparison to the average stress.

Determination of resistance based on allowable static stresses is specified in Article 6.6.1.

Where a component does not satisfy the criteria above, it shall be designed for the fatigue limit state as specified in Article 6.6.3.

C6.7.2.1

Static stresses are stresses that are uniform, usually caused by uniform loads acting on the stationary component.

6.7.2.2 UNIAXIAL NORMAL STRESS AND SHEAR STRESS

Where components comprised of ductile materials are subjected to static, steady loads, resistance to static uniaxial tension or bending shall satisfy the following:

$$\sigma_{max} \le \frac{\sigma_{yt}}{n_s} \qquad (6.7.2.2\text{-}1)$$

where:

σ_{yt} = specified minimum tensile yield stress (psi)

σ_{max} = maximum normal stress resulting from the applied loads (psi)

n_s = static factor of safety (DIM)

C6.7.2.2

It is not necessary to include stress concentration factors in the stress calculation when the failure mode is yielding.

For the values for n_s refer to Article 6.6.1.

SPECIFICATIONS

Where components comprised of ductile materials are subjected to static loads of direct shear or pure torsion, but are not subjected to axial or bending loads, resistance shall satisfy:

$$\tau_{max} \leq \left(\frac{\sigma_{yt}}{2n_s}\right) \tag{6.7.2.2-2}$$

where:

τ_{max} = maximum shear stress resulting from the applied loads (psi)

6.7.2.3 COMBINED STRESSES

Resistance of components subjected to simultaneous loads producing uniaxial normal stress and shear stress shall satisfy:

$$\tau_{max} \leq \frac{\sigma_{yt}}{2n_s} \tag{6.7.2.3-1}$$

in which:

$$\tau_{max} = \sqrt{\left(\frac{\sigma}{2}\right)^2 + \tau^2} \tag{6.7.2.3-2}$$

where:

σ = applied uniaxial normal stress due to loads producing tension, bending, or both (psi)

τ = applied shear stress due to loads producing torsion, direct shear, or both (psi)

6.7.3 General Requirements for Design with Fluctuating Stresses at the Fatigue Limit State

6.7.3.1 GENERAL

The fatigue limit state shall be considered where a machinery component is subjected more than about 10,000 cycles of stress during the component's lifetime.

COMMENTARY

C6.7.2.3

The most common case of combined stresses in machinery components is a combination of uniaxial normal, σ, and shear, τ, stresses, where the normal stress is caused usually by bending or axial loads and the shear stresses caused by torsion or direct shear.

For ductile materials and combined stresses, one of the most commonly used theories of failure by yielding is the maximum shear stress theory.

This is the maximum shear stress from Mohr's circle, due to combined stresses.

C6.7.3.1

For an indefinite life, and for steel components, it is possible to design parts that will not experience fatigue failure during its lifetime. The design process uses the modified endurance limit, σ_e as defined in Article 6.6.3.2.

SPECIFICATIONS

6.7.3.2 STRESS CONCENTRATION FACTORS - UNIAXIAL NORMAL STRESS AND SHEAR

Stress concentration factors for fluctuating stress conditions for normal stress, K_F, and shear stress, K_{FS}, may be determined as:

$$K_F = 1 + q\,(K_t - 1) \qquad (6.7.3.2\text{-}1)$$

$$K_{FS} = 1 + q\,(K_{ts} - 1) \qquad (6.7.3.2\text{-}2)$$

in which for ductile materials,

$$q = \frac{1}{\left(1 + \frac{\sqrt{a}}{\sqrt{r}}\right)} \qquad (6.7.3.2\text{-}3)$$

where:

r = radius of notch or fillet (in.)

$\sqrt{a}$ = Neuber constant corresponding to the minimum specified ultimate tensile stress as specified in Table 1 $(\text{in.})^{0.5}$

Table 6.7.3.2-1 - Value of Neuber Constant

σ_{ut} (psi)	$\sqrt{a}$ $(\text{in.})^{0.5}$
60,000	0.108
90,000	0.070
120,000	0.049
140,000	0.039
180,000	0.024

COMMENTARY

C6.7.3.2

Stress concentrations are probably the most critical criteria to consider when designing to prevent fatigue failure. Stress concentration factors depend on the loading and the shape of the part. The most critical stress concentrations occur at locations of size or shape discontinuities, especially fillets at shoulders on shafts where the diameters change. Other locations of possible stress concentrations include keyways, other grooves, threads, holes, and similar discontinuities.

The values for K_t and K_{ts} come from figures by Peterson, or others (Pilkey, 1997). See representative figures given at the end of this section, Appendix A6.

For ductile materials only, the theoretical stress concentration factors K_t or K_{ts} from the graphs, are modified to fatigue stress concentration factors, K_F or K_{FS}, which are used in the fatigue design equations.

The equations for the fatigue stress concentration factor, K_F for bending or axial stresses and K_{FS} for torsional shear stresses, are dependent on q, the notch or fillet radius sensitivity factor, and K_t or K_{ts}, the theoretical stress concentration factors.

K_t depends on the type of loading, i.e., bending moment, axial force, torsional moment, the shape of the part, i.e., round, flat, the fillet radius size r as related to the smaller section size d, usually r/d, and the ratio of the dimensions at a change in cross-section, usually D/d.

The tables in Appendix A6 provide values of K_F or K_{FS} for analyzing parts with threads and keyways. There is no modification required, using q, since these values are given as fatigue stress concentration factors.

For a fillet or notch radius equal to 0.4 in. Equation 3 yields the following values:

Table C6.7.3.2-1 - Value of Notch Sensitivity Factor

σ_{ut} (psi)	q
60,000	0.85
90,000	0.90
120,000	0.93
140,000	0. 94
180,000	0.96

SPECIFICATIONS

6.7.3.3 FATIGUE DESIGN

6.7.3.3.1 Mean and Amplitude Stresses

The provisions of this section require consideration of mean stresses, σ_m and τ_m, and amplitude stresses, σ_a and τ_a, which shall be determined as:

$$\sigma_a = \left(\frac{\sigma_{max} - \sigma_{min}}{2}\right) \qquad (6.7.3.3.1\text{-}1)$$

$$\sigma_m = \left(\frac{\sigma_{max} + \sigma_{min}}{2}\right) \qquad (6.7.3.3.1\text{-}2)$$

$$\tau_a = \left(\frac{\tau_{max} - \tau_{min}}{2}\right) \qquad (6.7.3.3.1\text{-}3)$$

$$\tau_m = \left(\frac{\tau_{max} + \tau_{min}}{2}\right) \qquad (6.7.3.3.1\text{-}4)$$

where:

σ_{max} = maximum applied normal stress (psi)

σ_{min} = minimum applied normal stress (psi)

τ_{max} = maximum applied shear stress (psi)

τ_{min} = minimum applied shear stress (psi)

6.7.3.3.2 Fatigue Failure Theory

Components subjected to loads producing, both uniaxial normal stresses and shear stresses, shall satisfy:

$$\frac{\sigma'_a}{\sigma_e} + \frac{\sigma'_m}{\sigma_{yt}} \leq 0.80 \qquad (6.7.3.3.2\text{-}1)$$

in which:

$$\sigma'_a = \sqrt{(K_F \sigma_a)^2 + 3(K_{FS} \tau_a)^2} \qquad (6.7.3.3.2\text{-}2)$$

$$\sigma'_m = \sqrt{(K_F \sigma_m)^2 + 3(K_{FS} \tau_m)^2} \qquad (6.7.3.3.2\text{-}3)$$

COMMENTARY

C6.7.3.3.1

Stresses may vary during a cyclic stress condition from some minimum to some maximum stress, whether normal or shear stresses.

C6.7.3.3.2

The fatigue failure theory presented here will use the Soderberg failure criteria, which is a conservative theory that uses the endurance limit σ_e and the material tensile yield strength (minimum), σ_{yt} .

Another fatigue theory of failure, which is less conservative, known as the nominal mean stress method, uses K_F and K_{FS} only with the alternating, amplitude stresses, and not with the mean stresses.

When dealing with combined cyclic fluctuating stresses, both uniaxial normal (usually bending) and shear (usually torsion), it is necessary to calculate the Von Mises stress, given by Equations 2 and 3, which is an equivalent normal stress that is used in the fatigue design equations.

The terms σ'_a and σ'_m are the amplitude and mean Von Mises stresses, respectively.

With these equations, the fatigue stress concentration factors, K_F and K_{FS} are used both with the amplitude stresses and the mean stresses.

SPECIFICATIONS

where:

σ_a = amplitude normal stress specified in Article 6.7.3.3.1 (psi)

σ_e = endurance limit of a steel shaft specified in Article 6.6.3.2 (psi)

σ_m = mean normal stress specified in Article 6.7.3.3.1 (psi)

τ_a = amplitude shear stress specified in Article 6.7.3.3.1 (psi)

τ_m = mean shear stress specified in Article 6.7.3.3.1 (psi)

K_F = stress concentration factor for fluctuating normal stress specified in Article 6.7.3.2 (DIM)

K_{FS} = stress concentration factor for fluctuating shear stress specified in Article 6.7.3.2 (DIM)

6.7.4 Shafts, Trunnions, Machine Elements Subjected to Cyclic Stresses

6.7.4.1 SHAFT AND TRUNNION DIAMETER

Unless specified otherwise by the Owner, the design of shafts, trunnions, and other machinery parts subjected to more than 1 million cycles of reversed bending moment due to rotation in combination with a steady torsional moment shall satisfy:

$$\frac{32}{\pi d^3}\left[\frac{K_F M_a}{\sigma_e} + \frac{\sqrt{3} K_{FS} T_m}{2\sigma_{yt}}\right] \le 0.8 \qquad (6.7.4.1\text{-}1)$$

where:

K_F = fatigue stress concentration factor (bending) (DIM)

K_{FS} = fatigue stress concentration factor (torsion) (DIM)

M_a = amplitude bending moment (lb.-in.)

T_m = mean (steady) torsional moment (lb.-in.)

σ_e = endurance limit of the steel shaft specified in Article 6.6.3.2 (psi)

σ_{yt} = minimum tensile yield strength of the steel shaft (psi)

The diameter of shafts used for transmitting power for

COMMENTARY

C6.7.4.1

The previous editions of the AASHTO Movable Bridge Specifications were essentially devoid of any reference to the possibility of fatigue failure of shafts, trunnions, or similar machinery parts that are subjected to high numbers of stress cycles during their life, that can lead to failure especially at locations of high stress concentration.

For a shaft or trunnion of multiple diameters, it is necessary to analyze all crucial cross-sections.

For trunnion type bascule bridges, the trunnions experience a single one-way bending cycle for each complete bridge operation. Therefore, Equation 1 may be taken as:

$$\frac{32}{\pi d^3}\left[\frac{K_F M_a}{\sigma_e} + \frac{\sqrt{3} K_{FS} T_m}{2\sigma_{yt}}\right] \le 1 \qquad (C6.7.4.1\text{-}1)$$

SPECIFICATIONS

the operation of the bridge, or for shafts 48 in. or more in length forming part of the operating machinery or bridge lock system, shall be not less than 2.5 in. in diameter.

Shafts and trunnions with a diameter more than 8 in. shall have a hole bored lengthwise through the center. The hole diameter should be about 20 percent of the shaft diameter.

6.7.4.2 SHAFT LENGTH AND DEFORMATION

For solid steel shafts supporting their own weight only, the unsupported length of the shaft shall not exceed:

$$L = 80\,(D^2)^{1/3} \qquad (6.7.4.2\text{-}1)$$

where:

L = length of shaft between bearings (in.)

D = diameter of solid shaft (in.)

Where shafts are considered to be subject to misalignment resulting from the deflection of the supporting structure, they shall be made in noncontinuous lengths, and the arrangement should be such that only angular misalignment need be accounted for by the couplings. Each length of shaft should be supported by not more than two bearings.

Shafts shall be proportioned so that the angular twist in degrees per in. of length, under maximum torsional loads, shall not exceed:

- For typical shafts and where the shaft diameter exceeds 7.5 in. .... (0.05/D)
- Where less twist is desirable, as in shafts driving end-lifting devices .... (0.006)

where:

D = solid shaft diameter (in.)

COMMENTARY

The requirement that large shafts and trunnions, i.e., greater than 8 in. diameter, have a hole of about 20 percent of the diameter bored lengthwise through the center of the shaft should not effect the calculation of the required diameter, giving less than 1 percent error.

Historically, these bores have been provided to reduce residual stresses and to remove nonhomogeneous material that may result from the forming process. Additionally, these bores allow for improved heat treating, as well as easier and more accurate field alignment.

C6.7.4.2

If shafts meet the requirements for couplings, offset misalignment is satisfied by a floating shaft.

SPECIFICATIONS

6.7.4.3 SHAFT CRITICAL SPEED

The maximum speed of a shaft shall not exceed 67 percent of the critical speed of any section of the shaft as specified herein.

For a solid steel floating shaft subjected to only its own mass, the critical speed in RPM shall be determined as:

$$n_c = 4.732 \times 10^6 \frac{D}{L^2} \quad (6.7.4.3\text{-}1)$$

where:

D = diameter of the solid shaft (in.)

L = distance between supports, usually flexible gear couplings (in.)

For a simply supported shaft with a concentrated mass at about center span, the critical shaft speed in RPM shall be determined as:

$$n_c = 1.55 \times 10^6 \left(\frac{D^2}{L^2}\right)\sqrt{\frac{L}{W}} \quad (6.7.4.3\text{-}2)$$

where:

D = diameter of the solid shaft (in.)

L = distance between supports, usually flexible gear coupling (in.)

W = weight of the concentrated mass (lb.)

6.7.4.4 SHAFTS INTEGRAL WITH PINIONS

Pinions should be forged integral with their shafts where the following conditions are satisfied:

- if the required shaft size is approximately equal to the root diameter of the teeth;

COMMENTARY

C6.7.4.3

Torque transmitted through line shafting should be kept as small as is practical in order to minimize the weight of line shafting and associated bearings, couplings, and supports. This results in higher line shaft speeds which must be verified to be safely under the shaft critical speed.

C6.7.4.4

SPECIFICATIONS

- if the minimum gear hub thickness is less than 40 percent of the shaft diameter.

6.7.5 Design of Open Spur Gearing

6.7.5.1 GENERAL

The equations specified herein apply only to full depth spur gear teeth.

Unless specified otherwise, gear teeth:

- shall be machine cut,
- shall be of the involute type,
- shall have a pressure angle of 20 degrees.

For spur gear pitch diameter tooth speeds over 600 fpm and where quiet operation is desired, an enclosed helical gear speed reducer should be considered.

Unless otherwise specified, all gear teeth shall be cut from solid rims. For open spur gears, the AGMA gear quality shall be Class 7 or higher and the backlash shall be as established by AGMA based on center distance and diametral pitch (Pd).

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/Pd, nor more than 14 Pd (not less than 8, nor more than 14, times the tooth module).

The diametral pitch of spur gears shall not be less than:

- 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.

COMMENTARY

Minimum gear hub thickness for gears with keyways shall be taken as the minimum length between the keyway and root of the teeth.

C6.7.5.1

The use of stub teeth is not recommended, however, AGMA does cover unequal addendum tooth systems. The tooth geometry factors, used for both fatigue and surface durability, for unequal addendum tooth systems are different than those presented below, under Articles C6.7.5.2.2 and C6.7.5.2.3. See AGMA 908-B89.

Spur gear design is based on diametral pitch, which is defined by the number of teeth on the gear divided by pitch diameter (in.). For large teeth (Pd<1), circular pitch is commonly used instead of diametral pitch.

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Pinions, including rack pinions and motor pinions, should have not less than 18 teeth.

Use of less than 18 teeth, with a 20 degree pressure angle, may cause undercutting of the teeth, thereby weakening the critical cross-section at the root of the teeth.

If less than 18 teeth is required for the pinion, consider using the AGMA unequal addendum tooth system.

6.7.5.2 AGMA SPUR GEAR DESIGN EQUATIONS

6.7.5.2.1 General

The provisions of this article shall be taken to apply only to the design of open spur gears.

$$W_t = \frac{2T}{d} \qquad (6.7.5.2.1\text{-}1)$$

where:

T = FLT (lb.-in.) applied at gear shaft

d = pinion pitch diameter (in.)

C6.7.5.2.1

The following equations give the relationship between pitch line tangential tooth force, W_t (lb.), the torque applied to the gear shaft, T (lb.-in.), and the power which the gear transmits, P (hp).

$$P = \frac{W_t d n_p}{126{,}000} \qquad (C6.7.5.2.1\text{-}1)$$

where:

n_p = pinion rotational velocity (RPM)

SPECIFICATIONS

COMMENTARY

AGMA presents design/analysis equations ANSI/AGMA 2001-C95) to determine the safe power that can be transmitted by the gear teeth, either safe power based on fatigue strength, P_{at} (hp), or safe power based on pitting resistance/wear/ surface durability, P_{ac} (hp).

These equations have been modified into equations to find the factored flexural resistance, W_{tat}, and factored pitting resistance, W_{tac}.

6.7.5.2.2 Design for the Fatigue Limit State

Compliance with Equation 1 is intended to prevent fatigue failure of the gear teeth.

The following must then be satisfied:

$$W_t \leq W_{tat} \qquad (6.7.5.2.2\text{-}1)$$

The factored flexural resistance, W_{tat} (lb.) of the spur gear teeth, based on fatigue, shall be determined as:

$$W_{tat} = \frac{F J S_{at} Y_N}{P_d K_o K_v K_s K_m K_B S_F K_T K_R} \qquad (6.7.5.2.2\text{-}2)$$

in which:

$$K_v = \left[\frac{A + \sqrt{V_t}}{A}\right]^B \qquad (6.7.5.2.2\text{-}3)$$

$$A = 50 + 56\,(1.0 - B) \qquad (6.7.5.2.2\text{-}4)$$

$$B = 0.25\,(12 - Q_v)^{0.667} \qquad (6.7.5.2.2\text{-}5)$$

$$v_t = \pi n_p \frac{d}{12} \qquad (6.7.5.2.2\text{-}6)$$

where:

K_v = dynamic factor (DIM)

F = tooth face width of the spur pinion or gear that is being analyzed/designed (in.)

P_d = diametral pitch taken as N_p/d (in.$^{-1}$)

v_t = pitch line velocity (fpm)

d = pitch diameter of the pinion (in.)

N_p = number of teeth on the pinion

C6.7.5.2.2

Tables C1 and C2 provide values of the geometry factor, J for values of 18, 19, 20, and 21 tooth pinions, for 20° full depth, equal addendum teeth only. (Modified from AGMA 908-B89)

For a gear quality of 6 or 7 use Table C2 for J and tooth loading at the tip; use the highest single tooth contact Table C1 for J only if the gears are of high quality and accurately aligned at assembly.

Note that the factor K_V is now greater than 1. Previous editions of the AGMA Standards used K_V as less than 1.

SPECIFICATIONS

S_{at} = allowable bending stress specified in Article 6.6.4.2 (psi)

K_o = overload factor taken as > 1.0 where momentary overloads up to 200 percent exceed 4 in 8 hours, and exceed one second duration (DIM)

H_B = Brinell hardness for the teeth (DIM)

N = number of load cycles

n_p = pinion (RPM)

Q_V = gear quality number taken as an integer between 7 and 12 (DIM)

K_S = tooth size factor to reflect nonuniformity of tooth material properties, due to large tooth size, gear diameter, and face width taken as >1 (DIM)

K_m = load distribution factor taken as $K_m = 1.21 + 0.0259F$ for open gearing, adjusted at assembly, with $F < 28$ in., and $F/d < 1$ (DIM)

K_B = rim thickness factor taken as 1.0 if $m_B = t_R / h_t > 1.2$ (DIM)

m_B = the backup ratio (DIM)

t_R = rim thickness (in.)

h_t = total tooth height (in.)

S_F = safety factor for bending strength (fatigue) $S_F \geq 1.2$ (DIM)

J = geometry factor for bending strength (DIM)

Y_N = life factor for bending resistance taken as (DIM):

- For $H_B \approx 250$ and $10^3 < N < 3 \times 10^6$

$$Y_N = 4.9404\ N^{-0.1045} \quad (6.7.5.2.2\text{-}6)$$

- For $N > 3\text{x}10^6$ load cycles, regardless of hardness

$$Y_N = 1.6831\ N^{-0.0323} \quad (6.7.5.2.2\text{-}7)$$

COMMENTARY

Refer to the AGMA Standards for a definition of the Gear Quality Number. The accuracy of a gear increases with an increase of the quality number. Therefore, tighter manufacturing tolerances must be met and thus may increase cost. However, the production methods and tooling of many gearing manufacturers is such that the minimum quality number they will produce may be $Q_v = 9$ or higher. In such cases, the designer may have little or no cost savings in specifying a lower quality number.

AGMA gives no further guidance on K_S, however other references recommend using K_S of 1.2 to 1.5 for large tooth size (say $P_d < 2.5$). (Norton, 1998; Shigley, 1983)

This is an approximate equation, derived from AGMA Standard 2001-C95. See this or the latest AGMA standard for a more accurate calculation of K_m.

A good design guideline is to have $m_B > 1.2$.

See Tables C6.7.5.2.2-1 and C6.7.5.2.2-2 for suggested values of J.

SPECIFICATIONS

K_t = temperature factor taken as 1.0 for gear temperatures less than 250°F (DIM)

K_R = reliability factor (DIM):

- for 99 percent reliability
- 1.25 for 99.9 percent reliability

COMMENTARY

Table C6.7.5.2.2-1 - J Factor for 20° Full Depth Spur Pinion/Gear (P, G)

Equal Addendum Loaded at Highest Point, Single Tooth Contact								
GEAR TEETH	PINION TEETH (N_p)							
	18		19		20		21	
	P	G	P	G	P	G	P	G
18	0.30	0.30	-	-	-	-	-	-
19	0.30	0.30	0.31	0.31	-	-	-	-
20	0.30	0.31	0.31	0.31	0.31	0.31	-	-
21	0.30	0.32	0.31	0.32	0.32	0.32	0.33	0.33
26	0.31	0.34	0.31	0.34	0.32	0.34	0.33	0.35
35	0.31	0.37	0.32	0.37	0.33	0.37	0.34	0.37
55	0.32	0.40	0.32	0.40	0.33	0.40	0.34	0.40
135	0.33	0.43	0.33	0.43	0.34	0.43	0.35	0.43

SPECIFICATIONS | COMMENTARY

Table C6.7.5.2.2-2 - J Factor for 20° Full Depth Spur Pinion/Gear (P, G)

Equal Addendum Loaded at Tooth Tip								
GEAR TEETH	PINION TEETH (N_p)							
	18		19		20		21	
	P	G	P	G	P	G	P	G
18	0.23	0.23	-	-	-	-	-	-
19	0.23	0.23	0.23	0.23	-	-	-	-
20	0.23	0.24	0.23	0.24	0.24	0.24	-	-
21	0.23	0.24	0.23	0.24	0.24	0.24	0.24	0.24
26	0.23	0.25	0.23	0.25	0.24	0.25	0.24	0.25
35	0.23	0.26	0.23	0.26	0.24	0.26	0.24	0.26
55	0.23	0.28	0.23	0.28	0.24	0.28	0.24	0.28
135	0.23	0.29	0.23	0.29	0.24	0.29	0.24	0.29

6.7.5.2.3 Surface Durability and Wear - Design Equations

Compliance with Equation 1 is intended to promote surface durability and pitting resistance.

The following must then be satisfied:

$$W_t \leq W_{tac} \qquad (6.7.5.2.3\text{-}1)$$

The factored surface durability resistance, F_{taz} (N), of the spur gear teeth based on pitting resistance shall be determined as:

$$W_{tac} = \frac{FdI}{K_o K_v K_s K_m C_F}\left(\frac{S_{ac} Z_N C_H}{C_p S_H K_T K_R}\right)^2 \qquad (6.7.5.2.3\text{-}2)$$

where:

F = tooth face width of the pinion or gear that has the narrowest face width (in.)

S_{ac} = allowable contact stress for the lower Brinell hardness number of the pinion/gear pair as specified in Article 6.6.4.3 (psi)

N = number of load cycles (DIM)

C6.7.5.2.3

SPECIFICATIONS

S_H = safety factor for pitting resistance, i.e., surface durability taken as >1 (DIM)

C_p = elastic coefficient taken as 2,300 for steel pinion-steel gear $(psi)^{0.5}$

Z_N = stress cycle factor for pitting resistance taken as $2.466\ N^{-0.056}$ for $10^4 < N < 10^{10}$ (DIM)

C_H = hardness ratio factor for pitting resistance, taken as 1.0 if $H_{BP}/H_{BG} < 1.2$ (DIM)

I = geometry factor for pitting resistance (DIM)

C_f = surface condition factor for pitting resistance taken as 1.0 for good tooth surface condition as specified in Article 6.7.8 for tooth surface finish depending on module (DIM)

COMMENTARY

Table C1 gives values of I modified from AGMA 908-B89, I geometry factor tables - values for 18, 19, 20, and 21 tooth pinions, for 20° full depth, equal addendum teeth only.

Table C6.7.5.2.3-1 - I Factors for 20° Full Depth Spur Pinions/Gears

Equal Addendum Factors same for both Pinion and Gear								
GEAR TEETH	PINION TEETH (N_p)							
	18		19		20		21	
	P	G	P	G	P	G	P	G
18	0.075		-		-		-	
19	0.077		0.076		-		-	
20	0.079		0.078		0.076		-	
21	0.080		0.080		0.078		0.078	
26	0.084		0.084		0.084		0.084	
35	0.091		0.091		0.091		0.091	
55	0.100		0.101		0.102		0.102	
135	0.112		0.114		0.116		0.118	

Usually the hardness of the gear is lower than that of the pinion. H_{BP} and H_{BG} are the Brinell hardness of the pinion and gear, respectively. The H_{BP}/H_{BG} ratio will usually be less than or equal to 1.2. For example, it is common to have the pinion hardness equal to 350 BHN and the gear equal to 300 BHN, for a ratio of 1.17.

For other material combinations, i.e., steel-cast iron or steel-bronze, refer to the AGMA Standards.

SPECIFICATIONS

6.7.5.2.4 Yield Failure at Intermittent Overload

Spur gear teeth shall be investigated for an infrequent overload condition at the overload limit state for which yield failure due to bending might occur.

The following must then be satisfied:

$$W_t\ (max) \leq W_{max} \quad (6.7.5.2.4\text{-}1)$$

based on the overload condition.

The maximum factored resistance, W_{max} (lb.), based upon yield failure of the gear teeth shall be taken as:

$$W_{max} = \frac{K_y F K_f J S_{ay}}{P_d K_{my}} \quad (6.7.5.2.4\text{-}2)$$

where:

K_y = yield strength factor taken as 0.50 (DIM)

K_f = stress correction factor = 1 (DIM)

S_{ay} = allowable yield stress number specified in Article 6.6.4.4 (psi)

K_{my} = load distribution factor for overload conditions, taken as $K_{my} > 1.1$ for straddle-mounted gear (DIM)

F = tooth face width of the spur pinion or gear that is being analyzed/designed (in.)

P_d = diametral pitch (in.$^{-1}$)

J = geometry factor for bending strength (DIM)

COMMENTARY

C6.7.5.2.4

The equation given in the AGMA Standards is modified to solve for W_{max} which is the maximum allowable peak tangential tooth load that can be transmitted, based on yielding.

AGMA suggests using $K_f = 1$, since this is a yield criteria for failure of a ductile material. K_f is defined by AGMA 908-B89 (Equation 5.72).

AGMA gives an equation only for an enclosed drive.

See Tables C6.7.5.2.2-1 and C6.7.5.2.2-2 for suggested values of J.

6.7.6 Enclosed Speed Reducers

SPECIFICATIONS

6.7.6.1 GENERAL

Whenever possible, enclosed speed reducers should be used instead of open gearing.

Enclosed speed reducers shall be specified on the basis of torque at the service limit state at an AGMA service factor of 1.0 and shall resist torque at the overload limit state without exceeding 75 percent of the yield strength of any component.

Enclosed reducer bearings shall be of the rolling element type and shall have a L-10 life of 40,000 hours.

COMMENTARY

C6.7.6.1

It is recommended that all gearing, except final drive gearing, e.g., rack and pinion, be designed using enclosed speed reducers wherever feasible.

SPECIFICATIONS

Gear quality for enclosed reducers shall be AGMA Class 9 or higher, and backlash shall be in accordance with AGMA standards.

Lubrication of the gears and bearings shall be automatic and continuous while the unit is being operated.

Provisions shall be made for filling, draining, and ventilating the housings and a sight gage or dip stick shall be mounted on the unit to facilitate monitoring the lubricant level.

The design of machinery shall accommodate a ±4 percent variation in the reducer exact ratio from the design ratio in the specifications.

6.7.6.2 PARALLEL SPUR, HELICAL, AND BEVEL GEAR REDUCERS

The contract documents shall specify that spur, helical, herringbone, and bevel enclosed gear speed reducers and increasers be manufactured in accordance with the requirements of the applicable AGMA standards and shall carry the AGMA symbol on the nameplate. The nameplate shall be specified to contain the AGMA horsepower rating, the thermal rating, input and output speeds, and the exact ratio.

6.7.6.3 WORM GEAR REDUCERS

Except for the end lifts and center wedges of swing bridges, worm gearing should not be used for transmitting power to move the span. Where used, worm gear reducers should be commercial units which shall be selected on the basis of their rating under AGMA recommended practice.

The contract documents shall specify that commercial worm gear reducers shall provide that, or custom designs shall provide that:

- the worms be heat-treated alloy steel and the worm gear shall be typically phosphor, tin, or manganese alloys of bronze,
- the thread of the worm be ground and polished, and the teeth of the gear shall be accurately cut to the correct profile,
- the worm and gear thrust loads be taken by rolling element bearings, mounted in water and oil-tight housings, and
- the unit shall be mounted in a cast-iron or steel/cast steel housing and the lubrication shall be continuous while in operation.

COMMENTARY

See the provisions of Article 6.10.4.2.

C6.7.6.3

It is recommended that use of worm gearing should be limited to enclosed gear reducers.

SPECIFICATIONS

Worm gear units that are used for end and center lifts or wedges of swing bridges shall be self-locking.

The contract documents shall specify that worm gear speed reducers and worm gear motors be manufactured in accordance with the requirements of applicable AGMA standards and shall carry the AGMA symbol on the nameplate. The nameplate shall be specified to contain the AGMA horsepower rating, the thermal rating, input and output speeds, and the exact ratio.

6.7.6.4 PLANETARY GEAR REDUCERS

The contract documents shall specify planetary gear reducers be manufactured in accordance with the requirements of applicable AGMA standards and shall carry the AGMA symbol on the nameplate. The nameplate shall be specified to contain the AGMA horsepower rating, the thermal rating, input and output speeds, and the exact ratio.

6.7.6.5 CYCLOIDAL SPEED REDUCERS

The contract documents shall specify that the nameplate contain the horsepower rating, the thermal rating, input and output speeds, and the exact ratio.

6.7.6.6 MECHANICAL ACTUATORS

The contract documents shall specify that mechanical actuators using ball screws, with recirculating balls, or using the Acme screw and nut, shall be standard manufactured enclosed units.

The ball screw actuators shall be specified to have a brake to lock the actuator in position.

The Acme screw actuators shall be specified self-locking, depending on the friction and the pitch of the screw.

6.7.7 Bearing Design

6.7.7.1 PLAIN BEARINGS

6.7.7.1.1 General

Bearings shall be placed close to the points of loading and located so that the applied unit bearing pressure will be as nearly uniform as possible.

Large journal bearings shall be of the split type with one half recessed into the other half. The length of a bearing shall be not less than its diameter. The base half of bearings for gear trains and for mating gears and pinions shall be in one piece. The caps of bearings shall be secured to the bases with turned bolts with square heads

COMMENTARY

C6.7.6.5

Cycloidal speed reducers do not use gears to produce a speed reduction, but cycloidal discs and pin rollers.

C6.7.6.6

These units are typically driven by electric motors through use of helical gearing or worm gear drives.

Ball screw actuators are typically nonlocking, because of low friction.

A brake unit is recommended as a precautionary measure. Also, brake units, usually motor brakes, aid in accurate positioning.

C6.7.7.1.1

Large journal pillow block bearings are generally greater than 4 in. bore diameter.

SPECIFICATIONS

recessed into the base or threaded dowels and with double hexagonal nuts. The nuts shall bear on finished bosses or spot-faced seats.

Where it is obvious that aligning and adjustment will be necessary during erection, provisions shall be made for the aligning of bearings by means of shims, and for the adjustment of the caps by means of laminated liners or other effective devices.

Large bearings shall be provided with effective means for cleaning lubrication passages without dismantling parts. Jacking holes shall be provided between machinery bearing caps and bases to facilitate maintenance.

The shaft (journal) should be specified to be at least 100 BHN points harder than the metallic bearing material.

Thrust loads shall be absorbed by using thrust flanges on the bearing, or by thrust collars or thrust washers.

6.7.7.1.2 Plain Bearing Design Equations

Plain cylindrical bearings, i.e., sleeve bearings, that are boundary lubricated shall be sized based on three main parameters: pressure, surface velocity of journal, determined as indicated below, and the product of the two.

$$p = \frac{F_{ur}}{(DL)} \quad (6.7.7.1.2\text{-}1)$$

$$V = \frac{p\,D n}{12} \quad (6.7.7.1.2\text{-}2)$$

where:

F_{ur} = applied radial load (lb.)

p = pressure (psi)

V = surface velocity (fpm)

D = diameter of the journal (bearing I.D.) (in.)

L = length of the bearing (in.)

n = journal rotational speed (RPM)

Where better data is not available, the maximum values for p, V and pV for various commonly used bronze bearing alloys may be taken from Table 1.

COMMENTARY

This requirement is specified because of the variability of the hardness of metallic bearing materials.

C6.7.7.1.2

It is common practice to reduce the projected area, D x L, by about 5 percent if grease grooves are present, unless a more accurate projected area is known.

Radial bearing wear is directly related to the product of pV whereas bearing life is indirectly related to pV. Refer to bearing manufacturers as the factors used to determine bearing life vary significantly with material, whether the material is metallic or nonmetallic, the type and method of lubrication, and contamination of the lubricant.

The relationship between D and L is generally that the length, L, should usually be between 100 percent and 150 percent of the diameter, D.

SPECIFICATIONS

Table 6.7.7.1.2-1 - Performance Parameters for Cast Bronze Bearings

UNS ALLOY	p (psi)	V (fpm)	pV (psi·fpm)	COMMON NAME
C 86300	8,000	25	70,000	Mang. Bronze
C 91100	2,500	50	30,000	Phos. Bronze
C 91300	3,000	50	30,000	Phos. Bronze
C 93700	1,000	250	30,000	Tin Bronze
C 95400	3,500	100	50,000	Alum. Bronze

6.7.7.1.3 Lubricated Plain Bearings

Journal bearings should have bronze bushings. For lightly loaded bearings, the bushings may be bronze or a nonmetallic substance as specified herein. For split bearings, the bushing shall be in halves and shall be provided with an effective device to prevent its rotation under load. The force tending to cause rotation shall be taken as 6 percent of the maximum load on the bearing and as acting at the outer circumference of the bushing. A clearance of approximately ¼ in. shall exist between the bushing of the cap and the bushing of the base into which laminated liners shall be placed. The inside longitudinal corners of both halves shall be rounded or chamfered, except for a distance of 0.4 in. from each end or from the shaft shoulder fillet tangent point.

Bushings for solid bearings shall be in one piece and shall be pressed into the bearing bore and effectively held against rotation.

6.7.7.1.4 Self-Lubricating; Low Maintenance Plain Bearings

6.7.7.1.4a Metallic Bearings

The oil-impregnated powdered metal bearings shall comply with the provisions of ASTM Standards B 438, oil-impregnated sintered bronze, B 439, oil impregnated iron-base sintered, and B 783, ferrous powdered metal.

COMMENTARY

C6.7.7.1.4a

The most common of this type of bearing is the oil-impregnated or graphite impregnated copper alloy (bronze) or iron alloy powdered metal bearings.

Caution should be used when specifying a stainless steel shaft (journal) with oil-impregnated bearings. The "300 Series" Austenitic Stainless Steel may not be as satisfactory as a "400 Series" Ferritic Stainless Steel because of the high nickel content in the 300 Series reacting with the normally used oil-impregnated lubricant.

Table C1 lists maximum values for p, V and pV for various commonly used oil-impregnated bearing materials:

SPECIFICATIONS

COMMENTARY

Table C6.7.7.1.4a-1 - Performance Parameters for Oil-Impregnated Metals

MATERIAL	ASTM NO.	p (psi)	V (fpm)	pV (psi·fpm)
Oilite Bronze	B-438-73 Gr 1 Type II	2,000	1,200	50,000
Super Oilite	B-439-70 Gr 4	4,000	225	35,000
Super Oilite - 16	B-426 Gr 4 Type II	8,000	35	75,000

6.7.7.1.4b NonMetallic Bearings

Plastic bearing materials, such as nylons, acetal resins (Delrin), TFE fluorocarbons (Teflon), PTFE, and fiber reinforced variations of these materials may be used where conditions permit.

C6.7.7.1.4b

As a general guide to the important properties of "plastic" bearings and other plastic parts, refer to ASTM D 5592 "Standard Guide for Material Properties Needed in Engineering Design Using Plastics."

Refer to manufacturers of "plastic" bearings for detailed information on the allowable p, V, and pV values, and any particular design methods. The properties of some nonmetallic bearing materials are given in Table C1.

Table C6.7.7.1.4b-1 - Performance Parameters for Nonmetal Bearings

MATERIAL	p (psi)	V (fpm)	pV (psi·fpm)
Acetal ("Delrin")	1,000	1,000	3,000
Nylon	1,000	1,000	3,000
Phenolics	6,000	2,500	15,000
TFE ("Teflon")	500	50	1,000
PTFE Composite	10,000	150	25,000

6.7.7.2 ROLLING ELEMENT BEARINGS

6.7.7.2.1 General

Rolling element bearings shall be designed at the service limit state so that the L-10 life shall be 40,000 hours at the average running speed of the bearing and shall also be designed at the overload limit state and shall satisfy:

$$P \geq F \qquad (6.7.7.2.1\text{-}1)$$

C6.7.7.2.1

The life of rolling element bearings is based on the L-10 life which is defined by the American Bearing Manufacturers Association as the life for which 90 percent of a group of identical bearings will survive under a given equivalent radial load (i.e.,10 percent failures).

SPECIFICATIONS

where:

P = factored radial resistance specified in Articles 6.7.7.2.2 and 6.7.7.2.4 (lb.)

F = factored radial load applied specified in Article 6.7.7.2.2 and 6.7.7.2.4 (lb.)

Where separately mounted in pillow blocks, bearings shall be self-aligning. Housings should be of cast steel and shall be split on the centerline. Seals shall be designed to retain lubricants and to keep dirt or moisture out of the bearing.

Foot-mounted pillow block bases for units with small bore, usually taken to mean under 3 in., should have slotted holes at the mounting feet to permit easy erection, adjustment and replacement. If the mounting feet have slotted holes, the feet shall have machined ends to permit the use of end chocks or the unit shall be doweled in place after installation and alignment.

6.7.7.2.2 Rolling Element Bearing Design

Where a radial roller bearing is intended to have an L-10 life greater than 40,000 hours of life with a minimum reliability of 90 percent, the equivalent dynamic factored radial resistance, P_r (lb.), shall be determined as:

$$P_r = 0.77 C_r \left(\frac{1}{n}\right)^{0.3} \quad (6.7.7.2.2\text{-}1)$$

where:

C_r = basic dynamic radial load rating of the bearing (lb.)

n = rotational speed of the bearing inner race (RPM)

For radial roller bearings, the factored dynamic radial load F_{xy} (lb.) may be determined as:

$$F_{xy} = X\,F_{ur} + Y\,F_{ua} \geq F_{ur} \quad (6.7.7.2.2\text{-}2)$$

where:

F_{ur} = applied radial load (lb.)

F_{ua} = applied axial load (lb.)

X = dynamic radial load factor (DIM)

Y = dynamic thrust load factor (DIM)

COMMENTARY

C6.7.7.2.2

Design of Rolling Element Bearings is covered by American Bearing Manufactures Association (ABMA) standards. Where rolling element bearings are used for trunnion bearings on bascule spans or sheave/counterweight trunnion bearings on vertical lift spans, refer to Articles 6.7.7.2.3 and 6.7.7.2.4)

For the design/sizing of single row roller bearings, the equations given in the standards (ANSI/ABMA Standard 11-1990 or Latest Edition) have been modified into a resistance equation format used in Equation 1.

Nomenclature varies slightly from ABMA standard practice in order to maintain continuity within this document.

The factored dynamic radial load F_{xy} shall not be less than the applied radial load F_{ur}.

Values for X and Y are obtained from manufacturer's catalogues, for the particular size and type of bearing under consideration.

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Where P_r exceeds 0.5 C_r, the provisions of Article 6.7.7.2.4 shall also apply.

6.7.7.2.3 Roller Bearings for Heavy Loads

Where specified in the contract documents, rolling element bearings shall be used to support the counterweight sheave shafts of vertical lift bridges, fixed trunnions on bascule spans, and similar shafts/trunnions carrying heavy loads.

Each roller bearing shall be of a type, or shall be so mounted, such that the deflection of the trunnion/shaft will produce no overloading of any part of the bearing or housing. The bearing rollers shall:

- be relatively short for their diameter,
- be closely spaced in separator cages,
- run between hardened-steel races mounted in the housing and on the shaft.

The bearing mountings on each shaft shall be such that the trunnion or shaft will be restrained from axial movement by one mounting, and be free to move in the other mounting.

The ratio of length to diameter of any cylindrical roller or roller segment shall not exceed 3.25. For segmented rollers, the ratio of the total length of roller to diameter ratio shall not exceed 6.5.

Cylindrical roller bearings shall be provided with rolling element thrust bearings capable of restraining an axial thrust equal to 15 percent of the total radial load on the shaft or trunnion. Spherical or tapered roller bearings shall be proportioned for a minimum design axial load equal to the greater of the applied axial load or 15 percent of the total radial load on the bearing.

Each roller bearing shall be mounted in an oil- and water-tight steel housing, which shall be provided with means for replenishing the lubricant and arranged for convenient access for thorough cleaning of the operating parts.

The rolling element bearing shall have a means for ease of removal from the trunnion/shaft by hydraulic or other acceptable process.

Rollers and races shall be of bearing quality steel, as specified in ASTM A 295 and A 485 for through hardened steels and ASTM A 534 for carburized steels. The typical hardness level ranges shall be Rockwell C hardness 60 to 65 for the rollers and 58 to 64 for the races. Bearings specified shall be made by a manufacturer of established

COMMENTARY

When the outer race of the bearing rotates and the inner race is stationary, some bearing manufacturers require an extra rotation multiplication factor in addition to X.

C6.7.7.2.3

Heavy loads are generally defined as being greater than 675,000 lb.

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reputation who has had bearings of comparable size of the same materials and type in successful service for at least ten years.

6.7.7.2.4 Sizing of Large Rolling Element Bearings

For slow rotation large rolling element bearings, or those that do not complete one full revolution during its loading cycle, sizing should be based on C_{or}, the basic static radial load rating of the bearing.

The factored radial design resistance, P_{or} (lb.) for a bearing subjected to static loads and low speeds shall be determined as:

$$P_{or} = \frac{C_{or}}{n_s} \qquad (6.7.7.2.4\text{-}1)$$

where:

- For counterweight sheave trunnion bearings of vertical lift bridges, the static design factor should be taken as...$n_S = 5$
- For bascule spans with a fixed trunnion, the static design factor should be taken as ....$n_S = 8.5$

For a combination of the applied radial and axial loads acting on the bearing, the factored bearing load, F_{oxy} (lb.), shall be determined as:

$$F_{oxy} = X_o F_{ur} + Y_o F_{ua} \qquad (6.7.7.2.4\text{-}2)$$

where:

F_{ur} = applied radial load (lb.)

F_{ua} = applied axial load taken as not less than 15 percent of F_{ur} (lb.)

X_o = a static axial load factor (DIM)

Y_o = a static radial load factor (DIM)

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C6.7.7.2.4

Large rolling element bearings generally have a bore larger than 4 in., and a rotational speed less than 5 RPM.

It is necessary to work closely with bearing manufacturers on the large rolling element bearings. The specific operating parameters may necessitate special lubricating or other requirements, especially in applications such as a bascule trunnion bearing whose inner race operates at less than one quarter revolution each cycle.

As an example, one manufacturer of large spherical roller bearings, for the 232 Series, uses $X_o = 1$ and $Y_o \approx 2$. (Y_o varies from about 1.75 for a 14 in. to about 2.06 for a 36 in. bore bearing in this series.)

Therefore, if the axial force is 15 percent of the radial force, this gives $F_{oxy} \approx 1.3\ F_{ur}$. For a vertical lift span, the required static radial load rating would be:

$$P_{or} = \frac{C_{or}}{5} \qquad (C6.7.7.2.4\text{-}1)$$

then:

$$\frac{C_{or}}{5}\ {}^{3}\ 1.3F_{ur} \quad \text{or} \quad C_{or}\ {}^{3}\ 6.5F_{ur}$$

The values of X_o and Y_o are values that depend on bearing size and type, and manufacturer.

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6.7.8 Fits and Finishes

The fits and surface finishes for machinery parts, specified in Table 1, are in accordance with ANSI B4.1, Preferred Limits and Fits for Cylindrical Parts, and ASME B46.1, Surface Texture.

Fits other than the preferred fits listed in this ANSI Standard may be used.

Surface finishes are given as the arithmetic average roughness height (R_a) in microinches; if additional limits are required for waviness and lay, they shall be specified.

The fits for cylindrical parts, specified in Table 1, shall also apply to the major dimensions of noncylindrical parts.

Table 6.7.8-1 - Fits and Finishes

PART	FIT	FINISH R_a(µin.)
Machinery base on steel	--	250
Machinery base on masonry	--	500
Shaft journals	RC6	8
Journal bushing	RC6	16
Split bushing in base	LC1	125
Solid bushing in base (to ¼ in. wall)	FN1	63
Solid bushing in base (over ¼ in. wall)	FN2	63
Hubs on shafts(to 2 in. bore)	FN2	32
Hubs on shafts (over 2 in. bore)	FN2	63
Hubs on main trunnions	FN3	63

COMMENTARY

C6.7.8

Roughness height does not define waviness or flatness, which is a separate criteria. A surface may be "smooth", but be wavy or warped, therefore, not a proper flat surface to mount machinery to.

Fits other than those listed in Table 6.7.8-1 may be used at the discretion of the Engineer.

The range of fits given for hubs on main trunnions allows the designer some flexibility to provide the most appropriate fit for the particular detail.

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Turned bolts in finished holes	LC6	63
Sliding bearings	RC6	32
Center discs	--	32
Keys and keyways		
Top and Bottom	LC4	63
Sides	FN2	63
Machinery parts in fixed contact	--	125
Teeth of open spur gears		
Over 3 diametral pitch	--	32
3 to 1.75 diametral pitch	--	63
Under 1.75 diametral pitch	--	125

6.7.9 Hubs, Collars, and Couplings

6.7.9.1 HUBS

If practical, the length of all wheel hubs should be not less than the diameter of the bore, and for gear wheels not less than 1.25 times the width of the teeth. The minimum thickness at any place on the hub, especially at keyways, should not be less than 0.4 of the gross section diameter of the bore.

All hubs shall be provided with keys, splines or mechanical shrink fit assemblies designed to carry the total torque to be transmitted to the shaft.

For bascule trunnion hubs, the provisions of Article 6.8.1.3.1 shall apply for required fit with structural members.

C6.7.9.1

See also the provisions of Article 6.7.4.4 which covers when to make pinions integral with their shafts.

6.7.9.2 COLLARS

Collars shall be provided wherever necessary to prevent the shaft from moving axially. Where the collar is held in position by set screws, there shall be at least two set screws, 120 degrees apart. The set screws shall have cone points, and the shafts shall be counterbored for the set screws. The edges of the holes shall be peened over the set screws after the collars are adjusted. If a shaft or

C6.7.9.2

There are other alternatives to this type of set-screwed collar, e.g., split clamp-type one or two piece collars, shrink disk friction locking hub, shrink disk friction locking assemblies, and keyless friction locking bushings. These types of assemblies usually require no shaft drilling, key slots, or other shaft preparation, which may also cause stress risers in the shaft. Also these assemblies will

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trunnion receives an axial force, there shall be a thrust bearing to prevent axial movement.

6.7.9.3 COUPLINGS

When practical, all couplings used in connection with the machinery should be standard manufactured flexible couplings placed close to the bearings.

Couplings between machinery units should be of the gear type, providing for angular misalignment or for both angular and offset misalignment.

Couplings connecting machinery shafts to electric motor or internal combustion engine shafts shall be flexible couplings, transmitting the torque through metal parts and providing for both misalignment and shock.

The use of shock resistant couplings, with nonmetal torque transmitting components, may be permitted only where the coupling design is such that normal operating torques can be transmitted by the coupling in the event of nonmetallic component failure.

All coupling and shaft fits and finishes shall comply with the provisions of Article 6.7.8 for hubs on shafts.

All couplings should be keyed to the shafts. Keyed couplings shall be fitted to their shafts in the shop, the couplings after manufacture being shipped to the manufacturers of the shafts as necessary to accomplish this result.

Where necessary, couplings utilizing mechanical shrink fit assemblies or some other means of indexing shall be provided to permit infinite and repeatable axial and angular machinery alignment.

6.7.10 Keys and Keyways

6.7.10.1 GENERAL

Keys for securing machinery parts to shafts should be parallel-faced, with square or rectangular cross-section.

All keys shall be fitted into keyways machined into the hub and shaft. Preferably, the keyways in the shaft should have closed ends, which shall be milled to a semi-circle equal to the width of the key. Keyways shall not extend into any bearing or shaft shoulder fillet. Keyways shall have a fillet in the bottom in each corner according to ANSI B17.1.

Keys shall have a width, b (in.), and a height, h (in.), conforming to ANSI Standards for keys. The total length of the key or keys shall be such that the allowable stresses in shear and bearing specified in Article 6.6.1 are not exceeded.

Keys that are not set into the closed-end keyways shall be held by safety set screws, or other effective means; in

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transmit a torque while preventing axial motion. See also the provisions of Article 6.7.12.

C6.7.9.3

Angular alignment couplings are known as single engagement type; angular and offset alignment couplings are known as double engagement type.

This criteria rules out almost all shock resistant couplings with nonmetallic elastomeric flexing elements usually made of neoprene. One type of coupling uses square jaws on each metal hub, with a nonmetallic or soft metal “spider” between the metal jaws of each hub. In the event of a spider failure, the square metal jaws would still transmit the torque, although poorly and with much noise, and with a lot of backlash between the jaws.

C6.7.10.1

Tapered keys may be used to meet special requirements.

A table of ANSI preferred square and rectangular key sizes, and the shaft size range they are to be used on, is given in Table C1.

Bearing stresses are usually the controlling factor when the cross-section of the key is rectangular, $h < b$.

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vertical shafts, collars clamped about the shafts, or similar devices shall be used.

In hubs of spoked wheels, the keyways shall be located in the centers of the spokes.

6.7.10.2 CAPACITY OF KEYS

Keys used to transmit loads generated by the prime mover from a shaft to another component, i.e., gears, couplings, etc. shall have sufficient resistance to develop the full torsional strength of the shaft.

If two keys are required, they shall be placed 120° apart. When using two keys, each key shall be capable of

COMMENTARY

Table C6.7.10.1-1 - Key Sizes

DIAMETER (in.) OVER-TO-INCLUDE		KEY SIZE (in.) (b x h)
5/16	7/16	3/32´ 3/32
7/16	9/16	1/8´ 1/8
9/16	⅞	3/16´ 3/16
⅞	1¼	¼ × ¼
1¼	1⅜	5/16´ 5/16
1⅜	1¾	⅜ × ⅜
1¾	2¼	½ × ½
2¼	2¾	⅝ × ⅝
2¾	3¼	¾ × ¾
3¼	3¾	⅞ × ⅞
3¾	4½	1 × 1
4½	5½	1¼ × 1¼
5½	6½	32 × 18
6½	7½	1¾ × 20
7½	9	2 × 1½
9	11	2½ × 1¾
11	13	3 × 2
13	15	3½ × 2½
15	18	4 × 3
18	22	5 × 3½
22	26	6 × 4
26	30	7 × 5

C6.7.10.2

It is desired to have the shaft fail before the key(s) since signs of distress in the shaft will normally be more visible. Key failure may more easily lead to an uncontrolled situation, but yield failure of the shaft may have a higher tendency to misalign components and jamb in place.

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carrying 60 percent of the full torsional strength of the shaft.

The resistance of keys used to connect components that are minor parts shall be greater than the force effect corresponding to the torsional requirement of the component.

For trunnions and similar parts which are designed chiefly for bending and bearing, the keys shall have sufficient resistance to hold the trunnion from rotating. The force tending to cause rotation shall be as specified in Article 6.8.1.3.1.

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A shaft that is part of the main drive train may also be used to drive an electrical component such as a selsyn or limit switch unit. The key is sized based on the torsional requirement of this component.

Keyslots cause a stress concentration which must be addressed in the fatigue analysis of the shaft.

6.7.11 Splines

Splined connections for securing machinery parts to shafts shall use standard involute splines complying with ANSI B 92.1, Involute Splines, providing either a major diameter fit or a side fit.

Where cut splines are used, the capacity of the splined shaft may be approximated as that of an equivalent shaft of a size equal to the root diameter of the spline. Stress concentration should be considered using a factor analogous to a "sled-runner" keyslot.

C6.7.11

(For guidance on design of metric involute splines, see ANSI Standard B 92.2M which is derived from ISO 4156 Standard. This standard is not a "soft metric" conversion of the inch-based standard, and, therefore, components would not be compatible. See also Machinery's Handbook., 25th ed. Pg 2064.)

See Appendix A6, Table A6.1-2 for K_f values.

6.7.12 Mechanical Shrink / Friction Fit Assemblies

Mechanical shrink fit assemblies for securing machinery parts to shafts shall be a frictional, keyless, shaft/hub locking device. Each assembly shall be furnished as a complete unit from one manufacturer. The assemblies shall provide an adjustable shrink fit between hubs and shaft for infinite axial positioning and rotational indexing without loss of transmissible torque capacity.

C6.7.12

Caution should be used when using a "frictional" device to transmit a torque. Careful sizing of the device and specifying of the correct assembly tightening torque is extremely important.

6.7.13 Motor and Machinery Brake Design

6.7.13.1 GENERAL

For all brakes and spans where practical, the pressure on the rubbing surface of the brake should not exceed 30 psi, and the product of the pressure on the rubbing surface times the velocity of the brake wheel surface in feet per minute (fpm) should not exceed 90,000 psi-fpm.

Brakes shall be provided with adjustable electrical, or preferably, mechanical means for delaying brake setting so that all brakes do not set at the same time, thereby inducing excessively high torques in the machinery.

C6.7.13.1

Velocity is usually taken at the rim surface for drum brakes, or disc speed at the disc-pad radius for disc brakes.

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6.7.13.2 REQUIREMENTS FOR ELECTRICALLY RELEASED MOTOR BRAKES

The brakes for the span-driving motors, designated as motor brakes, shall be fail-safe type disc or shoe (drum) brakes which are held in the set position by springs with such force as to provide the retarding torques specified herein. Disc hubs or brake wheels for the motor brakes shall be mounted on the main motor shaft, or on a rear motor shaft extension.

Brakes shall be designed for intermittent duty. The brakes shall be designed to release when the current is on and to set automatically when the current is cut off. Brakes for the span operation shall be provided with mechanical or electrical escapements, such that the brakes will not be applied simultaneously.

The brakes shall be equipped with a means for adjusting the torque and shall be set in the shop for the specified torque. Each brake shall be provided with a nameplate which shall be specified to state the rated torque range of the brake and the actual torque setting. Shoe-type brakes shall be designed so that it is possible to adjust the brakes or replace the shoe linings without changing the torque settings.

Brakes released by direct-current shall be released by hydraulic power units, thrustor units or shunt-coil solenoids. Shunt coils shall have discharge resistors so as to avoid high transient voltage upon opening of the shunt-coil circuit.

Brakes released by alternating-current shall be released by hydraulic power units, thrustor units, or motor operators. Hydraulic power units and thrustor motors exposed to the atmosphere shall be totally enclosed, nonventilated with special weatherproof insulation and conduit box.

For shoe-type (drum) brakes, the releasing mechanism shall be capable of exerting a force of not less than 130 percent of the force actually required to release the brake when set at the specified torque setting, and at the lowest ambient temperature expected at the site of the bridge.

The brakes for motors other than main drive motors shall be solenoid-released, spring-applied, shoe-type brakes or dry-type disc brakes. Brakes shall have an intermittent rating not less than the full load torque of the motors with which they are used.

All brakes shall be of a construction which ensures uniform wear, and shall be provided with independent adjustments for adjusting lining wear, equalizing clearance between friction surfaces, and adjusting the retarding torque. The brake linings shall be of materials which are not affected by moisture and are a nonasbestos material. The solenoids, hydraulic power units, thrustor units, and motor operators shall be moisture proof. All fittings shall be

COMMENTARY

C6.7.13.2

The term “fail-safe” shall mean that the brake will set when electrical power is discontinued.

The designer will normally be selecting brake units as a standard manufactured item with special modifications. The requirements set forth will affect layout on the contract drawings and will likely need to be included in the contract specifications.

corrosion resisting. Hydraulic power units and thrustors shall be provided with all-weather oil.

Shoe-type brakes shall be provided with a low force hand release lever permanently attached to the brake mechanism and arranged so that one person can operate the releases easily and rapidly. Means shall be provided for latching the lever in the set and released positions.

Disc-type brakes shall have provisions for hand release and arranged so one person can operate them easily and rapidly and which can be latched in the released position.

When brakes are located outside of the machinery house, they shall be of weatherproof construction or shall be provided with a weatherproof housing. The housing shall be arranged to permit operation of the hand release from outside the housing.

If installed on the moving portion of the span, the brakes shall be designed to operate satisfactorily in any position of the span.

Consideration should be given to specifying that brakes be provided with:

- heating elements to prevent the accumulation of moisture and frost,
- limit switches for control, and
- lights on the operator's control panel to indicate the position of the brakes and their hand release levers.

6.7.13.3 ELECTRICALLY RELEASED MACHINERY BRAKES

The brakes for the span-operating machinery which are designated as machinery brakes shall meet the requirements for the motor brakes specified in Article 6.7.13.2, except as otherwise specified herein.

6.7.13.4 HYDRAULICALLY RELEASED MACHINERY BRAKES

The brakes for the span-operating machinery which are designated as machinery brakes shall meet the requirements for the motor brakes specified in Article 6.7.13.2, except as otherwise specified herein.

6.7.13.5 HAND OR FOOT RELEASED BRAKES

For calculating the strength of the machinery parts under the action of manually-operated brakes, the force applied at the extreme end of a hand lever shall be assumed at <u>150 lb.</u> and the force applied on a foot pedal shall be assumed at <u>200 lb.</u> Under this condition, the

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normal allowable unit stresses for any linkage or mechanism may be increased 50 percent.

Hand brakes and foot brakes should be arranged so that the brake is applied by means of a weight or spring and released manually.

6.7.14 Machinery Support Members and Anchorage

6.7.14.1 MACHINERY SUPPORTS

In the design of structural parts subject to loads from machinery or from forces applied for moving or stopping the span, due consideration shall be given to securing adequate stiffness and rigidity and the avoidance of resonance. Beams subject to such stresses should have a depth not less than 1/8 of their span. If shallower beams are used, the section shall be increased so that the deflection will not be greater than if the above limiting depth had not been exceeded. Deflections shall be investigated sufficiently to insure that they will not interfere with proper machinery operation.

6.7.14.2 ANCHORAGE

Anchor bolts or other anchorages that resist uplift shall be designed to provide at least 1½ times the uplift and to support that force at the allowable stress.

6.7.15 Fasteners, Turned Bolts, and Nuts

All bolts for connecting machinery parts to each other or to supporting steelwork should be high-strength bolts conforming to AASHTO M 164 (ASTM A 325), or ASTM A 490.

SAE grade bolts may be used in place of AASHTO or ASTM grades at the discretion of the Engineer. In no case shall less than SAE Grade 5 be used.

Where specified, all turned bolts shall have turned shanks, semi-finished, washer-faced, hexagonal heads, and rolled, cut, or ground threads. The finished shanks shall be 1/16 in. larger in diameter than the major diameter of the thread. The bolt blank size, usually 1/8 in. larger than the thread size, shall determine the head dimensions.

The dimensions of all bolt heads shall be in accordance with the ANSI heavy hex structural bolt series specified in ANSI B18.2.6, and threads shall be in accordance with the ANSI coarse thread thread series specified in ANSI B1.1.

Turned bolts shall be fitted in reamed holes, to an LC6 fit.

COMMENTARY

C6.7.15

The use of ASTM A 325, Type 2, or ASTM A 490, Type 2, low carbon martensitic steel, is not recommended in any applications where the connection is subjected to fluctuating loads or the possibility of impact loadings.

Refer to ASTM F 568 for a description of the ASTM property classes.

For details of SAE fasteners, refer to SAE J1199.

Table C1 lists standard bolt sizes.

Rolled threads are preferred because this threading process provides the lowest stress concentrations in the thread roots.

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The dimensions of all hex nuts shall be in accordance with the ANSI heavy hex nut series specified in ANSI B18.2.6, and threads shall be in accordance with the ANSI coarse thread series specified in ANSI B1.1. Nuts shall conform to the requirements of ASTM A563.

Hardened steel washers shall be in accordance with ASTM F 436.

All bolt heads and nuts shall bear on seats square with the axis of the bolt. On castings, except where recessed, the bearing shall be on finished bosses or spot-faced seats. Bolt heads which are recessed in castings shall be square. All nuts shall be secured by effective locks. If double nuts are used, both nuts shall be of standard thickness.

COMMENTARY

Table C6.7.15-1 - Standard Bolt Sizes, Thread Pitch, Tensile Stress Areas

MAJOR DIA. D (in.)	THREADS PER INCH n ($in.^{-1}$)	MINOR DIA. (in.)	TENSILE STRESS AREA A_t ($in.^2$)
⅝	11	0.5135	0.226
¾	10	0.6273	0.334
⅞	9	0.7387	0.462
1	8	0.8466	0.606
1⅛	7	0.9497	0.763
1¼	7	1.0747	0.969
1⅜	6	1.7105	1.155
1½	8	1.2955	1.405

The values listed in Table C2 for proof and tensile loads are from ASTM. For ASTM A 325 bolts, the proof loads are based on a strength of 85,000 psi (½ to 1, incl.) and 74,000 psi (1⅛ to 1½ incl.); the tensile loads are based on a minimum tensile strength of 120,000 psi (½ to 1, incl.) and 105,000 psi (1⅛ to 1½, incl.). For ASTM A 490 bolts, the proof loads are based on a strength of 120,000 psi; the tensile loads are based on a minimum tensile strength of 150,000 psi.

Table C6.7.15-2 - Proof and Tensile Loads

DIA×TPI	A 325		A 490	
	PROOF (lb.)	TENSILE (lb.)	PROOF (lb.)	TENSILE (lb.)
⅝-11	19,200	27,100	27,100	33,900
¾-10	28,400	40,100	40,100	50,100
⅞-9	39,250	55,450	55,450	69,300
1-8	51,500	72,700	72,700	90,900
1⅛-7	56,450	80,100	91,550	114,450
1¼-7	71,700	101,700	116,300	145,350
1⅜-6	85,450	121,300	138,600	173,250
1½-6	104,000	147,500	168,600	210,750

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6.7.16 Miscellaneous Machinery Requirements

6.7.16.1 SAFETY COVERS AND GUARDS

Safety guards for protection of persons shall be specified. The contract documents shall specify that all safety regulations shall be observed.

Safety gear guards shall be provided for all gears in the machinery houses. If gears or sheaves are located where falling objects may foul them, they shall be protected by metal covers that are easily removable.

6.7.16.2 DUST AND WATER PROTECTION COVERS

Dust covers shall be shown in the contract documents wherever necessary to protect the sliding and rotating surfaces and prevent dust from mixing with the lubricant.

Counterweight sheave rims shall be covered to protect them from the weather.

Covers shall be designed such that access is provided for maintenance and inspection.

6.7.16.3 DRAIN HOLES

There shall be drain holes not less than 1 in. in diameter at places where water is likely to collect.

Semicircular holes may be used in sheave or gear web to rim interfaces. The semicircular holes shall be $\frac{9}{16}$ in. minimum radius.

6.7.16.4 DRIP PANS

Drip pans shall be provided where open gearing is used and located so as to prevent excess lubrication from falling on the machinery room floor, or prevent contamination of waterways where appropriate.

6.7.16.5 COMPRESSED AIR DEVICES

Mechanical devices using power transmitted by compressed air may be used for the operation of auxiliary span drives, span locks, and end lifting and centering devices. Operating air pressure shall be no more than 100 psi.

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6.8 BRIDGE TYPE SPECIFIC MECHANICAL MACHINERY DESIGN

6.8.1 Bascule Spans

6.8.1.1 DRIVE MACHINERY

Drive machinery for bascule spans shall normally include drive motor(s), main reducer, output shafts and two pinions driving two racks mounted to the two main girders.

C6.8.1.1

There are exceptions, e.g., machinery to drive one pinion/rack centrally located on the span.

6.8.1.2 RACKS AND PINIONS

6.8.1.2.1 General

Where a multiple rack and pinion drive is used, either there shall be mechanical devices, usually a differential gear reducer, on the bridge to equalize the torques at the main pinions, or another such equalization method.

6.8.1.2.2 Racks

Racks shall be made in segments, the number of which depends on the size of the bridge. If feasible, the number of teeth in each segment should be the same. The rack segments should normally be made of high-strength steel castings, or equivalent strength steel weldments. The teeth shall be machine cut in a fixture using the true pitch radius for curved racks with fixed pitch radius, and the joints between the segments shall not vary in circular pitch from the rest of the teeth. The joint shall be machined accurately and located at the middle of a tooth space. The rack segments shall be machined on all surfaces in contact with the structure, or mounting hardware.

C6.8.1.2.2

The tolerance at the joint is the same as the circular pitch tolerance for the other teeth, as this is covered by the gear quality number.

6.8.1.2.3 Pinions

Each pinion shall be supported by two bearings, preferably rolling element self-aligning pillow blocks.

The pinion may be forged integral with its shaft, or be separate and press fitted and keyed to its shaft.

The pinion shall have no fewer than 18 machine cut teeth, and normally be 20 degree full depth involute. The pinions should be made from an alloy steel, heat treated with through-hardened teeth with a minimum hardness of 180 BHN.

C6.8.1.2.3

See Article 6.7.4.4 for guidelines on when to make the shaft integral with the pinion.

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6.8.1.3 TRUNNIONS AND BEARINGS

6.8.1.3.1 Trunnions

Trunnions shall be designed to transfer span loads to the trunnion bearings which shall include loads from the span drive machinery during operation. Torsional loading shall be resisted by keys and/or turned bolts. The provisions of Article 6.7.4.1 shall apply to fatigue design.

Trunnion design may include distinct hubs that increase the bearing area of the trunnion girders and have hub flanges that bolt to the girder webs to transfer torsional and axial loads. These trunnion hubs shall fit tightly into structural parts with an ANSI FN3 fit and the fit between the hub and trunnion shall be as specified in Table 6.7.8-1.

Trunnion designs that do not require hubs shall have an ANSI FN3 fit between the trunnion and mating structural parts. Trunnion collars and/or retaining rings shall be used to transmit torsional and axial loads. The trunnion shall have an integrally forged collar for axial positioning.

Transmitters, resolvers or encoders should be geared to the trunnion shafts if suitable for the particular installation. If synchronous position indicators are used, the receivers in the control desk shall be geared to the indicators. The gearing shall be arranged so as to give the greatest practical accuracy in indication. Use of low backlash enclosed gearing should be considered for all position indication systems.

6.8.1.3.2 Trunnion Bearings

Trunnion bearings should be self-aligning spherical plain or rolling element bearings retained in a split steel housing.

The bearing assembly shall be designed to support:

- the dead load and ice load where applicable, live load and impact loads of the bascule span when closed,
- the dead load and wind loads when open, and
- a thrust (axial) load equal to approximately 15 percent of the maximum radial load.

The bearing housings shall be adjustable to proper elevation, alignment and position on the supporting pedestals in the field by the use of full length shims.

The holes through the supporting steel housing for the anchor bolts shall be oversized holes previously drilled in the shop.

COMMENTARY

C6.8.1.3.1

When the span is closed, trunnions are subject to dead load, live load, and impact.

6.8.1.4 BUFFERS

To aid in seating the moving span smoothly, spans may be equipped with:

- specially designed air buffers,
- industrial shock absorbers, or
- a control system capable of performing the smooth seating in a positive manner.

If the buffer is a specially designed air buffer, the following recommendations shall be considered:

- Provide a bore of the cylinder of the air buffer not less than 10 in. and the stroke not less than 24 in.
- Provide three cast iron or PTFE packing rings for each piston.
- Provide each air buffer with a needle valve and a check valve. The system shall be suitable for sustaining short intervals of air pressure of 1,000 psi and a temperature of 400°F.

6.8.1.5 SPAN AND TAIL LOCKS, CENTERING DEVICES

6.8.1.5.1 Locking Devices

Single leaf spans shall either have a locking device at the toe end for each outside girder or truss to force down and hold down the toe end to its seats or live load shoe locks.

Double leaf spans shall be provided with center locks to lock together the toe ends of the spans and tail locks or latches. Center locks shall transfer live load and impact from one leaf to the other.

The locking devices of single leaf bascules and tail locks of double leaf bascules shall resist the greater of:

- any uplift force that may result from live loads, and
- maximum uplift created by the drive machinery at a stalled condition.

6.8.1.5.2 Centering Devices

The bridges shall be equipped with self-centering devices at the toe end. Transverse centering shall be accomplished by a device preferably located on the centerline of the bridge, as near the roadway level as practicable, with a total clearance not to exceed ⅛ in.

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6.8.2 Swing Spans

6.8.2.1 DRIVE MACHINERY

Drive machinery for spans shall normally include drive motor(s), main reducer, output shafts, and pinions/gears driving the operating rack. There shall be a minimum of two pinions, diametrically opposite, providing equal torque to rotate the span. Either the main gear reducer shall be of the differential type, or equalization of torque shall be provided by another method acceptable to the Engineer.

Where four pinions are used, they shall be placed in pairs which shall be diametrically opposite.

Motor brakes shall be provided at each motor, and two machinery brakes should be placed on the reducer output shafts if practical.

6.8.2.2 RACKS AND PINIONS

Racks shall be made in sections not less than 72 in. long. The joints in the rack shall be accurately finished and at the center of a tooth space, the space at the joint having the same dimension as the other tooth spacings.

If a cast track is used and loads are light, as in some pivot-bearing bridges, the rack and balance wheel track segments should be cast in one piece. In rim bearing bridges, the rack shall be separate from the track, so that the parts may be easily removed for repairs.

Each main pinion shaft shall be supported in double bearings, which shall be provided with bolted caps to permit easy removal of the pinion shaft, and to provide adjustment for wear. A thrust locking means shall be provided at the top bearing to carry the weight of the pinion and shaft. Means shall be provided for holding the pinion against movement along the shaft. The double bearing shall be proportioned for the maximum pinion load and shall be adequately braced and attached to the rim girder or superstructure.

Sufficient shims shall be provided between the bearing base and the steelwork to accommodate any necessary adjustment in position of the bearing. Where feasible, the bearings should be shipped assembled to the support steelwork with the shims in place.

6.8.2.3 PIVOT BEARING

Pivot bearing bridges shall be designed so that the entire weight of the moving span is carried on the pivot bearing when the bridge is swinging.

Pivot bearings shall consist of disk bearings or rolling element thrust bearings upon which the span rotates together with supporting pedestal.

COMMENTARY

C6.8.2.2

The design may make use of rolling element bearings, mounted in pillow blocks, to support the vertical pinion shafts. A spherical roller bearing unit will support the thrust load, whereas a cylindrical roller bearing unit may require a separate thrust bearing unit to take the pinion/shaft weight and any misalignment loads.

C6.8.2.3

SPECIFICATIONS

Disk pivot bearings shall consist of two disks, one of phosphor bronze and one of hardened steel. The disks shall be so anchored that the sliding will take place only at the surface of contact of the disks.

Rolling element thrust pivot bearings shall be designed to support the bridge weight as a thrust load and any wind loads or other horizontal forces as radial loads.

Where possible, pivots shall be designed so that the bearings may be removed and replaced while the bridge span is closed, without stopping vehicular traffic over the bridge.

Adjustment for height shall be provided.

6.8.2.4 END LIFTS

End wedges, roller lifts, or equivalent end lifting devices shall lift the ends of the bridge an amount sufficient to produce a positive reaction at each end as defined in Article 2.4.2.2.

End lifting machinery shall be proportioned to exert lifting force(s) equal to the greater of:

- the lifting force(s) specified above plus the reaction due to temperature difference specified in Article 2.5.1.2.6, or,
- the lifting force(s) required to raise each end of the span 1 in.

End lifts and supports shall be designed at allowable stresses defined in Articles 6.6.2.3 and 6.6.2.6 for the maximum positive end reaction including impact and temperature differential.

6.8.2.5 CENTER WEDGES

Center wedges and supports shall be designed at allowable bearing stresses defined in Article 6.6.2.3 for the reaction of the maximum vehicular live load and impact.

6.8.2.6 BALANCE WHEELS

Where possible, no fewer than eight wheels, moving on a circular track, should be provided to resist the tilting of the bridge while the bridge is swinging. The maximum overturning moment shall be determined using ice and/or wind loading as defined in Article 5.4.3. The balance wheel clearance with the track shall be adjustable for height, preferably by shims between the superstructure and the seats of the bearings. For short, narrow bridges, four wheels may be used.

Unless there is compelling analytic evidence to the contrary, the full overturning moment shall be resisted by

COMMENTARY

The type of rolling element bearing recommended for this application is the spherical roller thrust bearing.

C6.8.2.4

Swings spans with unequal arms will have different end lift forces from one end to the other.

C6.8.2.5

Center wedges shall not lift the bridge, but shall be designed to drive firmly to resist only the vehicular live load and impact.

A different distribution of loads to balance wheels can be justified by a suitable three-dimensional analysis which

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a single balance wheel where there are only four, and shared, 60 percent to 40 percent, by two balance wheels when there are eight or more.

When wheels are not cast integral with their axles, they shall have pressed fits thereon; either the axles should rotate in bronze-bushed bearings or the shaft should be held from rotation at the ends, and bronze-bushed bearings pressed into the balance wheels.

Balance wheel bearings shall be provided with a means for lubrication.

Balance wheels and their bearings shall be designed for twice the allowable stresses at the service limit state.

6.8.2.7 RIM BEARING WHEELS

Rollers of rim bearing or combined rim and center bearing bridges shall be proportioned for:

- the dead load when the bridge is swinging, and
- the dead load plus vehicular live, and impact loads when the bridge is closed.

In computing the load on the rollers, the rim girder shall be considered as distributing the load uniformly over a distance equal to twice the depth of the girder, out-to-out of flanges. This distance shall be symmetrical about the vertical centerline of the concentrated load.

6.8.2.8 TRACKS

Tracks should be specified to be made in sections, preferably not less than 72 in. long. The track shall be deep enough to insure good distribution of the balance wheel or roller loads to the pier for rim bearing bridges, tracks shall not be less than 4 in. deep.

The joints in the track shall be detailed to be staggered. The track shall be anchored to the pier by bolts not less than 1.5 in. in diameter, extending at least 12 in. into the pier cap, and set in mortar or grout. The track of hand-operated, center bearing bridges shall have a sufficient number of anchor bolts so that the mortar or grout in which they are set will not be crushed by the tractive force developed when turning the bridge. When center bearing bridges are operated by mechanical power and a curved rack is attached or integral with the track, the track shall be anchored down by bolts, and the reactive force developed when turning the bridge shall be resisted by lugs extending from the bottom of the track downward into the pier cap and set in cement mortar, grout or concrete.

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includes allowances for installation inaccuracies and wheel and tread irregularities.

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COMMENTARY

6.8.2.9 CENTERING DEVICES

Bridges shall be equipped with self-centering devices at one or both ends of the swing span. The centering device(s) shall preferably be located on the centerline of the bridge, as near the roadway level as practicable, with a total clearance not to exceed 5⁄16 in.

6.8.2.10 SPAN LOCKS

When the centering devices are not designed to hold the span in the closed position, bridges shall be equipped with span locks. If the swing span is left normally in the open position, it shall also be locked in this position. The span locks shall resist the greatest turning moment created by the span operating machinery while stalled.

6.8.3 Vertical Lift Spans

6.8.3.1 SPAN DRIVE VERTICAL LIFTS

6.8.3.1.1 Drive Machinery

The drive machinery for span drives should include drive motor(s), main reducer, output shafts, and pinions/gears driving the operating drums or ring gears. Motor brakes shall be provided at each motor, and two machinery brakes should be placed on the reducer output shafts where practical. The main gear reducer shall not be of the differential type.

6.8.3.1.2 Operating Ropes

The transverse deviation of a rope from a plane through the groove of a drum or sheave at right angles to the axis of the shaft of the drum or sheave shall not exceed 1 in 30, and where practical should not exceed 1 in 40.

There shall be at least two full turns of the rope on the operating drum when the span is in the fully open or closed position and, in addition, the end of the rope shall be rigidly clamped to the drum, the attachment being such as to avoid sharp bends in the wires.

6.8.3.1.3 Operating Drums and Deflector Sheaves

For operating ropes, the diameter of the drums and deflector sheaves shall be not less than 45 times the diameter of the rope, and preferably should not be less than 48 times, except for deflector sheaves with small angles of contact (less than 45 degrees) between rope and sheave. The provisions of Article 6.8.3.3.5 shall apply.

C6.8.3.1.3

SPECIFICATIONS

Operating drums shall be press fitted on their shafts, and in addition shall have keys of sufficient resistance to carry the total torque to be transmitted to the shafts.

The shape of the groove on operating drums shall conform as closely as feasible to the rope section. The center-to-center distance of the grooves shall be not less than ⅛ in. more than the diameter of the wire rope.

Deflector sheaves should have the same diameter as the drums. Intermediate deflector sheaves shall be provided as necessary to prevent rubbing of the ropes on other parts of the machinery or the bridge, and to avoid excessive sag of the ropes.

Where operating ropes have small angles of contact with deflector sheaves, the sheaves shall be supported on roller or ball bearings and shall be designed as light as practicable to insure easy turning and minimum rope slippage in starting and stopping.

All deflector sheaves shall have deep grooves to prevent displacement of the ropes.

6.8.3.1.4 Take-Up Assemblies

There shall be take-ups for controlling slack in the operating ropes consisting of turnbuckles or other devices, such as manually-operated take-up reels. The take-up assemblies shall be readily accessible and capable of being operated by one person.

6.8.3.2 TOWER DRIVE VERTICAL LIFTS

6.8.3.2.1 Drive Machinery

The drive machinery for tower drives should include drive motor(s), main reducer, output shafts and pinions driving the sheave ring gears. Motor brakes shall be provided at each motor, and, where feasible, two machinery brakes should be placed on the pinion/output shafts, as close to the pinion as practical.

6.8.3.2.2 Ring Gears and Pinions

Two pinions engaging ring gears on the two counterweight sheaves in each tower should be provided for tower drives.

The counterweight sheave ring gears should be specified to be made in segments. The segments shall be accurately fit together, with the joint at the center of a tooth space, and with no variation in the circular pitch. The segments shall be machined to accurately bolt to the counterweight sheave.

COMMENTARY

Generally sheaves with a groove depth at least equal to the wire rope diameter are considered deep grooved.

C6.8.3.1.4

The take-ups shall be such as to prevent any rotation of the ropes about their axes during adjustment.

Refer to the provisions of Article 5.7.2.1 for the maximum manual effort one person can apply to any manual device.

C6.8.3.2.2

Occasionally, four sheaves may be required per tower. In such cases, four pinions engaging ring gears on each sheave should be provided.

SPECIFICATIONS

COMMENTARY

6.8.3.2.3 Equalizing Devices

On tower drive vertical lift spans operated through pinions engaging ring gears on the counterweight sheaves, devices shall be specified to equalize the forces at the ring gear pinions when two counterweight sheaves and two pinions are used at each corner of the span, i.e., four sheaves per tower. Equalizing devices should not be used between pinions at opposite sides of the span, but adjusting devices shall be provided between such pinions to permit leveling of the span.

6.8.3.3 WIRE ROPES AND SOCKETS

6.8.3.3.1 Diameter of Wire Ropes

The diameter of counterweight ropes shall be not less than 1 in. nor more than 2.5 in. The diameter of operating ropes shall be not less than ¾ in. The actual diameter of a wire rope, taken as the diameter of the circumscribed circle, shall be measured when the rope is unstressed. The amount by which the actual diameter of a rope may differ from the nominal diameter shall be not greater than the tolerances specified in Table 1.

C6.8.3.3.1

Most common nominal diameters of wire rope in the 6 x 19 class, fiber core are in Table 6.8.3.3.6-1.

Table 6.8.3.3.1-1 - Rope Diameter Tolerance (in.)

NOMINAL DIA	UNDERSIZE	OVERSIZE
5/8 to 3/4	0	1/32
13/16 to 1 1/8	0	3/64
1 3/16 to 1 1/2	0	1/16
1 9/16 to 2 1/4	0	3/32
2 5/16 to 2 1/2	0	1/8

6.8.3.3.2 Construction

Wire ropes shall be specified to be made of improved plow steel (IPS) or extra improved plow steel (EIPS) wire. All operating ropes shall be preformed wire rope.

Wire rope shall be specified to be 6 x 19 class wire rope of 6 x 25 filler wire construction with a hard fiber core. Each strand shall consist of 19 main wires and 6 filler wires fabricated in one operation, with all wires interlocking. There shall be four sizes of wires in each strand; 12 outer wires of one size, 6 filler wires of one size, 6 inner wires of one size and a core wire. The hard fiber core should be polypropylene. Use of independent wire rope core (IWRC) may be permitted.

C6.8.3.3.2

Double extra improved plow steel (EEIPS) grade wire rope is also available in some sizes.

The use of IWRC has not been expressly permitted until this time, however, its use has wide acceptance in general industry and growing use and acceptance in movable railroad bridges.

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6.8.3.3.3 Lay

Unless otherwise specified, all wire ropes shall be right regular lay, and the maximum length of rope lay shall be taken as:

- Operating ropes: 6.75 times nominal rope diameter.
- Counterweight ropes: 7.5 times nominal rope diameter

The lay of the wires in the strands shall be such as to make the wires approximately parallel to the axis of the rope where they come in contact with a circular cylinder circumscribed on the rope.

6.8.3.3.4 Wire Rope Stresses

Where a wire rope is bent over a sheave, the maximum bending stress, σ_b in psi, on the rope may be conservatively determined as:

$$\sigma_b = E_w \frac{d_w}{D} \quad (6.8.3.3.4\text{-}1)$$

where:

d_W = diameter of the outer wires in the wire rope (in.)

D = tread diameter of sheave rope grooves (in.)

E_W = tensile modulus of elasticity of the steel wire = 30×10^6 psi

The maximum total stress in the rope, σ_t in psi, may be determined as:

$$\sigma_t = \frac{P}{A} + \sigma_b + \frac{P_o}{A} \quad (6.8.3.3.4\text{-}2)$$

where:

P = direct load on the ropes (lb.)

A = effective cross-sectional area of the ropes (in.^{2})

P_o = operating loads, e.g., the larger of starting or inertial loads (lb.)

COMMENTARY

C6.8.3.3.4

The maximum bending stress occurs on the outer-most wires at the least bending radius.

For rope of the 6 x 19 class - 6 strands of 19 main wires each, $d_W \approx d / 16$

therefore:

$$\sigma_b = \frac{1.875 \times 10^6}{D/d} \quad (C6.8.3.3.4\text{-}1)$$

where:

d = diameter of the wire rope (in.)

When determining P_o for counterweight ropes, only inertial loads are effective.

SPECIFICATIONS

The maximum total stress shall not exceed the allowable tensile stresses specified in Article 6.6.5.

6.8.3.3.5 Short Arc of Contact

When operating ropes have less than a 45 degree arc of contact with a deflector sheave, the minimum sheave diameter shall be at least 26 times the wire rope diameter.

6.8.3.3.6 Wire Rope Tensile Strengths

The minimum tensile breaking resistance, P_{ut} , for the 6x19 class (6x25 FW), fiber core wire ropes, both improved plow steel (IPS) and extra improved plow steel (EIPS), based on WRTB and manufacturers' specifications shall be taken as specified in Table 1.

Table 6.8.3.3.6-1 - Physical Properties of Rope

D (in.)	WEIGHT / LENGTH (lb./ft.)	P_{ut} (lb.) IPS	EIPS
¾	0.095	47,600	52,400
⅞	1.29	64,400	70,800
1	1.68	83,600	92,000
1⅛	2.13	105,200	115,600
1¼	2.63	129,200	142,200
1⅜	3.18	155,400	171,000
1½	3.78	184,000	202,000
1⅝	4.44	214,000	236,000
1¾	5.15	248,000	274,000
1⅞	5.91	282,000	312,000
2	6.72	320,000	352,000
2⅛	7.59	358,000	394,000
2¼	8.51	400,000	440,000
2⅜	9.48	–	488,000
2½	10.5	–	538,000

COMMENTARY

C6.8.3.3.5

Where a rope is in contact with a small deflector sheave over a short arc, taken as 45 degrees or less, the actual radius of curvature of the rope is usually larger than the deflector sheave radius.

C6.8.3.3.6

FW = Filler wire construction, total of 25 wires per strand, 19 main wires, and 6 filler wires.

WRTB Wire Rope Users Manual, 1993.

EIPS with fiber core is not listed in WRTB manual.

Values are based on a 9 percent increase in tensile strength over the IPS fiber core.

Table 1 is for bright, uncoated wire rope. For galvanized wire rope, refer to specific manufacturer's specifications. Galvanized wire rope typically has about a 10 percent lower breaking strength than the above values.

To find the ultimate tensile strength, σ_{ut} in psi, divide the values of the tensile breaking resistance, P_{ut} in lb., in the table by the area of the wire rope in in.^{2}:

$$s_{ut} = \frac{P_{ut}}{A} \qquad \text{(C6.8.3.3.6-1)}$$

in which:

$$A = 0.417\, d^2$$

for 6 x 25 FW wire rope with a fiber core.

The elongation of wire rope under load may be determined using the following E_R values :

E_R (psi)	Percent of Ultimate Load
10.8×10^6 (74 500)	0-20
12×10^6 (83 000)	21-65

and the equation:

$$d = \frac{PL}{AE_R} \qquad \text{(C6.8.3.3.6-2)}$$

SPECIFICATIONS COMMENTARY

The contract documents shall require testing to destruction of all wire ropes to verify minimum tensile breaking resistances.

6.8.3.3.7 Wire Rope Sockets

All sockets used with wire ropes, except those for 2½ in. diameter ropes, shall be specified to conform to the requirements of Federal Specification RR-S-550, latest revision. Sockets for 2½ in. diameter ropes may be cast steel conforming to ASTM A148 (A 148M), Grade 80-50.

Sockets shall be specified to be attached to the ropes by using zinc of a quality not less than that defined for high grade in the current specifications for slab zinc (Spelter) of AASHTO M 120, and using a method that will not permit the rope to slip more than 16.7 percent of the nominal diameter of the rope when stressed to 80 percent of its specified ultimate strength. The contract documents shall specify that if a greater movement should occur, the method of attachment shall be changed until a satisfactory one is found.

The sockets shall be stronger than the rope with which they are used. The contract documents shall require testing of sockets and shall specify that:

- If a socket should break during testing, two others shall be selected and attached to another piece of rope and the test repeated, and this process shall be continued until the inspector is satisfied with their reliability, in which case the lot shall be accepted, and

- If 10 percent or more of all the sockets tested break at a load less than the specified minimum ultimate strength, the entire lot shall be rejected, and new ones, of greater resistance, shall be furnished.

Pin and socket fits different from those specified by the Federal Specification may be specified by the Engineer.

Sockets shall be specified to be painted in the shop as specified for structural steel.

6.8.3.4 SHEAVES

6.8.3.4.1 General

Sheaves shall be designed based on internal stresses determined using stress analysis methods, including finite element analysis (FEA) method whenever possible. Sheaves shall be designed so that deflection of the rim under action of the ropes is within allowable tolerances for the pitch or tread diameter.

Sheaves fabricated by welding shall be specified to be made of structural steel, AASHTO M 183 (M 183M),

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COMMENTARY

M 222 (M 222M), or M 223 (M 223M) or forged carbon steel, AASHTO M 102, Class D (ASTM A 668, Class D, S4).

The resistance of the sheave shall be such that the stresses under dead load with the sheave rotating and with impact do not exceed:

- tension or compression in base metal or filler metal: 10,000 psi with stress range not to exceed 10,000 psi,

- shear in base metal or filler metal: 5,000 psi with stress range not to exceed 5,000 psi,

The contract documents shall specify that:

- the rim be fabricated from not more than three pieces of plate and stiffened by transverse ribs if necessary to carry the load,

- the rim be welded into a complete ring and the welds ground flush on all four sides before being welded into the sheave assembly,

- each web be fabricated from not more than two pieces of plate,

- web welds, if used, shall be ground flush on both sides,

- the hub shall be made from a one-piece forging,

- all welds shall be full penetration welds made with low hydrogen procedures,

- automatic submerged-arc welding shall be used to the greatest extent practicable,

- after completion of the weldment and before final machining, the sheave be stress relieved, and

- unless otherwise specified, the sheave assembly shall be stress relieved by heat treatment prior to final machining.

The contract documents shall specify that:

- the grooves be accurately machined to insure uniformity of the pitch diameter for all of the grooves, and

- the pitch diameter variation shall not exceed plus or minus 0.01 in.

SPECIFICATIONS

Operating rope drums shall conform to all general requirements of this article.

6.8.3.4.2 Counterweight Sheaves

For main counterweight ropes, the pitch diameter of the counterweight sheave, center-to-center of ropes, shall be not less than 72 times the diameter of the rope, and preferably not less than 80 times. For auxiliary counterweight ropes, the pitch diameter of the sheave shall be not less than 60 times the diameter of the rope.

The shape of the grooves shall be detailed to conform as closely as feasible to the rope section so that the ropes run freely in the grooves. The sides of the grooves shall prevent the ropes from flattening under static loads. The distance center-to-center of grooves shall be at least 6.5 mm more than the diameter of the rope.

6.8.3.4.3 Sheave Trunnions and Bearings

Counterweight sheaves shall be detailed to be shrink fitted on their trunnions, and then secured by driving-fit dowels set in holes drilled into the sheave hub and the trunnion.

Counterweight sheave bearings shall be designed so that they can be aligned in the field at proper elevation, alignment and position on the supporting steel parts by the use of full length shims, with due allowance for movements of the bearings. The holes through the supporting steel parts for the connecting bolts shall be drilled through the holes in the bearings, which are previously drilled in the shop.

COMMENTARY

C6.8.3.4.3

The dead load to be placed on the bearings may result in structural deflection which must be accounted for in design and alignment requirements.

The use of tapered full-length shims may be required to achieve a horizontal trunnion installation and maintain full bearing.

SPECIFICATIONS

6.8.3.5 COUNTERWEIGHTS AND ROPE ANCHORAGES

6.8.3.5.1 Counterweights

The balance-block pockets shall be detailed in the contract documents and shall be placed as near the ends of the counterweights as practical.

COMMENTARY

C6.8.3.5.1

This provision is required in order to aid in securing the required balance between the lift span and the counterweights at each of the four corners of the span.

SPECIFICATIONS

6.8.3.5.2 Counterweight Rope Anchorages

The connections of the counterweight ropes to the lift span and counterweights shall be detailed in the contract documents to permit replacement of any one rope without disturbing the other ropes. Provision shall also be made for replacement of all the ropes simultaneously.

On the lift span side, the counterweight ropes shall be detailed to be separated sufficiently to prevent wind induced slapping of the ropes against each other while the span is in the closed position.

COMMENTARY

C6.8.3.5.2

Replacement of all ropes simultaneously will usually require details for supporting the counterweights from the towers.

This may be accomplished either by use of widely spaced grooves on the sheaves, or by using deviations of the ropes from a vertical plane.

SPECIFICATIONS

The transverse deviation of a counterweight rope from a vertical plane through the center of the groove on the sheaves should not exceed one-half the spacing of the grooves, and shall be the same for all the ropes on a sheave. In no case shall transverse deviation of slope exceed 1 in 40. The longitudinal deviation of a counterweight rope leading from the sheave, measured from a vertical plane tangent to the pitch diameter of the sheave, shall not exceed 1 in 30, and shall be the same for all the ropes on a sheave. These deviations shall not be exceeded on the span side for the lift span in its highest possible position, and on the counterweight side for the span in the closed position.

The connections of all ropes shall be made in such manner as to prevent rope rotation and give equal loads on several ropes of a group.

The connections of all ropes shall be so made that the centerline of the rope above the socket is at all times at right angles to the axis of the socket pin for pin sockets and to the bearing face of the socket for block sockets. Rope deflector castings or equivalent devices shall be provided near the sockets, as necessary, to accomplish this.

6.8.3.5.3 Clearance Below Counterweights

The counterweights should be detailed to clear the roadway by not less than 60 in. when the span is fully open. In computing this clearance, the counterweight ropes shall be assumed to stretch 1 percent of their length.

6.8.3.6 BUFFERS

The contract documents should require provisions of either:

- air buffers or industrial shock absorbers to aid in seating the moving span smoothly, or
- a control system capable of performing the smooth seating in a positive manner.

Vertical lift spans should also be equipped with air buffers, industrial shock absorbers, or other type of bumper to aid in stopping the movable span at maximum lift height, without damage to the structure.

The provisions of Article 6.8.1.4 shall apply to the design of a custom air buffer.

COMMENTARY

This may be accomplished either by adjustment of the tension in the ropes using jacks and shims, turnbuckles, or by use of equalizers.

C6.8.3.5.3

The 1 percent stretch shall be applied in addition to calculated elastic stretch of the wire ropes from the dead load.

C6.8.3.6

The purpose of this provision is to prevent damage in the event that the span is accidentally moved beyond the prescribed limits of lift.

SPECIFICATIONS

6.8.3.7 SPAN LOCKS AND CENTERING DEVICES

6.8.3.7.1 Locking Devices

Bridge design should include locking devices at each end of the lift span to prevent the span from rising after it has been seated by the operating machinery. The locking device may be self-locking, i.e., spring loaded, with power release, or a power driven/release locking bar.

Locking devices shall be designed at two times the overload limit state to resist the maximum load produced by the prime mover at a stalled condition.

6.8.3.7.2 Centering Devices

Bridges shall be equipped with self-centering devices at each end. Transverse centering shall be accomplished by devices located on the center line of bridge, as near the roadway level as practicable, with a total clearance not to exceed ⅛ in. For truss bridges these centering devices shall be supplemented by close transverse centering of the unloaded chords, to be accomplished by special centering devices or by the span guides.

6.8.3.8 SPAN AND COUNTERWEIGHT GUIDES

The span and its counterweights shall be detailed so that they are held in position transversely and longitudinally during their movement by means of guides engaging guide flanges on the towers. Truss spans shall have transverse guides at both top and bottom chord. The guides may be of either the sliding or the rolling type. The ends of guide flanges shall be planed smooth. The guides shall be adjustable, and shall be set to provide a normal running clearance of ⅜ in., except for the transverse span guides for the seated position of the span where the clearance shall not exceed ⅛ in.

COMMENTARY

C6.8.3.8

Roller guides are strongly recommended to reduce friction and wear.

6.9 EMERGENCY DRIVES

6.9.1 Engines - For Driving Generators, Hydraulic Power Units, and for Span Drive

These requirements apply to separately mounted engines and to engines forming part of an engine-generator set or an engine driven hydraulic power unit; the provisions of Article 8.3.9 apply to electric generators.

For determining the required engine size, allowable prime mover torque overloads shall be taken as specified in Article 5.4.1.

C6.9.1

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COMMENTARY

The rated engine torque, as referred to above, shall be taken to mean the lesser of:

- the rated torque of the engine at the speed to be used for operation, measured at the flywheel, with all metal housing in place, and with radiator, fan, and all other power-consuming accessories in place, or
- 85 percent of the rated torque of the stripped engine.

Engines shall be of the industrial duty truck or marine type. The engines should be diesel and operate at a speed of not more than 1,800 rpm and preferably not more than 1,400 rpm, and shall be equipped with a governor to limit the maximum speed to the designated value. Unless otherwise specified, the engine shall have not less than 4 cylinders. If the engine is used for span operation, the contract documents shall specify that the engine be tested by the manufacturer prior to shipment to demonstrate that it will develop the rated torque specified herein.

The contract documents shall specify that the engine shall be equipped with reversing gears, preferably of the helical type, and preferably in a separate gear unit, with a gear ratio of not less than 2 to 1. Reversing shall be by means of approved friction clutch or clutches on the countershaft operated by a lever or other approved device. The arrangement shall be such that the machinery may be operated in either direction without stopping the engine. The reversing-gear unit shall satisfy the requirements given in Article 6.7.6.

All engines having a rating of 20 hp or more shall be equipped with an electric starter with generator and storage battery. Where electric current is available at the bridge, a battery charging unit shall also be provided. All engines having a rating of 60 hp or less shall also be provided with a hand cranking device, if feasible.

Provision shall be made for effectively cooling the engine. Smaller engines may be air cooled, whereas larger engines are typically liquid cooled. There shall also be provided a corrosion resisting metallic exhaust pipe and muffler discharging outside the engine room. Air inlets, including louvers, shall be arranged to assure an adequate air supply to the engines at all times.

The fuel tank shall be located outside the engine room, below the level of the intake. The tank shall be made of corrosion-resisting metal and shall be large enough to hold fuel for 30 days of normal operation. It shall be protected from the sun. It shall be equipped with an automatic gage to show the quantity of fuel in the tank. The fuel pipe and fittings connecting the fuel tank and the engine shall be of copper or brass, so arranged and supported as to effectively provide for temperature and vibration movements tending to produce fracture and leakage at

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connections. Protective fill and vent seal units shall be included to prevent accidental vapor ignition. A day-tank, including pumps, shall be provided for engines over 60 hp. The installation shall be in accordance with the requirements of the National Fire Protection Association.

A small control board containing throttle and choke controls, ignition switch, starter button, and oil and temperature gages shall be provided at the engine.

The engine shall be enclosed in readily removable metal housing unless located in a protected space, and together with reversing gears and all other engine accessories, shall be mounted in the shop on a rigid steel frame so as to form a complete engine unit ready for installation.

Indicators shall be provided in the engine room to show the position of the moving span and, if so specified, of the lifting and locking apparatus.

If cold ambient temperatures may affect starting reliability, a water jacket heater or other suitable means to warm the fuel shall be provided. Protective features shall include low oil pressure cut-out, high water temperature cut-out, engine overspeed shutdown, and overcranking protection if applicable.

The contract documents shall specify the type and quantity of spare parts for engines to be furnished.

On all bridges operated by engines, means shall be provided for interlocking the span movement with operation of the locks and wedges so that power cannot be applied to the span until locks or wedges are released. For swing spans, such interlocking between span and lock mechanisms can generally be accomplished by means of mechanical trips which will allow the gears to be engaged only in proper sequence.

When engines for span or lock operation are used in conjunction with electrically operated lights, gates or other safety devices, interlocking shall be provided which will not permit the locks to be retracted until the safety devices are in operation, nor permit the safety devices to go out of operation until the span is seated and the locks reseated.

Means shall be provided for by-passing the interlocking system in an emergency.

6.9.2 Manual Operation

6.9.2.1 GENERAL

Provision for manual operation in case of power failure shall be provided on all brakes, locks, gates and barriers. Interlock with the span mechanisms shall be provided to prevent bridge openings before gate closure.

COMMENTARY

Such interlocking can generally be accomplished by magnetically actuated trips and relays which will allow the switches and gear to be engaged only in proper sequence. Interlocking by connection to the ignition system of the engine will generally be unsatisfactory.

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6.9.2.2 HAND OR FOOT POWER

On any machinery component having a manual, i.e., hand or foot, override or emergency operation arrangement, the maximum forces to cause the action shall be no more than the values listed in Articles 5.7.2.1 and 6.7.13.5.

6.9.2.3 HAND OR FOOT BRAKES

Brakes for bridges operated by power other than electricity shall be operated manually, or where so specified, they shall be electrically operated from an auxiliary electric generator.

The provisions of Article 6.7.13.5 shall apply as applicable.

6.9.2.4 MANUAL OPERATION OF SPAN LOCKS AND LIFTS

Gear motors and other electrical actuators should be provided with an extension of the high-speed shaft to allow emergency hand operation of the mechanism, by attaching a wheel or crank. Interlocking shall be provided such that electrical operation is disabled during hand operation.

If the actuators are hydraulic, driven by an electric HPU, emergency operation shall be available, either with a hand operated pump in small installations, or emergency diesel engine driven HPU.

6.10 LUBRICATION

6.10.1 General

Provision shall be made for effective lubrication of all sliding surfaces, gearing, and of roller and ball bearings. Lubricating devices shall be easily accessible.

The contract documents shall specify that two hand-operated grease guns shall be provided for each type of grease used to service all lubrication fittings, and that all necessary adapters shall be provided for the equipment.

6.10.2 Lubrication Fittings

Lubrication fittings should be of the giant button head pressure type, with built in check valve.

If feasible, all lube fittings should be standardized to one size and type, for ease of maintenance.

COMMENTARY

C6.10.1

Consideration should be given by the designer to verification of lubrication when designing the lubrication provisions. Maintenance experience exists demonstrating that even though lubrication is being faithfully attempted, lubrication lines and grooves blocked by hardened grease sometimes create lubrication starvation, unbeknown to maintenance personnel. A significant amount of premature bearing and trunnion wear, including scoring and galling, has been attributed to this type of lubrication failure.

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COMMENTARY

6.10.3 Lubrication of Bearings

6.10.3.1 PLAIN JOURNAL BEARINGS

The contract documents shall specify that each sliding bearing requiring lubrication shall have a high-pressure grease fitting, containing a small receiving ball or cone check valve, made of steel, that will receive the grease and close against back pressure. These fittings shall be connected to the bushings of bearings by means of corrosion resisting pipe, which shall be screwed into the bushing through a hole in the cap. If the bearings are not readily accessible, the fittings shall be placed where they will be accessible, and shall be connected to the bearings by means of corrosion resisting pipe.

Grease ducts shall be so located that the lubricant will tend to flow, by gravity, toward the bearing surface. Grooves shall be provided, wherever necessary, for proper distribution of the lubricant.

The grooves for plain trunnion bearings shall be cut in the bushing. Such grooves shall be straight, parallel to the axis of the shaft, and for large bearings no fewer than three; they shall be so located that the entire bearing surface will be swept by lubricant in the lesser of:

- one movement of opening or closing the bridge, or
- 90 degrees rotation of the shaft.

Each such groove shall be served with lubricant by a separate pressure fitting. The grooves shall be of such size that an 5⁄16 in. diameter wire will lie wholly within the groove; their bottoms shall be rounded to a ¼ in. radius. The grooves shall be accessible for cleaning with a wire.

The grooves for the counterweight sheave bearings may either be in accordance with the requirements of the foregoing paragraph, or they may be spiral grooves cut in the bushing and served with pressure fittings. A clean-out hole shall be provided in the bearing base and connected to the lowest point of the spiral grooves so that the journal surface can be cleaned and the grooves flushed out. Lubricants and greases used for journal bearings shall be of a composition which will not solidify in lubrication passages during the bridge life.

In disk bearings, grooves emanating at the center and extending to the outer edge shall be specified in the upper of the two rubbing surfaces in contact. The grooves shall not be less than ¼ in. wide and deep, and the corners shall be rounded to a radius not less than half the width of the groove. The corners at the bottom of the grooves shall be filleted so there shall be no sharp corners.

Small bearings with bearing pressures less than 1,000 psi and slow or intermittent motion, and not readily accessible,

SPECIFICATIONS

may be self-lubricating bushings. Bushings shall be of a character which will not be injured by the application of oil, and the bearings shall be provided with oil holes for emergency lubrication, and oil holes shall be fitted with readily removable screw plugs.

6.10.3.2 ROLLING ELEMENT BEARINGS

Each roller bearing shall be specified to be mounted in a steel housing, which shall be provided with means for replenishing the lubricant and arranged for convenient access for thorough cleaning of the operating parts. The housing shall contain a sealing system that will prevent water and contaminant from entering while retaining the grease or oil lubricant.

6.10.4 Lubrication of Gears

6.10.4.1 OPEN SPUR GEARING

To insure that open gearing receive proper lubrication, the contract documents shall specify that the gear teeth first be clean and properly aligned for the required distribution of the lubricant on the surface of the gear teeth.

Intermittent methods of lubrication shall be permitted to open gears having a pitch line velocity of less than about 1,000 fpm.

6.10.4.2 ENCLOSED GEARING

Lubrication of the gears and bearings in enclosed gearboxes shall be automatic and continuous while in operation. Provisions shall be made for filling, draining and ventilating the housings and a sight gage or dip stick shall be mounted on the unit to read the lubricant level.

For worm gear enclosed gear boxes, the anti-friction bearings shall be mounted in water and oil-tight housings. The unit shall be mounted in a cast-iron or steel housing and the lubrication shall be continuous while in operation.

COMMENTARY

C6.10.4.1

Viscosity of the gear lubricant varies with temperature, therefore it is necessary to give the selection of the lubricants careful attention, especially in regions where temperature changes are considerable between winter and summer.

For most open gearing applications, an extreme pressure (EP) lubricant designed for open gearing is recommended. These lubricants have the necessary properties to carry heavy tooth loads, while giving the necessary lubrication to the teeth surfaces.

Heavy grades of EP lubricants have the necessary properties to allow for intermittent application of the lubricant. Many of these grades contain an asphaltic base to provide extra film adhesion to the tooth surface. The lubricant may usually be hand brushed onto the gear teeth.

C6.10.4.2

Automatic and continuous lubrication refers to the reducer gears being lubricated during every operation of the movable span. The gears may be lubricated by use of a mechanically or electrically driven pressurized pump, or they may be splash lubricated by use of slingers, or the reducer may be filled with sufficient lubricating oil to bath the gears.

For reducers subjected to long periods of inactivity, care must be taken so that the exposed portions of the gears do not suffer from rusting and corrosion.

Some reducers are filled with oil so that the gears are completely covered, thereby offering protection from rusting. However, the efficiency of this reducer may suffer due to power loss from oil churning.

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6.10.5 Lubrication of Couplings and Miscellaneous Mechanical Components

The contract documents shall specify that couplings be re-lubricated at approximately six month intervals, unless subjected to excessive misalignment, shock loads, sudden reversals, axial movement, or extreme variations in temperature or humidity.

6.10.6 Lubrication of Wire Ropes

The contract documents shall require that during fabrication, each wire shall be coated with a rust-inhibiting lubricant as it is laid in the strand, and preferably receive an additional application of lubricant during the rope closing operation. In ropes with a fiber core, the fibers shall be prelubricated before the rope closing operation.

6.10.7 Lubrication of Wedges and Strike Plates

Wedge plates, strike plates and other sliding flat surfaces should be detailed to have the capability of being lubricated while the span is closed, through the use of lube fittings and grease grooves to distribute and retain the lubricant.

COMMENTARY

C6.10.5

Gear or grid couplings require adequate lubrication for satisfactory operation and longevity.

Extreme pressure (EP) greases are normally recommended, with NLGI #0 or #1 being the normal grades, depending on coupling RPM. NLGI #0 is recommended for low speed applications.

Refer to manufacturer's catalogues for specific guidelines with regard to lubrication procedure, type to use, and interval requirements.

C6.10.6

This factory lubrication treatment protects against corrosion during shipping and storage, as well as providing lubrication during the initial service period.

Based on manufacturer's guidelines, the wire rope must be periodically re-lubricated at intervals during its lifetime. The lubricant shall be specifically manufactured as a wire rope dressing.

A good wire rope lubricant in the Wire Rope Users Manual (1993) should:

- be free from acids and alkalis,
- have sufficient adhesive strength to remain on the ropes,
- be of a viscosity capable of penetrating the interstices between wires and strands,
- not be soluble in the medium surrounding it under the actual operating conditions,
- have a high film strength, and
- resist oxidation.

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APPENDIX A6

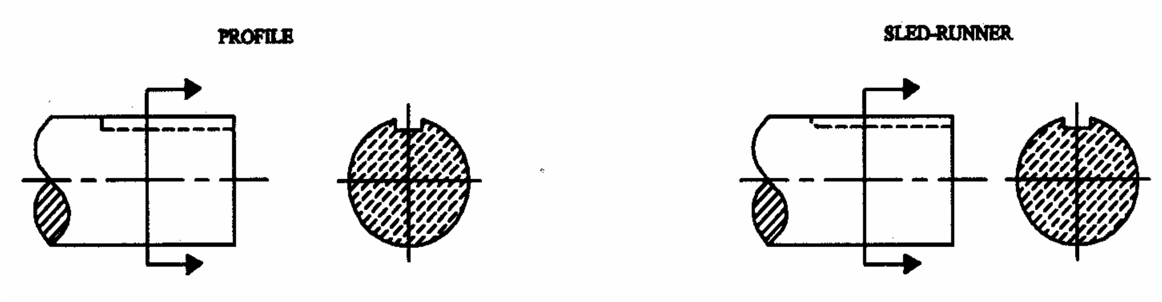
A6.1 STRESS CONCENTRATION FACTORS FOR KEYWAYS AND THREADS

The following tables give the values for K_F and K_{FS} directly. Use these when analyzing keyways and threads. There are no further calculations required using q.

Table A6.1-1 - Fatigue Stress Concentration Factors for Threads

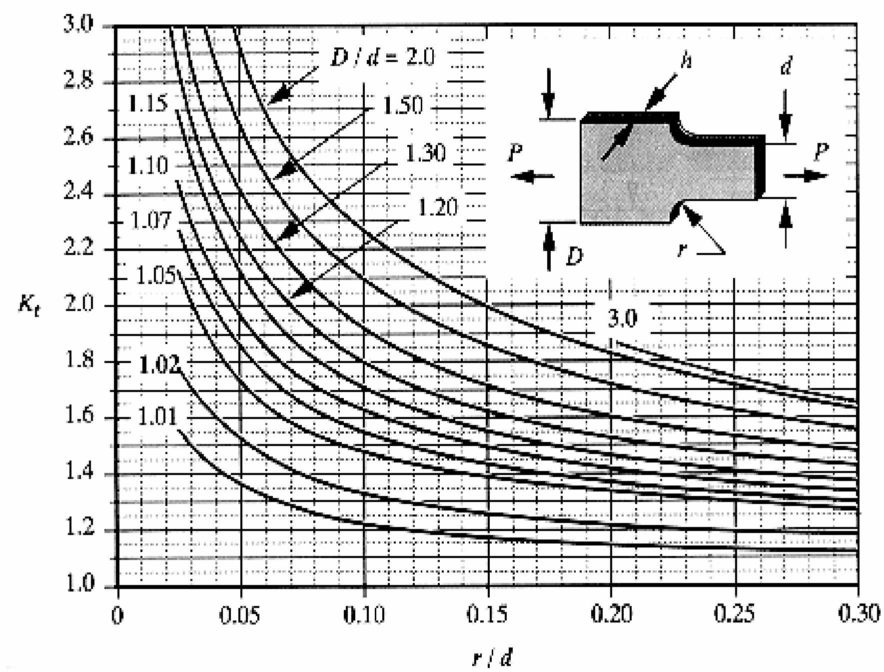
		BENDING OR TENSION	
		ROLLED	CUT
K_F or K_{FS}	ANNEALED	2.2	2.3
	QUENCHED & DRAWN	3.0	3.8

Table A6.1-2 - Fatigue Stress Concentration Factors for Keyways



	PROFILE		SLED-RUNNER	
	BENDING K_F	TORSION K_{FS}	BENDING K_F	TORSION K_{FS}
ANNEALED	1.6	1.3	1.3	1.3
QUENCHED & DRAWN	2.0	1.6	1.6	1.6

A6.2 CHARTS OF THEORETICAL STRESS CONCENTRATION FACTORS

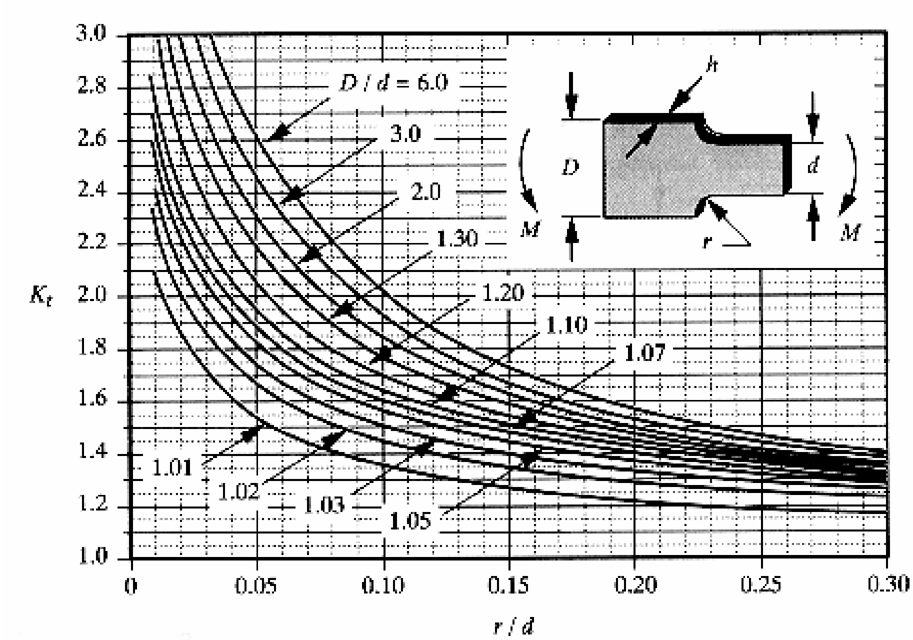


$$K_t \cong A\left(\frac{r}{d}\right)^b$$

where :

D/d	A	b
2.00	1.099 60	−0.320 77
1.50	1.076 90	−0.295 58
1.30	1.054 40	−0.270 21
1.20	1.035 10	−0.250 84
1.15	1.014 20	−0.239 35
1.10	1.013 00	−0.215 35
1.07	1.014 50	−0.193 66
1.05	0.987 97	−0.138 48
1.02	1.025 90	−0.169 78
1.01	0.976 62	−0.106 56

Figure A6.2-1: Geometric Stress Concentration Factor K_t for a Filleted Flat Bar in Axial Tension (Norton 1998).

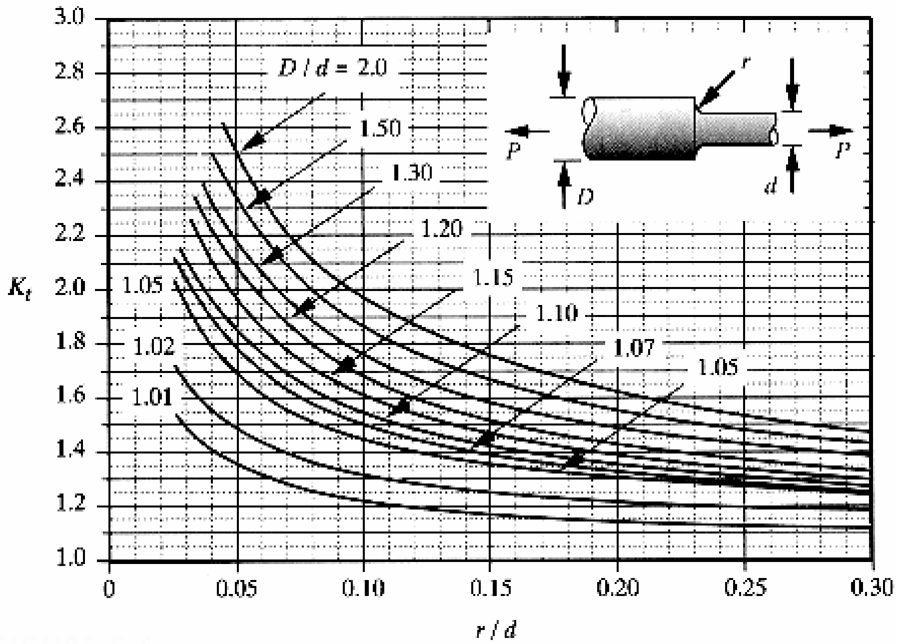


$$K_t \cong A\left(\frac{r}{d}\right)^b$$

where :

D/d	A	b
6.00	0.895 79	−0.358 47
3.00	0.907 20	−0.333 33
2.00	0.932 32	−0.303 04
1.30	0.958 80	−0.272 69
1.20	0.995 90	−0.238 29
1.10	1.016 50	−0.215 48
1.07	1.019 90	−0.203 33
1.05	1.022 60	−0.191 56
1.03	1.016 60	−0.178 02
1.02	0.995 28	−0.170 13
1.01	0.966 89	−0.154 17

Figure A6.2-2: Geometric Stress Concentration Factor K_t for a Filleted Flat Bar in Bending (Norton 1998).

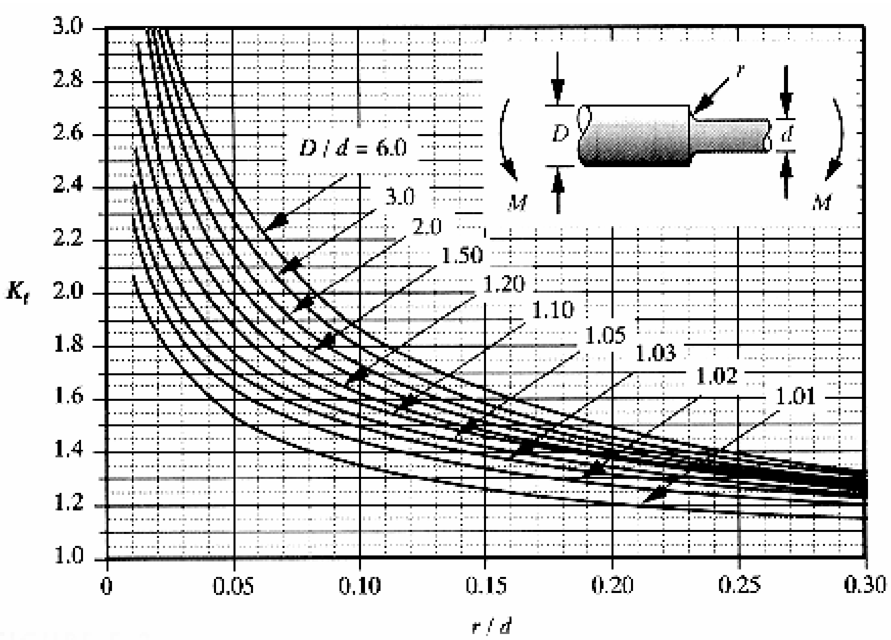


$$K_t \cong A\left(\frac{r}{d}\right)^b$$

where :

D/d	A	b
2.00	1.014 70	–0.300 35
1.50	0.999 57	–0.282 21
1.30	0.996 82	–0.257 51
1.20	0.962 72	–0.255 27
1.15	0.980 84	–0.224 85
1.10	0.984 50	–0.208 18
1.07	0.984 98	–0.195 48
1.05	1.004 80	–0.170 76
1.02	1.012 20	–0.124 74
1.01	0.984 13	–0.104 74

Figure A6.2-3: Geometric Stress Concentration Factor K_t for a Shaft with a Shoulder Fillet in Axial Tension (Norton 1998).

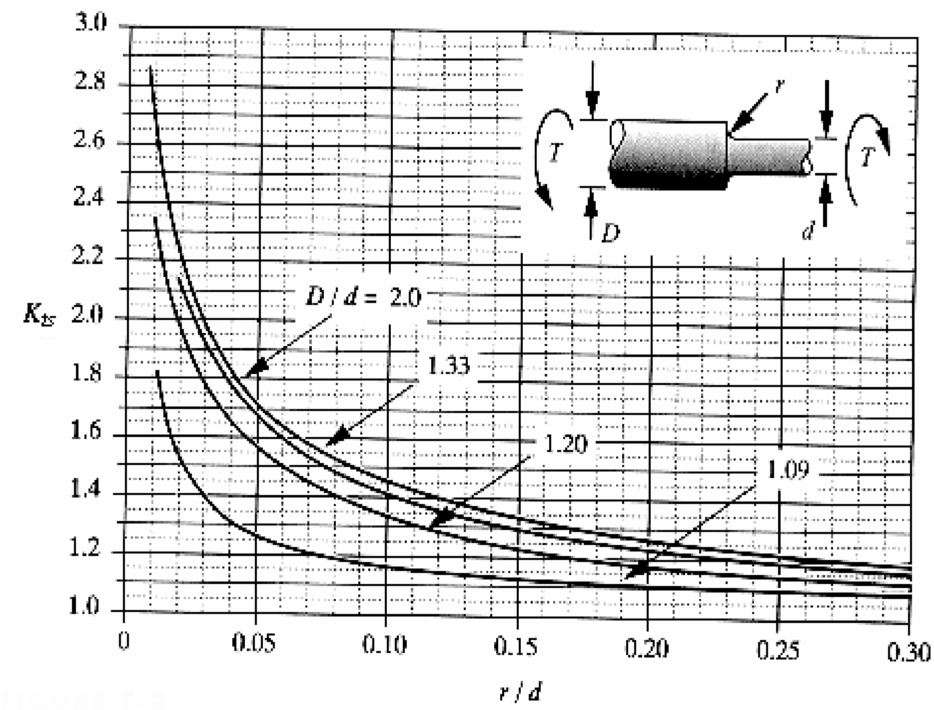


$$K_t \cong A\left(\frac{r}{d}\right)^b$$

where :

D/d	A	b
6.00	0.878 68	–0.332 43
3.00	0.893 34	–0.308 60
2.00	0.908 79	–0.285 98
1.50	0.938 36	–0.257 59
1.20	0.970 98	–0.217 96
1.10	0.951 20	–0.237 57
1.07	0.975 27	–0.209 58
1.05	0.981 37	–0.196 53
1.03	0.980 61	–0.183 81
1.02	0.960 48	–0.177 11
1.01	0.919 38	–0.170 32

Figure A6.2-4: Geometric Stress Concentration Factor K_t for a Shaft with a Shoulder Fillet in Bending (Norton 1998).

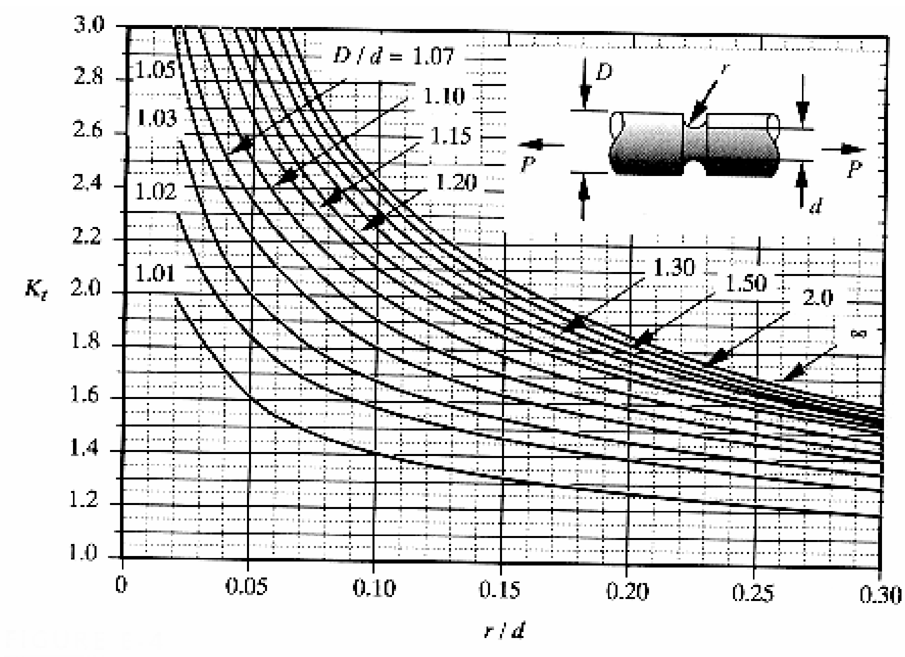


$$K_{ts} \cong A\left(\frac{r}{d}\right)^b$$

where :

D/d	A	b
2.00	0.863 31	–0.238 65
1.33	0.848 97	–0.231 61
1.20	0.834 25	–0.216 49
1.09	0.903 37	–0.126 92

Figure A6.2-5: Geometric Stress Concentration Factor K_{ts} for a Shaft with a shoulder Fillet in Torsion (Norton 1998).

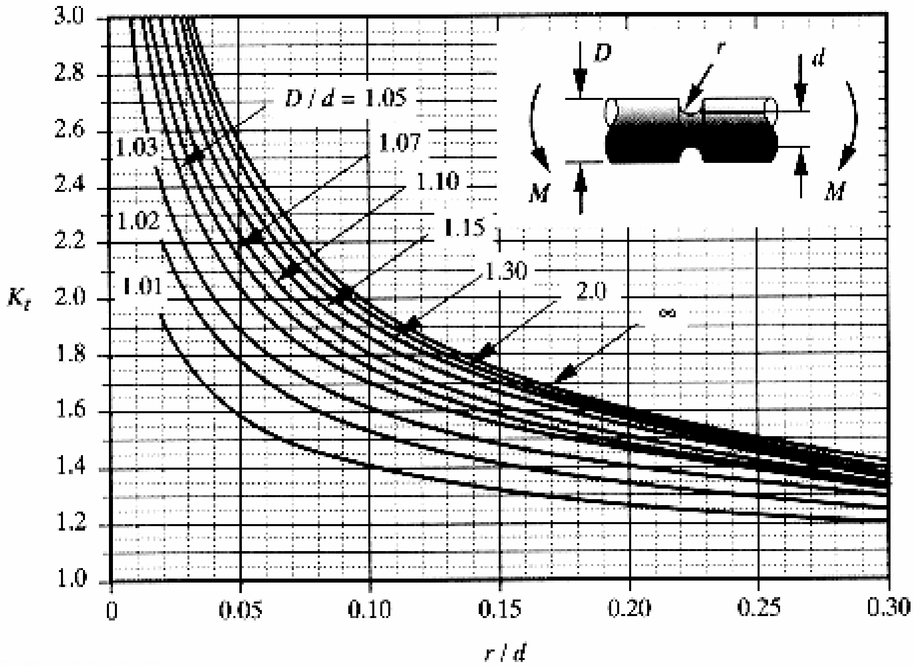


$$K_t \cong A\left(\frac{r}{d}\right)^b$$

where :

D/d	A	b
∞	0.993 72	–0.393 52
2.00	0.993 83	–0.382 31
1.50	0.998 08	–0.369 55
1.30	1.004 90	–0.355 45
1.20	1.010 70	–0.337 65
1.15	1.026 30	–0.316 73
1.10	1.027 20	–0.294 84
1.07	1.023 80	–0.276 18
1.05	1.027 20	–0.252 56
1.03	1.036 70	–0.216 03
1.02	1.037 90	–0.187 55
1.01	1.000 30	–0.156 09

Figure A6.2-6: Geometric Stress Concentration Factor K_t for a Grooved Shaft in Axial Tension (Norton 1998).

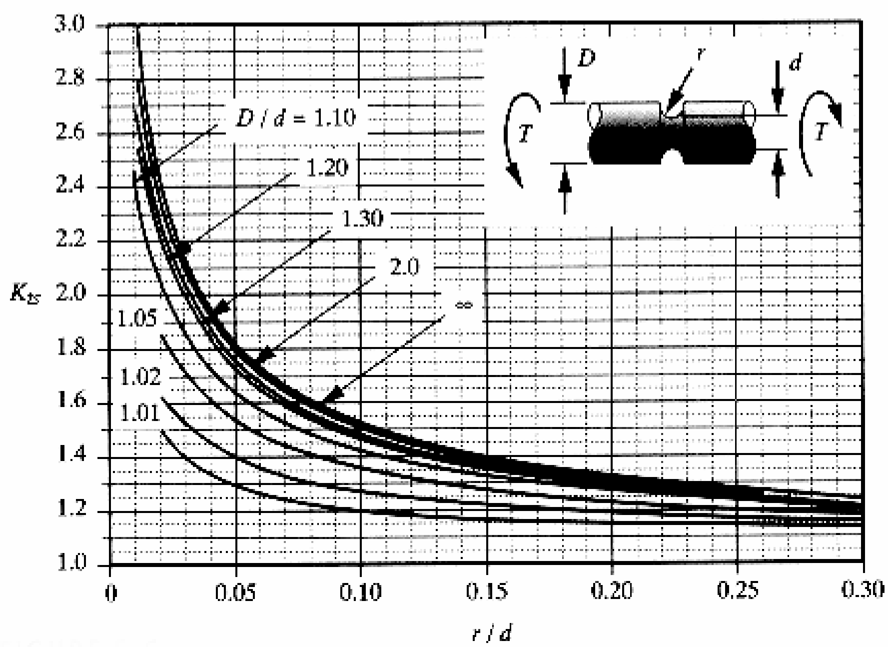


$$K_t \cong A\left(\frac{r}{d}\right)^b$$

where :

D/d	A	b
∞	0.948 01	−0.333 02
2.00	0.936 19	−0.330 66
1.50	0.938 94	−0.323 80
1.30	0.942 99	−0.315 04
1.20	0.946 81	−0.305 82
1.15	0.953 11	−0.297 39
1.12	0.955 73	−0.288 86
1.10	0.954 54	−0.282 68
1.07	0.967 74	−0.264 52
1.05	0.987 55	−0.241 34
1.03	0.990 33	−0.215 17
1.02	0.977 53	−0.197 93
1.01	0.993 93	−0.152 38

Figure A6.2-7: Geometric Stress Concentration Factor K_t for a Grooved Shaft in Bending (Norton 1998).

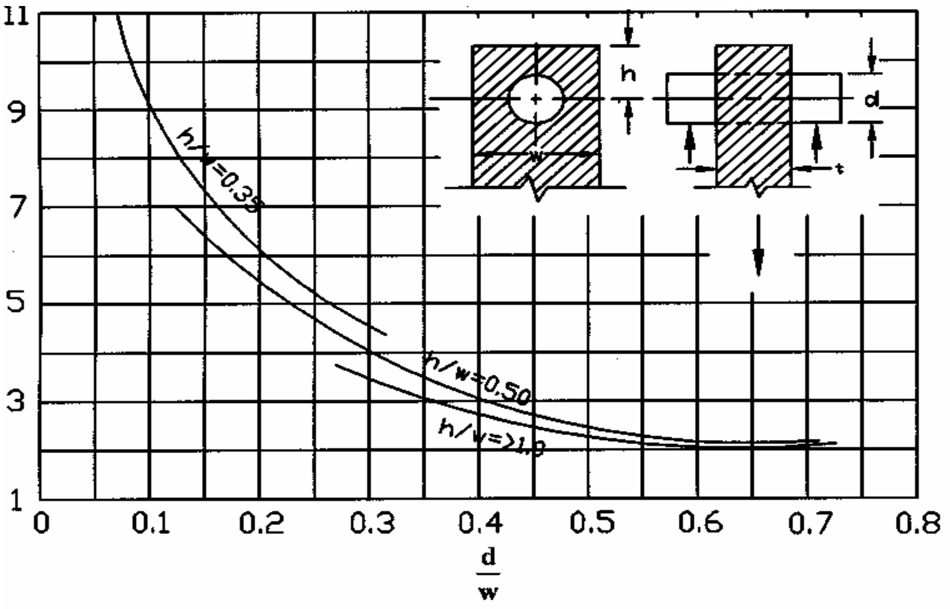


$$K_{ts} \cong A\left(\frac{r}{d}\right)^b$$

where :

D/d	A	b
∞	0.881 26	−0.252 04
2.00	0.890 35	−0.240 75
1.30	0.894 60	−0.232 67
1.20	0.901 82	−0.223 34
1.10	0.923 11	−0.197 40
1.05	0.938 53	−0.169 41
1.02	0.968 77	−0.126 05
1.01	0.972 45	−0.101 62

Figure A6.2-8: Geometric Stress Concentration Factor K_{ts} for a Grooved Shaft in Torsion (Norton 1998).



$$K_t \cong A\left(\frac{d}{w}\right)^b$$

where:

$\frac{h}{w}$	A	b
$\geq$ 1.00	1.15	–0.9723
0.50	1.537	–0.7923
0.35	2.071	–0.6447

Figure A6.2-9: Geometric Stress Concentration Factor K_t for a Plate Loaded in Tension by a Pin Through a Hole (Shigley, 1989).

SECTION 7 - TABLE OF CONTENTS - HYDRAULIC DESIGN

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SPECIFICATIONS

7.1 GENERAL REQUIREMENTS

7.1.1 Design Objectives

Hydraulic systems for movable bridges and their ancillary devices shall be designed for specified limit states to achieve the objectives of constructibility, safety and serviceability.

7.1.2 Contract Documents

Contract drawings for hydraulic systems shall clearly identify the performance requirements of the hydraulic system, regardless of the level of design detail presented. Performance requirements included in the design shall include working pressures, or loads, and flows, at critical locations in the system, for all design load conditions, including overhauling loads.

7.1.3 Symbols

Hydraulic symbols used in contract drawings, working drawings, and shop drawings shall be in accordance with ANSI Y32.10.

7.1.4 Related Standards

In addition to the required standards specified within this section, there are a number of related standards for hydraulic systems and components which provide useful reference material for movable bridge hydraulic systems and which may be used for reference or specified as binding to the design and/or construction of movable bridge hydraulic systems, including:

ISO 4413 - *Hydraulic Fluid Power - General rules relating to systems.*

NFPA/T2.24.1 R1-2000 - *Hydraulic Fluid Power - Systems standard for stationary industrial machinery - Supplement to ISO 4413:1998 - Hydraulic Fluid Power - General rules relating to systems.*

ISO/DIS 4406 - *Hydraulic Fluid Power - Fluids - Code for defining the level of contamination by solid particles.*

ISO 16889:1999 - *Hydraulic Fluid Power Filters - Multi-pass method for evaluating filtration performance of a filter element.*

COMMENTARY

C7.1.1

Design in this case includes specification of manufactured products or components.

C7.1.2

Various components from different manufacturers may meet the design intent of a component shown in a hydraulic schematic. Similarly, many components with identical hydraulic symbols will not function per the intent of the design. Therefore, it is important that hydraulic schematics and details be accompanied by specific performance requirements.

C7.1.4

Designers of hydraulic systems for movable bridges should be familiar with the current standards for hydraulic system design, construction, maintenance, safety, etc. There are numerous standards not listed herein which provide applicable information.

SPECIFICATIONS

COMMENTARY

ISO/FDIS 4413 - *Hydraulic Fluid Power - General rules relating to systems.*

ISO/FDIS 4414 - *Pneumatic Fluid Power - General rules relating to systems.*

NFPA/T2.25.1 R2-2003 - *Pneumatic Fluid Power - Systems standard for industrial machinery - Supplement to ISO 4414:1998 - Pneumatic Fluid Power - General rules relating to systems (second edition).*

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7.2 DEFINITIONS

Accumulator - An energy storage device for storing hydraulic fluid under pressure. The energy absorbing mechanism may be a spring, external weight, or an inert gas with a precharge pressure.

Beta Ratio - A measure of the effectiveness of filters.

Closed Loop - A hydraulic circuit in which the pump output, after passing through an actuator, returns directly to the pump inlet.

Component - An individual unit comprising one or more parts designed to be a functional part of a fluid power system, e.g., cylinder, valve, filter, excluding piping.

Counter Balance Valves - Valves specifically intended to retard actuator movement by providing back pressure on the downstream side of the actuator during overhauling load situations.

Design Working Pressure - The established criteria for maximum working pressure allowed by design.

DIN - Deutsches Institut fur Normang e.V. (German Standards Institute)

Hydraulic Regenerative Braking - A method of energy absorption whereby fluid power flow is reversed and directed toward the system hydraulic pumps. This energy is then absorbed by resistance to speed change in the prime mover.

Maximum Working Pressure - The highest pressure at which the system or part of the system is intended to operate in steady-state conditions without amplification due to impact; a physically established value - controlled and limited by physical devices such as relief valves (see Article 7.6.2).

Normal Working Pressure - The pressure at which a system or part of the system is intended to operate in steady-state conditions without amplification due to impact as established by the design setting of a relief valve. Normal working pressure differs from maximum working pressure in that it is established by setting an adjustable relief valve to a specific pressure lower than the maximum pressure setting. If a non-adjustable relief valve is used, the normal working pressure and maximum working pressure will be the same for the part of the system whose pressure is controlled by that valve.

Open Loop - A hydraulic circuit design in which fluid is drawn from a reservoir, routed through an actuator, and returned, at low pressure to the reservoir.

Overall Efficiency - The total efficiency of a component of a system; the product of the mechanical efficiency and the volumetric efficiency.

Rated Pressure - The highest pressure at which the component is intended to operate for a number of repetitions sufficient to ensure adequate service life.

7.3 NOTATION

A = additional thickness allowance to compensate for material removed during threading; to provide for mechanical strength of the conductor; and to provide for corrosion and/or erosion (in.) (C7.6.9.2)
D = diameter of rod (in.) (7.5.12.3)
D_o = outside diameter of pipe or tube (in.) (C7.6.9.2)
E = modulus of elasticity (psi) (7.5.12.3)
Eff_{ov} = overall efficiency (DIM, decimal equivalent) (7.5.2.2)
I_{rod} = moment of inertia of the rod ($in.^4$) (7.5.12.3)
I_{shell} = moment of inertia of the cylinder body ($in.^4$) (7.5.12.3)

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K = effective length factor (DIM) (7.5.12.3)
L_{eff} = $L_{total} \cdot K$ (in.) (7.5.12.3)
L_{rod} = the length of the rod (in.) (7.5.12.3)
L_{shell} = the length of the cylinder body (in.) (7.5.12.3)
L_{total} = length of cylinder between points of attachment on support (in.) (7.5.12.3)
P = pressure (psi); internal design pressure (maximum working pressure) (psi) (7.5.2.2) (C7.6.9.2)
P_E = Euler buckling load (lb.) (7.5.12.3)
P_{kw} = power (hp) (7.5.2.2)
Q = flow (gpm) (7.5.2.2)
SE = allowable stress in material due to internal pressure and joint efficiency at the design temperature (psi) (C7.6.9.2)
t_m = minimum wall thickness of the pipe or tube (in.) (C7.6.9.2)
y = coefficient for pipe geometry that varies with temperature (DIM) (C7.6.9.2)

7.4 DESIGN LOADING CRITERIA

7.4.1 Power Requirements for Hydraulic System Design

Except as noted within this section, power requirements for hydraulic system design shall be taken as specified for machinery design in Section 5. Prime movers for span drive hydraulic systems shall be sized to satisfy the provisions of Article 7.5.2.

C7.4.1

The prime mover of a hydraulic system is the pump motor, the electric motor which imparts energy to the pump.

7.4.2 Machinery Design Criteria and Limit States

Machinery actuated by hydraulic power shall be designed at the service limit state using resistances specified in Section 6 for the following loads:

- Hydraulic motor torques at the Normal Working Pressure neglecting motor efficiency.
- Hydraulic cylinder forces at the Normal Working Pressure of the circuit which actuates the cylinder, neglecting cylinder efficiency.

Machinery actuated by hydraulic power shall also be designed for the overload limit state using resistances specified in Section 6 for the following loads:

- Hydraulic motor torques at the maximum working pressure neglecting motor efficiency.
- Hydraulic cylinder forces at the maximum working pressure of the circuit which actuates the cylinder, neglecting cylinder efficiency.

For seismic design at the extreme event limit state, the provisions of Article 6.4.1 shall apply.

SPECIFICATIONS

7.4.3 Hydraulic Cylinder Connections

Clevis pins, bearings, trunnions, and other machinery for support of hydraulic cylinders shall be designed as specified for machinery. Structural connections for cylinder support, including clevis brackets, weldments, and high strength bolts shall be designed as structural elements in accordance with the provisions of Section 2, for the most severe of the following:

- 150 percent maximum working pressure of the circuit which actuates the cylinder or the cylinder relief valve, whichever is greater, span in any position, taken as DAD in Table 2.4.2.3-1.
- Calculated operating loads plus 100 percent impact, span in any position, taken as M_o in Table 2.4.2.3-1.
- Normal Working Pressure of the circuit which actuates the cylinder, without impact, span restrained physically by load shoes, bumper blocks, end lifts, wedges, etc., taken as DAD in Table 2.4.2.3-1.

In each load case above, the appropriate load factors and load combinations shall be applied as specified in Sections 2 and 5.

The rated pressure of manufactured components shall be determined by the manufacturer in accordance with the requirements stated herein or in accordance with industry standards. Where no standards exist, or the standards are deemed unacceptable to the Engineer, the contact documents shall require the manufacturer to present physical test data verifying stated component ratings.

7.4.4 Hydraulic System Limit States

7.4.4.1 GENERAL REQUIREMENTS

Hydraulic systems and components shall be designed for the service and/or overload limit states as stated herein. Unless specified otherwise, the resistance factors, φ, shall be taken as 1.0 for hydraulic system and component design.

COMMENTARY

C7.4.3

This section differentiates between elements to be designed as machinery and those to be designed as structural elements. The intent is for structural elements to be designed using the maximum loads anticipated, i.e., those associated with maximum working pressures and dynamic impact, as the normal design loads and for machinery to be designed using typical operating loads as the normal design load and maximum loads as an overload case. The basis for this is in the difference in the approach to calculating the nominal resistance of structural systems and machinery. Structural systems, which will be designed in accordance with the AASHTO LRFD Bridge Design Specifications, have well defined resistances related to strength and fatigue resistance. Machinery on the other hand is typically designed using conservatively estimated resistances based on specified allowable stresses to account for cyclic loading fatigue and stress concentrations. While these specifications introduce an endurance basis for fatigue design of machinery in Section 6, strength design of machinery has not yet been fully codified.

No additional impact, as in DAM, shall be applied beyond the 100 percent specified herein.

SPECIFICATIONS

7.4.4.2 COMPONENT RESISTANCE

Hydraulic system components shall be designed for the service limit state such that the component is rated for continuous duty at the normal working pressure. The component's rated fatigue pressure, per Article 7.6.4, shall be greater than or equal to the normal working pressure.

Hydraulic system components shall also be designed for the overload limit state such that the component is rated for intermittent duty at the maximum working pressure. The component's rated burst pressure, per Article 7.6.4, shall be greater than or equal to the maximum working pressure.

7.4.4.3 PIPE, TUBING AND FITTINGS

Pipe, tubing, and fittings shall be designed for the overload limit state such that the allowable working stresses established in ASME B31.1 are not exceeded at the maximum working pressure.

7.4.4.4 HYDRAULIC CYLINDERS

Hydraulic cylinder assemblies shall be designed for the service limit state such that the cylinder is rated for continuous duty at the normal working pressure.

Hydraulic cylinder shells shall be designed for the overload limit state such that the allowable working stresses established in ASME B31.1 are not exceeded at the maximum working pressure. Hydraulic cylinder rods and tubes shall be designed for the overload limit state such that a minimum factor of safety of n_s=3.0 is provided at the load resulting from the maximum working pressure. See Article 7.5.12.3.

7.5 COMPONENTS

7.5.1 Hydraulic Fluid

Hydraulic fluids shall be specified based on the hydraulic equipment being employed and the environmental conditions at the facility. As a minimum, hydraulic fluids shall have a viscosity index of 200 with an ISO Viscosity Grade of 32 to 46 at a temperature of 100°F.

Hydraulic fluids should be specified as nonharmful to the environment and readily biodegradable, conforming to the following standards:

ISO 9439 - *International Water Quality Evaluation Standard*

ASTM D-5864 - *Standard Test Method for Determining the Aerobic/Aquatic Biodegradation of Lubricants*

COMMENTARY

C7.5.1

For hydraulic systems in cold climates, consideration must be given to the temperature limitations of biodegradable hydraulic fluids. In these cases alternative fluids, including nonbiodegradable fluids, may be used provided that appropriate components and seals are used and that containment of potential fluid leaks and spills is addressed to the satisfaction of the owner and agencies having environmental jurisdiction. Containment by way of full enclosure of the hydraulic system and installation of sumps to contain and remove spilled fluid is one method of addressing this issue.

SPECIFICATIONS

Designs and details shall consider containment of leaks and spills.

7.5.2 Electric Motors

7.5.2.1 GENERAL

All specified motors shall comply to NEMA dimensional standards, NEMA Design B motor guidelines, and ANSI/NEMA MG 1 specifications for Motors and Generators.

Electrical motors equal to or greater than 1 hp used for driving hydraulic pumping equipment shall be specified as chemical duty rated, 1,800 nominal RPM, three phase, 480 VAC, TEFC, squirrel cage induction motors. Motors rated below 1 hp may be specified as single phase, capacitor start at the discretion of the Engineer.

In computing the power requirements of electric motors, the pressure, P, shall be taken as the pressure at the pump outlet. This pressure shall be computed taking into account all pressure drops in the circuit at the corresponding flow rate at the pump. Pressure drops for components shall be taken from manufacturer's data for typical components. If the pressure drop of a particular component is assumed, the anticipated pressure drop should be specified in the contract documents.

7.5.2.2 OPEN LOOP SYSTEMS

Electric motors for operation of pumps in open loop applications shall be sized to provide 120 percent of the power required of the pump under constant velocity conditions. Calculation of motor power requirements, P_{hp}, shall include the maximum fluid power, i.e. flow times the pressure at the pump, plus pump and motor efficiencies and be taken as:

$$P_{hp} = \frac{QP}{Eff_{ov}\,1{,}715} \tag{7.5.2.2-1}$$

where:

P_{hp} = power (hp)

Q = flow in (gal./min.)

P = pressure in (psi)

Eff_{ov} = overall efficiency (DIM, decimal equivalent of percentage)

COMMENTARY

C7.5.2.1

In sizing electric motors for driving hydraulic pumps, consideration should be given to the following, in addition to the power required to operate the span under constant velocity conditions:

- Charge pump or control pressure pump power requirements.
- Minimum motor size to start and drive large pumps.

C7.5.2.2

The 20 percent extra power requirement is to account for future changes to components within the circuit and field adjustment of counterbalance valves.

SPECIFICATIONS

7.5.2.3 CLOSED LOOP SYSTEMS

Electric motors for operation of pumps in closed loop hydrostatic drives shall be sized to provide:

- 110 percent of the power required of the pump under constant velocity driving conditions, and
- the power required of the pump under dynamic braking conditions.

Provisions shall be made in design of the electric motor power supply for regenerative conditions.

7.5.2.4 AUXILIARY DEVICES

Electric motors for operation of auxiliary device hydraulic systems in which the pumps are started under no-load conditions shall be sized for the constant velocity loads and satisfy the requirements of Articles 7.5.2.2 and 7.5.2.3.

Electric motors for operation of auxiliary device hydraulic systems in which the pumps are started under loaded conditions shall be sized for the constant loads resulting from the maximum working pressure and satisfy the requirements of Section 5.

7.5.3 Internal Combustion Engines

The use of internal combustion engines as prime movers for hydraulic systems may be specified for emergency operation only, and only in the absence of redundancy in the electrical power systems.

Internal combustion engines for use as prime movers in hydraulic drive systems shall be specified to have a clutch.

7.5.4 Couplings

Flexible couplings for electric motor and hydraulic pump shafts shall be specified to assist in the assembly process of the hydraulic power unit. Couplings shall be specified based on available power and shall contain flexible inserts designed to accommodate shaft misalignment without inducing additional axial thrust into shafts.

COMMENTARY

C7.5.2.3

The extra 10 percent is to account for future changes to components within the circuit.

C7.5.3

The purpose of the clutch is to provide a smooth and gradual increase in torque and to prevent stalling of the engine during engagement of the drive.

SPECIFICATIONS

7.5.5 Pumps

7.5.5.1 MAIN DRIVE SYSTEM PUMPS

Hydraulic pumps intended for fluid power generation shall be specified as positive displacement type with pressure and flow compensation. The contract documents shall specify that pumps adhere to the minimum efficiencies specified in Article 7.6.3. When sizing pumps, consideration shall be made for maximum flow requirements and pump efficiencies at actual anticipated system pressures.

Pumps shall be specified as being manufactured for use in the type of circuit intended by the design, i.e., for "open loop" or "closed loop" circuits.

Pump mounting shall be specified as front face with SAE (or ISO) standard dimensions. Orientation for drain ports shall be specified as not to allow pump chamber to drain during periods of inactivity.

Provision shall be made in the design and detailing of hydraulic systems for span operation such that the pumps are started under no-load conditions.

7.5.5.2 AUXILIARY PUMPS

Auxiliary pumps may be specified as positive displacement pumps with or without pressure and flow compensation. Where fixed displacement pumps are specified, excessive heat generated by the system shall be avoided.

Pump mounting shall be specified as front face with SAE (or ISO) standard dimensions.

COMMENTARY

C7.5.5.2

Auxiliary pumps include charge pumps, circulation pumps, cooling pumps, reservoir fill pumps, and pumps for operation of auxiliary devices such as locks, wedges, end lifts, etc.

Fixed displacement charge pumps for flow control systems shall be properly sized to deliver required flow and pressure without excessive losses over relief valves. Pump volume shall be considered when sizing all valves in such a subcircuit.

7.5.6 Control Valves

7.5.6.1 GENERAL

All valves necessary for proper span control shall be specified to have a rated pressure equal to or greater than the maximum working pressure.

Requirements for internal or external draining of valve chambers shall be considered. The contract documents shall specify these types of general details and shall assign the responsibility for draining of valves to the Contractor.

7.5.6.2 DIRECTIONAL AND SPEED CONTROL VALVES

Contract documents for hydraulic power units shall provide a detailed methodology for controlling speed and direction of movable spans. Directional and speed control valving shall be specified based on system requirements

C7.5.6.2

The designer must address full speed and creep speed when sizing valves for speed control. In lieu of more specific information, creep speed may be taken as 10 percent of full speed. Valves must be sized large enough

SPECIFICATIONS

with control and power valves sized according to anticipated flow and pressure requirements. All valve pressure ratings shall be specified to meet the maximum working pressure.

Whenever possible, valves shall be specified as manifold mounted with a subplate-type interfacing conforming to ANSI B93.7 - *Dimensions for Mounting Surfaces of Sub-Plate Type Hydraulic Fluid Power Valves*.

Unless an alternative mode of manual operation is provided, or the owner waives the requirements for manual operation, any valve required for span movement shall be specified with a manual override system. Design and valve configuration shall be such that one operator could manually shift the necessary valves in order to direct the system to move the associated span without the use of the electrical control system.

7.5.6.3 SOLENOID OPERATED VALVES

Solenoid valves shall be specified as 120 VAC or variable current loop signal when proportional, variable shifting is required. Electrical requirements of solenoid valves shall be in accordance with NFPA/T2.24.1 R1-2000 - *Electrically Operated Valves*.

7.5.6.4 PRESSURE CONTROL VALVES

Pressure control valves shall be specified based on system requirements for power delivery. The rated pressure for pressure relief valves shall be equal to or greater than the maximum pressure rating of any fluid power generator or fluid power storage device located immediately downstream of this valve.

7.5.7 Accumulators

When accumulators are specified for fluid power storage and discharge, the design shall comply with requirements of the NFPA/T2.24.1 R1-2000 - *Gas-loaded Accumulators*.

Accumulator circuits shall be required to automatically discharge stored fluid power anytime the hydraulic power unit is not in operation. Discharge orifices shall be sized to relieve pressure within the amount of time determined by the Engineer. Appropriate placards shall be specified to aid in proper operation of safety shutoff valve, verification of fluid power discharge, and type of unit precharge.

7.5.8 Fluid Reservoirs

Fluid reservoirs for hydraulic power units shall be specified as heavy-duty construction, designed for the environmental conditions of the facility.

COMMENTARY

to avoid excessive pressure losses at full bridge speed. However, valves must be capable of speed regulation at creep speed.

Manual operation of span movement control valves is required to allow for emergency and maintenance operation in the event of control system failure without taking the primary circuit offline.

C7.5.6.3

In the NFPA/T2.24.1 R1-2000, see Sub-Section 7.4 - *Electrically Operated Valves* (Page 5).

C7.5.7

In the NFPA/T2.24.1 R1-2000, see Sub-Section 6.3 - *Gas-Loaded Accumulators* *(Page 4)*.

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Minimum volume of any reservoirs shall be specified as 2.5 times the flow per minute of the system. Additional reservoir volume may be necessary for adequate heat dissipation. Calculations for reservoir heat dissipation through convection shall consider only the area of the vertical sides of the reservoir. Material for reservoir construction shall be stainless steel conforming to ASTM A 276, Type 316, standards.

The designer shall specify that hydraulic power unit reservoirs be equipped with removable clean out covers, suitable baffles, immersion heaters, sight and temperature gauges, electrical temperature and level indication, fluid sampling ports, drain ports, and breather ports.

7.5.9 Hydraulic Power Unit Accessories

7.5.9.1 HEAT EXCHANGER

Heat generation in hydraulic circuits shall be estimated through analysis of the energy losses in the circuit. Energy losses shall be calculated for components, flow resistances, and throttles.

If design calculations indicate the possibility of hydraulic power unit oil temperatures exceeding 140°F during any frequency of operation, a heat exchanging system, either offline or inline, which will cycle automatically and remove sufficient heat to maintain an acceptable operating temperature as specified by the Engineer shall be required in the contract documents. Heat exchanger equipment specifications shall conform to all other specifications for hydraulic power units in these applicable sections.

7.5.9.2 EMERGENCY PUMPS

Where redundancy of hydraulic pumps is not provided, the contract documents shall specify that a backup hydraulic pump system for emergency operation of hydraulic equipment be provided. Where practical, backup pumps should be hand operated, not requiring normal or emergency electrical power.

7.5.10 Filters

All hydraulic power units shall be specified with filtration for the pressure flow from pumps, including charge pumps, and for the system return flow. Filters and filter housings shall be sized to minimize losses based on maximum system flow requirements.

Closed loop system configurations shall be specified with filtration at pressure output of charge pumps, and bidirectional filter units in the system loop.

COMMENTARY

C7.5.9.1

In designing heat exchangers, consideration should be made for the fact that the most severe conditions may occur during start up, testing, or maintenance and not during normal operation. Offline heat exchangers can also provide for additional filtration of hydraulic oil.

Energy losses in flow control valves, such as counterbalance valves, may be significant. Minimum pressure drops across these components, based upon load control requirements, must be considered.

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COMMENTARY

Suction filters or suction strainers shall not be permitted on any hydraulic power unit.

Filtration design shall be specified as to allow for ease of element replacement with a minimum of fluid loss during servicing.

Filter ratings shall be specified as "absolute" with a Beta ratio $\geq$ 100, i.e., 99 percent efficient, for any micron rating. A minimum filter rating of 10 micron, absolute, shall be specified for any hydraulic system. For systems containing high precision servo-type valves or when indicated by a component manufacturer, higher absolute filtration ratings may be specified in the contract documents.

Filtration specifications shall conform to NFPA/T2.24.1 R1-2000, Filtration and Fluid Conditioning, and Article 7.6.7 of this specification to determine fluid cleanliness levels.

All filters shall contain electrical indication of clogging or, when equipped with a bypass check valve, when bypass occurs.

7.5.11 Hydraulic Motors

7.5.11.1 HYDRAULIC MOTORS FOR SPAN OPERATION

Hydraulic motors for span operation shall be sized to meet the power requirements of Section 5 and the working pressures of Article 7.6.2. Hydraulic motors shall be sized and rated to provide the maximum starting torque at a pressure equal to or less than the rated pressure of the motor. Hydraulic motors for span operation shall be of the fixed displacement Low-Speed-High-Torque (LSHT) or High-Speed-Low-Torque (HSLT) configuration.

Low-Speed-High-Torque motors for span operation shall be of the radial piston design.

High-Speed-Low-Torque motors for span operation shall be of the axial piston, fixed displacement, design.

All hydraulic motor drive systems for span operation shall include brakes meeting the requirements of Article 6.7.13.

Specifications for span drive hydraulic motors shall detail calculated requirements for maximum motor output torque, RPM, flow, and pressure. Consideration shall be given to volumetric and mechanical motor efficiencies at calculated operating parameters. Motors shall satisfy the minimum efficiencies as specified in Article 7.6.3. Motor drive pressures shall be less than or equal to the Design Working Pressure established per Article 7.6.2. Maximum pressure developed from any motor in an overhauling span load condition shall not be greater than the Design Working Pressure. This pressure shall be regulated directly at the motor ports.

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The contract documents shall specify that hydraulic motors be capable of smooth operation at full bridge speed and at creep speed, without evidence of cogging. The contract documents shall include acceptance criteria and tolerances for smooth operation. For radial piston motors, the minimum number of power pistons shall be specified in the contract documents.

Couplings for motor power delivery shall be required to accommodate the maximum possible motor torque which could be developed based on motor port relief valve settings or braking torque requirements, whichever is greater.

The contract documents shall specify torque arms and any other necessary components for motor installation. Motor orientation at time of installation shall establish requirements for piped case drain ports which shall be positioned to prevent case discharge during system inactivity.

7.5.11.2 HYDRAULIC MOTORS FOR AUXILIARY DEVICES

Hydraulic motors for auxiliary devices should satisfy the provisions of Article 7.5.11.1. At the discretion of the Owner, this specification may be modified to incorporate specific requirements of a particular application.

7.5.12 Hydraulic Cylinders

7.5.12.1 CYLINDERS FOR SPAN OPERATION

Hydraulic cylinders for span operation shall be sized to satisfy the power requirements of Article 5.4 and the working pressures of Article 7.6.2. Cylinders shall also be sized to satisfy the holding requirements of Article 5.5. Hydraulic cylinders shall be sized and rated to provide the greater of the maximum starting torque and the force required for holding the span, at a pressure equal to or less than the rated pressure of the cylinder. Cylinder rods shall be sized for maximum loads and buckling resistance requirements.

The contract documents shall specify that hydraulic cylinders for span operation be mill duty with bolted heads and rod seals serviceable without cylinder removal. Cylinder heads shall contain an appropriate rod scraper to limit intrusion of foreign materials into the hydraulic oil and a collar reservoir system to contain small rod seal leaks.

COMMENTARY

C7.5.12.1

There are several details which can be used to introduce freedom of rotation at the cylinder connections. The most common is to provide spherical bearings at the rod and blind end of the cylinder, either as part of the cylinder or part of the connecting fixed bracket. Freedom of rotation is required not only in the plane of primary rotation, but in all directions to allow for construction tolerances and flexibility of the movable span and its supports.

Eliminating bending forces in hydraulic cylinders is required to improve resistance to buckling and to maximize the service life of seals and bearings.

As rod lengths increase, rod boots may become less practical. If necessary, for protection of rod finish, a guard may be specified to cover the rod and protect it from falling debris.

In the NFPA/T2.24.1 R1-2000, see Sub-Section 6.2, Cylinders (Page 4). The dimensional requirements of NFPA/T2.24.1 R1-2000 are intended for tie rod cylinders and may not be sufficient for mill-type cylinders. The ASME Boiler and Pressure Vessel Code

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Cylinder design shall conform to NFPA/T2.24.1 R1-2000, Cylinders, except that cylinder dimensions and mounting shall be as required to meet the strength and serviceability criteria specified herein. Strength of hydraulic cylinders for span operation shall be in accordance with the ASME Boiler and Pressure Vessel Code. Pressure test ports shall be specified at cylinder ports. Test ports may be provided in the cylinder manifold for improved access. Air bleed ports shall be provided at practical locations for bleeding trapped air during installation and servicing.

The contract documents shall require that shop testing of cylinders specified in Article 7.8.4.2.2 be performed by the cylinder manufacturer.

Cylinders for span operation shall be mounted such that bending is not introduced into the cylinder due to the restraint of the supports and/or connections. If design geometry dictates that cylinder mounting will require positioning other than vertical, consideration shall be given to the added effects of cylinder dead loads in the specification of cylinder rod bearings and analysis of cylinder buckling. As cylinder mounting approaches the horizontal position, provisions for a rod protection guard for normally extended cylinders should be considered.

The contract documents shall require that cylinder tubes have a permanently attached stainless steel information placard providing cylinder model number, manufacturer, pressure rating, bore and rod diameter, stroke length, cushion information, and details of any nonstandard features.

COMMENTARY

specifies requirements for thickness of shells and tubes under internal pressure.

7.5.12.2 CYLINDERS FOR AUXILIARY DEVICES

Cylinders for auxiliary devices such as span locks, wedges, etc., shall adhere to the requirements of Article 7.5.12.1, except that cylinders with bores 5 in. in diameter and smaller may be tie rod cylinders meeting the requirements of ISO/DIS 6020 in lieu of mill type cylinders. Such cylinders shall be rated for 3,000 psi minimum.

If the design requires the cylinder rod to remain in the extended position, the cylinder shall be specified with a flexible boot for rod protection.

C7.5.12.2

Cylinder type and ratings are given to establish quality and durability in addition to strength. Cylinders rated at 2,300 psi (16 MPa) may be used for auxiliary devices in controlled environments when mounted such that side loading is prevented.

7.5.12.3 CYLINDER BUCKLING

Hydraulic cylinders shall be designed with a minimum factor of safety of n_s=3.0 against buckling at the maximum design loads. Maximum design loads for buckling shall be taken as the load in the cylinder at maximum working pressure specified in Article 7.6.2. In lieu of a more detailed stability analysis, the ultimate buckling strength of the cylinder, P_E, shall be taken as:

C7.5.12.3

Hydraulic cylinders are composed of two main structural sections, the shell, also called the tube or body, and the rod. Each of these has a significantly different stiffness. This creates a complex structure with varying section properties which does not have a direct correlation to the model basis for Euler's formula for buckling resistance. One acceptable, and conservative method, of calculating the buckling resistance of a hydraulic cylinder

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$$P_E = \frac{\pi^3 E D^4}{64\left(L_{mod}\right)^2} \qquad (7.5.12.3\text{-}1)$$

in which:

$$L_{mod} = X_l K L_{total} \qquad (7.5.12.3\text{-}2)$$

$$X_l = \frac{1}{\frac{L_{total}}{L_{shell}} \times \sqrt{\frac{I_{shell}}{I_{rod}}}} + \frac{1}{\left(\frac{L_{total}}{L_{rod}}\right)} \qquad (7.5.12.3\text{-}3)$$

and

$$L_{Total} = L_{ROD} + L_{shell} \qquad (7.5.12.3\text{-}4)$$

where:

P_E = Euler buckling load (lb.)

D = diameter of rod (in.)

E = modulus of elasticity of rod (psi)

L_{total} = length of cylinder between points of attachment or support (in.)

L_{shell} = length of the cylinder body (in.)

L_{rod} = length of the rod (in.)

I_{shell} = moment of inertia of the cylinder body (in.^{4})

I_{rod} = moment of inertia of the rod (in.^{4})

X_l = buckling length reduction factor (DIM)

K = effective length factor for support conditions (DIM)

Effective length factor, "K", may be taken as 1.0 for cylinders supported by spherical bearings at the rod end and blind end, i.e, pinned-pinned support conditions. Other conditions should be evaluated using accepted engineering practice for determination of K.

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is to apply Euler's buckling formula, assuming the entire cylinder for its effective length has the section properties of the rod.

The approach presented herein provides a means of using Euler's formula in a less conservative manner. In this case, a buckling length reduction factor is calculated which when multiplied by the effective length of the cylinder produces a simplified yet equivalent structural model of the actual cylinder. This model has constant section properties of the rod, but a shorter effective length. The result is a simplified model which allows for calculation of buckling resistance using the Euler formula.

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7.5.12.4 CYLINDER CUSHIONS

Cylinders for span operation shall be equipped with cushions at both ends. Cushions shall be adequate to slow the span from full speed to creep speed in the event of uncontrolled motion at the anticipated end of travel, e.g., seating. Cushions may be either fixed or adjustable. Adjustable cushions shall be equipped with tamper proof covers or locking devices.

COMMENTARY

C7.5.12.4

Typical cylinder cushions consist of a fixed or adjustable orifice through which the fluid existing the cylinder must pass once the cushion is engaged. Adjustable cushions offer the advantage of flexibility during testing and operation to achieve the desired amount of cushioning back pressure. The disadvantage of adjustable cushions is that they may be set improperly or inadvertently altered such that they do not perform as required. Where fixed orifices are provided as cushion elements, the orifice should be drilled in a removable plug, thus allowing for modification if cushion performance is not satisfactory.

Other cushion types, such as multiple orifice or stepped cushions, are available through some cylinder manufacturers. These systems may be specified at the discretion of the Engineer.

7.5.12.5 CYLINDER CIRCUITS

Cylinder circuits shall include a cylinder manifold mounted directly to each cylinder used for span operation. Manifolds shall contain pilot operated check valves or similar means to hold fluid in the cylinder when the cylinder is not intended to be in motion. Manifolds shall also contain cylinder relief valves for limiting the pressure in both ends, i.e., rod and blind, of the cylinder. The manifold shall be hard piped to the cylinder ports.

Circuits with hydraulic cylinders shall have anticavitation check valves and plumbing to allow makeup fluid to be drawn from a reservoir into the suction end of the cylinder in the event loads exceed the relief valve settings.

Cylinder manifolds for span drive cylinders shall be equipped with manual release needle valves to allow for maintenance or emergency operation of the cylinder. Manual release valves shall allow for variable flow adjustment which facilitates controlled release of fluid from the cylinder under pressure.

7.5.13 Hydraulic Brakes

Hydraulic brake system shall conform to the requirements of Article 6.7.13. The contract documents shall specify that hydraulic brake units be a multidisk design with spring loading for fail-safe set brake in the unenergized condition. Brake release shall be accomplished by the application of a specified hydraulic pressure at brake port. Brake housing shall be equipped with limit switches to provide positive indication of the fully released and fully set positions of the brake.

C7.5.13

Control of hydraulic brake application is critical to the service life of the machinery. This is particularly true for LSHT motor applications where the brakes have very large torque capabilities. It is recommended that fixed orifices be used to assure brake application times. Brake application times and torques should be shop tested prior to implementation.

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Hydraulic circuits for brake control shall include provisions to adjust and set the brake application time such that hydraulic brakes do not set instantaneously under normal operation or power interruption.

Hydraulic brakes shall be specified with a manual means of release other than with fluid pressure, such as jacking screws.

7.5.14 Pressure Indicators

At a minimum, permanently mounted hydraulic pressure gauges shall be specified in the contract documents for measuring fluid pressures at the main system pumps and manifold pressures at the field piping connections. Additional quick disconnect type ports with "no mess" check valves and protective caps, secured by chains, shall be specified at pertinent locations within the hydraulic system. Connections shall also be specified at the cylinder manifolds to measure cylinder rod and blind end pressures.

Gauges shall be located to avoid contact during maintenance of the hydraulic unit. Mounting shall be rigid and capable of accommodating vibrations during operation. Housing shall be stainless steel with the internal area being glycerin filled for component protection and dampening. Gauge working design pressure shall be specified for 150 percent of the maximum working pressure at the port location.

COMMENTARY

C7.5.14

Additional pressure indicators may be placed at locations at the main system manifold to measure charge pressures, control line pressures, and connections to field piping.

7.6 GENERAL DESIGN PROVISIONS

7.6.1 Safety

Design of hydraulic systems shall consider methods to limit the modes of failure and increase system safety with prime importance placed on the safety of personnel. The contract documents shall require that all elements be designed for safe operation at their rated pressure and specified use. Components critical to safety, such as relief valves, load control valves, high pressure hoses and fittings, accumulators, and hydraulic cylinders, shall be given additional consideration with regards to safety.

C7.6.1

Failure may occur in components, piping, fittings, actuators, controls, etc.

7.6.2 Working Pressures

Design Working Pressures shall be established by the designer and/or the owner, giving consideration to site specific conditions, safety, ratings of available components, and design limit states. The established Design Working Pressure is to be used in initial calculation of prime mover size and circuit design. Once the basic circuit and components are selected, design shall utilize

C7.6.2

To assist in verification, it is recommended that pressure gauges or gauge test ports are located at convenient positions within the circuit.

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the Normal Working Pressure and/or Maximum Working Pressure at the appropriate limit state. Unless a detailed analysis of all components is performed, and except as provided in Article 7.6.5.3, Maximum Design Working Pressures shall not be taken as greater than the following:

- Hydraulic cylinder drives for span operation ....3,000 psi
- Hydraulic motor drives for span operation ...4,500 psi
- Hydraulic drives for auxiliary devices...........3,000 psi

Design Working Pressure shall not exceed 6,000 psi without prior approval of the owner.

The Maximum Working Pressure in a system or subsystem shall be controlled physically through use of pressure control valves. The Maximum Working Pressure shall be taken as the pressure setting of a nonadjustable relief valve or the maximum pressure setting of an adjustable relief valve. Normal Working Pressure in a system or subsystem shall be established either as the maximum working pressure or as a value less than the maximum working pressure through use of an adjustable relief valve.

Provision for verification of maximum pressure control shall be provided in the contract documents. Maximum working pressure control settings shall be verified in the field or shop to assure that the design working pressures are not exceeded.

Contract documents should require that adequate warnings be placed to assure that the devices are not modified or replaced such that the maximum working pressure intended is exceeded.

Pressure intensifiers shall not be used in hydraulic systems for movable bridge span operation.

7.6.3 System and Component Efficiency

Hydraulic system design shall account for:

- overall efficiency, E_0,
- volumetric efficiency, E_v, and
- mechanical efficiency, E_m.

Determination of the required displacement of hydraulic pump(s) shall account for the volumetric efficiency of the pumps. Determination of the required power of pump motor(s) shall account for the total efficiency of the pump and the hydraulic circuit.

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Unless a more detailed analysis is performed and the corresponding minimum efficiencies are specified in the contract documents, the following maximum component efficiencies shall be used in computing the required power of the prime mover:

- Axial Piston or Gear Pumps ....80 percent
- Hydraulic Cylinders.....................................95 percent
- LSHT Radial Piston Motors ....85 percent
- Axial Piston Motors.....................................85 percent

7.6.4 Component Ratings

Hydraulic components shall be specified with certified manufacturer's pressure ratings, i.e. rated pressure, which meet or exceed the maximum working pressure. Testing of custom components shall be specified in the contract documents to establish certification criteria and acceptance. Component ratings may require testing and documentation in accordance with several industry standards based on the type of component and intended use. As a minimum, components shall be certified for Rated Fatigue Pressure, RFP, and Rated Burst Pressure, RBP, in accordance with NFPA/T2.6.1 - *Fluid power components - Method for verifying the fatigue and establishing the burst pressure ratings of the pressure containing envelope of a metal fluid power component*.

7.6.5 Controls

7.6.5.1 CLOSED LOOP SYSTEMS

C7.6.5.1

Closed loop hydraulic systems may be used for primary span drive and control, and for auxiliary device operation. Closed loop drives for span operation shall be hydrostatic drives whereby the speed and direction of the actuator is controlled by varying the output of the pump(s).

Load control in closed loop drives shall be provided by regenerative braking of the electric motor(s) driving the pump(s). Counterbalance valves shall not be used in closed loop drives.

The maximum pressure within the loop shall be controlled with pressure relief valves equipped with a fixed maximum setting or adjustable settings with maximum pressure protection.

Closed loop hydrostatic drives that are equipped with power limited pumps shall be equipped with a fail-safe means of detecting overspeed conditions and braking the span to a speed at which the drive can provide adequate

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control. The above requirement may be waived if the electric motors are sized to provide dynamic braking for maximum starting torque loads at normal speed.

Hydrostatic transmissions shall be equipped with charge pumps and a flushing valve to allow fluid to be continuously circulated out of the low pressure side of the loop, filtered, cooled and returned to the loop. Flushing valves shall continuously remove a minimum of 20 percent of the flow within the loop.

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Flushing of the loop provides a means for off line fluid conditioning. The flow removed from the loop must be adequate to remove heat at a sufficient rate to maintain acceptable fluid temperatures under the most severe operating conditions.

7.6.5.2 OPEN LOOP SYSTEMS

Open loop hydraulic systems may be used for primary span drive and control, and for auxiliary device operation.

Motion control in open loop systems should be specified and designed as either variable volume pump control or proportional valve technology. Servo valve control should be specified and designed where positioning tolerances are required which exceed the capabilities of proportional valve technology.

Counterbalance valves, or components designed to perform a similar function, should be specified to provide load control in open loop hydraulic systems. Counterbalance valves shall be designed to provide for steady load control over the entire range of anticipated loads and speeds. Counterbalance valves which also act as relief valves should be set at 1.3 times the maximum load induced pressure or higher.

C7.6.5.2

Counterbalance valves must be carefully sized and selected to modulate, inversely proportional to the control pressure upstream of the load, such that they provide fail safe control pressure downstream of the load. Under maximum overhauling loads and deceleration, the counterbalance valve(s) must provide sufficient back pressure to maintain the desired speed or decelerate the span in a smooth controlled fashion. Under normal driving loads, the counterbalance valves must provide adequate pressure to control oscillations without adversely affecting the desired speed. During seating, when flow is reduced to a fraction of the maximum flow, counterbalance valves must also be effective. In general, a minimum pressure drop must be maintained in the counterbalance valves to avoid span oscillations. This may require sets of different size counterbalance valves or sequencing of valves. Attempts to improve system efficiency by oversizing the counterbalance valves will result in instability under normal loads, particularly at creep speed.

Counterbalance valves are available in a range of pilot ratios. In general higher ratios will result in higher system efficiency and greater instability. Unless the designer is familiar with the performance of a particular valve, specification of ratios greater than 3.0:1.0 is not recommended.

7.6.5.3 SEATING PRESSURE CONTROL

Where designs incorporate a reduced pressure setting for seating of the span, the reduced pressure shall be achieved through either varying a proportional relief valve or engaging a sequence valve. In determining the maximum buckling loads on cylinders which extend to

C7.6.5.3

Cylinders which extend to lower the span are subjected to compressive, i.e., buckling loads, when the span contacts the load shoes. The buckling load in the cylinder under this condition is the controlled seating pressure times the blind area of the cylinder. A 50 percent

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lower the span, 1.5 times the specified seating pressure may be used as the maximum working pressure in lieu of the maximum setting of the relief valve.

7.6.6 Shock and Surge Suppression

Hydraulic systems for span operation, and their controls, shall be designed and detailed to minimize shock loads resulting from pressure surges during operation or holding of the span. The maximum pressure in actuators shall be controlled with relief valves. Control systems for hydraulic systems shall be designed such that smooth controlled deceleration occurs under normal conditions, and in the event of inadvertent changes in speed and direction.

Deceleration valves and/or accumulators shall be used in systems employing hydraulic cylinders for span actuation to provide for smooth deceleration in the event of power failure during operation or emergency stopping of the span. Accumulators used for this purpose shall be sized so as not to interfere with normal operation or to delay emergency stops beyond the specified deceleration time.

7.6.7 Filtration and Fluid Conditioning

Hydraulic systems shall be specified with filtration components as specified in Article 7.5.10. Standards for measuring levels of filtration shall be specified based on ISO 4406 - *Hydraulic Fluid Power-Fluids-Methods for Coding Level of Contamination by Solid Particles*.

The contract documents shall require that newly installed hydraulic power units exhibit proper levels of fluid cleanliness before acceptance. Minimum levels of cleanliness shall be as specified below:

- Servo Valve Applications.......................ISO 15/13/11
- Span Drive Applications ....ISO 16/14/12
- Aux. Drive Applications..........................ISO 17/15/13

7.6.8 Temperature Control

Hydraulic systems shall be designed to operate under the effects of the ambient conditions at the facility and for the duty cycle anticipated. Unless special conditions justify otherwise, hydraulic systems for span operation shall be equipped with immersion and/or unit heaters and heat exchangers. In determining the controlling duty cycle for heat dissipation, the designer should consider that installation and testing operations may produce the most severe heat generating conditions.

COMMENTARY

increase in this load is applied to account for pressure spikes which may occur upon impact of the span with the load shoes.

C7.6.7

The values indicated are minimum values and the designer shall consider specific components and manufacturer's recommendations when specifying minimum required ISO levels.

The Contractor may be required to supply higher levels of filtration during initial system charging and startup in order to obtain specified levels of cleanliness. This may require temporary or off-line filters of a smaller micron value than specified for final design.

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Hydraulic systems shall be designed and detailed so that reservoir hydraulic fluid temperature does not fall below 50°F or exceed 120°F, unless the specified hydraulic fluid and components are rated for service beyond this range of temperature. Reservoirs shall be sized and designed to dissipate heat and shall be located to have adequate free air circulation. Heat exchangers shall provide cooling capacity for the difference between radiant cooling and heat generation in the system such that fluid temperatures in the reservoir, or closed loop of a hydrostatic drive, are maintained within the limits specified above.

Where hydraulic actuators and fluid conductors are exposed to temperatures below 32°F, consideration shall be made for design of heat tracing or thermal blankets to maintain acceptable minimum fluid temperatures.

COMMENTARY

7.6.9 Fluid Conductors

7.6.9.1 MAXIMUM FLOW RATES

Fluid conductors shall be sized to maintain the following maximum flow velocities:

- Suction Lines..4 fps
- Pressure Lines ....20 fps
- Return Lines...13 fps

Suction lines shall be sized such that the pressure at the pump inlet port is not less than the minimum specified by the pump manufacturer to assure that cavitation does not occur under the design operating conditions.

C7.6.9.1

Fluid conductors include pipe, tube, ports, hose and associated fittings and hardware. It is preferable that pump inlets be located below the tank level to insure positive pressure at the inlet.

7.6.9.2 PIPE AND PIPE FITTINGS

Pressure rating of pipe for hydraulic systems shall be greater than or equal to the maximum working pressure in the piping. Minimum safety factors shall be as specified in ASME B31.1 code for pressure piping.

Pressure rating and safety factors for pipe fittings shall be consistent with ratings for associated pipe.

C7.6.9.2

The ASME B31.1 code for pressure piping establishes allowable working stresses for common pipe and tube materials used as fluid conductors. The following basic formula is included in this code:

$$P = \frac{2SE(t_m - A)}{D_o - 2y(t_m - A)} \qquad \text{(C7.6.9.2-1)}$$

where:

P = internal design pressure (maximum working pressure) (psi)

t_m = minimum wall thickness of the pipe or tube (in.)

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7.6.9.3 TUBING AND TUBE FITTINGS

Pressure rating of tubing for hydraulic systems shall be greater than or equal to the maximum working pressure in the piping. Minimum safety factors shall be as specified in ASME B31.1 code for pressure piping.

Pressure rating and safety factors for tube fittings shall be consistent with ratings for associated tubing. Dimensions shall conform with SAE J514 JUN96 standards for 37 degree flare connections and SAE J1453 specifications for O-ring connections.

7.6.9.4 HOSE ASSEMBLIES

Flexible hydraulic hose assemblies shall be specified for the interconnection of moving or vibrating components to reduce stresses caused by vibration. Hose lengths shall not exceed minimum requirements for the application.

Pressure rating and safety factors for hose end connections shall be consistent with ratings for associated hose. Assembly pressure testing shall be in accordance with SAE J343 standards for testing SAE 100R series hose.

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SE = allowable stress in material due to internal pressure and joint efficiency at the design temperature (psi)

D_o = outside diameter of pipe or tube (in.)

A = additional thickness allowance to compensate for material removed during threading; to provide for mechanical strength of the conductor; and to provide for corrosion and/or erosion (in.)

y = coefficient for pipe geometry that varies with temperature (DIM)

For typical carbon steel and stainless steel pipe and tube used for bridge hydraulic systems, additional thickness allowance, “A”, may be neglected and coefficient “y” may be conservatively taken as zero. Values for SE are provided in the tables in the Appendix of ASME B31.1. These values typically provide for an allowable working stress of 0.25 times the tensile strength of the material for materials under internal pressure and at temperatures below 200°F. In other words, this design approach provides for a minimum factor of safety of 4 against bursting. Designers should consult ASME B31.1 for additional information on application of this equation.

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7.6.9.5 QUICK DISCONNECTS

Quick disconnect-type connectors shall only be permitted where specified in the contract documents. Specified uses for these connectors shall be limited to temporary hydraulic connections or where emergency fluid power may be provided by connecting an auxiliary power unit.

7.7 DETAILING OF HYDRAULIC SYSTEMS

7.7.1 Power Units

As a minimum, power units for bridge operation shall be specified with maximum permissible envelope dimensions. Details of fabricated assembly showing pumps, motors, valve manifolds, and reservoirs shall be provided as necessary to aid in locating equipment within specified areas.

7.7.2 Plumbing

Plumbing between hydraulic power units and actuators shall be detailed to clearly indicate location, routing, and method of attachment of pipe and tube for the transmission of hydraulic fluid. These details shall provide clear identification of special fittings such as quick disconnects. Additional details shall be provided as necessary for protective line covers, guards, and similar appurtenances.

7.7.3 Serviceability

Hydraulic power units shall be designed to provide serviceability at a common area of the unit. Service related items to be located in this area shall include pressure gauges, additional gauge ports, pressure filter(s), and valve manifold with adjustable valves. Pumps containing flow or pressure limiting valves shall be oriented and located in open areas for clear access. Other items which cannot be conveniently located in a common area, e.g. return filters, breathers, shall have unhindered accessability for service and maintenance.

7.7.4 Identification and Accessibility

Hydraulic power units shall be fabricated and installed in order to provide maintenance access to all areas of the unit containing components requiring adjustment, replacement, monitoring, or similar servicing.

Each component of any hydraulic system shall contain permanently attached labels identifying the component,

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C7.7.1

The Contractor must submit shop assembly drawings detailing exact component locations for the entire hydraulic power unit including valve manifolds. Detailing will show all dimensions for the unit reservoir, support frame, motors, pumps, manifolds, hydraulic control panel, and any other items which may influence the final location of the power unit and any other equipment located in the adjacent area.

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manufacturer and part number. This tag shall also display a call-out number which can be referenced to the hydraulic schematic located in maintenance manuals. These labels shall be specified as stainless steel with permanent markings appropriate for a corrosive environment.

7.8 FABRICATION AND CONSTRUCTION

7.8.1 General

As a minimum, the contract documents shall contain the provisions for fabrication and construction specified herein.

7.8.2 Flushing and Filling

Before charging hydraulic units, reservoir covers or inspection covers shall be removed and a complete inspection of the interior of the reservoir shall be performed. This inspection shall be specified regardless of any prior inspections which may have occurred during or after fabrication and regardless of integrity of unit protection from contamination during shipping or storage.

Fluid for flushing shall be the same as specified for final use. Flushing shall be performed with the fluid flow path in a "loop" configuration including the hydraulic power unit manifolds and field piping with bridge actuators bypassed from the circuit. This technique shall be applicable for both open and closed loop designs. The designer shall specify maximum fluid flow and duration of flushing. The contract documents shall require observation of filter contamination and final degree of cleanliness as specified in Article 7.6.7. This cleanliness level shall be verified through a qualified fluid testing facility and a copy of the results shall be forwarded to the owner for use as baseline data.

All motor and pump case drains shall be filled in accordance with instructions provided by the Manufacturer, and air from hydraulic cylinders and motors be purged prior to operation.

7.8.3 Painting

Paint systems shall be specified for all noncorrosion resistant materials used in hydraulic systems.

COMMENTARY

C7.8.2

The designer should be aware that fluid cleanliness level based on fluid sampling of reservoir may require several days. There is a great likelihood the bridge will be in operation before this level is ever determined. Therefore, effort should be made to correctly specify exact flushing techniques per recommended NFPA standards and have the Contractor submit his proposed procedure for approval prior to implementing the flushing process.

The contract documents should be written to clearly indicate the Contractor's responsibility for obtaining the appropriate cleanliness level and verifying this level through resampling in the event of a deficient report from the first sample. The procedure of further cleaning the fluid, collecting, and processing additional samples should be at the expense of the Contractor.

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7.8.4 Testing

7.8.4.1 GENERAL

The contract documents shall contain detailed specifications outlining the required tests and acceptance criteria.

7.8.4.2 SHOP TESTS

Shop tests shall be specified for all custom components that are required to be factory set to a specific performance value. Manifolds shall be pressure tested to 3 times the maximum working pressure.

7.8.4.2.1 Power Units

The contract documents shall require hydraulic power unit testing requirements including pressure testing at 1.5 times the maximum working pressure, verification of flow and pressure control, and verification of power output. Temperature control, offline filtration systems, and all diagnostic control systems of the unit design shall also be tested. These tests shall be performed by the hydraulic equipment manufacturer and in conjunction with the hydraulic power unit control panel. The Contractor shall be required to submit the complete test procedure for approval prior to testing. Upon successful completion of test, the results shall be complied and a copy delivered to the owner for their records.

7.8.4.2.2 Hydraulic Cylinders

The contract documents shall require that hydraulic cylinders for span operation be pressure tested to 3 times the maximum working pressure. Cylinder pressure tests shall also be made with manifolds, manifold valves, and hard piping in place as an assembly.

Assembly testing shall be performed at 1.5 times the maximum working pressure. Specification shall further require dynamic cylinder cushion performance testing at design rod speeds.

7.8.4.3 FIELD TESTS

After installation of any hydraulic system, complete field testing of system functionally shall be required. This testing shall demonstrate the correct operation of all functions of the hydraulic system using the designed electrical and mechanical interfacing, i.e., hydraulic cylinders or motors permanently installed and operator

COMMENTARY

C7.8.4.1

Hydraulic systems, subsystems, and components require detailed testing prior to being placed in service.

C7.8.4.2

Pressure testing of components and assemblies will require pressures greater than the design specified relief valve settings. The test specification should require the contractor to provide test stands, shop power units, and/or temporary relief valves as are necessary to provide the required test pressure.

C7.8.4.3

The designer should specify an appropriate acceptance period in which all hydraulic main drive and auxiliary systems will be fully exercised. This period should have a sufficient duration to allow for all systems to demonstrate operation without failure. During this time, consideration for unit overheating should be given and any

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control desk electrically connected to system. All modes of operation in which the system is designed for operation shall also be demonstrated.

During functional testing, the contract documents shall further require the Contractor to record system pressures, flows, and operating times from the hydraulic system during all modes of operation. This data shall be submitted to the engineer for approval prior to system acceptance.

7.9 MATERIALS

7.9.1 Hydraulic Plumbing

7.9.1.1 PIPE AND PIPE FITTINGS

All hydraulic piping material shall be specified as seamless, low carbon stainless steel conforming to ASTM A 312, TP 316 or TP 304. Pipe shall be specified as pickled, cleaned and capped before shipping.

Pipe fitting materials shall be specified as similar to the pipes in which they are fitted. Acceptable welded pipe fittings shall be 37 degree flare type or SAE straight thread for conductors sizes up to and including 1.5 in. nominal. Mating 37 degree surface shall have an O-ring and O-ring boss specified to provide a leak-free connection. For nominal sizes of 1 in. to 1.5 in., 37 degree flare fittings shall be specified to contain a soft metallic washer for a leak-free connection. This washer shall be in lieu of an O-ring and O-ring boss. All connections involving piping over 1.5 in. shall be specified as butt welded or welded four-bolt flange utilizing captive O-ring pressure seal system connection. Specifications for flange dimensions shall conform to SAE J518 JUN93 standards. No pipe thread shall be permitted on any portion of the hydraulic system where continuous or intermittent pressures could exceed 200 psi. Where pipe threads are allowed by the criteria above, no pipe sealant shall be permitted.

Material for all associated hardware required for fittings shall be specified as similar to fitting material. Flange bolts shall be provided with locking washers and be of size and material to fit application.

7.9.1.2 TUBING AND TUBE FITTINGS

All hydraulic tubing material shall be specified as seamless, annealed, low carbon stainless steel conforming to ASTM A 269, TP304, ISO 10763:1994, and ANSI B31.1-2004 (2001 out now) standards. Maximum tubing shall be specified as no larger than 1.5 in. nominal. For conductor requirements greater than 1.5 in., tubing shall not be considered.

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active heat removal systems on hydraulic equipment should be demonstrated.

For staged construction, the designer should specify additional test periods between stages. This will reduce the possibility of disruption of traffic (vehicular or marine) by demolishing an existing bridge prior to fully verifying the operation of a new span.

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Tube connections shall be accomplished with 37 degree tube end flares and flare nuts. Mating 37 degree surface shall only be specified as Type 304 stainless steel and shall have an O-ring and O-ring boss to provide a leak-free connection. For nominal sizes of 1 in. to 1.5 in., 37 degree flare fittings shall contain a soft metallic washer for a leak-free connection. This washer shall be in lieu of an O-ring and O-ring boss. Other methods for tube fitting not requiring tube flaring shall only be permitted with written permission from the Engineer. Specifications for flange dimensions shall conform to SAE J518 JUN93 standards.

Specifications for all tube connections shall allow for unlimited break and remake of connections without cutting or creating loss of sealing integrity.

Material for all associated hardware required for fittings shall be specified as similar to fitting material. Flange bolts shall be provided with locking washers.

7.9.1.3 HOSE AND HOSE FITTINGS

Hose material shall be specified as hydraulic duty rated for maximum operating pressure consistent with pressures specified for components in which the hose is connected. Hoses shall be specified such that the maximum working pressure in the hose does not exceed the recommended maximum operating pressure for that hose as defined in SAE J517, Hydraulic Hose.

Hose end connections shall be specified as Type 304 stainless steel for 37 degree female JIC swivel connections or Type 316 for four-bolt, O-ring flange connections. Specifications for flange dimensions shall conform to SAE J518 JUN93 standards. The contract documents shall require hose assemblies to be shop assembled by the hose supplier. Dimensional standards shall conform to SAE 516 JUL94 standards.

Material for all associated hardware required for hose fittings shall be specified as similar to fitting material. Flange bolts shall be provided with locking washers.

7.9.1.4 QUICK DISCONNECTS

Quick disconnect type fittings shall be attached to pipe fittings through SAE standard straight threads with an O-ring boss and machined wrench flats for installation. Both male and female couplings shall be provided with an internal checking valve to prevent fluid loss when not coupled.

Coupling material shall be specified as Type 304 or Type 316 stainless steel.

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7.9.1.5 MANIFOLDS

Manifold material for hydraulic valving shall be specified as steel or aluminum alloy possessing the necessary strength for the system pressure including safety factors. Carbon steel and aluminum manifolds shall be painted for protection per the requirements of structural steel.

SECTION 8 - TABLE OF CONTENTS - ELECTRICAL DESIGN

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8.1 GENERAL DESIGN REQUIREMENTS

8.1.1 Scope, Codes and Standards

This section specifies the general design requirements for the electrical power and control systems on movable bridges. These specifications are based on the use of 60-Hz alternating current (AC) as the standard electric supply, but acknowledges that direct current (DC) supply may be utilized in some cases.

In general, the electrical system design shall comply with the codes and standards identified herein. Where governmental agencies have their own codes, local ordinances, regulations, or requirements which must also be complied with, the designer shall include them in the design and construction requirements.

Applicable Standards and Codes:

NFPA - National Fire Protection Association

NEC - National Electrical Code (NFPA 70)

NFPA 780 - Standard for the Installation of Lightning Protection Systems

NESC -National Electrical Safety Code (ANSI C2)

NEMA - applicable standards for electrical components and equipment

IEEE - applicable standards for electrical components and equipment

As a minimum, the electrical system design shall adhere to current standard practices for industry, such as the NEC and the NESC.

Corrosion resistant materials and heavy-duty construction techniques for electrical system components exceeding those found in many standard outdoor industrial facilities should be considered, as appropriate, for the given site.

The contract documents shall adequately describe, in as quantified and measurable a manner as practical, the minimum acceptable criteria to be met during the construction and installation of the bridge electrical system. Explicit acceptance test criteria shall be specified in the contract documents, consisting of all features or parameters critical to the proper performance of electrical system components, and the system as a whole. Such acceptance test criteria shall be specified for materials, components, and workmanship to the extent practical.

C8.1.1

Where only the name of standards or regulatory organization is given, specific codes or standards by the respective organization or agency will be given later in this section.

Movable bridges generally present a harsh environment in terms of corrosion and severe vibration.

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8.1.2 Safety

Design and detailing shall consider the safety of all personnel and traffic as are likely to be on, under, or near the bridge. In particular, where maintenance personnel may need to gain access to components of the electrical system, safe access and adequate lighting for periodic maintenance and inspection shall be provided by the design, as specified elsewhere herein.

8.2 DEFINITIONS

Corrosion Resistant - Corrosion resistant materials are materials which are considered inherently corrosion resistant without the addition of an external coating, such as stainless steel, bronze, brass, cast aluminum in mild environments, and some nonmetallic materials. Corrosion resistance as used herein implies resistance to rain, sunlight, salt water, and the alkalinity of concrete without experiencing noticeable degradation.

Exposed - In a location exposed to sunlight or sky light, or direct or blowing rain or mist.

Flux Vector Control - An AC inverter-based motor speed control with the ability to independently control the magnetic flux and torque-producing components of current in an AC induction motor, resulting in very high accuracy of torque, speed, and power control.

SCR Speed Control - An AC motor speed control based on silicon controlled rectifiers (SCR's), which are connected in a bridge configuration and switched on and off, chopping the AC waveform in order to vary the effective power to the motor thereby achieving speed control.

Semiexposed - In a location exposed to occasional blowing rain or mist, or a wet or damp location.

Squirrel Cage Motor - An AC induction motor utilizing solid bars as conductors in the rotor slots instead of discrete windings. The ends of the bars are electrically connected by shorting rings on both ends of the rotor. These motors display a fixed speed-torque characteristic.

True-RMS Meter - A meter designed such that its indicating system responds to the actual root-mean-square (RMS) value of the electrical current, voltage, or power. Standard metering systems generally respond to the average values, and apply a scale calibration factor to indicate the equivalent RMS value, based on a pure sine wave. Standard meters will not accurately respond to the distorted waveforms typically encountered with the various electronic motor speed controls.

Wound Rotor Motor - An AC induction motor utilizing a rotor with discrete windings placed in the rotor slots and terminated on slip rings. Brushes ride the slip rings, offering provision for completing the connection to the rotor circuit. This allows some flexibility in application since the motor's speed-torque characteristics can be altered by altering the impedance of the rotor circuit termination.

8.3 ELECTRIC SUPPLY AND POWER DISTRIBUTION

8.3.1 Commercial Electric Service

The designer shall coordinate with the owner and the commercial electric utility company early in the design phase to establish the significant utilization parameters of electric service. As a minimum, the following parameters

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shall be established and identified in the electrical service design calculations:

- service voltage,
- current capacity,
- available fault current, and
- voltage regulation taken as voltage drop per ampere at the connection to the utility

8.3.2 Circuit Breakers

8.3.2.1 LOW VOLTAGE CIRCUIT BREAKERS (600 VOLTS AND BELOW)

Circuit breakers for circuits 600 volts and below shall be the molded case-type with thermal-magnetic trip.

Low voltage circuit breakers located in exposed or semiexposed locations shall be specified with NEMA 4X weatherproof enclosures.

8.3.2.2 HIGH VOLTAGE CIRCUIT BREAKERS (ABOVE 600 VOLTS)

Circuit breakers for circuits rated above 600 volts shall be the oil-filled or vacuum break-type.

Circuit breakers for voltages above 600 volts shall be in metal-clad switchgear enclosures, located in one of the following:

- nonexposed locations, such as electrical rooms,
- weatherproof switchgear shelters, or
- larger weatherproof enclosures

In general, temperature and humidity control shall be provided.

8.3.3 Fuses

8.3.3.1 FUSES RATED 20 AMPS AND HIGHER

Fuses may be used as electric service or feeder overcurrent or fault protective devices only in the following applications:

- when incorporated in load-break rated motor-operated fused disconnect switches on systems rated above 600 volts, or

COMMENTARY

The latter two parameters will generally have to be obtained from the electric utility by the designer.

C8.3.3.1

In general, circuit breakers are preferred over fuses, except for specific applications where the required tripping characteristics or ratings cannot be obtained with circuit breakers.

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- as current-limiting components of circuit breakers where necessary to achieve the required fault current interrupting capacity, or as an integral component of a manufacturer's equipment, other than disconnect switches

Fuses shall be selected according to their ampere rating and tripping characteristics, in coordination with available fault current calculations and tripping time calculations.

Where fuses are utilized on three-phase services or feeders, phase-loss detection devices shall be provided in the contract documents.

COMMENTARY

The purpose of phase-loss detection devices is to ensure the protection of any equipment or components that could be damaged, or which may malfunction with dangerous or damaging consequences to personnel or other equipment, in the event of loss of one phase.

8.3.3.2 FUSES RATED BELOW 20 AMPS

Fuses rated below 20 amps may be used in applications where the tripping characteristics of circuit breakers are not acceptable, or where the necessary ampere rating does not match the ratings of commonly available molded case circuit breakers.

8.3.4 Disconnect Switches

Blade type disconnect switches may be used to fulfill the NEC requirements for in-sight disconnecting means.

Disconnect switches shall be nonfused, heavy-duty, load-break rated. Where used in exposed or semiexposed locations, the disconnect switch enclosures shall be rated NEMA 4X.

Molded case switches or molded case circuit breakers may be used as disconnect switches. When circuit breakers are used, the affects of their noninstantaneous tripping characteristics to the overall protective system coordination during overloads and faults shall be considered and identified in the design calculations.

C8.3.4

It is the intent of this specification that overcurrent or fault protection be provided by circuit breakers rather than fuses. Special circumstances may require exceptions to this requirement.

8.3.5 Transformers

8.3.5.1 GENERAL PURPOSE TRANSFORMERS

Oil-filled transformers may be specified where required by high voltages. Otherwise, transformers shall be the dry-type.

Dry-type transformers rated 10 kVA and higher single phase, and 15 kVA and higher three phase, shall be provided with primary voltage-compensating taps, providing both above- and below-normal full capacity compensation of 5 percent each, preferably in 2-1/2 percent increments.

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8.3.5.2 DRIVE ISOLATION TRANSFORMERS

Where electrical noise from a variable speed electronic drive may generate harmonics or other electrical noise which could interfere with proper operation of electronic components of the control system, or other electronic systems at the site, shielded isolation transformers specifically manufactured for use with AC or DC variable speed drives shall be included in the design.

Drive isolation transformers shall be distinguished from general purpose dry-type transformers by incorporating the following features:

- a grounded electrostatic shielding between primary and secondary windings, and
- strip conductor windings or other equivalent means of ruggedizing the windings against mechanical stresses during high-current transients.

COMMENTARY

C8.3.5.2

The designer should evaluate the need for drive isolation transformers by investigating the site for the presence of sensitive electronic equipment, and conferring with drive manufacturers concerning drive performance with regard to generation of electrical noise and harmonics.

8.3.6 High Voltage Switchgear (600 Volts and Above)

High voltage switchgear shall be utilized as the primary disconnecting means whenever the utility company supply voltage exceeds 600 volts. All switchgear shall conform to ANSI C37.20.3 - Metal - enclosed Interrupter Switchgear, and NEMA SG-5 - Power Switchgear Assemblies. Switchgear shall also comply with any local utility requirements. Metal enclosed circuit breakers shall not be used as the primary disconnecting means.

As a minimum, each switchgear assembly shall consist of an interrupter assembly and a power fuse assembly. Switchgear shall have a short circuit rating adequate to withstand the maximum calculated load side fault current.

The interrupter assembly may either be manually or electrically operated; all switches shall utilize a quick-make, quick-break mechanism which shall swiftly and positively open and close the switch independent of the speed of the operating mechanism. Switchgear shall be constructed such that the compartment doors cannot be opened with the switch closed, and that the switch cannot be closed with the door open. All compartment doors shall be padlockable and shall be provided with a window for visual verification of switch position.

Power fuses shall be of the replaceable solid material type, equipped with blown fuse indication, and shall be sized such that tripping coordination is provided between the switchgear and load side circuit breakers and also between the switchgear and utility company fusing.

All low voltage components of high voltage switch gear shall be located in grounded, metal-enclosed compartments separate from the high voltage to provide

C8.3.6

For high voltage applications, interrupter switches in combination with power fuses are preferred over circuit breakers due to their reduced maintenance, stable time-current characteristics, and they possess no relays which require periodic testing.

Low voltage applications are covered in Article 8.3.2.1.

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insulation and shall be arranged for complete accessibility without exposure to high voltage.

Consideration shall be given for the need for additional equipment such as mechanically interlocked load side ground switches, utility metering compartments, phasing receptacles, key interlocks, and instrumentations and indicating lights.

High voltage switchgear shall be located in nonexposed locations, such as electrical rooms, or in weatherproof equipment shelters.

8.3.7 Surge Protection

Surge arresters shall be provided for all electric services for protection against lightning and switching-induced voltage surges. The arresters required by this article shall be considered first level, or course, surge protection. Higher levels, or finer, surge protection may be provided as recommended in other articles herein depending on the requirements of more sensitive electronic equipment, if present.

The surge arresters for the electric service shall be located at the service entrance equipment, connected to the load side of the service disconnect. Where the bridge is supplied by long feeder cables run from a service disconnect to the bridge, additional surge arresters shall be installed at the electrical equipment on the bridge.

8.3.8 Transfer Switches

8.3.8.1 GENERAL

Where more than one source of electrical power is to be available, one or more transfer switches shall be provided for switching the load from one source to another. The transfer switches may be either automatic or nonautomatic, according to site specific criteria and owner preference.

All transfer switches, whether automatic or nonautomatic, shall be electrically powered, with a backup manual operating mechanism provided for emergency operation in the event of failure of the electrically operated transfer mechanism.

All transfer switches shall be load break rated, with contact transfer time in either direction not to exceed one sixth of a second.

All transfer switches shall be mechanically held, electrically operated, and mechanically interlocked so that only one of two possible positions, i.e., power sources, can be engaged at a time. The switch shall be so designed to mechanically prevent an intermediate position where neither source is connected. The switch operating mechanism shall be powered from the live source.

COMMENTARY

C8.3.8.1

The purpose for utilizing electrically operated transfer switches, even for non-automatic, is personnel safety. Generally, these non-automatic switches may be operated by remote switch or button, affording the operator protection by distance.

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Unless required by specific site conditions, all transfer switches shall provide open-transition transfer, completely disconnecting one source before connecting the other.

8.3.8.2 AUTOMATIC TRANSFER SWITCHES

Automatic transfer switches shall be listed under UL 1008 - Standard for Safety for Automatic Transfer Switches.

Automatic transfer switches shall be selected and specified with the following control features:

- adjustable close differential voltage sensing on all phases of normal source,
- adjustable voltage sensing of standby source,
- adjustable time delay transfer from normal to standby source to override momentary normal source outages,
- adjustable time delay retransfer to normal source,
- test switch to simulate normal source outage,
- adjustable time delay engine start contact for engine-generator applications, and
- adjustable unloaded running cooldown timer for engine-generator applications.

8.3.8.3 NONAUTOMATIC TRANSFER SWITCHES

Electrically operated, remote control transfer switches shall be used for nonautomatic transfer switches. Nonautomatic transfer switches shall be identical to and meet the same basic requirements of automatic transfer switches, except that the control features may be simplified consistent with manual initiated operation.

8.3.9 Engine Generator Sets

8.3.9.1 GENERAL

C8.3.9.1

Where engine generator sets are utilized, consisting of an internal combustion engine and electric generator, the engine and generator shall be direct-coupled and assembled on a common base frame.

Generators shall be sized so as to limit the momentary voltage drop during any possible load step, to the lesser of the following: 15 percent, or less than the lamp extinguishing voltage drop tolerance of any HID lamp/ballast combinations used at the site.

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Where the electric motor load for operating the bridge is significantly larger than the continuous load for lighting, heating/ventilation/air conditioning, and other miscellaneous loads, separate generator sets should be provided for bridge operation and for continuous lighting and heating loads. The engine generator set provided for lighting and heating loads shall be sized, to the extent practical, so that it will be loaded to at least 60 percent of its kW capacity during typical operation.

Liquid cooled engines shall be provided with thermostatically controlled coolant heaters.

When installed in buildings or walk-in enclosures, all exhaust manifolds and all other exhaust components located within the room or enclosure shall be insulated with premanufactured aluminized cloth covered insulating pads, custom fitted to the exhaust components, and secured with stainless steel wire tires.

Complete-cutoff, fully automatic battery chargers, powered from the commercial service, shall be utilized to maintain the starting batteries at full charge.

8.3.9.2 ENGINE INSTRUMENTS AND CONTROLS

Engine instruments and controls shall be provided in a panel or cabinet on or immediately adjacent to the engine. These instruments and controls may be combined with the generator instruments and controls and mounted on the generator. Remote instrumentation may be provided, but shall be in addition to and not in lieu of the local instruments at the engine generator set.

Engine instrumentation shall include:

- gauges for coolant temperature, DC volts, oil pressure, and vacuum for diesels,
- a running time meter,
- indicating lights and warning alarm contact for low oil pressure shutdown, high coolant temperature shutdown, and shutdown for failure to start after four cranking cycles.

Engine controls shall include start/stop switch for manual control, and manual emergency shutdown switch.

Where regenerative drives, either AC or DC, are used, the designer shall coordinate the regenerated power absorption rating of the engine generator set with the bridge drives and intended operating conditions.

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Where engine generators are not operated at 50 percent of their load capability or higher, they tend to "wet stack" and build up carbon deposits in the cylinders leading to reduced engine life.

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8.3.9.3 GENERATOR INSTRUMENTS AND CONTROLS

Generators shall be provided with voltmeters, ammeters, and meter selector switches to enable switching the meters to monitor all phase currents, and all phase-to-phase and phase-to-neutral voltages.

8.3.9.4 SUPPLEMENTAL GENERATOR LOADING

Where the engine generator will be loaded to less than 50 percent of its capacity for the majority of its duty cycle, consideration shall be given to providing supplemental loading, such as a resistance load bank, sized to load the generator to approximately 60 percent of capacity.

When resistance load banks are used, they shall be switched off during bridge operation or at other times of significant loading.

8.3.9.5 VENTILATION

The room or enclosure in which the engine generator is located shall be provided with adequate ventilation to meet engine combustion requirements and to maintain ambient temperatures within the range of the engine generator manufacturer's recommended operating temperatures.

Where exhaust louvers are specified, they should either be gravity or motor controlled so as to close against back drafts when the engine is not running. Intake louvers shall be sized at least 50 percent greater area than exhaust louvers, and shall be motor operated.

Where motorized louver operators are specified, they shall automatically close louvers with spring pressure when de-energized, and open louvers against spring pressure when energized. Louver motors shall be powered directly from the generator(s) through appropriate dedicated transformers and fuses so as to run at all times the engine is running.

8.3.9.6 REMOTE RADIATORS

Where it is not practical or desirable to provide adequate ventilation for engine generator cooling with a conventional attached radiator, consideration shall be given to utilizing a detached, remotely located radiator for engine cooling. Remote radiators and their electrically operated fans shall be sized for the engine cooling requirements. The radiator/fan design and orientation shall take into account any derating effects of prevailing winds and possible ice or snow build-up.

The electric fan motor shall be powered directly from the generator, through a small transformer and fuse or circuit breaker, circuited to run any time the engine is

COMMENTARY

C8.3.9.4

Resistance load banks are recommended as a means of loading the engine for cleaner running, avoidance of carbon deposit buildup in the engine, and wet-stacking in the exhaust system.

C8.3.9.5

Generally, this would be accomplished via intake and exhaust louvers with motorized dampers or shutters.

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running, independent of any transfer switches or generator circuit breakers.

Piping for engine coolant to remote radiators shall be sized for adequate flow, in accordance with the engine generator manufacturer's recommendations. All piping located within the room or enclosure shall be insulated with preformed foam insulation and covered with a rigid protective jacket.

8.4 ELECTRICAL CONTROL SYSTEMS

8.4.1 Operating Sequence and Interlocking Requirements

8.4.1.1 BASCULE BRIDGES, SINGLE LEAF AND DOUBLE PARALLEL LEAF

The operating sequence described below shall be considered for bascule bridges, modified as necessary for the specific bridge and site conditions.

Raise Span

Step 1: Bridge operator performs the proper traffic control operations to safely signal and stop roadway and pedestrian traffic. This includes use of traffic signals, followed by warning gates, which are then followed by resistance gates or barriers.

Step 2: Unlock all locking devices.

Step 3: Release the machinery brakes.

Step 4: Release the motor brakes. This step should generally be performed simultaneously with energizing the drive motors, in which case a permissive brake released check should be performed within a few seconds to confirm brake release, or to shutdown drive motors if confirmation fails. Where electronic drives equipped with a motor torque confirmation circuit are used, drive motor torque confirmation should be performed and so circuited so as to set brakes if the drive does not confirm torque within a few seconds of releasing the motor brakes.

Step 5: Accelerate drive motors to running speed.

Step 6: At nearly open, decelerate motors to slow speed.

Step 7: De-energize motors at fully open position. Set motor and machinery brakes.

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Lower Span

Step 1: Release machinery brakes.

Step 2: Release motor brakes and energize motors.

Step 3: Accelerate motors to full speed.

Step 4: At the nearly closed position, decelerate the motors to slow speed.

Step 5: Approximately 2 to 3 seconds after span has fully seated, set all brakes while motors are stalled in reduced torque mode. After brakes have set, de-energize the drive motors.

Step 6: Lock all locking devices.

Step 7: With permissive interlock from locking devices, operator raises, i.e., opens, traffic barriers, followed by warning gates. Alternatively, all gates and barriers may be raised simultaneously.

Step 8: Operator manually returns traffic signals to green, or other condition normally utilized at span fully closed.

8.4.1.2 BASCULE BRIDGES, DOUBLE OPPOSING LEAF

Except as modified herein, the recommended sequence for opening double leaf bascule bridges is the same as Article 8.4.1.1 for single leaf bascule bridges.

The following seating sequence is recommended for double leaf bridges, in lieu of Steps 4, 5 and 6 in Article 8.4.1.1:

- At the nearly closed position, the far leaf shall decelerate and stop.
- The near leaf shall decelerate to slow speed and continue to fully closed. After a short delay at fully closed, the brakes shall be set, followed by de-energizing of the drive motors.
- The far leaf shall then proceed to fully closed and de-energize through the same sequence as the near leaf.
- With permissive contact received indicating both spans are fully seated and de-energized, span locking devices shall be driven to their locked positions.

COMMENTARY

C8.4.1.2

This provision is intended to address certain situations in which simultaneous seating of both leaves could, as a result of misalignment of centering or locking devices, cause a jamming or interference condition.

The seating sequence from nearly closed to fully closed may have to be modified for bridges utilizing tongue-and-groove or similar passive shear locks that require precise coordination of the position of both leaves as the lock components come together.

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8.4.1.3 VERTICAL LIFT BRIDGES

This recommended sequence for both span drive and tower drive vertical lift bridges is the same as Article 8.4.1.1 for single leaf bascule bridges.

Tower drive vertical lift bridges require skew monitoring and control. The designer shall determine allowable skew and provide appropriate skew limiting provisions in the control circuit.

8.4.1.4 SWING SPANS

Design of swing span control systems and the associated sequence of operation must be coordinated with the mechanical systems unique to swing span bridges.

The following sequence shall be considered for typical swing span machinery arrangements.

Open Span

Step 1: Bridge operator performs the proper traffic control operations to safely signal and stop roadway and pedestrian traffic. This includes use of traffic signals, followed by warning gates, which are then followed by resistance gates or barriers.

Step 2: Pull center supports.

Step 3: Pull end supports.

Step 4: Pull span centering devices.

Step 5: Release the machinery brakes.

Step 6: Release the motor brakes. See Step 4, Article 8.4.1.1.

Step 7: Accelerate drive motors to running speed.

Step 8: At nearly open, decelerate motors to slow speed.

Step 9: De-energize motors at fully open position. Set motor and machinery brakes.

Close Span

Step 1: Release machinery brakes.

Step 2: Release motor brakes and energize motors.

Step 3: Accelerate motors to full speed.

COMMENTARY

C8.4.1.4

Some swing spans utilize center support wedge at the pivot pier and span centering devices at the rest piers, while they may not all be required on other bridges. Typically, swing spans will require end wedges at the rest piers. The correct sequence of operation of these devices is critical to swing span operation.

The recommended operating sequence assumes all referenced wedges and centering devices will be required.

The designer must consider the operating range of the swing span. Some bridges may be designed to open 90° in either direction such that all bridge positioning devices, and the control circuit, must be capable of 180° operation.

Center and end supports generally consist of wedges, jacks, rollers, or similar mechanical equipment driven in place to stabilize the swing span for live load, and retracted when turning the span.

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Step 4: At the nearly closed position, decelerate the motors to slow speed.

Step 5: When span is fully closed, de-energize motors, set motor brakes followed by machinery brakes.

Step 6: Drive span centering devices.

Step 7: Drive end supports.

Step 8: Drive center supports.

Step 9: Permissive contact from center supports shall allow operator to manually raise, i.e., open, traffic barriers, followed by warning gates.

Step 10: Operator manually returns traffic signals to green, or other condition normally utilized at span fully closed.

8.4.2 Control Logic

8.4.2.1 GENERAL

Control logic shall be designed to ensure the correct sequence of operation, interlocking for protection of machinery and personnel, control of dynamics such as acceleration, deceleration, speed and skew of the span.

Power for control circuits shall be derived from either a single dedicated transformer, or from a single dedicated branch circuit.

The supply voltage to the control system shall not exceed 120 volts between any two conductors, or any conductor and ground, and shall be derived from a solidly grounded system, as defined by the NEC.

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C8.4.2.1

The extent of bridge operating control logic can range from minimal for manual operation to quite complex for fully automatic operation. Methods of applying control logic to the bridge operating system may be through the use of relays, PLC's, i.e., programmable logic controller's, and PC's, i.e., personal computers.

As a minimum, limit switch contacts from various components of machinery and position monitoring should be electrically interlocked to prohibit improper sequence of operation.

8.4.2.2 RELAY CONTROL LOGIC

Control relays shall be heavy-duty industrial relays, UL listed, with replaceable contacts. Relay contacts shall be rated for the peak load which can be applied at any given time, but not less than 10 amps.

C8.4.2.2

Relay control logic is highly reliable and possibly perferred for bridges where the level of maintenance may be limited. The designer is, however, somewhat restricted to the basic bridge operation and interlocking. Features such as continuous control of speed and skew, maintenance alarms, messaging and reporting are generally not possible with relay control logic.

8.4.2.3 PROGRAMMABLE LOGIC CONTROLLER (PLC)

PLC designs shall include redundant processors. Input power shall be provided through an uninterruptible power supply with power conditioning.

Designers shall consider specifying PLC I/O modules that provide visual indication of the I/O status.

C8.4.2.3

PLC control logic generally affords high reliability and high levels of speed and skew control. Additionally, all settings of limits and timers can be accessed and changed easily.

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PLC ladder logic diagrams shall be specified to include cross-referencing of all contacts and coils. Sufficient comments shall be included on the ladder logic diagrams to describe all logic operations.

The designer may consider providing trouble alarms and maintenance messaging for all major span operating devices. Such alarms and messaging may be stored in a memory module for downloading to printer or computer on demand.

The designer shall review the operating environment and specify heating, cooling, and humidity control as may be required to maintain the PLC system hardware and accessories well within the manufacturers specified operating parameters.

The contract documents shall require compatibility demonstration of all components of the PLC control system prior to delivery to the project site.

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PLC control allows remote devices to be interfaced utilizing fiber optic cable, eliminating interference from adjacent cables and lightning.

However, the technical demands of servicing PLC systems should be considered along with an evaluation of the owner's means of acquiring the technically trained personnel needed to perform PLC maintenance and troubleshooting.

8.4.2.4 INDUSTRIAL COMPUTER CONTROL

The designer shall consider the operating environment and specify heating, cooling, and humidity control as may be required to maintain the computer system hardware and accessories well within the manufacturers specified operating parameters.

C8.4.2.4

Computer-based control systems have the advantage of higher communications speed than PLC I/O systems.

8.4.2.5 EMERGENCY STOP

All bridge control systems shall be designed to include an emergency stop mode stopping span movement without the normal deceleration.

The emergency stop pushbutton shall be prominent on the operating console. Designer shall consider use of a large, red, mushroom head pushbutton.

The emergency stop shall be wired to directly remove power from the operating coils of all bridge machinery motor starters, including, but not limited to drive and auxiliary motors, brakes, locks and wedges, regardless of the state of any I/O devices.

C8.4.2.5

It is common practice on bridge control consoles to use a large mushroom-head, maintained contact, push-off, pull-on, pushbutton for the emergency stop.

This pushbutton is a multicontact switch that not only interrupts the control logic sequence, but also directly interrupts power feeding the motor contactor coils.

8.4.2.6 NORMAL STOP

Control systems shall incorporate a normal stop function intended to provide a smooth method of stopping the span without the undesirably high machinery stresses typically experienced with an emergency stop. Normal stop shall be actuated by pressing a normal stop pushbutton on the control console. Normal stop shall initiate controlled deceleration of the drive motor(s) via regenerative and/or dynamic braking in electronic drives, and after a preset time delay to allow deceleration to approximately 10-15 percent of running speed, shall de-energize the drive(s) and set the brakes. In stepped

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resistance-type motor control, the brakes shall be set sequentially with sufficient time delay between succeeding brakes so as to limit maximum braking torque to approximately 150 percent of full load motor torque.

Normal stops shall generally be adjusted to decelerate and stop the span with the same torques and times as are experienced for accelerating the span to running speed.

8.4.3 Bypass Switches

Bypass switches shall be provided to permit overriding the safety interlock circuits.

Bypass switches shall be located on, or in close proximity of the bridge control console. Provisions shall be included for installing keyed locks or seals on each bypass switch cover.

Bypassable devices may include traffic signals, gates, barriers, locks and fully seated limit switches.

The designer may consider additional bypass switches, located at the span locks, and connected in series with the span lock bypass switches on the console to ensure visual check of the locks when a span lock bypass is invoked.

8.4.4 Limit Switches

8.4.4.1 GENERAL

Provisions shall be made to monitor critical bridge machinery and span positioning with a variety of limit switch configurations.

The designer shall specify heavy-duty industrial limit switches with all components water-proof, oil-tight and corrosion resistant. Switches to be utilized outdoors shall be specified to be rated for harsh environments, such as marine duty.

The designer shall specify switch contact material and contact arrangements such that contact closure is positive at the low levels of control voltage and current often encountered on PLC or PC control.

8.4.4.2 LEVER ARM LIMIT SWITCHES

Lever arm limit switches shall be used to monitor motor and machinery brakes, span fully open and extreme lift, and may be used for locks, wedges and span fully seated condition. Operating components of the lever arm switches such as the arm, wheels or rollers shall be specified to be corrosion resistant.

The designer shall consider providing adjustable mounting provisions such that the trip point may be adjusted without the field drilling of additional mounting holes.

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C8.4.3

It should be recognized that all bypass switches introduce additional risk into bridge operation by allowing personnel to defeat the electrical means for enforcing correct operating sequence. However, from a practical consideration, some by-passing provision is typically necessary to allow continuation of the bridge operation in the event of a limit switch failure.

C8.4.4.1

Bridge positions of fully open, fully seated, extreme lift and the position of brakes, locks, lifts and wedges are most reliably monitored with directly actuated limit switches.

Problems have been experienced with standard industrial-type limit switches in cold temperatures. Switches should be specified which are rated for temperatures substantially lower than are likely to be encountered in the actual application.

Wiping contact action may be necessary for control voltages of 24 volts and below.

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8.4.4.3 ROTARY CAM LIMIT SWITCHES

Rotary cam limit switches may be used to provide indication of nearly closed, nearly open and fully open span positions. Rotary cam switches may also be configured to indicate excess skew on vertical lift bridges.

Rotary cam switches shall be coupled to machinery through low backlash flexible couplings. Cam adjustment shall be possible without mechanical disassembly of the switch.

For span position monitoring, rotation of the cams should not exceed 330 degrees.

8.4.4.4 PLUNGER LIMIT SWITCHES

Plunger limit switches shall be utilized only in those applications not subject to overtravel.

8.4.4.5 PROXIMITY SWITCHES

Proximity switches may be used for monitoring the positions of the movable span and ancillary machinery, such as span locks, wedges, and brakes. Where proximity switches are used, the designer shall investigate and select the proximity switches which will be compatible with anticipated variations in target distances for each specific application. Variations in target distance shall be considered for temperature effects and running clearances.

Where proximity switches will be used with PLC control systems, the designer shall investigate the compatibility of the proximity switch to be specified with the particular PLC I/O module to be specified.

8.4.5 Position Indicator Systems

8.4.5.1 GENERAL

Position indicators shall be sufficiently accurate to provide indication of span position and skew condition to the bridge operator within:

- one-half degree for bascule and swing bridges
- 6 in. for vertical lift bridges
- 1 in. for skew of vertical lift bridges.

The designer shall consider supplementing digital position indicators with analog displays to provide the operator with

COMMENTARY

C8.4.4.4

Plunger limit switches are generally limited for use as span seated limit switches where overtravel is inherently limited. Even in these situations, lever limit switches may be the better choice because they are more tolerant of misadjustment without damaging the switch.

C8.4.4.5

Problems have been experienced with PLC's and proximity switches. These problems may be due to the high impedance PLC input not presenting a sufficient load to the proximity switch to maintain consistent and reliable operation.

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a readily recognizable indication of the relative span position.

All position indicators must be coupled to bridge machinery using low backlash couplings and, when required, through low backlash gear reduction.

Position indicator transmitters shall be specified to be oil, water and corrosion resistant.

8.4.5.2 SYNCHRONOUS SYSTEMS

Synchronous position receivers may be mounted on the control console and connected to dial indicators custom calibrated for the specific application.

Synchronous position transmitters may exceed one full revolution when appropriate gear reduction is provided in the indicator assembly.

8.4.5.3 POTENTIOMETER SYSTEMS

Output from the high precision potentiometer shall be processed through appropriate circuitry such that correct scale factor is provided for an analog or digital display.

8.4.5.4 RESOLVER SYSTEMS

Absolute position, brushless resolvers shall be specified.

Resolvers shall be oil, water, and corrosion resistant, preferably mill-duty or NEMA 4 rated. Connections to rotating machinery shaft shall be through flexible couplings.

Resolver cables shall be specified as twisted, shielded pairs, properly grounded, and kept away from motor power cables.

The designer shall determine whether long cable runs require the specification of an optional signal converter located near the resolver.

Resolver systems may be used for position monitoring. Resolvers shall be absolute, and the decoder or other digital interface shall be nonvolatile, i.e., interruptions to decoder or resolver power shall not affect the accuracy of the position indication once power is restored, regardless of whether or not the shaft being monitored has moved.

Cable lengths and types shall be specified in strict conformance with manufacturer recommendations.

COMMENTARY

C8.4.5.2

Synchronous position indicators have a reputation for being rugged, highly reliable and accurate. "Synchros", also called “selsyns”, are the traditional analog method of indicating movable bridge span position. A synchro differential can compare inputs from two synchro transmitters on a vertical lift bridge to provide an accurate indication of span skew.

Synchro driven position indicators have the advantage of operating at high signal level, usually 120 volts AC, and are relatively immune to problems associated with electrical noise and the distance between devices.

However, synchro systems are often inordinately expensive. Hence, the trend in position indication is toward alternatives to synchro systems.

C8.4.5.3

Precision potentiometers have the advantage of being available as multiturn devices. Therefore, some positioning applications may not require gear reducers when connected to multiturn machinery shafts.

C8.4.5.4

Resolvers have the advantage of providing precise shaft position information while being rugged and having a wider temperature range than encoders.

Cable length restrictions are a function of the type of resolver output. A line driver may be required on some bridge applications.

Long runs of resolver cabling may introduce external electrical noise, resulting in false or erratic position measurements.

SPECIFICATIONS

Resolvers shall be specified in NEMA 4, corrosion resistant, heavy duty housings with sealed bearings. When resolvers are mounted in wet or damp areas, they shall be specified with weatherproof electrical cable connections.

8.4.5.5 ABSOLUTE ENCODER SYSTEMS

Absolute encoder systems may be used for position monitoring. Absolute encoders and any decoder or other digital interface shall be nonvolatile, i.e., interruptions to decoder or encoder power shall not affect the accuracy of the position indication once power is restored, regardless of whether or not the shaft being monitored has moved.

Cable lengths and types shall be specified in strict conformance with manufacturer recommendations. Absolute encoders shall be specified in NEMA 4, corrosion resistant, heavy duty housings with sealed bearings. When absolute encoders are mounted in wet or damp areas, they shall be specified with weatherproof electrical cable connections.

8.4.6 Control Console

8.4.6.1 GENERAL

The control console should be located in the bridge operator's house. The console location shall provide the bridge operator with an unobstructed view of the marine channel and of highway traffic in both directions.

The designer shall consider the console size with respect to access into the operators room through doorways, windows and stairways. The console shall be designed for partial disassembly, if required, for installation or later removal.

8.4.6.2 CONTROL CONSOLE DEVICES

Console devices should be legible under all ambient lighting conditions. Direct sunlight may require consideration of window shading and the brightest indicator lights.

The designer shall use a systematic approach for indicator light colors. Red shall be reserved to indicate an unsafe or fault condition, e.g., motor overload; green to indicate a safe "rest" condition, e.g., brake "set", and yellow for a normal transitory operating function, e.g., "brake release" or "nearly open". All lighted indicators shall have a push-to-test feature, either individually or group test.

Lighted indicators shall be provided to show the state of all bridge operating devices and for the span positions of fully seated, nearly closed, nearly open and fully open.

COMMENTARY

C8.4.6.2

The designer should consider a logical layout of control devices for the console that relates to the bridge operating sequence and required physical movements of the operator. Panel devices for operating equipment located on the bridge to the operators right, when facing the console, should be on the right side of the console. The control devices used in the beginning of the operating sequence should be located toward either end of the console such that the operator works toward raise-lower controls located near the center.

When lever or "pistol grip" controls are used for span raise and lower, or open and close, to comply with general practice they would operate as follows:

SPECIFICATIONS

Operating switches, pushbuttons and indicator lights shall be industrial-type, UL listed, oil and water-tight. All devices shall be identified with integral or separately attached legend plates. Legends shall be engraved with white letters on a black background.

Legend plates, when not part of an installed panel device, shall be engraved metal or multilayer acrylic, with high contrast between characters and background, and shall be attached with brass or stainless steel screws.

COMMENTARY

- Vertical Lift Bridge - Clockwise rotation of the control from center "off" will raise the bridge, counterclockwise rotation to lower the bridge.
- Swing Span Bridge - Rotate from center “off” position in the same direction as the swing span as observed by the operator.
- Bascule Bridge - Rotate from center "off" position, in the same direction as the bascule leaf as observed by the operator. On a two leaf bascule bridge where the operator is facing the nearside leaf, the switch for the nearside leaf operates counterclockwise to raise the bridge, while the switch for the far side leaf will operate clockwise.

8.4.6.3 CONTROL CONSOLE CONSTRUCTION

The console shall be free standing with a frame of No. 11 U.S. Standard Gage minimum thickness steel. The console top should be nonreflecting stainless steel, No. 10 U.S. Standard Gage minimum thickness, and continuously hinged for access to panel components.

A raised, sloped instrument panel should be provided at the rear of the console top such that metering may be easily viewed.

All seams shall be continuously welded. Outside surfaces shall be seamless and smooth and all corners shall be rounded.

The front of the console shall include one or more gasketed doors with three-point latches and chromed operating handles. The console, with exception of the stainless steel top, shall be phosphate coated and finished with epoxy paint.

The interior sides and back of the console shall be furnished with mounting panels. The mounting panels shall be finished with enamel paint. The console bottom may be open, with interior flanges for attachment to the floor.

Interior lighting shall be provided inside the console, above each door, and operated by door switches.

Consoles located within unheated areas should include cabinet heaters and thermostats.

8.4.7 Electrical Signal Multiplexing

Multiplexing control signals may be considered in situations requiring limited number of conductors.

Shielded, twisted pairs should be used for signal multiplexing in electrically noisy environments. Surge protection shall be provided on long, exposed cable runs to protect the multiplexing equipment.

The designer shall determine the additional communication time for data transmission and reception

C8.4.7

Signal multiplexing is utilized to send multiple control signals over a single pair of wires. This technology is best applied when few spare conductors are available, for example, in existing submarine cables.

Hardwired control conductors are preferred over multiplexing, except for PLC and PC systems, due to the delay times often encountered in multiplexers.

SPECIFICATIONS

anticipated for the system under consideration. The designer shall review manufacturers engineering data to assure that proposed signal multiplexing does not introduce unacceptable delays to any portion of the bridge operating sequence.

8.4.8 Fiber Optics

Fiber optic communication may be utilized for data, CCTV and PLC communication on bridges.

Testing after installation shall be specified by the designer. At a minimum, testing shall include continuity tests of the entire fiber optic cable system.

The designer shall consider suitable jacket material for the particular environment, cable support methods, protection from mechanical stress and compliance with the manufacturers recommended minimum bending radius.

Low loss, fusion splices shall be specified.

8.4.9 Radio Data Links

When determining the suitability of radio data links, consideration shall be given to the distance involved, type of data to be transmitted, and presence of local radio interference.

The designer shall specify that the radio system is to operate error free in the presence of all anticipated local services such as, but not limited to, cellular telephones, CB radios, marine radios and marine ship-board radar.

Control system shop testing shall include the radio data link system. The test shall duplicate or exceed the transmission distance required at the actual final installed location.

Directional antennas may be required to ensure reliable communications.

8.5 ELECTRIC MOTORS

8.5.1 General Requirements

All motors used for bridge drives shall be specified to be crane and hoist or mill duty motors. Motors for other bridge operating equipment shall be crane and hoist, or heavy-duty industrial motors.

Motors should be specified to have either cast iron or cast aluminum frames. Where mounting feet are utilized

COMMENTARY

C8.4.8

Fiber optic communication is immune to electrical and magnetic fields, electrical noise, lightning, and radio frequency interference.

Modules are available with the capability to transmit and receive many contact closures, such as limit switches, over a single fiber.

C8.4.9

The use of radio data links may eliminate the need for expensive or difficult cable installation. Radio data links are particularly useful for supplementing existing submarine cables of limited capacity.

The designer has many options in the selection of radio data link equipment with respect to operating frequency and mode of transmission. Delay times similar to those encountered in multiplexing should be investigated and evaluated.

The designer is cautioned that movement of the span may interfere with an adjacent radio signal path such that antenna locations need to be chosen carefully and include cable allowances for trial-and-error antenna adjustment.

Increasing the number of inputs and outputs on a radio data link system may add noticeable time to the bridge operation cycle.

C8.5.1

NEMA governs the terminology, the performance, the testing, and the standard dimensions of motors. Most, if not all, motors manufactured for industrial use in the United States will be manufactured according to NEMA standards, particularly MG 1, Motors and Generators. All references in this section to NEMA standards concerning electric motors refer to NEMA MG 1.

SPECIFICATIONS

for mounting motors, cast feet shall be specified in lieu of stamped or bent metal plate motor mounts.

Where motors are to be installed in exposed, wet, or damp locations, the motors shall be specified to be weatherproof, with stainless steel shafts, when available.

Motors rated for 60 minutes or continuous duty shall be either totally enclosed, or installed in weather protected climate-controlled houses, in which case squirrel cage motors may be drip-proof. Motors rated for 30 minute duty cycle shall be totally enclosed. Motors rated for running times less than 30 minutes shall not be used.

Totally enclosed motors shall be specified with drain holes in bottom of frame and frame heaters where practical.

All DC motors and AC wound rotor motors shall be specified with totally enclosed frames. Drip-proof frames shall not be used for any DC or AC wound rotor motors in any environment.

Motors of 0.9 kW (1.2 HP) and larger shall be specified with ball or roller bearings. Where available, the motor bearings shall be specified to be regreasable and provided with grease fittings suitable to the owner, and vent plugs for purging the grease.

COMMENTARY

Because of the sometimes severe vibration encountered on bridges, the heavy-duty construction used for crane and hoist or mill duty motors is essential. Historically, cast-frame motors, and cast mounting feet seem to be more robust and better suited to the rough vibration and corrosive environment of typical movable bridges.

The dampness and abrasive grit and dust often encountered on movable bridges are detrimental to motor life and reliability, hence the requirement for totally enclosed motors. DC motors and AC wound rotor motors are especially vulnerable to increased maintenance if their brushes and commutators or slip rings are exposed to such an environment. However, these motors have proven to be very reliable in bridge drive service when furnished with totally enclosed frames.

Totally enclosed nonventilated (TENV) frames have traditionally been used with very good success on movable bridges, in lieu of totally enclosed fan-cooled (TEFC) or drip-proof. This is because the running times are short. This is very intermittent operation according to the motor industry's standards, even on a busy bridge. Hence 30 minute or 60 minute rated TENV motors are the norm on movable bridges.

8.5.2 Application-Specific Criteria

8.5.2.1 GENERAL

Electric motors for use on movable bridges are considered in two main categories, according to the following applications:

- Span Drive Motors, including skew control, or synchronizing motors
- Ancillary Device Motors, i.e., span locks, brakes, wedges, hydraulic pumps, air compressors, etc.

8.5.2.2 SPAN DRIVE MOTORS

Electric motors used as the prime mover for driving the movable span(s) shall be one of the following three types:

- AC Squirrel Cage Induction Motor
- AC Wound Rotor Induction Motor
- DC Motor

C8.5.2.2

Squirrel cage motors are quickly becoming popular as movable bridge drive motors, particularly when used with flux vector drives. Only inverter duty or flux vector duty rated motors should be used in these applications, due to the historically high failure rate of the windings insulation of standard AC squirrel cage motors with these drives.

Wound rotor motors are still popular and have proven themselves historically to be very rugged and reliable motors when used with stepped resistance control or variable voltage thyristor drives.

SPECIFICATIONS

In addition to the requirements of this and following sections, span drive motors shall also be selected and specified in accordance with the criteria above in 8.5.1.

Motor base speeds should be either 900 or 1,200 RPM for AC motors, and 850 or 1,150 RPM for DC motors.

Motors to be applied in any location other than a climate controlled house shall be specified with marine duty, anti-fungal, moisture resistant sealed or encapsulated windings and internal frame heaters sized per manufacturer's recommendations. Internal frame heaters shall not be thermostatically controlled.

COMMENTARY

DC motors in conjunction with modern digital DC drives excel in smooth control over speed ranges of 100:1. This reason, and the ease of indirectly controlling torque simply by controlling armature current, has kept the DC motors in contention as a movable bridge prime mover.

8.5.2.2.1 AC Squirrel Cage Motors

The squirrel cage motor NEMA Design Letter shall be selected to provide the appropriate speed-torque characteristics compatible with the load requirements of the hydraulic system or machinery design. Unless specifically determined otherwise, Design B characteristics shall be specified.

Only inverter duty or flux vector duty rated motors shall be utilized with inverter or flux vector drives. Standard squirrel cage motors shall not be used with these drives.

Standard squirrel cage motors may be considered for use with high ratio gearing where span travel speeds are low, i.e., 0.5–0.75 in./sec., and full voltage starting and running are employed. Standard squirrel cage motors are also permissible for use with other nonspeed-controlled applications, such as span locks, wedges, brakes, gates and barriers, hydraulic pumps, and air compressors.

C8.5.2.2.1

AC squirrel cage motors should be utilized as bridge drive motors in the following applications:

- Variable speed application with electronic inverter drive
- Constant speed applications via high gear reduction, resulting in very slow span movement, such as for emergency drive
- Hydraulic pump motors for hydraulically driven bridges

Use of squirrel cage motors with full voltage starting and running as bridge drive motors should be restricted to emergency or auxiliary drives.

NEMA Design C or D motors are often used with span locks or wedges due to the high starting torque requirements for these applications.

8.5.2.2.2 AC Wound Rotor Motors

When wound rotor motors are used in stepped rotor resistance applications, the control system shall be designed to prevent the resistance values from being stepped prematurely, relative to the speed-torque characteristics of the motor-resistor combination, thereby allowing the motor to develop excessively high torque.

The control system and machinery designs shall be coordinated to ensure that the machinery can withstand the unusually high torque peaks that are typically encountered with wound rotor motors in stepped resistance applications.

In variable voltage thyristor drive applications, the control system designer shall specify the required value of current limiting necessary to adequately limit the maximum torque developed at any given speed, based on the speed-torque characteristics for the value(s) of external rotor resistance selected. This value of necessary current limiting shall be clearly specified on the contract

C8.5.2.2.2

AC wound rotor motors should be utilized as bridge drive motors in the following applications:

- Variable torque applications utilizing stepped rotor resistance
- Variable speed applications utilizing analog or digital thyristor variable voltage drives

An AC wound rotor motor control using stepped resistance in the rotor circuit is primarily torque control, not speed control. This characteristic of the AC wound rotor motor is often misunderstood. It is often thought to be speed control because it has historically been used to control the bridge acceleration rate and starting current surges. But an AC wound rotor motor, being an induction motor, will always try to reach its synchronous base speed, regardless of resistance value.

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documents as part of the drive or motor performance specification.

Wound rotor motors shall not be used with flux vector or other inverter-type drives.

8.5.2.2.3 DC Motors

DC motors utilized as bridge drive motors shall be shunt wound and rated for the type of DC power source with which they will be applied, according to the power supply letter designations in NEMA MG 1.

DC motors shall be controlled by a closed-loop speed control system employing either analog tachometer or digital encoder feedback.

DC motors shall be capable of developing 150 percent full load rated torque at 100 percent base speed for at least 30 seconds, and 175 percent full load rated torque at 75 percent base speed for at least 10 seconds. Motors shall be suitably rated for stalling at 75 percent rated full load torque for 30 seconds.

8.5.2.3 SKEW CONTROL, OR SYNCHRONIZING MOTORS

The provisions of this article shall apply to the use of motors for providing skew control by synchronizing the hoisting motors in a tower drive vertical lift bridge.

Motors used for this application shall be identical wound rotor motors.

The full load rated torque of each of these motors shall be equal to the drive motor full load rated torque.

These motors shall be connected as synchronous transformers, with their stator circuits and their rotor circuits connected together in the configuration commonly referred to as a 'power synchrotie'. In movable bridge applications, the skew control motors shall be operated against their magnetic field direction of rotation.

The skew control motors shall be energized in a timed three step procedure immediately prior to energizing the movable bridge drive motors, as follows:

- Step 1: energize stators, single phase
- Step 2: close rotor tie contactor
- Step 3: close stator third phase supply contactor

COMMENTARY

An induction motor normally only develops counter-torque when the load overhauls it and attempts to drive it higher than its synchronous speed. Adding resistance to the rotor circuit of a bridge drive motor in an overhauling load condition decreases its counter-torque capability, thereby allowing the bridge to increase its speed, which is usually contrary to the operator's intent.

Without a speed-dependent feedback and error correction system, AC motor speed controls generally cannot accomplish true speed control.

C8.5.2.2.3

DC motors should be utilized as bridge drive motors in the following applications:

- Variable speed, controlled-rectifier power
- Variable speed, controlled-generator power

C8.5.2.3

Historically, the power synchro tie has been an excellent skew control system, providing dynamic turn-for-turn synchronizing of the bridge drive motors at normal running speeds. This results in skew control on the order of perhaps 0.2 in., or less. When skew does occur, it is generally during the braking and stopping at the end of travel.

Despite good past performance, this system is generally considered antiquated due to the high precision performance and lower cost of current digital DC drives and AC flux vector drives, and the simplicity of their application. It is rarely used in new system design. However, in certain rehabilitation situations where power synchrotie systems are currently installed, some owners have elected to retain the existing wound rotor motors, apply thyristor speed control to the drive motors, and continue utilizing the power synchrotie for skew control.

The designer unfamiliar with the power synchrotie is encouraged to research this application of wound rotor motors before applying or modifying this type of system.

The rotors are always connected between the two motors. The stators are connected together, but have a contactor in one phase between them, and a second

SPECIFICATIONS

COMMENTARY

contactor in one phase of the supply feeder, in addition to the normal three-phase motor starter.

The timing between the steps is a matter of judgement and is best adjusted in the field after observing system performance. There have been numerous articles written concerning applying wound rotor motors in synchronizing applications, particularly in paper mill and conveyor applications.

Two articles concerning bridge applications are: “Synchrotie Systems” by Campbell (1961) and “Induction Motors as Selsyn Drives” by Nowacki (1934).

8.5.2.4 ANCILLARY DEVICE MOTORS

In general, the motors used for all functions on the bridge, other than movable bridge drives, should be standard heavy-duty industrial AC squirrel cage motors. Motor selection and equipment design shall be coordinated to utilize 1,200 RPM or 1,800 RPM motors.

C-face, D-flange, or any other face or flange-mount motor shall be used with caution and consideration for the vibration likely to be encountered in the intended mounting location. Whenever possible, foot-mount motors shall be used in high vibration applications.

8.6 ELECTRIC MOTOR CONTROLS

8.6.1 Speed Control of Span Drive Motors

8.6.1.1 GENERAL

The designer shall consider the following factors when selecting a method of span drive motor speed control including:

- Constraints due to reuse of existing motors on rehabilitation projects.
- The owner’s level of technical capability to maintain the system.
- Design, installation and material costs.

All span drive speed control equipment shall be housed in NEMA 12 or NEMA 4 cabinets when located indoors, and NEMA 4X enclosures when located outdoors or otherwise exposed to the weather. Enclosures in unheated environments shall include heaters, thermostats, and, or air conditioners as required to maintain the equipment within the manufacturers recommended temperature range and to prevent condensing humidity.

C8.6.1.1

A recommended criteria for selecting any motor-drive combination for movable bridge drives is its suitability for crane and hoist application. If the crane and hoist industry is using a given motor-drive combination with good results, then it probably warrants further investigation for movable bridge drive application. If it's not suited for crane and hoist application, or is no longer popular in that industry, it may well not be suitable for movable bridge drive application.

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Indoor drive equipment shall include metering of voltage and current on the cabinet doors. For current metering on electronic drives, three identical ammeters shall be utilized and arranged side-by-side, one per phase, for phase balance observation. Meters shall be switchboard-type. All metering for electronic drives shall be true-RMS. Frequency response of the metering shall be considered when used with inverter-type drives.

All cabinets shall be clearly labeled with permanently attached, engraved legend plates.

Motor overloads, circuit breakers and contactors may be located in the span speed control cabinets.

COMMENTARY

True-RMS metering is essential on electronic drives due to the inherent wave form distortion, and resulting meter errors encountered. Measurement errors of nearly 40 percent have been experienced with standard meters and thyristor voltage controls.

8.6.1.2 STEPPED RESISTANCE CONTROL

Stepped resistance control may be considered where wound rotor drive motors are to be used.

Values for the secondary resistance shall be calculated for each step of acceleration.

C8.6.1.2

Stepped resistance control is highly reliable and can be maintained with a minimum of technical expertise.

Older motors, unless refurbished to include new windings and insulation, may not be capable of withstanding peak voltages produced by other AC drive methods.

Because stepped resistance is torque control, precise control of motor speed cannot be achieved with it. It is, therefore, not suitable for automatic sequencing control circuits.

8.6.1.3 SCR (AC THYRISTOR) SPEED CONTROL

SCR speed controls should be used exclusively with wound rotor motors.

Drives shall be specified to be four-quadrant, regenerative, with tachometer feedback.

Loss of the tachometer signal shall automatically shutdown the drive.

The capability to adjust acceleration and deceleration ramp time, slow speed, running speed and torque limiting shall be provided in the design.

C8.6.1.3

SCR-based speed control has been used successfully with NEMA Design D squirrel cage motors, but is generally more difficult to adjust, and usually results in higher motor temperatures than with wound rotor motors.

8.6.1.4 DC SPEED CONTROL

DC drives shall be specified to include tachometer or encoder feedback. Drives shall have separately adjustable acceleration and deceleration times, shall be capable of producing full torque at zero speed, and shall be provided with reduced torque feature for reduced-torque stalled motor seating.

C8.6.1.4

DC drives provide the most precise speed and torque control available.

Precise speed control of multiple motors make digital DC drives well suited to control skew on vertical lift bridges.

DC drives have the advantage of not requiring secondary resistors, but do, however, require an additional power supply for the motor field.

8.6.1.5 VARIABLE FREQUENCY SPEED CONTROL

In general, variable frequency drives should not be used for movable bridge drives.

C8.6.1.5

Variable frequency speed control has seen limited success for speed control of movable bridges.

SPECIFICATIONS

COMMENTARY

These drives have the advantage of compatibility with standard squirrel cage motors, although inverter duty motors should be considered for their ability to withstand the higher harmonic voltages that may be produced.

Variable frequency drives have historically lacked the ability for precise control of torque at low speed. Control of torque under overhauling loads and stall conditions, such as seating, has sometimes been unsuccessful.

Add-on dynamic braking units with external resistors are generally required for decelerating and overhauling loads.

8.6.1.6 FLUX VECTOR SPEED CONTROL

Flux vector drives may be specified when precise speed and torque control are required utilizing squirrel cage motors.

The designer shall investigate the need for including:

- Line reactors to reduce standing wave or reflection voltage rise on the feeder to the motor
- Filters
- Motor feeder cable with higher voltage ratings
- Shielded power cable to minimize electrical noise interference to other equipment
- Cable terminators

Application of flux vector drives shall include the use of inverter duty rated squirrel cage motors, matched to the drive systems, and having higher voltage rated insulation to withstand the higher voltages encountered with these drives.

The designer shall investigate the prospective drive manufacturers' recommendations for the control and reduction of electrical noise generated by the drive circuitry, and properly accommodate them in the system design.

C8.6.1.6

Flux vector drives are capable of producing full torque at zero speed through rated motor speed.

Feedback devices, preferability encoder, are required for precision control at low speed and when using a master/slave configuration for skew control on vertical lift bridges.

Dynamic braking resistors are required to dissipate regenerated motor power from overhauling loads. Specify resistor rating to exceed the regenerative capacity of the drive by a minimum of 150 percent.

Voltage reflections are inherent to flux vector drives, and, depending on cable lengths and motor horsepower, may contribute to early insulation failure, particularly when using existing noninverter duty motors.

8.6.2 Master Switches

The use of full voltage drum switches should be limited to in-kind replacements on existing systems. Switches shall be specified such that horsepower and current ratings of the switch meet or exceed the voltage and current requirements of the bridge drive equipment.

New system design shall utilize detent or cam-type master switches to switch contactor coils only. The contract documents shall require that switches shall be heavy-duty industrial switches with self-cleaning contacts,

C8.6.2

Master switches were historically used to directly operate and control wound rotor motors without intermediate contactors. These "drum switches" can be horsepower rated, directly switching secondary resistors for speed control and operating brakes. Horsepower rated drum switches find limited use in new control systems as they introduce motor voltage, usually 480 volts, into the control console.

SPECIFICATIONS

suitable for mounting in the control console. Contact ratings shall exceed the current and voltage requirements for the largest contactor coil to be controlled. Switches shall have positive detents for all switch positions and large pistol-grip control handle.

COMMENTARY

Personnel safety and system maintenance are enhanced by utilizing contactors, remotely located in metal enclosed cabinets. Switching is then accomplished using multisection detent, or cam switches connected to provide the same control sequence as performed by the drum switches.

8.6.3 Resistors

Resistors shall be specified to be corrosion and vibration resistant.

Resistor enclosures in exposed areas shall be specified to be corrosion resistant, preferably stainless steel.

As a minimum, vertical surfaces of resistor enclosures shall be covered with safety screening.

Enclosures with limited ventilation area may include fans in the bottom of the enclosure. Metal fan blades shall be specified.

Field changeable taps shall be specified for resistors.

Resistors utilized in the rotor circuits of wound rotor motors with thyristor drives shall be continuous rated for at least 150 percent of the full load rotor current.

Only high temperature wire rated 250°C shall be permitted within the resistor enclosures.

C8.6.3

Resistance values for secondaries, or rotors, of wound rotor motors should be calculated from motor rotor characteristics as furnished by the motor manufacturer or found on the motor nameplate.

Resistance values should not be used that result in less than 150 percent starting torque at zero speed.

Punched-grid or edge-wound ribbon stainless steel resistors are typically used for new bridge motor control applications.

8.6.4 Tachometers, Encoders, and Overspeed Switches

8.6.4.1 TACHOMETERS

Specifications for tachometers shall state the voltage output at 1,000 RPM.

Generally, tachometers located in machinery areas should be specified to include NEMA 4 enclosures. High-misalignment flexible couplings with keyways shall be specified for tachometer attachment to either the motor shaft, or the motor shaft adapter.

C8.6.4.1

Tachometers are traditionally utilized as analog devices generating a given voltage output, usually DC, for a given RPM of the tachometer shaft. Some manufacturers list encoders, which provide outputs as pulses per revolution, as tachometers.

Generally, higher tachometer voltages, such as 100 volts/1,000 RPM, are preferred for maximum immunity to induced electrical noise on the tachometer circuit wiring.

8.6.4.2 ENCODERS

Encoders shall be specified as incremental or absolute depending on the drive requirements.

Encoders enclosures to be located in machinery areas shall be specified as corrosion resistant, oil and weather proof. Enclosures shall be rugged, mill-duty type to provide protection to the encoder components. Flexible couplings shall be specified.

Specifications shall include the desired format of digital output and line drivers if required. Encoders shall be bi-directional and have a LED light source. Maximum expected RPM shall be specified.

C8.6.4.2

Motor speed controllers can use incremental optical encoders to determine shaft position by counting the total number of pulses during shaft rotation. Incremental encoders are also used as digital tachometers to monitor motor speed.

The designer is cautioned that the low-end operating temperature range of encoders is limited, usually to 0°C, unless extended range options are available.

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8.6.4.3 OVERSPEED SWITCHES

Overspeed switches, when utilized, shall be specified as oil water and corrosion resistant, preferably NEMA 4 rated. Switches shall be attached to rotating machinery shafts with flexible couplings.

Switches shall be selected to provide the best available trip setting resolution for the drive motor speed being monitored. High speed trip setting shall not exceed motor running speed by more than 20 percent.

8.6.5 Motor Control Centers

Motor Control Center (MCC) assemblies shall be specified to be gasketed NEMA 12 construction. Panel mounted devices shall be corrosion and vibration resistant to the extent possible. Unit latches, door hardware, wireway barriers, unit support pans, and other implanted parts shall be suitably plated for resistance to corrosion.

MCCs shall be UL listed. Wiring shall be NEMA Class II, Type C, with master terminal boards.

All weight bearing members and conduit entry roof plates shall be No. 12 U.S. Standard Gage thick steel or heavier.

The main horizontal bus bars shall be tin-plated copper.

Horizontal wireways shall be provided both top and bottom. The wireways shall be completely isolated from all buses. A full height vertical wireway shall be provided in each standard vertical section, and shall be completely isolated from the buses. A separately removable hinged door with 1/4 turn pawl-type latches shall cover the vertical wireway.

Each combination starter unit or other unit configuration shall have an individual door giving access only to that unit.

Operating handles on each unit shall be padlockable in the "off" position.

Consideration should be given to providing the MCC with ammeters and voltmeters. Where voltmeters are provided, they shall be provided with a switch to select phase-to-phase or phase-to-ground voltage and phase amperage for each phase.

Nameplates bearing the name of the equipment served shall be provided on each unit of each MCC cabinet. Nameplates shall be permanently attached to the front of each unit with corrosion-proof machine screws.

Each contactor, circuit breaker, relay, and transformer shall also be labeled with its own individual nameplate according to its designation on the drawings, and located inside each MCC unit adjacent to the respective device.

COMMENTARY

C8.6.4.3

Preferably, the overspeed switches should be specified to include two overspeed switch assemblies, independently adjustable. Resolvers have the advantage of providing precise shaft position information while being extremely rugged and having a much wider temperature range than encoders.

Voltage sensing of tachometers or frequency sensing of encoders is sometimes utilized as overspeed protection in lieu of a mechanical speed switch.

C8.6.5

Cabinet heaters and thermostats should be specified if condensing humidity is anticipated. Standard MCC cabinets cannot be located in wet or damp locations.

SPECIFICATIONS

Cabinet finish shall be baked enamel, or epoxy, color as specified. Cabinet interior surfaces shall be enamel coated.

Half-height units shall not be permitted.

8.6.6 Contactors

Contactors shall be full voltage magnetic, horse power rated with weld resistant contacts. Contacts shall be gravity drop-out. Arc covers shall be easily removable for inspection and replacement of the contacts. Contactors shall have provisions for installation of auxiliary contacts.

8.6.7 Electronic Reduced Voltage Starters

Specifications for reduced voltage starters shall include motor horsepower, voltage, maximum current, and duty cycle. If the starter is self-contained, pilot devices and the enclosure shall be specified. Closed circuit transition switching should be specified where run contactors are used to eliminate transient switching currents.

8.6.8 Overload Relays

General purpose motor overload relays, other than those for main drives, shall be ambient compensated bimetallic-type. Overload relays shall be specified to include removable heater elements and test trip button. Overload relays shall be specified as manual or automatic reset as may be required. The overload class shall be specified.

Overload relays for the main drive motors shall have adjustable trip time and adjustable trip current. These adjustments shall be easily made in the field without replacing any components of the overload relay.

Overload relays shall not be used as torque limiting devices to protect machinery from excessive motor torque.

8.7 ELECTRICALLY OPERATED BRAKES

To minimize shock to the machinery, electrically operated brakes used with the span drive machinery should be provided with a means for controlling the rate that the brakes set. This may be accomplished via adjustable mechanical damping, such as an adjustable

COMMENTARY

C8.6.6

NEMA rated contactors with their conservative ratings and long operating life have been traditionally preferred for movable bridge applications. IEC contactors, used internationally, should be selected carefully within well defined parameters. Although the operating cycle life of IEC contactors is rated about one third that of NEMA contactors, the designer may have applications where the smaller size of the IEC contactors are advantageous.

C8.6.7

Reduced voltage starters are used to soft-start standard squirrel cage induction motors by limiting inrush current. Current and torque are adjusted by setting current limits and ramp time on solid state reduced voltage starters. Reduced voltage starters have limited duty cycles when used with high inertia loads and are not recommended for use with the bridge drive motors. They do not incorporate speed sensing feedback circuits, and are, therefore, not suitable for speed control.

C8.6.8

Overload relays shall be connected such that an overload on any one phase trips all three phases of the motor circuit.

Auxiliary contacts are recommended.

Class 20 overload relays, tripping in 20 seconds at 600 percent overload, are typical.

Overload relay requirements for main drive motors can be satisfied through the use of solid state motor controllers or oil filled dashpot inverse time relays. Standard motor overload relays seldom protect a movable bridge motor properly, particularly if electronic drives are involved.

Instantaneous-trip overload relays are generally not suitable for bridge drive motors.

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orifice for air or hydraulic fluid, or by voltage control for solenoid brakes.

Electrically operated brakes shall be equipped with means for manually releasing the brakes in the event of electrical actuator failure.

Span drive brakes shall be provided with separate limit switches to indicate the following conditions:

- set
- released
- hand-released

8.8 CONTROL CABINETS

Control cabinets shall be minimum NEMA Class 1 construction and shall be UL listed. With the exception of factory assembled MCCs, cabinets shall be constructed from minimum No. 12 U.S. Standard Gage thick steel. Doors shall be gasketed with three point latching mechanisms and corrosion resistant hardware.

Control cabinets shall include door operated flourescent lights, steel back panels for equipment mounting.

All cabinets shall have lifting eye bolts.

Cabinets shall include space heaters with thermostats, ventilating fans or air conditioning when required.

All miscellaneous hardware shall be corrosion resistant. Exterior shall be finished with baked enamel or epoxy, and interior with baked enamel.

All cabinets shall be identified with screw attached, engraved or stamped nameplates.

COMMENTARY

C8.8

The enclosures described in this section are generally large cabinets used for control components, motor starters and MCCs.

Although No. 12 U.S. Standard Gage thick steel is permitted, the designer should consider specifying No. 10 U.S. Standard Gage thick steel when cabinets are in machinery areas or other locations subject to damage or abuse.

The specifications listed are for equipment to be used in a clean and dry environment. The designer should consider NEMA 12 enclosure requirements if dust, dirt or condensing humidity are present. Enclosures in a corrosive atmosphere, including salt water, or located outdoors, should be NEMA 4X with stainless steel hardware.

8.9 ELECTRICAL CONDUCTORS

8.9.1 General Requirements

General purpose wiring shall be copper with ASTM Class B stranding. Insulation shall be 600 volt rated and shall be listed for use in wet locations. Conductors shall be UL listed and shall be types THHW, THW, or XHHW.

Conductors shall be sized to limit the maximum voltage drop to 5 percent from the incoming service to the end device on any circuit.

Minimum conductor size shall be AWG 12 on the bridge structure. AWG 14 conductors shall be allowed within control consoles and factory wired, UL listed control panels.

The contract documents may permit shielded control wires with insulation rated as low as 300 volts when routed through raceways or enclosures containing circuits not exceeding 120 volts.

C8.9.1

An AWG/Metric conversion table for wire sizes can be found in the appendix.

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8.9.2 Splicing and Tapping Conductors

Splices and taps shall be made only within junction boxes, terminal cabinets, and conduit bodies. Splices and taps made in terminal cabinets shall be made on terminal blocks. In-line splices shall be made using tubular compression splices covered with heat-shrink sleeves or mold- poured epoxy resin splice kits, with a voltage rating equal to or exceeding the conductor insulation rating.

Splices in wet locations shall be insulated with waterproof materials listed for use in wet locations, such as epoxy encapsulation.

Splices of power cables and cable connections to motor terminals shall utilize compression-type bolted lugs with insulating covers.

Splices and taps shall not be permitted in inaccessible locations.

Wire nuts shall be restricted to use only within switch and receptacle boxes for 120 volt branch circuits rated 20 amps or less serving lights and convenience receptacles.

Split bolt connectors shall not be permitted.

8.9.3 Labeling and Identifying Conductors

All conductors shall be labeled with permanent, oil and water-resistant, preprinted identification bands or sleeves. Conductors shall be identified at every tap, termination and in all junction boxes, pull boxes and equipment enclosures.

Contract documents shall require that conductors shown on multiple drawings, supplied by multiple vendors, or supplied by multiple subcontractors be consistently and uniquely identified, such that any given conductor will be assigned only a single wire number which is the same on all drawings and diagrams on which it appears.

8.9.4 Exposed Conductors and Cables

Conductors and cables to be installed in exposed locations shall be specified as resistant to sunlight, weather, oil or other products of the environment of the installation.

8.9.5 Flexible Loop and Droop Cables

Flexible loop or droop cables shall be comprised of extra flexible, round portable cables with a heavy-duty black neoprene jacket, same or similar to UL Type SO or Type W. Conductors shall be stranded copper in accordance with ASTM Class K. The flexible cables shall be connected to the conventional wiring on terminal strips in terminal cabinets.

COMMENTARY

8.9.2

All wiring should be continuous between terminals. Taps and splices, when required, should be made on terminal blocks inside a terminal cabinet or junction box.

C8.9.5

Flexible cables should be specified for applications between fixed and moving parts. Examples are the electrical connections between a bascule pier and a bascule span, between tower and movable span on a vertical lift bridge, between a fixed pier and a timber fender, or between a fixed junction box and an adjustable floodlight.

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Stainless steel or bronze wire mesh cable grips shall be specified wherever strain relief to the flexible cables is required, such as where they enter terminal cabinets.

8.9.6 Aerial Cables

The contract documents shall specify products and materials for aerial cables which will be suitable for the anticipated environmental condition and the anticipated service life. Aerial cables shall be rated for long-term exposure to various weather conditions such as ice, wind, rain, and direct sunlight.

Aerial cables shall be round, covered with a heavy-duty jacket which is resistant to sunlight, weather and aging, such as black Neoprene, or polyethylene.

Each electrical cable shall be attached to a messenger cable at intervals of not more than 2 ft. All messenger cables shall be of high-strength corrosion resistant material and shall be adequately anchored.

Messenger cables, cable hangers, and all accessories shall be corrosion resistant. The messenger cable shall be suspended with sag as required to safely support the entire assembly under various conditions of wind, ice, and temperature appropriate for the location of the bridge without exceeding the safe working tension of the messenger.

The amount of cable sag shall be determined and specified by the designer for each particular installation.

8.9.7 Submarine Cables

Submarine cables shall be specified as rated for long-term underwater installations and suitable for use in wet or dry locations which may be exposed to direct sunlight.

General configuration of the submarine cables shall consist of stranded copper conductors insulated with crosslinked polyethylene (90°C), cabled with fillers as necessary, binder tape, polyethylene inner jacket, polyethylene coated spiral-wound galvanized steel armor wires, separator tape, and a polyethylene outer jacket.

All cables shall be manufactured in accordance with ICEA S-66-524, NEMA WC-7-1988, as applicable.

8.9.7.1 CONDUCTORS

Wires shall be annealed uncoated or tinned copper in accordance with ASTM B 3. Conductors shall be stranded in accordance with ASTM B 8, Class "B" stranding.

Conductor insulation shall be a chemically crosslinked polyethylene, XLPE.

The thickness of the conductor insulation shall comply with ICEA S-95-658/NEMA WC-70.

COMMENTARY

C8.9.6

Aerial cables are generally stationary overhead cables between two fixed points.

Aerial cables are intended to be messenger supported. Clearances, loadings and messenger strength are governed by the NESC.

For examples of standard industry practice, refer to sag and tension calculations in any recent edition of the Standard Handbook for Electrical Engineers by Fink and Carroll, McGraw Hill Book Company.

Use appropriate ice and wind loading for the locality as given in the NESC.

C8.9.7

Unless otherwise specified, the cables should be placed at least 5 ft. below the bed of the channel. The U. S. Army Corps of Engineers and the U. S. Coast Guard may impose greater depth requirements, which should be confirmed for each bridge site.

Cables shall be long enough to provide sufficient slack for terminating.

C8.9.7.1

ICEA S-66-524 revision has deleted color coding methods and tables.

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COMMENTARY

Color coding of insulated conductors shall be accomplished by surface printed legends consisting of numbers and color designations in accordance with ICEA S-73-532, NEMA WC57-1990, Method No.3, and Table E1 or E-5, as required.

The contract documents shall require that conductor identification color and print be legible after normal handling during installation.

8.9.7.2 CABLE CONSTRUCTION

Cable components shall be assembled into a tight concentric configuration. The direction of lay for adjacent layers of cabled conductors shall be reversed. Maximum lay lengths and lay directions shall conform to ICEA S-95-658/NEMA WC-70.

Fillers shall be employed as necessary within the cabled core to produce a substantially circular cross-section. Fillers shall be nonhygroscopic polypropylene or polyethylene.

The cables shall be covered with a 2 mil. corrugated polyester binder tape prior to application of the inner jacket. The tape shall be applied helically with a minimum overlap of 25 percent.

After cable fabrication is completed, the cable ends shall be suitably sealed by the cable manufacturer to prevent moisture from entering the conductor core area during shipment and storage.

8.9.7.3 INNER AND OUTER JACKET MATERIAL

Both the inner jacket which covers the cabled core assembly, and the outer jacket covering the armor wire shall be homogeneous layers of high density polyethylene in accordance with ICEA S-95-658/NEMA WC-70. The thickness of the inner and outer jackets shall be taken as specified in Table 1:

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COMMENTARY

Table 8.9.7.3-1 - Jacket Sizes

Calculated Diameter of Cable Under the Respective Jacket (in.)	Jacket Average Thickness (in.)
0-0.425	45
0.426-0.700	60
0.701-1.500	80
1.501-2.500	110
2.501 and larger	140

8.9.7.4 CABLE ARMOR WIRE

The armor wire shall consist of galvanized steel wires. Each armor wire shall be coated with a layer of high density polyethylene. The coating shall be sunlight and weather resistant.

The coated armor wires shall be applied at a lay angle of 17 to 25 degrees and provide a coverage of 92 to 98 percent. The coated armor wires shall be applied in a left lay helix.

The armor wires shall be sized as specified in Table 1:

Table 8.9.7.4-1 - Armor Wire Sizes

Calculated Diameter of Jacketed Core (mm)	Nominal Size of Armor Wire (mm)	Nominal Thickness of PE Coating (mm)
0-19.05	2.77	0.508
19.06-25.40	3.40	0.635
25.41-43.18	4.19	0.762
43.19-63.51	5.16	0.889
63.52 and larger	6.05	1.016

A layer of separator tape shall be installed over the spiral-wound armor wires, prior to the application of the outer jacket. A 2 mil. corrugated polyester separator tape shall be applied helically with a minimum overlap of 25 percent.

8.9.7.5 TESTING

The finished cable shall withstand, between each conductor and all other conductors, including armor, an AC rms voltage in accordance with ICEA S-95-658/NEMA WC-70 for nonshielded 0–2 kV cables.

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COMMENTARY

The insulation resistance shall be measured after the completed cable AC voltage tests. The measurement method shall be in accordance with ICEA S-95-658/NEMA WC-70.

Contract documents shall specify that all tests be certified, with results given to the owner prior to installation of the cable.

8.9.7.6 SUPPORT

Submarine cables shall be anchored against the pier with anchor brackets constructed of corrosion resistant materials.

Submarine cables shall be supported vertically with anchor heads designed to support the weight of the cable by the armor wires.

C8.9.7.6

A slack loop of cable between the anchor head and the terminal cabinet may prevent undue strain on the conductors entering the cabinet.

Where waterway and trench geometry permit, a horizontal loop in the submarine cable may help protect against errant vessels dragging anchor.

8.10 CONDUITS, WIREWAYS, BOXES AND CABINETS

8.10.1 Conduit, General Requirements

Metal conduits shall be specified to comply with American National Standards Institute requirements as follows:

- ANSI C80.1 - Hot Dip Galvanized Rigid Steel Conduit
- ANSI C80.3 - Electrical Metallic Tubing
- ANSI C80.5 - Rigid Aluminum Conduit
- Nonmetallic conduits shall be specified to comply with:
    - NEMA TC2
    - UL Standards 651
    - Federal Specification WC-1094A

C8.10.1

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All conduits shall be specified to be UL listed and installed in accordance with the appropriate section of the NEC.

Size of conduits shall be specified in accordance with the NEC. In no case shall conduits less than 19 mm (3/4 inch) diameter be used.

Metal conduits shall be provided with insulated throat grounding bushings and shall be electrically bonded with a bonding wire to all metal enclosures or wireways that they enter.

Couplings and fittings shall be specifically designed and manufactured for the application and for the conduit material.

Where conduit runs are attached to structures subject to movement, the contract documents shall contain provisions for the associated expansion or deflection. Fittings for accommodating expansion or deflection must consider the location, accessability and the calculated amount of movement.

Exposed conduit shall be supported on spacers, designed for the purpose, to support conduits at least 0.5 in. off the surface to which they are mounted.

Rigid conduit mounted on bridge structures should be supported at intervals of 6 ft.

COMMENTARY

Pressure or "C" clamp attachments should be avoided when bolted connections can be utilized. Bolted attachments are perferred on the movable spans due to generally higher vibration in those areas.

(At the time of this writing (1998), metric size conduit is not available in the United States or Canada. Therefore, the currently available size in English units is shown with the a3pproximate metric value in parentheses.)

The vibration on bridge structures requires that conduit supports should be used at spacings considerably less than those required by the NEC. The recommended 6 ft. maximum applies to all conduit diameters and should not be exceeded, except due to structural limitations, in which case the maximum support spacing shall be maintained as close to 6 ft. as possible.

8.10.1.1 RIGID STEEL CONDUIT

Rigid steel conduit and fittings shall be specified to be hot-dip galvanized.

Where specified, rigid steel galvanized conduit and fittings shall be coated with a factory applied plastic coating with a nominal 40 mil. thickness. The contract documents shall require that the bond between the PVC coating and the steel exceed the tensile strength of the plastic coating, taken as a minimum of 2,000 psi in lieu of better information. The contract documents shall specify that the interior and threads of plastic coated conduits have a urethane coating, 2 mil. nominal thickness.

C8.10.1.1

All field cuts, or other interruptions in the galvanized coating shall be touched up with zinc rich cold galvanizing compound.

All nicks, cuts and scratches in the PVC coating shall be touched up with touch-up compound supplied by the conduit manufacturer.

Uncoated rigid steel conduit shall be protected with a bituminous coating or hot-applied bituminous tape at locations where conduit exits concrete or soil into air, or exits concrete into soil.

8.10.1.2 RIGID ALUMINUM CONDUIT

Rigid aluminum conduit shall be installed so as to be isolated off of structural steel and masonry by at least 0.5 in.

Aluminum conduit shall not be directly buried or encased in concrete.

C8.10.1.2

Aluminum conduit, where specified, should be used in exposed locations only. Precautions should be taken to isolate aluminum conduit from contact with soil or concrete.

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8.10.1.3 ELECTRICAL METALLIC TUBING (EMT)

EMT shall be specified as galvanized steel with a laquer coating. Installation, where permitted, shall be in accordance with the NEC, Article 348.

Electrical metallic tubing shall only be used within dry finished areas, such as operator's houses, and shall not extend outdoors or into unfinished areas such as piers or bridge structures, or be concrete encased.

8.10.1.4 RIGID NONMETALLIC CONDUIT

Rigid nonmetallic conduit may be specified for use only in nonexposed locations, in concrete encasements or where it is buried. Rigid nonmetallic conduit shall be specified in accordance with the NEC, Article 347, and as specified herein.

Schedule 40 PVC conduit may be specified only when concrete encased.

Fiberglass reinforced epoxy or other underground duct material shall be specified only where concrete encased.

8.10.1.5 FLEXIBLE CONDUIT

All flexible metal conduit shall be specified as UL listed and labeled liquid-tight.

Jacket shall be specified to be water, oil, and, where required, sunlight resistant.

Flexible conduit shall not be permitted for installation in concealed locations, buried, or concrete encased.

The designer shall specify all flexible conduits to include a separate ground conductor.

Installation shall be specified to comply with Article 351 of the NEC.

Fittings for liquid-tight flexible conduit shall be specified to be of a type specifically designed for the purpose, and corrosion resistant.

The designer shall specify that all connections to motors, brakes, limit switches, span sensing devices and other equipment shall include a minimum length of 18 in. of flexible conduit.

COMMENTARY

C8.10.1.5

An external grounding braid or strap provides visual confirmation that the ground connection is intact.

8.10.2 Wireways

Wireways shall be specified to be of steel or stainless steel construction with gasketed covers, UL listed, and suitable for the environment where it is to be installed. Wireways used in exposed locations shall be specified oil tight, water tight and corrosion resistant.

Installation and conductor fill shall be specified in accordance with Article 362 of the NEC.

Removable covers shall be accessible.

C8.10.2

Separation of power and control circuits should be considered when specifying wireways, in accordance with NEC.

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8.10.3 Junction Boxes and Terminal Cabinets

Junction boxes, pull boxes and terminal cabinets specified for outdoor locations or where not otherwise protected from oil and water, shall be rated NEMA 4 or 4X.

Boxes and cabinets in exposed locations shall be constructed of cast aluminum, hot-dip galvanized cast iron, or stainless steel. Hinges, bolts, screws and other hardware shall be specified to be brass or stainless steel.

Cast aluminum enclosures shall be specified to be isolated from concrete through the use of neoprene or similar material.

The size of boxes and cabinets, and the installation thereof, shall be based on requirements of the NEC, Articles 370 and 373, respectively.

Terminal cabinets shall have permanent labels attached indicating the designation of the cabinet as shown on the contract plans.

Terminal cabinets shall be specified to include interior mounting panels for terminal strips.

Cabinets and boxes in wet locations or subject to condensation shall be specified to include a minimum 6mm drain hole at the low point of the enclosure.

General purpose cabinets, boxes and enclosures to be installed indoors shall be NEMA 12.

Cabinets and boxes in dry, environmentally controlled areas and exceptionally clean may be specified as NEMA 1.

COMMENTARY

C8.10.3

Preferably all boxes exposed to the weather should be specified to conform to NEMA 4X requirements for maximum corrosion resistance.

It is common practice to isolate cast aluminum enclosures from structural steel.

Conduit entry should be to the bottom of enclosures whenever possible. The contract documents should require that requests for use of top entry for conduits be reviewed on a case-by-case basis.

The NEC should be considered the minimum requirements. In many cases, for ease of maintenance or longevity, it will be necessary to exceed the NEC requirements.

Larger drain holes if specified, should be screened to prevent entry of insects and rodents. Preferably, drain fittings, designed for the purpose, could be specified.

NEMA 12 rated enclosures should be specified in lieu of NEMA 1 as the minimum enclosure type in dry, environmentally controlled areas. NEMA 12 enclosures provide added protection from dust and dirt.

8.11 SERVICE LIGHTS AND RECEPTACLES

The contract documents shall contain provisions for complete electric lighting systems for:

- the operator’s house,
- machinery house or deck,
- tower top machinery rooms,
- all stairways and walkways,
- the end lifting and locking areas,
- all other areas where inspection or maintenance of equipment is required.

The lighting systems shall be designed so that the following intensities are achieved:

- Operator’s house - 300 lx

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- Machinery room - 200 lx
- Walkways, stairways, ladders, elevators - 20 fc
- Unhoused equipment - 15 fc

The lighting systems may utilize flourescent, incandescent, or high intensity discharge lighting fixtures or any combination thereof.

Incandescent lighting fixtures shall be rated for 100 watt lamps minimum, although smaller lamps may be specified to be installed. Similarly, conductors which supply incandescent fixtures shall be sized based on a minimum of 100 watts per fixture.

Control room lighting shall preferably be designed with dimming capability, adjustable from or near the control console.

Exterior lighting shall consist of enclosed high intensity discharge fixtures or vaportite incandescent fixtures equipped with globes and guards.

All lighting fixtures should be equipped with shock absorbing porcelain sockets, where practical.

Duplex-type receptacles shall be provided in each room of the operator's house, in machinery rooms, the end lifting and locking apparatus, and all other areas where the inspection or maintenance of equipment is required. Consideration shall also be given to furnishing receptacles inside, or in close proximity to the operator's console, control logic cabinets, terminal cabinets and span drive cabinets.

All receptacles shall be duplex, three wire, ground fault interrupter.

All receptacles exposed to the weather shall be housed in weatherproof enclosures, and all exposed parts shall be corrosion resistant.

Lighting circuits shall not be supplied from branch circuits that supply receptacles.

8.12 GROUNDING

8.12.1 General

A grounding system shall be provided to meet or exceed the requirements of the NEC. The power system supplying the bridge shall be a solidly grounded system.

8.12.2 Equipment Grounding

Grounding for all equipment, cabinets and enclosures containing electric equipment shall be by dedicated grounding conductors run in each conduit and raceway from each piece of equipment, cabinet and enclosure back

COMMENTARY

The designer should consider the lamp starting delays associated with high intensity discharge lighting. These lamps may not be suitable for use on certain walkways or stairways where such a delay could compromise safety.

Where using flourescent lamps outdoors or in unheated locations, the lamp performance characteristics should be investigated for the lowest ambient temperatures likely to be encountered.

C8.12.2

Some control equipment utilizes isolated ground systems which may not be connected to the system ground. The specifier should be aware of this situation and

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to the system ground bus. Conduit and raceways shall not be utilized as the sole grounding means for electric equipment. The grounding conductors shall be sized according to the NEC.

8.12.3 Structure Grounding

The bridge structure steel and attached electrical equipment shall be grounded by an explicit connection, such as a copper cable exothermically welded to structural steel, and connected to a suitable grounding electrode of one of the following types:

- steel pipe piles, steel H piles, or permanently placed steel sheet piling;
- copper ground rods driven at least 10 ft. into mud immediately adjacent to pier or substructure;
- stainless steel plate of at least 10 $ft.^2$ of exposed area mounted as low as possible below water on the pier or substructure, such that the entire plate is completely below the lowest water elevation at all times.

Concrete or masonry substructures and piers shall not be considered adequately grounded. Reinforcing steel in concrete shall not be used in lieu of copper electrical cables for grounding.

Copper cable, size AWG 1/0 or larger, shall be used to connect the bridge structure to the grounding electrode.

Bridge grounding, as specified above, shall be provided for the various bridge types as follows:

- Bascule Bridges - two bridge grounding connections on each side of the waterway or channel, on the pier or substructure closest to the channel, placed on opposite ends of each such pier or substructure.
- Swing Bridges - one bridge grounding connection at each end of the swing span, connected to an end wedge or end lift device bearing plate on each rest pier; and two bridge grounding connections at the pivot pier, placed diametrically opposite.
- Vertical Lift Bridges - two bridge grounding connections on each tower on the pier or substructure closest to the channel, placed at diagonally opposite corners on that pier or substructure.

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should follow the manufacturer's recommendations for connection of this equipment.

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8.13 LIGHTNING PROTECTION

8.13.1 Standards and Codes

NFPA 780

8.13.2 Requirements

8.13.2.1 GENERAL

Lightning protection shall be provided for movable bridges, including operator and machinery houses, and any appurtenances or equipment that project significantly above the surrounding structure.

Lightning protection shall consist of the air terminals, main and bonding conductors, and ground terminals, as defined by NFPA 780.

The lightning protection system shall be designed and specified in accordance with NFPA 780 and the NEC, and the contract documents shall require conformance of the installation to these same standards. Early streamer emission systems, or lightning dissipater arrays may be used in designs which comply with NFPA 780 and the NEC.

Only corrosion resistant materials, such as copper, bronze, and stainless steel shall be used for the lightning protection system components. Aluminum, iron, or coated carbon steel shall not be considered corrosion resistant for this purpose.

8.13.2.2 AIR TERMINALS

Air terminals shall be manufactured of solid copper, copper bronze, or stainless steel. Multipoint or brush-type static dissipating air terminals may also be used in conjunction with conventional air terminals.

8.13.2.3 CONDUCTORS

Main and bonding conductors shall be stranded copper, sized per NFPA 780.

The structural steel framework of a structure may be used as the main conductor per NFPA 780. Main or bonding conductors for lightning protection systems shall not be run in steel conduit.

COMMENTARY

C8.13.2.3

NFPA 780, Paragraph 3-19.1 allows the structural steel framing of structures to be used as the main conductor for the lightning protection system, provided it is electrically continuous. It is suggested that where steel member bolted faying surfaces are unpainted, they should be considered electrically continuous. However, painted faying surfaces may present questionable current carrying capability and objectionable voltage rise, in which case a copper main conductor would better ensure the safe conduction to ground of the lightning stroke.

Reinforcing bars in concrete structures and substructures should not be considered electrically continuous. Historic evidence exists to show that a lightning stroke traveling across a gap between embedded

SPECIFICATIONS

8.13.2.4 ATTACHMENT HARDWARE

Attachment hardware shall be solid copper, bronze, or stainless steel. Fastening details of the hardware and components to the structure or substructure shall be shown in the contract documents in sufficient detail so as to constrain the installer to a methodology that will have no detrimental effects on the structure.

8.13.2.5 GROUND TERMINALS

The grounding system described previously for the structure, where in conformance with NFPA 780, may also be utilized for the lightning protection system. When necessary, additional ground rods or other suitable grounding electrodes shall be provided.

8.14 SPARE PARTS

The contract documents shall enumerate spare parts for the electrical system to be provided by the contractor. In lieu of owner-supplied requirements, the following spare parts may be specified:

- One spare processor and one spare I/O module of each type used for PLC systems.
- Three fuses of each size and type used.
- One complete set of movable and stationary contacts for each size of contactor and starter used.
- One coil for each size of contactor and starter used.
- One brake coil or thruster motor for each size of brake used.

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reinforcing bars in a bridge pier can dissipate sufficient energy in the concrete to fracture the pier. A jacketed or PVC-conduit-protected main copper conductor may be embedded in bridge piers and brought out near, and or exothermically welded to, the bridge bearing plate or anchor bolts. Such practice must be carefully coordinated with and approved by the structural designer.

When protecting lightning system conductors in conduit, only nonferrous conduits should be used. The magnetic properties of steel conduit surrounding the conductor behave as a magnetic choke and raise the impedance of the conductor, as seen by the lightning, to unacceptably high values. This impedance may not be readily apparent at 60 Hz power frequencies and normal current amplitudes.

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- Six indicating lights for each type used.
- One spare limit switch for each type used.
- One spare control relay for each type used.
- One complete navigation light assembly for each type used, including one lens of each color and size used.
- Additional spare parts shall be specified as recommended by the manufacturers for engines, engine-generator sets, position indicating devices, electronic control components, tachometers, and other parts.

REFERENCES

Campbell, S. J. “Synchrotie Systems,” Letter #1389A, an internal document in the Industrial Engineering Department of Westinghouse Electric Corporation, East Pittsburgh Regional Engineering Department, March 24, 1961.

Nowacki , L. M. “Induction Motors as Selsyn Drivers,” a paper presented at the January 1934 A.I.E.E. winter convention, and published in the A.I.E.E. Transactions, Page 1721.

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A swing span having unequal lengths or unbalanced dead loads shall be balanced by counterweighting.

1.5.2 Counterweight Details

Provision shall also be made for independent supports for the counterweights of vertical lift bridges for the purpose of replacing counterweight ropes.

Counterweights usually shall be of concrete, supported by a steel frame. Boxes shall be rigidly braced and stiffened to prevent warping or bulging. All surfaces of the boxes in contact with the concrete shall be provided with open holes ~~about~~ 700 in.^{2} to each 10 ft.^{2} of surface to ~~permit~~ escape of water from the box as the concrete cures, 1 or otherwise a low-slump concrete shall be used with any excess water drawn off as the concrete is placed. In the design of counterweight attachments, details which may produce fatigue effects due to vibration of the structure shall be avoided.

Concrete counterweights not enclosed in steel boxes shall be adequately reinforced.

Counterweights shall be made so as to be adjusted for variations in the weight of the span and in the unit weight of the concrete. Pockets shall be provided in the counterweights to house the balance blocks necessary to compensate for not less than 3.5 percent underrun and 5 percent overrun in the weight of the span. Each completed counterweight shall contain not less than one percent of its weight in balance blocks, arranged so as to be readily removable for future adjustment. Additional blocks for

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no greater than 20 degrees above or below a horizontal line passing through the trunnion.

- For single leaf bascules, the equivalent downward reaction at the toe should be 1,000 lb. per bascule girder with the leaf fully seated.
- For double leaf bascules, the equivalent downward reaction at the toe should be 1,500 lb. per bascule girder with the leaf fully seated.
- Bascule bridges operated hydraulically, especially with cylinders, should be balanced such that they are span heavy in all positions. The amount of imbalance must be carefully considered by the designer. The imbalance may range from the values specified herein to a non-counterweighted design. There have been several single leaf bascule spans constructed without counterweights.
- Vertical lift bridges should have a downward reaction of no less than 1,000 lb. per corner with the span fully seated. This value should increase with the size and mass of the vertical lift up to a maximum of 2,700 lb. per corner.

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future adjustment in the amount of 0.5 percent of the weight of the counterweight shall also be provided and shall be stored at the site as specified by the Engineer. All balance blocks shall be firmly held in place so that they will not move during the operation of the bridge. Balance blocks shall be provided with recessed handles and shall weigh not more than 100 lb. each. Balance blocks shall be furnished only as necessary to meet the specified requirements for future adjustment and to secure the required balance of the span and the counterweights.

Pockets in counterweights shall be provided with drain holes not less than 2 in. in diameter. The pockets should be covered where exposed to the weather. The cover shall be weatherproof. The pockets should provide for both vertical and horizontal load adjustment.

1.5.3 Counterweight Concrete

Unless otherwise specified, concrete shall be made with Type II cement, and shall be proportioned as directed by the Engineer, with not more than 5½ gal of water per sack taken as 43 kg of cement. Where heavy concrete is required for counterweights, the coarse aggregate shall be trap rock, magnetic iron ore, or other heavy material, or the concrete may consist of steel punchings or scrap metal, and mortar composed of 1 part cement and 2 parts of fine aggregate. The maximum weight of heavy concrete shall be 315 pcf and preferably not more than 275 pcf. Heavy concrete shall be placed in layers and consolidated with vibrators or tampers. Methods of mixing and placing shall be such as to give close control of the unit weight of the concrete and uniformity of unit weight throughout the mass. Steel ingots or billets shall preferably be used in counterweights when required to obtain specified counterweight density. They shall be distributed uniformly throughout the counterweight, shall be individually supported within the counterweight and shall be clean and free of oil or grease and not galvanized or coated with other materials.

For ascertaining the weight of the concrete, test blocks, having a volume of not less than 4 $ft.^3$ for ordinary concrete, 1 $ft.^3$ for heavy concrete and 1 $ft.^3$ for the mortar for heavy concrete, shall be cast at least 30 days before concreting is begun. Two test blocks of each kind shall be provided with one weighed immediately after casting and the other after it has seasoned. If samples show a tendency to swell, they shall be rejected.

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1.5.4 Counterweight Pits and Pit Pumps

If practical, counterweight pits shall be drained by gravity. Otherwise, provision shall be made for pumping water out and for cleaning of the counterweight pit. A sump shall be placed in the floor of the pit so that water will collect in it. The pump shall preferably be placed in the sump. Sump pumps should be automatically controlled with level switches for on and off, and shall be specified as trash pumps to pass small debris without clogging. Consideration should be given to environmental containments in pit effluent.

1.5.5 Diversion of Drainage

Provision shall be made for preventing excessive roadway drainage from entering the counterweight pit.

1.6 MACHINERY AND OPERATOR'S HOUSES

X Article

See section 1.4.4 for additional safety design considerations. See Section 8 for additional electrical requirements for machinery and operator's houses.

1.6.1 Machinery House

A suitable house or houses shall be provided for the protection of the machinery wherever practical. Houses shall be large enough for easy access to all machinery, and shall be fireproof and weatherproof. Houses or rooms containing electric control equipment shall have thermal insulation and climate control as specified. Windows, when provided, shall be glazed with shatterproof glass. Openings shall be large enough to admit passage of the largest unit of machinery.

The floor shall be concrete, steel, or other fireproof material, as specified. It shall have a nonslip surface. Floors in rooms containing electrical equipment such as motor control centers or control cabinets shall be covered with vinyl, rubber tile, or other electrically insulating materials as specified on areas surrounding such electrical equipment.

Consideration shall be given for a hand-operated overhead traveling crane or hoist, of sufficient capacity for handling the heaviest piece of machinery, to be specified in the machinery house.

Adequate heating and ventilation provisions shall be made for times when maintenance personnel are performing maintenance tasks in the house during the climatic extremes for the location.

Where a machinery house is not feasible or practical, exposed or semi-exposed machinery decks may be utilized for mounting weatherproof machinery.

2.3 NOTATION

Insert the following additional notation in the proper order:

ADO = estimated average daily number of openings (2.4.1.5)

DAM = force effects from operation of machinery including 100 percent dynamic allowance (lb. or lb.-in.) (2.4.1.2.3)

N_o = number of operational open and closed cycles in 75 years (2.4.1.5)

n_o = number of stress range cycles per operation (2.4.1.5)

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girders shall coincide with the corresponding central planes of the webs of the track girders. The treads attached to the segmental girders and track girders shall be steel castings or rolled steel plates and shall not be considered as part of the flanges of these girders.

The allowable load for line bearing P_{LB} (lb./in.) between treads of segments having a diameter of 10 ft. or more shall not exceed:

X

$$P_{LB} = (12{,}000 + 80D)\left(\frac{F_y - 13{,}000}{20{,}000}\right) \qquad (2.5.1.1.3\text{-}1)$$

Throughout document.
Due to conversion?

where:

D = diameter of the segment (in.)

F_y = specified minimum yield strength of the material in tension (psi)

The thickness t (in.) of sole plates and of the flanges of flange-and-web castings shall not be less than t = 3.00 + 0.004 D. Tread plates may be flange-and-web castings. The edge thickness of the rolling flange shall not be less than 3 in., and the flange thickness at any horizontal section of the web of the casting shall be such that the unit bearing on the web of the casting shall not exceed one-half of the yield strength of the material in tension, with the length of the bearing taken as twice the depth from the rolling face to the plane under consideration.

The effective length of the line bearing for each web shall not exceed the thickness of the web of the segmental or track girder, including the effective thickness of the side plates, plus 1.6 times the least depth of the tread. The edge of the web shall be machined so as to bear continuously upon the tread. The thickness of the web shall be such that the quotient obtained by dividing the load by the area of a portion of the edge of the web whose length equals twice the thickness of the tread, shall not exceed one-half of the minimum yield strength of the material in tension.

Flange angles shall not be considered as transmitting any load from the web to the treads. The bearing value of side plates shall not exceed the resistance of those fasteners or welds connecting them to the web, which are included between diverging lines, in the plane of the web, that intersect at the line contact between the treads and track. These lines make an angle whose tangent is 0.8 with the normal to the rolling surface. The load, as used in this article, shall be the weight of the structure, no addition being made for rolling impact.

Solid tread plates on segmental girders shall have a radius slightly smaller than the segmental girders in order to facilitate the securing of tight contact with the girders throughout their length when drawn up with the attaching bolts.

5.3 NOTATION

Insert the following additional notation in the proper order:

D = diameter of sheave (in.) (5.8.3)

d = diameter of rope (in.) (5.8.3)

n = full-load speed (rpm) (5.4.1)

T_{min} = minimum required full load torque (lb.-in.) (5.4.1)

u = worm gear ratio (5.8.4.2.2)

φ = resistance factor (dim) (5.7.3)

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$$T_{min} = \frac{T_A}{OL_A} \quad (5.4.1\text{-}2)$$

$$T_{min} = \frac{T_{cv}}{OL_{cv}} \quad (5.4.1\text{-}3)$$

where:

T_s = maximum torque required for starting (lb.-in.)

T_A = maximum torque required for acceleration (lb.-in.)

T_{cv} = maximum torque required for constant velocity (lb.-in.)

OL_s = starting overload factor specified in Table 1 (DIM)

OL_A = acceleration overload factor specified in Table 1 (DIM)

dim (search + replace throughout)

OL_{cv} = constant velocity overload factor specified in Table 1 (DIM)

When determining the maximum starting torque, T_s, and maximum constant velocity torque, T_{cv}, every possible bridge position shall be considered for both directions of travel.

When determining the maximum accelerating torque, T_A, the maximum acceleration time shall not be taken greater than ten seconds. Loading shall be equal to that of T_{cv}, but shall include inertial loads from the span and machinery.

Inertial loading of the span shall include the span and counterweight.

After determining T_{min}, the prime mover shall be selected at the lowest standard power, P_m (hp), rating such that:

- $FLT \geq T_{min}$, and
- $FLT = \frac{63{,}000 P_m}{n}$ (5.4.1-4)

where:

rpm (search + replace throughout doc.)

n = full-load speed (RPM)

P_m = standard motor size, power (hp)

Where consideration of ice accretion loads specified in various Articles herein is appropriate, the exposed area of

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speed. An 1800 RPM motor may develop rated power output at 1775 RPM, only 1.4 percent below synchronous speed.

Add new ¶

Past editions of AASHTO Specifications for movable bridges did not include inertia forces in T_s. At this writing (2006) some owners do include the inertia forces. The applicable provisions have been modified to permit a choice of including the inertia forces or not at the owners discretion.

The constant velocity torque (T_{cv}) will vary according to the position of the movable span. The movable span can be stopped at any position by the operator, therefore, every span position must be considered for T_s.

Machinery inertia includes all mechanical items and the motor rotor, if an electrical drive is used, which can be a substantial load.

At the time of this writing, 2006 ~~1998~~, most motor manufacturers in this Country are manufacturing motors to NEMA Standard MG 1. The motor frame dimensions and power are expressed in inches and horsepower, respectively. (Motor manufacturers reportedly will, upon special request, provide nameplates in metric units, by soft conversion of the power rating using a conversion factor of 0.75 kW/HP.

In some cases, metric motors can be acquired based on the European IEC standards. Some dimensional incompatibility should be anticipated where metric motors would be mounted on English-dimensioned machinery, such as integral-type gear motors and C-face or flange-mounted pumps or gear reducers.)

Ice accretion is considered in determining the power requirement. Power requirements lead to design loads for the structural supports for the machinery system via the

SPECIFICATIONS

the deck shall be taken as specified for wind loads in Article 2.4.1.3.1.

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machinery loads specified in Section 2 of these Specifications.

There are no general requirements for ice accretion loads on superstructures in the AASHTO LRFD Highway Bridge Design Specifications. Rather, ice accretion is treated as a site specific load in Article 3.9.6 therein. For most sites, it was considered that significant ice accretion on the superstructure would reduce the live load and dynamic load amplification due to the resulting poor driving conditions, i.e. there would be an offset in the loads. Thus ice accretion is generally not considered in the design of fixed bridge superstructures, and it follows that the same reasoning applies to the movable bridges when they are functioning as a fixed bridge.

Since ice accretion would not necessarily be inconsistent with the need to open the bridge, ice loads are applied, where appropriate, when determining power requirements and the structural ramifications thereof.

5.4.2 Bascule Spans

The maximum bridge starting torque, T_s, shall be determined using the friction coefficients for starting and neglecting inertia.

The following load conditions shall be used to size the prime mover:

- **Maximum Starting Torque (T_s)** - Shall be determined for span operation against static frictional resistance, unbalanced conditions specified in Article 1.5, a wind load of 10 psf on any vertical projection, and an ice loading of 2.5 psf on the area specified in Article 2.4.1.3.1.
- **Maximum Constant Velocity Torque (T_{cv})** - Shall be determined for span operation against dynamic frictional resistances, unbalanced conditions specified in Article 1.5, and a wind load of 2.5 psf acting normal to the floor on the area specified in Article 2.4.1.3.1.

C5.4.2

Bascule types using operation struts, control arm linkages, etc., must include their effect on imbalance which may significantly effect maximum span torque values and angles of occurrence.

T_s is usually maximum with the span near fully open.

T_{cv} is usually maximum with the span near fully closed.

and may at the owner's discretion include inertial resistance due to acceleration.

5.4.3 Swing Spans

The maximum bridge starting torque shall be determined using the friction coefficients for starting and neglecting inertia.

The following load conditions shall be used to size the prime mover:

- **Maximum Starting Torque (T_s)** - Shall be determined for span operation against static frictional resistances, a wind load of 10 psf on any vertical projection of the open bridge, and an ice loading of 2.5 psf on the area specified in Article 2.4.1.3.1. Provision shall

C5.4.3

Ice loading will add to the dead weight of the swing span and, therefore, the frictional torque will increase.

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also be made for a wind load of 10 psf on one arm and 5 psf on the other arm.

- **Maximum Constant Velocity Torque (T_{cv})** - Shall be determined for span operation against dynamic frictional resistances and a wind load of 2.5 psf acting horizontally against the vertical projection of one arm. For unequal arm swing spans the vertical projection of the longer arm shall be used. Under this loading, span operation shall occur in the normal time for operation.

Wind loading shall be applied to both arms and shall be considered as blowing in a single direction even when wind pressure varies between arms.

5.4.4 Vertical Lift Spans

The following load conditions shall be used to size the prime mover:

- **Maximum Starting Torque (T_s)** - Shall be determined for span operation against static frictional resistances, rope bending, unbalanced conditions specified in Article 1.5, a wind load of 2.5 psf on the area specified in Article 2.4.1.3.1 acting normal to the floor, and an ice loading of 2.5 psf on the area specified in Article 2.4.1.3.1 and may at the owner's discretion include inertial resistance due to acceleration.

- **Maximum Constant Velocity Torque (T_{cv})** - Shall be determined for span operation against dynamic frictional resistances, rope bending, unbalanced conditions specified in Article 1.5, and a wind load of 2.5 psf on the area specified in Article 2.4.1.3.1 acting normal to the floor. This wind loading shall be considered to include frictional resistances from span and counterweight guides caused by horizontal wind on the moving span.

5.5 HOLDING REQUIREMENTS

Machinery for holding the span against the loads and under the conditions specified herein may be proportioned for the overload limit state.

When bascule or swing spans are normally left in the open position, the span can be held in the fully open position against the wind loads specified in Article 2.4.1.3.1 by either:

- proportioning the machinery alone; or

- proportioning separate holding or locking devices, such that when combined with the holding capacity of the

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Table 5.8.2-1 - Friction Factors for Typical Bearings

Bearing Type	For Starting	For Motion
Plain Radial Type:		
For Trunnion or Journal Friction		
• Sleeve bearings, one or more complete rotations	0.14	0.09
• Sleeve bearings, less than one complete rotation	0.18	0.12
For linear sliding bearings	0.18	0.12
Plain Thrust Bearings:		
For friction on center disks	0.15	0.10
For collar friction at ends of conical rollers	0.15	0.10
Rolling Element Bearings and Rollers:		
Rolling element bearings	0.004	0.003
For rolling friction of bridges having rollers with flanges, or build-up segmental girders	0.009	0.006
X For rolling friction of solid cast rollers without flanges	~~0.063/√D~~	~~0.063/√D~~
where:		
X D = diameter of roller (in.)	0.0113/√D	0.0113/√D

move up

5.8.3 Wire Rope Bending Losses

For 180 degrees bending of wire ropes, for each sheave, the coefficient of direct tension in rope for starting and motion is 0.3 d/D.

where:

d = diameter of rope (in.)

D = diameter of sheave (in.)

5.8.4 Efficiency Factors for Gearing

5.8.4.1 OPEN SPUR GEARING

The efficiency of any pair of open spur gears, bearing friction not included, may be taken as 0.96.

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6.1 SCOPE

The provisions in this section apply to the design of the machinery for moving, aligning, and locking a movable bridge span. The section addresses the requirements for bascule, swing, and vertical lift movable spans.

6.2 DEFINITIONS

Addendum - Portion of gear tooth outside (greater than) the pitch radius.

AGMA - American Gear Manufacturer's Association

Allowable Static Design Stress - Permissible value of stress for calculations involving components subjected to static loading.

Average (mean) Stress - One-half of the sum of the maximum and minimum stress.

Backlash - The smallest amount of space between the faces of mating gears.

Bevel Gear - Type of gear commonly used when shafts intersect and utilizes the concept of rolling cones.

Brittle - Materials designed against ultimate strength for which failure means fracture; easily broken snapped or cracked.

Contact Stress Failure - Failure of gear teeth based on projected area of contact.

Crank Pins - Joint between linkages where stress alternates between application and release.

Cyclic Stress - Stress range which follows a pattern over and over.

Dedendum - Portion of gear tooth from the root to the pitch line.

Deflector Sheaves - Component used on span drive vertical lift bridges to guide operating ropes from the top chord (horizontal) to the tower attachments (vertical).

Diametral Pitch - Index of gear tooth sizes that is defined as the number of teeth divided by the pitch diameter (in.) $(in.^{-1})$

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Ductile - Materials designed against yield strength and failure is visible before fracture.

Enclosed Gearing - Gear set of which all moving elements are included in a given frame and cover that is dust-proof and oil-tight.

Endurance Limit Strength - Stress level at which completely reversing cyclic stress (fatigue) causes failure in one million (1×10^6) cycles; the ability to withstand fatigue loads.

Fatigue Failure - Point at which cyclic loading causes fracture or permanent deformation.

Fatigue Limit State - Limit state relating to cyclic stress and crack propagation.

Fatigue Strength - Ability to withstand cyclic loading.

Helical - A gear with a cylindrical pitch surface and teeth that are at an angle to the axis.

Herringbone - A type of helical gearing where half of the teeth are right-handed and the other half are left-handed.

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Idler Gear - A gear that has the same number of teeth as the mating gear and, therefore, introduces no change in shaft RPM, but changes the rotational direction.

L-10 Life - Basic rating life of a component for 90% reliability based on load and speed data, typically given as a number of revolutions.

Lay - The manner in which the wires in a strand or the strands in a rope are helically positioned.

Mechanical Shrink-fit Assembly - Mechanical connection where assembly is performed by heating or cooling one element relative to the other, and when an equilibrium temperature is reached, an interference fit is produced.

Minimum Yield Strength - The lowest value of stress a material shows a specified limiting deviation from the proportionality of stress to strain.

Module - Metric index of gear tooth sizes that is defined as pitch diameter (mm) divided by number of teeth.

Open Gearing - Gear set that is not sealed and may have moving elements exposed to the environment.

Pitting Resistance/Wear/Surface Durability - AGMA terms used for rating gearing from the aspect of contact surface stress.

Service Limit State - Limit state relating to stress, deformation and cracking applied to normal operating loads.

Sheave - A pulley or wheel having a grooved rim, typically used for wire ropes on vertical lift bridges.

Spur Gear Teeth -Teeth on the cylindrical pitch surface of a gear that are parallel to the axis.

Stress Range - Maximum stress minus the minimum stress.

Uniaxial Tensile Stress - Stress acting along only one axis.

Yield Failure/Intermittent Overload - Overload condition for which yield failure may occur in spur gear teeth experiencing less than 100 cycles in its design life.

6.3 NOTATION

6.3.1 General

A	=	constant cross-sectional area (in.^{2}); projected area (in.^{2}) (6.6.1) (6.6.2.5)
a	=	constant for finding surface roughness factor (DIM) (6.6.3.2)
$\sqrt{a}$	=	Neuber constant for finding q (notch sensitivity) (in.$^{0.5}$) (6.7.3.2)
B	=	exponent quantity (6.7.5.2.2)
b	=	exponent for finding surface roughness factor (DIM); pinion/gear face width (mm) key width (rectangular or square) (in.) (6.6.3.2) (6.7.10.1)
$^{\circ}C$	=	degrees Celsius (6.7.5.2.2)
C_D	=	endurance limit modifying factor based on diameter (DIM) (6.6.3.2)
C_f	=	gear design surface finish factor (DIM) (6.7.5.2.3)
C_H	=	gear design hardness ratio factor (DIM) (6.7.5.2.3)
C_M	=	endurance limit modifying factor for miscellaneous condition (DIM) (6.6.3.2)
C_{or}	=	basic static load rating, element bearing (lb.) (6.7.7.2.4)
C_p	=	gear design elastic coefficient (psi$^{-0.5}$) (6.7.5.2.3)
C_R, C_S, C_T	=	reliability, surface, temperature factors, respectively (DIM) (6.6.3.2)
C_r	=	basic dynamic load rating, rolling element bearing (lb.) (6.7.7.2.2)

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X

(psi$^{0.5}$)

SPECIFICATIONS COMMENTARY

d = pitch diameter of pinion (in.); diameter of the wire rope (in.) (6.7.5.2.1) (C6.8.3.3.4)
D = diameter for use in various sliding bearings (in.); rolling bearings (in.); shaft diameter (in.); tread diameter of sheave rope grooves (in.) (6.6.2.5)&(6.7.7.1.2) (6.6.2.6) (6.6.3.2) (6.7.4.3) (6.8.3.3.4)
d_w = diameter of outer wires in the wire rope (in.) (6.8.3.3.4)
E_R = tensile modulus of elasticity of the wire rope (psi) (C6.8.3.3.6)
E_w = tensile modulus of elasticity of the steel wire (psi) (6.8.3.3.4)
°F = degrees Fahrenheit (6.7.5.2.2)
F = pinion/gear face width (in.) (6.7.5.2.2)
F_{oxy} = factored large bearing load (lb.) (6.7.7.2.4)
F_{ua} = applied axial load (lb.) (6.7.7.2.2)
F_{ur} = radial load (lb.) (6.7.7.1.2)
G = gear for J and I tables (DIM) (C6.7.5.2.2)
g = acceleration due to gravity = 32.2 (ft./s.^{2}) (6.7.4.3)
H_B = Brinell hardness number (BHN) (DIM) (6.6.4.2)
H_{BP}, H_{BG} = Brinell hardness number for pinion/gear (DIM) (6.7.5.2.3)
h = key height (in.) (6.7.10.1)
h_t = total gear tooth height (in.) (6.7.5.2.2)
I = gear design tooth geometry factor - surface durability (DIM) (6.7.5.2.3)
J = gear design tooth geometry factor - fatigue (DIM) (6.7.5.2.2)
K_B = gear design rim factor (DIM) (6.7.5.2.2)
K_F = fatigue stress concentration factor (normal stress) (DIM) (6.7.3.2)
K_{FS} = fatigue stress concentration factor (shear stress) (DIM) (6.7.3.2)
K_f = gear design stress correction factor (DIM) (6.7.5.2.4)
K_m = gear design load distribution factor (DIM) (6.7.5.2.2)
K_{my} = gear design load distribution factor for overload (DIM) (6.7.5.2.4)
K_o = gear design overload factor (DIM) (6.7.5.2.2)
K_R = gear design reliability factor (DIM) (6.7.5.2.2)
K_S = gear design tooth size factor (DIM) (6.7.5.2.2)
K_t = theoretical stress concentration factor (normal stress) (DIM) (6.7.3.2)
K_T = gear design temperature factor (DIM) (6.7.5.2.2)
K_{ts} = theoretical stress concentration factor (shear stress) (DIM) (6.7.3.2)
K_v = gear design velocity (dynamic) factor (DIM) (6.7.5.2.2)
K_y = gear design yield strength factor (DIM) (6.7.5.2.4)
k = radius of gyration (in.) (6.6.1)
L = length of shaft between supports (in.) (6.7.4.2)
L_{act} = actual length (in.) (6.6.1)
L_{eff} = effective length (in.) (6.6.1)
M_a = bending moment amplitude (lb.-in.) (6.7.4.1)
m_B = gear design backup ratio (DIM) (6.7.5.2.2)
m_t = ~~gear design tooth module transverse (6.7.5.1)~~
N = gear design number of load cycles (DIM) (6.7.5.2.2)
Ω = rotational speed (RPM); rotational speed of bearing inner race (RPM) (6.6.2.5) (6.7.7.2.2)
Ω_c = critical shaft rotational speed (RPM) (6.7.4.3)
Ω_p = pinion speed (RPM) (C6.7.5.2.1)
N_p = gear design number of pinion teeth (DIM) (6.7.5.2.2)
n_S = static design factor (DIM) (6.6.1)
P = power which the gear transmits (hp); pinion for J and I tables; direct load on wire rope (lb.) (C6.7.5.2.1) (C6.7.5.2.2) (6.8.3.3.4)
P_{ac} = allowable transmitted power for gear design surface durability (hp) (C6.7.5.2.1)
P_{at} = allowable transmitted power for gear design fatigue (hp) (C6.7.5.2.1)
P_d = diametral pitch (in.$^{-1}$) (6.7.5.1)
P_o = operating loads, e.g., the larger of starting or inertial loads (lb.) (6.8.3.3.4)
P_{or} = factored radial design resistance (lb.) (6.7.7.2.4)
P_r = equivalent dynamic radial load for rolling element bearings (lb.) (6.7.7.2.2)

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(lb)(6.6.1);

P_{ut} = wire rope minimum ultimate breaking load (lb.) (6.8.3.3.6)
p = allowable static design resistance in compression; bearing design pressure (psi) (6.6.1) (6.7.7.1.2)
Q_v = gear quality number (DIM) (6.7.5.2.2)
q = fatigue design notch sensitivity factor (DIM) (6.7.3.2)
R_a = surface arithmetic average roughness (μin.) (6.6.3.2)
R_b = bearing resistance (lb.) (6.6.2.5)
R_R = line bearing resistance on rollers (lb./in.) (6.6.2.6)
r = notch/fillet radius or radius of a hole (in.) (6.7.3.2)
S_{at} = gear design allowable stress - fatigue (psi) (6.6.4.2)
S_{ac} = gear design allowable stress - surface durability (psi) (6.6.4.3)
S_{ay} = gear design allowable yield (psi) (6.6.4.4)
S_F = safety factor for bending strength (DIM) (6.7.5.2.2)
S_H = surface durability design factor (DIM) (6.7.5.2.3)
T = torque, torsional moment (lb.-in.) (6.7.5.2.1)
T_m = mean-steady torque (lb.-in.) (6.7.4.1)
t_R = rim thickness (in.)(6.7.5.2.2)
V = velocity; journal surface speed (fpm) (6.7.7.1.2)
v_t = gear design pitch line velocity (fpm) (6.7.5.2.2)
W = weight (lb.) (6.7.4.3)
W_{max} = maximum peak transmitting gear force (lb.) (6.7.5.2.4)
W_t = tangential transmitting gear force (lb.) (6.7.5.2.1)
W_{tac} = factored surface durability resistance of spur gear teeth (lb.) (6.7.5.2.3)
W_{tat} = factored flexural resistance of spur gear teeth (lb.) (6.7.5.2.2)
X, X_o = bearing design factors (radial) (DIM) (6.7.7.2.2), (6.7.7.2.4)
Y, Y_o = bearing design factors thrust (axial) (DIM) (6.7.7.2.2), (6.7.7.2.4)
Y_N = gear design life factor - fatigue (DIM) (6.7.5.2.2)
Z_N = gear design life factor (DIM) (6.7.5.2.3)
α = factor for allowable bearing resistance (lb./in.•RPM); factor for allowable line bearing resistance (lb./in.); endurance limit factor (DIM) (6.6.2.5), (6.6.2.6), (6.6.3.2)
σ = normal stress (psi) (6.7.2.3)
σ'_a, σ'_m = Von Mises stresses (psi) (6.7.3.3.2)
σ_a = amplitude stress - fluctuating (psi) (6.7.3.3.1)
σ_b = maximum wire rope bending stress (psi) (6.8.3.3.4)
σ_e = endurance limit (psi) (6.6.3.2)
σ_m = mean or average stress (psi (6.7.3.3.1)
σ_{max}, σ_{min} = maximum and minimum cyclic stress (psi) (6.7.2.2) (6.7.3.3.1)
σ_t = maximum total stress in wire rope (psi) (6.8.3.3.4)
σ_{ut} = ultimate tensile strength (psi) (6.6.3.2)
$\sigma_y, \sigma_{yt}, \sigma_{yc}$ = yield strength of material - min. (psi) (C6.6.1)
τ = shear stress (psi) (6.7.2.3)
τ_a, τ_m = amplitude and mean cyclic shear stresses (psi) (6.7.3.3.1)
τ_{max} = maximum shear stress resulting from applied loads (psi) (6.7.2.2)

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6.4 GENERAL REQUIREMENTS

6.4.1 Machinery

6.4.1.1 LIMIT STATES AND RESISTANCE FACTORS

Unless otherwise stated, machinery design shall be based on the service and fatigue limit states using the loads and resistances specified herein.

C6.4.1.1

The design of bridge machinery in the United States is based on allowable working stress design, therefore, this section follows the accepted industry design practice. As

6.3 NOTATION

Insert the following additional notation in the proper order:

φ = resistance factor (dim) (6.4.1.1)

SPECIFICATIONS | COMMENTARY

6.6 RESISTANCE OF MACHINERY PARTS

6.6.1 Resistance at the Service Limit State

C6.6.1

For commonly used materials, resistance values shall be computed using allowable static stresses in Table 1 which include the factors of safety and unsupported length provisions specified herein as applicable.

Table 6.6.1-1 - Allowable Static Stresses in psi

Material	AASHTO	ASTM	Tension	Compression	Fixed Bearing	Shear
Structural Steel	M 183	A 36 /A36M	12,000	12,000–55 L_{eff}/k	16,000	6,000
Forged Carbon Steel (except keys)	M 102	A 668 (CL D) /668M	15,000	15,000–65 L_{eff}/k	18,000	7,500
Forged Carbon Steel (keys)	M 102	A 668 (CL D) /668M	-	-	15,000	7,500
Forged Alloy Steel	M 102	A 668 (CL G) /668M	16,000	16,000–70 L_{eff}/k	21,000	8,000
Cast Steel	M 103	A 27 /A27M (~~GR 485-250~~)	9,000	10,000–45 L_{eff}/k	13,000	5,000
Cast Steel	-	A 148 /A148M (~~GR 620-415~~)	5,000	15,000–65 L_{eff}/k	21,000	7,500
Bronze	M 107	B 22 ALLOY UNS C90500, C91100, C91300, C93700	7,000	7,000	-	-
Hot Rolled Steel Bar	M 255	A 675 /A675M (GR 515)	12,000	12,000–55 L_{eff}/k	16,000	6,000

(GR 75)

For materials not included in Table 1, resistance shall be determined by applying the remaining provisions of this article.

The minimum static design resistance at the service limit state shall be determined by applying the following factors of safety, n_S, to minimum tensile yield:

- Forged, drawn, rolled, wrought steel.................$n_S = 3$
- Cast steel ....$n_S = 4$

The static shear resistance shall be based upon one-half the allowable tensile design resistance.

(GR 70-36)

(GR 90-60)

SPECIFICATIONS

For crank pins and similar joints with alternating application and release of stress, the bearing resistance determined as specified above may be doubled.

6.6.2.4 INTERMEDIATE SPEEDS

For intermediate motion at speeds exceeding 50 fpm, bearing resistance shall be determined using the gross projected bearing area and shall be based on the stresses specified below:

- Shaft journals, rolled or forged steel on AASHTO M 107 (ASTM B 22 Alloy UNS C93700) bronze .... 600 psi
- Thrust collars, rolled or forged steel on AASHTO M 107 (ASTM B 22 Alloy UNS C93700) bronze .... 200 psi

The provisions of Article 6.6.2.5 shall not be exceeded.

6.6.2.5 HEATING AND SEIZING

The maximum allowable bearing resistance, R_b (lb.), shall be taken as:

$$R_b = a\left(\frac{A}{nD}\right) \qquad \alpha \qquad (6.6.2.5\text{-}1)$$

where:

- A = planar or projected area over which the load is acting (in.^{2})
- n = rotational speed - revolutions per minute (RPM)
- D = diameter of journal or step bearing, or mean diameter of collar or screw (in.)
- α = a factor specified herein (lb./in.·RPM)

The factor α shall be taken as follows:

- For sleeve bearing shaft, journals rolled or forged steel on bronze 250,000
- For step bearings, hardened steel on bronze 60,000
- For thrust collars, rolled or forged steel on bronze 50,000

COMMENTARY

C6.6.2.4

The maximum journal surface velocity (fpm) is different for various plain bearing materials. Most plain journal bearings are intended for slow speed applications only. Use rolling element bearings for most intermediate and high speeds, based on journal surface velocity.

C6.6.2.5

The purpose of this provision is to avoid heating and seizing on sleeve bearing shaft journals, step bearings for vertical shafts, thrust collars, and Acme thread power screws.

At high rotational speeds, rolling element bearings are strongly preferred. Use of plain bearings in this type of application can lead to excessive wear and shortened life.

SPECIFICATIONS

- For acme screws, rolled or forged steel on bronze 220,000

6.6.2.6 BEARING ON ROLLERS

The maximum allowable line bearing resistance, R_R per unit width (lb./in.) on rollers used in steel-steel interfaces shall be taken as:

$$R_R = \alpha\left(\frac{\sigma_y - 13{,}000}{20{,}000}\right) \qquad (6.6.2.6\text{-}1)$$

where:

σ_y = minimum yield strength of the weaker material (psi)

D = diameter of the roller (in.)

α = factor specified herein (lb./in.)

	Diameters Up to 25 in.	Diameters 25 to 125 in.
For rollers in motion:		
	400 D	$2{,}000\sqrt{D}$
For rollers at rest:		
	600 D	$3{,}000\sqrt{D}$

For manufactured trunnion and counterweight sheave roller bearings, the provisions of Article 6.7.7.2.4 shall apply.

COMMENTARY

C6.6.2.6

The guidelines in this article are typically used for special steel rollers designed for the following uses: swing span rim bearing rollers, swing span balance wheels (center bearing), end lift rollers or rockers, span guide rollers, counterweight guide rollers, etc.

Equation 1 is based on steel-steel interface only. Use of Equation 1 for other materials would be erroneous.

For steel and most other ductile materials, the yield strength in compression is the same as the yield strength in tension.

6.6.3 Design for the Fatigue Limit State

6.6.3.1 GENERAL

The loads generated from operation at full load torque or normal operating pressure shall be used unless otherwise specified.

For steel parts other than spur gears subjected to cyclic stresses, when the expected number of load cycles is expected to be more than one million, design shall be based on the endurance limit specified herein.

C6.6.3.1

From field measurements of several movable bridges, it has been found that the operating machinery constant velocity running loads usually range from 40 percent to 70 percent of full-load torque. Where the movable span is likely to be subject to frequent operation under heavy wind or ice loading, a load factor of up to 1.5 may be applied.

Where mechanical components are subjected to high numbers of cyclic stresses during the expected life of the part, designing statically may not be adequate, and a component may fail by fatigue.

SPECIFICATIONS

For operating ropes, the respective corresponding limits shall be taken as 30 percent and 16.7 percent.

6.7 MECHANICAL MACHINERY DESIGN

6.7.1 General

Machinery shall be designed for loading from:

- the prime mover as specified in Article 5.7,
- holding as specified in Article 5.5, and
- braking as specified in Article 5.7.3.

6.7.2 Requirements for Design with Static Stresses

6.7.2.1 GENERAL

These provisions shall be taken to apply at the service limit state.

Stresses which are cyclic and changing shall not be treated as static unless the total number of stress cycles during the life of the part are less than about 10,000 cycles, or the range of stress is small in comparison to the average stress.

Determination of resistance based on allowable static stresses is specified in Article 6.6.1.

Where a component does not satisfy the criteria above, it shall be designed for the fatigue limit state as specified in Article 6.6.3.

6.7.2.2 UNIAXIAL NORMAL STRESS AND SHEAR STRESS

Where components comprised of ductile materials are subjected to static, steady loads, resistance to static uniaxial tension or bending shall satisfy the following:

$$\sigma_{max} \leq \frac{\sigma_{yt}}{n_s} \qquad (6.7.2.2\text{-}1)$$

where:

σ_{yt} = specified minimum tensile yield stress (psi)

σ_{max} = maximum normal stress resulting from the applied loads (psi)

n_s = static factor of safety (DIM)

COMMENTARY

C6.7.2.1

Static stresses are stresses that are uniform, usually caused by uniform loads acting on the stationary component.

C6.7.2.2

It is not necessary to include stress concentration factors in the stress calculation when the failure mode is yielding.

For the values for n_s refer to Article 6.6.1.

SPECIFICATIONS

Where components comprised of ductile materials are subjected to static loads of direct shear or pure torsion, but are not subjected to axial or bending loads, resistance shall satisfy:

$$\tau_{max} \le \left(\frac{\sigma_{yt}}{2n_s}\right) \tag{6.7.2.2-2}$$

where:

τ_{max} = maximum shear stress resulting from the applied loads (psi)

6.7.2.3 COMBINED STRESSES

Resistance of components subjected to simultaneous loads producing uniaxial normal stress and shear stress shall satisfy:

$$\tau_{max} \le \frac{\sigma_{yt}}{2n_s} \tag{6.7.2.3-1}$$

in which:

$$\tau_{max} = \sqrt{\left(\frac{\sigma}{2}\right)^2 + \tau^2} \tag{6.7.2.3-2}$$

where:

σ = applied uniaxial normal stress due to loads producing tension, bending, or both (psi)

τ = applied shear stress due to loads producing torsion, direct shear, or both (psi)

6.7.3 General Requirements for Design with Fluctuating Stresses at the Fatigue Limit State

6.7.3.1 GENERAL

The fatigue limit state shall be considered where a machinery component is subjected more than about 10,000 cycles of stress during the component's lifetime.

COMMENTARY

C6.7.2.3

The most common case of combined stresses in machinery components is a combination of uniaxial normal, σ, and shear, τ, stresses, where the normal stress is caused usually by bending or axial loads and the shear stresses caused by torsion or direct shear.

For ductile materials and combined stresses, one of the most commonly used theories of failure by yielding is the maximum shear stress theory.

This is the maximum shear stress from Mohr's circle, due to combined stresses.

C6.7.3.1

For an indefinite life, and for steel components, it is possible to design parts that will not experience fatigue failure during its lifetime. The design process uses the modified endurance limit, σ_e as defined in Article 6.6.3.2.

SPECIFICATIONS

COMMENTARY

6.7.3.3 FATIGUE DESIGN

6.7.3.3.1 Mean and Amplitude Stresses

The provisions of this section require consideration of mean stresses, σ_m and τ_m, and amplitude stresses, σ_a and τ_a, which shall be determined as:

$$\sigma_a = \left(\frac{\sigma_{max} - \sigma_{min}}{2}\right) \qquad (6.7.3.3.1\text{-}1)$$

$$\sigma_m = \left(\frac{\sigma_{max} + \sigma_{min}}{2}\right) \qquad (6.7.3.3.1\text{-}2)$$

$$\tau_a = \left(\frac{\tau_{max} - \tau_{min}}{2}\right) \qquad (6.7.3.3.1\text{-}3)$$

$$\tau_m = \left(\frac{\tau_{max} + \tau_{min}}{2}\right) \qquad (6.7.3.3.1\text{-}4)$$

where:

σ_{max} = maximum applied normal stress (psi)

σ_{min} = minimum applied normal stress (psi)

τ_{max} = maximum applied shear stress (psi)

τ_{min} = minimum applied shear stress (psi)

C6.7.3.3.1

Stresses may vary during a cyclic stress condition from some minimum to some maximum stress, whether normal or shear stresses.

6.7.3.3.2 Fatigue Failure Theory

Components subjected to loads producing, both uniaxial normal stresses and shear stresses, shall satisfy:

$$\left(\frac{\sigma'_a}{\sigma_e} + \frac{\sigma'_m}{\sigma_{yt}}\right) \leq 0.80 \qquad (6.7.3.3.2\text{-}1)$$

in which:

$$\sigma'_a = \sqrt{(K_F \sigma_a)^2 + 3(K_{FS} \tau_a)^2} \qquad (6.7.3.3.2\text{-}2)$$

$$\sigma'_m = \sqrt{(K_F \sigma_m)^2 + 3(K_{FS} \tau_m)^2} \qquad (6.7.3.3.2\text{-}3)$$

C6.7.3.3.2

The fatigue failure theory presented here will use the Soderberg failure criteria, which is a conservative theory that uses the endurance limit σ_e and the material tensile yield strength (minimum), σ_{yt}.

Another fatigue theory of failure, which is less conservative, known as the nominal mean stress method, uses K_F and K_{FS} only with the alternating, amplitude stresses, and not with the mean stresses.

When dealing with combined cyclic fluctuating stresses, both uniaxial normal (usually bending) and shear (usually torsion), it is necessary to calculate the Von Mises stress, given by Equations 2 and 3, which is an equivalent normal stress that is used in the fatigue design equations.

The terms σ'_a and σ'_m are the amplitude and mean Von Mises stresses, respectively.

With these equations, the fatigue stress concentration factors, K_F and K_{FS} are used both with the amplitude stresses and the mean stresses.

SPECIFICATIONS

where:

σ_a = amplitude normal stress specified in Article 6.7.3.3.1 (psi)

σ_e = endurance limit of a steel shaft specified in Article 6.6.3.2 (psi)

σ_m = mean normal stress specified in Article 6.7.3.3.1 (psi)

τ_a = amplitude shear stress specified in Article 6.7.3.3.1 (psi)

τ_m = mean shear stress specified in Article 6.7.3.3.1 (psi)

K_F = stress concentration factor for fluctuating normal stress specified in Article 6.7.3.2 (DIM)

K_{FS} = stress concentration factor for fluctuating shear stress specified in Article 6.7.3.2 (DIM)

6.7.4 Shafts, Trunnions, Machine Elements Subjected to Cyclic Stresses

6.7.4.1 SHAFT AND TRUNNION DIAMETER

Unless specified otherwise by the Owner, the design of shafts, trunnions, and other machinery parts subjected to more than 1 million cycles of reversed bending moment due to rotation in combination with a steady torsional moment shall satisfy:

$$\frac{32}{\pi d^3}\left[\frac{K_F M_a}{\sigma_e} + \frac{\sqrt{3}K_{FS}T_m}{2\sigma_{yt}}\right] \le 0.8 \qquad (6.7.4.1\text{-}1)$$

where:

K_F = fatigue stress concentration factor (bending) (DIM)

K_{FS} = fatigue stress concentration factor (torsion) (DIM)

M_a = amplitude bending moment (lb.-in.)

T_m = mean (steady) torsional moment (lb.-in.)

σ_e = endurance limit of the steel shaft specified in Article 6.6.3.2 (psi)

σ_{yt} = minimum tensile yield strength of the steel shaft (psi)

The diameter of shafts used for transmitting power for

COMMENTARY

C6.7.4.1

The previous editions of the AASHTO Movable Bridge Specifications were essentially devoid of any reference to the possibility of fatigue failure of shafts, trunnions, or similar machinery parts that are subjected to high numbers of stress cycles during their life, that can lead to failure especially at locations of high stress concentration.

For a shaft or trunnion of multiple diameters, it is necessary to analyze all crucial cross-sections.

For trunnion type bascule bridges, the trunnions experience a single one-way bending cycle for each complete bridge operation. Therefore, Equation 1 may be taken as:

$$\frac{32}{\pi d^3}\left[\frac{K_F M_a}{\sigma_e} + \frac{\sqrt{3}K_{FS}T_m}{2\sigma_{yt}}\right] \le \qquad (C6.7.4.1\text{-}1)$$

SPECIFICATIONS

- if the minimum gear hub thickness is less than 40 percent of the shaft diameter.

6.7.5 Design of Open Spur Gearing

6.7.5.1 GENERAL

The equations specified herein apply only to full depth spur gear teeth.

Unless specified otherwise, gear teeth:

- shall be machine cut,
- shall be of the involute type,
- shall have a pressure angle of 20 degrees.

For spur gear pitch diameter tooth speeds over 600 fpm and where quiet operation is desired, an enclosed helical gear speed reducer should be considered.

Unless otherwise specified, all gear teeth shall be cut from solid rims. For open spur gears, the AGMA gear quality shall be Class 7 or higher and the backlash shall be as established by AGMA based on center distance and diametral pitch (Pd). P_d

X

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). P_d

The face width of a spur gear should be not less than 8/Pd nor more than 14 Pd (not less than 8, nor more than 14, times the tooth module). P_d

X X

The diametral pitch of spur gears shall not be less than: e

X

- for pinions other than motor pinions, transmitting power for moving the span .... 3.14 in. — 3.14 in.$^{-1}$
- for motor pinions .... 4.19 in. — 4.19 in.$^{-1}$
- for main rack teeth .... 2.09 in. — 2.09 in.$^{-1}$

COMMENTARY

Minimum gear hub thickness for gears with keyways shall be taken as the minimum length between the keyway and root of the teeth.

C6.7.5.1

The use of stub teeth is not recommended, however, AGMA does cover unequal addendum tooth systems. The tooth geometry factors, used for both fatigue and surface durability, for unequal addendum tooth systems are different than those presented below, under Articles C6.7.5.2.2 and C6.7.5.2.3. See AGMA 908-B89.

Spur gear design is based on diametral pitch, which is defined by the number of teeth on the gear divided by pitch diameter (in.). For large teeth (Pd<1), circular pitch is commonly used instead of diametral pitch. X

P_d ← d is subscript

SPECIFICATIONS

COMMENTARY

AGMA presents design/analysis equations ANSI/AGMA 2001-C95) to determine the safe power that can be transmitted by the gear teeth, either safe power based on fatigue strength, P_{at} (hp), or safe power based on pitting resistance/wear/ surface durability, P_{ac} (hp).

These equations have been modified into equations to find the factored flexural resistance, W_{tat}, and factored pitting resistance, W_{tac}.

6.7.5.2.2 Design for the Fatigue Limit State

Compliance with Equation 1 is intended to prevent fatigue failure of the gear teeth.

The following must then be satisfied:

$W_t \leq W_{tat}$ (6.7.5.2.2-1)

The factored flexural resistance, W_{tat} (lb.) of the spur gear teeth, based on fatigue, shall be determined as:

$$W_{tat} = \frac{F J S_{a} Y_N}{P_d K_o K_v K_s K_m K_B S_F K_T K_R} \qquad (6.7.5.2.2\text{-}2)$$

JS_{at}

in which:

$$K_v = \left[\frac{A + \sqrt{V_t}}{A}\right] \qquad (6.7.5.2.2\text{-}3)$$

$$A = 50 + 56\,(1.0 - B) \qquad (6.7.5.2.2\text{-}4)$$

$$B = 0.25\,(12 - Q_v)^{0.667} \qquad (6.7.5.2.2\text{-}5)$$

✓

$$v_t = p\,n_p \frac{d}{12} \qquad (6.7.5.2.2\text{-}6)$$

π ← Due to conversion problem?

where:

K_v = dynamic factor (DIM)

F = tooth face width of the spur pinion or gear that is being analyzed/designed (in.)

P_d = diametral pitch taken as N_p/d (in.$^{-1}$)

v_t = pitch line velocity (fpm)

d = pitch diameter of the pinion (in.)

N_p = number of teeth on the pinion

C6.7.5.2.2

Tables C1 and C2 provide values of the geometry factor, J for values of 18, 19, 20, and 21 tooth pinions, for 20° full depth, equal addendum teeth only. (Modified from AGMA 908-B89)

For a gear quality of 6 or 7 use Table C2 for J and tooth loading at the tip; use the highest single tooth contact Table C1 for J only if the gears are of high quality and accurately aligned at assembly.

Note that the factor K_V is now greater than 1. Previous editions of the AGMA Standards used K_V as less than 1.

SPECIFICATIONS

S_{at} = allowable bending stress specified in Article 6.6.4.2 (psi)

K_o = overload factor taken as > 1.0 where momentary overloads up to 200 percent exceed 4 in 8 hours, and exceed one second duration (DIM)

H_B = Brinell hardness for the teeth (DIM)

N = number of load cycles

n_p = pinion (RPM)

Q_V = gear quality number taken as an integer between 7 and 12 (DIM)

K_S = tooth size factor to reflect nonuniformity of tooth material properties, due to large tooth size, gear diameter, and face width taken as >1 (DIM)

K_m = load distribution factor taken as $K_m = 1.21 + 0.0259F$ for open gearing, adjusted at assembly, with $F < 28$ in., and $F/d < 1$ (DIM)

K_B = rim thickness factor taken as 1.0 if $m_B = t_R / h_t > 1.2$ (DIM)

m_B = the backup ratio (DIM)

t_R = rim thickness (in.)

h_t = total tooth height (in.)

S_F = safety factor for bending strength (fatigue) $S_F \geq 1.2$ (DIM)

J = geometry factor for bending strength (DIM)

Y_N = life factor for bending resistance taken as (DIM):

- For $H_B \approx 250$ and $10^3 < N < 3 \times 10^6$

$$Y_N = 4.9404\ N^{-0.1045} \quad (6.7.5.2.2\text{-}6)$$

6.7.5.2.2-7

- For $N > 3\text{x}10^6$ $_{load}$ cycles, regardless of hardness

$$Y_N = 1.6831\ N^{-0.0323} \quad (6.7.5.2.2\text{-}7)$$

6.7.5.2.2-8

X

COMMENTARY

Refer to the AGMA Standards for a definition of the Gear Quality Number. The accuracy of a gear increases with an increase of the quality number. Therefore, tighter manufacturing tolerances must be met and thus may increase cost. However, the production methods and tooling of many gearing manufacturers is such that the minimum quality number they will produce may be Q_v = 9 or higher. In such cases, the designer may have little or no cost savings in specifying a lower quality number.

AGMA gives no further guidance on K_S, however other references recommend using K_S of 1.2 to 1.5 for large tooth size (say $P_d < 2.5$). (Norton, 1998; Shigley, 1983)

This is an approximate equation, derived from AGMA Standard 2001-C95. See this or the latest AGMA standard for a more accurate calculation of K_m.

A good design guideline is to have m_B > 1.2.

See Tables C6.7.5.2.2-1 and C6.7.5.2.2-2 for suggested values of J.

SPECIFICATIONS

COMMENTARY

Table C6.7.5.2.2-2 - J Factor for 20° Full Depth Spur Pinion/Gear (P, G)

Equal Addendum Loaded at Tooth Tip								
GEAR TEETH	PINION TEETH (N_p)							
	18		19		20		21	
	P	G	P	G	P	G	P	G
18	0.23	0.23	-	-	-	-	-	-
19	0.23	0.23	0.23	0.23	-	-	-	-
20	0.23	0.24	0.23	0.24	0.24	0.24	-	-
21	0.23	0.24	0.23	0.24	0.24	0.24	0.24	0.24
26	0.23	0.25	0.23	0.25	0.24	0.25	0.24	0.25
35	0.23	0.26	0.23	0.26	0.24	0.26	0.24	0.26
55	0.23	0.28	0.23	0.28	0.24	0.28	0.24	0.28
135	0.23	0.29	0.23	0.29	0.24	0.29	0.24	0.29

6.7.5.2.3 Surface Durability and Wear - Design Equations

C6.7.5.2.3

Compliance with Equation 1 is intended to promote surface durability and pitting resistance.

The following must then be satisfied: ✓ Due to conversion?

X

$W_t £ W_{tac}$ ≤ (6.7.5.2.3-1)

The factored surface durability resistance, F_{taz} (N), of the spur gear teeth based on pitting resistance shall be determined as:

$$W_{tac} = \frac{F d I}{K_o K_v K_s K_m C_F} \left(\frac{S_{ac} Z_N C_H}{C_p S_H K_T K_R} \right)^2 \quad (6.7.5.2.3\text{-}2)$$

where:

F = tooth face width of the pinion or gear that has the narrowest face width (in.)

S_{ac} = allowable contact stress for the lower Brinell hardness number of the pinion/gear pair as specified in Article 6.6.4.3 (psi)

N = number of load cycles (DIM)

(Small f subscript) f

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SPECIFICATIONS

S_H = safety factor for pitting resistance, i.e., surface durability taken as >1 (DIM)

$\underline{C_p}$ = elastic coefficient taken as 2,300 for steel pinion-steel gear (psi)$^{0.5}$

Z_N = stress cycle factor for pitting resistance taken as 2.466 $\underline{N}^{-0.056}$ for $10^4 < \underline{N} < 10^{10}$ (DIM)

$\underline{C_H}$ = hardness ratio factor for pitting resistance, taken as 1.0 if $H_{BP}/H_{BG} < 1.2$ (DIM)

$\underline{I}$ = geometry factor for pitting resistance (DIM)

$\underline{C_f}$ = surface condition factor for pitting resistance taken as 1.0 for good tooth surface condition as specified in Article 6.7.8 for tooth surface finish depending on module (DIM)

diametral pitch

COMMENTARY

Table C1 gives values of $\underline{I}$ modified from AGMA 908-B89, $\underline{I}$ geometry factor tables - values for 18, 19, 20, and 21 tooth pinions, for 20° full depth, equal addendum teeth only.

Table C6.7.5.2.3-1 - $\underline{I}$ Factors for 20° Full Depth Spur Pinions/Gears

Equal Addendum Factors same for both Pinion and Gear				
GEAR TEETH	PINION TEETH ($\underline{N_p}$)			
	18	19	20	21
	P | G	P | G	P | G	P | G
18	0.075	-	-	-
19	0.077	0.076	-	-
20	0.079	0.078	0.076	-
21	0.080	0.080	0.078	0.078
26	0.084	0.084	0.084	0.084
35	0.091	0.091	0.091	0.091
55	0.100	0.101	0.102	0.102
135	0.112	0.114	0.116	0.118

Usually the hardness of the gear is lower than that of the pinion. H_{BP} and H_{BG} are the Brinell hardness of the pinion and gear, respectively. The H_{BP}/H_{BG} ratio will usually be less than or equal to 1.2. For example, it is common to have the pinion hardness equal to 350 BHN and the gear equal to 300 BHN, for a ratio of 1.17.

For other material combinations, i.e., steel-cast iron or steel-bronze, refer to the AGMA Standards.

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recessed into the base or threaded dowels and with double hexagonal nuts. The nuts shall bear on finished bosses or spot-faced seats.

Where it is obvious that aligning and adjustment will be necessary during erection, provisions shall be made for the aligning of bearings by means of shims, and for the adjustment of the caps by means of laminated liners or other effective devices.

Large bearings shall be provided with effective means for cleaning lubrication passages without dismantling parts. Jacking holes shall be provided between machinery bearing caps and bases to facilitate maintenance.

The shaft (journal) should be specified to be at least 100 BHN points harder than the metallic bearing material.

This requirement is specified because of the variability of the hardness of metallic bearing materials.

Thrust loads shall be absorbed by using thrust flanges on the bearing, or by thrust collars or thrust washers.

6.7.7.1.2 Plain Bearing Design Equations

Plain cylindrical bearings, i.e., sleeve bearings, that are boundary lubricated shall be sized based on three main parameters: pressure, surface velocity of journal, determined as indicated below, and the product of the two.

$$p = \frac{F_{ur}}{(DL)} \qquad (6.7.7.1.2\text{-}1)$$

π

$$V = \frac{pDn}{12} \qquad (6.7.7.1.2\text{-}2)$$

where:

F_{ur} = applied radial load (lb.)

p = pressure (psi)

V = surface velocity (fpm)

D = diameter of the journal (bearing I.D.) (in.)

L = length of the bearing (in.)

n = journal rotational speed (RPM)

Where better data is not available, the maximum values for p, V and pV for various commonly used bronze bearing alloys may be taken from Table 1.

C6.7.7.1.2

It is common practice to reduce the projected area, D x L, by about 5 percent if grease grooves are present, unless a more accurate projected area is known.

Radial bearing wear is directly related to the product of pV whereas bearing life is indirectly related to pV. Refer to bearing manufacturers as the factors used to determine bearing life vary significantly with material, whether the material is metallic or nonmetallic, the type and method of lubrication, and contamination of the lubricant.

The relationship between D and L is generally that the length, L, should usually be between 100 percent and 150 percent of the diameter, D.

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Table C6.7.7.1.4a-1 - Performance Parameters for Oil-Impregnated Metals

MATERIAL	ASTM NO.	p (psi)	V (fpm)	pV (psi·fpm)
Oilite Bronze	B-438-73 Gr 1 Type II	2,000	1,200	50,000
Super Oilite	B-439-70 Gr 4	4,000	225	35,000
Super Oilite - 16	B-426 Gr 4 Type II	8,000	35	75,000

6.7.7.1.4b NonMetallic Bearings

Plastic bearing materials, such as nylons, acetal resins (Delrin), TFE fluorocarbons (Teflon), PTFE, and fiber reinforced variations of these materials may be used where conditions permit.

C6.7.7.1.4b

As a general guide to the important properties of "plastic" bearings and other plastic parts, refer to ASTM D 5592 "Standard Guide for Material Properties Needed in Engineering Design Using Plastics."

Refer to manufacturers of "plastic" bearings for detailed information on the allowable p, V, and pV values, and any particular design methods. The properties of some nonmetallic bearing materials are given in Table C1.

Table C6.7.7.1.4b-1 - Performance Parameters for Nonmetal Bearings

MATERIAL	p (psi)	V (fpm)	pV (psi·fpm)
Acetal ("Delrin")	1,000	1,000	3,000
Nylon	1,000	1,000	3,000
Phenolics	6,000	2,500	15,000
TFE ("Teflon")	500	50	1,000
PTFE Composite	10,000	150	25,000

6.7.7.2 ROLLING ELEMENT BEARINGS

6.7.7.2.1 General

Rolling element bearings shall be designed at the service limit state so that the L-10 life shall be 40,000 hours at the average running speed of the bearing and shall also be designed at the overload limit state and shall satisfy:

$$P \; {}^{3} \; F \qquad (6.7.7.2.1\text{-}1)$$

C6.7.7.2.1

The life of rolling element bearings is based on the L-10 life which is defined by the American Bearing Manufacturers Association as the life for which 90 percent of a group of identical bearings will survive under a given equivalent radial load (i.e.,10 percent failures).

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reputation who has had bearings of comparable size of the same materials and type in successful service for at least ten years.

6.7.7.2.4 Sizing of Large Rolling Element Bearings

For slow rotation large rolling element bearings, or those that do not complete one full revolution during its loading cycle, sizing should be based on C_{or}, the basic static radial load rating of the bearing.

The factored radial design resistance, P_{or} (lb.) for a bearing subjected to static loads and low speeds shall be determined as:

$$P_{or} = \frac{C_{or}}{n_s} \qquad (6.7.7.2.4\text{-}1)$$

where:

- For counterweight sheave trunnion bearings of vertical lift bridges, the static design factor should be taken as .... $n_S = 5$
- For bascule spans with a fixed trunnion, the static design factor should be taken as .... $n_S = 8.5$

For a combination of the applied radial and axial loads acting on the bearing, the factored bearing load, F_{oxy} (lb.), shall be determined as:

$$F_{oxy} = X_o F_{ur} + Y_o F_{ua} \qquad (6.7.7.2.4\text{-}2)$$

where:

F_{ur} = applied radial load (lb.)

F_{ua} = applied axial load taken as not less than 15 percent of F_{ur} (lb.)

X_o = a static axial load factor (DIM)

Y_o = a static radial load factor (DIM)

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C6.7.7.2.4

Large rolling element bearings generally have a bore larger than 4 in., and a rotational speed less than 5 RPM.

It is necessary to work closely with bearing manufacturers on the large rolling element bearings. The specific operating parameters may necessitate special lubricating or other requirements, especially in applications such as a bascule trunnion bearing whose inner race operates at less than one quarter revolution each cycle.

As an example, one manufacturer of large spherical roller bearings, for the 232 Series, uses $X_o = 1$ and $Y_o \approx 2$. (Y_o varies from about 1.75 for a 14 in. to about 2.06 for a 36 in. bore bearing in this series.)

Therefore, if the axial force is 15 percent of the radial force, this gives $F_{oxy} \approx 1.3\,F_{ur}$. For a vertical lift span, the required static radial load rating would be:

$$P_{or} = \frac{C_{or}}{5} \qquad (C6.7.7.2.4\text{-}1)$$

then:

$$\frac{C_{or}}{5} \geq 1.3F_{ur} \quad \text{or} \quad C_{or} \geq 6.5F_{ur}$$

The values of X_o and Y_o are values that depend on bearing size and type, and manufacturer.

≥

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6.7.8 Fits and Finishes

The fits and surface finishes for machinery parts, specified in Table 1, are in accordance with ANSI B4.1, Preferred Limits and Fits for Cylindrical Parts, and ASME B46.1, Surface Texture.

Fits other than the preferred fits listed in this ANSI Standard may be used.

Surface finishes are given as the arithmetic average roughness height (R_a) in microinches; if additional limits are required for waviness and lay, they shall be specified.

The fits for cylindrical parts, specified in Table 1, shall also apply to the major dimensions of noncylindrical parts.

Table 6.7.8-1 - Fits and Finishes

PART	FIT	FINISH R_a(μin.)
Machinery base on steel	--	250
Machinery base on masonry	--	500
Shaft journals	RC6	8
Journal bushing	RC6	16
Split bushing in base	LC1	125
Solid bushing in base (to ¼ in. wall)	FN1	63
Solid bushing in base (over ¼ in. wall)	FN2	63
Hubs on shafts(to 2 in. bore)	FN2	32
Hubs on shafts (over 2 in. bore)	FN2	63
Hubs on main trunnions	FN3	63

COMMENTARY

C6.7.8

Roughness height does not define waviness or flatness, which is a separate criteria. A surface may be "smooth", but be wavy or warped, therefore, not a proper flat surface to mount machinery to. 1

Fits other than those listed in Table 6.7.8-1 may be used at the discretion of the Engineer.

The range of fits given for hubs on main trunnions allows the designer some flexibility to provide the most appropriate fit for the particular detail.

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vertical shafts, collars clamped about the shafts, or similar devices shall be used.

In hubs of spoked wheels, the keyways shall be located in the centers of the spokes.

6.7.10.2 CAPACITY OF KEYS

Keys used to transmit loads generated by the prime mover from a shaft to another component, i.e., gears, couplings, etc. shall have sufficient resistance to develop the full torsional strength of the shaft.

If two keys are required, they shall be placed 120° apart. When using two keys, each key shall be capable of

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Table C6.7.10.1-1 - Key Sizes

DIAMETER (in.) OVER-TO-INCLUDE		KEY SIZE (in.) (b x h)	
5/16	7/16	3/32 · 3/32	
7/16	9/16	1/8 · 1/8	
9/16	7/8	3/16 · 3/16	
7/8	1 1/4	1/4 x 1/4	
1 1/4	1 3/8	5/16 · 5/16	
1 3/8	1 3/4	3/8 x 3/8	
1 3/4	2 1/4	1/2 x 1/2	
2 1/4	2 3/4	5/8 x 5/8	
2 3/4	3 1/4	3/4 x 3/4	
3 1/4	3 3/4	7/8 x 7/8	
3 3/4	4 1/2	1 x 1	
4 1/2	5 1/2	1 1/4 x 1 1/4	1 1/2
5 1/2	6 1/2	32 x 18	
6 1/2	7 1/2	1 3/4 x 20	1 1/2
7 1/2	9	2 x 1 1/2	
9	11	2 1/2 x 1 3/4	
11	13	3 x 2	
13	15	3 1/2 x 2 1/2	
15	18	4 x 3	
18	22	5 x 3 1/2	
22	26	6 x 4	
26	30	7 x 5	

C6.7.10.2

It is desired to have the shaft fail before the key(s) since signs of distress in the shaft will normally be more visible. Key failure may more easily lead to an uncontrolled situation, but yield failure of the shaft may have a higher tendency to misalign components and jamb in place.

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normal allowable unit stresses for any linkage or mechanism may be increased 50 percent.

Hand brakes and foot brakes should be arranged so that the brake is applied by means of a weight or spring and released manually.

6.7.14 Machinery Support Members and Anchorage

6.7.14.1 MACHINERY SUPPORTS

In the design of structural parts subject to loads from machinery or from forces applied for moving or stopping the span, due consideration shall be given to securing adequate stiffness and rigidity and the avoidance of resonance. Beams subject to such stresses should have a depth not less than 1/8 of their span. If shallower beams are used, the section shall be increased so that the deflection will not be greater than if the above limiting depth had not been exceeded. Deflections shall be investigated sufficiently to insure that they will not interfere with proper machinery operation.

6.7.14.2 ANCHORAGE

Anchor bolts or other anchorages that resist uplift shall be designed to provide at least 1½ times the uplift and to support that force at the allowable stress.

6.7.15 Fasteners, Turned Bolts, and Nuts

All bolts for connecting machinery parts to each other or to supporting steelwork should be high-strength bolts conforming to AASHTO M 164 (ASTM A 325), or ASTM A 490.

SAE grade bolts may be used in place of AASHTO or ASTM grades at the discretion of the Engineer. In no case shall less than SAE Grade 5 be used.

Where specified, all turned bolts shall have turned shanks, semi-finished, washer-faced, hexagonal heads, and rolled, cut, or ground threads. The finished shanks shall be ¹⁄₁₆ in. larger in diameter than the major diameter of the thread. The bolt blank size, usually ⅛ in. larger than the thread size, shall determine the head dimensions.

The dimensions of all bolt heads shall be in accordance with the ANSI heavy hex structural bolt series specified in ANSI B18.2.6, and threads shall be in accordance with the ANSI coarse thread thread series specified in ANSI B1.1.

Turned bolts shall be fitted in reamed holes, to an LC6 fit.

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For metric classes

C6.7.15

The use of ASTM A 325, Type 2, or ASTM A 490, Type 2, low carbon martensitic steel, is not recommended in any applications where the connection is subjected to fluctuating loads or the possibility of impact loadings.

Refer to ASTM F 568 for a description of the ASTM property classes.

For details of SAE fasteners, refer to SAE J1199. J429

Table C1 lists standard bolt sizes.

Rolled threads are preferred because this threading process provides the lowest stress concentrations in the thread roots.

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The dimensions of all hex nuts shall be in accordance with the ANSI heavy hex nut series specified in ANSI B18.2.6, and threads shall be in accordance with the ANSI coarse thread series specified in ANSI B1.1. Nuts shall conform to the requirements of ASTM A563.

Hardened steel washers shall be in accordance with ASTM F 436.

All bolt heads and nuts shall bear on seats square with the axis of the bolt. On castings, except where recessed, the bearing shall be on finished bosses or spot-faced seats. Bolt heads which are recessed in castings shall be square. All nuts shall be secured by effective locks. If double nuts are used, both nuts shall be of standard thickness.

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Table C6.7.15-1 - Standard Bolt Sizes, Thread Pitch, Tensile Stress Areas

MAJOR DIA. D (in.)	THREADS PER INCH n ($in.^{-1}$)	MINOR DIA. (in.)	TENSILE STRESS AREA A_t ($in.^2$)
⅝	11	0.5135	0.226
¾	10	0.6273	0.334
⅞	9	0.7387	0.462
1	8	0.8466	0.606
1⅛	7	0.9497	0.763
1¼	7	1.0747	0.969
1⅜	6	1.7105	1.155
1½	8	1.2955	1.405

1.1705

6

X

The values listed in Table C2 for proof and tensile loads are from ASTM. For ASTM A 325 bolts, the proof loads are based on a strength of 85,000 psi (½ to 1, incl.) and 74,000 psi (1⅛ to 1½ incl.); the tensile loads are based on a minimum tensile strength of 120,000 psi (½ to 1, incl.) and 105,000 psi (1⅛ to 1½, incl.). For ASTM A 490 bolts, the proof loads are based on a strength of 120,000 psi; the tensile loads are based on a minimum tensile strength of 150,000 psi.

Table C6.7.15-2 - Proof and Tensile Loads

DIA×TPI	A 325		A 490	
	PROOF (lb.)	TENSILE (lb.)	PROOF (lb.)	TENSILE (lb.)
⅝-11	19,200	27,100	27,100	33,900
¾-10	28,400	40,100	40,100	50,100
⅞-9	39,250	55,450	55,450	69,300
1-8	51,500	72,700	72,700	90,900
1⅛-7	56,450	80,100	91,550	114,450
1¼-7	71,700	101,700	116,300	145,350
1⅜-6	85,450	121,300	138,600	173,250
1½-6	104,000	147,500	168,600	210,750

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6.8.3.3.3 Lay

Unless otherwise specified, all wire ropes shall be right regular lay, and the maximum length of rope lay shall be taken as:

- Operating ropes: 6.75 times nominal rope diameter.
- Counterweight ropes: 7.5 times nominal rope diameter

The lay of the wires in the strands shall be such as to make the wires approximately parallel to the axis of the rope where they come in contact with a circular cylinder circumscribed on the rope.

6.8.3.3.4 Wire Rope Stresses

Where a wire rope is bent over a sheave, the maximum bending stress, σ_b in psi, on the rope may be conservatively determined as:

$$\sigma_b = E_w \frac{d_w}{D} \quad (6.8.3.3.4\text{-}1)$$

where:

d_W = diameter of the outer wires in the wire rope (in.)

D = tread diameter of sheave rope grooves (in.)

E_W = tensile modulus of elasticity of the steel wire = 30 × 10^6 psi

The maximum total stress in the rope, σ_t in psi, may be determined as:

$$\sigma_t = \frac{P}{A} + \sigma_b + \frac{P_o}{A} \quad (6.8.3.3.4\text{-}2)$$

where:

P = direct load on the ropes (lb.)

A = effective cross-sectional area of the ropes (in.^{2})

P_o = operating loads, e.g., the larger of starting or inertial loads (lb.)

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C6.8.3.3.4

The maximum bending stress occurs on the outer-most wires at the least bending radius.

For rope of the 6 x 19 class - 6 strands of 19 main wires each, $d_W \approx d / 16$

therefore:

$$\sigma_b = \frac{1.875 \times 10^6}{D/d} \quad (C6.8.3.3.4\text{-}1)$$

where:

d = diameter of the wire rope (in.)

When determining P_o for counterweight ropes, only inertial loads are effective.

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The maximum total stress shall not exceed the allowable tensile stresses specified in Article 6.6.5.

6.8.3.3.5 Short Arc of Contact

When operating ropes have less than a 45 degree arc of contact with a deflector sheave, the minimum sheave diameter shall be at least 26 times the wire rope diameter.

6.8.3.3.6 Wire Rope Tensile Strengths

The minimum tensile breaking resistance, P_{ut}, for the 6x19 class (6x25 FW), fiber core wire ropes, both improved plow steel (IPS) and extra improved plow steel (EIPS), based on WRTB and manufacturers' specifications shall be taken as specified in Table 1.

Table 6.8.3.3.6-1 - Physical Properties of Rope

D (in.)	WEIGHT / LENGTH (lb./ft.)	P_{ut} (lb.) IPS	EIPS
¾	0.095	47,600	52,400
⅞	1.29	64,400	70,800
1	1.68	83,600	92,000
1⅛	2.13	105,200	115,600
1¼	2.63	129,200	142,200
1⅜	3.18	155,400	171,000
1½	3.78	184,000	202,000
1⅝	4.44	214,000	236,000
1¾	5.15	248,000	274,000
1⅞	5.91	282,000	312,000
2	6.72	320,000	352,000
2⅛	7.59	358,000	394,000
2¼	8.51	400,000	440,000
2⅜	9.48	–	488,000
2½	10.5	–	538,000

COMMENTARY

C6.8.3.3.5

Where a rope is in contact with a small deflector sheave over a short arc, taken as 45 degrees or less, the actual radius of curvature of the rope is usually larger than the deflector sheave radius.

C6.8.3.3.6

FW = Filler wire construction, total of 25 wires per strand, 19 main wires, and 6 filler wires.

WRTB Wire Rope Users Manual, 1993.

EIPS with fiber core is not listed in WRTB manual.

Values are based on a 9 percent increase in tensile strength over the IPS fiber core.

Table 1 is for bright, uncoated wire rope. For galvanized wire rope, refer to specific manufacturer's specifications. Galvanized wire rope typically has about a 10 percent lower breaking strength than the above values.

To find the ultimate tensile strength, σ_{ut} in psi, divide the values of the tensile breaking resistance, P_{ut} in lb., in the table by the area of the wire rope in in.^{2}:

$$\sigma_{ut} = \frac{P_{ut}}{A} \qquad \text{(C6.8.3.3.6-1)}$$

in which:

$A = 0.417\, d^2$

for 6 x 25 FW wire rope with a fiber core.

The elongation of wire rope under load may be determined using the following E_R values :

E_R (psi)	Percent of Ultimate Load
10.8×10^6 (74 500)	0-20
12×10^6 (83 000)	21-65

and the equation:

$$\delta = \frac{PL}{AE_R} \qquad \text{(C6.8.3.3.6-2)}$$

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M 222 (M 222M), or M 223 (M 223M) or forged carbon steel, AASHTO M 102, Class D (ASTM A 668, Class D, S4).

The resistance of the sheave shall be such that the stresses under dead load with the sheave rotating and with impact do not exceed:

- tension or compression in base metal or filler metal: 10,000 psi with stress range not to exceed 10,000 psi,
- shear in base metal or filler metal: 5,000 psi with stress range not to exceed 5,000 psi,

The contract documents shall specify that:

- the rim be fabricated from not more than three pieces of plate and stiffened by transverse ribs if necessary to carry the load,
- the rim be welded into a complete ring and the welds ground flush on all four sides before being welded into the sheave assembly,
- each web be fabricated from not more than two pieces of plate,
- web welds, if used, shall be ground flush on both sides,
- the hub shall be made from a one-piece forging,
- all welds shall be full penetration welds made with low hydrogen procedures,
- automatic submerged-arc welding shall be used to the greatest extent practicable,
- after completion of the weldment and before final machining, the sheave be stress relieved, and
- unless otherwise specified, the sheave assembly shall be stress relieved by heat treatment prior to final machining.

The contract documents shall specify that:

- the grooves be accurately machined to insure uniformity of the pitch diameter for all of the grooves, and
- the pitch diameter variation shall not exceed plus or minus 0.01 in.

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connections. Protective fill and vent seal units shall be included to prevent accidental vapor ignition. A day-tank, including pumps, shall be provided for engines over 60 hp. The installation shall be in accordance with the requirements of the National Fire Protection Association.

A small control board containing throttle and choke controls, ignition switch, starter button, and oil and temperature gages shall be provided at the engine.

The engine shall be enclosed in readily removable metal housing unless located in a protected space, and together with reversing gears and all other engine accessories, shall be mounted in the shop on a rigid steel frame so as to form a complete engine unit ready for installation.

Indicators shall be provided in the engine room to show the position of the moving span and, if so specified, of the lifting and locking apparatus.

If cold ambient temperatures may affect starting reliability, a water jacket heater or other suitable means to warm the fuel shall be provided. Protective features shall include low oil pressure cut-out, high water temperature cut-out, engine overspeed shutdown, and overcranking protection if applicable.

The contract documents shall specify the type and quantity of spare parts for engines to be furnished.

On all bridges operated by engines, means shall be provided for interlocking the span movement with operation of the locks and wedges so that power cannot be applied to the span until locks or wedges are released. For swing spans, such interlocking between span and lock mechanisms can generally be accomplished by means of mechanical trips which will allow the gears to be engaged only in proper sequence.

When engines for span or lock operation are used in conjunction with electrically operated lights, gates or other safety devices, interlocking shall be provided which will not permit the locks to be retracted until the safety devices are in operation, nor permit the safety devices to go out of operation until the span is seated and the locks reseated.

Means shall be provided for by-passing the interlocking system in an emergency.

6.9.2 Manual Operation

6.9.2.1 GENERAL

Provision for manual operation in case of power failure shall be provided on all brakes, locks, gates and barriers. Interlock with the span mechanisms shall be provided to prevent bridge openings before gate closure.

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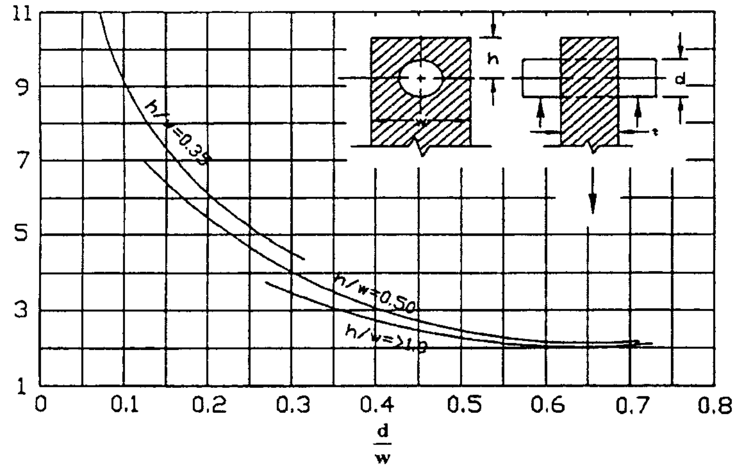
Such interlocking can generally be accomplished by magnetically actuated trips and relays which will allow the switches and gear to be engaged only in proper sequence. Interlocking by connection to the ignition system of the engine will generally be unsatisfactory.



$K_t \cong A\left(\frac{d}{w}\right)^b$

where:

$\frac{h}{w}$	*A*	*b*
1.00	1.15	–0.9723
0.50	1.537	–0.7923
0.35	2.071	–0.6447

Figure A6.2-9: Geometric Stress Concentration Factor K_t for a Plate Loaded in Tension by a Pin Through a Hole (Shigley, 1989).

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7.2 DEFINITIONS

Accumulator - An energy storage device for storing hydraulic fluid under pressure. The energy absorbing mechanism may be a spring, external weight, or an inert gas with a precharge pressure.

Beta Ratio - A measure of the effectiveness of filters.

Closed Loop - A hydraulic circuit in which the pump output, after passing through an actuator, returns directly to the pump inlet.

Component - An individual unit comprising one or more parts designed to be a functional part of a fluid power system, e.g., cylinder, valve, filter, excluding piping.

Counter Balance Valves - Valves specifically intended to retard actuator movement by providing back pressure on the downstream side of the actuator during overhauling load situations.

Design Working Pressure - The established criteria for maximum working pressure allowed by design.

DIN - Deutsches Institut fur Normang e.V. (German Standards Institute)

Hydraulic Regenerative Braking - A method of energy absorption whereby fluid power flow is reversed and directed toward the system hydraulic pumps. This energy is then absorbed by resistance to speed change in the prime mover.

Maximum Working Pressure - The highest pressure at which the system or part of the system is intended to operate in steady-state conditions without amplification due to impact; a physically established value - controlled and limited by physical devices such as relief valves (see Article 7.6.2).

Normal Working Pressure - The pressure at which a system or part of the system is intended to operate in steady-state conditions without amplification due to impact as established by the design setting of a relief valve. Normal working pressure differs from maximum working pressure in that it is established by setting an adjustable relief valve to a specific pressure lower than the maximum pressure setting. If a non-adjustable relief valve is used, the normal working pressure and maximum working pressure will be the same for the part of the system whose pressure is controlled by that valve.

Open Loop - A hydraulic circuit design in which fluid is drawn from a reservoir, routed through an actuator, and returned, at low pressure to the reservoir.

Overall Efficiency - The total efficiency of a component of a system; the product of the mechanical efficiency and the volumetric efficiency.

Rated Pressure - The highest pressure at which the component is intended to operate for a number of repetitions sufficient to ensure adequate service life.

7.3 NOTATION

A	=	additional thickness allowance to compensate for material removed during threading; to provide for mechanical strength of the conductor; and to provide for corrosion and/or erosion (in.) (C7.6.9.2)
D	=	diameter of rod (in.) (7.5.12.3)
D_o	=	outside diameter of pipe or tube (in.) (C7.6.9.2)
E	=	modulus of elasticity (psi) (7.5.12.3)
Eff_{ov}	=	overall efficiency (DIM, decimal equivalent) (7.5.2.2)
I_{rod}	=	moment of inertia of the rod ($in.^4$) (7.5.12.3)
I_{shell}	=	moment of inertia of the cylinder body ($in.^4$) (7.5.12.3)

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K = effective length factor (DIM) (7.5.12.3)
L_{eff} = $L_{total} \cdot K$ (in.) (7.5.12.3)
L_{rod} = the length of the rod (in.) (7.5.12.3)
L_{shell} = the length of the cylinder body (in.) (7.5.12.3)
L_{total} = length of cylinder between points of attachment on support (in.) (7.5.12.3)
P = pressure (psi); internal design pressure (maximum working pressure) (psi) (7.5.2.2) (C7.6.9.2)
P_E = Euler buckling load (lb.) (7.5.12.3)
P_{hp}
P_{kw} = power (hp) (7.5.2.2)
Q = flow (gpm) (7.5.2.2)
SE = allowable stress in material due to internal pressure and joint efficiency at the design temperature (psi) (C7.6.9.2)
t_m = minimum wall thickness of the pipe or tube (in.) (C7.6.9.2)
y = coefficient for pipe geometry that varies with temperature (DIM) (C7.6.9.2)

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7.4 DESIGN LOADING CRITERIA

7.4.1 Power Requirements for Hydraulic System Design

Except as noted within this section, power requirements for hydraulic system design shall be taken as specified for machinery design in Section 5. Prime movers for span drive hydraulic systems shall be sized to satisfy the provisions of Article 7.5.2.

C7.4.1

The prime mover of a hydraulic system is the pump motor, the electric motor which imparts energy to the pump.

7.4.2 Machinery Design Criteria and Limit States

Machinery actuated by hydraulic power shall be designed at the service limit state using resistances specified in Section 6 for the following loads:

- Hydraulic motor torques at the Normal Working Pressure neglecting motor efficiency.
- Hydraulic cylinder forces at the Normal Working Pressure of the circuit which actuates the cylinder, neglecting cylinder efficiency.

Machinery actuated by hydraulic power shall also be designed for the overload limit state using resistances specified in Section 6 for the following loads:

- Hydraulic motor torques at the maximum working pressure neglecting motor efficiency.
- Hydraulic cylinder forces at the maximum working pressure of the circuit which actuates the cylinder, neglecting cylinder efficiency.

For seismic design at the extreme event limit state, the provisions of Article 6.4.1 shall apply.

7.3 NOTATION

Insert the following additional notation in the proper order:

X_I = buckling length reduction factor (dim) (7.5.12.3)

SPECIFICATIONS

COMMENTARY

Designs and details shall consider containment of leaks and spills.

7.5.2 Electric Motors

7.5.2.1 GENERAL

All specified motors shall comply to NEMA dimensional standards, NEMA Design B motor guidelines, and ANSI/NEMA MG 1 specifications for Motors and Generators.

Electrical motors equal to or greater than 1 hp used for driving hydraulic pumping equipment shall be specified as chemical duty rated, 1,800 nominal RPM, three phase, 480 VAC, TEFC, squirrel cage induction motors. Motors rated below 1 hp may be specified as single phase, capacitor start at the discretion of the Engineer.

In computing the power requirements of electric motors, the pressure, P, shall be taken as the pressure at the pump outlet. This pressure shall be computed taking into account all pressure drops in the circuit at the corresponding flow rate at the pump. Pressure drops for components shall be taken from manufacturer's data for typical components. If the pressure drop of a particular component is assumed, the anticipated pressure drop should be specified in the contract documents.

C7.5.2.1

In sizing electric motors for driving hydraulic pumps, consideration should be given to the following, in addition to the power required to operate the span under constant velocity conditions:

- Charge pump or control pressure pump power requirements.
- Minimum motor size to start and drive large pumps.

7.5.2.2 OPEN LOOP SYSTEMS

Electric motors for operation of pumps in open loop applications shall be sized to provide 120 percent of the power required of the pump under constant velocity conditions. Calculation of motor power requirements, P_{hp}, shall include the maximum fluid power, i.e. flow times the pressure at the pump, plus pump and motor efficiencies and be taken as:

$$P_{hp} = \frac{QP}{Eff_{ov}\, 1{,}715} \qquad (7.5.2.2\text{-}1)$$

where:

P_{hp} = power (hp)

Q = flow in (gal./min.) (gpm)

P = pressure in (psi)

Eff_{ov} = overall efficiency (DIM, decimal equivalent of percentage)

C7.5.2.2

The 20 percent extra power requirement is to account for future changes to components within the circuit and field adjustment of counterbalance valves.

SPECIFICATIONS

control desk electrically connected to system. All modes of operation in which the system is designed for operation shall also be demonstrated.

During functional testing, the contract documents shall further require the Contractor to record system pressures, flows, and operating times from the hydraulic system during all modes of operation. This data shall be submitted to the engineer for approval prior to system acceptance.

7.9 MATERIALS

7.9.1 Hydraulic Plumbing

7.9.1.1 PIPE AND PIPE FITTINGS

All hydraulic piping material shall be specified as seamless, low carbon stainless steel conforming to ASTM A 312, TP 316 or TP 304. Pipe shall be specified as pickled, cleaned and capped before shipping.

Pipe fitting materials shall be specified as similar to the pipes in which they are fitted. Acceptable welded pipe fittings shall be 37 degree flare type or SAE straight thread for conductors sizes up to and including 1.5 in. nominal. Mating 37 degree surface shall have an O-ring and O-ring boss specified to provide a leak-free connection. For nominal sizes of 1 in. to 1.5 in., 37 degree flare fittings shall be specified to contain a soft metallic washer for a leak-free connection. This washer shall be in lieu of an O-ring and O-ring boss. All connections involving piping over 1.5 in. shall be specified as butt welded or welded four-bolt flange utilizing captive O-ring pressure seal system connection. Specifications for flange dimensions shall conform to SAE J518 JUN93 standards. No pipe thread shall be permitted on any portion of the hydraulic system where continuous or intermittent pressures could exceed 200 psi. Where pipe threads are allowed by the criteria above, no pipe sealant shall be permitted.

Material for all associated hardware required for fittings shall be specified as similar to fitting material. Flange bolts shall be provided with locking washers and be of size and material to fit application.

7.9.1.2 TUBING AND TUBE FITTINGS

All hydraulic tubing material shall be specified as seamless, annealed, low carbon stainless steel conforming to ASTM A 269, TP304, ISO 10763:1994, and ANSI B31.1-2004(2001 out now) standards. Maximum tubing shall be specified as no larger than 1.5 in. nominal. For conductor requirements greater than 1.5 in., tubing shall not be considered.

COMMENTARY

active heat removal systems on hydraulic equipment should be demonstrated.

For staged construction, the designer should specify additional test periods between stages. This will reduce the possibility of disruption of traffic (vehicular or marine) by demolishing an existing bridge prior to fully verifying the operation of a new span.

SPECIFICATIONS

for mounting motors, cast feet shall be specified in lieu of stamped or bent metal plate motor mounts.

Where motors are to be installed in exposed, wet, or damp locations, the motors shall be specified to be weatherproof, with stainless steel shafts, when available.

Motors rated for 60 minutes or continuous duty shall be either totally enclosed, or installed in weather protected climate-controlled houses, in which case squirrel cage motors may be drip-proof. Motors rated for 30 minute duty cycle shall be totally enclosed. Motors rated for running times less than 30 minutes shall not be used.

Totally enclosed motors shall be specified with drain holes in bottom of frame and frame heaters where practical.

All DC motors and AC wound rotor motors shall be specified with totally enclosed frames. Drip-proof frames shall not be used for any DC or AC wound rotor motors in any environment.

than 1.0 HP

Motors ~~of~~ 0.9 kW (1.2 HP) ~~and~~ larger shall be specified with ball or roller bearings. Where available, the motor bearings shall be specified to be regreasable and provided with grease fittings suitable to the owner, and vent plugs for purging the grease.

~~1.2 hp~~

COMMENTARY

Because of the sometimes severe vibration encountered on bridges, the heavy-duty construction used for crane and hoist or mill duty motors is essential. Historically, cast-frame motors, and cast mounting feet seem to be more robust and better suited to the rough vibration and corrosive environment of typical movable bridges.

The dampness and abrasive grit and dust often encountered on movable bridges are detrimental to motor life and reliability, hence the requirement for totally enclosed motors. DC motors and AC wound rotor motors are especially vulnerable to increased maintenance if their brushes and commutators or slip rings are exposed to such an environment. However, these motors have proven to be very reliable in bridge drive service when furnished with totally enclosed frames.

Totally enclosed nonventilated (TENV) frames have traditionally been used with very good success on movable bridges, in lieu of totally enclosed fan-cooled (TEFC) or drip-proof. This is because the running times are short. This is very intermittent operation according to the motor industry's standards, even on a busy bridge. Hence 30 minute or 60 minute rated TENV motors are the norm on movable bridges.

8.5.2 Application-Specific Criteria

8.5.2.1 GENERAL

Electric motors for use on movable bridges are considered in two main categories, according to the following applications:

- Span Drive Motors, including skew control, or synchronizing motors
- Ancillary Device Motors, i.e., span locks, brakes, wedges, hydraulic pumps, air compressors, etc.

8.5.2.2 SPAN DRIVE MOTORS

Electric motors used as the prime mover for driving the movable span(s) shall be one of the following three types:

- AC Squirrel Cage Induction Motor
- AC Wound Rotor Induction Motor
- DC Motor

C8.5.2.2

Squirrel cage motors are quickly becoming popular as movable bridge drive motors, particularly when used with flux vector drives. Only inverter duty or flux vector duty rated motors should be used in these applications, due to the historically high failure rate of the windings insulation of standard AC squirrel cage motors with these drives.

Wound rotor motors are still popular and have proven themselves historically to be very rugged and reliable motors when used with stepped resistance control or variable voltage thyristor drives.

SPECIFICATIONS

COMMENTARY

Stainless steel or bronze wire mesh cable grips shall be specified wherever strain relief to the flexible cables is required, such as where they enter terminal cabinets.

8.9.6 Aerial Cables

The contract documents shall specify products and materials for aerial cables which will be suitable for the anticipated environmental condition and the anticipated service life. Aerial cables shall be rated for long-term exposure to various weather conditions such as ice, wind, rain, and direct sunlight.

Aerial cables shall be round, covered with a heavy-duty jacket which is resistant to sunlight, weather and aging, such as black Neoprene, or polyethylene.

Each electrical cable shall be attached to a messenger cable at intervals of not more than 2 ft. All messenger cables shall be of high-strength corrosion resistant material and shall be adequately anchored.

Messenger cables, cable hangers, and all accessories shall be corrosion resistant. The messenger cable shall be suspended with sag as required to safely support the entire assembly under various conditions of wind, ice, and temperature appropriate for the location of the bridge without exceeding the safe working tension of the messenger.

The amount of cable sag shall be determined and specified by the designer for each particular installation.

C8.9.6

Aerial cables are generally stationary overhead cables between two fixed points.

Aerial cables are intended to be messenger supported. Clearances, loadings and messenger strength are governed by the NESC.

For examples of standard industry practice, refer to sag and tension calculations in any recent edition of the Standard Handbook for Electrical Engineers by Fink and Carroll, McGraw Hill Book Company.

Use appropriate ice and wind loading for the locality as given in the NESC.

8.9.7 Submarine Cables

Submarine cables shall be specified as rated for long-term underwater installations and suitable for use in wet or dry locations which may be exposed to direct sunlight.

General configuration of the submarine cables shall consist of stranded copper conductors insulated with crosslinked polyethylene (90°C), cabled with fillers as necessary, binder tape, polyethylene inner jacket, polyethylene coated spiral-wound galvanized steel armor wires, separator tape, and a polyethylene outer jacket.

All cables shall be manufactured in accordance with ICEA S-66-524, NEMA WC-7-1988, as applicable.

C8.9.7

Unless otherwise specified, the cables should be placed at least 5 ft. below the bed of the channel. The U. S. Army Corps of Engineers and the U. S. Coast Guard may impose greater depth requirements, which should be confirmed for each bridge site.

Cables shall be long enough to provide sufficient slack for terminating.

Industry standards rate cable insulation using the SI system.

ICEA S-95-658/NEMA WC-70-1999

8.9.7.1 CONDUCTORS

Wires shall be annealed uncoated or tinned copper in accordance with ASTM B 3. Conductors shall be stranded in accordance with ASTM B 8, Class "B" stranding.

Conductor insulation shall be a chemically crosslinked polyethylene, XLPE.

The thickness of the conductor insulation shall comply with ICEA S-95-658/NEMA WC-70.

C8.9.7.1

ICEA S-66-524 revision has deleted color coding methods and tables.

SPECIFICATIONS

Color coding of insulated conductors shall be accomplished by surface printed legends consisting of numbers and color designations in accordance with ICEA S-73-532, NEMA WC57-1990 1995 ~~Method No.3, and Table E1 or E-5, as required.~~ .

The contract documents shall require that conductor identification color and print be legible after normal handling during installation.

8.9.7.2 CABLE CONSTRUCTION

Cable components shall be assembled into a tight concentric configuration. The direction of lay for adjacent layers of cabled conductors shall be reversed. Maximum lay lengths and lay directions shall conform to <u>ICEA S-95-658/NEMA WC-70</u>.

Fillers shall be employed as necessary within the cabled core to produce a substantially circular cross-section. Fillers shall be nonhygroscopic polypropylene or polyethylene.

The cables shall be covered with a <u>2 mil</u> corrugated polyester binder tape prior to application of the inner jacket. The tape shall be applied helically with a minimum overlap of 25 percent.

After cable fabrication is completed, the cable ends shall be suitably sealed by the cable manufacturer to prevent moisture from entering the conductor core area during shipment and storage.

8.9.7.3 INNER AND OUTER JACKET MATERIAL

Both the inner jacket which covers the cabled core assembly, and the outer jacket covering the armor wire shall be homogeneous layers of high density polyethylene in accordance with <u>ICEA S-95-658/NEMA WC-70</u>. The thickness of the inner and outer jackets shall be taken as specified in Table 1:

COMMENTARY

e

← search + replace this throughout document

SPECIFICATIONS

Table 8.9.7.3-1 - Jacket Sizes

Calculated Diameter of Cable Under the Respective Jacket (in.)	Jacket Average Thickness (in.) (mil)
0-0.425	45
0.426-0.700	60
0.701-1.500	80
1.501-2.500	110
2.501 and larger	140

8.9.7.4 CABLE ARMOR WIRE

The armor wire shall consist of galvanized steel wires. Each armor wire shall be coated with a layer of high density polyethylene. The coating shall be sunlight and weather resistant.

The coated armor wires shall be applied at a lay angle of 17 to 25 degrees and provide a coverage of 92 to 98 percent. The coated armor wires shall be applied in a left lay helix.

The armor wires shall be sized as specified in Table 1:

Table 8.9.7.4-1 - Armor Wire Sizes

Calculated Diameter of Jacketed Core (mm)	Nominal Size of Armor Wire (mm)	Nominal Thickness of PE Coating (mm)
0-19.05	2.77	0.508
19.06-25.40	3.40	0.635
25.41-43.18	4.19	0.762
43.19-63.51	5.16	0.889
63.52 and larger	6.05	1.016

A layer of separator tape shall be installed over the spiral-wound armor wires, prior to the application of the a outer jacket. A 2 mil ~~corrugated polyester separator~~ tape shall be applied helically with a minimum overlap of 25 percent.

8.9.7.5 TESTING

The finished cable shall withstand, between each conductor and all other conductors, including armor, an AC rms voltage in accordance with ICEA S-95-658/NEMA WC-70 for nonshielded 0–2 kV cables.

COMMENTARY

(mil)

Table 8.9.7.4-1 - Armor Wire Sizes

Calculated Diameter of Jacketed Core ~~(mm)~~ (inches)	Nominal Size of Armor Wire (BWG) ~~(mm)~~ (mils) ~~(inches)~~	Nominal Thickness of PE Coating ~~(mm)~~ (mils)
0-19.05 0 - 0.750	12 ~~2.77~~ 109	~~0.508~~ 20
0.751 ~~19.06-25.40~~ 1.000	10 ~~3.40~~ 134	~~0.635~~ 25
1.001 ~~25.41-43.18~~ 1.700	8 ~~4.19~~ 165	~~0.762~~ 30
1.701 ~~43.19-63.51~~ 2.500	6 ~~5.16~~ 203	~~0.889~~ 35
2.501 ~~63.52~~ & larger	4 ~~6.05~~ 238	~~1.016~~ 40

replace with English equivalent table (previously provided)

2006

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All conduits shall be specified to be UL listed and installed in accordance with the appropriate section of the NEC.

Size of conduits shall be specified in accordance with the NEC. In no case shall conduits less than 19 mm (3/4 inch) diameter be used.

in.

Metal conduits shall be provided with insulated throat grounding bushings and shall be electrically bonded with a bonding wire to all metal enclosures or wireways that they enter.

Couplings and fittings shall be specifically designed and manufactured for the application and for the conduit material.

Where conduit runs are attached to structures subject to movement, the contract documents shall contain provisions for the associated expansion or deflection. Fittings for accommodating expansion or deflection must consider the location, accessability and the calculated amount of movement.

Exposed conduit shall be supported on spacers, designed for the purpose, to support conduits at least 0.5 in. off the surface to which they are mounted.

Rigid conduit mounted on bridge structures should be supported at intervals of 6 ft.

COMMENTARY

Pressure or "C" clamp attachments should be avoided when bolted connections can be utilized. Bolted attachments are perferred on the movable spans due to generally higher vibration in those areas.

(At the time of this writing (1998), metric size conduit is not available in the United States or Canada. Therefore, the currently available size in English units is shown with the approximate metric value in parentheses.)

The vibration on bridge structures requires that conduit supports should be used at spacings considerably less than those required by the NEC. The recommended 6 ft. maximum applies to all conduit diameters and should not be exceeded, except due to structural limitations, in which case the maximum support spacing shall be maintained as close to 6 ft. as possible.

8.10.1.1 RIGID STEEL CONDUIT

Rigid steel conduit and fittings shall be specified to be hot-dip galvanized.

Where specified, rigid steel galvanized conduit and fittings shall be coated with a factory applied plastic coating with a nominal 40 mil thickness. The contract documents shall require that the bond between the PVC coating and the steel exceed the tensile strength of the plastic coating, taken as a minimum of 2,000 psi in lieu of better information. The contract documents shall specify that the interior and threads of plastic coated conduits have a urethane coating, 2 mil nominal thickness.

C8.10.1.1

All field cuts, or other interruptions in the galvanized coating shall be touched up with zinc rich cold galvanizing compound.

All nicks, cuts and scratches in the PVC coating shall be touched up with touch-up compound supplied by the conduit manufacturer.

Uncoated rigid steel conduit shall be protected with a bituminous coating or hot-applied bituminous tape at locations where conduit exits concrete or soil into air, or exits concrete into soil.

8.10.1.2 RIGID ALUMINUM CONDUIT

Rigid aluminum conduit shall be installed so as to be isolated off of structural steel and masonry by at least 0.5 in.

Aluminum conduit shall not be directly buried or encased in concrete.

C8.10.1.2

Aluminum conduit, where specified, should be used in exposed locations only. Precautions should be taken to isolate aluminum conduit from contact with soil or concrete.

SPECIFICATIONS

8.10.3 Junction Boxes and Terminal Cabinets

Junction boxes, pull boxes and terminal cabinets specified for outdoor locations or where not otherwise protected from oil and water, shall be rated NEMA 4 or 4X.

Boxes and cabinets in exposed locations shall be constructed of cast aluminum, hot-dip galvanized cast iron, or stainless steel. Hinges, bolts, screws and other hardware shall be specified to be brass or stainless steel.

Cast aluminum enclosures shall be specified to be isolated from concrete through the use of neoprene or similar material.

The size of boxes and cabinets, and the installation thereof, shall be based on requirements of the NEC, Articles 370 and 373, respectively.

Terminal cabinets shall have permanent labels attached indicating the designation of the cabinet as shown on the contract plans.

Terminal cabinets shall be specified to include interior mounting panels for terminal strips.

Cabinets and boxes in wet locations or subject to condensation shall be specified to include a minimum 6mm drain hole at the low point of the enclosure.

General purpose cabinets, boxes and enclosures to be installed indoors shall be NEMA 12.

Cabinets and boxes in dry, environmentally controlled areas and exceptionally clean may be specified as NEMA 1.

COMMENTARY

C8.10.3

Preferably all boxes exposed to the weather should be specified to conform to NEMA 4X requirements for maximum corrosion resistance.

It is common practice to isolate cast aluminum enclosures from structural steel.

Conduit entry should be to the bottom of enclosures whenever possible. The contract documents should require that requests for use of top entry for conduits be reviewed on a case-by-case basis.

The NEC should be considered the minimum requirements. In many cases, for ease of maintenance or longevity, it will be necessary to exceed the NEC requirements.

Larger drain holes if specified, should be screened to prevent entry of insects and rodents. Preferably, drain fittings, designed for the purpose, could be specified.

NEMA 12 rated enclosures should be specified in lieu of NEMA 1 as the minimum enclosure type in dry, environmentally controlled areas. NEMA 12 enclosures provide added protection from dust and dirt.

8.11 SERVICE LIGHTS AND RECEPTACLES

The contract documents shall contain provisions for complete electric lighting systems for:

- the operator's house,
- machinery house or deck,
- tower top machinery rooms,
- all stairways and walkways,
- the end lifting and locking areas,
- all other areas where inspection or maintenance of equipment is required.

The lighting systems shall be designed so that the following intensities are achieved:

- Operator's house - 300 lx 30 fc

C8.11

SPECIFICATIONS

- Machinery room - 200 lx - 20 fc
- Walkways, stairways, ladders, elevators - 20 fc
- Unhoused equipment - 15 fc

The lighting systems may utilize flourescent, incandescent, or high intensity discharge lighting fixtures or any combination thereof.

Incandescent lighting fixtures shall be rated for 100 watt lamps minimum, although smaller lamps may be specified to be installed. Similarly, conductors which supply incandescent fixtures shall be sized based on a minimum of 100 watts per fixture.

Control room lighting shall preferably be designed with dimming capability, adjustable from or near the control console.

Exterior lighting shall consist of enclosed high intensity discharge fixtures or vaportite incandescent fixtures equipped with globes and guards.

All lighting fixtures should be equipped with shock absorbing porcelain sockets, where practical.

Duplex-type receptacles shall be provided in each room of the operator's house, in machinery rooms, the end lifting and locking apparatus, and all other areas where the inspection or maintenance of equipment is required. Consideration shall also be given to furnishing receptacles inside, or in close proximity to the operator's console, control logic cabinets, terminal cabinets and span drive cabinets.

All receptacles shall be duplex, three wire, ground fault interrupter.

All receptacles exposed to the weather shall be housed in weatherproof enclosures, and all exposed parts shall be corrosion resistant.

Lighting circuits shall not be supplied from branch circuits that supply receptacles.

COMMENTARY

The designer should consider the lamp starting delays associated with high intensity discharge lighting. These lamps may not be suitable for use on certain walkways or stairways where such a delay could compromise safety.

Where using flourescent lamps outdoors or in unheated locations, the lamp performance characteristics should be investigated for the lowest ambient temperatures likely to be encountered.

8.12 GROUNDING

8.12.1 General

A grounding system shall be provided to meet or exceed the requirements of the NEC. The power system supplying the bridge shall be a solidly grounded system.

8.12.2 Equipment Grounding

Grounding for all equipment, cabinets and enclosures containing electric equipment shall be by dedicated grounding conductors run in each conduit and raceway from each piece of equipment, cabinet and enclosure back

C8.12.2

Some control equipment utilizes isolated ground systems which may not be connected to the system ground. The specifier should be aware of this situation and

APPENDIX A
SI Versions of Equations, Tables, and Figures
for the *AASHTO LRFD Movable Bridge Design Specifications*

[This Appendix was added in its entirety in the 2005 interim.]

Section 2

$$P_{LB} = \left(\overset{2100}{1200} + 0.55D\right)\left(\frac{F_y - 90}{140}\right) \qquad (2.5.1.1.3\text{-}1)$$

where:

D = diameter of the segment (mm)

F_y = specified minimum yield strength of the material in tension (MPa)

Section 5

$$FLT = \frac{9\,550\,000P_m}{n} \qquad (5.4.1\text{-}4)$$

where:

n = full-load speed (RPM)

P_m = standard motor size, power (kW)

Appendix A - SI Versions of Equations, Tables, and Figures

Table 5.8.2-1 - Friction Factors for Typical Bearings

Bearing Type	For Starting	For Motion
Plain Radial Type:		
For Trunnion or Journal Friction		
• Sleeve bearings, one or more complete rotations	0.14	0.09
• Sleeve bearings, less than one complete rotation	0.18	0.12
For linear sliding bearings	0.18	0.12
Plain Thrust Bearings:		
For friction on center disks	0.15	0.10
For collar friction at ends of conical rollers	0.15	0.10
Rolling Element Bearings and Rollers:		
Rolling element bearings	0.004	0.003
For rolling friction of bridges having rollers with flanges, or build-up segmental girders	0.009	0.006
For rolling friction of solid cast rollers without flanges	$0.063/\sqrt{D}$	$0.063/\sqrt{D}$
where: D = diameter of roller (in.) mm	$0.0113/\sqrt{D}$	$0.0113/\sqrt{D}$

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2005

Appendix A - SI Versions of Equations, Tables, and Figures

Section 6

Table 6.6.1-1 - Allowable Static Stresses in MPa

Material	AASHTO	ASTM	Tension	Compression	Fixed Bearing	Shear
Structural Steel	M 183M	A36/A 36M	83	83–0.38 L_{eff}/k	110	41
Forged Carbon Steel (except keys)	M 102M	A 668/A 668M (CL D)	103	103–0.45 L_{eff}/k	124	52
Forged Carbon Steel (keys)	M 102M	A 668/A 668M (CL D)	-	-	103	52
Forged Alloy Steel	M 102M	A 668/A 668M (CL G)	110	110–0.48 L_{eff}/k	145	55
Cast Steel	M 103M	A 27/A27M (GR 485-250)	62	69–0.31 L_{eff}/k	90	34
Cast Steel	-	A 148/A 148M (GR 620-415)	103	103–0.45 L_{eff}/k	145	52
Bronze	M 107	B 22 ALLOY UNS C90500, C91100, C91300, C93700	48	48	-	-
Hot Rolled Steel Bar	M 255M/M 255M	A 675/A 675M (GR 515)	83	83–0.38 L_{eff}/k	110	41

$$R_b = \left(\frac{A}{\omega D}\right) \qquad (6.6.2.5\text{-}1)$$

where:

A = planar or projected area over which the load is acting (mm^2)
ω = rotational speed—revolutions per minute (rpm)
D = diameter of journal or step bearing, or mean diameter of collar or screw (mm)
α = a factor specified herein (N/mm-rpm)

$$R_R = \left(\frac{\sigma_y - 90}{140} \right) \qquad (6.6.2.6\text{-}1)$$

where:

σ_y = minimum yield strength of the weaker material (Mpa)
D = diameter of the roller (mm)
α = factor specified herein (N/mm)

	Diameters up to 635 mm	Diameters 635 to 32,000 mm
For rollers in motion:	2.80 D	$70\sqrt{D}$
For rollers at rest:	4.20 D	$105\sqrt{D}$

For $D \leq 8\text{mm}$, $C_D = 1$ (6.6.3.2-2)

For $D > 8\text{mm}$, $C_D = (D/7.6)^{-0.113}$ (6.6.3.2-3)

Table 6.6.3.2-1 - Variables for Determining C_s (Surface Roughness Factor) (Shigley, 1989)

Condition	a	b
For a ground surface	1.58	-0.085
For a cold finished or smooth machined surface with $R_a \leq 0.8$ µm	4.51	-0.265
For a hot rolled or rough machined with $R_a > 0.8$ µm, or as a heat treated surface	57.7	-0.718
For an as cast or as forged surface	272.0	-0.995

Appendix A - SI Versions of Equations, Tables, and Figures

$\sigma_{FP} = 0.533H_B + 88.3$ (6.6.4.2-1)

where:

H_B = Brinell hardness for the teeth (dim)

$\sigma_{HP} = 2.22H_B + 200$ (6.6.4.3-1)

where:

σ_{HP} = gear design allowable stress—surface durability (MPa)

2.069H_B

$\sigma_S = 0.014H_B^2 - 2{,}069H_B + 213.8$ (6.6.4.4-1)

where:

σ_S = gear design allowable yield (MPa)

$\sigma_S = 3.324H_B - 226.2$ (6.6.4.4-2)

Table 6.7.3.2-1 - Value of Neuber Constant

σ_{ut} (MPa)	$\sqrt{a}$ $(mm)^{0.5}$
420	0.54
630	0.35
840	0.25
980	0.20
1260	0.12

Table C6.7.3.2-1 - Value of Notch Sensitivity Factor

MPa

σ_{ut} (p)	q
420	0.85
630	0.90
840	0.93
980	0. 94
1260	0.96

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$$L = 220(D^2)^{1/3} \quad (6.7.4.2\text{-}1)$$

where:

L = length of shaft between bearings (mm)
D = diameter of solid shaft (mm)

$$\omega_c = 120 \times 10^6 \frac{D}{L^2} \quad (6.7.4.3\text{-}1)$$

× ⟵ multiplication sign

where:

ω_c = critical shaft rotation speed (rpm)
D = diameter of the solid shaft (mm)
L = distance between supports, usually flexible gear couplings (mm)

$$\omega_c = 0.207 \times 10^6 \left(\frac{D^2}{L^2}\right)\sqrt{\frac{L}{m}} \quad (6.7.4.3\text{-}2)$$

in which:

$$m = \frac{W}{g} \quad (6.7.4.3\text{-}3)$$

where:

ω_c = critical shaft rotation speed (rpm)
D = diameter of the solid shaft (mm)
L = distance between supports, usually flexible gear coupling (mm)
W = weight of the concentrated mass (N)
M = concentrated mass (kg)
G = acceleration due to gravity taken as 9.81 m/s^2

$$F_t = \frac{2T}{d_{w1}} \quad (6.7.5.2.1\text{-}1)$$

where:

F_t = tangential transmitting gear force (N)
T = FLT (N-mm) applied at gear shaft
d_{w1} = pinion pitch diameter (mm)

$$P = \frac{F_t d_{w1} \omega_1}{1.91 \times 10^7} \quad (C6.7.5.2.1\text{-}1)$$

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where:

ω_1 = pinion rotational velocity (rpm)

$$F_1 \le F_{tay} \qquad (6.7.5.2.2\text{-}1)$$

F_{tay} = factored flexural resistance of spur gear teeth (N)

$$F_{tay} = \frac{b m_t Y_J \sigma_{FP} Y_N}{K_o K_v K_s K_H K_B S_F Y_\theta Y_z} \qquad (6.7.5.2.2\text{-}2)$$

in which:

$$K_v = \left(\frac{A + \sqrt{200 v_t}}{A}\right) \qquad (6.7.5.2.2\text{-}3)$$

$$A = 50 + 56(1.0 - B) \qquad (6.7.5.2.2\text{-}4)$$

$$B = 0.25(12 - Q_v)^{0.667} \qquad (6.7.5.2.2\text{-}5)$$

$$v_t = \pi \omega_1 \frac{d_{w1}}{60{,}000} \qquad (6.7.5.2.2\text{-}6)$$

where:

K_v = dynamic factor (dim)
b = tooth face width of the spur pinion or gear that is being analyzed/designed (mm)
m_t = transverse tooth module taken as d_{w1}/N_{w1} (mm)
v_t = pitch line velocity (m/s)
d_{w1} = pitch diameter of the pinion (mm)
N_{w1} = number of teeth on the pinion
σ_{FP} = allowable bending stress specified in Article 6.6.4.2 (MPa)
K_o = overload factor taken as >1.0 where momentary overloads up to 200 percent exceed 4 in 8 hours, and exceed one second duration (dim)
H_B = Brinell hardness for the teeth (dim)
n_L = number of load cycles
ω_1 = pinion (rpm)
Q_V = gear quality number taken as an integer between 7 and 12 (dim)
K_S = tooth size factor to reflect nonuniformity of tooth material properties, due to large tooth size, gear diameter, and face width taken as ≥ 1 (dim)

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[redacted]

Appendix A - SI Versions of Equations, Tables, and Figures

K_H = load distribution factor taken as $K_H = 1.26 + 0.00102b$ for open gearing, adjusted at assembly, with $b<700$ mm, and $b/d_{wl} <1$ (dim)

K_B = rim thickness factor taken as 1.0 if $m_B = t_R/h_t > 1.2$ (dim)

m_B = the backup ation (dim) ratio

t_R = rim thickness (dim)

h_t = total tooth height (mm)

S_F = safety factor for bending strength (fatigue) $S_F \geq 1.2$ (dim)

Y_J = geometry factor for bending strength (dim)

Y_N = life factor for bending resistance taken as (dim):

- For $H_B \approx 250$ and $10^3 < n_L < 3 \times 10^6$

$$Y_N = 4.940 n_L^{-0.1045} \quad (6.7.5.2.2\text{-}7)$$

4.9404

- For $n_L > 3 \times 10^6$ load cycles, regardless of hardness

$$Y_N = 1.6831 n_L^{-0.0323} \quad (6.7.5.2.2\text{-}8)$$

THERE ARE A LOT OF THESE

Y_θ = temperature factor taken as 1.0 for gear temperatures less than 120 °C (dim)

Y_z = reliability factor (dim)

- For 99 percent reliability
- 1.25 for 99.9 percent reliability

$$F_t \leq F_{taz} \quad (6.7.5.2.3\text{-}1)$$

where:

F_t = tangential transmitting gear force (N)

F_{taz} = factored surface durability resistance of spur gear teeth (N)

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$$F_{taz} = \frac{b d_{wI} Z_I}{K_o K_v K_s K_H Z_R} \left(\frac{\sigma_{HP} Z_N Z_W}{Z_E S_H Y_\theta Y_Z} \right) \qquad (6.7.5.2.3\text{-}2)$$

where:

b = tooth face width of the pinion or gear that has the narrowest face width (mm)

σ_{HP} = allowable contact stress for the lower Brinell hardness number of the pinion/gear pair as specified in Article 6.6.4.3 (MPa)

n_L = number of load cycles (dim)

S_H = safety factor for pitting resistance, i.e., surface durability taken as >1 (dim)

Z_E = elastic coefficient taken as 190 for steel pinion-steel gear $(MPa)^{0.5}$

Z_N = stress cycle factor for pitting resistance taken as $2.466 n_L^{-0.056}$ for $10^4 < n_L <$ (dim) 10^{10}

Z_W = hardness ratio factor for pitting resistance, taken as 1.0 if $H_{BP}/H_{BG} < 1/2$ (dim) 1.2

Z_I = geometry factor for pitting resistance (dim)

Z_R = surface condition factor for pitting resistance taken as 1.0 for good tooth surface condition as specified in Article 6.7.8 for tooth surface finish depending on module (dim)

$$F_t\,(max) \le F_{max} \qquad (6.7.5.2.4\text{-}1)$$

where:

F_t = tangential transmitting gear force (N)

F_{max} = maximum peak transmitting gear force (N)

$$F_{max} = \frac{K_y b m_t K_f Y_J \sigma_s}{K_{HS}} \qquad (6.7.5.2.4\text{-}2)$$

where:

K_y = yield strength factor taken as 0.50 (dim)

K_f = stress correction factor = 1 (dim)

σ_S = allowable yield stress number specified in Article 6.6.4.4 (MPa)

K_{HS} = load distribution factor for overlaod conditions taken as $K_{HS} > 1.1$ for straddle-mounted gear (dim)

b = tooth face width of the spur pinion or gear that is being analyzed/designed (mm)

m_t = transverse tooth module (mm)

Y_J = geometry factor for bending strength (dim)

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Appendix A - SI Versions of Equations, Tables, and Figures

$$p = \frac{F_w}{(DL)}$$ Fur (6.7.7.1.2-1)

$$V = \frac{\pi D w}{(60{,}000)}$$ (6.7.7.1.2-2)

where:

F_{ur} = applied radial load (N)
p = pressure (MPa)
V = surface velocity (m/s)
D = diameter of the journal (bearing I.D.) (mm)
L = length of the bearing (mm)
ω = journal rotational speed (rpm)

Table 6.7.7.1.2-1 - Performance Parameters for Cast Bronze Bearings

UNS ALLOY	p (MPa)	V (m/s)	pV (MPa·m/s)	COMMON NAME
C 86300	55.2	0.12	2.45	Mang. Bronze
C 91100	17.2	0.25	1.05	Phos. Bronze
C 91300	20.7	0.25	1.05	Phos. Bronze
C 93700	6.9	1.25	1.05	Tin Bronze
C 95400	24.1	0.50	1.75	Alum. Bronze

Table C6.7.7.1.4a-1 - Performance Parameters for Oil-Impregnated Metals

MATERIAL	ASTM NO.	p (MPa)	V (m/s)	pV (MPa·m/s)
Oilite Bronze	B-438-73 Gr 1 Type II	13.8	6.10	1.75
Super Oilite	B-439-70 Gr 4	27.6	1.14	1.23
Super Oilite - 16	B-426 Gr 4 Type II	55.2	0.18	2.63

cut off ?

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Table C6.7.7.1.4b-1 - Performance Parameters for Nonmetal Bearings

MATERIAL	p (MPa)	V (m/s)	pV (MPa·m/s)
Acetal ("Delrin")	6.9	5.08	0.105
Nylon	6.9	5.08	0.105
Phenolics	41.0	12.70	0.525
TFE ("Teflon")	3.5	0.25	0.035
PTFE Composite	69.0	0.76	0.875

shift numbers in columns so they are not cut-off by column border.

$$P_r = 0.77 C_r \left(\frac{1}{\omega}\right)^{0.3} \qquad (6.7.7.2.2\text{-}1)$$

where:

C_r = basic dynamic radial load rating of the bearing (N)

X ω = rotational speed of the bearing inner race (RPM)

rpm ✓ ← Search+replace this throughout document

Table 6.7.8-1 - Fits and Finishes

PART	FIT	FINISH R_a(μm)
Machinery base on steel	--	6.3
Machinery base on masonry	--	12.5
Shaft journals	H8/f7	0.2
Journal bushing	H8/f7	0.4
Split bushing in base	H7/h6	3.2
Solid bushing in base (to 6 mm wall)	H7/p6	1.6
Solid bushing in base (over 6 mm wall)	H7/s6	1.6
Hubs on shafts (to 50 mm bore)	H7/s6	0.8
Hubs on shafts (over 50 mm bore)	H7/s6	1.6
Hubs on main trunnions	H7/s6 to H7/u6	1.6
Turned bolts in finished holes	H7/h6	1.6
Sliding bearings	H8/f7	0.82
Center discs	--	0.8
Keys and keyways		
Top and Bottom	H7/h6	1.6
Sides	H7/s6	1.6
Machinery parts in fixed contact	--	3.2
Teeth of open spur gears		
Under 8 mm module	--	0.8
8 to 16 mm module	--	1.6
Over 16 mm module	--	3.2

0.8 ✓

X

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Table C6.7.10.1-1 - Key Sizes

change to MULTIPLICATION SIGN

DIAMETER (mm) OVER-TO-INCLUDE		KEY SIZE (mm) (b x h)
12	17	5 x 5
17	22	6 × 6
22	30	8 x 7
30	38	10 × 8
38	44	12 × 8
44	50	14 × 9
50	58	16 × 10
58	65	18 × 11
65	75	20 × 12
75	85	22 × 14
85	95	25 × 14
95	110	28 × 16
110	130	32 × 18
130	150	36 × 20
150	170	40 × 22
170	200	45 × 25
200	230	50 × 28
230	260	56 × 32
260	290	63 × 32
290	330	70 × 36
330	380	80 × 40
380	440	90 × 45

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← Due to conversion?

$$\sigma \quad S_b = E_w \frac{d_w}{D} \qquad (6.8.3.3.4\text{-}1)$$

where:

d_W = diameter of the outer wires in the wire rope (mm)

D = tread diameter of sheave rope grooves (mm)

E_W = tensile modulus of elasticity of the steel wire = 200 000 MPa

$$\sigma \quad S_b = \frac{125\ 000}{D/d} \qquad (C6.8.3.3.4\text{-}1)$$

where:

d = diameter of the wire rope (mm)

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Section 8

Table C8.9.1-1 - AWG/Metric Conversion Table

AWG or MCM	Circular Mils	Cross-Sectional Area mm^2	Metric Conductor Size mm^2
14	4110 4930	2.08 2.50	 2.5
12	6530 7890	3.31 4.00	 4.0
10	10380 11800		 6.0
8	16510 19700		 10.0
6	26240 31600		 16.0
4	41740 49300		 25.0
2	66360 69100		 35.0
1	83690 98700		 50.0
1/0	105600		-
2/0	133100 138000		 70.0
3/0	167800 18700 187000		 95.0
4/0	211600 23700 237000		 120.0
250 MCM	250000 296000		 150.0
300	300000		-
350	350000 365000		 185.0
400	400000 474000		 240.0
500	500000 592000		 300.0
600	600000		-
750	750000 789000 987000		 400.0 500.0
1000	1000000		-

Cross-Sectional Area mm^2
2.08 2.50
3.31 4.00
5.26 6.00
8.37 10.00
13.30 16.00
21.15 25.00
33.63 35.00
42.41 50.00
53.48
67.43 70.00
85.03 95.00
107.20 120.00
126.64 150.00
152.00
177.35 185.00
202.71 240.00
253.35 300.00
303.96
379.95 400.00 500.00
506.60

Please insert into table

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Appendix A - SI Versions of Equations, Tables, and Figures

Table 8.9.7.3-1 - Jacket Sizes

Calculated Diameter of Cable Under the Respective Jacket (mm)	Jacket Average Thickness (mm)
0-10.80	1.143
10.81-17.79	1.524
17.80-38.11	2.032
38.12-63.51	2.794
63.52 and larger	3.556

Table 8.9.7.4-1 - Armor Wire Sizes

Calculated Diameter of Jacketed Core (mm)	Nominal Size of Armor Wire (mm)	Nominal Thickness of PE Coating (mm)
0-19.05	2.77	0.508
19.06-25.40	3.40	0.635
25.41-43.18	4.19	0.762
43.19-63.51	5.16	0.889
63.52 and larger	6.05	1.016

MOVABLE BRIDGE SPECIFICATIONS – 2005 INTERIM

Search List

Replace 1998 with 2006 in the following Sections:

Section 1

Article

C1.3.1

Section 2

None

Section 3

None

Section 4

None

Section 5

Article

C5.4.1

Section 6

Article

C6.4.1.1

Section 7

None

Section 8

Article

C8.10.1

2006 AASHTO BRIDGE COMMITTEE AGENDA ITEM: 8 (REVISION 1)

SUBJECT: LRFD Bridge Design Specifications: Section 5, Articles 5.2 and 5.10.11.4.1 (WAI 95)

TECHNICAL COMMITTEE: T-10 Concrete

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

☒ **DESIGN SPEC** ☐ **CONSTRUCTION SPEC** ☐ **MOVABLE SPEC**
☐ **LRFR MANUAL** ☐ **OTHER**

☐ **US VERSION** ☐ **SI VERSION** ☒ **BOTH**

DATE PREPARED: 1/5/06
DATE REVISED: 5/16/06

AGENDA ITEM:

Item #1

Add the following to Article 5.2 DEFINITIONS:

Seismic Hoop – A cylindrical noncontinuously wound tie with closure made using a butt weld or a mechanical coupler.

Item #2

Add the following to Article C5.10.11.4.1c, as a new 1st paragraph:

Seismic hoops may offer the following advantages over spirals:

- Improved constructability when the transverse reinforcement cage must extend up into a bent cap or down into a footing. Seismic hoops can be used at the top and bottom of the column in combination with spirals, or full height of the column in place of spirals.
- Ability to sample and perform destructive testing of in-situ splices prior to assembly.
- Breakage at a single location vs. potential unwinding and plastic hinge failure.

Item #3

Revise Article 5.10.11.4.1d Transverse Reinforcement for Confinement at Plastic Hinges, 2nd paragraph as follows:

For a circular column, the volumetric ratio of spiral or seismic hoop reinforcement, ρ_s shall satisfy either that required in Article 5.7.4.6….(*rest of paragraph is unchanged*)

In the commentary to Article 5.10.11.4.1d, revise the current 3rd paragraph as follows:

Careful detailing of the confining steel in the plastic hinge zone is required because of spalling and l~~L~~oss of concrete cover ~~in the plastic hinge zone as a result of spalling requires careful detailing of the confining steel~~. With deformation associated with plastic hinging, the strains in the transverse reinforcement increase. ~~It is clearly~~

~~inadequate to simply lap the spiral reinforcement;~~ ~~U~~ultimate-level splices are required. ~~If the concrete cover spalls, the spiral will be able to unwind.~~ Similarly, rectangular hoops should be anchored by bending ends back into the core.

Item #4

Revise Paragraph 2 in Article 5.10.11.4.1f Splices, as follows:

Lap splices in longitudinal reinforcement shall not be used ~~only within the center half of column height, and the splice length shall not be less than 16.0 in. or 60.0 bar diameters~~.

Add the following as Paragraphs 2 and 3 in the Commentary to Article 5.10.11.4.1f as follows:

Splices in seismic-critical elements should be designed for ultimate behavior under seismic deformation demands. Recommendations for acceptable strains are provided in Table 1. The strain demand at a cross-section is obtained from the deformation demand at that cross-section and the corresponding moment-curvature relationship. Traditional service level splices are only appropriate in components such as bent caps, girders, and footings, when not subjected to or protected from seismic damage by careful location and detailing of plastic hinge regions.

Table C5.10.11.4.1f-1 Recommended Strain Limits in A706 Bars, and Bars with Splices for Seismic Zones 3 and 4.

	Minimum Required Resisting Strain, ε Bar only	Minimum Required Resisting Strain, ε Bar with Splice	Maximum Allowable Load Strain, ε	Resulting Factor of Safety
Ultimate	6% for #11 and larger 9% for #10 and smaller	6% for #11 and larger 9% for #10 and smaller	<2%	3 to 4.5
Service	(same as above)	≥ 2 %	≤0.2%	≥ 10
Lap (or welded / mechanical lap in lieu of lap splice)	(same as above)	≥ 0.2%	≤0.15% (unfactored loads) ≤0.2% (factored)	1.33

Limits are based on tests done by the California Department of Transportation and University of California-Berkeley, the latter of which is described in ACI (*2001*). The demonstrated strain at ultimate resistance of butt-welded details was divided by the typical demand strain in order to document the factor of safety. Although current experimental limitations of other splice details performing at the service level preclude strain measurements, known values are shown in Table C1 for comparison. The variability of strain along the potential plastic hinge justifies the much higher factor of safety. Use of traditional splice details to resist extreme loading conditions where nonlinear behavior is desired and analyzed as such, are shown to be inefficient. A615 steel is generally not permitted by Caltrans because of weldability and ductility concerns, and was not investigated.

Item #5

Add the following to REFERENCES at the end of Section 5:

Lehman, Dawn et. al. "Repair of Severely Damaged Bridge Columns"; *ACI Structural Journal*; Vol. 98 Issue 2, March 2001, p233-242.

OTHER AFFECTED ARTICLES:

Article 5.13.4.6.3, Cast-in-Place Piles in Seismic Zones 3 and 4.

BACKGROUND:

The terminology "seismic hoop" is proposed to distinguish between the present AASHTO definition for hoops, and non-continuously wound circular hoops with a butt-weld or mechanical closure detail. The later is prudent in some seismic situations and has been used throughout this ballot item.

Current seismic requirements in the *AASHTO LRFD Specifications* for compression members resemble those used for buildings. While bridges may be thought of as single-story buildings, its frame tends to have fewer bracing elements. Bridge column diameters are generally more massive. Past performance of bridges in seismic events must be taken into account. This experience has shown that:

- Seismic performance of spiral reinforcement in short, large-diameter columns, is questionable
- Lap-splicing of longitudinal bars in seismic critical components (components essential for non-collapse of the structure i.e. non-collapse design criteria) is undependable.
- Bounds of plastic behavior aren't certain, and the cost difference of alternate splicing methods is insignificant compared to the risk

Caltrans' practices are suggested as Commentary for designers who wish to increase confidence in their Seismic Zones 3 and 4 bridges:

- Seismic hoops in seismic-critical components
- Ultimate splice requirements for seismic-critical components

ANTICIPATED EFFECT ON BRIDGES:

Improved performance of columns, piles, and shafts during an Extreme Event.

REFERENCES:

ACI 318-05

"Splices in Bar Reinforcing Steel", California Department of Transportation *Bridge Design Memo to Designers* #20-9, p6-7 and Attachment 1; August 2001.

OTHER:

None

2006 AASHTO BRIDGE COMMITTEE AGENDA ITEM: 9 (REVISION 1)

SUBJECT: LRFD Bridge Design Specifications: Section 5, Articles 5.5.3.1, 5.5.3.2 and 5.9.4.2.1 (WAI 83)

TECHNICAL COMMITTEE: T-10 Concrete

☒ **REVISION**	☒ **ADDITION**	☐ **NEW DOCUMENT**
☒ **DESIGN SPEC** ☐ **LRFR MANUAL**	☐ **CONSTRUCTION SPEC** ☐ **OTHER**	☐ **MOVABLE SPEC**
☒ **US VERSION**	☐ **SI VERSION**	☐ **BOTH**

DATE PREPARED: 1/5/06
DATE REVISED: 5/24/06

AGENDA ITEM:

Item # 1

Delete the fourth paragraph of Article 5.5.3.1.

~~Where consideration of fatigue is required, the stress range shall be determined using twice the fatigue load combination as specified in Table 3.4.1-1, i.e. using a load factor of 1.5.~~

~~Add the following paragraph to the end of Article C5.3.3.1, opposite the paragraph revised in Item #1 above:~~

Item # 2

Revise Article 5.5.3.2 as follows:

5.5.3.2 Reinforcing Bars

The stress range in straight reinforcement and welded wire reinforcement without a cross weld in the high stress region resulting from ~~twice~~ the fatigue load combination, specified in Table 3.4.1-1, shall satisfy:

$$\cancel{f_f \leq 21 - 0.33 f_{\min} + 8\left(\frac{r}{h}\right)} \qquad (5.5.3.2\text{-}1)$$

$$f_f \leq 24 - 0.33 f_{\min} \qquad (5.5.3.2\text{-}1)$$

The stress range in straight welded wire reinforcement with a cross weld in the high stress region resulting from ~~twice~~ the fatigue load combination ~~as~~, specified in Table 3.4.1-1, shall satisfy:

$$f_f \leq 16 - 0.33 f_{\min} \qquad (5.5.3.2\text{-}2)$$

where:

f_f = stress range (ksi)

f_{min} = ~~algebraic minimum stress level, tension positive, compression negative~~ the minimum live load stress resulting from ~~twice~~ the fatigue load ~~combination specified in Table 3.4.1-1~~, 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)

The definition of the high stress region for application of Eqs. 1 and 2 ~~is:~~ for flexural reinforcement~~,~~ shall be taken as one-third of the span on each side of the section of maximum moment.

- ~~For shear reinforcement in I-beams, box beams, and similar members, the clear web height between fillets~~
- ~~For shear reinforcement in rectangular beams and other members, the middle 2/3rds of the total member depth~~
- ~~For flexural reinforcement, one-third of the span on each side of the section of maximum moment~~

~~r/h = ratio of base radius to height of rolled-on transverse deformations; if the actual value is not known, 0.3 may be used.~~

Add the following to the end of Article C5.5.3.2:

Structural welded wire reinforcement has been increasingly used in bridge applications in recent years, especially as auxiliary reinforcement in bridge I- and box beams and as primary reinforcement in slabs. Design for shear has traditionally not included a fatigue check of the reinforcement as the member is expected to be uncracked under service conditions and the stress range in steel minimal. ~~However, with increasing use of thin web members, high strength concrete, and high maximum shear limits, it is possible that diagonal tension at service limit states would result in diagonal cracking and increased stress range in stirrups.~~ The stress range for steel bars has existed in previous editions. It is based on Hansen et al (*1976*). The simplified form in this edition replaces the (*r*/*h*) parameter with the default value 0.3 recommended by the Hansen et al. Inclusion of limits for WWR is based on recent studies by Hawkins et al (*1971*, *1987*) and Tadros et al (*2004*). ~~Use of WWR as flexural reinforcement in nonprestressed slabs and other members requires that fatigue limits be checked in a manner consistent with treatment of ordinary reinforcing bars.~~

~~Twice the fatigue load combination specified in Table 3.4.1-1 for Eq. 2 is appropriate as this value represents the provides the equivalent maximum stress range for fatigue considerations, and the various fatigue-resistance equations of Section 5 are lower-bound equations providing theoretically infinite fatigue life.~~

~~Item #3~~

~~Revise the first paragraph of Article 5.9.4.2.1 as follows:~~

~~Compression shall be investigated using the Service Limit State Load Combination I specified in Table 3.4.1-1, except as noted. The limits in Table 1 shall apply.~~

~~Revise the third bullet item in Table 5.9.4.2.1-1 as follows:~~

- ~~In other than segmentally constructed bridges due to twice the live load of the Fatigue Load Combination and one-half the sum of the effective prestress and permanent loads of the Service Limit State Load Combination I, specified in Table 3.4.1-1.~~

~~Add the following paragraph to the beginning of Article C5.9.4.2.1:~~

~~The compression stress limit at 0.40 f'_c is an infinite-life fatigue resistance. As such, the live load to be considered shall be twice that of the Fatigue Load Combination. See Article C5.5.3.1.~~

Item #3

Add the following references at the end of Section 5:

1. Jongpitaksseel, N., Bowers, J., Amornrattanepong, W., and Tadros, M. K., "Development of Fatigue Limit Formula for Deformed Welded Wire Reinforcement," 2004 Concrete Bridge Conference Paper #79, Charlotte, NC, 2004.
2. Tadros, M. K., Bowers, J. and Amornrattanepong, W., "A Unified Approach to Fatigue Design in Concrete Bridges," PCI Journal, (scheduled for publication in early 2006).

OTHER AFFECTED ARTICLES:

None

BACKGROUND:

Item #1

This paragraph is redundant since it is repeated, and modified for WWR, in the revised Article 5.5.3.2.

Item #2

Welded wire reinforcement (WWR) has many advantages over traditional mild reinforcing steel. WWR boasts a higher yield strength, higher quality control, and significantly lower construction labor costs. However, many designers are reluctant to use WWR as a structural reinforcement alternative to mild steel reinforcing bars, due to a lack of fatigue design guidance. The proposed changes will allow designers to confidently utilize the advantages of WWR.

The current rebar fatigue formula was developed in a research project conducted for the National Cooperative Highway Research Program by Hanson et al (1976). The research report was published as NCHRP Report 164, titled "Fatigue Strength of High Yield Reinforcing Bars." Grade 60, #8 bars were the primary targets. Other sizes and grades were also tested. The formula was adopted without any revision in ACI documents and in the 1994 Edition of the AASHTO LRFD Bridge Design Specifications. The research was done on single bars embedded in small beams as flexural reinforcement with two-point loading applied at the rate of 250 to 500 cycles per minute. A total of 353 concrete beams were tested. However, most of the tests were done for the so-called "finite-life" region where the number of cycles was between 10,000 and 1,000,000.

The researchers concluded that the "long-life" region, of 1 to 5 million cycles, was the more important one for design purposes and based their conclusions of that region on Phase II of their work. The formula was, essentially, based on only the very few tests performed on #8 Grade 60 bars from "Manufacturer A" with a minimum stress of 6 ksi, and a fatigue life in excess of 2 million cycles. The researchers also reported that the fatigue limit of interest to the bridge designer is not sensitive to concrete beam dimensions, concrete material properties, bar size, steel grade (in the range of 40 to 75 ksi tested), or steel metallurgy.

Most fatigue experiments (ACI Committee 215, 1998) in the past were focused on testing reinforcing bars embedded in structural concrete members. MacGregor et. al (1971) reported fatigue tests on reinforced concrete beams, containing #5, #8, and #10 reinforcing bars with yield strength of 40, 60, and 75 ksi. They concluded that fatigue strength of reinforcing bars was relatively insensitive to the tensile strength of the bar metal. However, the fatigue strength of the bars was appreciably lower than that of the base metal. This difference resulted from the stress concentration at the base of the deformations and a decarburized layer on the outside of the bars. However, this investigation was performed on fatigue tests of hot rolled deformed reinforcing bars embedded in concrete beams.

Pasko (1971) performed fatigue tests on #5 deformed reinforcing bars conforming to ASTM A615 Grade 60, welded to #3 plain transverse reinforcing. Hawkins (1971) performed fatigue test on plain straight wires with cross welds, cut from 6 x 6 – W2 x W2, and fatigue loading on slabs with the same reinforcement. The only application of concern to the researchers was one-way reinforced slabs with a mid-span concentrated load. Only one value of dead load (minimum) stress was considered: 8 ksi. Only three stress range (live load) values were considered: 31.4, 41.5, and 53.1 ksi. The report did not conclude with a fatigue limit. The high stress range values resulted in fatigue fracture of the wire before the endurance limit was reached. Hawkins (1987) performed fatigue testing on twelve concrete slabs with welded wire reinforcement, both with and without epoxy coating. These investigations do not represent the practical WWR applications utilized in structural bridge components

The proposed fatigue formula is based on the latest research conducted at the University of Nebraska (Jongpitaksseel et al, 2004 and Tadros et al, 2006). This research takes what was learned from the NCHRP study and builds upon it for WWR. Therefore, the research was focused on the “long-life” region and the recommended formulas were developed based on specimens with a fatigue life in excess of five million cycles. The testing program studied two conditions A) Wire with no cross weld in the high tension zone, and B) Wire with cross weld in the high tension zone.

It was concluded that wire with no cross weld in the high tension zone could be safely treated with the same formula as the current formula for rebar. The wire with cross weld in the high tension zone is most appropriately treated with the above proposed formula.

Instead of testing the reinforcement embedded in concrete members, the Nebraska research focused on the WWR material itself. It is true that testing in concrete members would be valuable, but only after the fatigue properties of the reinforcement itself is established. The NCHRP tests have shown that testing in concrete beams did not impact the developed fatigue formula. While embedded testing should be undertaken in the future, rapidly establishing a conservative fatigue formula now is a significant contribution to the state of the art, and of great benefit to bridge designers.

Additionally, the proposed formulas eliminate the current (r/h) term. In practice, this term has been almost universally been taken as the suggested 0.3. The proposed change recognizes this practice, and uses it to further simplify the formulas.

ANTICIPATED EFFECT ON BRIDGES:

The proposed changes will allow designers to more often consider WWR as an alternative reinforcement to conventional rebar. Applications include bridge deck reinforcement, girder web shear reinforcement, and other structural reinforcement situations. When appropriately applied, the increased use of WWR has the potential to save time and money.

REFERENCES:

1. ACI Committee 215, “Considerations for Design of Concrete Structures Subjected to Fatigue Loading,” ACI Manual of Concrete Practice 1998 Part 1, American Concrete Institute, Farmington Hills, MI.
2. ASTM, 2002, “Standard Specification for Steel Welded Wire Reinforcement, Deformed, for Concrete.” *ASTM A497/A497M-02,* American Society for Testing and Materials International, West Conshohocken, Pa., 2002, 5 pp.
3. Hanson, J.M., Somes, N.F., Helgason, T, Corley, W.G., and Hognestad, E., “Fatigue Strength of High Yield Reinforcing Bars,” NCHRP Report 164, National Cooperative Highway Research Program, Transportation Research Board, Washington, D.C., 1976, 90 pp.
4. Hawkins, N.M. and Heaton, L.W., “The Fatigue Properties of Welded Wire Fabric,” Report SM 71-3, University of Washington, Seattle, WA, September 1971.
5. Hawkins, N.M. and Takebe, Y., “Long Life Fatigue Characteristics of Large Diameter Welded Wire Fabric,” Report SM 87-1, University of Washington, Seattle, WA, January 1987.
6. MacGregor, J.G., Jhamb, I.C., and Nuttall, N., “Fatigue Strength of Hot Rolled Deformed Reinforcing Bars,” ACI Journal, March 1971, pp. 169-179.

7. Pasko, T.J., "Effect of Welding on Fatigue Life of High-Strength Reinforcing Steel Used in Continuously Reinforced Concrete Pavements," Report No. FHWA-RD-72-32, Federal Highway Administration, Washington, D.C., November 1971.
8. Jongpitaksseel, N., Bowers, J., Amornrattanepong, W., and Tadros, M. K., "Development of Fatigue Limit Formula for Deformed Welded Wire Reinforcement," 2004 Concrete Bridge Conference Paper #79, Charlotte, NC, 2004.
9. Tadros, M. K., Bowers, J. and Amornrattanepong, W., "A Unified Approach to Fatigue Design in Concrete Bridges," PCI Journal, (scheduled 2006).

OTHER:

None

2006 AASHTO BRIDGE COMMITTEE AGENDA ITEM: 10 (REVISION 1)

SUBJECT: LRFD Bridge Design Specifications: Section 5, Article 5.8.3 (WAI 108)

TECHNICAL COMMITTEE: T-10 Concrete

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

☒ **DESIGN SPEC** ☐ **CONSTRUCTION SPEC** ☐ **MOVABLE SPEC**
☐ **LRFR MANUAL** ☐ **OTHER**

☒ **US VERSION** ☐ **SI VERSION** ☐ **BOTH**

DATE PREPARED: 1/9/06
DATE REVISED: 5/22/06

AGENDA ITEM:

Item #1

In Article 5.4.2.6, add the following sub-bullet in the first set of bullets as follows:

- o When used to calculate the cracking moment of a member in Article 5.8.3.4.3……..$0.20\sqrt{f'_c}$

Item #2

Revise Article 5.8.3 in various articles as indicated in the attached file.

OTHER AFFECTED ARTICLES:

None

BACKGROUND:

At the request of the Subcommittee on Bridges and Structures at its 2000 Annual Meeting, research into more simplified methods of calculating the shear capacity of concrete bridge members was undertaken. The research was initiated under NCHRP 12-61. The culmination of the research will be publication of NCHRP Report 549, "Simplified Shear Design of Structural Concrete Members".

The proposed provisions reintroduce the Vci and Vcw method for calculating the shear capacity of a member. The proposed relationships differ from the current Standard Specifications in the expression for V_{cw} and the angle θ of the parallel chord truss model used for evaluating V_s. The expressions for V_{cw} were developed so that they can be applied seamlessly to beams with deformed bar reinforcement only, with prestressed reinforcement only, and all combinations of those reinforcements. The angle θ may be conservatively taken as 45 degree or the effects of axial compression on the angle of diagonal cracking may be considered for evaluating the contributions of the shear reinforcement. The new equations are found to offer a more uniform level of safety across a broad range of design cases, a means of assessing the state of cracking under service load levels, and a direct solution for capacity rating.

In addition to the changes presented above, the only other significant change is that the maximum allowable shear design stress is reduced to $0.18f'_c$ unless the member is cast integral with the support or there is a direct check of diagonal compressive strength using a strut-and-tie approach.

ANTICIPATED EFFECT ON BRIDGES:

Improved ease in calculating shear capacity

REFERENCES:

NCHRP Report 549

OTHER:

None

5.8.3 Sectional Design Model

5.8.3.1 General

The sectional design model may be used for shear design where permitted in accordance with the provisions of Article 5.8.1.

In lieu of the methods specified herein, the resistance of members in shear or in shear combined with torsion may be determined by satisfying the conditions of equilibrium and compatibility of strains and by using experimentally verified stress-strain relationships for reinforcement and for diagonally cracked concrete. Where consideration of simultaneous shear in a second direction is warranted, investigation shall be based either on the principles outlined above or on a three-dimensional strut-and-tie model.

C5.8.3.1

In the sectional design approach, the component is investigated by comparing the factored shear force and the factored shear resistance at a number of sections along its length. Usually this check is made at the tenth points of the span and at locations near the supports.

See Article 5.10.11.4.1c for additional requirements for Seismic Zones 3 and 4.

An appropriate nonlinear finite element analysis or a detailed sectional analysis would satisfy the requirements of this article. More information on appropriate procedures and a computer program that satisfies these requirements are given by Collins and Mitchell (1991). One possible approach to the analysis of biaxial shear and other complex loadings on concrete members is outlined in Rabbat and Collins (*1978*), and a corresponding computer-aided solution is presented in Rabbat and Collins (*1976*). A discussion of the effect of biaxial shear on the design of reinforced concrete beam-to-column joints can be found in Pauley and Priestley (*1992*).

5.8.3.2 Sections Near Supports

The provisions of Article 5.8.1.2 shall be considered.

Where the reaction force in the direction of the applied shear introduces compression into the end region of a member the location of the critical section for shear shall be taken as d_v from the internal face of the support as illustrated in Figure 1.

C5.8.3.2

Loads close to the support are transferred directly to the support by compressive arching action without causing additional stresses in the stirrups.

The traditional approach to proportioning transverse reinforcement involves the determination of the required stirrup spacing at discrete sections along the member. The stirrups are then detailed such that this spacing is not exceeded over a length of the beam extending from the design section to the next design section out into the span. In such an approach, the shear demand and resistance provided is assumed to be as shown in Figure C1. There are, however, more theoretically exact stirrup designs. Knowledge of these may help to reconcile published research to traditional design practice.

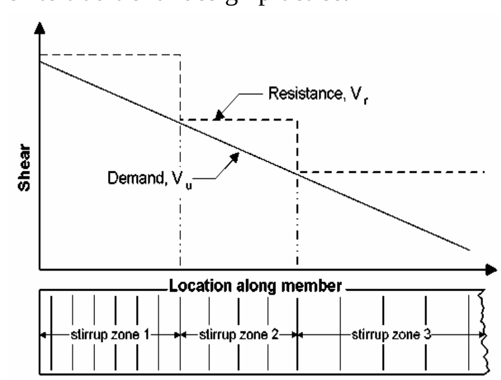


Figure C5.8.3.2-1 Traditional Shear Design.

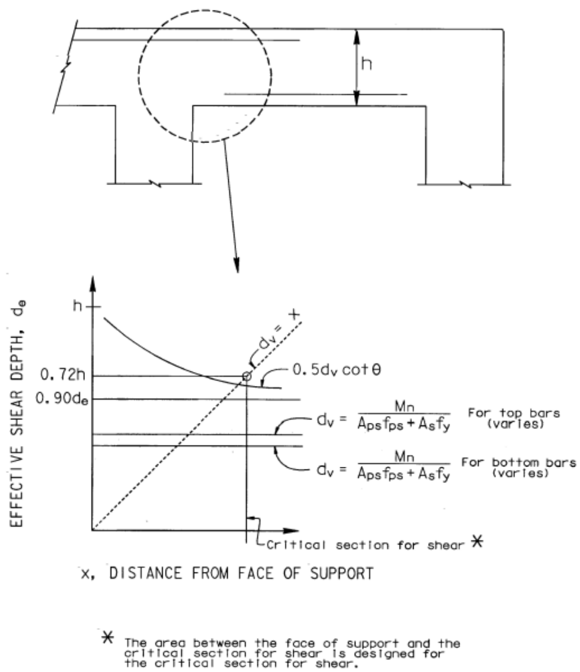


Figure 5.8.3.2-1 Critical Section for Shear.

Otherwise, the design section shall be taken at the internal face of the support. Where the beam-type element extends on both sides of the reaction area, the design section on each side of the reaction shall be determined separately based upon the loads on each side of the reaction and whether their respective contribution to the total reaction introduces tension or compression into the end region.

For post-tensioned beams, anchorage zone reinforcement shall be provided as specified in Article 5.10.9. For pretensioned beams, a reinforcement cage confining the ends of strands shall be provided as specified in Article 5.10.10. For nonprestressed beams supported on bearings that introduce compression into the member, only minimal transverse reinforcement may be provided between the inside edge of the bearing plate or pad and the end of the beam.

Unlike flexural failures, shear failures occur over an inclined plane and a shear crack typically intersects a number of stirrups. The length of the failure along the longitudinal axis of the member is approximately $d_v \cot\theta$. Each of the stirrups intersected by this crack participates in resisting the applied shear. The relationship between the location of the design section and the longitudinal zone of stirrups that resist the shear at that design section is a function of the vertical position of the load applied to the member, including its selfweight, and the projection along the longitudinal axis of the beam of the inclined cracks at that location. Ideally, the design section could be located by determining where the vertical centroid of the applied loads intersects a shear crack inclined at an angle θ as shown in Figure C2.

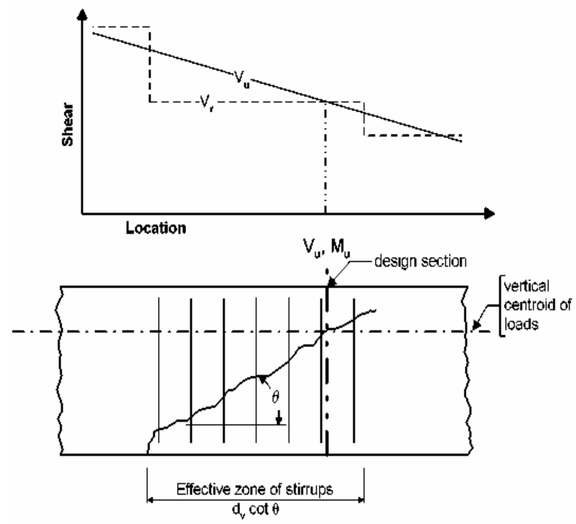


Figure C5.8.3.2-2 Theoretical Shear Design Section Location.

For typical cases where the applied load acts at or above the middepth of the member, it is more practical to take the traditional approach as shown in Figure C1 or a more liberal yet conservative approach as shown in Figure C3. The approach taken in Figure C3 has the effect of extending the required stirrup spacing for a distance of $0.5 d_v \cot\theta$ toward the bearing.

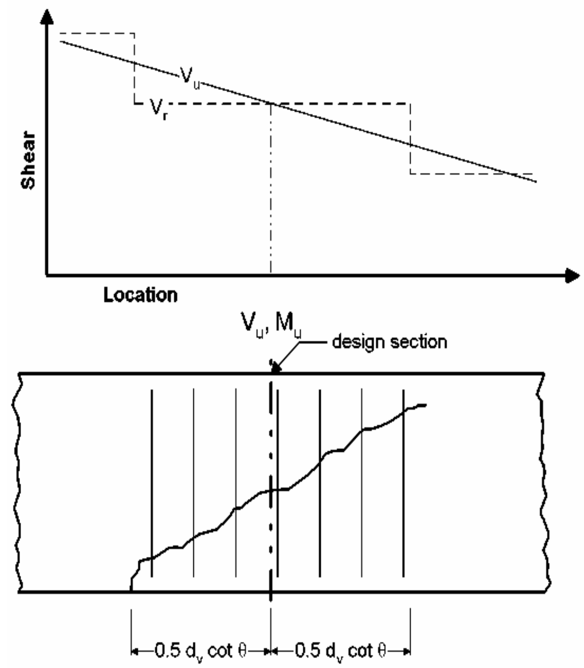


Figure C5.8.3.2-3 Simplified Design Section For Loads Applied at or Above the Middepth of the Member.

If the significant portion of the loads being resisted by the member are applied at a bearing resting on top of the member, the shear failure zone extends for a distance of approximately d_v cot θ beyond the point of load application as shown in Figure C4. As with the previous case, all of the stirrups falling within the failure zone may be assumed effective in resisting the applied shear force. The traditional approach shown in Figure C1 is even more conservative in this case.

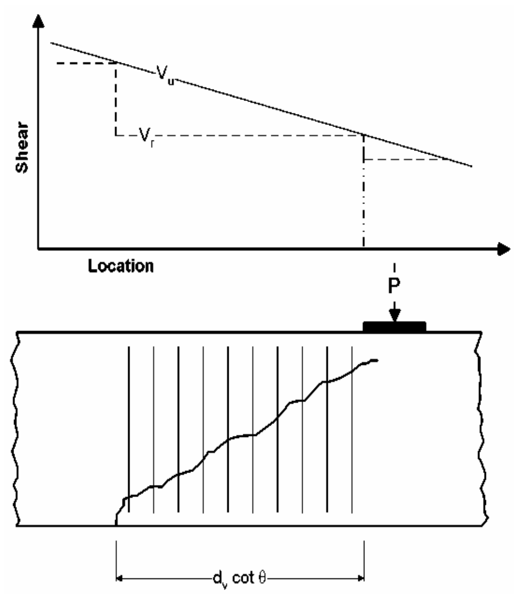


Figure C5.8.3.2-4 Effective Transverse Reinforcement to Members Subjected Primarily to Concentrated Loads.

Figure C5 shows a case where an inverted T-beam acts as a pier cap and the longitudinal members are supported by the flange of the T. In this case, a significant amount of the load is applied below the middepth of the member, and it is more appropriate to use the traditional approach to shear design shown in Figure C1.

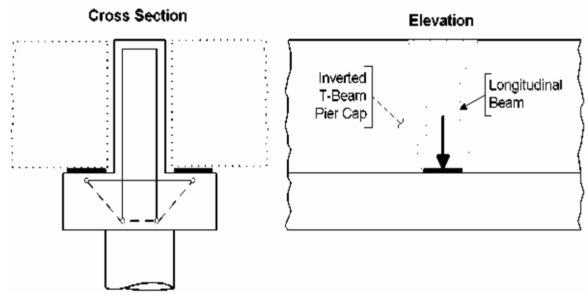


Figure C5.8.3.2-5 Inverted T-Beam Pier Cap.

The T-beam pier cap shown in Figure C5 acts as a beam ledge and should be designed for the localized effects caused by the concentrated load applied to the T-beam flange. Provisions for beam ledge design are given in Article 5.13.2.5.

If the shear stress at the design section calculated in accordance with 5.8.2.9 exceeds $0.18f'_c$ and the beam-type element is not built integrally with the support, its end region shall be designed using the strut-and-tie model specified in Article 5.6.3.

Where a beam is loaded on top and its end is not built integrally into the support, all the shear funnels down into the end bearing. Where the beam has a thin web so that the shear stress in the beam exceeds $0.18f'_c$, there is the possibility of a local diagonal compression or horizontal shear failure along the interface between the web and the lower flange of the beam. Usually the inclusion of additional transverse reinforcement cannot prevent this type of failure and either the section size must be increased or the end of the beam designed using a strut-and-tie model.

5.8.3.3 Nominal Shear Resistance

The nominal shear resistance, V_n, shall be determined as the lesser of:

$$V_n = V_c + V_s + V_p \tag{5.8.3.3-1}$$

$$V_n = 0.25 f'_c b_v d_v + V_p \tag{5.8.3.3-2}$$

in which:

$V_c = 0.0316\beta\sqrt{f'_c}\, b_v d_v$, if the procedures of Articles 5.8.3.4.1 or 5.8.3.4.2 are used (5.8.3.3-3)

V_c = the lesser of V_{ci} and V_{cw}, if the procedures of Article 5.8.3.4.3 are used.

$$V_s = \frac{A_v f_y d_v (\cot\theta + \cot\alpha)\sin\alpha}{s} \tag{5.8.3.3-4}$$

C5.8.3.3

The shear resistance of a concrete member may be separated into a component, V_c, that relies on tensile and shear stresses in the concrete, a component, V_s, that relies on tensile stresses in the transverse reinforcement, and a component, V_p, that is the vertical component of the prestressing force.

The expressions for V_c and V_s apply to both prestressed and nonprestressed sections, with the terms β and θ depending on the applied loading and the properties of the section.

The upper limit of V_n, given by Eq. 2, is intended to ensure that the concrete in the web of the beam will not crush prior to yield of the transverse reinforcement.

where α = 90°, Eq. 4 reduces to:

$$V_s = \frac{A_v f_y d_v \cot\theta}{s} \tag{C5.8.3.3-1}$$

where:

b_v = effective web width taken as the minimum web width within the depth d_v as determined in Article 5.8.2.9 (in.)

d_v = effective shear depth as determined in Article 5.8.2.9 (in.)

s = spacing of stirrups (in.)

β = factor indicating ability of diagonally cracked concrete to transmit tension and shear as specified in Article 5.8.3.4

θ = angle of inclination of diagonal compressive stresses as determined in Article 5.8.3.4 (°); if the procedures of Article 5.8.3.4.3 are used, cot θ is defined therein

α = angle of inclination of transverse reinforcement to longitudinal axis (°)

A_v = area of shear reinforcement within a distance s (in.^{2})

V_p = component in the direction of the applied shear of the effective prestressing force; positive if resisting the applied shear (kip); V_p is taken as zero if the procedures of Article 5.8.3.4.3 are used

The angle θ is, therefore, also taken as the angle between a strut and the longitudinal axis of a member.

5.8.3.4 Procedures for Determining Shear Resistance ~~Determination of β and θ~~

C5.8.3.4

Design for shear may utilize any of the three methods identified herein provided that all requirements for usage of the chosen method are satisfied.

Three complementary methods are given for evaluating shear resistance. Method 1, specified in Article 5.8.3.4.1, as described herein, is only applicable for nonprestressed sections ~~not greater than 16.0 in. deep~~. Method 2, as described in Article 5.8.3.4.2, is applicable for all prestressed and nonprestressed members, with and without shear reinforcement, with and without axial load. Method 3, specified in Article 5.8.3.4.3, is applicable for both prestressed and nonprestressed sections in which there is no net axial tensile load and at least minimum shear reinforcement is provided. Axial load effects can otherwise be accounted for through adjustments to the level of effective precompression stress f_{pc}. In regions of overlapping applicability between the latter 2 methods, Method 3 will generally lead to somewhat more shear reinforcement being required, particularly in areas of negative moment and near points of contraflexure. Method 3 provides a direct capacity rating while Method 2 may require iterative evaluation. If Method 3 leads to an unsatisfactory rating, it is permissible to use Method 2.

5.8.3.4.1 Simplified Procedure for Nonprestressed Sections ~~Not Greater than 16.0 IN Deep~~

For concrete footings in which the distance from point of zero shear to the face of the column, pier or wall is less than $3d_y$ with or without transverse reinforcement, and for other nonprestressed concrete sections not subjected to axial tension and containing at least the minimum amount of transverse reinforcement specified in Article 5.8.2.5, or having an overall depth of less than 16.0 in., the following values may be used:

$\beta = 2.0$
$\theta = 45°$

C5.8.3.4.1

With β taken as 2.0 and θ as 45°, the expressions for shear strength become essentially identical to those traditionally used for evaluating shear resistance. Recent large-scale experiments (*Shioya et al. 1989*), however, have demonstrated that these traditional expressions can be seriously unconservative for large members not containing transverse reinforcement ~~and therefore an overall depth limit of 16.0 in. is imposed for use of this article~~.

5.8.3.4.2 General Procedure

For sections containing at least the minimum amount of transverse reinforcement specified in Article 5.8.2.5, the values of β and θ shall be as specified in Table 1. In using this table, ε_x shall be taken as the calculated longitudinal strain at the middepth of the member when the section is subjected to M_u, N_u, and V_u as shown in Figure 1.

For sections containing less transverse reinforcement than specified in Article 5.8.2.5, the values of β and θ shall be as specified in Table 2. In using this table, ε_x shall be taken as the largest calculated longitudinal strain which occurs within the web of the member when the section is subjected to N_u, M_u, and V_u as shown in Figure 2.

Unless more accurate calculations are made, ε_x shall be determined as:

- If the section contains at least the minimum transverse reinforcement as specified in Article

C5.8.3.4.2

The shear resistance of a member may be determined by performing a detailed sectional analysis that satisfies the requirements of Article 5.8.3.1. Such an analysis, see Figure C1, would show that the shear stresses are not uniform over the depth of the web and that the direction of the principal compressive stresses changes over the depth of the beam. The more direct procedure given herein assumes that the concrete shear stresses are uniformly distributed over an area b_v wide and d_v deep, that the direction of principal compressive stresses (defined by angle θ) remains constant over d_v, and that the shear strength of the section can be determined by considering the biaxial stress conditions at just one location in the web. See Figure C2.

Members containing at least the minimum amount of transverse reinforcement have a considerable capacity to redistribute shear stresses from the most highly strained portion of the cross-section to the less highly strained portions. Because of this capacity to

5.8.2.5:

$$\varepsilon_x = \frac{\left(\frac{|M_u|}{d_v} + 0.5N_u + 0.5|V_u - V_p|\cot\theta - A_{ps}f_{po}\right)}{2(E_sA_s + E_pA_{ps})}$$

(5.8.3.4.2-1)

The initial value of ε_x should not be taken greater than 0.001.

- If the section contains less than the minimum transverse reinforcement as specified in Article 5.8.2.5:

$$\varepsilon_x = \frac{\left(\frac{|M_u|}{d_v} + 0.5N_u + 0.5|V_u - V_p|\cot\theta - A_{ps}f_{po}\right)}{E_sA_s + E_pA_{ps}}$$

(5.8.3.4.2-2)

The initial value of ε_x should not be taken greater than 0.002.

- If the value of ε_x from Eqs. 1 or 2 is negative, the strain shall be taken as:

$$\varepsilon_x = \frac{\left(\frac{|M_u|}{d_v} + 0.5N_u + 0.5|V_u - V_p|\cot\theta - A_{ps}f_{po}\right)}{2\left(E_cA_c + E_sA_s + E_pA_{ps}\right)}$$

(5.8.3.4.2-3)

where:

A_c = area of concrete on the flexural tension side of the member as shown in Figure 1 (in.^{2})

A_{ps} = area of prestressing steel on the flexural tension side of the member, as shown in Figure 1 (in.^{2})

A_s = area of nonprestressed steel on the flexural tension side of the member at the section under consideration, as shown in Figure 1. In calculating A_s for use in this equation, bars which are terminated at a distance less than their development length from the section under consideration shall be ignored (in.^{2})

f_{po} = a parameter taken as modulus of elasticity of prestressing tendons multiplied by the locked-in difference in strain between the prestressing tendons and the surrounding concrete (ksi). For the usual levels of prestressing, a value of 0.7 f_{pu} will be appropriate for both pretensioned and post-tensioned members

N_u = factored axial force, taken as positive if tensile and negative if compressive (kip)

redistribute, it is appropriate to use the middepth of the member as the location at which the biaxial stress conditions are determined. Members that contain no transverse reinforcement, or contain less than the minimum amount of transverse reinforcement, have less capacity for shear stress redistribution. Hence, for such members, it is appropriate to perform the biaxial stress calculations at the location in the web subject to the highest longitudinal tensile strain, see Figure 2.

The longitudinal strain, ε_x, can be determined by the procedure illustrated in Figure C3. The actual section is represented by an idealized section consisting of a flexural tension flange, a flexural compression flange, and a web. The area of the compression flange is taken as the area on the flexure compression side of the member, i.e., the total area minus the area of the tension flange as defined by A_c. After diagonal cracks have formed in the web, the shear force applied to the web concrete, $V_u–V_p$, will primarily be carried by diagonal compressive stresses in the web concrete. These diagonal compressive stresses will result in a longitudinal compressive force in the web concrete of $(V_u–V_p)$ cot θ. Equilibrium requires that this longitudinal compressive force in the web needs to be balanced by tensile forces in the two flanges, with half the force, that is 0.5 $(V_u–V_p)$ cot θ, being taken by each flange. To avoid a trial and error iteration process, it is a convenient simplification to take this flange force due to shear as $V_u–V_p$. This amounts to taking 0.5 cot θ = 1.0 in the numerator of Eqs. 1, 2 and 3. This simplification is not expected to cause a significant loss of accuracy. After the required axial forces in the two flanges are calculated, the resulting axial strains, ε_t and ε_c, can be calculated based on the axial force-axial strain relationship shown in Figure C4.

For members containing at least the minimum amount of transverse reinforcement, ε_x can be taken as:

$$\varepsilon_x = \frac{\varepsilon_t + \varepsilon_c}{2} \tag{C5.8.3.4.2-1}$$

where ε_t and ε_c are positive for tensile strains and negative for compressive strains. If, for a member subject to flexure, the strain ε_c is assumed to be negligibly small, then ε_x becomes one half of ε_t. This is the basis for the expression for ε_x given in Eq. 1. For members containing less than the minimum amount of transverse reinforcement, Eq. 2 makes the conservative simplification that ε_x is equal to ε_t.

In some situations, it will be more appropriate to determine ε_x using the more accurate procedure of Eq. C1 rather than the simpler Eqs. 1 through 3. For example, the shear capacity of sections near the ends of precast, pretensioned simple beams made continuous for live load will be estimated in a very conservative manner by Eqs. 1 through 3 because, at these locations, the prestressing strands are located on the flexural compression side and, therefore, will not be included in A_{ps}. This will result in the benefits of prestressing not

M_u = factored moment, not to be taken less than $V_u d_v$ (kip-in.)

V_u = factored shear force (kip)

Within the transfer length, f_{po} shall be increased linearly from zero at the location where the bond between the strands and concrete commences to its full value at the end of the transfer length.

The flexural tension side of the member shall be taken as the half-depth containing the flexural tension zone, as illustrated in Figure 1.

The crack spacing parameter s_{xe}, used in Table 2, shall be determined as:

$$s_{xe} = s_x \frac{1.38}{a_g + 0.63} \leq 80 \text{ in.} \tag{5.8.3.4.2-4}$$

where:

a_g = maximum aggregate size (in.)

s_x = the lesser of either d_v or the maximum distance between layers of longitudinal crack control reinforcement, where the area of the reinforcement in each layer is not less than $0.003 b_v s_x$, as shown in Figure 3 (in.)

being accounted for by Eqs. 1 through 3.

Absolute value signs were added to Eqs. 1 through 3 in 2004. This notation replaced direction in the nomenclature to take M_u and V_u as positive values. For shear, absolute value signs in Eqs. 1 through 3 are needed to properly consider the effects due to V_u and V_p in sections containing a parabolic tendon path which may not change signs at the same location as shear demand, particularly at midspan.

For pretensioned members, f_{po} can be taken as the stress in the strands when the concrete is cast around them, i.e., approximately equal to the jacking stress. For post-tensioned members, f_{po} can be conservatively taken as the average stress in the tendons when the post-tensioning is completed.

Note that in both Table 1 and Table 2, the values of β and θ given in a particular cell of the table can be applied over a range of values. Thus from Table 1, θ=34.4° and β=2.26 can be used provided that ε_x is not greater than 0.75×10^{-3} and v_u/f'_c is not greater than 0.125. Linear interpolation between the values given in the tables may be used, but is not recommended for hand calculations. Assuming a value of ε_x larger than the value calculated using Eqs. 1, 2 or 3, as appropriate, is permissible and will result in a higher value of θ and a lower value of β. Higher values of θ will typically require more transverse shear reinforcement, but will decrease the tension force required to be resisted by the longitudinal reinforcement. Figure C5 illustrates the shear design process by means of a flow chart. This figure is based on the simplified assumption that $0.5 \cot\theta = 1.0$.

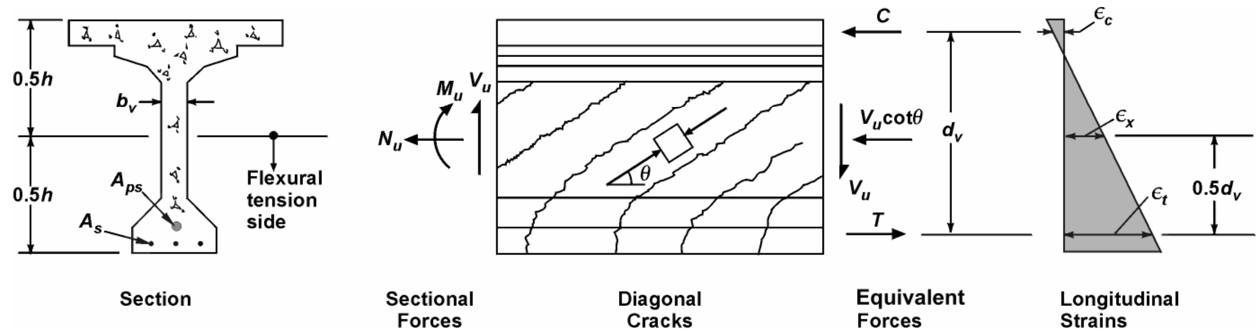
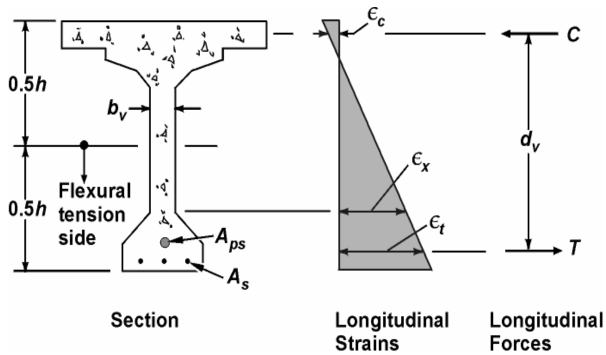


Figure 5.8.3.4.2-1 Illustration of Shear Parameters for Section Containing at Least the Minimum Amount of Transverse Reinforcement, V_p=0.



For sections containing a specified amount of transverse reinforcement, a shear-moment interaction diagram, see Figure C6, can be calculated directly from the procedures in this article. For a known concrete strength and a certain value of ε_x, each cell of Table 1 corresponds to a certain value of v_u/f'_c, i.e., a certain value of V_n. This value of V_n requires an amount of transverse reinforcement expressed in terms of the parameter $A_v f_y/(b_v s)$. The shear capacity corresponding to the provided shear reinforcement can be found by linearly interpolating between the values of V_n corresponding to two consecutive cells where one cell

Figure 5.8.3.4.2-2 Longitudinal Strain, ε_x for Sections Containing Less than the Minimum Amount of Transverse Reinforcement.

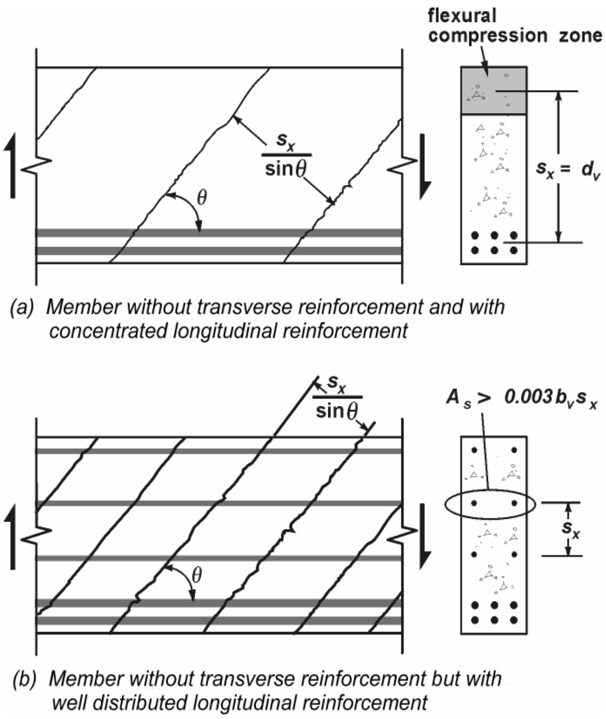


Figure 5.8.3.4.2-3 Definition of Crack Spacing Parameter s_x.

requires more transverse reinforcement than actually provided and the other cell requires less reinforcement than actually provided. After V_n and θ have been found in this manner, the corresponding moment capacity M_n can be found by calculating, from Eqs. 1 through 3, the moment required to cause this chosen value of ε_x, and calculating, from Eq. 5.8.3.5-1, the moment required to yield the reinforcement. The predicted moment capacity will be the lower of these two values. In using Eqs. 5.8.2.9-1, 5.8.3.5-1 and Eqs. 1 through 3 of the procedure to calculate a $V_n - M_n$ interaction diagram, it is appropriate to replace V_u by V_n, M_u by M_n and N_u by N_n and to take the value of ϕ as 1.0. With an appropriate spreadsheet, the use of shear-moment interaction diagrams is a convenient way of performing shear design and evaluation.

The values of β and θ listed in Table 1 and Table 2 are based on calculating the stresses that can be transmitted across diagonally cracked concrete. As the cracks become wider, the stress that can be transmitted decreases. For members containing at least the minimum amount of transverse reinforcement, it is assumed that the diagonal cracks will be spaced about 12.0 in. apart. For members without transverse reinforcement, the spacing of diagonal cracks inclined at $\theta°$ to the longitudinal reinforcement is assumed to be $s_x/\sin\theta$, as shown in Figure 3. Hence, deeper members having larger values of s_x are calculated to have more widely spaced cracks and hence, cannot transmit such high shear stresses. The ability of the crack surfaces to transmit shear stresses is influenced by the aggregate size of the concrete. Members made from concretes that have a smaller maximum aggregate size will have a larger value of s_{xe} and hence, if there is no transverse reinforcement, will have a smaller shear strength.

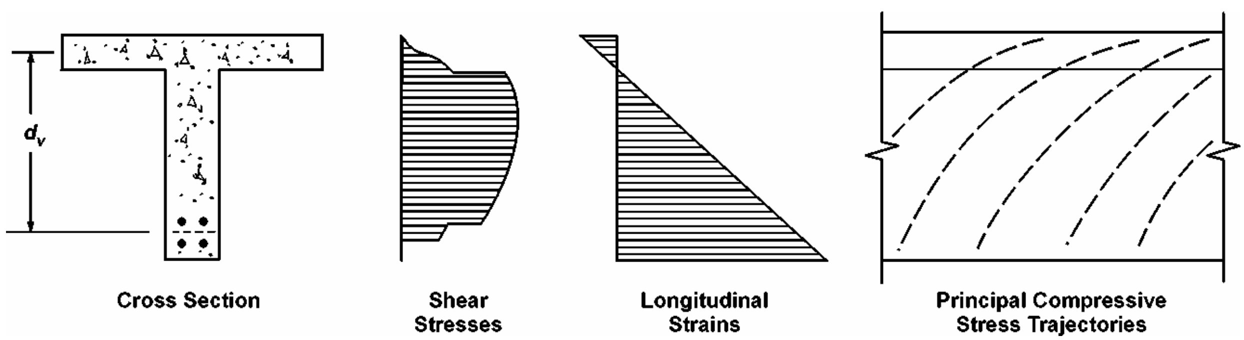


Figure C5.8.3.4.2-1 Detailed Sectional Analysis to Determine Shear Resistance in Accordance with Article 5.8.3.1.

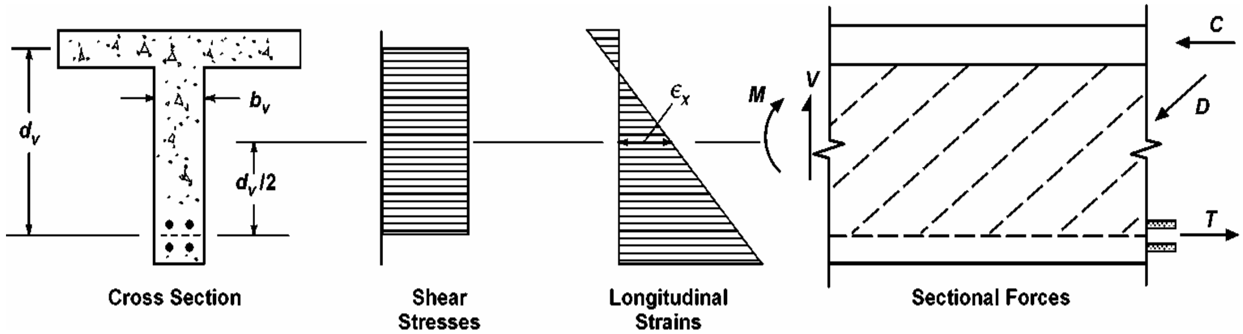


Figure C5.8.3.4.2-2 More Direct Procedure to Determine Shear Resistance in Accordance with Article 5.8.3.4.3.

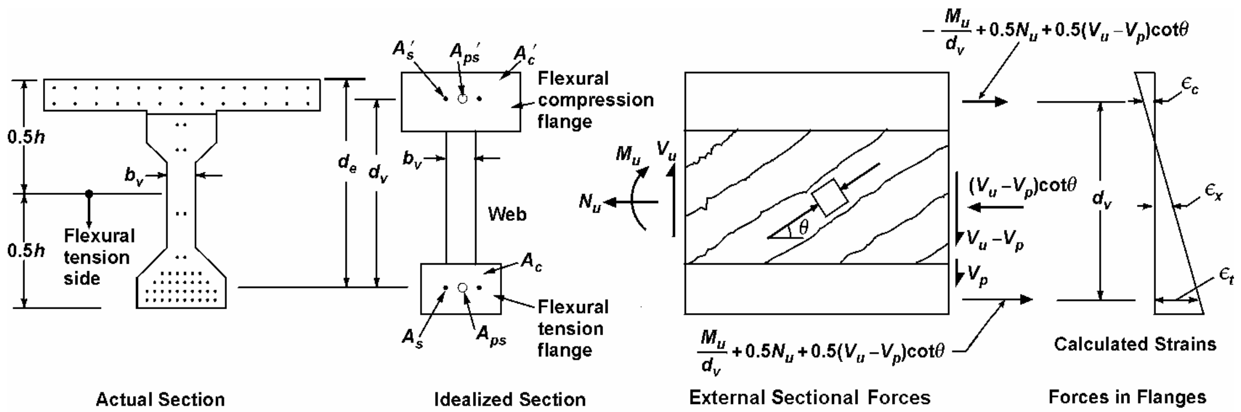


Figure C5.8.3.4.2-3 More Accurate Calculation Procedure for Determining ε_x.

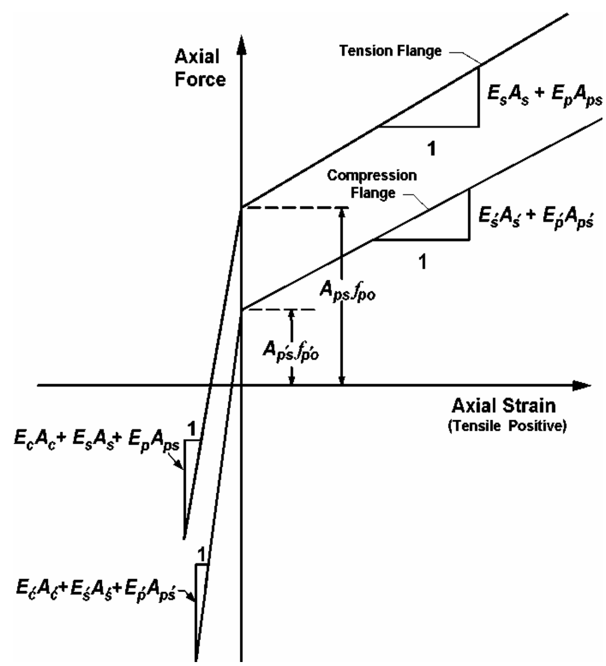


Figure C5.8.3.4.2-4 Assumed Relations Between Axial Force in Flange and Axial Strain of Flange.

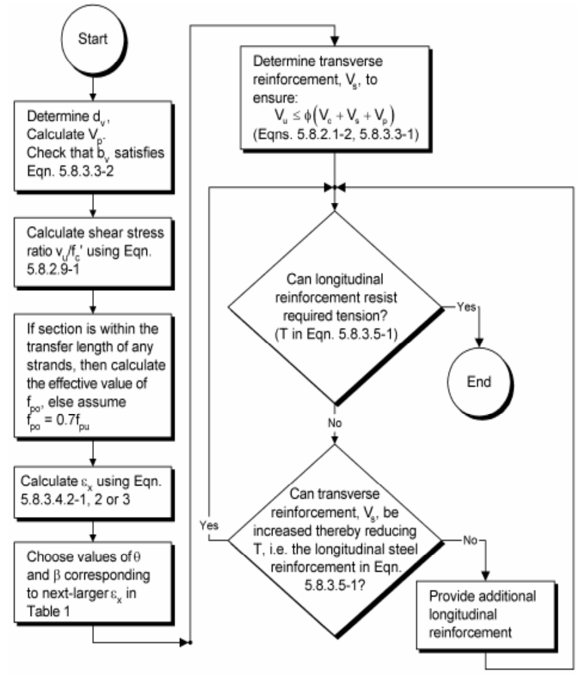


Figure C5.8.3.4.2-5 Flow Chart for Shear Design of Section Containing at Least Minimum Transverse Reinforcement.

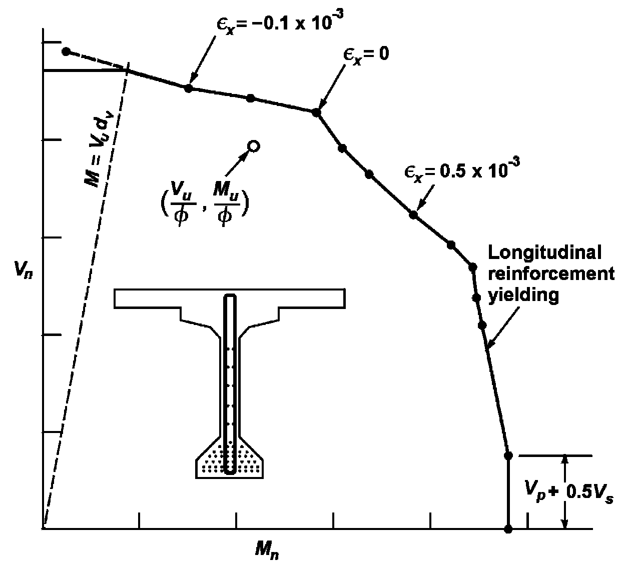


Figure C5.8.3.4.2-6 Typical Shear-Moment Interaction Diagram.

More details on the procedures used in deriving the tabulated values of θ and β are given in Collins and Mitchell (*1991*).

Table 5.8.3.4.2-1 Values of θ and β for Sections with Transverse Reinforcement.

$\frac{v_u}{f'_c}$	$\varepsilon_x \times 1{,}000$								
	≤−0.20	≤−0.10	≤−0.05	≤0	≤0.125	≤0.25	≤0.50	≤0.75	≤1.00
≤0.075	22.3 6.32	20.4 4.75	21.0 4.10	21.8 3.75	24.3 3.24	26.6 2.94	30.5 2.59	33.7 2.38	36.4 2.23
≤0.100	18.1 3.79	20.4 3.38	21.4 3.24	22.5 3.14	24.9 2.91	27.1 2.75	30.8 2.50	34.0 2.32	36.7 2.18
≤0.125	19.9 3.18	21.9 2.99	22.8 2.94	23.7 2.87	25.9 2.74	27.9 2.62	31.4 2.42	34.4 2.26	37.0 2.13
≤0.150	21.6 2.88	23.3 2.79	24.2 2.78	25.0 2.72	26.9 2.60	28.8 2.52	32.1 2.36	34.9 2.21	37.3 2.08
≤0.175	23.2 2.73	24.7 2.66	25.5 2.65	26.2 2.60	28.0 2.52	29.7 2.44	32.7 2.28	35.2 2.14	36.8 1.96
≤0.200	24.7 2.63	26.1 2.59	26.7 2.52	27.4 2.51	29.0 2.43	30.6 2.37	32.8 2.14	34.5 1.94	36.1 1.79
≤0.225	26.1 2.53	27.3 2.45	27.9 2.42	28.5 2.40	30.0 2.34	30.8 2.14	32.3 1.86	34.0 1.73	35.7 1.64
≤0.250	27.5 2.39	28.6 2.39	29.1 2.33	29.7 2.33	30.6 2.12	31.3 1.93	32.8 1.70	34.3 1.58	35.8 1.50

Table 5.8.3.4.2-2 Values of θ and β for Sections with Less than Minimum Transverse Reinforcement.

s_{xe} (in.)	$\varepsilon_x \times 1000$										
	≤–0.20	≤–0.10	≤–0.05	≤0	≤0.125	≤0.25	≤0.50	≤0.75	≤1.00	≤1.50	≤2.00
≤5	25.4 6.36	25.5 6.06	25.9 5.56	26.4 5.15	27.7 4.41	28.9 3.91	30.9 3.26	32.4 2.86	33.7 2.58	35.6 2.21	37.2 1.96
≤10	27.6 5.78	27.6 5.78	28.3 5.38	29.3 4.89	31.6 4.05	33.5 3.52	36.3 2.88	38.4 2.50	40.1 2.23	42.7 1.88	44.7 1.65
≤15	29.5 5.34	29.5 5.34	29.7 5.27	31.1 4.73	34.1 3.82	36.5 3.28	39.9 2.64	42.4 2.26	44.4 2.01	47.4 1.68	49.7 1.46
≤20	31.2 4.99	31.2 4.99	31.2 4.99	32.3 4.61	36.0 3.65	38.8 3.09	42.7 2.46	45.5 2.09	47.6 1.85	50.9 1.52	53.4 1.31
≤30	34.1 4.46	34.1 4.46	34.1 4.46	34.2 4.43	38.9 3.39	42.3 2.82	46.9 2.19	50.1 1.84	52.6 1.60	56.3 1.30	59.0 1.10
≤40	36.6 4.06	36.6 4.06	36.6 4.06	36.6 4.06	41.2 3.20	45.0 2.62	50.2 2.00	53.7 1.66	56.3 1.43	60.2 1.14	63.0 0.95
≤60	40.8 3.50	40.8 3.50	40.8 3.50	40.8 3.50	44.5 2.92	49.2 2.32	55.1 1.72	58.9 1.40	61.8 1.18	65.8 0.92	68.6 0.75
≤80	44.3 3.10	44.3 3.10	44.3 3.10	44.3 3.10	47.1 2.71	52.3 2.11	58.7 1.52	62.8 1.21	65.7 1.01	69.7 0.76	72.4 0.62

5.8.3.4.3 Simplified Procedure for Prestressed and Nonprestressed Sections

For concrete beams not subject to significant axial tension, prestressed and nonprestressed, and containing at least the minimum amount of transverse reinforcement specified in Article 5.8.2.5, V_n in Article 5.8.3.3 may be determined with V_p taken as zero and V_c taken as the lesser of V_{ci} and V_{cw}, where:

V_{ci} = nominal shear resistance provided by concrete when inclined cracking results from combined shear and moment (kip)

V_{cw} = nominal shear resistance provided by concrete when inclined cracking results from excessive principal tensions in web (kip)

- V_{ci} shall be determined as:

$$V_{ci} = 0.02\sqrt{f'_c}\, b_v d_v + V_d + \frac{V_i M_{cre}}{M_{max}} \geq 0.06\sqrt{f'_c}\, b_v d_v \tag{5.8.3.4.3-1}$$

where:

V_d = shear force at section due to unfactored dead load and includes both DC and DW (kip)

V_i = factored shear force at section due to externally applied loads occurring simultaneously with M_{max} (kip)

M_{cre} = moment causing flexural cracking at section due to externally applied loads (kip-in)

M_{max} = maximum factored moment at section due to externally applied loads (kip-in)

M_{cre} shall be determined as:

~~M_{cr} = $(I_c/y_t)(0.2\sqrt{f'_c} + f_{pe} - f_d)$~~ (5.8.3.4.3-2)

$$M_{cre} = S_c \left(f_r + f_{cpe} - \frac{M_{dnc}}{S_{nc}} \right)$$

where:

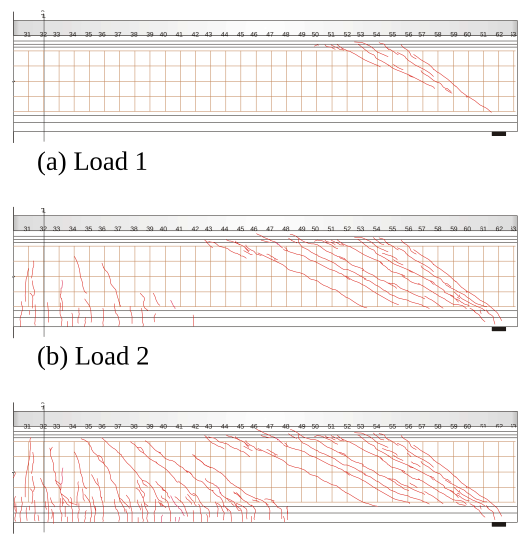
~~I_c = moment of inertia of section resisting externally applied factored loads (in^4)~~

~~y_t = distance from centroidal axis of gross section resisting externally applied factored loads, neglecting reinforcement, to extreme fiber in~~

C5.8.3.4.3

Article 5.8.3.4.3 is based on the recommendations of NCHRP Report 549 (*Hawkins et al.* 2005). The concepts of this article are compatible with the concepts of ACI Code 318-05 and AASHTO Standard Specifications for Highway Bridges 1996 for evaluations of the shear resistance of prestressed concrete members. However, those concepts are modified so that this article applies to both prestressed and nonprestressed sections.

The nominal shear resistance V_n is the sum of the shear resistances V_c and V_s provided by the concrete and shear reinforcement, respectively. Both V_c and V_s depend on the type of inclined cracking that occurs at the given section. There are two types of inclined cracking: flexure-shear cracking and web-shear cracking for which the associated resistances are V_{ci} and V_{cw}, respectively. Figure C1 shows the development of both types of cracking when increasing uniform load was applied to a 63-inch bulb-tee girder. NCHRP Report XX2 (*Hawkins et al.*2005).



(a) Load 1

(b) Load 2

(c) Load 3

Figure C5.8.3.4.3-1 – Development of Shear Cracking with Increasing Loads for Uniformly Loaded Bulb Tee Beam. Load 1< Load 2< Load 3.

Web-shear cracking begins from an interior point in the web of the member before either flange in that region cracks in flexure. In Figure C1, at load 1, web-shear cracking developed in the web of the member adjacent to the end support. Flexure-shear cracking is initiated by flexural cracking. Flexural cracking increases the shear stresses in the concrete above the flexural crack. In Figure C1, flexural cracking had developed in the central region of the beam by load 2 and by load 3, the flexural cracks had become inclined cracks as flexural cracking extended towards the end support with increasing load

~~tension (in)~~

~~f_{pe} = compressive stress in concrete due to effective prestress forces only (after allowance for all prestress losses), at extreme fiber of section where tensile stress is caused by externally applied loads (ksi)~~

~~f_d = stress due to unfactored dead load, at extreme fiber of section where tensile stress is caused by externally applied loads (ksi)~~

f_{cpe} = compressive stress in concrete due to effective prestress forces only (after allowance for all prestress losses) at extreme fiber of section where tensile stress is caused by externally applied loads (ksi)

M_{dnc} = total unfactored dead load moment acting on the monolithic or noncomposite section (kip-ft.)

S_c = section modulus for the extreme fiber of the composite section where tensile stress is caused by externally applied loads (in.^{3})

S_{nc} = section modulus for the extreme fiber of the monolithic or noncomposite section where tensile stress is caused by externally applied loads (in.^{3})

In Eq. 5.8.3.4.3-1, M_{max} and V_i shall be determined from the load combination causing maximum moment at the section.

- V_{cw} shall be determined as:

$$V_{cw} = (0.06\sqrt{f_c^{'}} + 0.30 f_{pc})b_v d_v + V_p \tag{5.8.3.4.3-3}$$

where:

f_{pc} = compressive stress in concrete (after allowance for all prestresss losses) at centroid of cross section resisting externally applied loads or at junction of web and flange when the centroid lies within the flange (ksi). In a composite member, f_{pc} is the resultant compressive stress at the centroid of the composite section, or at junction of web and flange, due to both prestresss and moments resisted by precast member acting alone.

- V_s shall be determined using Eq. 5.8.3.3-4

For sections with shear reinforcement equal to or greater than that required by Article 5.8.2.5, the shear carried by the concrete may drop below V_c shortly after inclined cracking, and the shear reinforcement may yield locally. However, sections continue to resist increasing shears until resistances provided by the concrete again reach V_c. Thus, V_{ci} and V_{cw} are measures of the resistance that can be provided by the concrete at the nominal shear resistance of the section and are not directly equal to the shears at inclined cracking.

The angle θ of the inclined crack, and therefore of the diagonal compressive stress, is less for a web-shear crack than a flexure-shear crack. Consequently, for a given section the value of V_s associated with web-shear cracking is greater than that associated with flexure-shear cracking.

V_{ci} is the sum of the shear ($V_i M_{cr}/M_{max}$) required to cause flexural cracking at the given section plus the increment of shear necessary to develop the flexural crack into a shear crack, For a non-composite beam, the total cross section resists all applied shears, dead and live, I_c equals the moment of inertia of the gross section and V_d equals the unfactored dead load shear acting on the section. In this case Eq. 1 can be used directly.

For a composite beam, part of the dead load is resisted by only part of the final section. Where the final gross concrete section is achieved with only one addition to the initial concrete section, (two-stage construction), Eq. 1 can be used directly. In Eq. 2 appropriate section properties are used to compute f_d and in Eq. 1 the shear due to dead load V_d and that due to other loads V_i are separated. V_d is the total shear force due to unfactored dead loads acting on the part of the section carrying the dead loads acting prior to composite action plus the unfactored superimposed dead load acting on the composite member. The term V_i may be taken as (V_u -V_d) and M_{max} as $M_u – M_d$ where V_u and M_u are the factored shear and moment at the given section due to the total factored loads M_d is the moment due to unfactored dead load at the same section.

Where the final gross section is developed with more than one concrete composite addition to the initial section (multiple-stage construction), it is necessary to trace the build up of the extreme fiber flexural stresses to compute M_{cr}. For each stage in the life history of the member, the increments in the extreme fiber flexural stress at the given section due to the unfactored loads acting on that section are calculated using the section properties existing at that stage. V_d, V_i and M_{max} are calculated in the same manner as for two-stage construction.

A somewhat lower modulus of rupture is used in evaluating M_{cre} by Eq. 2 to account for the effects of differential shrinkage between the slab and the girder, and the effects of thermal gradients that can occur over the depth of the girder.

with cot θ taken as follows:

Where V_{ci} is less than V_{cw}, cot θ = 1.0

Where V_{ci} is greater than V_{cw},

$$\cot\theta = 1.0 + 3(f_{pc} / \sqrt{f'_c}) \leq 1.8 \qquad (5.8.3.4.3\text{-}4)$$

5.8.3.5 Longitudinal Reinforcement

At each section the tensile capacity of the longitudinal reinforcement on the flexural tension side of the member shall be proportioned to satisfy:

$$A_{ps} f_{ps} + A_s f_y \geq \frac{|M_u|}{d_v \phi} + 0.5\frac{N_u}{\phi} + \left(\left|\frac{V_u}{\phi} - V_p\right| - 0.5V_s\right)\cot\theta \qquad (5.8.3.5\text{-}1)$$

where:

V_s = shear resistance provided by the transverse reinforcement at the section under investigation as given by Eq. 5.8.3.3-4, except V_s shall not be taken as greater than V_u/ϕ (kip)

θ = angle of inclination of diagonal compressive stresses used in determining the nominal shear resistance of the section under investigation as determined by Article 5.8.3.4 (°); if the procedures of Article 5.8.3.4.3 are used, cot θ is defined therein

$\phi_f \phi_v \phi_c$ = resistance factors taken from Article 5.5.4.2 as appropriate for moment, shear and axial resistance

The area of longitudinal reinforcement on the flexural tension side of the member need not exceed the area required to resist the maximum moment acting alone. This provision applies where the reaction force or the load introduces direct compression into the flexural compression face of the member.

Eq. 1 shall be evaluated where simply-supported girders are made continuous for live loads. Where longitudinal reinforcement is discontinuous, Eq. 1 shall be reevaluated.

C5.8.3.5

Shear causes tension in the longitudinal reinforcement. For a given shear, this tension becomes larger as θ becomes smaller and as V_c becomes larger. The tension in the longitudinal reinforcement caused by the shear force can be visualized from a free-body diagram such as that shown in Figure C1.

Taking moments about Point 0 in Figure C1, assuming that the aggregate interlock force on the crack, which contributes to V_c, has a negligible moment about Point 0, and neglecting the small difference in location of V_u and V_p leads to the requirement for the tension force in the longitudinal reinforcement caused by shear.

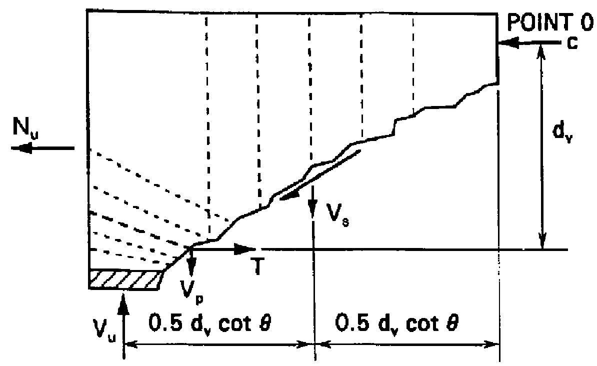


Figure C5.8.3.5-1 Forces Assumed in Resistance Model Caused by Moment and Shear.

At maximum moment locations, the shear force changes sign, and hence the inclination of the diagonal compressive stresses changes. At direct supports including simply-supported girder ends and bent/pier caps pinned to columns, and at loads applied directly to the top or bottom face of the member, this change of inclination is associated with a fan-shaped pattern of compressive stresses radiating from the point load or the direct support as shown in Figure C2. This fanning of the diagonal stresses reduces the tension in the longitudinal reinforcement caused by the shear; i.e., angle θ becomes steeper. The tension in the reinforcement does not exceed that due to the maximum moment alone. Hence, the longitudinal reinforcement requirements can be met by extending the flexural reinforcement for a distance of $d_v\cot\theta$ or as specified in Article 5.11, whichever is greater.

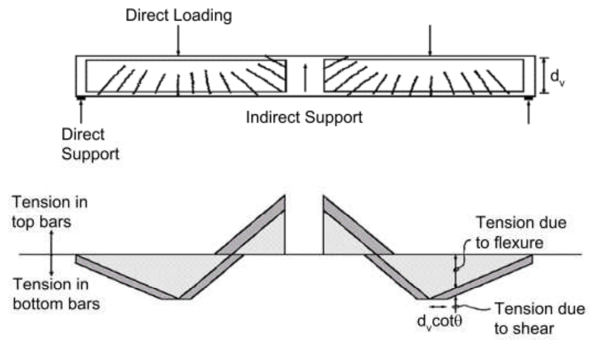


Figure C5.8.3.5-2 Force Variation in Longitudinal Reinforcement Near Maximum Moment Locations.

At the inside edge of the bearing area of simple end supports to the section of critical shear, the longitudinal reinforcement on the flexural tension side of the member shall satisfy:

$$A_s f_y + A_{ps} f_{ps} \geq \left(\frac{V_u}{\phi_v} - 0.5V_s - V_p \right) \cot\theta \qquad (5.8.3.5\text{-}2)$$

Eqs. 1 and 2 shall be taken to apply to sections not subjected to torsion. Any lack of full development shall be accounted for.

In determining the tensile force that the reinforcement is expected to resist at the inside edge of the bearing area, the values of V_u, V_s, V_p, and θ, calculated for the section d_v from the face of the support may be used. In calculating the tensile resistance of the longitudinal reinforcement, a linear variation of resistance over the development length of Article 5.11.2.1.1 or the bi-linear variation of resistance over the transfer and development length of Article 5.11.4.2 may be assumed.

5.8.3.6 Sections Subjected to Combined Shear and Torsion

5.8.3.6.1 Transverse Reinforcement

The transverse reinforcement shall not be less than the sum of that required for shear, as specified in Article 5.8.3.3, and for the concurrent torsion, as specified in Articles 5.8.2.1 and 5.8.3.6.2.

C5.8.3.6.1

The shear stresses due to torsion and shear will add on one side of the section and offset on the other side. The transverse reinforcement is designed for the side where the effects are additive.

Usually the loading that causes the highest torsion differs from the loading that causes the highest shear. Although it is sometimes convenient to design for the highest torsion combined with the highest shear, it is only necessary to design for the highest shear and its concurrent torsion, and the highest torsion and its concurrent shear.

5.8.3.6.2 Torsional Resistance

The nominal torsional resistance shall be taken as:

$$T_n = \frac{2 A_o A_t f_y \cot\theta}{s} \qquad (5.8.3.6.2\text{-}1)$$

where:

A_o = area enclosed by the shear flow path, including any area of holes therein (in.^{2})

A_t = area of one leg of closed transverse torsion reinforcement in solid members, or total area of transverse torsion reinforcement in the exterior web of cellular members (in.^{2})

θ = angle of crack 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 (°)

5.8.3.6.3 Longitudinal Reinforcement

The provisions of Article 5.8.3.5 shall apply as amended, herein, to include torsion.

The longitudinal reinforcement in solid sections shall be proportioned to satisfy Eq. 1:

$$A_{ps} f_{ps} + A_s f_y \geq \frac{|M_u|}{\phi d_v} + \frac{0.5 N_u}{\phi} + \cot\theta \sqrt{\left(\left|\frac{V_u}{\phi} - V_p\right| - 0.5 V_s\right)^2 + \left(\frac{0.45 p_h T_u}{2 A_o \phi}\right)^2} \quad (5.8.3.6.3\text{-}1)$$

In box sections, longitudinal reinforcement for torsion, in addition to that required for flexure, shall not be less than:

$$A_\ell = \frac{T_n p_h}{2 A_o f_y} \quad (5.8.3.6.3\text{-}2)$$

where:

p_h = perimeter of the centerline of the closed transverse torsion reinforcement (in.)

C5.8.3.6.3

To account for the fact that on one side of the section the torsional and shear stresses oppose each other, the equivalent tension used in the design equation is taken as the square root of the sum of the squares of the individually calculated tensions in the web.

REFERENCES – To be added to end of Section 5

Hawkins, N.M., D.A. Kuchma, R.F. Mast, M.L. Marsh, and K-H. Reineck. Simplified Shear Design of Structural Concrete Members. NCHRP Report XX1. TRB, National Research Council, Washington, D.C. 2005

Hawkins, N.M., D.A. Kuchma, H.G. Russell, G.J. Klein, and N.S. Anderson. Application of the LRFD Bridge Design Specifications to High-Strength Structural Concrete: Shear Provisions. NCHRP Report XX2. TRB, National Research Council, Washington, D.C. 2006.

2006 AASHTO BRIDGE COMMITTEE AGENDA ITEM: 11

SUBJECT: LRFD Bridge Design Specifications: Section 5, Article 5.10.8 (WAI 107)

TECHNICAL COMMITTEE: T-10 Concrete

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

☒ **DESIGN SPEC** ☐ **CONSTRUCTION SPEC** ☐ **MOVABLE SPEC**
☐ **LRFR MANUAL** ☐ **OTHER**

☒ **US VERSION** ☐ **SI VERSION** ☐ **BOTH**

DATE PREPARED: 1/5/06
DATE REVISED:

AGENDA ITEM:

Item #1

Modify Article 5.10.8 as follows:

5.10.8 SHRINKAGE AND TEMPERATURE REINFORCEMENT

Reinforcement for shrinkage and temperature stresses shall be provided near surfaces of concrete exposed to daily temperature changes and in structural mass concrete. Temperature and shrinkage reinforcement shall ensure that the total reinforcement on the exposed surfaces is not less than that specified herein.

Reinforcement for shrinkage and temperature may be in the form of bars, welded wire fabric or prestressing tendons.

For bar or welded wire fabric, the area of reinforcement per foot, on each face and in each direction shall satisfy:

$$A_s \geq \frac{1.30bh}{2(b+h)f_y} \qquad (5.10.8\text{-}1)$$

$$0.11 \leq A_s \leq 0.60 \qquad (5.10.8\text{-}2)$$

where:

A_s = area of reinforcement in each direction and each face (in^2/ft)
b = least width of component section (in.)
h = least thickness of component section (in.)
f_y = specified yield strength of reinforcing bars $\leq$ 75 ksi

<u>Where the least dimension varies along the length of wall, footing, or other component, multiple sections should be examined to represent the average condition at each section. Spacing shall not exceed:</u>

- <u>3.0 times the component thickness, or 18.0 in.</u>
- <u>12.0 in. for walls and footings greater than 18.0 in. thick</u>
- <u>12.0 in. for other components greater than 36.0 in. thick</u>

<u>For components 6.0 in. or less in thickness the minimum steel specified may be placed in a single layer. Shrinkage</u>

and temperature steel shall not be required for:

- End face of walls 18 in. or less in thickness
- Side faces of buried footings 36 in. or less in thickness
- Faces of all other components, with smaller dimension less than or equal to 18.0 in.

~~For components 6.0 in. or less in thickness the minimum steel specified may be placed in a single layer. Faces of components~~ that ~~which are less than 18.0 in. thick need not be reinforced for shrinkage and temperature effects. Shrinkage and temperature reinforcement shall not be spaced further apart than 3.0 times the component thickness or 18.0 in. For components over 36.0 in. thickness, the shrinkage and temperature reinforcement shall be not spaced further apart that 12.0 in.~~

If prestressing tendons are used as steel for shrinkage and temperature reinforcement, the tendons shall provide a minimum average compressive stress of 0.11 ksi on the gross concrete area through which a crack plane may extend, based on the effective prestress after losses. Spacing of tendons should not exceed either 72.0 in. or the distance specified in Article 5.10.3.4. Where the spacing is greater than 54.0 in., bonded reinforcement shall be provided between tendons, for a distance equal to the tendon spacing.

~~For solid structural concrete walls and footings, bar spacing shall not exceed 12.0 in. in each direction on all faces greater than 18.0 in. thick, and the area of shrinkage and temperature steel shall satisfy the requirements herein, with *b* and *h* being the least dimensions of any section through the component. Where the least dimensions vary along the length of the wall or footing, multiple sections should be examined to represent the average condition at each section of wall or footing.~~

OTHER AFFECTED ARTICLES:

None

BACKGROUND:

See 2005 AASHTO Agenda Item #25 (Revision 1)

ANTICIPATED EFFECT ON BRIDGES:

Further clarifies and simplifies application of this specification resulting in improved crack control.

REFERENCES:

None

OTHER:

None

2006 AASHTO BRIDGE COMMITTEE AGENDA ITEM: 12

SUBJECT: LRFD Bridge Design Specifications: Section 5, Article 5.10.11.4.1b (WAI 99)

TECHNICAL COMMITTEE: T-10 Concrete

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

☒ **DESIGN SPEC** ☐ **CONSTRUCTION SPEC** ☐ **MOVABLE SPEC**
☐ **LRFR MANUAL** ☐ **OTHER**

☒ **US VERSION** ☐ **SI VERSION** ☒ **BOTH**

DATE PREPARED: 1/5/06
DATE REVISED:

AGENDA ITEM:

Item #1

Revise Article 5.10.11.4.1b as follows:

The biaxial strength of columns shall not be less than that required for flexure, as specified in Article 3.10.9.4. The column shall be investigated for both extreme load cases, as specified in Article 3.10.8, at the extreme event limit state. The resistance factors of Article 5.5.4.2 shall be replaced for ~~both spirally and tied reinforcement~~ columns with either spiral or tie reinforcement by the value of 0.5 where the extreme factored axial load for the column exceeds $0.20f'_cA_g$. The ~~value of ϕ~~ resistance factor may be increased linearly from 0.50 to ~~0.90~~ the value of ϕ specified in Article 5.5.4.2 for flexure with no axial load when the extreme factored axial load is between $0.20f'_cA_g$ and 0.0.

Item #2

Revise Article C5.10.11.4.1b as follows:

Columns are required to be designed biaxially and to be investigated for both the minimum and maximum axial forces. For columns with a maximum axial ~~stress~~ load exceeding $0.20f'_cA_g$, the resistance factor, ϕ, is reduced to 0.50 as shown in Figure C1. This requirement was added because of the trend toward a reduction in ductility capacity as the axial load increases. Implicit in this requirement is the recommendation that design axial force be less than $0.20f'_cA_g$. Columns with axial forces greater than this value are permitted, but they are designed for higher force levels, i.e., lower ϕ factor, in lieu of the lower ductility capacity.

On the y-axis of Figure C1, the origins of the solid lines are the values of ϕ specified in Article 5.5.4.2.1 for tension-controlled prestressed and nonprestressed members. The value of ϕ to be used on the y-axis is determined by the strain condition at a cross-section at nominal flexural strength with no axial load. If the analysis indicates a tension-controlled member in pure flexure, the equations shown for the solid lines in Figure C1 may be used to calculate the value of ϕ to be used in the column design based on the maximum extreme factored axial load. If the cross-section analyzed for pure flexure is compression-controlled, or is in the transition region between tension-controlled and compression-controlled members, the appropriate value of ϕ at an axial load of 0.0 may be calculated by Eqs. 5.5.4.2.1-1 or 5.5.4.2.1-2. The value of ϕ to be used in the column design is then interpolated between this value of ϕ at 0.0 axial load and 0.50 at an axial load of $0.20f'_cA_g$, as indicated by the dashed line in Figure C1.

Item #3

Revise Figure C5.10.11.4.1b-1 as follows:

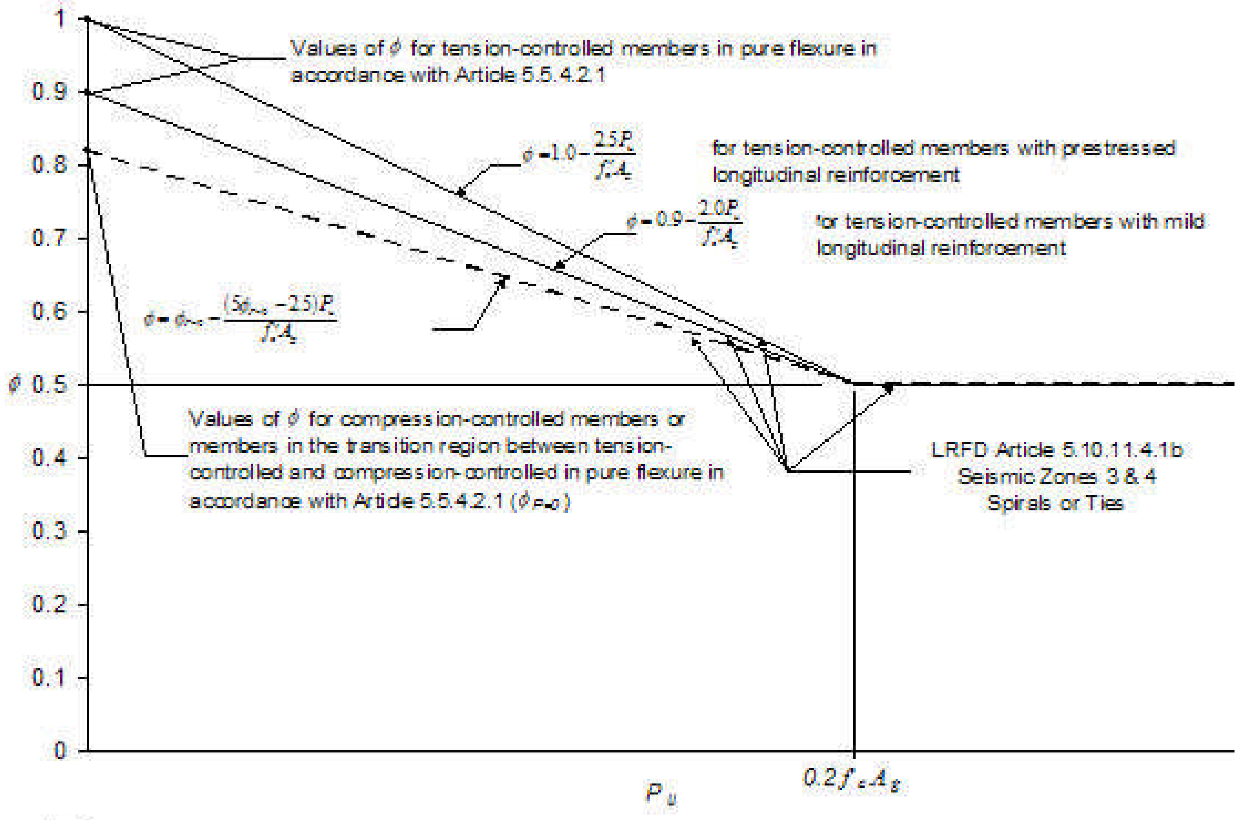


Figure C5.10.11.4.1b-1 Variation of Resistance Factor in Seismic Zones 3 and 4.

OTHER AFFECTED ARTICLES:

None

BACKGROUND:

In the 2006 Interim to the Third Edition of the LRFD Bridge Design Specifications, changes were made to the manner in which resistance factors for prestressed and non-prestressed members are determined. In lieu of determining the resistance factor based on the type of loading, ϕ is now determined by the strain condition at a cross-section at nominal strength. Consequently, the linear variation of ϕ between axial load levels of 0.0 and $0.10f'_cA_g$ is no longer applicable, and is proposed to be removed from Figure C5.10.11.4.1b-1.

In existing Figure C5.10.11.4.1b-1, the assumption is made that a column with zero axial load is under-reinforced, hence $\phi = 0.9$. This was previously true as, prior to the 2006 Interim, over-reinforced nonprestressed members were not allowed. This is no longer the case. The proposed change requires that ϕ for the cross-section at nominal flexural strength with zero axial load be calculated to establish the starting point for the interpolation of ϕ for the column design in Seismic Zones 3 & 4. This is consistent with the changes made in the 2006 Interim, since reduced ductility is compensated with increasing over-strength (i.e., a reduced ϕ factor). The change also provides for the potential use of prestressed columns.

As an example, three square and three circular column sizes were considered. Three square columns were analyzed at the maximum allowed reinforcement ratio of about 6%. The longitudinal bars were assumed to be centered 3.5” from each face of the column, and were distributed evenly about the perimeter. Three circular columns were analyzed for 1% and 4% reinforcement with the longitudinal bars at 2.5” from face of column. The table below represents the results of a non-linear strain compatibility analysis of the strain condition of each member at nominal flexural strength and 0.0 axial load. The 18” square column is in the transition region between tension-controlled and compression-controlled members. The y-axis value of ϕ in Figure C1 would be 0.88 (see the dashed line), and the interpolated value of ϕ for the column design would be between 0.88 at an axial load of 0.0 and 0.50 at an axial load of $0.20f'_cA_g$. The resistance factors for circular columns at zero axial load would be 0.9 in all cases except the 24” diameter column with (12) #11 is slightly less. The resistance factor decreases to 0.5 when column axial load increases to $0.2f'_cA_g$ as shown in Figure C1.

Column Size	Bars	c (in)	d_t (in)	ε_t	ϕ
18” Square	(12) #11	5.67	14.5	0.00468	0.88
25” Square	(24) #11	7.35	21.5	0.00578	0.90
35” Square	(32) #14	9.27	31.5	0.00720	0.90
24” Dia. Column	(6) #9	5.39	21.31	0.00897	0.90
24” Dia. Column	(12)#11	8.02	21.17	0.00492	0.896
48” Dia. Column	(46)#11	8.78	45.17	0.01248	0.90
48” Dia. Column	(26)#11	14.9	45.17	0.006093	0.90
72” Dia. Column	(26)#11	12.42	69.17	0.01371	0.90
72” Dia. Column	(104)#11	21.98	69.17	0.00644	0.90

ANTICIPATED EFFECT ON BRIDGES:

These changes will establish consistency in the determination of resistance factors for columns in Seismic Zones 3 & 4 with changes made to the determination of ϕ factors in the 2006 Interim.

REFERENCES:

None

OTHER:

None

2006 AASHTO BRIDGE COMMITTEE AGENDA ITEM: 13 (REVISION 2)

SUBJECT: LRFD Bridge Design Specifications: Section 5, Article 5.14.1.4 (WAI 73)

TECHNICAL COMMITTEE: T-10 Concrete

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

☒ **DESIGN SPEC** ☐ **CONSTRUCTION SPEC** ☐ **MOVABLE SPEC**
☐ **LRFR MANUAL** ☐ **OTHER**

☒ **US VERSION** ☐ **SI VERSION** ☐ **BOTH**

DATE PREPARED: 1/5/06
DATE REVISED: 5/22/06

AGENDA ITEM:

Item #1

Delete the following items as shown:

~~5.14.1.4 Simple Span Precast Girders Made Continuous~~

~~5.14.1.4.1 General~~

~~Bridges consisting of simple span precast concrete girders and cast-in-place concrete slabs may be made continuous for transient and/or permanent loads by using a cast-in-place closure joint at piers.~~

~~Where the cast-in-place closure joint is contained within a diaphragm, the design at the closure joint may be based on the strength of the concrete in the precast elements.~~

~~5.14.1.4.2 Reinforcement~~

~~Longitudinal reinforcement that makes or contributes to making the precast girder continuous over a pier shall be anchored in regions of the slab that can be shown to be crack-free at strength limit states. Such reinforcement shall satisfy the requirements specified in Article 5.11.1.2.3. This anchorage reinforcement shall be staggered. Regular longitudinal slab reinforcement may be utilized as part of the total longitudinal reinforcement required.~~

~~5.14.1.4.3 Degree of Continuity at Various Limit States~~

~~If the calculated stress at the bottom of the joint for the combination of superimposed permanent loads, settlement, creep, shrinkage, 50 percent live load, and temperature gradient, if applicable, is compressive, the joint may be considered fully effective.~~

~~Structures with fully effective construction joints at the piers shall be designed as fully continuous structures at all limit states for load applied after closure.~~

~~Structures with partially effective construction joints at piers shall be designed as continuous structures for loads applied after closure for strength and extreme event limit states only.~~

~~If the negative moment resistance of the joint at a pier is less than the total amount required, positive moment resistances in the adjacent spans shall be increased appropriately for each limit state investigated~~

~~C5.14.1.4.2~~

~~Recent limited tests on continuous model and full-size structural components indicate that, unless the~~

~~reinforcement is anchored in a compressive zone, its effectiveness becomes questionable at strength limit state (Priestley 1993). The intent of staggering bar ends is to distribute local force effects.~~

~~C5.14.1.4.3~~

~~It has been observed that in isolated cases, especially where the beams were green at the time of placing of the slab concrete, upward bowing of the superstructure due to creep resulted in separation of the closure joint. Such separation may cause the structure to act as simply supported at service and fatigue limit states, with continuity regained only at or close to strength limit state.~~

~~In addition to providing for a pouring sequence of cast-in-place concrete, the Designer may impose an age requirement on the use of precast concrete components.~~

Item #2

Replace the current headings for the subarticles of Article 5.14.1.4 with the following:

5.14.1.4 Bridges Composed of Simple Span Precast Girders Made Continuous

5.14.1.4.1 General
5.14.1.4.2 Restraint Moments
5.14.1.4.3 Material Properties
5.14.1.4.4 Age of Girder when Continuity is Established
5.14.1.4.5 Degree of Continuity at Various Limit States
5.14.1.4.6 Service Limit State
5.14.1.4.7 Strength Limit State
5.14.1.4.8 Negative Moment Connections
5.14.1.4.9 Positive Moment Connections
5.14.1.4.10 Continuity Diaphragms

Item #3

5.3 NOTATION

Revise or add the following definitions:

A_c = area of core of spirally reinforced compression member measured to the outside diameter of the spiral (in.^{2}); gross area of concrete deck slab (in.^{2}) (5.7.4.6) (C5.14.1.4.3)

A_s = area of nonprestressed tension reinforcement (in.^{2}); total area of longitudinal deck reinforcement (in.^{2}) (5.5.4.2.1) (C5.14.1.4.3)

A_{tr} = area of concrete deck slab with transformed longitudinal deck reinforcement (in.^{2}) (C5.14.1.4.3)

$E_{c\ deck}$ = modulus of elasticity of deck concrete (ksi) (C5.14.1.4.3)

f_{psl} = stress in the strand at the SERVICE limit state. Cracked section shall be assumed (ksi) (C5.14.1.4.9)

f_{pul} = stress in the strand at the STRENGTH limit state (ksi) (C5.14.1.4.9)

ℓ_{dsh} = total length of extended strand (in.) (C5.14.1.4.9)

n = modular ratio between deck concrete and reinforcement (C5.14.1.4.3)

$\varepsilon_{effective}$ = effective concrete shrinkage strain (in./in.) (C5.14.1.4.3)

ε_{sh} = concrete shrinkage strain at a given time (in./in.); unrestrained shrinkage strain for deck concrete (in./in.) (5.4.2.3.3) (C5.14.1.4.3)

Item #4

Revise the current subarticles of Article 5.14 as follows:

5.14.1.4 Bridges Composed of Simple Span Precast Girders Made Continuous

5.14.1.4.1 General

The provisions of this Article shall apply at the service and strength limit states as applicable.

When the requirements of Article 5.14.1.4 are satisfied, multi-span bridges composed of simple-span precast girders with continuity diaphragms cast between ends of girders at interior supports may be considered continuous for loads placed on the bridge after the continuity diaphragms are installed and have cured.

The connection between girders at the continuity diaphragm shall be designed for all effects that cause moment at the connection, including restraint moments from time-dependent effects, except as allowed in Article 5.14.1.4.

The requirements specified in Article 5.14.1.4 supplement the requirements of other sections of these Specifications for fully prestressed concrete components that are not segmentally constructed.

Multi-span bridges composed of precast girders with continuity diaphragms at interior supports that are designed as a series of simple spans are not required to satisfy the requirements of Article 5.14.1.4.

5.14.1.4.2 Restraint Moments

The bridge shall be designed for restraint moments that may develop because of time-dependent or other deformations, except as allowed in Article 5.14.1.4.4.

Restraint moments shall not be included in any combination when the effect of the restraint moment is to reduce the total moment.

5.14.1.4.3 Material Properties

Creep and shrinkage properties of the girder concrete and the shrinkage properties of the deck slab concrete shall be determined from either:

- Tests of concrete using the same proportions and materials that will be used in the girders and deck slab. Measurements shall include the time-dependent rate of change of these properties.

- The provisions of Article 5.4.2.3.

The restraining effect of reinforcement on concrete shrinkage may be considered.

5.14.1.4.4 Age of Girder when Continuity is Established

The minimum age of the precast girder when continuity is established should be specified in the contract documents. This age shall be used for calculating restraint moments due to creep and shrinkage. If no age is specified, a reasonable, but conservative estimate of the time continuity is established shall be used for all calculations of restraint moments.

The following simplification may be applied if acceptable to the owner and if the contract documents require a minimum girder age of at least 90 days when continuity is established:

- Positive restraint moments caused by girder creep and shrinkage and deck slab shrinkage may be taken to be 0.

- Computation of restraint moments shall not be required.

- A positive moment connection shall be provided with a factored resistance, φMn, not less than 1.2 M_{cr}, as specified in Article 5.14.1.4.9.

For other ages at continuity, the age-related design parameters should be determined from the literature, approved by the owner, and documented in the contract documents.

5.14.1.4.5 Degree of Continuity at Various Limit States

Both a positive and negative moment connection, as specified in Articles 5.14.1.4.8 and 5.14.1.4.9, are required for all continuity diaphragms, regardless of the degree of continuity as defined in this article.

The connection between precast girders at a continuity diaphragm shall be considered fully effective if either of the following are satisfied:

- The calculated stress at the bottom of the continuity diaphragm for the combination of superimposed permanent loads, settlement, creep, shrinkage, 50 percent live load and temperature gradient, if applicable, is compressive.

- The contract documents require that the age of the precast girders shall be at least 90 days when continuity is established and the design simplifications of Article 5.14.1.4.4 are used.

If the connection between precast girders at a continuity diaphragm does not satisfy these requirements, the joint shall be considered partially effective.

Superstructures with fully effective connections at interior supports may be designed as fully continuous structures ~~at all limit states~~ for loads applied after continuity is established.

Superstructures with partially effective connections at interior supports shall be designed as continuous structures for loads applied after continuity is established for strength ~~and extreme event~~ limit states only.

Gross composite girder section properties, ignoring any deck cracking, may be used for analysis as specified in Article 4.5.2.2.

If the negative moment resistance of the section at an interior support is less than the total amount required, the positive design moments in the adjacent spans shall be increased appropriately for each limit state investigated.

5.14.1.4.6 Service Limit State for Girder Stress Limits

Simple-span precast girders made continuous shall be designed to satisfy service limit state stress limits given in Article 5.9.4. For service load combinations that involve traffic loading, tensile stresses in prestressed members shall be investigated using the Service III load combination specified in Table 3.4.1-1.

At the service limit state after losses, when tensile stresses develop at the top of the girders near interior supports, the tensile stress limits specified in Table 5.9.4.1.2-1 for other than segmentally constructed bridges shall apply. The specified compressive strength of the girder concrete, f'_c, shall be substituted for f'_{ci} in the stress limit equations. The Service III load combination shall be used to compute tensile stresses for these locations.

Alternatively, the top of the precast girders at interior supports may be designed as reinforced concrete members at the strength limit state. In this case, the stress limits for the service limit state shall not apply to this region of the precast girder.

A cast-in-place composite deck slab shall not be subject to the tensile stress limits for the service limit state after losses specified in Table 5.9.4.2.2-1.

5.14.1.4.7 Strength Limit State

The connections between precast girders and a continuity diaphragm shall be designed for the strength limit state.

The reinforcement in the deck slab shall be proportioned to resist negative design moments at the strength limit state.

5.14.1.4.8 Negative Moment Connections

The reinforcement in a cast-in-place, composite deck slab in a multi-span precast girder bridge made continuous shall be proportioned to resist negative design moments at the strength limit state.

Longitudinal reinforcement used for the negative moment connection over an interior pier shall be anchored in regions of the slab that are in compression at strength limit states and shall satisfy the requirements of Article 5.11.1.2.3. The termination of this reinforcement shall be staggered. All longitudinal reinforcement in the deck slab may be used for the negative moment connection.

Negative moment connections between precast girders into or across the continuity diaphragm shall satisfy the requirements of Article 5.11.5. These connections shall be permitted where the bridge is designed with a composite deck slab and shall be required where the bridge is designed without a composite deck slab. Additional connection details shall be permitted if the strength and performance of these connections is verified by analysis or testing.

The requirements of Article 5.7.3 shall apply to the reinforcement in the deck slab and at negative moment connections at continuity diaphragms.

5.14.1.4.9 Positive Moment Connections

5.14.1.4.9a General

Positive moment connections at continuity diaphragms shall be made with reinforcement developed into both the girder and continuity diaphragm. Three types of connections shall be permitted:

- Mild reinforcement embedded in the precast girders and developed into the continuity diaphragm.
- Pretensioning strands extended beyond the end of the girder and anchored into the continuity diaphragm. These strands shall not be debonded at the end of the girder.
- Any connection detail shown by analysis, testing or as approved by the bridge owner to provide adequate positive moment resistance.

Additional requirements for connections made using each type of reinforcement are given in subsequent articles.

The critical section for the development of positive moment reinforcement into the continuity diaphragm shall be taken at the face of the girder. The critical section for the development of positive moment reinforcement into the precast girder shall consider conditions in the girder as specified in this article for the type of reinforcement used.

The requirements of Article 5.7.3, except Article 5.7.3.3.2, shall apply to the reinforcement at positive moment connections at continuity diaphragms. This reinforcement shall be proportioned to resist the larger of the following, except when using the design simplifications of Article 5.14.1.4.4:

- Factored positive restraint moment, or
- 0.6 M_{cr}

The cracking moment M_{cr} shall be computed using Eq. 5.7.3.6.2-2 with the gross composite section properties for the girder and the effective width of composite deck slab, if any, and the material properties of the concrete in the continuity diaphragm.

The precast girders shall be designed for any positive restraint moments that are used in design. Near the ends of girders, the reduced effect of prestress within the transfer length shall be considered.

5.14.1.4.9b Positive Moment Connection using Mild Reinforcement

The anchorage of mild reinforcement used for positive moment connections shall satisfy the requirements of Article 5.11 and the additional requirements of this Article. Where positive moment reinforcement is added between pretensioned strands, consolidation of concrete and bond of reinforcement shall be considered.

The critical section for the development of positive moment reinforcement into the precast girder shall consider conditions in the girder. The reinforcement shall be developed beyond the inside edge of the bearing area. The reinforcement shall also be detailed so that, for strands considered in resisting positive moments within the end of the girder, debonding of strands does not terminate within the development length.

Where multiple bars are used for a positive moment connection, the termination of the reinforcement shall be staggered in pairs symmetrical about the centerline of the precast girder.

5.14.1.4.9c Positive Moment Connection using Prestressing Strand

Pretensioning strands that are not debonded at the end of the girder may be extended into the continuity diaphragm as positive moment reinforcement. The extended strands shall be anchored into the diaphragm by bending the strands into a 90° hook or by providing a development length as specified in Article 5.11.4.

The stress in the strands used for design, as a function of the total length of the strand, shall not exceed:

$$f_{psl} = \frac{(\ell_{dsh} - 8)}{0.228} \quad (5.14.1.4.9c\text{-}1)$$

$$f_{pul} = \frac{(\ell_{dsh} - 8)}{0.163} \quad (5.14.1.4.9c\text{-}2)$$

where:

ℓ_{dsh} = total length of extended strand (in.)
f_{psl} = stress in the strand at the service limit state
Cracked section shall be assumed (ksi)
f_{pul} = stress in the strand at the strength limit state (ksi)

Strands shall project at least 8.0 in. from the face of the girder before they are bent.

5.14.1.4.9d Details of Positive Moment Connection

Positive moment reinforcement shall be placed in a pattern that is symmetrical, or as nearly symmetrical as possible, about the centerline of the cross-section.

Fabrication and erection issues shall be considered in the detailing of positive moment reinforcement in the continuity diaphragm. Reinforcement from opposing girders shall be detailed to mesh during erection without significant conflicts. Reinforcement shall be detailed to enable placement of anchor bars and other reinforcement in the continuity diaphragm.

5.14.1.4.10 Continuity Diaphragms

The design of continuity diaphragms at interior supports may be based on the strength of the concrete in the precast girders.

Precast girders may be embedded into continuity diaphragms.

If horizontal diaphragm reinforcement is passed through holes in the precast beam or is attached to the precast element using mechanical connectors, the end precast element shall be designed to resist positive moments caused by superimposed dead loads, live loads, creep and shrinkage of the girders, shrinkage of the deck slab, and temperature effects. Design of the end of the girder shall account for the reduced effect of prestress within the transfer length.

Where ends of girders are not directly opposite each other across a continuity diaphragm, the diaphragm must be designed to transfer forces between girders. Continuity diaphragms shall also be designed for situations where an angle change occurs between opposing girders.

Item #5

Revise the current subarticles of the Commentary to Article 5.14.1.4 as follows:

C5.14.1.4.1

This type of bridge is generally constructed with a composite deck slab. However, with proper design and detailing, precast members used without a composite deck may also be made continuous for loads applied after continuity is established. Details of this type of construction are discussed in Miller et al. (*2004*).

The designer may choose to design a multi-span bridge as a series of simple spans but detail it as continuous with continuity diaphragms to eliminate expansion joints in the deck slab. This approach has been used successfully in several parts of the country.

Where this approach is used, the designer should consider adding reinforcement in the deck adjacent to the interior supports to control cracking that may occur from the continuous action of the structure.

Positive moment connections improve the structural integrity of a bridge, increasing its ability to resist extreme event and unanticipated loadings. These connections also control cracking that may occur in the continuity diaphragm. Therefore, it is recommended that positive moment connections be provided in all bridges detailed as continuous for live load.

C5.14.1.4.2

Deformations that occur after continuity is established from time-dependent effects such as creep, shrinkage and temperature variation cause restraint moments.

Restraint moments are computed at interior supports of continuous bridges but affect the design moments at all locations on the bridge. Studies show that restraint moments can be positive or negative. The magnitude and direction of the moments depend on girder age at the time continuity is established, properties of the girder and slab concrete, and bridge and girder geometry. (*Mirmiran, et al., 2001*). The data show that the later continuity is formed, the lower the predicted values of positive restraint moment which will form. Since positive restraint moments are not desirable, waiting as long as possible after the girders are cast to establish continuity and cast the deck appears to be beneficial.

Several methods have been published for computing restraint moments (*Mirmiran, et al. 2001*). While these methods may be useful in estimating restraint moments, designers should be aware that these methods may overestimate the restraint moments – both positive and negative. Existing structures do not show the distress that would be expected from the moments computed by some analysis methods.

Most analysis methods indicate that differential shrinkage between the girder and deck mitigates positive moment formation. Data from various projects (*Miller, et al. 2004 and Russell, et al., 2003*) does not show the effects of differential shrinkage. Therefore, it is questionable whether negative moments due to differential shrinkage form to the extent predicted by analysis. Since field observations of significant negative moment distress have not been reported, negative moments caused by differential shrinkage are often ignored in design.

Estimated restraint moments are highly dependent on actual material properties and project schedules and the computed restraint moments may never develop. Therefore, a critical design moment must not be reduced by a restraint moment in case the restraint moment does not develop.

C5.14.1.4.3

The development of restraint moments is highly dependent on the creep and shrinkage properties of the girder and deck concrete. Since these properties can vary widely, measured properties should be used when available to obtain the most accurate analysis. However, these properties are rarely available during design. Therefore, the provisions of Article 5.4.2.3 may be used to estimate these properties.

Because longitudinal reinforcement in the deck slab restrains the shrinkage of the deck concrete, the apparent shrinkage is less than the free shrinkage of the deck concrete. This effect may be estimated using an effective concrete shrinkage strain, $\varepsilon_{effective}$, which may be taken as:

$$\varepsilon_{effective} = \varepsilon_{sh}\left(\frac{A_c}{A_{tr}}\right) \qquad \text{(C5.14.1.4.3-1)}$$

where:

ε_{sh} = unrestrained shrinkage strain for deck concrete (in./in.)

A_c = gross area of concrete deck slab (in.^{2})

A_{tr} = area of concrete deck slab with transformed longitudinal deck reinforcement (in.^{2})

= $A_c + A_s(n-1)$

A_s = total area of longitudinal deck reinforcement (in.^{2})
n = modular ratio between deck concrete and reinforcement
= $E_s / E_{c\,deck}$
$E_{c\,deck}$ = modulus of elasticity of deck concrete (ksi)

Eq. C1 is based on simple mechanics (*Abdalla et al. 1993*). If the amount of longitudinal reinforcement varies along the length of the slab, the average area of longitudinal reinforcement may be used to calculate the transformed area.

C5.14.1.4.4

Analytical studies show that the age of the precast girder when continuity is established is an important factor in the development of restraint moments (*Mirmiran, et al. 2001*). According to analysis, establishing continuity when girders are young causes larger positive moments to develop. Therefore, if no minimum girder age for continuity is specified, the earliest reasonable age must be used. Results from surveys of practice (*Miller et al. 2004*) show a wide variation in girder ages at which continuity is established. An age of 7 days was reported to be a realistic minimum. However, the use of 7 days as the age of girders when continuity is established results in a large positive restraint moment. Therefore, a specified minimum girder age at continuity of at least 28 days is strongly recommended.

If girders are 90 days or older when continuity is established, the provisions of Article 5.4.2.3 predict that approximately 60 percent of the creep and 70 percent of the shrinkage in the girders, which could cause positive moments, has already occurred prior to establishing continuity. The owner may allow the use of k_{td} in Eq. 5.4.2.3.2-5 set at 0.7 to determine the time at which continuity can be established and, therefore, utilize the 90-day provisions of this Article. Since most of the creep and shrinkage in the girder has already occurred before continuity is established, the potential development of time-dependent positive moments is limited. Differential shrinkage between the deck and the girders, to the extent to which it actually occurs (refer to Article C5.14.1.4.2) would also tend to limit positive moment development.

Even if the girders are 90 days old or older when continuity is established, some positive moment may develop at the connection and some cracking may occur. Research (*Miller, et al. 2004*) has shown that if the connection is designed with a capacity of 1.2 M_{cr}, the connection can tolerate this cracking without appreciable loss of continuity.

This provision provides a simplified approach to design of precast girder bridges made continuous that eliminates the need to evaluate restraint moments. Some states allow design methods where restraint moments are not evaluated when continuity is established when girders are older than a specified age. These design methods have been used for many years with good success. However, an owner may require the computation of restraint moments for all girder ages.

C5.14.1.4.5

A fully effective joint at a continuity diaphragm is a joint that is capable of full moment transfer between spans, resulting in the structure behaving as a continuous structure.

In some cases, especially when continuity is established at an early girder age, continuing upward cambering of the girders due to creep may cause cracking at the bottom of the continuity diaphragm (*Mirmiran et al. 2001*). Analysis and tests indicate that such cracking may cause the structure to act as a series of simply supported spans when resisting some portion of the permanent or live loads applied after continuity is established, however, this condition only occurs when the cracking is severe and the positive moment connection is near failure (*Miller et al. 2004*). Where this occurs, the connections at the continuity diaphragm are partially effective.

Theoretically, the portion of the permanent or live loads required to close the cracks would be applied to a simply supported span, neglecting continuity. The remainder of the load would then be applied to the continuous span, assuming full continuity. However, in cases where the portion of the live load required to close the crack is less than 50 percent of the live load, placing part of the load on simple spans and placing the remainder on the continuous bridge results in only a small change in total stresses at critical sections due to all loads. Tests have shown that the connections can tolerate some positive moment cracking and remain continuous (*Miller, et. al., 2004*). Therefore, if the conditions of the first bullet point are satisfied, it is reasonable to design the member as continuous for the entire load placed on the structure after continuity is established.

The second bullet follows from the requirements of Article 5.14.1.4.4 where restraint moments may be neglected if continuity is established when the age of the precast girder is at least 90 days. Without positive moment, the potential cracks in the continuity diaphragm would not form and the connection would be fully effective.

Partially effective construction joints are designed by applying the portion of the permanent and live loads applied after continuity is established to a simple span (neglecting continuity). Only the portion of the loads required to close the assumed cracks are applied. The remainder of the permanent and live loads would then be applied to the continuous span. The load required to close the crack can be taken as the load causing zero tension at the bottom of the continuity diaphragm. Such analysis may be avoided if the contract documents require the age of the girder at continuity to be at least 90 days.

C5.14.1.4.6

Tensile stresses under service limit state loadings may occur at the top of the girder near interior supports. This region of the girder is not a precompressed tensile zone, so there is not an applicable tensile stress limit in Table 5.9.4.2.2-1. Furthermore, the tensile zone is close to the end of the girder, so adding or debonding pretensioned strands has little effect in reducing the tensile stresses. Therefore, the limits specified for temporary stresses before losses have been used to address this condition, with modification to use the specified concrete strength. This provision provides some relief for the potentially high tensile stresses that may develop at the ends of girders because of negative service load moments.

This option allows the top of the girder at the interior support to be designed as a reinforced concrete element using the strength limit state rather than a prestressed concrete element using the service limit state.

The deck slab is not a prestressed element. Therefore, the tensile stress limits do not apply. It has been customary to apply the compressive stress limits to the deck slab.

C5.14.1.4.7

The continuity diaphragm is not prestressed concrete so the stress limits for the service limit state do not apply. Connections to it are therefore designed using provisions for reinforced concrete elements.

C5.14.1.4.8

Research at PCA (*Kaar et al. 1961*) and years of experience show that the reinforcement in a composite deck slab can be proportioned to resist negative design moments in a continuous bridge.

Limited tests on continuous model and full size structural components indicate that, unless the reinforcement is anchored in a compressive zone, the effectiveness becomes questionable at the strength limit state (*Priestly 1993*). The termination of the longitudinal deck slab reinforcement is staggered to minimize potential deck cracking by distributing local force effects.

A negative moment connection between precast girders and the continuity diaphragm is not typically provided, because the deck slab reinforcement is usually proportioned to resist the negative design moments. However, research (*Ma et al. 1998*) suggests that mechanical connections between the tops of girders may also be used for negative moment connections, especially when continuity is established prior to placement of the deck slab. If a composite deck slab is not used on the bridge, a negative moment connection between girders is required to obtain continuity. Mechanical reinforcement splices have been successfully used to provide a negative moment connection between box beam bridges that do not have a composite deck slab.

C5.14.1.4.9a

Positive moment connections improve the structural integrity of a bridge, increasing its ability to resist extreme event and unanticipated loadings. Therefore, it is recommended that positive moment connections be provided in all bridges detailed as continuous for live load.

Both embedded bar and extended strand connections have been used successfully to provide positive moment resistance. Test results (*Miller et al. 2004*) indicate that connections using the two types of reinforcement perform similarly under both static and fatigue loads and both have adequate strength to resist the applied moments.

Analytical studies (*Mirmiran, et. al. 2001*) suggest that a minimum amount of reinforcement, corresponding to a capacity of 0.6 M_{cr} is needed to develop adequate resistance to positive restraint moments. These same studies show that a positive moment connection with a capacity greater than 1.2 M_{cr} provides only minor improvement in continuity behavior over a connection with a capacity of 1.2 M_{cr}. Therefore, it is recommended that the positive moment capacity of the connection not exceed 1.2 M_{cr}. If the computed positive moment exceeds 1.2 M_{cr}, the section should be modified or steps should be taken to reduce the positive moment.

The cracking moment M_{cr} is the moment that causes cracking in the continuity diaphragm. Since the continuity diaphragm is not a prestressed concrete section, the equation for computing the cracking moment for a reinforced section is used. The diaphragm is generally cast with the deck concrete, so the section properties are computed using uniform concrete properties, so the deck width is not transformed.

Article 5.7.3.3.2 specifies a minimum capacity for all flexural sections. This is to prevent sudden collapse at the formation of the first crack. However, the positive moment connection that is being discussed here is not intended to resist applied live loads. Even if the positive moment connection were to fail completely, the system may, at worst, become a series of simple spans. Therefore, the minimum reinforcement requirement of Article 5.7.3.3.2 does not apply. Allowing positive moment connections with lower quantities of reinforcement will relieve congestion in continuity diaphragms.

C5.14.1.4.9b

The positive moment connection is designed to utilize the yield strength of the reinforcement. Therefore, the connection must be detailed to provide full development of the reinforcement. If the reinforcement cannot be detailed for full development, the connection may be designed using a reduced stress in the reinforcement.

Potential cracks are more likely to form in the precast girder at the inside edge of the bearing area and locations of termination of debonding. Since cracking within the development length reduces the effectiveness of the development, the reinforcement should be detailed to avoid this condition. It is recommended that reinforcement be developed beyond the location where a crack radiating from the inside edge of the bearing may cross the reinforcement.

The termination of the positive moment reinforcement is staggered to reduce the potential for cracking at the ends of the bars.

C5.14.1.4.9c

Strands that are debonded or shielded at the end of a member may not be used as reinforcement for the positive moment connection. There are no requirements for development of the strand into the girder because the strands run continuously through the precast girder.

Eqs. 1 and 2 were developed for 0.5 in. strand by Salmons, et al. (*1980*). These are for prestressing strand extended from the end of the girder and given 90° hooks. Other equations are also available to estimate stress in bent strands (*Noppakunwijai et al. 2002*).

C5.14.1.4.9d

Tests (*Miller et al. 2004*) suggest that reinforcement patterns that have significant asymmetry may result in unequal bar stresses that can be detrimental to the performance of the positive moment connection.

With some girder shapes, it may not be possible to install prebent hooked bars without the hook tails interfering with the formwork. In such cases, a straight bar may be embedded and then bent after the girder is fabricated. Such bending is generally accomplished without heating and the bend must be smooth with a minimum bend diameter conforming to the requirements of Table 5.10.2.3-1. If the Engineer allows the reinforcement to be bent after the girder is fabricated, the contract documents shall indicate that field bending is permissible and shall provide requirements for such bending. Since requirements regarding field bending may vary, the preferences of the owner should be considered.

Hairpin bars (a bar with a 180° bend with both legs developed into the precast girder) have been used for positive moment connections to eliminate the need for post-fabrication bending of the reinforcement and reduce congestion in the continuity diaphragm.

C5.14.1.4.10

The use of the increased concrete strength is permitted because the continuity diaphragm concrete between girder ends is confined by the girders and by the continuity diaphragm extending beyond the girders. It is recommended that this provision be applied only to conditions where the portion of the continuity diaphragm that is in compression is confined between ends of precast girders.

The width of the continuity diaphragm must be large enough to provide the required embedment for the development of the positive moment reinforcement into the diaphragm. An anchor bar with a diameter equal to or greater than the diameter of the positive moment reinforcement may be placed in the corner of a 90^{o} hook or inside the loop of a 180^{o} hook bar to improve the effectiveness of the anchorage of the reinforcement.

Several construction sequences have been successfully used for the construction of bridges with precast girders made continuous. When determining the construction sequence, the Engineer should consider the effect of girder rotations and restraint as the deck slab concrete is being placed.

Test results (*Miller et al. 2004*) have shown that embedding precast girders 6.0 in. into continuity diaphragms improves the performance of positive moment connections. The observed stresses in the positive moment reinforcement in the continuity diaphragm were reduced compared to connections without girder embedment.

The connection between precast girders and the continuity diaphragm may be enhanced by passing horizontal reinforcement through holes in the precast beam or attaching the reinforcement to the beam by embedded connectors. Test results (*Miller et al. 2004; Salmons 1980*) show that such reinforcement stiffens the connection. The use of such mechanical connections requires that the end of the girder be embedded into the continuity diaphragm. Tests of continuity diaphragms without mechanical connections between the girder and diaphragm show the failure of connection occurs by the beam end pulling out of the diaphragm with all of the damage occurring in the diaphragm. Tests of connections with horizontal bars show that cracks may form in the end of the precast girder outside the continuity diaphragm if the connection is subjected to a significant positive moment. Such cracking in the end region of the girder may not be desirable.

A method such as given in Article 5.6.3 may be used to design a continuity diaphragm for these conditions.

Item #6

Add the following references to Section 5:

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Salmons, J.R., *Behavior of Untensioned-Bonded Prestressing Strand*, Final Report 77-1, Missouri Cooperative Highway Research Program , Missouri State Highway Department, June 1980, 73 p.

OTHER AFFECTED ARTICLES:

None

BACKGROUND:

In the early 1960's, the Portland Cement Association showed that single span precast girders could be successfully connected to create a multiple span structure which would be continuous for superimposed dead load and live load (Kaar, *et al*. 1960, 1961; Hanson, 1960, Mattock *e. al.* 1960, 1961a, 1961b). The connection was made by casting a diaphragm between the ends of the girders and placing negative moment reinforcement in the deck above the diaphragm. This research also showed that positive moments develop at the diaphragms due to creep and shrinkage, cracking the interface. A positive moment connection was needed at the diaphragm to maintain continuity. National Co-operative Highway Research Program (NCHRP) Report 322 (Oesterle, *et al.,* 1989) disputed the effectiveness of this bridge type. Report 322 concluded that presence of the positive moment connection caused the formation of a positive restraint moment at the girder ends. When the effect of this positive restraint moment was added to the positive midspan moment, the author of NCHRP Report 322 concluded that any benefit of continuity was lost. This report further concluded that positive moment connections were expensive and difficult to construct. Therefore, NCHRP Report 322 recommended not using continuous for live load bridges.

However, these results were not universally accepted. NCHRP Project 12-53 was commissioned to further study this type of bridge. The results of this project are published as NCHRP Report 519 (Miller, *et al.*2004). The main conclusions of the study are:

1) This research program tested two types of positive moment connections: ones using 90^o bent reinforcing bars and ones using 90^o bent extended prestressing strand. Both types of connections were found to perform adequately.

2) Bent strand positive moment connections can be adequately designed using the equations for strand length proposed by Salmons (1973, 1974a, 1974b, 1980). These connections were easy to construct as the strand was flexible enough to move during assembly. These connections tend to fail by gradual pull-out of the strand.

3) Bent bar connection were more difficult to construct than bent strand connections. Embedding the bar in the end of the girders caused additional congestion in an already congested area. For some type of girders, most notably I and bulb T sections, it is not possible to install the bars pre-bent as the bottom flange formwork interferes with hook tails. In these cases, the bars must be installed straight and field bent, resulting in non-uniform hooks which may cause unequal stresses in the bars. To allow the bars to mesh in the diaphragm, the bent bars must be installed in an asymmetrical pattern. This may also cause unequal stresses in the bars. Because the bars are not flexible, it is more difficult to assemble the bent bar connection. The provisions of Articles 5.10 (Details of Reinforcement) and 5.11 (Development and Splices of Reinforcement) are adequate for designing this type of connection. Failure occurred due to fatigue of the bars.

4) Embedding the girder ends in the diaphragm seems to improve the connection capacity, but the effect is hard to quantify due to the uncertain nature of concrete to concrete bond capacity.

5) Placing additional stirrups in the diaphragm just outside of the bottom flange of the girder does not increase connection strength, but does increase ductility. This detail may be useful in seismic zones.

6) Use of horizontal bars through the web increases the connection strength, but at failure the girder webs crack. This may be an undesirable failure mode.

7) Some states have positive moment connection details require that bottom ½ or 1/3 of the diaphragm be cast before the remainder of the diaphragm and the slab. It is thought that the weight of the slab, when it is cast, will rotate the girder ends into the partial diaphragm and compress the connection. This detail did not work and seems to need a tension tie at the top of the girders to work properly.

8) Expansion and contraction of the deck due to heat of hydration significantly affect the reactions and stresses in the girders. In this research, contraction of the deck caused negative moment to form at the diaphragm. However, an anticipated formation of negative moment due to differential deck shrinkage did not occur.

9) Daily temperature changes greatly affect the stresses in the girders. It was found that reaction changed as much as ±25% per day due to thermal effects.

10) Positive moment cracking at the diaphragm does not necessarily affect the continuity of the system. It was found that there is no degradation of the continuity until the positive moment connection is near failure and the cracks at the face of the diaphragm have propagated into the deck. In the extreme case of the connection being near failure, the continuity was only reduced by 30%. Continuity was restored when the positive moment was removed.

11) Use of a properly designed reinforced deck is an adequate connection for negative moment. Positive moment cracking does not affect negative moment capacity.

ANTICIPATED EFFECT ON BRIDGES:

Many states use continuous for live load bridges. This type of construction is covered under Article 5.14.1.4 of the current LRFD Specifications (w/ 2005 and 06 interim). The proposed changes update this Article to reflect recent research findings.

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Salmons, J.R. and May, G.W., *Strand Reinforcing for End Connection of Pretensioned I-Beam Bridges*, Interim Report 73-5B, Missouri Cooperative Highway Research Program, Missouri State Highway Department, May 1974, 142 p.

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OTHER:

None

2006 AASHTO BRIDGE COMMITTEE AGENDA ITEM: 14 (REVISION 1)

SUBJECT: LRFD Bridge Design Specifications: Section 5, Various Articles (WAI 109)

TECHNICAL COMMITTEE: T-10 Concrete

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

☒ **DESIGN SPEC** ☐ **CONSTRUCTION SPEC** ☐ **MOVABLE SPEC**
☐ **LRFR MANUAL** ☐ **OTHER**

☒ **US VERSION** ☐ **SI VERSION** ☐ **BOTH**

DATE PREPARED: 11/3/05
DATE REVISED: 5/15/06

AGENDA ITEM:

Item #1

In Article 5.3 Notation, change current variable f_s *to* f_{ss}*:*

~~f_s~~f_{ss} = tensile stress in mild steel reinforcement at the service limit state (ksi) (5.7.3.4)

Add the following two definitions:

f_s = stress in the mild steel tension reinforcement at nominal flexural resistance (ksi) (5.7.3.1) (5.7.3.2)

f'_s = stress in the mild steel compression reinforcement at nominal flexural resistance (ksi) (5.7.3.1) (5.7.3.2)

Item #2

Add the following bulleted item to the list in Article 5.7.2.1:

- In the approximate flexural resistance equations of Articles 5.7.3.1 and 5.7.3.2, f_y and f'_y may replace f_s and f'_s, respectively, subject to the following conditions:
  - f_y may replace f_s when, using f_y in the calculation, the resulting ratio c/d_s does not exceed 0.6. If c/d_s exceeds 0.6, strain compatibility shall be used to determine the stress in the mild steel tension reinforcement.
  - f'_y may replace f'_s when, using f'_y in the calculation, $c > 3d'_s$. If $c < 3d'_s$, ~~the compression reinforcement may be conservatively ignored ($A'_s = 0$), or~~ strain compatibility shall be used to determine the stress in the mild steel compression reinforcement. The compression reinforcement may be conservatively ignored, i.e., $A'_s = 0$.

Add the following commentary to Article C5.7.2.1:

When using the approximate flexural resistance equations in Articles 5.7.3.1 and 5.7.3.2, it is important to assure that both the tension and compression mild steel reinforcement are yielding to obtain accurate results. In previous editions of AASHTO LRFD Bridge Design Specifications, the maximum reinforcement limit of $c/d_e \leq$ 0.42 assured that the mild tension steel would yield at nominal flexural resistance, but this limit was eliminated in the 2006 Interim revisions. The current limit of $c/d_s \leq 0.6$ assures that the mild tension steel will be at or near yield, while $c > 3d'_s$ assures that the mild compression steel will yield. It is conservative to ignore the compression steel when calculating flexural resistance. In cases where either the tension or compression steel does not yield, it is more accurate to use a method based on the conditions of equilibrium and strain compatibility to determine the flexural resistance.

The mild steel tension reinforcement limitation does not apply to prestressing steel used as tension reinforcement. The equations used to determine the stress in the prestressing steel at nominal flexural resistance already consider the effect of the depth to the neutral axis.

Item #3

In Eq. 5.7.3.1.1-3, replace f_y with f_s and f'_y with f'_s:

$$c = \frac{A_{ps} f_{pu} + A_s \cancel{f_y}\,\underline{f_s} - A'_s \cancel{f'_y}\,\underline{f'_s} - 0.85 f'_c (b - b_w) h_f}{0.85 f'_c \beta_1 b_w + k A_{ps} \dfrac{f_{pu}}{d_p}}$$

In Eq. 5.7.3.1.1-4, replace f_y with f_s and f'_y with f'_s:

$$c = \frac{A_{ps} f_{pu} + A_s \cancel{f_y}\,\underline{f_s} - A'_s \cancel{f'_y}\,\underline{f'_s}}{0.85 f'_c \beta_1 b + k A_{ps} \dfrac{f_{pu}}{d_p}}$$

In the where list in Article 5.7.3.1.1, delete the definitions for f_y and f'_y and replace with:

f_s = stress in the mild steel tension reinforcement at nominal flexural resistance (ksi), as specified in Article 5.7.2.1

f'_s = stress in the mild steel compression reinforcement at nominal flexural resistance (ksi), as specified in Article 5.7.2.1

Delete the last paragraph of Article 5.7.3.1.1.

In Eq. 5.7.3.1.2-3, replace f_y with f_s and f'_y with f'_s:

$$c = \frac{A_{ps} f_{pu} + A_s \cancel{f_y}\,\underline{f_s} - A'_s \cancel{f'_y}\,\underline{f'_s} - 0.85 f'_c (b - b_w) h_f}{0.85 f'_c \beta_1 b_w}$$

In Eq. 5.7.3.1.2-4, replace f_y with f_s and f'_y with f'_s:

$$c = \frac{A_{ps} f_{pu} + A_s \cancel{f_y}\,\underline{f_s} - A'_s \cancel{f'_y}\,\underline{f'_s}}{0.85 f'_c \beta_1 b}$$

Delete the last paragraph of Article 5.7.3.1.2.

In Eq. 5.7.3.2.2-1, replace f_y with f_s and f'_y with f'_s:

$$M_n = A_{ps} f_{ps}\left(d_p - \frac{a}{2}\right) + A_s \cancel{f_y}\,\underline{f_s}\left(d_s - \frac{a}{2}\right) - A_s' \cancel{f'_y}\,\underline{f'_s}\left(d_s' - \frac{a}{2}\right) + 0.85 f'_c (b - b_w) h_f \left(\frac{a}{2} - \frac{h_f}{2}\right)$$

In the where list in Article 5.7.3.2.2, delete the definitions for f_y and f'_y and replace with:

f_s = stress in the mild steel tension reinforcement at nominal flexural resistance (ksi), as specified in Article 5.7.2.1

f'_s = stress in the mild steel compression reinforcement at nominal flexural resistance (ksi), as specified in Article 5.7.2.1

Item #4

In Eq. 5.7.3.4-1, replace f_s with f_{ss}:

$$s \leq \frac{700\gamma_c}{\beta_s \cancel{f_s} \underline{f_{ss}}} - 2d_c$$

where:

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

OTHER AFFECTED ARTICLES:

None

BACKGROUND:

The approximate equations of Article 5.7.3 are intended to give designers a simplified method of calculating the flexural resistance of reinforced and prestressed concrete members. The current LRFD equations inherently assume that both the tension and compression mild reinforcement are yielding at nominal resistance, which is not always the case. However, simply replacing f_y and f'_y with f_s and f'_s does little good to simplify the designer's task, since a strain compatibility analysis is necessary to determine f_s and f'_s.

For compression reinforcement, current LRFD contains a caveat to the use of the approximate equations that requires the stress in the compression reinforcement to be checked for yielding. As a result, it is more convenient for designers to ignore the compression reinforcement, in lieu of performing a strain compatibility analysis, when calculating flexural resistance. A simple check that $c \geq 3d'_s$ can assure that the compression steel is at or near yield at nominal flexural resistance. If not, the compression steel can either be conservatively ignored, or a strain compatibility analysis can be performed.

For mild steel tension reinforcement, the previous maximum reinforcement limit of $c/d_e \leq 0.42$ assured that the mild steel reinforcement would yield at nominal resistance. However, this limit was eliminated in the 2006 Interims in favor of a varying resistance factor. A new limit of $c/d_s \leq 0.60$ is proposed to assure yielding of the mild steel tension reinforcement in conjunction with the approximate equations. This limit is entirely compatible with the varying resistance factor approach. In fact, if a beam or slab has one layer of mild steel flexural tension reinforcement, this limit corresponds to the compression-controlled limit of the member.

Parametric studies were used to verify these limitations for prestressed and non-prestressed flexural members varying in depth from 12 inches to over 7 ft. Note that the mild steel tension reinforcement limitation does not apply to prestressed members, as the equations for the stress in prestressing steel at nominal flexural resistance already consider the effect of the depth to the neutral axis. The studies compared the approximate equations to a non-linear strain compatibility analysis for different types of flexural members. For non-prestressed members, beyond the limit $c/d_s = 0.60$, the approximate equations overestimate the flexural resistance because they assume the tension reinforcement is yielding when in fact it is not. This did not occur for members with prestressed tension reinforcement.

Adoption of these limits on the applicability of the approximate equations will enable designers to use them with confidence while maintaining their simplicity.

ANTICIPATED EFFECT ON BRIDGES:

None

REFERENCES:

None

OTHER:

None

2006 AASHTO BRIDGE COMMITTEE AGENDA ITEM: 15 (REVISION 1)

SUBJECT: LRFD Bridge Design Specifications: Section 5, Various Articles (WAI 57)

TECHNICAL COMMITTEE: T-10 Concrete

☒ **REVISION**	☒ **ADDITION**	☐ **NEW DOCUMENT**
☒ **DESIGN SPEC** ☐ **LRFR MANUAL**	☐ **CONSTRUCTION SPEC** ☐ **OTHER**	☐ **MOVABLE SPEC**
☒ **US VERSION**	☐ **SI VERSION**	☐ **BOTH**

DATE PREPARED: 1/12/06
DATE REVISED: 5/15/06

AGENDA ITEM:

Item #1

Delete entire existing Article 5.8.4

Item #2

Insert proposed Article 5.8.4 as presented in the attached file.

Item #3

Delete the two paragraphs in Article 5.13.2.5.2 and replace with the following:

Design of beam ledges for shear shall be in accordance with the requirements for shear friction specified in Article 5.8.4. Nominal interface shear resistance shall satisfy Eqs. 5.13.2.4.2-1 through 5.13.2.4.2-4 wherein the width of the concrete face, b_w, assumed to participate in resistance to shear shall not exceed S, (W+4a_v), or 2c, as illustrated in Figure 1.

Item #4

Revise second paragraph of Article 5.13.3.8 as follows:

Lateral forces shall be transferred from the pier to the footing in accordance with shear-transfer provisions specified in Article 5.8.4 on the basis of the appropriate bulleted item in Article 5.8.4.~~2~~3.

Item #5

Revise second sentence of second paragraph of Article 5.14.1.1 as follows:

….Where a structurally separate concrete deck is applied, it shall be made composite with the precast beams in accordance with the provisions of Article 5.8.4.

OTHER AFFECTED ARTICLES:

None

BACKGROUND:

Item #1

The existing LRFD Specifications require substantially more interface shear reinforcement for girder/slab bridges than had been required by the Standard Specifications. So much so that interface shear reinforcement requirements generally govern over vertical (transverse) shear reinforcement requirements. An extensive review of available literature indicated that the interface shear resistance equation is extremely conservative relative to experimental data. In the absence of data and literature supporting the existing Article 5.8.4 provisions, an effort has been made to reevaluate the content and format of the entire Article.

The overall objective is to eliminate over-design, introduce proper LRFD notation, eliminate a significant dependence on Commentary equations for Code application and eliminate numerous changes in units from one portion of the Article to another.

Item #2

5.8.4.1:

Notation is now consistent with the other Articles of the LRFD Specification with respect to factored resistance, nominal resistance and factored design force. Specific definition of the loads to be included in the factored design force for girder/slab bridges, absent from prior editions of the Specifications is now provided.

The position taken within this proposal is predicated on the underlying fact that these are Strength Limit State design computations as opposed to Service Limit State design computations. As such, the predicted factored resistance must exceed all factored forces acting on the system after cracking and plastic deformations have occurred. That is to say, noncomposite loads having no initial influence on interface shear **stress** in an elastic girder/slab system, will ultimately be redistributed within the system as the ultimate moment capacity is approached. In order for the system to reach full nominal moment capacity the interface shear **strength** must exceed interface stresses produced by all loads associated with equilibrium of the section at full nominal moment capacity. And equilibrium at full nominal moment capacity is established in relation to all noncomposite and composite loads.

Although the preceding paragraph should be sufficient to stand on its own, the position expressed is consistent with text in Article 8.14.2 of the 17th Edition of the Standard Specifications (specifically Articles 8.14.2.1, 8.14.2.2 and 8.14.2.4).

C5.8.4.1:

Wording is included to acknowledge Article 5.13.2.4 where cohesion is neglected for certain types of structural systems. The proposed language recognizes that cohesion **may** or **may not** contribute to interface shear resistance.

Language is provided to aid designers in visualizing the mechanics of interface shear behavior. This language emphasizes the importance of proper detailing of reinforcement crossing the interface in consideration of the unintended, adverse consequences that could result from the inclusion of superfluous reinforcement. It is not uncommon to see interface reinforcement details that do not satisfy anchorage requirements.

Full-depth deck panels are not addressed by the proposed provisions. As is consistent with other unaddressed aspects of the Specifications, the matter is deferred to the Owner.

C5.8.4.2:

Notation is incorporated to assure that design forces and resistances are those associated with interface shear. Prior editions were confusing to many engineers as the units on load (demand) and resistance varied through the text. To eliminate confusion and simplify the application of the provisions the demand side of the equation has been “derived” based on equilibrium to establish that the interface shear force can be taken as the vertical shear force at the section under consideration (as ultimately expressed by Equations 5.8.4.2-1 and 5.8.4.2-2). This simply reestablishes the approach taken in the Standard Specifications for many years.

5.8.4.3:

Through a review of extensive experimental data taken from refereed journals published from 1960 to the present new values for cohesion, the limiting fraction of concrete strength that can be considered in design, and the maximum interface shear stress permitted have been adopted for use in Equation 5.8.4.1-3 to establish lower bound design expressions that correlate with the experimental data.

Bulleted items of Article 5.8.4.3 provide for the individual structural characteristics pertinent to a design. The parameter values (c, μ, K_1 and K_2) for the bulleted items were established as described in the following: (Noting that, in relation to each bullet item that has been re-evaluated based on a review of experimental data, the predicted resistance is **increased relative to the existing Specifications**. Albeit increases in the case of bullets 2, 3 and 4, that have been "arbitrarily" tempered by engineering judgment by application of the historical "strength reduction factor for shear" of 0.85).

Bullet 1: Cast-in-place concrete slab on clean concrete girder surfaces, free of laitance with surface roughened to an amplitude of 0.25-in.:

A substantial body of experimental data supports the parameter values as providing a close lower bound to the data. The inherent redundancy of girder/slab bridges and the load transfer mechanics further assure the lower bound resistances computed with these values are conservative for design.

Bullet 2: Normal weight concrete placed monolithically:

Commentary is provided to point out to designers that this is a special case that is somewhat in conflict with the underlying assumption of Article 5.8.4. That is, Article 5.8.4 is predicated on an existing **or potential** crack being assumed. For reinforced concrete this bullet would have limited application ("all concrete cracks") and should be used judiciously by designers.

Relatively limited data is available upon which to base definitive parameter values, particularly in view of the various geometric and loading conditions to which the design interface may be subjected. Consequently, the laboratory experimental data was plotted to establish a close lower bound and then a "strength reduction" factor for reinforced concrete (0.85) was applied to the lower bound expression. This approach was adopted in response to concern that the laboratory data seemed high; that laboratory specimens are typically of higher quality than that which results from field construction; and that data was particularly limited at low levels of clamping stress where concrete quality is likely to be of greater influence.

Bullet 3: Lightweight concretes placed monolithically, or nonmonolithically, against a clean concrete surface, free of laitance with surface intentionally roughened to an amplitude of 0.25-in.:

Experimental data does not warrant distinguishing between sand-lightweight and all-lightweight concrete. Due to limited data the same 0.85 "strength reduction" approach was taken as described above for normal weight monolithic concrete.

Bullet 4: Concrete placed against a clean concrete surface, free of laitance, with surface intentionally roughened to an amplitude of 0.25-in.:

Although extensive data exists, to account for variability in the geometry, loading and lack of redundancy at interfaces other than girder/slab interfaces, the same 0.85 "strength reduction" approach was applied to the data as for normal weight monolithic concrete and lightweight concretes.

Bullets 5 & 6: The parameter values are carried over from previous editions as they have not been re-evaluated at this time.

5.8.4.3:

Alternate minimum interface shear reinforcement requirements are provided for the case of cast-in-place slabs on girders. Minimum area requirements need not apply at low levels of interface shear demand, however, a means of achieving redundancy is available in girder/slab bridges by virtue of the presence of vertical shear reinforcement and such redundancy must be incorporated in the design by extending the vertical shear reinforcement across the interface.

C5.8.4.3:

The objective of eliminating the necessity of adding interface reinforcement as a function of interface width rather than interface shear demand is described. Designers are encouraged to recognize the dual role vertical shear reinforcement can serve, make prudent use of it by extending it across the interface, and satisfy resistance requirements with effective, efficient details.

Items #3 through #5 – Self-explanatory

ANTICIPATED EFFECT ON BRIDGES:

More economical design of components designed on the basis of interface shear transfer. With respect to girder/slab bridges, a reduction in mild reinforcing steel within the beam, increased job site safety by virtue of fewer bars projecting from the top of the beam that construction workers might trip over, and cost savings associated with future slab removal.

REFERENCES:

Hofbeck, J. A., Ibrahim, I. O., and Mattock, A. H., "Shear Transfer in Reinforced Concrete," *ACI Journal*, V. 66, No. 2, February 1969, pp. 119-128.

Mattock, A. H., Li, W. K., and Wang, T. C., "Shear Transfer in Lightweight Reinforced Concrete," *PCI Journal*, V. 21, No. 1, January-February 1976, pp. 20-39.

Loov, R. E., and Patnaik, A. K., "Horizontal Shear Strength of Composite Concrete Beams with a Rough Interface," *PCI Journal*, V. 39, No. 1, January-February 1994, pp. 48-69.

Patnaik, A. K., "Longitudinal Shear Strength of Composite Concrete Beams with a Rough Interface and no Ties," *Australian Journal of Structural Engineering*, Vol. SE1, No. 3, 1999, pp. 157-166.

Mattock, A. H., "Shear Friction and High-Strength Concrete," *ACI Structural Journal*, V. 98, No. 1, January-February 2001, pp. 50-59.

Mitchell, Andrew D., and Kahn, Lawrence F., "Shear Friction Behavior of High Strength Concrete," Structural Engineering, Mechanics and Materials Research Report No. 01-3, prepared for the Office of Materials and Research, Georgia Department of Transportation (GDOT Research Project No. 2005), School of Civil and Environmental Engineering, Georgia Institute of Technology, Atlanta, GA, January 2001, 183 pp.

Slapkus, Adam, and Kahn, Lawrence F., "Interface Shear in High Strength Composite T-Beams," *PCI Journal*, Vol. 49, No. 4, July/August 2004, pp. 102-110.

OTHER:

None

ATTACHMENT – 2006 AGENDA ITEM 15 - T-10 (REVISION 1) – 5/15/06

5.8.4 Interface Shear Transfer—Shear Friction

5.8.4.1 General

Interface shear transfer shall be considered across a given plane at:

- An existing or potential crack,
- An interface between dissimilar materials,
- An interface between two concretes cast at different times, or
- The interface between different elements of the cross-section.

Reinforcement for interface shear may consist of single bars, multiple leg stirrups, or welded wire fabric.

All reinforcement present where interface shear transfer is to be considered shall be fully developed on both sides of the interface by embedment, hooks, mechanical methods such as headed studs or welding to develop the design yield stress.

C5.8.4.1

Shear displacement along an interface plane may be resisted by cohesion, aggregate interlock, and shear-friction developed by the force in the reinforcement crossing the plane of the interface. Roughness of the shear plane causes interface separation in a direction perpendicular to the interface plane. This separation induces tension in the reinforcement balanced by compressive stresses on the interface surfaces.

Adequate shear transfer reinforcement must be provided perpendicular to the vertical planes of web/flange interfaces in box girders to transfer flange longitudinal forces at the strength limit state. The factored design force for the interface reinforcement is calculated to account for the interface shear force, ΔF, as shown in Figure C1, as well as any localized shear effects due to the prestressing force anchorages at the section.

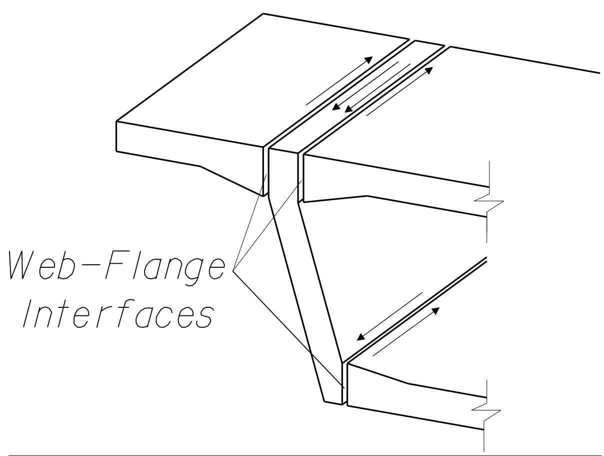

Figure C5.8.4.1-1 Longitudinal Shear Transfer Between Flanges and Webs of Box Girder Bridges.

Any reinforcement crossing the interface is subject to the same strain as the designed interface reinforcement. Insufficient anchorage of any reinforcement crossing the interface could result in localized fracture of the surrounding concrete.

When the required interface shear reinforcement in girder/slab design exceeds the area required to satisfy vertical (transverse) shear requirements, additional reinforcement must be provided to satisfy the interface shear requirements. The additional interface shear reinforcement need only extend into the girder a sufficient depth to develop the design yield stress of the reinforcement rather than extending the full depth of the girder as is required for vertical shear reinforcement.

The minimum area of interface shear reinforcement specified in Article 5.8.4.4 shall be satisfied.

The factored interface shear resistance, V_{ri}, shall be taken as:

$$V_{ri} = \phi V_{ni} \qquad (5.8.4.1\text{-}1)$$

and the design shall satisfy:

$$V_{ri} \geq V_{ui} \qquad (5.8.4.1\text{-}2)$$

where:

V_{ni} = nominal interface shear resistance (kip)

V_{ui} = factored interface shear force due to total load based on the applicable strength and extreme event load combinations in Table 3.4.1-1 (kip), and

Total load shall include all noncomposite and composite loads.

ϕ = resistance factor for shear specified in Article 5.5.4.2.1. In cases where different weight concretes exist on the two sides of an interface, the lower of the two values of ϕ shall be used.

For the extreme limit state event ϕ may be taken as 1.0.

The nominal shear resistance of the interface plane shall be taken as

$$V_{ni} = c A_{cv} + \mu [A_{vf} f_y + P_c] \qquad (5.8.4.1\text{-}3)$$

The nominal shear resistance, V_{ni}, used in the design shall not be greater than the lesser of:

$$V_{ni} \leq K_1 f'_c A_{cv}, \text{ or} \qquad (5.8.4.1\text{-}4)$$

$$V_{ni} \leq K_2 A_{cv} \qquad (5.8.4.1\text{-}5)$$

in which:

$$A_{cv} = b_{vi} L_{vi} \qquad (5.8.4.1\text{-}6)$$

A pure shear friction model assumes interface shear resistance is directly proportional to the net normal clamping force ($A_{vf}f_y + P_c$), through a friction coefficient (μ). Eq. 3 is a modified shear-friction model accounting for a contribution, evident in the experimental data, from cohesion and/or aggregate interlock depending on the nature of the interface under consideration given by the first term. For simplicity, the term "cohesion factor" is used throughout the body of this Article to capture the effects of cohesion and/or aggregate interlock such that Eq. 3 is analogous to the vertical shear resistance expression of $V_c + V_s$.

Eq. 4 limits V_{ni} to prevent crushing or shearing of aggregate along the shear plane.

Eqs. 3 and 4 are sufficient, with an appropriate value for K_1, to establish a lower bound for the available experimental data; however, Eq. 5 is necessitated by the sparseness of available experimental data beyond the limiting K_2 values provided in Article 5.8.4.3.

The ~~proposed~~ interface shear strength Eqs. 3, 4 and 5 are based on experimental data for normal weight, nonmonolithic concrete strengths ranging from 2.5 ksi to 16.5 ksi; normal weight, monolithic concrete strengths from 3.5 ksi to 18.0 ksi; sand-lightweight concrete strengths from 2.0 ksi to 6.0 ksi and all-lightweight concrete strengths from 4.0 ksi to 5.2 ksi.

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.

where:

A_{cv} = area of concrete considered to be engaged in interface shear transfer (in.^{2})

A_{vf} = area of interface shear reinforcement crossing the shear plane within the area A_{cv} (in.^{2})

b_{vi} = interface width considered to be engaged in shear transfer (in.)

L_{vi} = interface length considered to be engaged in shear transfer (in.)

c = cohesion factor specified in Article 5.8.4.3 (ksi)

μ = friction factor specified in Article 5.8.4.3 (dim.)

f_y = yield stress of reinforcement but design value not to exceed 60 (ksi)

P_c = permanent net compressive force normal to the shear plane; if force is tensile, P_c = 0.0 (kip)

f'_c = specified 28-day compressive strength of the weaker concrete on either side of the interface (ksi)

K_1 = fraction of concrete strength available to resist interface shear, as specified in Article 5.8.4.3.

K_2 = limiting interface shear resistance specified in Article 5.8.4.3 (ksi)

A_{vf} used in Eq. 3 is the interface shear reinforcement within the interface area A_{cv}. For a girder/slab interface, the area of the interface shear reinforcement per foot of girder length is calculated by replacing A_{cv} in Eq. 3 with 12 b_{vi} and P_c corresponding to the same one foot of girder length.

In consideration of the use of stay-in-place deck panels, or any other interface details, the designer shall determine the width of interface, b_{vi}, effectively acting to resist interface shear.

The interface reinforcement is assumed to be stressed to its design yield stress, f_y. However, f_y used in determining the interface shear resistance is limited to 60 ksi because interface shear resistance computed using higher values have overestimated the interface shear resistance experimentally determined in a limited number of tests of pre-cracked specimens.

It is conservative to neglect P_c if it is compressive, however, if included, the value of P_c shall be computed as the force acting over the area, A_{cv}. If P_c is tensile, additional reinforcement is required to resist the net tensile force as specified in Article 5.4.8.2.

5.8.4.2 Computation of the factored interface shear force, V_{ui}, for girder/slab bridges

Based on consideration of a free body diagram and utilizing the conservative envelope value of V_{u1}, the factored interface shear stress for a concrete girder/slab bridge may be determined as:

$$v_{ui} = V_{u1} / (b_{vi} \times d_v) \qquad (5.8.4.2\text{-}1)$$

where:

d_v = the distance between the centroid of the tension steel and the mid-thickness of the slab to compute a factored interface shear stress

The factored interface shear force in kips per foot for a concrete girder/slab bridge may be determined as:

C5.8.4.2

The following illustrates a free body diagram approach to computation of interface shear in a girder/slab bridge. In reinforced concrete, or prestressed concrete, girder bridges, with a cast-in-place slab, horizontal shear forces develop along the interface between the girders and the slab. The classical strength of materials approach, which is based on elastic behavior of the section, has been used successfully in the past to determine the design interface shear force. As an alternative to the classical elastic strength of materials approach, a reasonable approximation of the factored interface shear force at the strength or extreme event limit state for either elastic or inelastic behavior and cracked or uncracked sections, can be derived with the defined notation and the free body diagram shown in Figure C1 as follows:

$V_{ui} = v_{ui} \times A_{cv} = v_{ui} \times 12 \times b_{vi}$ (5.8.4.2-2)

If the net force, P_c, across the interface shear plane is tensile, additional reinforcement, A_{vpc}, shall be provided as:

$A_{vpc} = P_c / \phi f_y$. (5.8.4.2-3)

For beams and girders, the longitudinal spacing of the rows of interface shear transfer reinforcing bars shall not exceed 24.0 in.

M_{u2} = maximum factored moment at section 2

V_1 = the factored vertical shear at section 1 concurrent with M_{u2}

M_1 = the factored moment at section 1 concurrent with M_{u2}

Δl = unit length segment of girder

C_1 = compression force above the shear plane associated with M_1

C_{u2} = compression force above the shear plane associated with M_{u2}

$M_{u2} = M_1 + V_1 \Delta l$ (C5.8.4.2-1)

$C_{u2} = M_{u2} / d_v$ (C5.8.4.2-2)

$C_{u2} = M_1 / d_v + V_1 \Delta l / d_v$ (C5.8.4.2-3)

$C_1 = M_1 / d_v$ (C5.8.4.2-4)

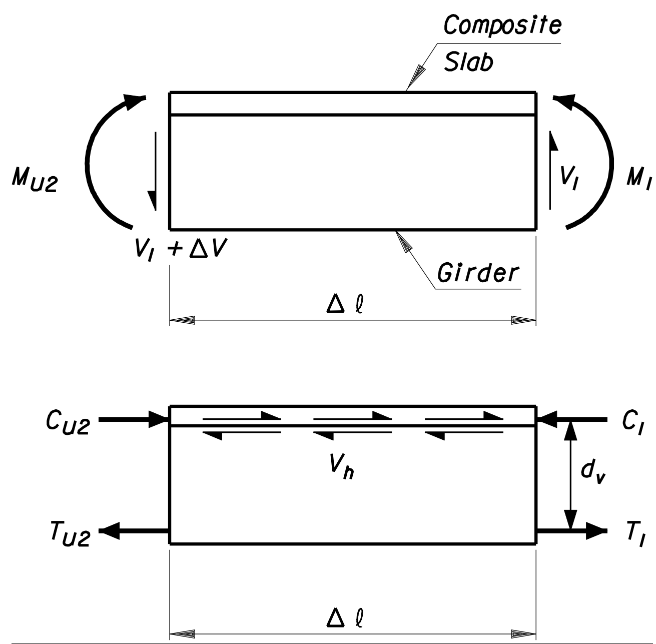

Figure C5.8.4.2-1 Free Body Diagrams.

$V_h = C_{u2} - C_1$ (C5.8.4.2-5)

$V_h = V_1 \Delta l / d_v$ (C5.8.4.2-6)

Such that for a unit length segment,

$V_{hi} = V_1 / d_v$ (C5.8.4.2-7)

where,

V_{hi} = factored interface shear force per unit length (kips/length)

The variation of V_1 over the length of any girder segment reflects the shear flow embodied in the classical strength of materials approach. For simplicity of design, V_1 can be conservatively taken as V_{u1} (since V_{u1}, the maximum factored vertical shear at section 1, is not likely to act concurrently with the factored moment at section 2); and further, the depth, d_v, can be taken as the distance between the centroid of the tension steel and the mid-thickness of the slab to compute a factored interface shear stress.

For design purposes the computed factored interface shear stress of Eq. 1 is converted to a resultant interface shear force computed with Eq. 2 acting over an area, A_{cv}, within which the computed area of reinforcement, A_{vf}, shall be located. The resulting area of reinforcement, A_{vf}, then defines the area of interface reinforcement required per foot of girder for direct comparison with vertical shear reinforcement requirements.

5.8.4.3 Cohesion and Friction Factors

The following values shall be taken for cohesion, c, and friction factor, μ:

- For a cast-in-place concrete slab on clean concrete girder surfaces, free of laitance with surface roughened to an amplitude of 0.25-in.

  $c = 0.28$ ksi
  $\mu = 1.0$
  $K_1 = 0.3$
  $K_2 = 1.8$ ksi for normal weight concrete
  $= 1.3$ ksi for lightweight concrete

- For normal weight concrete placed monolithically

  $c = 0.40$ ksi
  $\mu = 1.4$
  $K_1 = 0.25$
  $K_2 = 1.5$ ksi

- For lightweight concrete placed monolithically, or nonmonolithically, against a clean concrete surface, free of laitance with surface intentionally roughened to an amplitude of 0.25-in.

  $c = 0.24$ ksi
  $\mu = 1.0$
  $K_1 = 0.25$
  $K_2 = 1.0$ ksi

C5.8.4.3

The values presented provide a lower bound of the substantial body of experimental data available in the literature (Loov & Patnaik, 1994; Patnaik, 1999; Mattock, 2001; Slapkus & Kahn, 2004). Furthermore, the inherent redundancy of girder/slab bridges distinguishes this system from other structural interfaces.

The values presented apply strictly to monolithic concrete. These values are not applicable for situations where a crack may be anticipated to occur at a Service Limit State.

The factors presented provide a lower bound of the experimental data available in the literature (Hofbeck, Ibrahim and Mattock, 1969; Mattock, Li and Wang, 1976; Mitchell and Kahn, 2001).

Available experimental data demonstrates that only one modification factor is necessary, when coupled with the resistance factors of Article 5.5.4.2, to accommodate both all-lightweight and sand-lightweight concrete. Note this deviates from earlier specifications that distinguished between all-lightweight and sand-lightweight concrete.

Due to the absence of existing data the prescribed cohesion and friction factors for nonmonolithic lightweight concrete are accepted as conservative for application to monolithic lightweight concrete.

- For normal weight concrete placed against a clean concrete surface, free of laitance, with surface intentionally roughened to an amplitude of 0.25-in.

  $c = 0.24$ ksi
  $\mu = 1.0$
  $K_1 = 0.25$
  $K_2 = 1.5$ ksi

- For concrete placed against a clean concrete surface, free of laitance, but not intentionally roughened

  $c = 0.075$ ksi
  $\mu = 0.6$
  $K_1 = 0.2$
  $K_2 = 0.8$ ksi

- For concrete anchored to as-rolled structural steel by headed studs or by reinforcing bars where all steel in contact with concrete is clean and free of paint

  $c = 0.025$ ksi
  $\mu = 0.7$
  $K_1 = 0.2$
  $K_2 = 0.8$ ksi

Tighter constraints have been adopted for roughened interfaces, other than cast-in-place slabs on roughened girders, even though available test data does not indicate more severe restrictions are necessary. This is to account for variability in the geometry, loading and lack of redundancy at other interfaces.

For brackets, corbels, and ledges, the cohesion factor, c, shall be taken as 0.0.

Since the effectiveness of cohesion and aggregate interlock along a vertical crack interface is unreliable the cohesion component in Eq. 5.8.4.1-3 is set to 0.0 for brackets, corbels, and ledges

5.8.4.4 MINIMUM AREA OF INTERFACE SHEAR REINFORCEMENT

Except as provided herein, the cross-sectional area of the interface shear reinforcement, A_{vf}, crossing the interface area, A_{cv}, shall satisfy:

$$A_{vf} \geq \frac{0.05\ A_{cv}}{f_y} \qquad (5.8.4.4\text{-}1)$$

C5.8.4.4

For a girder/slab interface, the minimum area of interface shear reinforcement per foot of girder length is calculated by replacing A_{cv} in Eq. 1 with 12 b_{vi}.

For a cast-in-place concrete slab on clean concrete girder surfaces, free of laitance ~~with surface roughened to an amplitude of 0.25 in.~~, the following provisions shall apply:

- The minimum interface shear reinforcement, A_{vf}, need not exceed the lesser of the amount determined using Eq. 1 and the amount needed to resist 1.33 V_{ui}/ϕ as determined using Eq. 5.8.4.1-3.
- The minimum reinforcement provisions specified herein shall be waived for girder/slab interfaces with surface roughened to an amplitude of 0.25 in. where the factored interface shear stress, v_{ui} of Eq. 5.8.4.2-1, is less than 0.210 ksi, and all vertical (transverse) shear reinforcement required by the provisions of Article 5.8.1.1 is extended across the interface and adequately anchored in the slab.

Previous editions of these specifications and of the AASHTO Standard Specifications have required a minimum area of reinforcement based on the full interface area; similar to Eq. 1, irrespective of the need to mobilize the strength of the full interface area to resist the applied factored interface shear. In 2006, the additional minimum area provisions, applicable only to girder/slab interfaces, were introduced. The intent of these provisions was to eliminate the need for additional interface shear reinforcement due simply to a beam with a wider top flange being utilized in place of a narrower flanged beam.

The additional provision establishes a rational upper bound for the area of interface shear reinforcement required based on the interface shear demand rather than the interface area as stipulated by Eq. 1. This treatment is analogous to minimum reinforcement provisions for flexural capacity where a minimum additional overstrength factor of 1.33 is required beyond the factored demand.

With respect to a girder/slab interface, the intent is that the portion of the reinforcement required to resist vertical shear which is extended into the slab also serves as interface shear reinforcement.

2006 AASHTO BRIDGE COMMITTEE AGENDA ITEM: 15A (NEW)

SUBJECT: LRFD Bridge Design Specifications: Section 5, Various Articles, Editorial Changes

TECHNICAL COMMITTEE: T-10 Concrete

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

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DATE PREPARED: 5/22/06
DATE REVISED:

AGENDA ITEM:

Item #1

Revise Articles 5.4.2.3.2 as follows:

5.4.2.3.2 Creep

The creep coefficient may be taken as:

$$\psi(t, t_i) = 1.9 k_s k_{hc} k_f k_{td} t_i^{-0.118} \qquad (5.4.2.3.2\text{-}1)$$

for which:

$$k_s = 1.45 - 0.13(V/S) \geq 1.0 \qquad (5.4.2.3.2\text{-}2)$$

$$k_{hc} = 1.56 - 0.008H \qquad (5.4.2.3.2\text{-}3)$$

$$k_f = \frac{5}{1 + f'_{ci}} \qquad (5.4.2.3.2\text{-}4)$$

$$k_{td} = \left(\frac{t}{61 - 4f'_{ci} + t}\right) \qquad (5.4.2.3.2\text{-}5)$$

where:

H = relative humidity (%). In the absence of better information, H may be taken from Figure 5.4.2.3.3-1.
k_s = factor for the effect of the volume-to-surface ratio of the component
k_f = factor for the effect of concrete strength
k_{hc} = humidity factor for creep
k_{td} = time development factor
t = maturity of concrete (day), defined as age of concrete between time of loading for creep calculations, or

end of curing for shrinkage calculations, and time being considered for analysis of creep or shrinkage effects.

t_i = age of concrete ~~when load is initially applied~~ <u>at time of load application</u> (day)

V/S = volume-to-surface ratio (in.)

f'_{ci} = specified compressive strength of concrete at time of prestressing for pretensioned members and at time of initial loading for non-prestressed members. If concrete age at time of initial loading is unknown at design time, f'_{ci} may be taken as 0.80 f'_c (ksi)

~~In determining the maturity of concrete at initial loading, t_i, one day of accelerated curing by steam or radiant heat may be taken as equal to seven days of normal curing.~~

The surface area used in determining the volume to area ratio should include only the area that is exposed to atmospheric drying. For poorly ventilated enclosed cells, only 50 percent of the interior perimeter should be used in calculating the surface area. <u>For precast members with cast-in-place topping, the total precast surface should be used.</u> For pretensioned stemmed members (I-beams, T-beams, and box beams), with an average web thickness of 6 to 8 IN, the value of k_s may be taken as 1.00.

<u>Item #2</u>

Revise Equation 5.4.2.3.3-1 as follows:

$$\varepsilon_{sh} = \cancel{-} k_s k_{hs} k_f k_{td} \, 0.48 \times 10^{-3}$$ <u>(5.4.2.3.3-1)</u>

<u>Item #3</u>

In Article 5.9.5.2.3a, revise the following definition in the where list as follows:

f_{cgp} = the concrete stress at the center of gravity of prestressing tendons due to the prestressing force immediately after transfer and the self-weight of the member at the section of maximum moment (ksi). ~~For the purpose of estimating f_{cgp}, the prestressing force immediately after transfer may be assumed to be equal to 0.9 of the force just before transfer; also, change of concrete stress at the center of gravity of prestressing tendons due to subsequent applied loads, when considered.~~

<u>Item #4</u>

In Article 5.9.5.3, revise the following definition in the where list as follows:

Δf_{pR} = an estimate of relaxation loss taken as ~~2.5~~ <u>2.4</u> ksi for low relaxation strand, 10.0 ksi for stress relieved strand, and in accordance with manufacturers recommendation for other types of strand (ksi)

<u>Item #5</u>

In Article 5.9.5.4.1, revise the following definition in the where list as follows:

Δf_{pSS} = prestress ~~loss~~ <u>gain</u> due to shrinkage of deck <u>in</u> composite section (ksi)

<u>Item #6</u>

In Article 5.9.5.4.2a, revise the following definition in the where list as follows:

e_{pg} = eccentricity of ~~strands~~ <u>prestressing force</u> with respect to centroid of girder (in.)<u>, positive in common construction</u>

where it is below girder centroid

Item #7

In Article 5.9.5.4.3a, revise the following definitions in the where list as follows:

e_{pc} = eccentricity of ~~strands~~ prestressing force with respect to centroid of composite section (in.), positive in typical construction where prestressing force is below centroid of section

A_c = area of section calculated using the ~~net~~ gross composite concrete section properties of the girder and the deck and the deck-to-girder modular ratio

I_c = moment of inertia of section calculated using the ~~net~~ gross composite concrete section properties of the girder and the deck and the deck-to-girder modular ratio at service

Item #8

Revise Article 5.9.5.4.3b as follows:

5.9.5.4.3b Creep of Girder Concrete

The change in prestress ~~loss~~ (loss is positive, gain is negative) due to creep of girder concrete between time of deck placement and final time, Δf_{pCD}, shall be determined as:

$$\cancel{\Delta f_{pCD} = \frac{E_p}{E_{ci}} f_{cgp}\left(\psi_b\left(t_f, t_i\right) - \psi_b\left(t_d, t_i\right)\right)K_{df} + \frac{E_p}{E_c}\Delta f_{cd}\psi_b\left(t_f, t_d\right)K_{df} \geq 0.0} \qquad \cancel{(5.9.5.4.3b\text{-}1)}$$

$$\Delta f_{pCD} = \frac{E_p}{E_{ci}} f_{cgp}\psi_b\left[\left(t_f, t_i\right) - \psi_b\left(t_d, t_i\right)\right]K_{df} + \frac{E_p}{E_c}\Delta f_{cd}\psi_b\left(t_f, t_d\right)K_{df} \qquad (5.9.5.4.3b\text{-}1)$$

(Remainder of the Article is unchanged)

Item #9

Delete Article C5.9.5.4.3b in its entirety.

Item #10

Revise Article 5.9.5.4.3d as follows:

5.9.5.4.3d Shrinkage of Deck Concrete

The prestress ~~loss~~ gain due to shrinkage of deck composite section, Δf_{pSS}, shall be determined as:

$$\Delta f_{pSS} = \frac{E_p}{E_c}\Delta f_{cdf} K_{df}\left(1 + 0.7\psi_b\left(t_f, t_d\right)\right) \qquad (5.9.5.4.3d\text{-}1)$$

in which:

$$\cancel{\Delta f_{cdf} = \frac{\varepsilon_{ddf} A_d E_{cd}}{\left(1+0.7\psi_d(t_f,t_d)\right)}\left(\frac{1}{A_c}+\frac{e_{pc}e_d}{I_c}\right)} \quad \text{~~(5.9.5.4.3d-2)~~}$$

$$\underline{\Delta f_{cdf} = \frac{\varepsilon_{ddf} A_d E_{cd}}{\left(1+0.7\psi_d(t_f,t_d)\right)}\left(\frac{1}{A_c}-\frac{e_{pc}e_d}{I_c}\right)} \quad \underline{\text{(5.9.5.4.3d-2)}}$$

where:

Δf_{cdf} = change in concrete stress at centroid of prestressing strands due to shrinkage of deck concrete (ksi)

ε_{ddf} = shrinkage strain of deck concrete between placement and final time per Eq. 5.4.2.3.3-1

A_d = area of deck concrete (in.^{2})

E_{cd} = modulus of elasticity of deck concrete (ksi)

e_d = eccentricity of deck with respect to the ~~transformed net~~ gross composite section, ~~taken negative~~ positive in ~~common~~ typical construction where deck is above girder (in.)

(Remainder of the Article is unchanged)

OTHER AFFECTED ARTICLES:

None

BACKGROUND:

None

ANTICIPATED EFFECT ON BRIDGES:

None

REFERENCES:

None

OTHER:

None

2005 AASHTO BRIDGE COMMITTEE AGENDA ITEM: 18

SUBJECT: LRFD Bridge Design Specifications: Section 6, Article 6.10, Various Articles

TECHNICAL COMMITTEE: T-14 Steel

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

☒ **DESIGN SPEC** ☐ **CONSTRUCTION SPEC** ☐ **MOVABLE SPEC**
☐ **LRFR MANUAL** ☐ **OTHER**

☐ **US VERSION** ☐ **SI VERSION** ☒ **BOTH**

DATE PREPARED: 08-10-05
DATE REVISED:

AGENDA ITEM:

Item #1

Revise the first sentence of the last paragraph of Article 6.10.8.2.3 to read as follows:

For unbraced lengths in which the member is nonprismatic, the lateral torsional buckling resistance of the compression flange F_{nc} at each section within the unbraced length may be taken as the smallest resistance within the unbraced length under consideration determined from Eq. 1, 2, or 3, as applicable, assuming the unbraced length is prismatic.

Add the following after the first sentence of the last paragraph of Article 6.10.8.2.3:

The moment gradient modifier, C_b, shall be taken equal to 1.0 in this case and L_b shall not be modified by an effective length factor.

Start a new paragraph with the current second sentence of the last paragraph of Article 6.10.8.2.3 and add the following to the end of that sentence:

provided the lateral moment of inertia of the flange or flanges of the smaller section is equal to or larger than one-half the corresponding value in the larger section.

Item #2

Revise the last paragraph of Article C6.10.8.2.3 to read as follows:

For unbraced lengths containing a transition to a smaller section at a distance greater than 20 percent of the unbraced length from the brace point with the smaller moment, the lateral torsional buckling resistance should be taken as the smallest resistance, F_{nc}, within the unbraced length under consideration. ~~This resistance is to be compared to the largest value of the compressive stress due to the factored loads,~~ ~~f_{bu},~~ ~~throughout the unbraced length calculated using the actual properties of the section.~~ This approximation is based on replacing the nonprismatic member with an equivalent prismatic member. The cross-section of the equivalent member that gives the correct lateral torsional buckling resistance is generally some weighted average of all the cross-sections along the unbraced length. If the cross-section within the unbraced length that gives the smallest uniform bending resistance is used, and the calculated resistance is not exceeded at any section along the unbraced length, a conservative solution is obtained. ~~The moment gradient modifier,~~ ~~C_b,~~ ~~should be taken equal to 1.0 in this case and~~ ~~L_b~~ ~~should not be modified by an effective length factor.~~ A suggested procedure to provide a more refined estimate of the lateral torsional buckling resistance for this case is presented in Grubb and Schmidt *(2004).*

Add the following new paragraph to the end of Article C6.10.8.2.3:

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.

Item #3

Add the following after the first sentence of the last paragraph of Article A6.3.3:

The flexural resistance M_{nc} at each section within the unbraced length shall be taken equal to this resistance multiplied by the ratio of S_{xc} at the section under consideration to S_{xc} at the section governing the lateral torsional buckling resistance. The moment gradient modifier, C_b, shall be taken equal to 1.0 in this case and L_b shall not be modified by an effective length factor.

Start a new paragraph with the current second sentence of the last paragraph of Article A6.3.3 and add the following to the end of that sentence:

provided the lateral moment of inertia of the flange or flanges of the smaller section is equal to or larger than one-half the corresponding value in the larger section.

Item #4

Revise the next-to-the-last paragraph of Article CA6.3.3 to read as follows:

The effect of the variation in the moment along the length between brace points is accounted for by using the moment gradient modifier, C_b. Article C6.10.8.2.3 discusses the C_b parameter in detail. ~~Also, this article provides guidelines for handling of~~ Article 6.10.8.2.3 addresses unbraced lengths in which the member is nonprismatic. Article A6.3.3 extends the provisions for such unbraced lengths to members with compact and noncompact webs. ~~These guidelines are equally applicable to the types of sections addressed within this article with the following exception. For unbraced lengths containing a transition to a smaller section at a distance greater than 20 percent of the unbraced length from the brace point with the smaller moment, the lateral torsional buckling resistance,~~ $\cancel{M_{nc}}$~~, is to be taken as the smallest resistance within the unbraced length under consideration times the ratio of~~ $\cancel{S_{xc}}$ ~~at the section under consideration to~~ $\cancel{S_{xc}}$ ~~at the section governing the lateral torsional buckling resistance.~~

Item #5

Remove the last paragraph of Article D6.4.1.

Item #6

Remove the last paragraph of Article D6.4.2.

OTHER AFFECTED ARTICLES:

None

BACKGROUND:

The proposed revisions are intended to clarify the application of the suggested approximate procedure for determining the flexural resistance based on lateral torsional buckling for unbraced lengths in which the member is nonprismatic.

ANTICIPATED EFFECT ON BRIDGES:

None

REFERENCES:

None

OTHER:

None

2006 AASHTO BRIDGE COMMITTEE AGENDA ITEM: 19 (REVISION 1)

SUBJECT: LRFD Bridge Design Specifications: Section 6, Article 6.11, Various Articles

TECHNICAL COMMITTEE: T-14 Steel

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

☒ **DESIGN SPEC** ☐ **CONSTRUCTION SPEC** ☐ **MOVABLE SPEC**
☐ **LRFR MANUAL** ☐ **OTHER**

☐ **US VERSION** ☐ **SI VERSION** ☒ **BOTH**

DATE PREPARED: 08-11-05
DATE REVISED: 05-22-06

AGENDA ITEM:

Item #1

Add the following to the end of the last paragraph of Article C6.11.8.2.2:

In calculating R_b and R_h for a tub section, use one-half of the effective box flange width in conjunction with one top flange and a single web, where the effective box flange width is defined in Article 6.11.1.1. For a closed-box section, use one-half of the effective top and bottom box flange width in conjunction with a single web.

Item #2

In Article 6.11.8.2.3, change the range on the plate-buckling coefficient *k* underneath Eq. 6.11.8.2.3-2 to read as follows:

$\underline{1.0 \le k \le 4.0}$ and:

Item #3

Revise the fifth sentence of the second paragraph of Article C6.11.8.2.3 to read as follows:

k can take any value ranging from ~~2.0~~ 1.0 to 4.0. However, a value of *k* ranging from 2.0 to 4.0 generally should be assumed.

Revise the second sentence of the third paragraph of Article C6.11.8.2.3 to read as follows:

Another option in lieu of using Eq. 1 or 2 is to assume a *k* value ~~ranging from 2.0 to 4.0~~ and then determine the minimum required moment of inertia for each longitudinal flange stiffener from Eq. 6.11.11.2-2 that will provide the assumed value of *k*.

Item #4

Revise the next-to-the-last paragraph of Article 6.11.9 to read as follows:

For box flanges, b_{fc} or b_{ft}, as applicable, shall be taken as one-half of the effective flange width between webs in checking Eq. 6.10.9.3.2-1~~.~~, where the effective flange width shall be taken as specified in Article 6.11.1.1, but not to exceed $18t_f$ where t_f is the thickness of the box flange.

Item #5

In Article 6.11.11.2, change the range on the plate-buckling coefficient *k* in the where list to read as follows:

$1.0 \leq k \leq 4.0$

Item #6

Revise the second sentence of the third paragraph of Article C6.11.11.2 to read as follows:

Alternatively, a k value ~~ranging from 2.0 to 4.0~~ can be assumed in lieu of using Eq. 6.11.8.2.3-1 or 6.11.8.2.3-2. *k* can take any value ranging from 1.0 to 4.0. However, a value of *k* ranging from 2.0 to 4.0 generally should be assumed.

OTHER AFFECTED ARTICLES:

None

BACKGROUND:

A revision to Article C6.11.8.2.2 is proposed to clarify that in the calculation of the flange-strength reduction factors, R_b and R_h, a tub or closed-box section can essentially be treated as an equivalent I-section using one-half of the effective box-flange width (note that a box flange is defined in the specification as a flange connected to two webs). Also, a revision is proposed to Article 6.11.9 to clarify that when checking Eq. 6.10.9.3.2-1 for a single web in a tub or closed-box section to determine whether or not the full tension-field shear resistance can be utilized, one-half of the effective box flange width is to be used in the equation for b_{fc} or b_{ft}, as applicable, but not to exceed $18t_f$ where t_f is the thickness of the box flange.

Revisions are proposed to Articles 6.11.8.2.3, C6.11.8.2.3, 6.11.11.2 and C6.11.11.2 to revise the lower limit on the box flange plate-buckling coefficient *k* from 2.0 to 1.0 to allow the Engineer greater flexibility in the selection of *k*. However, it is also strongly suggested that a *k* value ranging from 2.0 to 4.0 be assumed to ensure greater overall economy and stability.

ANTICIPATED EFFECT ON BRIDGES:

None

REFERENCES:

None

OTHER:

None

2006 AASHTO BRIDGE COMMITTEE AGENDA ITEM: 20

SUBJECT: AASHTO/NSBA Collaboration Documents (G 1.4, G 4.2, G 4.4 and S 8.1)

TECHNICAL COMMITTEE: T-14 Steel

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

☐ **DESIGN SPEC** ☐ **CONSTRUCTION SPEC** ☐ **MOVABLE SPEC**
☐ **LRFR MANUAL** ☒ **OTHER AASHTO/NSBA Steel Bridge Collaboration**

☐ **US VERSION** ☐ **SI VERSION** ☐ **BOTH**

DATE PREPARED: 01/13/06
DATE REVISED:

AGENDA ITEM:

See attached Guidelines:

G 1.4 – Guidelines for Design Details
G 4.2 – Recommendations for the Qualification of Structural Bolting Inspectors
G 4.4 – Sample Owners Quality Assurance Manual
S 8.1 – Guide for Application of Coating Systems with Zinc-Rich Primers to Steel Bridges

OTHER AFFECTED ARTICLES:

None

BACKGROUND:

G 1.4, G 4.2, and G 4.4 are new Collaboration documents.

This edition S8.1 is an update to the 2002 edition of S8.1. A significant change is that S8.1 now also incorporates organic zinc rich primer systems as well as inorganic systems. Note that the title has been changed accordingly.

ANTICIPATED EFFECT ON BRIDGES:

None

REFERENCES:

None

OTHER:

None

AASHTO/NSBA Steel Bridge Collaboration

G 1.4 - 2006

Guidelines for Design Details

AASHTO/NSBA Steel Bridge Collaboration

Preface

This document is a standard developed by the AASHTO/NSBA Steel Bridge Collaboration. The primary goal of the Collaboration is to achieve steel bridge design and construction of the highest quality and value through standardization of the design, fabrication, and erection processes. Each standard represents the consensus of a diverse group of professionals. A listing of those serving on the Committee which developed the *Guide Specification for Application of Coating Systems with Zinc-Rich Primers* will be included in future editions.

As consensus documents, the Collaboration standards represent the best available current approach to the processes they cover. It is intended that Owners adopt and implement Collaboration standards in their entirety to facilitate the achievement of standardization. It is understood, however, that local statutes or preferences may prevent full adoption of the document. In such cases Owners should adopt these documents with the exceptions they feel are necessary.

Disclaimer

INDEX OF SHEETS

NOTES:

NUMBER AND DIAMETER OF BOLTS, WELD SIZES AND TYPES , LINES ,ROWS, COLUMNS OF HOLES, AND MATERIAL DIMENSIONS SHOWN ON ALL THESE SHEETS ARE FOR PRESENTATION PURPOSES ONLY. IT IS THE RESPONSIBILITY OF THE DESIGNER TO DETERMINE THE NUMBER AND DIAMETER OF BOLTS, WELD SIZES AND TYPES , AS WELL AS THE SIZES OF MEMBERS AND CONNECTION PLATE SIZES.

DISCLAIMER NOTE

INFORMATION SHOWN IS FOR CONCEPT ONLY. APPLICATION TO SPECIFIC STRUCTURES IS THE DESIGNER'S RESPONSIBILITY.

STRUCTURAL STEEL INDEX OF SHEETS

AASHTO/NSBA STEEL BRIDGE COLLABORATION

TASK GROUP 1, SUBTASK - GROUP 1.4
GUIDELINES FOR DESIGN DETAILS

PAGE NO. 101

MATERIAL : (UNLESS NOTED OTHERWISE)

ALL STRUCTURAL STEEL, FOR PAINTED PROJECTS, SHALL BE ASTM A709, GRADE 50. EXCEPT THAT STIFFENERS, INTERMEDIATE AND END CROSSFRAMES AND LATERAL BRACING MAY BE GRADE 36.
USE ASTM A709-50W FOR ALL MATERIAL ON UNPAINTED JOBS.
ALL BOLTS FOR PAINTED STEEL SHALL BE A325-TYPE 1 (MECH GALV) ROTATIONAL CAPACITY TESTED (RCT)
ALL BOLTS FOR UN-PAINTED STEEL SHALL BE A325-TYPE 3 ROTATIONAL CAPACITY TESTED (RCT)
ALL STRUCTURAL STEEL SHALL BE OF PRODUCED DOMESTICALLY ON FEDERAL AID PROJECTS.

FRACTURE CRITICAL MEMBERS:

DESIGN PLANS SHALL DESIGNATE EACH MEMBER OR COMPONENT THAT MUST MEET THE AASHTO REQUIREMENTS FOR FRACTURE CRITICAL MEMBERS. THE SPECIAL PROVISIONS FOR "FRACTURE CRITICAL" SHALL CONFORM TO THE PROVISIONS OF THE CURRENT AASHTO FRACTURE CONTROL PLAN. THE FABRICATOR MUST HAVE THE AISC FRACTURE CRITICAL ENDORSEMENT.

CHARPY V-NOTCH:

DESIGN DRAWINGS SHALL IDENTIFY ALL MAIN LOAD CARRYING MEMBERS.
ALL STRUCTURAL STEEL AS DESIGNATED ON THE PLANS SHALL RECEIVE CHARPY V-NOTCH TESTING IN ACCORDANCE WITH ASTM A709 AND SUPPLEMENTAL REQUIREMENT S83 OR S84 AS NOTED BELOW.

1. TENSION COMPONENTS OF REDUNDANT MEMBERS, DESIGNATED "T" ON THE PLANS, SHALL BE TESTED IN ACCORDANCE WITH TABLE S1.2 OF ASTM A709 (S83)

2. TENSION COMPONENTS OF FRACTURE CRITICAL MEMBERS, DESIGNATED "F" ON THE PLANS, SHALL BE TESTED IN ACCORDANCE WITH TABLE S1.3 OF ASTM A709 (S84)

STEEL FABRICATION:

FABRICATION SHALL BE PERFORMED IN ACCORDANCE WITH THE NSBA/AASHTO COLLABORATION DOC S2.1-2002 "STEEL BRIDGE FABRICATION GUIDE SPEC."
FABRICATORS OF STRUCTURAL STEEL SHALL HAVE THE APPROPRIATE AISC QUALITY CERTIFICATION.

WELDING:

1. WELDING DETAILS AND THE WELDING OPERATIONS SHALL BE IN ACCORDANCE WITH THE CURRENT EDITION OF THE AASHTO/AWS D1.5 BRIDGE WELDING CODE. WELDING PROCEDURES SHALL BE SUBMITTED AND APPROVED PRIOR TO WELDING ON PROJECT. NON-DESTRUCTIVE TESTING SHALL BE PERFORMED AS REQUIRED BY THE CURRENT EDITION OF THE ANSI/AASHTO/AWS D1.5 BRIDGE WELDING CODE.

2. THE FOLLOWING MEMBERS ARE CLASSIFIED AS ANCILLARY MEMBERS IN ACCORDANCE WITH THE CURRENT EDITION OF THE ANSI/AWS D1.5 BRIDGE WELDING CODE:

   A. EXPANSION JOINT

   B. DRAINAGE SYSTEM

   C. OTHER MISCELLANEOUS MATERIAL

FIELD CONNECTIONS:

ALL BOLTED CONNECTIONS SHALL USE 7/8 " DIAMETER ASTM A325 BOLTS UNLESS OTHERWISE NOTED, INSTALLED PER THE RESEARCH COUNCIL ON STRUCTURAL CONNECTIONS (RCSC). BOLTS FOR SLIP CRITICAL CONNECTIONS (SPLICES, CURVED BEAM/GIRDER BRACING) SHALL HAVE ROTATIONAL CAPACITY TESTING (RCT).

PAINTING IF REQUIRED:

STRUCTURAL STEEL SHALL BE PAINTED WITH A COATING SYSTEM IN ACCORDANCE WITH THE APPROPIATE SECTION OF THE OWNERS SPECIFICATIONS, AND THE AASHTO/NSBA DOCUMENT ON PAINTING (TG8). S8.1-2002,"GUIDE SPECIFICATIONS FOR INORGANIC ZINC-RICH PRIMER BASED COATING SYSTEMS" AND S2.1-2002,"STEEL BRIDGE FABRICATION GUIDE SPEC."

DRAWING PRESENTATION AND APPROVAL GUIDELINES:

DESIGN DRAWINGS SHOULD BE PREPARED IN ACCORDANCE WITH THE AASHTO/NSBA "DESIGN PLAN PRESENTATION GUIDELINES" G1.2
DETAIL DRAWINGS SHOULD BE PREPARED IN ACCORDANCE WITH THE AASHTO/NSBA "SHOP DETAIL DRAWINGS PRESENTATION GUIDELINES" G1.3
SHOP DETAIL DRAWINGS SHOULD BE REVIEWED IN ACCORDANCE WITH THE AASHTO/NSBA "SHOP DETAIL DRAWING REVIEW/APPROVAL GUIDELINES" G1.1

NOTES TO DESIGNERS:

A... 1" DIAMETER HSB CAN BE USED TO REDUCE THE NUMBER OF BOLTS REQUIRED IN LARGE SPLICES OR CONNECTIONS BASED ON A RE-DESIGN BY A LICENSED ENGINEER AT THE CONTRACTOR'S EXPENSE.

DISCLAIMER NOTE

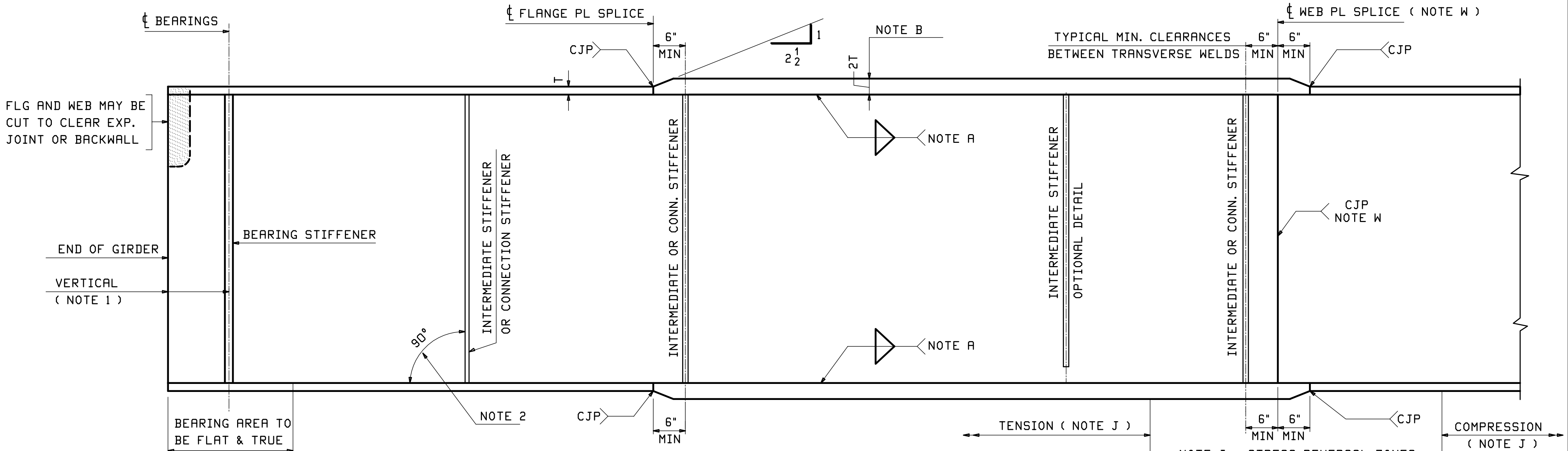
INFORMATION SHOWN IS FOR CONCEPT ONLY. APPLICATION TO SPECIFIC STRUCTURES IS THE DESIGNER'S RESPONSIBILITY.

GENERAL NOTES STRUCTURAL STEEL

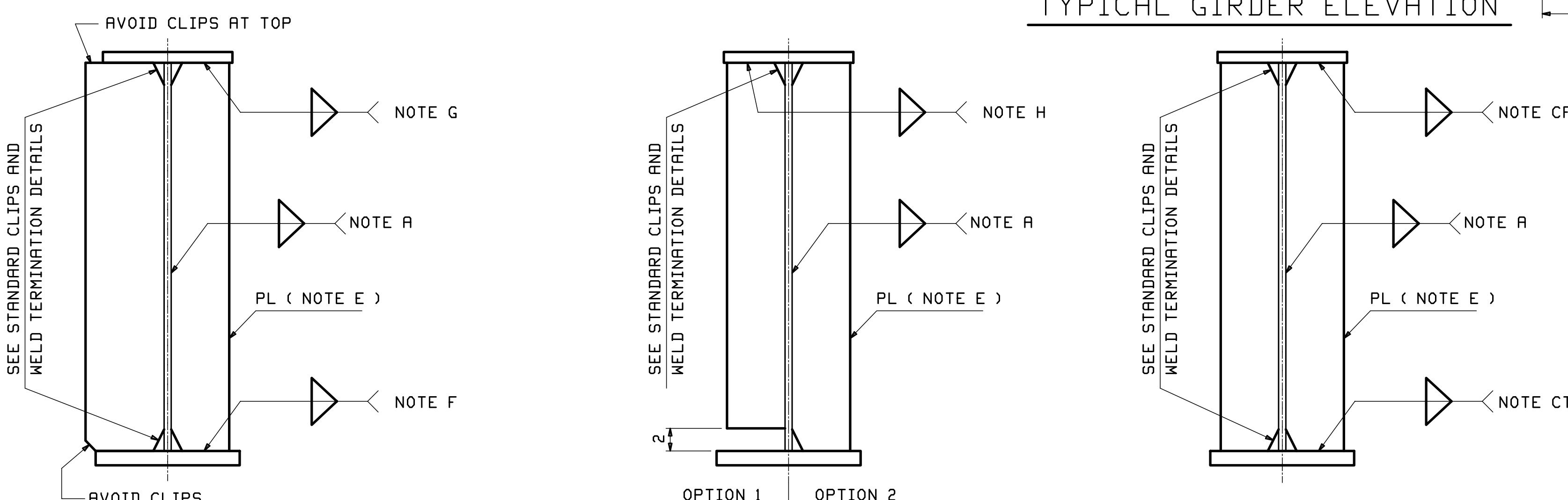
AASHTO/NSBA STEEL BRIDGE COLLABORATION

TASK GROUP 1, SUBTASK - GROUP 1.4
GUIDELINES FOR DESIGN DETAILS

PAGE NO. 102



TYPICAL GIRDER ELEVATION



BEARING STIFFENER

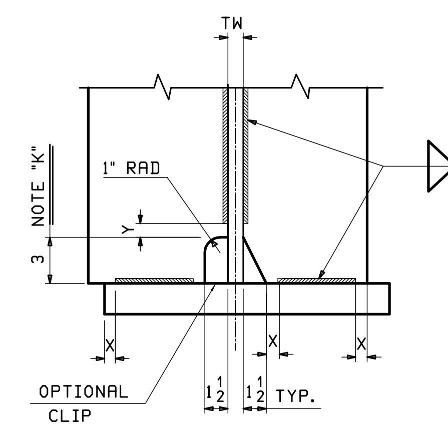
BEARING STIFFENER TO FLANGE WELDING IS REQUIRED IF A DIAPHRAGM OR CROSSFRAME IS ATTACHED TO THE STIFFENER.

INTERMEDIATE STIFFENER

REQUIRED ON ONE SIDE AS A WEB STIFFENER ONLY (NOT A CONN PL.) EITHER WELD TO COMPRESION AND GAP AT TENSION FLANGE OR TIGHT FIT TO BOTH.

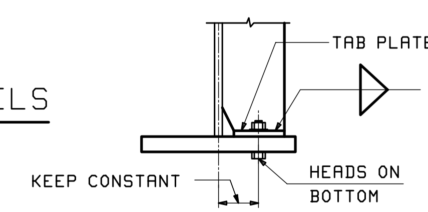
CONNECTION STIFFENER

TYPICAL STIFFENER DETAILS



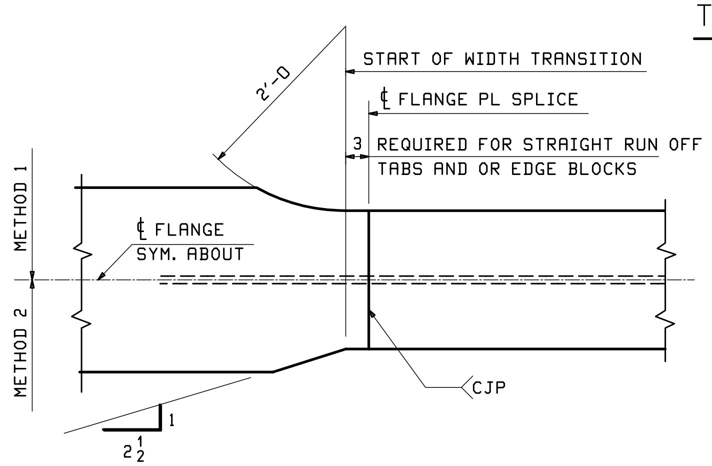
STANDARD CLIP & WELD TERMINATION DETAIL

X=	1/4" +/- 1/8"
Y=	1/2" +/- 1/4"



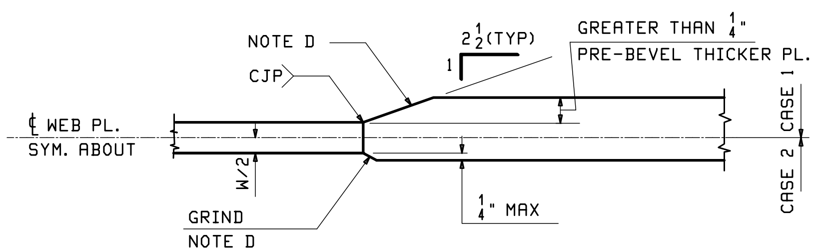
TAB PLATE DETAIL

NOTE CT AND J



FLANGE PLATE WIDTH TRANSITION

AVOID FLANGE WIDTH TRANSITIONS (NOTE C)



WEB PLATE THICKNESS TRANSITION

DISCLAIMER NOTE

INFORMATION SHOWN IS FOR CONCEPT ONLY. APPLICATION TO SPECIFIC STRUCTURES IS THE DESIGNERS RESPONSIBILITY.

NOTES FOR DESIGN DRAWINGS:

1 ... ENGINEER SHOULD SPECIFY THAT UNDER FULL DEAD LOAD, GIRDER ENDS AND ALL BEARING STIFFENERS, INCLUDING BEARING STIFFENERS AT PIERS, ARE VERTICAL OR NORMAL TO GRADE LONGITUDINALLY.

2 ... INTERMEDIATE STIFFENERS AND CONNECTION STIFFENERS SHOULD BE NORMAL TO THE FLANGE UNLESS OTHERWISE REQUIRED BY DESIGNER OR REQUESTED BY THE FABRICATOR, SUCH AS A CONNECTION STIFFENER IN AN AREA OF COMPLEX FRAMING. PLACE INTERMEDIATE STIFFENERS ON THE INTERIOR SIDE OF FASCIA GIRDERS.

NOTES TO DESIGNERS:

A ... FILLET WELD SIZES SHALL BE SHOWN ON THE DESIGN DRAWINGS OR A NOTE STATING THAT MINIMUM AASHTO/AWS WELDS SHALL BE USED. WHERE EVER POSSIBLE AVOID FILLET WELDS OVER 5/16. AVOID CJP WELDS, FOR OTHER THAN WEB AND FLANGE SPLICES.

B ... THICKER PLATE PREFERABLY SHOULD NOT BE MORE THAN 2X THE THICKNESS OF THINNER PLATE AT CJP WELDS.

C ... AVOID SHOP SPLICE FLANGE WIDTH TRANSITIONS WITHIN A GIRDER SHIPPING UNIT, CHANGES IN THE THICKNESS OF FLANGE PLATES ARE PREFERRED.

D ... WHERE DIFFERENCES IN WEB PLATE THICKNESS ARE LESS THAN 1/4" TRANSITIONS MAYBE ACCOMPLISHED BY GRINDING AFTER WELDING.

E ... DESCRIBE STIFFENER PLATE SIZES HERE OR ON GIRDER ELEVATIONS. USE 7 1/2" MINIMUM WIDTH FOR CONNECTION STIFFENERS WITH 2 VERTICAL ROWS OF BOLTS. USE 5" MINIMUM WIDTH CONNECTION PL'S FOR ONE ROW OF BOLTS.

F ... FINISH TO BEAR (TYP) AND FILLET WELD IF A DIAPHRAGM OR CROSSFRAME ATTACHES TO STIFFENER. INSURE CLEARANCE TO BRG. ATTACHMENT BOLTS. INVESTIGATE LOAD FOR REQUIREMENT OF CJP WELDS.

G ... USE FILLET WELD IN LIEU OF TIGHT FIT WHEN A DIAPHRAGM OR CROSSFRAME ATTACHES TO STIFFENER. TIGHT FIT ONLY WITHOUT A DIAPHRAGM OR CROSSFRAME ATTACHED.

H ... USE FILLET WELD TO COMPRESSION FLANGE AND GAP AT TENSION FLANGE OR TIGHT FIT TO BOTH FLANGES.

J ... SHOW LIMITS WITH DIMENSIONS OF TENSION ZONES AND STRESS REVERSAL ZONES. THESE ARE NEEDED FOR WELD TESTING AND FIT OF STIFFENERS.

K ... KEEP THE SIZE OF CLIPS THE SAME FOR ANY ONE PROJECT. 3" SHOULD WORK FOR ALMOST ALL STRUCTURES.

W ... DO NOT SHOW A CJP DETAIL FOR A WEB SPLICE UNLESS THE THICKNESS CHANGES. SPECIFY WHERE GRINDING IS REQUIRED.

X ... THESE DETAILS & MATERIAL SIZES ARE SUGGESTED AND ARE FOR A GUIDE ONLY. ENGINEER SHOULD CHECK WITH OWNER FOR POSSIBLE PREFERRED STANDARDS.

CF .. FILLET WELD TO COMPRESSION FLANGE.

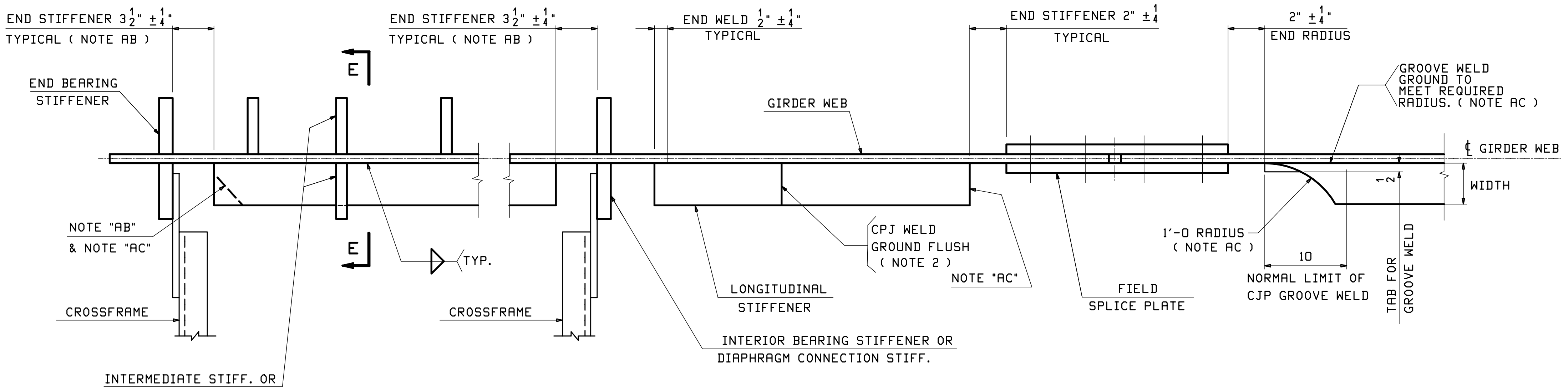
CT .. FILLET WELDING TO TENSION FLANGE IS PREFERRED. DESIGNER MUST CHECK STRESS RANGE FOR CATEGORY C . INVESTIGATE THICKENING FLANGE PLATE IN LIEU OF USING A BOLTED TAB PLATE. THEN USE TAB PLATES ONLY AT LOCATIONS WHERE REQUIRED. TAB PLATES COST ABOUT 150 DOLLARS EACH BASED ON 2003 COSTS.

TYPICAL GIRDER DETAILS - I

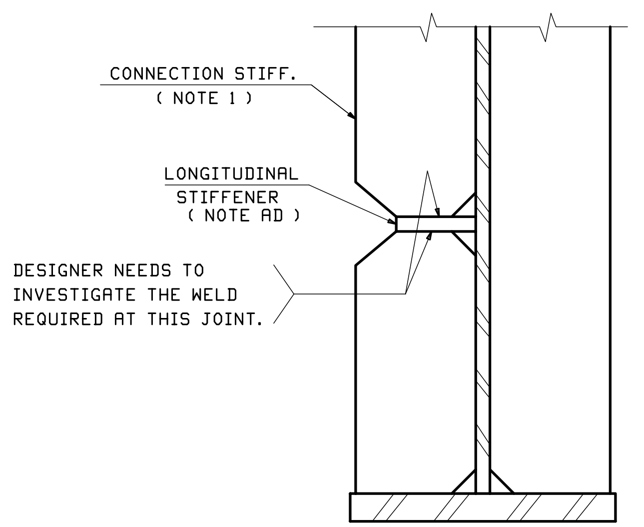
AASHTO/NSBA STEEL BRIDGE COLLABORATION

TASK GROUP 1, SUBTASK - GROUP 1.4
GUIDELINES FOR DESIGN DETAILS

PAGE NO. 103

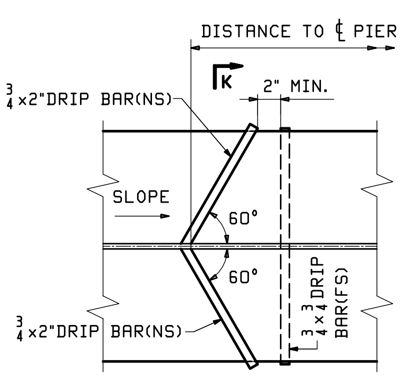


LONGITUDINAL - TRANSVERSE STIFFENER INTERSECTION DETAIL

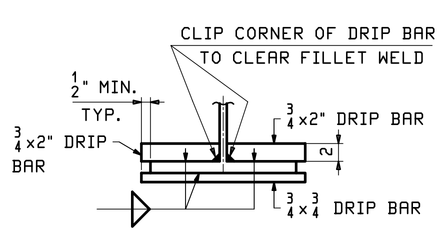


SECTION E-E

(IN TENSION OR REVERSAL ZONE ONLY)
(AT INT. STIFF. OR CONN. PL.)



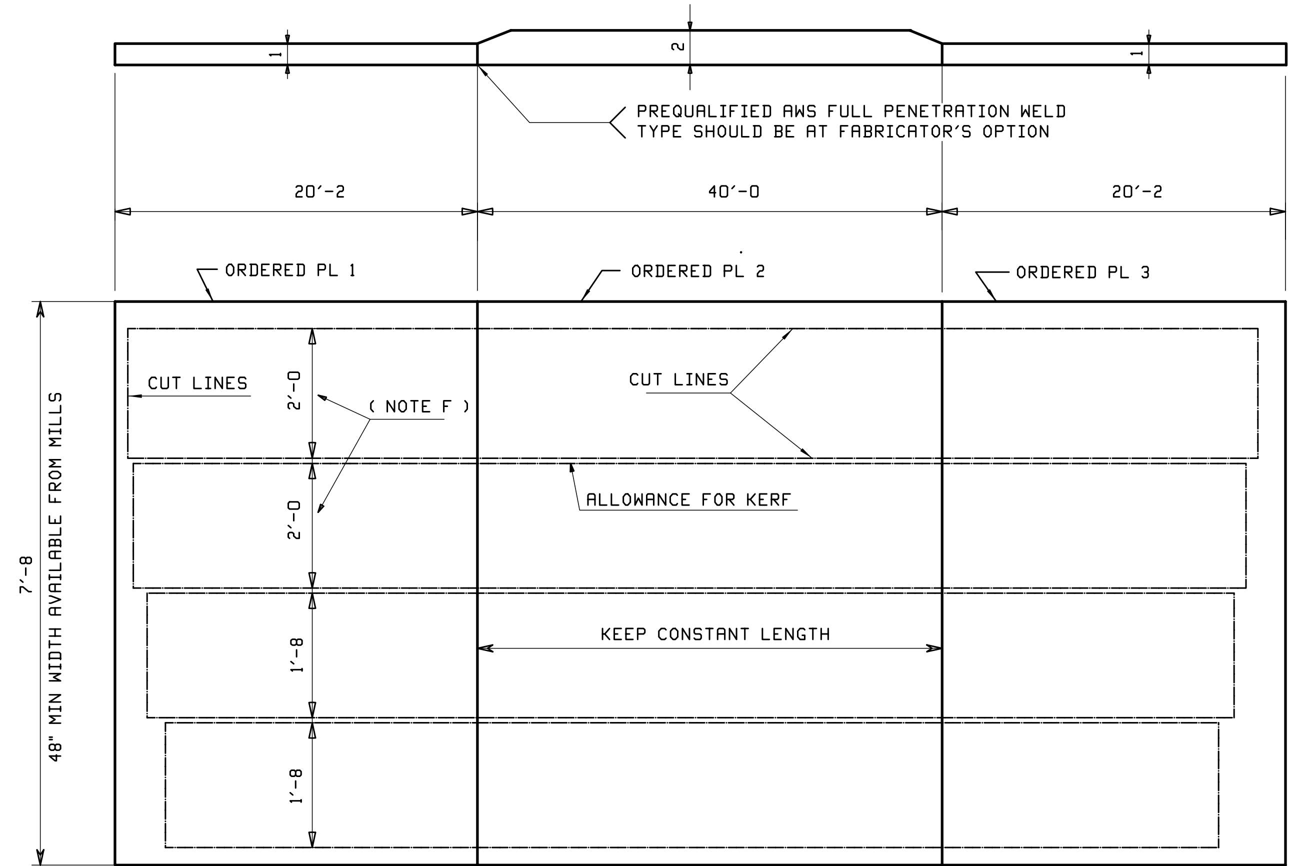
DRIP BAR DETAIL



SECTION K

NOTE:
DRIP BAR ON TOP OF BOTTOM FLANGE SHALL BE CAULKED AGAINST FLANGE, WEB & FILLET WELD SUBJECT TO APPROVAL OF THE ENGINEER.

FOR INFORMATION ONLY - NOT PART OF DESIGN PLANS



ALLOWANCE FOR SQUARING AND CUTTING BOTH SIDES TO AVOID INDUCING SWEEP.

SLABBING AND STRIPPING

LONGITUDINAL & TRANSVERSE STIFFENER

NOTES FOR DESIGN DRAWINGS:

1...LOCATE ALL INTERMEDIATE STIFFENERS ON OPPOSITE SIDE OF WEB FROM LONGITUDINAL STIFFENERS WHEN POSSIBLE. LOCATE LONGITUDINAL STIFFENER ON EXTERIOR FACE OF FASCIA GIRDERS.

2...PERFORM NON-DESTRUCTIVE TESTING ON LONGITUDINAL STIFFENER BUTT WELDS PRIOR TO ATTACHMENT TO GIRDER WEB. WELD NOT DETAILED ON DESIGN PLANS.

3...SEE SECTION E-E FOR STRESS REVERSAL ZONE AT STIFFENERS.

NOTES TO DESIGNERS:

AA...AVOID THE USE OF LONGITUDINAL STIFFENERS WHEREVER THICKENING THE WEB PLATE MIGHT BE MORE ECONOMICAL.

AB...CROSSFRAME MAY INTERFERE WITH LONGITUDINAL STIFFENER DURING ERECTION, IF SO CLIP STIFFENER AS REQUIRED.

AC...USE GROOVE WELD TERMINATION AND RADIUS CUT ONLY WHEN REQUIRED BY DESIGN.

AD...AVOID INTERSECTING TRANSVERSE AND LONGITUDINAL STIFFENERS BY LIMITING LENGTH OF THE LONGITUDINAL STIFFENER OR THICKENING THE WEB PLATE..

DISCLAIMER NOTE
INFORMATION SHOWN IS FOR CONCEPT ONLY. APPLICATION TO SPECIFIC STRUCTURES IS THE DESIGNER'S RESPONSIBILITY.

SLABBING & STRIPPING DETAIL

NOTES TO DESIGNERS

A...STEEL MILLS NO LONGER ROLL PLATES IN WIDTHS LESS THAN 48 INCHES. THE FABRICATOR'S ARE REQUIRED TO COMBINE PLATES AND NEST THEM IN ORDER TO ECONOMIZE AND REDUCE SCRAP.

B...THE ENDS OF ORDERED PLATES ARE PREPARED AND THEN WELDED TOGETHER AS SHOWN. THE INDIVIDUAL FLANGE PLATE ASSEMBLIES ARE THEN FLAME CUT TO THEIR FINISHED WIDTHS BY MULTIPLE TORCHES. NON-DESTRUCTIVE TESTING MAYBE PERFORMED PRIOR TO OR AFTER STRIPPING BUT BEFORE WELDING THE FLANGE PLATE ASSEMBLIES TO THE WEB PLATES.

C...THE COST OF SPLICING THE SLAB WHICH INCLUDES HANDLING, BEVELING, GRINDING AND TESTING, WOULD BE SUBSTANTIALLY LESS THAN FABRICATING EACH FLANGE PLATE ASSEMBLY INDIVIDUALLY.

D...AVOID TRANSITIONS IN FLANGE WIDTH IN ANY ONE GIRDER SHIPPING LENGTH (VARY THICKNESS INSTEAD) THIS WILL ELIMINATE EXTENSION TABS AND RUN-OFF BARS.

E...MINIMIZE CHANGES IN FLANGE PLATE THICKNESS. IT MAY BE MORE ECONOMICAL TO ELIMINATE A FLANGE SPLICE AND EXTEND THE THICKER FLANGE. THE COST OF A SPLICE MAY EXCEED THE MATERIAL COSTS.

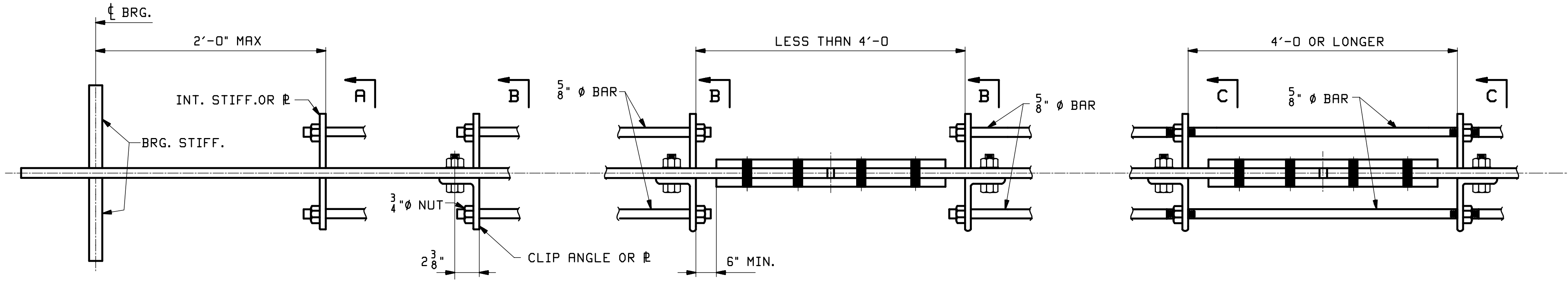
F...IF DIFFERENT FLANGE AREAS ARE NEEDED FOR ADJACENT CURVED GIRDERS, VARY WIDTHS INSTEAD IF THICKNESS.

TYPICAL GIRDER DETAILS - II & FLANGE SLABBING AND STIPPING DETAILS

AASHTO/NSBA STEEL BRIDGE COLLABORATION

TASK GROUP 1, SUBTASK - GROUP 1.4
GUIDELINES FOR DESIGN DETAILS

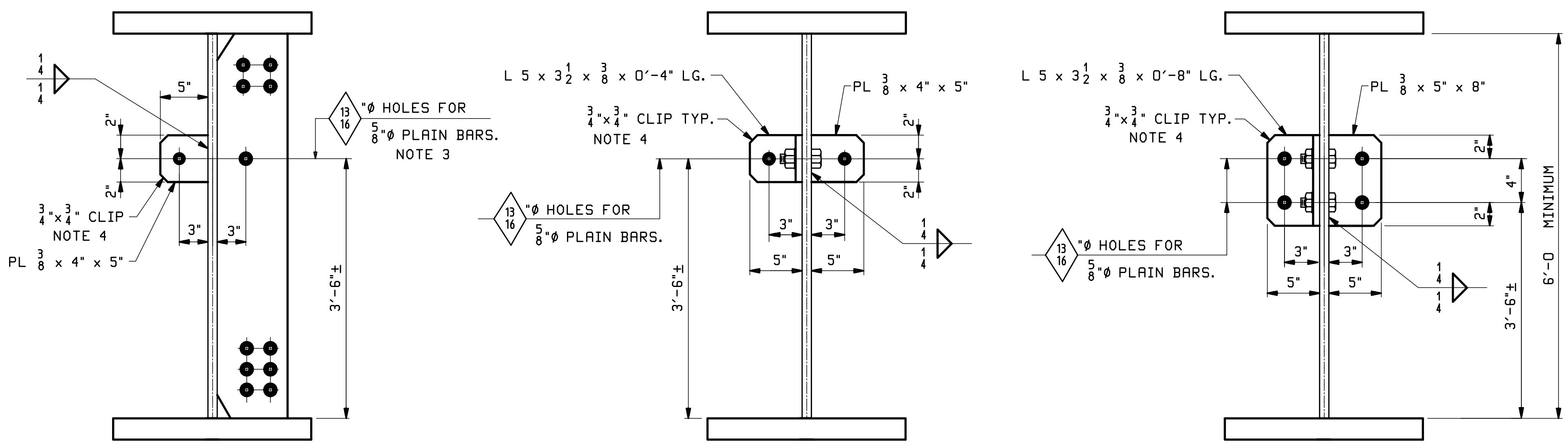
PAGE NO. 104



AT GIRDER END

AT GIRDER FIELD SPLICE

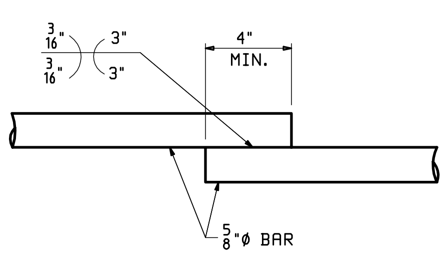
SAFETY HANDRAIL - END CONNECTIONS



SECTION-A

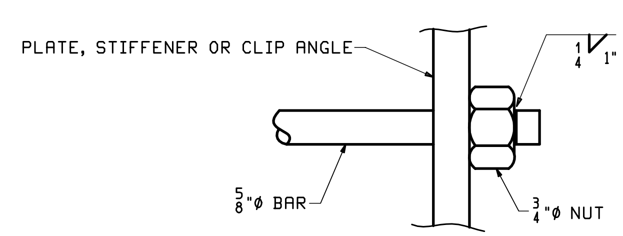
SECTION-B

SECTION - C



SHOP OR FIELD WELDED SPLICE DETAIL

NOTE 1



END CONNECTION DETAIL

DISCLAIMER NOTE

INFORMATION SHOWN IS FOR CONCEPT ONLY. APPLICATION TO SPECIFIC STRUCTURES IS THE DESIGNERS RESPONSIBILITY.

NOTES FOR DESIGN DRAWINGS:

1...BARS TO BE MADE CONTINUOUS THROUGH USE OF WELDED SPLICES.

2...DISTANCE BETWEEN HANDRAIL SUPPORTS NOT TO EXCEED 8'-6 MAXIMUM SPACING.

3...HOLE FOR HANDRAIL IN CONNECTION STIFFENERS MAY BE THE SIZE AS OTHER HOLES IN THE STIFFENER. (MIN. $\frac{13}{16}$"ø)

4...GRIND CORNERS AT CLIPS SMOOTH.

NOTES TO DESIGNERS

A...AVOID HANDRAILING FOR GIRDERS UNDER 6'-0 DEEP.

B...HANDRAILS ARE ONLY REQUIRED WHEN SPECIFIED BY THE OWNER.

C...CONNECTION PLATES ARE PREFERRED IN LIEU OF CLIP ANGLES. ANGLES REQUIRE LAYOUT AND DRILLING OF THE WEB PLATE AND CONTACT SURFACE PAINTING.

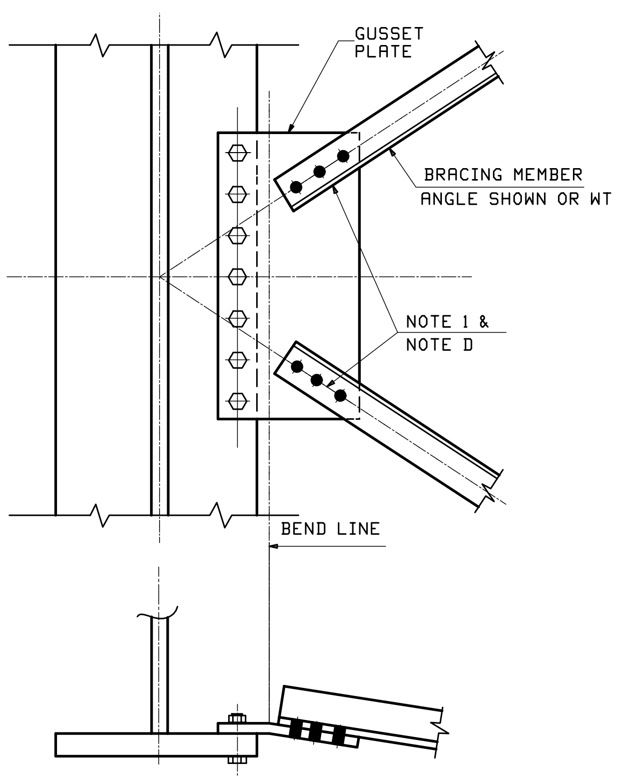
D...THESE DETAILS & MATERIAL SIZES ARE SUGGESTED AND ARE FOR A GUIDE ONLY. ENGINEER SHOULD CHECK WITH OWNER FOR POSSIBLE PREFERRED STANDARDS.

TYPICAL GIRDER DETAILS - III

AASHTO/NSBA STEEL BRIDGE COLLABORATION

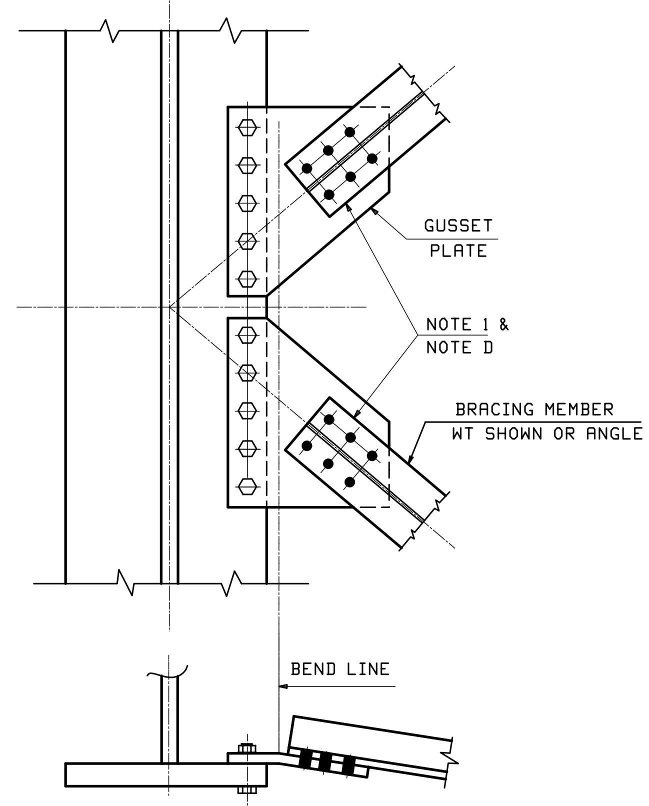
TASK GROUP 1, SUBTASK - GROUP 1.4
GUIDELINES FOR DESIGN DETAILS

PAGE NO. 105



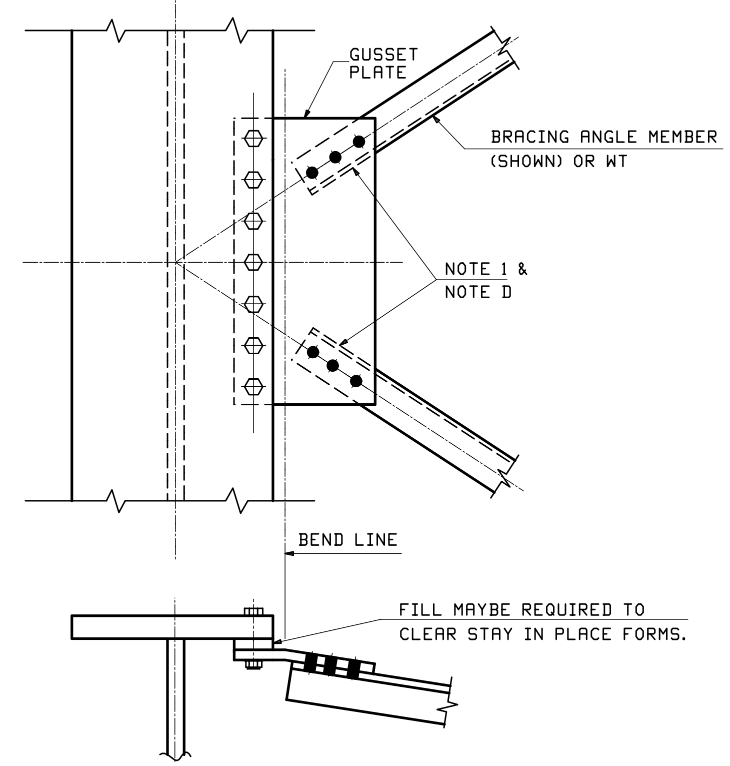
LATERAL BRACING CONN.

(AT BOTTOM FLANGE)



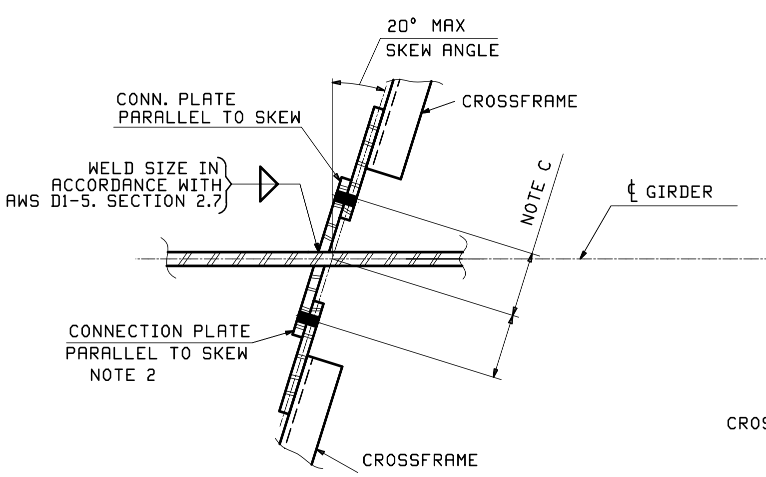
LATERAL BRACING CONN.

(AT BOTTOM FLANGE)
ALTERNATE DETAIL WHEN
BRACING SLOPE CHANGES



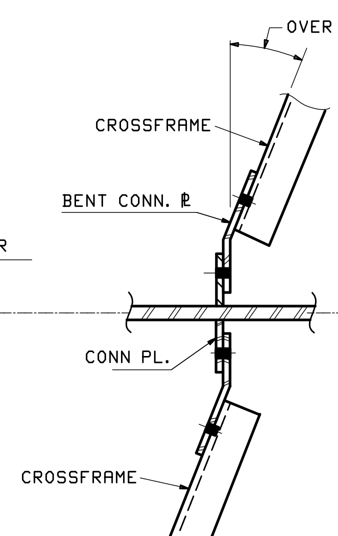
LATERAL BRACING CONN.

(AT TOP FLANGE)

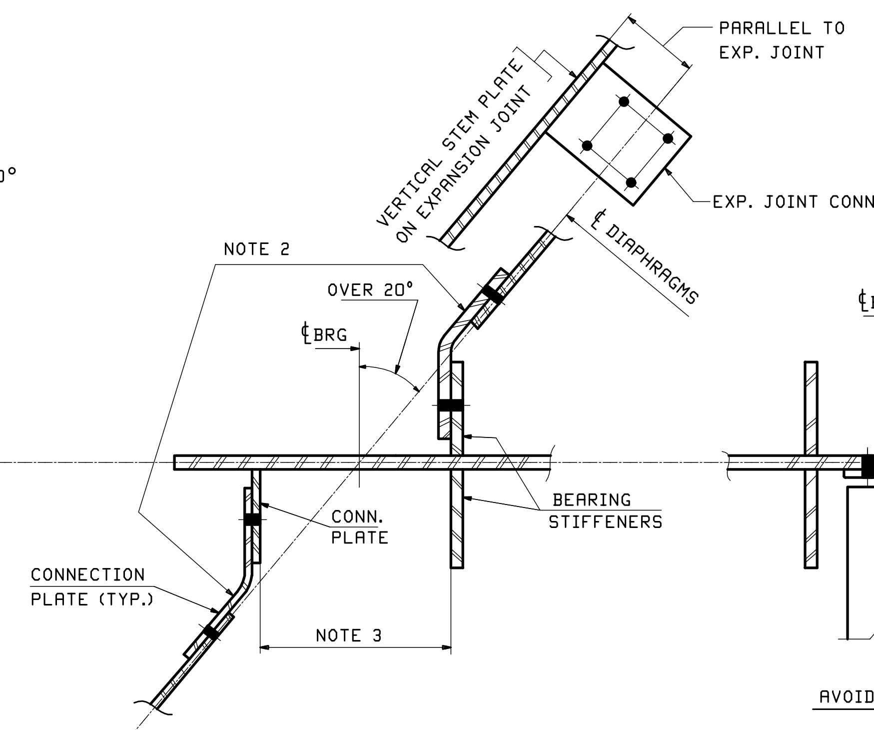


SKEWED CONN. PLATE

NOTE B

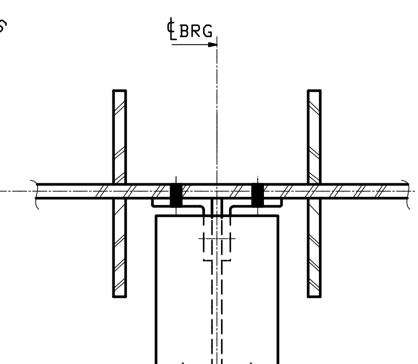


BENT GUSSET PLATE
AT INTERMEDIATE CF



ALTERNATE BENT CONN. PLATE

CONSIDER THIS DETAIL IF DIAPHRAGM/CF MUST
ALIGN TO SUPPORT EXPANSION JOINT SUPPORTS.



AVOID THIS DETAIL

NOTES FOR DESIGN DRAWINGS:

1...USE OVERSIZED HOLES IN BRACING MEMBERS, AND STANDARD SIZE HOLES IN GUSSET PLATES.

2...ORIENT CONNECTION PLATES TO FACILITATE ERECTION AS REQUIRED AT ABUTMENTS.

3...CLEAR SPACING BETWEEN STIFFENERS SHALL NOT BE LESS THAN 8" FOR MANUAL AND SEMI-AUTOMATIC WELDS, AND CLEAR SPACING BETWEEN STIFFENERS SHALL NOT BE LESS THAN THE FOLLOWING FOR AUTOMATIC WELDS:
- 10" OR 1 1/2 TIMES THE MAXIMUM STIFFENER WIDTH, WHICHEVER IS LARGER.

NOTES TO DESIGNERS:

A...AVOID LATERAL BRACING EXCEPT WHERE ABSOLUTELY NECESSARY, THEN USE IN EXTERIOR BAYS. BRACING MAY NOT BE REQUIRED IN ALL BAYS.

B...ON SMALL SKEWS IT IS MORE ECONOMICAL TO USE A SKEWED CONNECTION PLATE THAN A SQUARE STIFFENER AND BENT CONNECTION PLATE.
FOR SKEWS GREATER THAN 20° A BENT CONNECTION PLATE IS MORE EFFICIENT SINCE AUTOMATIC WELDING EQUIPMENT CANNOT BE USED TO WELD STIFFENER TO WEB.

C...MAKE SURE GAGE IN CONNECTION PLATE IS LARGE ENOUGH TO INSTALL AND TORQUE BOLTS.

D...FIELD WELDED OPTIONS MAYBE PERMITTED IN LIEU OF BOLTING.

DISCLAIMER NOTE

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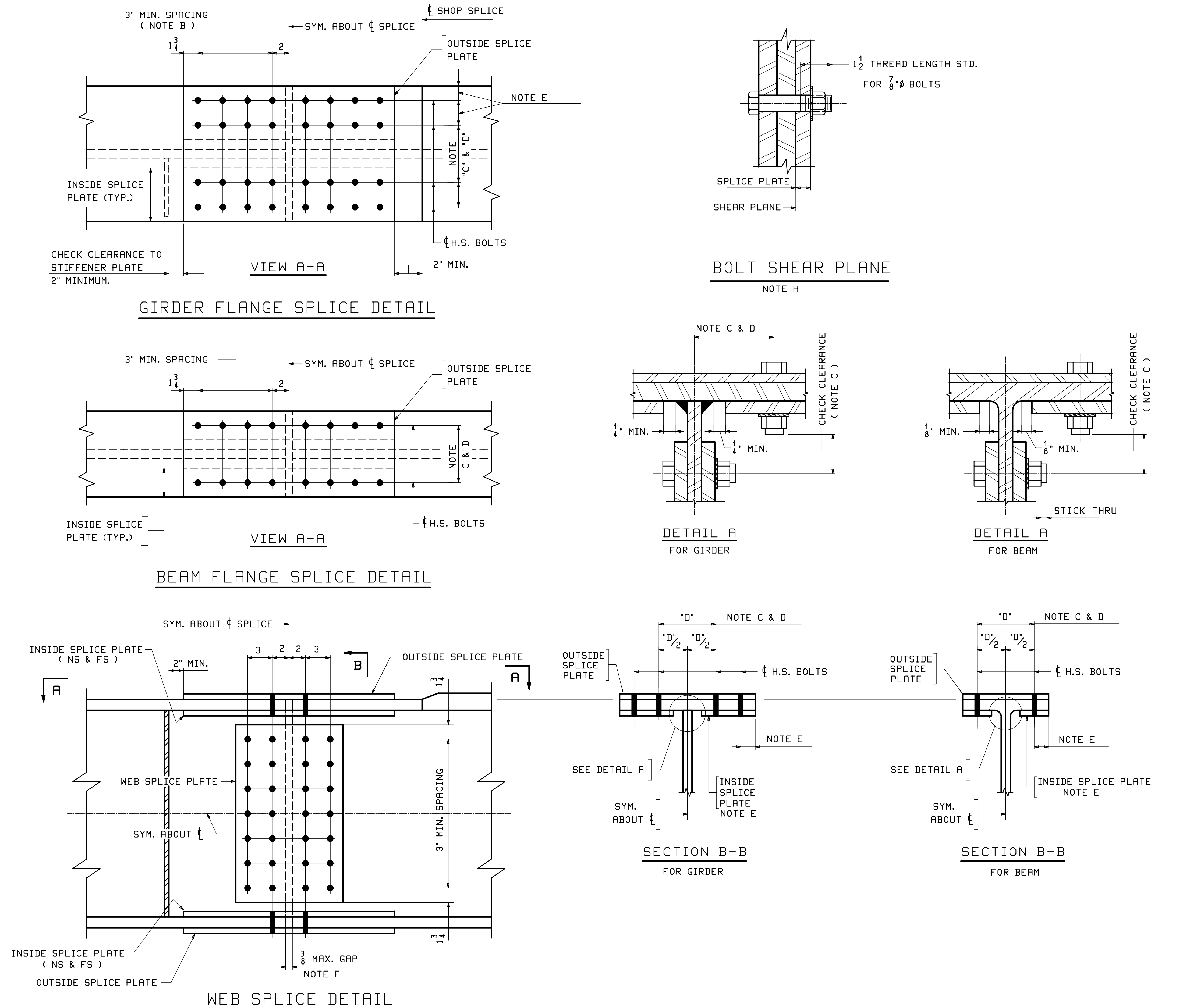
TYPICAL GIRDER DETAILS - IV

AASHTO/NSBA STEEL BRIDGE COLLABORATION

TASK GROUP 1, SUBTASK - GROUP 1.4
GUIDELINES FOR DESIGN DETAILS

PAGE NO. 106



NOTES FOR DESIGN DRAWINGS:

1...DETAILS SHOWN FOR $\frac{7}{8}$" DIAMETER HIGH STRENGTH BOLT. DIMENSIONS WILL VARY FOR OTHER BOLT DIAMETERS.

2...BOLT SPACING SHOWN ARE PREFERRED MINIMUMS.

3...EDGE DISTANCES SHOWN ARE MINIMUMS BASED ON SHEARED OR GAS CUT EDGES PLUS AN ADDITIONAL $\frac{1}{4}$" MATERIAL. THIS WILL PROVIDE A TOLERANCE FOR PUNCHING, DRILLING AND REAMING.

4...ALL SPLICE PLATES SHALL BE DETAILED WITH DIRECTION OF ROLLING PARALLEL TO STRESS DIRECTION.

5...ALL SPLICE PLATES ARE SUBJECT TO CHARPY V-NOTCH REQUIREMENTS.

NOTES TO DESIGNERS:

A...AVOID USING A490 BOLTS.

B...AVOID STAGGERED HOLE PATTERNS.

C...VERIFY INSTALLATION CLEARANCE. REFER TO THE AISC MANUAL OF STEEL CONSTRUCTION, "BOLT CLEARANCE TABLES"

D...VERIFY THAT BOLT SPACING FOR FLANGE SPLICES AND WEB SPLICES DO NOT EXCEED BOLT SEALING REQUIREMENTS.

E...MAKE INSIDE SPLICE PLATES SYMMETRICAL ($1\frac{3}{4}$" PREFERRED EDGE FOR $\frac{7}{8}$ø BOLTS). DO NOT SPECIFY MINIMUM EDGE DISTANCES. REFER TO LRFD FOR MINIMUM EDGE DISTANCE CRITERIA.

F...NORMALLY MAXIMUM GAP BETWEEN END OF MEMBERS IS TO BE $\frac{3}{8}$". FOR DEEP GIRDERS AT THE FABRICATOR'S REQUEST, THE GAP MAY BE GREATER AS LONG AS MINIMUM EDGE DISTANCE IS MAINTAINED.

G...FILL MAY BE REQUIRED AT FLANGE AND WEB SPLICE. USE $\frac{1}{8}$" MINIMUM THICKNESS, CVN NOT REQUIRED FOR FILLS. FILLS CAN BE GR. 36

DISCLAIMER NOTE

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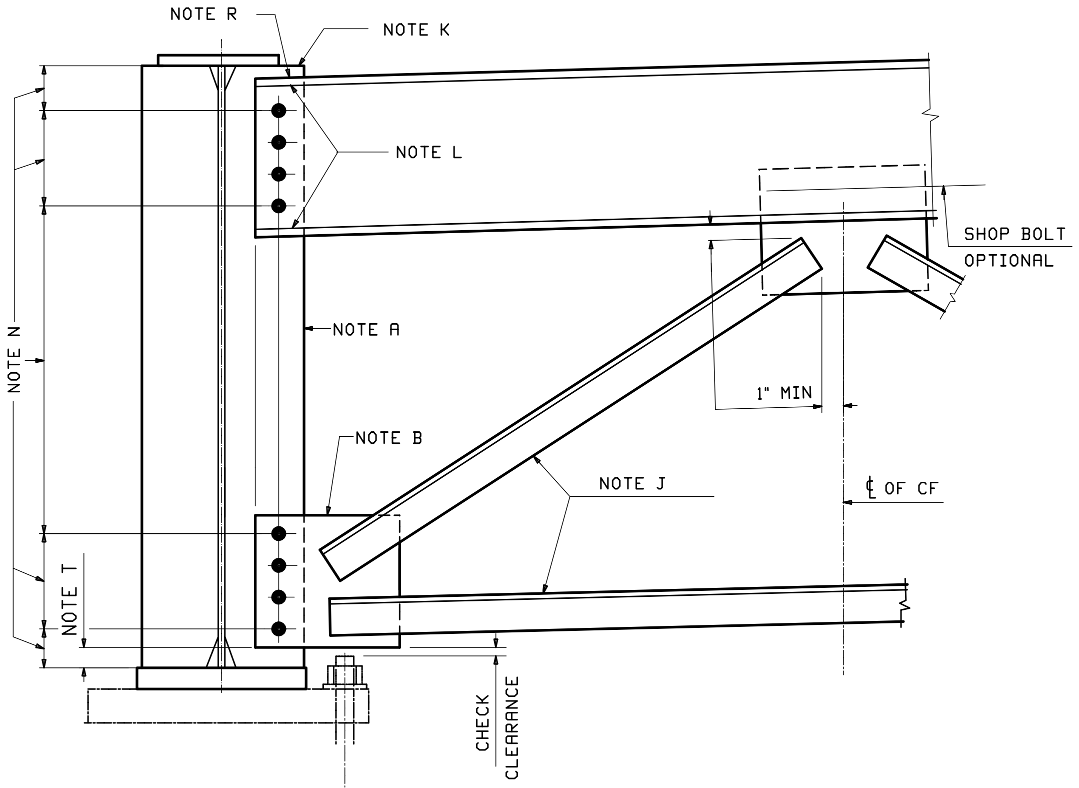
STANDARD BOLTED FIELD SPLICES

AASHTO/NSBA STEEL BRIDGE COLLABORATION

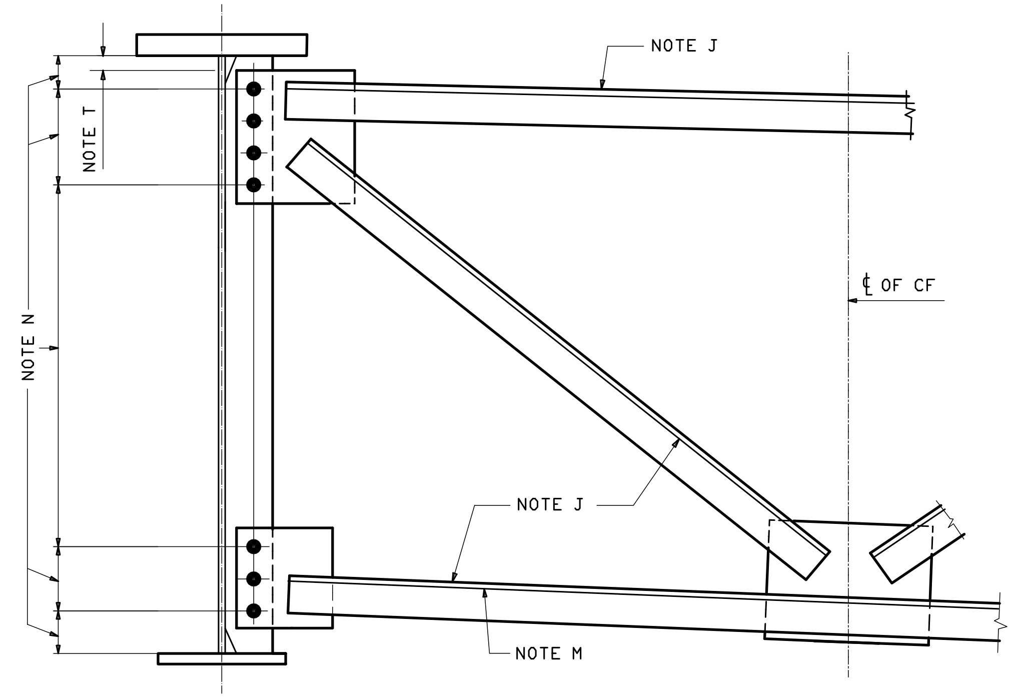
TASK GROUP 1, SUBTASK - GROUP 1.4
GUIDELINES FOR DESIGN DETAILS

PAGE NO. 107



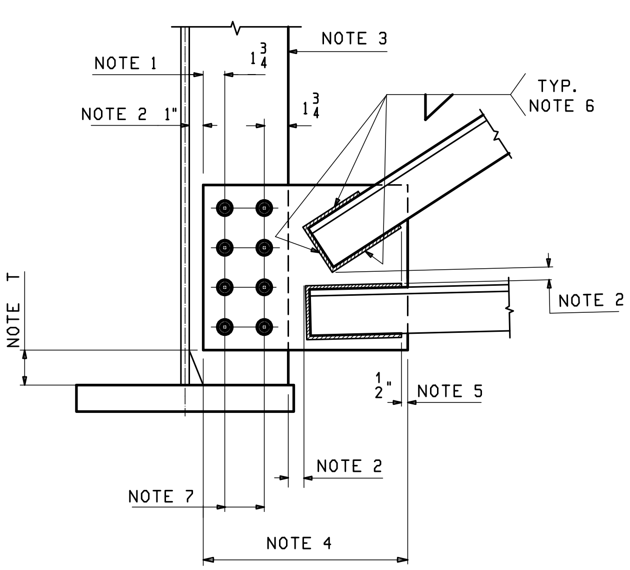
PREFERRED BEARING TYPE CROSSFRAME

FOR BEARING DIAPHRAGMS AT DECK JOINTS ONLY.
SEE NOTES A THRU T



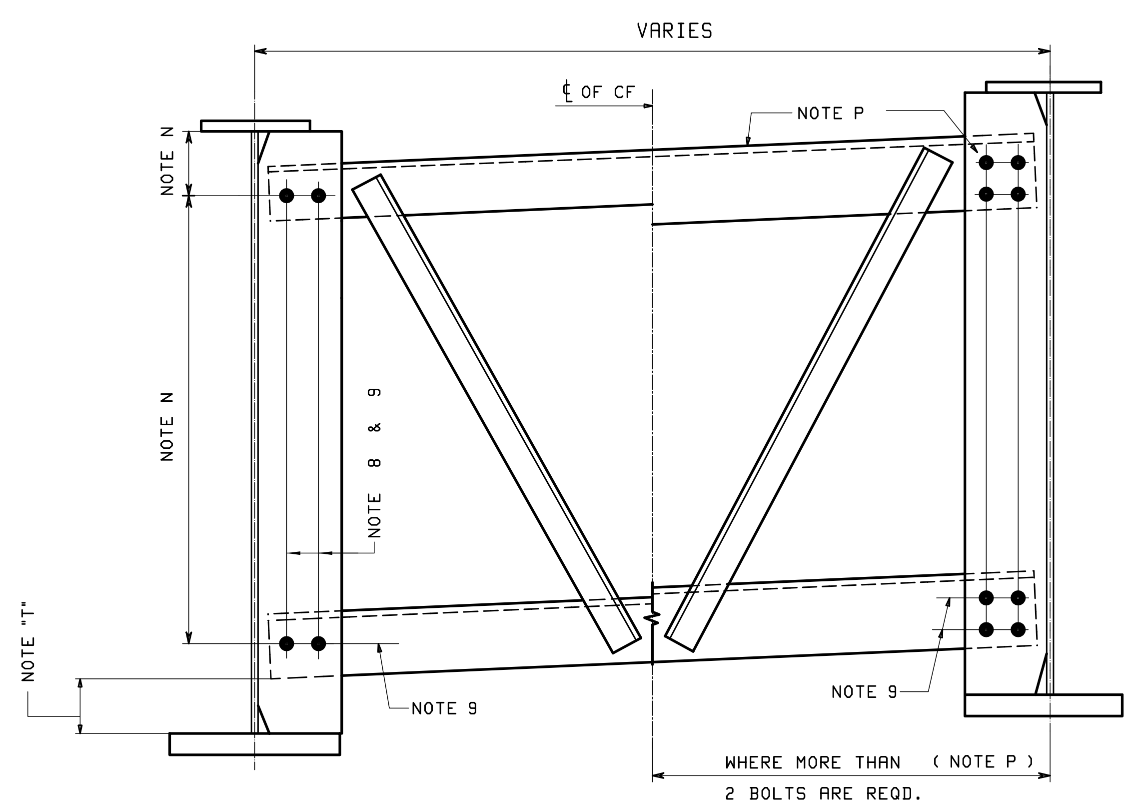
PREFERRED INTERMEDIATE "K" TYPE CROSSFRAME

SEE NOTES A THRU T



PREFERRED CONNECTION PLATE DETAILS

SEE NOTES 1 THRU 8



ALTERNATE INTERMEDIATE "K" TYPE CROSSFRAME - STRAIGHT BRIDGES

SEE NOTES A THRU T

ONE PIECE INTERMEDIATE CROSSFRAMES "K" TYPE ADVANTAGES OVER KNOCKED DOWN FRAMES

(SEE PAGE NO. 109 FOR EXAMPLE OF KNOCKED DOWN FRAME)

A... ALL STIFFENERS WOULD HAVE THE SAME LAYOUT AND THE SAME PIECE MARK

B... NO LAYOUTS ARE REQUIRED SINCE CONNECTION PLATES ARE RECTANGULAR

C... DIAGONAL MEMBERS CAN CONNECT TO EITHER THE TOP STRUT FOR BEARING TYPE OR BOTTOM STRUT FOR INTERMEDIATE TYPE.

D... ALL ANGLES CAN BE CUT WITHOUT ANY LAYOUTS

E... ERECTION IS QUICKER, DUE TO FEWER ERECTION PIECES

F... SINCE FRAMES ARE JIGGED, IN FABRICATION, CHANCE OF FIELD MISFITS ARE MINIMIZED

G... ALL PLATES CAN BE STACK DRILLED OR MULTIPLE PUNCHED , SINCE THE HOLE PATTERNS ARE IDENTICAL.

H... CHANGES IN THE GEOMETRY OF THE FRAME CAN EASILY BE ACCOMMODATED BY MOVING ONE SIDE OF THE JIG FOR DIFFERENCES IN ELEVATIONS.

I... ALL WELDING CAN BE DONE FROM THE NEAR SIDE, THEREFORE THE ASSEMBLY DOES NOT HAVE TO BE TURNED OVER.

J... USE SINGLE MEMBERS WHERE EVER POSSIBLE. TYPICAL ALL CROSSFRAMES. AVOID DOUBLE MEMBERS (ANGLES, CHANNELS, ETC.) DOUBLE MEMBERS CANNOT BE PAINTED PROPERLY.

K... SHOW CLIPPING OF STIFFENERS AS "OPTIONAL".

L... CHANNELS ARE PREFERRED IN LIEU OF "WF" SHAPES SINCE BEAM FLANGES WOULD HAVE TO BE CUT AND GROUND FLUSH ON FAR SIDE, AND MAY COLLECT DEBRIS AT EXPANSION JOINTS.

M... ORIENT MEMBERS TO MINIMIZE COLLECTING DEBRIS AND MOISTURE.

N... KEEP THESE DIMENSIONS THE SAME AND SLOPE THE CROSSFRAME MEMBERS.

P... INCREASE SIZE OF MEMBERS AS REQUIRED TO ACCOMODATE DIFFERENT BOLT REQUIREMENTS.

R... LOWER CHANNEL TO AVOID COPING.

T... USE CONSTANT DIMENSION AT ALL GIRDERS, USUALLY APPROX. 2" TOP & BOTTOM

NOTES TO DESIGNERS:

1... PROVIDE FOR PREFERRED EDGE DISTANCES EG: 1 3/4 FOR 7/8 DIA BOLTS. AVOID LRFD MINIMUMS.

2... PROVIDE ABOUT 1" MINIMUM CLEARANCE FROM EDGE OF FILLET WELD

3... INCLUDE ALLOWANCES FOR NOTES 1 & 2 IN DETERMINING STIFFENER WIDTH. A 7 1/2" MINIMUM WIDTH STIFFENER IS REQ'D FOR THE CONNECTION SHOWN.

4... KEEP GUSSET PLATES RECTANGULAR

5... TERMINATE WELDS $\frac{1}{2}$" (+/- $\frac{1}{4}$") SHORT OF EDGE

6... AVOID ALL AROUND WELDS , OMIT WELD ON FAR SIDE. KEEP ALL WELDING ON ONE SIDE.

7... PERMIT THE USE OF OVERSIZED HOLES FOR CROSSFRAME CONNECTIONS THIS COULD BE AN OPTION IF REQUESTED BY CONTRACTOR OR FABRICATOR. OVERSIZED HOLES SHOULD NOT BE USED ON CURVED STRUCTURES.

8... SHOWN FOR TWO ROWS OF BOLTS, NOTES FOR ONE ROW ARE SIMILAR.

9... FOR 2 LINES OF BOLTS, HOLES ARE HORIZONTALLY ALIGNED ON CONNECTION PLATE, SO WILL BE SKEWED ON CROSSFRAME MEMBERS. INSURE ADEQUATE SPACE/EDGE DISTANCE IS PROVIDED.

DISCLAIMER NOTE
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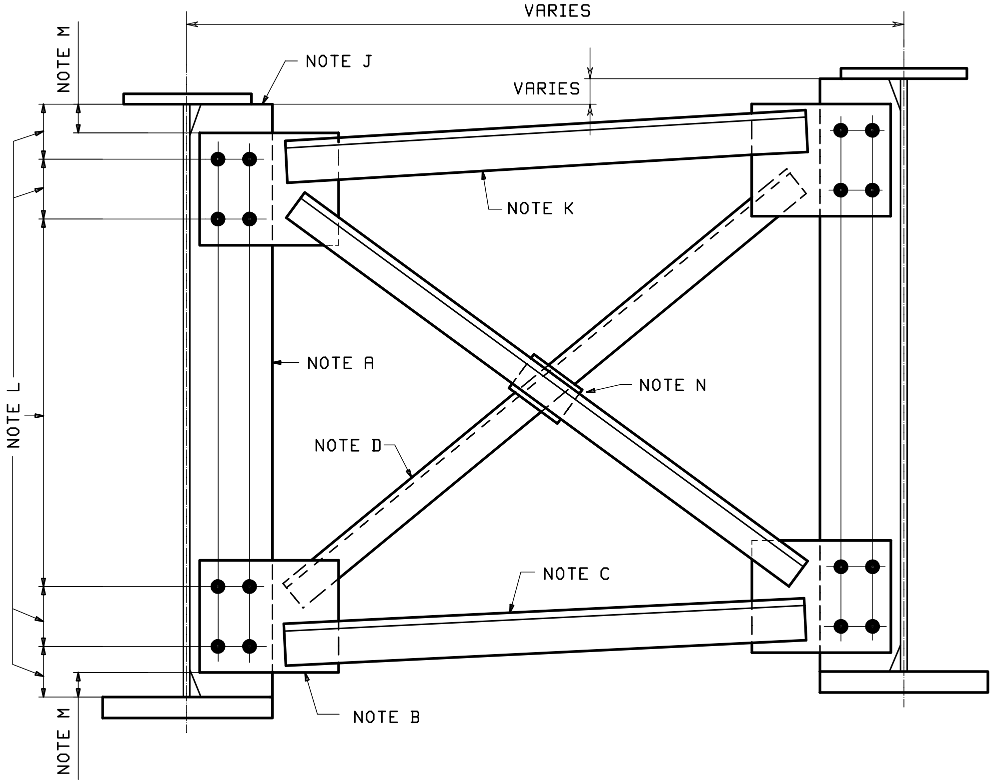
TYPICAL CROSSFRAME DETAILS "K" TYPE

AASHTO/NSBA STEEL BRIDGE COLLABORATION

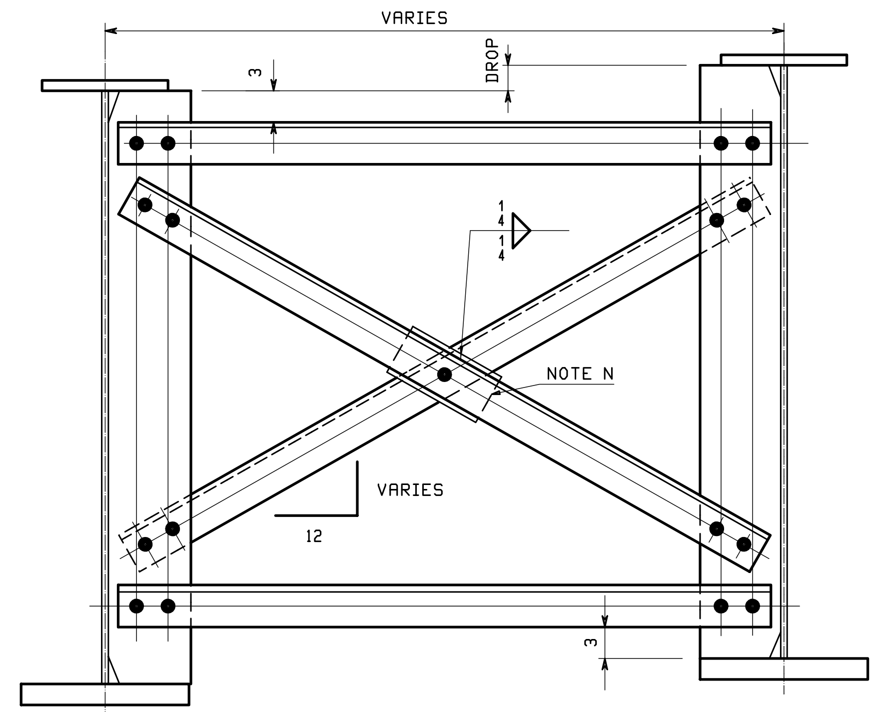
TASK GROUP 1, SUBTASK - GROUP 1.4
GUIDELINES FOR DESIGN DETAILS

PAGE NO. 108



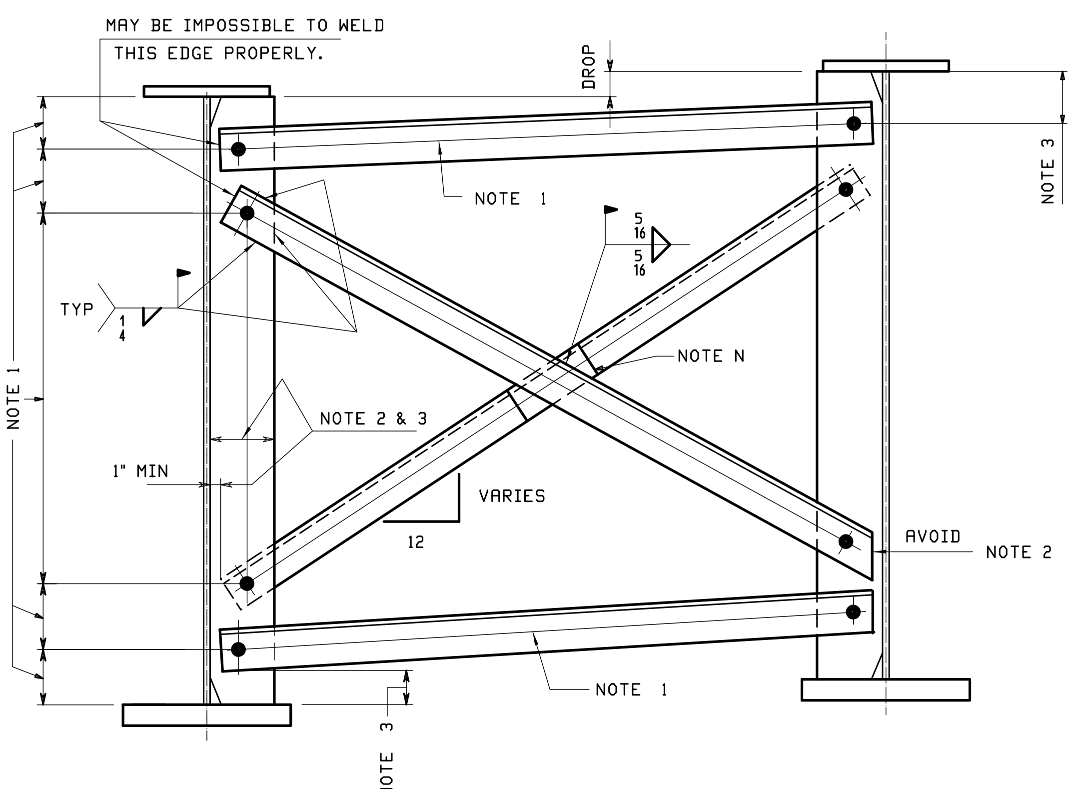
PREFERRED INTERMEDIATE "X" TYPE CROSSFRAME

SEE NOTES A THRU M



NON-PREFERRED "KNOCK-DOWN" TYPE CROSSFRAME

(4 SEPARATE SHIPPING PIECES)
USE THIS TYPE OF CROSSFRAME SPARINGLY.
AVOID THIS TYPE WHEN DROPS VARY.
SEE NOTES AA THRU AD



FIELD WELDED "KNOCK-DOWN" TYPE CROSSFRAME

IN STATES THAT PREFER FIELD WELDED CROSSFRAMES THIS METHOD
IS PREFERRED , PROVIDING NOTES 1 THRU 4 ARE INCORPORATED.

NOTES TO DESIGNERS:

PREFERRED INTERMEDIATE CROSSFRAMES "X" TYPE ADVANTAGES OVER KNOCKED DOWN FRAMES

(SEE PAGE NO. 108 FOR PREFERRED CONNECTION PLATE DETAIL)

A... ALL STIFFENERS WOULD HAVE THE SAME LAYOUT AND THE SAME MARK

B... NO LAYOUTS ARE REQUIRED SINCE CONNECTION PLATES ARE RECTANGULAR

C... ORIENT MEMBERS TO MINIMIZE COLLECTING DEBRIS AND MOISTURE.

D... ALL ANGLES CAN BE CUT WITHOUT ANY LAYOUTS

E... ERECTION IS MUCH FASTER, DUE TO FEWER ERECTION PIECES

F... SINCE FRAMES ARE JIGGED , CHANCE OF FIELD MISFITS ARE MINIMIZED

G... ALL PLATES CAN BE STACK DRILLED OR MULTIPLE PUNCHED , SINCE THE HOLE PATTERNS ARE IDENTICAL.

H... CHANGES IN THE GEOMETRY OF THE FRAME CAN EASILY BE ACCOMMODATED BY MOVING ONE SIDE OF THE JIG FOR DIFFERENCES IN ELEVATIONS.

I... USE SINGLE MEMBERS WHERE EVER POSSIBLE. TYPICAL ALL CROSSFRAMES. AVOID DOUBLE MEMBERS (ANGLES, CHANNELS, ETC.) DOUBLE MEMBERS CANNOT BE PAINTED PROPERLY.

J... SHOW CLIPPING OF STIFFENERS AS "OPTIONAL".

K... ELIMINATE TOP STRUT ON STRAIGHT GIRDER BRIDGES IF POSSIBLE. DESIGNER MUST CHECK STABILITY DURING ERECTION

L... KEEP THESE DIMENSIONS THE SAME AND SLOPE CROSSFRAME MEMBERS

M... USE CONSTANT DIMENSIONS AT ALL GIRDERS, USUALLY 2" TOP & BOTTOM

N... FILL MAY BE OMITTED WHERE POSSIBLE.

NON-PREFERRED "KNOCK-DOWN" TYPE CROSSFRAMES

AA... MAY BE ECONOMICAL TO FABRICATE IF C/C GIRDERS AND DROPS ARE THE SAME BUT STILL DIFFICULT TO ERECT. AVOID THIS TYPE WHEN DROPS VARY.

AB... KNOCKED DOWN CROSSFRAMES REQUIRE MORE SHOP & FIELD HANDLING THEY ARE COSTLY TO ERECT DUE TO THE INCREASED NUMBER OF DIFFERENT PIECES TO TRACK , HANDLE AND HOIST .

AC... WITH VARYING DROPS AND VARYING DISTANCES BETWEEN GIRDERS THIS FRAME WOULD REQUIRE A DIFFERENT LAYOUT FOR EACH STIFFENER , EACH DIAGONAL , EACH STRUT AND EACH FILL PLATE . THE CHANCE OF AN ERROR ON THIS TYPE OF FRAME IS GREATER THAN A JIGGED CROSSFRAME AND WIDER CONNECTION PLATES ARE REQUIRED IF USING 3 BOLTS PER END ON DIAGONALS.

AD... USE SINGLE MEMBERS WHERE EVER POSSIBLE. TYPICAL ALL CROSSFRAMES. AVOID DOUBLE MEMBERS (ANGLES, CHANNELS, ETC.) DOUBLE MEMBERS CANNOT BE PAINTED PROPERLY.

FIELD WELDED TYPE CROSSFRAME

1... SLOPE THE TOP & BOTTOM STRUTS AND KEEP THESE DIMENSIONS THE SAME ON EACH GIRDER, MAKING THE STIFFENERS IDENTICAL.

2... DO NOT CUT MEMBERS ON THE SKEW. IT IS MORE ECONOMICAL TO MAKE THE STIFFENER WIDER SINCE MOST FABRICATORS USE AN ANGLEMATIC MACHINE WHICH PUNCHES AND CUTS THE ANGLE SQUARE AUTOMATICALLY. A BEVELED CUT REQUIRES A BURNING OR SAWING OPERATION AT EACH END.

3... MAKE THIS DIMENSION LARGE ENOUGH TO PROVIDE ROOM BETWEEN THE STRUT AND FLANGE OR WEB FOR FIELD WELDING.

4... USE SINGLE MEMBERS WHERE EVER POSSIBLE. TYPICAL ALL CROSSFRAMES. AVOID DOUBLE MEMBERS (ANGLES, CHANNELS, ETC.) DOUBLE MEMBERS CANNOT BE PAINTED PROPERLY.

DISCLAIMER NOTE

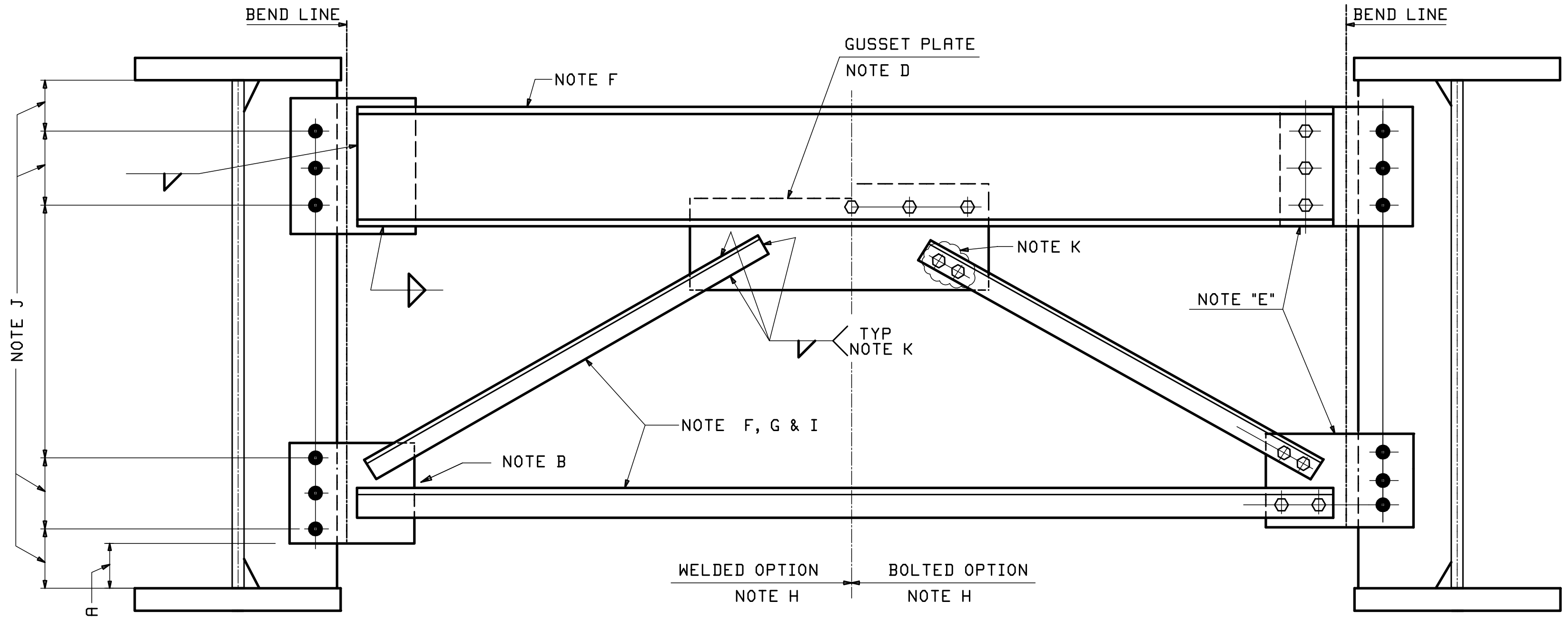
INFORMATION SHOWN IS FOR CONCEPT ONLY. APPLICATION TO SPECIFIC STRUCTURES IS THE DESIGNER'S RESPONSIBILITY.

TYPICAL CROSSFRAME DETAILS "X" TYPE

AASHTO/NSBA STEEL BRIDGE COLLABORATION

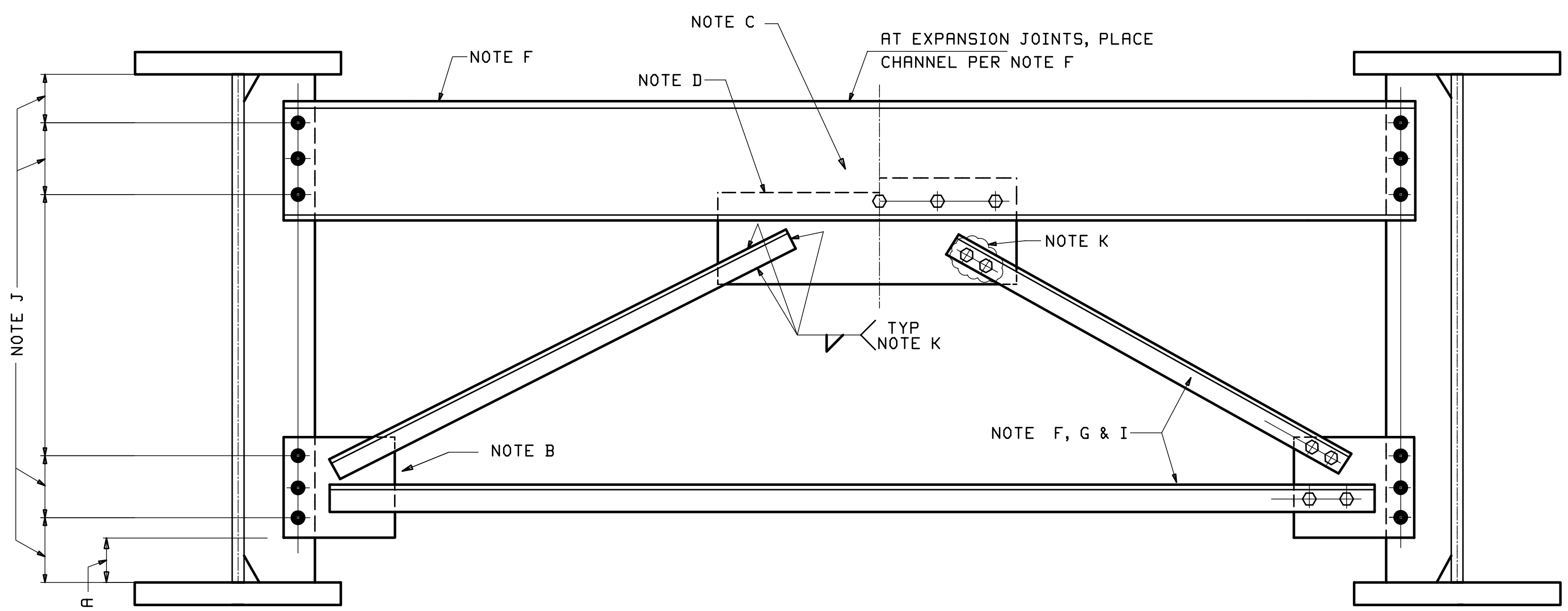
TASK GROUP 1, SUBTASK - GROUP 1.4
GUIDELINES FOR DESIGN DETAILS

PAGE NO. 109



END CROSSFRAME

FOR SKEWED CROSSFRAMES WITH BENT PLATE CONNECTIONS



END CROSSFRAME

FOR CROSSFRAMES WITHOUT BENT PLATE CONNECTIONS
CONNECT TOP MEMBER DIRECTLY TO CONNECTION PLATE.
FOR ADDITIONAL DETAILS SEE "PREFERRED BEARING TYPE
CROSSFRAME" ON SHEET 108.

PREFERRED BEARING TYPE CROSSFRAME

DISCLAIMER NOTE

INFORMATION SHOWN IS FOR CONCEPT ONLY. APPLICATION TO SPECIFIC STRUCTURES IS THE DESIGNER'S RESPONSIBILITY.

NOTES TO DESIGNERS:

END CROSSFRAME DETAIL

(SEE PAGE NO. 108 FOR ADDITIONAL NOTES AND PREFERRED CONNECTION PLATE DETAILS)

A... MODIFY THE DISTANCE BETWEEN THE BOTTOM GIRDER FLANGE AND THE LOWER DIAPHRAGM COMPONENT WHEN LOWER LATERAL BRACING IS USED. INDICATE MODIFICATIONS ON THE DESIGN DRAWINGS

B... SEE PREFERRED CONNECTION PLATE DETAIL ON PAGE NO. 108 FOR WELDING.

C... FILLS ARE NOT NECESSARY.

D... PLACE CHANNEL ON NEAR SIDE OF PLATES TO KEEP PLATES IN ONE PLANE FOR JIGGING. GUSSET PLATE MAY BE WELDED TO CHANNEL PRIOR TO ASSEMBLY IN JIG.

E... SHOW BOLTED OPTIONS FOR ALL BENT PLATES.

F... AT EXPANSION JOINTS, ORIENT CHANNEL FLANGES AND OUTSTANDING ANGLE LEGS AWAY FROM JOINT TO AVOID DEBRIS & CORROSION.

G... DESIGN DIAGONALS FOR WHEEL LOAD REACTIONS IF DECK HAUNCHES DOWN TO TOP STRUT.

H... DESIGN TO ALLOW FABRICATOR TO CHOOSE OPTION.

I... USE SINGLE MEMBERS WHERE POSSIBLE. AVOID DOUBLE MEMBERS (ANGLES, CHANNELS, ETC.) DOUBLE MEMBERS CANNOT BE PROPERLY PAINTED.

J... KEEP THESE DIMENSIONS THE SAME AND SLOPE THE CROSSFRAME MEMBERS.

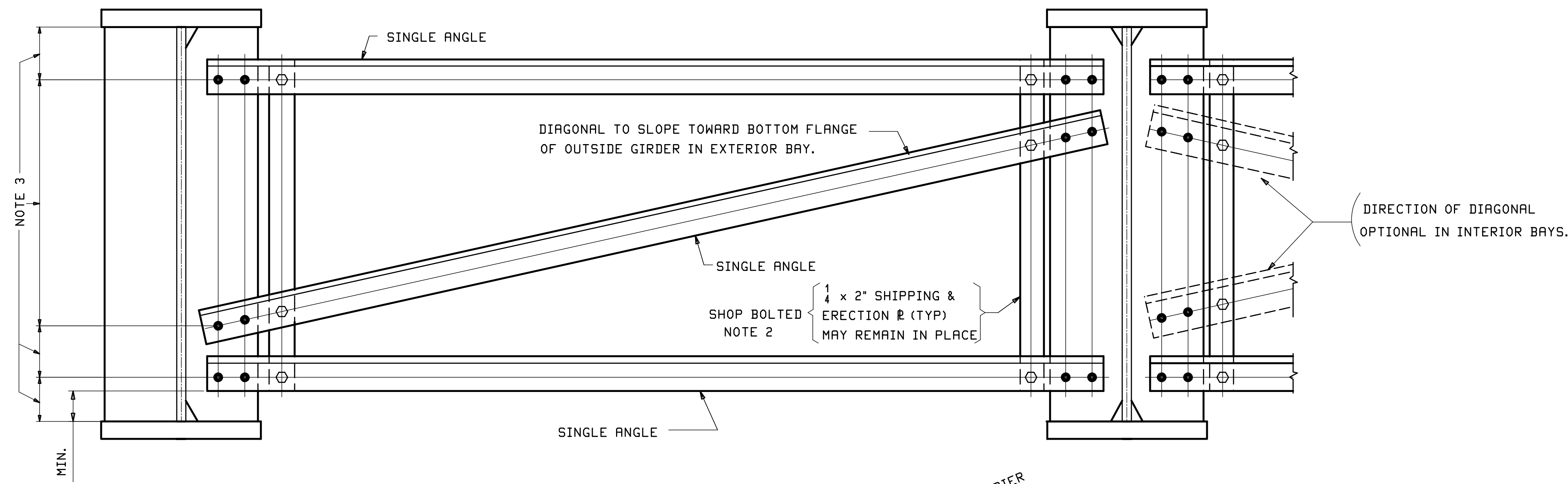
K... ON SKEWED END CROSSFRAMES, THE GEOMETRY OF THE CROSSFRAME DIAGONALS VARY DUE TO THE END CONNECTION PLATE ROTATION CAUSED BY THE DEAD LOAD CAMBER. SLOTTED HOLES OR FIELD WELDING MAY BE REQUIRED.

END CROSSFRAMES

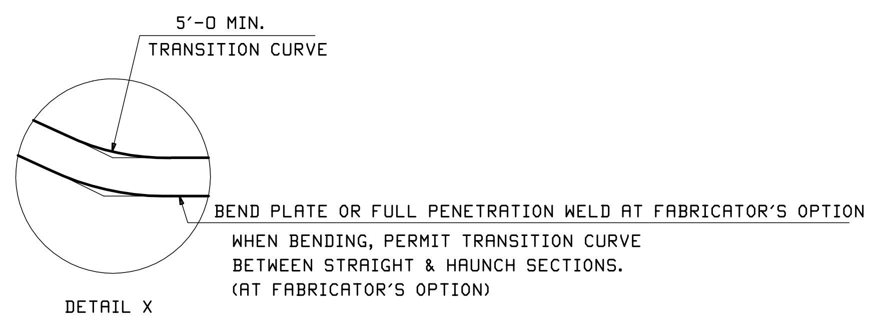
AASHTO/NSBA STEEL BRIDGE COLLABORATION

TASK GROUP 1, SUBTASK - GROUP 1.4
GUIDELINES FOR DESIGN DETAILS

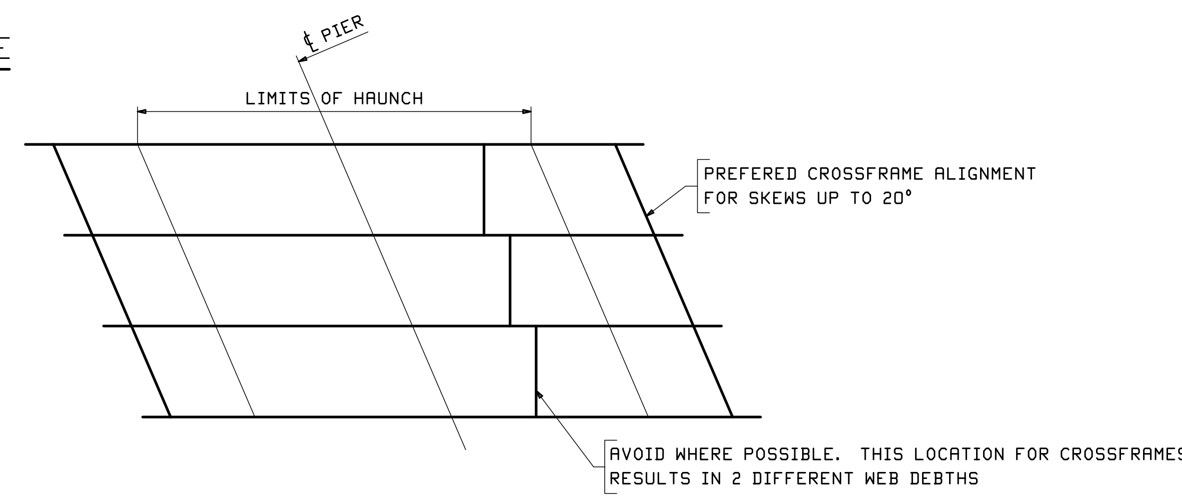
PAGE NO. 110



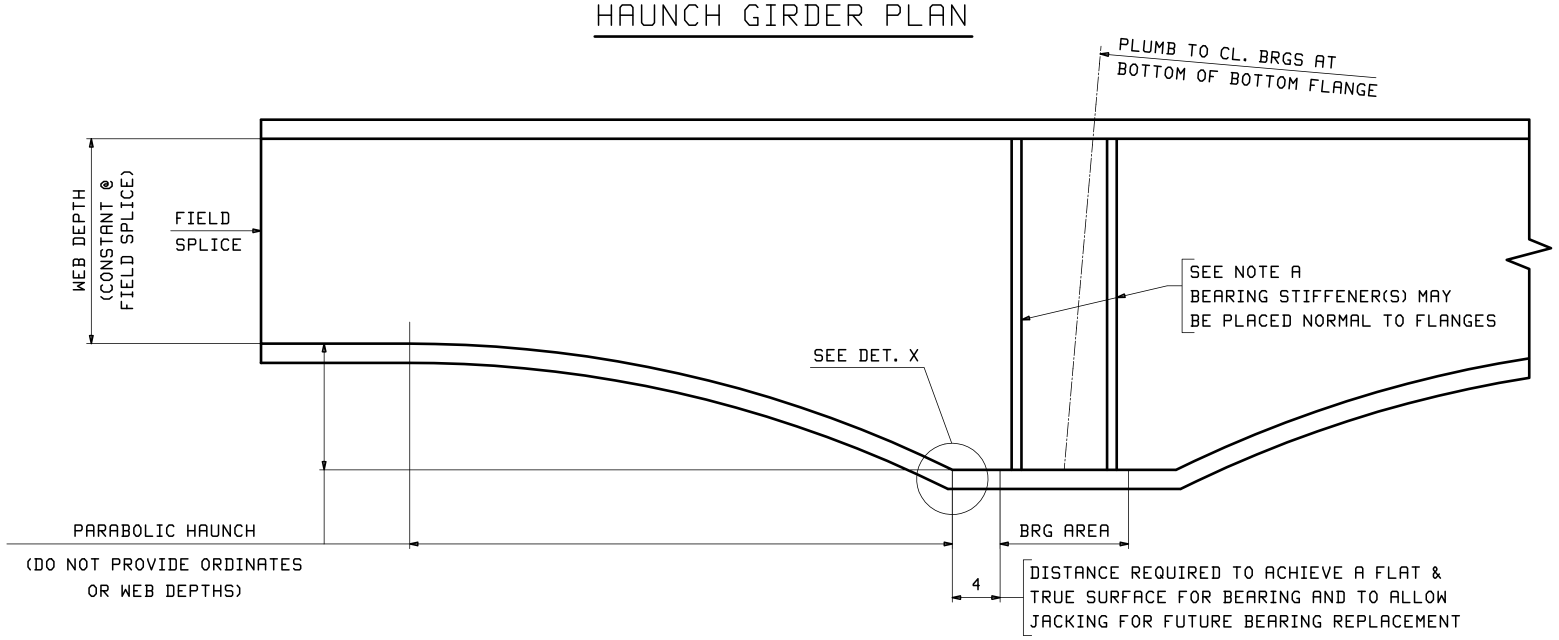
ALTERNATE-FRAME



DETAIL X



HAUNCH GIRDER PLAN



HAUNCH GIRDER ELEVATION

ALTERNATE-FRAME NOTES:

1...MODIFY THE DISTANCE BETWEEN THE BOTTOM GIRDER FLANGE AND THE LOWER DIAPHRAGM COMPONENT WHEN LOWER LATERAL BRACING IS USED.

2...NOT TO BE USED WITH ROLLED BEAMS OR PLATE GIRDERS < 42" DEEP. SIZE ANGLES FOR STRAIGHT GIRDER KL/r < 140 OR CURVED GIRDER KL/r < 120

3...KEEP THESE DIMENSIONS THE SAME AND KEEP THE CROSSFRAME LEVEL, MAKE LOW SIDE STIFFENER CONSTANT.

NOTES TO DESIGNERS:

A...PLACE BEARING STIFFENER NORMAL TO GRADE. NORMAL TO GRADE WILL BE MORE ECONOMICAL FOR FABRICATION. DESIGNER TO INVESTIGATE BEFORE PLACING STIFFENER VERTICAL.

DISCLAIMER NOTE

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INTERMEDIATE CROSSFRAMES & HAUNCH GIRDERS

AASHTO/NSBA STEEL BRIDGE COLLABORATION

TASK GROUP 1, SUBTASK - GROUP 1.4
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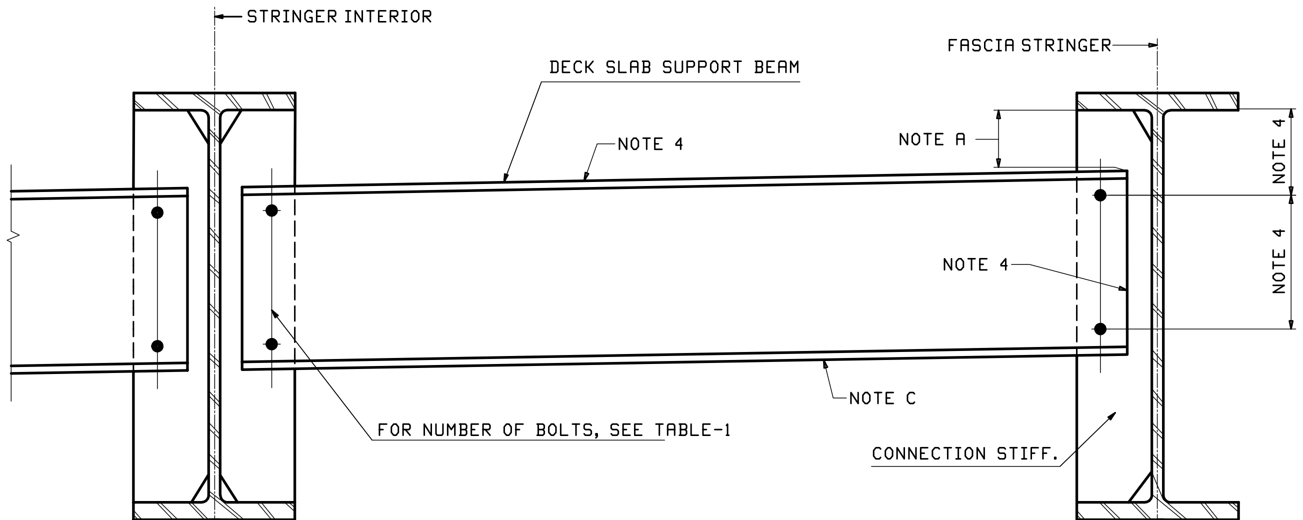
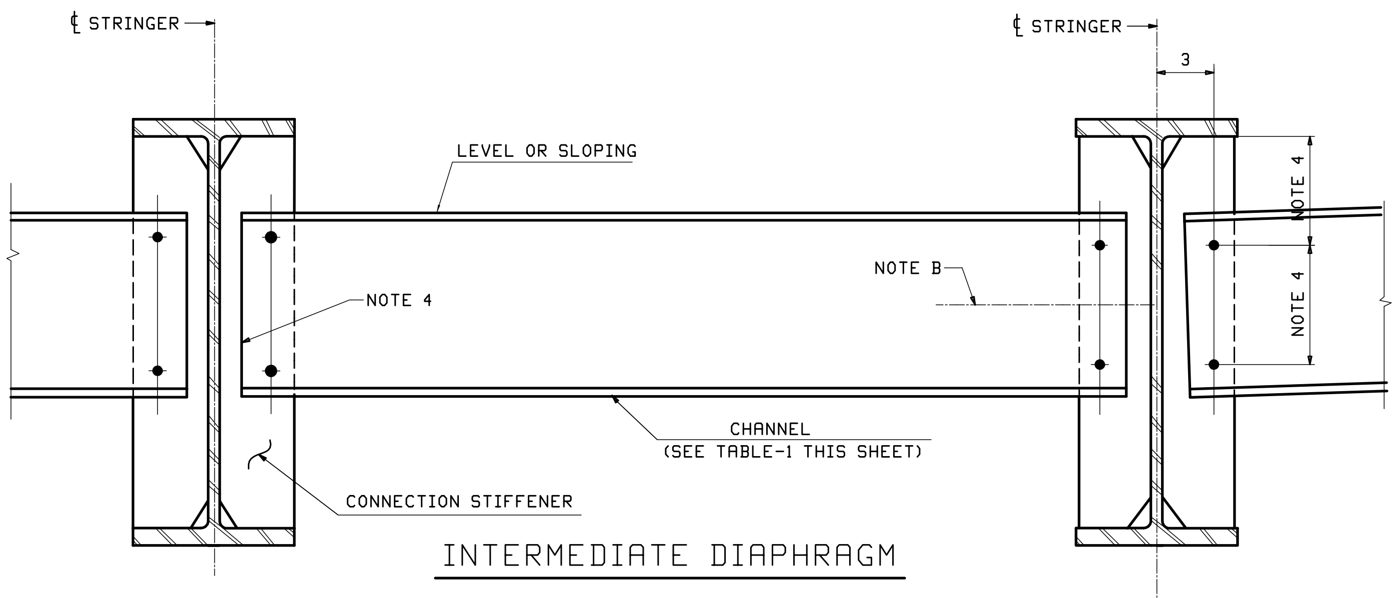
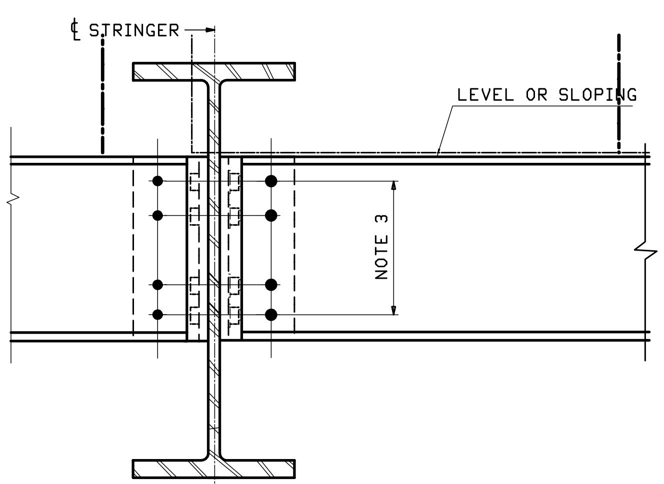


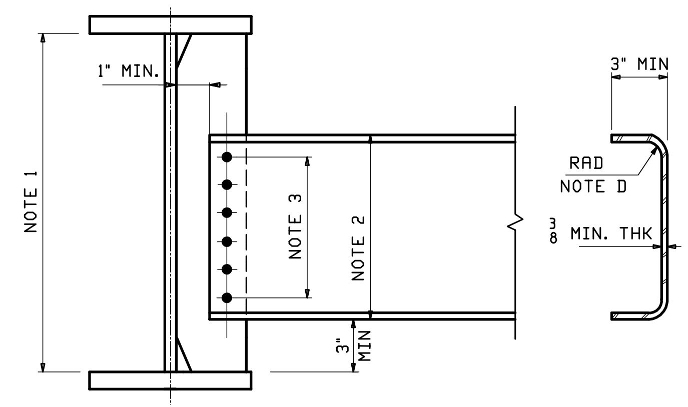
END DIAPHRAGM



INTERMEDIATE DIAPHRAGM



ALTERNATE DETAIL



BENT PLATE DIAPHRAGM

TABLE-1

STRINGER SIZE	DIAPHRAGM SIZE	NO. OF BOLTS
28 TO 36	C 18x42.7	5
25" TO 27"	C 15x33.9	4
UP TO 24" DEPTH	C 12x25	3

ROLLED SHAPE & BENT PLATE DIAPH. NOTES:

1...FOR WEB PLATE UP TO 48" DEEP.

2...DIAPHRAGM DEPTH - 0.4 TO 0.6 WEB DEPTH FOR ROLLED BEAMS AND 0.5 TO 0.75 WEB DEPTH FOR PLATE GIRDERS.

3...3" MIN and 7" MAX HOLE SPACING

4 ..SLOPE DIAPHRAGM AND KEEP HOLES VERTICAL IN STIFFENER AT CONSTANT DIMENSIONS (TO KEEP ALL STIFFENERS THE SAME) AND CUT ENDS OF DIAPHRAGM SQUARE.

NOTES TO DESIGNERS:

A... CONSTANT AT EACH STRINGER. DIMENSION BASED ON REQUIRED SLAB HAUNCH DEPTH.

B... MID DEPTH OF DIAPHRAGM AND STRINGER. TYPICAL AT EACH STRINGER FOR SLOPING DIAPH.

C... AT EXPANSION JOINT, ORIENT CHANNEL FLANGES AWAY FROM JOINT OPENING.

D... MINUMUM RADIUS AS PER AASHTO/NSBA FABRICATION S2.1

DISCLAIMER NOTE

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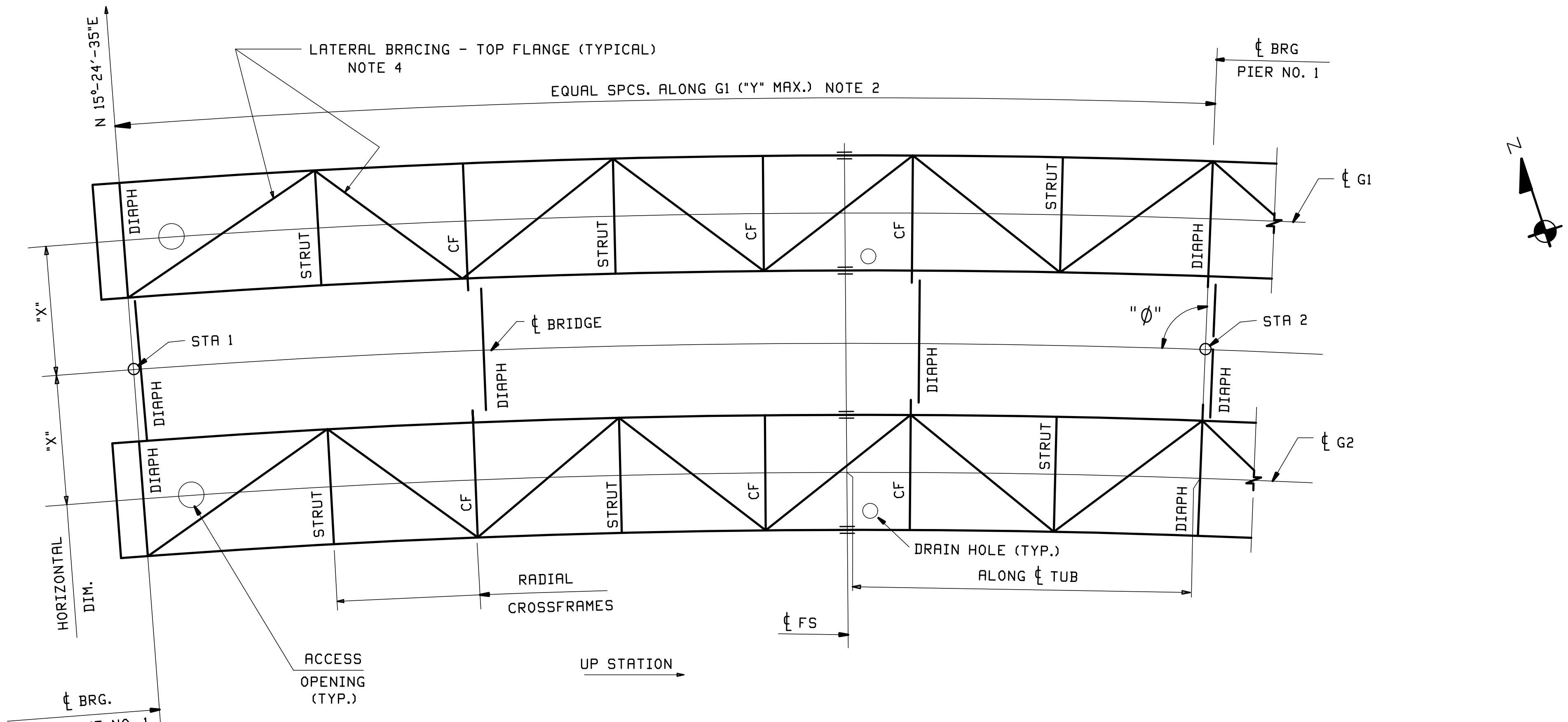
ROLLED SHAPE & BENT PLATE DIAPHRAGMS

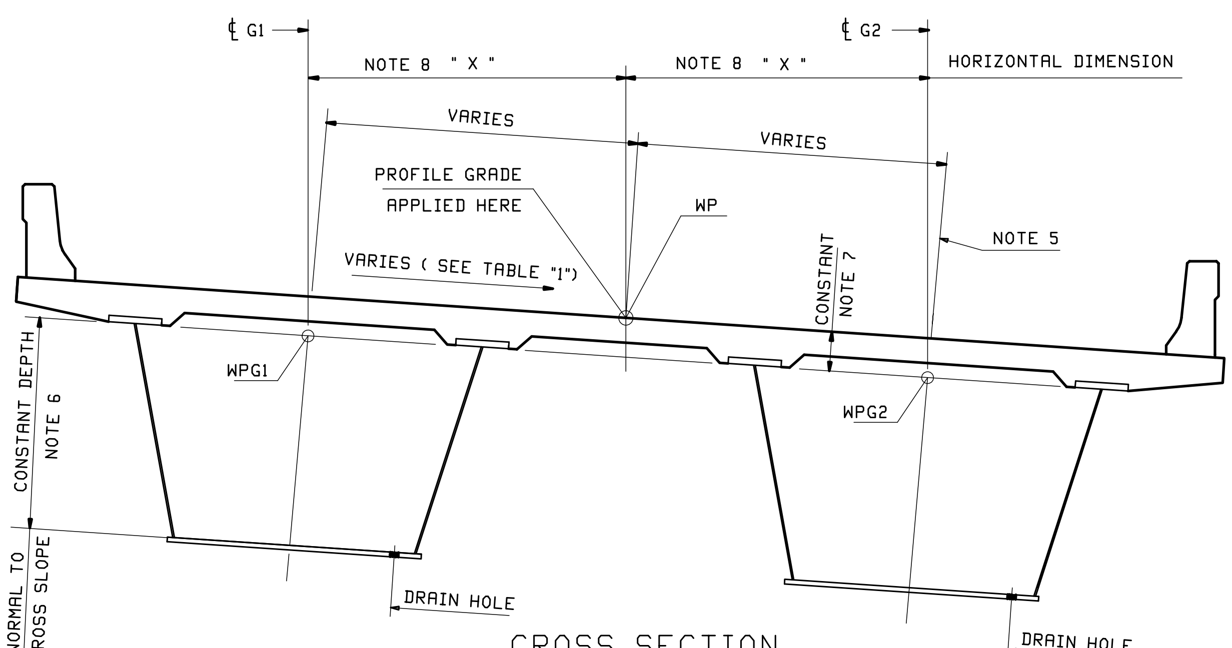
AASHTO/NSBA STEEL BRIDGE COLLABORATION

TASK GROUP 1, SUBTASK - GROUP 1.4
GUIDELINES FOR DESIGN DETAILS

PAGE NO. 112



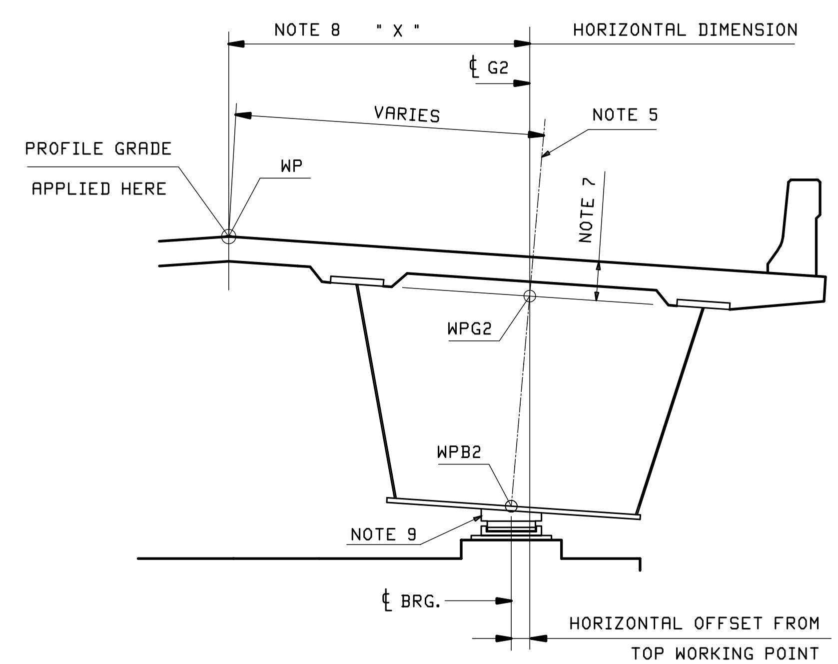
PLAN



CROSS SECTION

LOOKING UP STATION

TABLE "1"	
STATION	CROSS SLOPE OR SUPERELEVATION RATE



CROSS SECTION SHOWING BEARING OFFSET

GENERAL GEOMETRY FOR STEEL TUB GIRDERS

KEEP THE HORIZONTAL DIMENSION BETWEEN TUB GIRDERS TOP WP'S CONSTANT AND ROTATE TUBS FOR THE REQUIRED BRIDGE CROSS SLOPE. ALSO SEE NOTE 6. THIS MAY RESULT IN THE THE LENGTH OF THE EXTERNAL DIAPHRAGMS BEING DIFFERENT DUE TO CHANGES IN THE SUPERELEVATION BUT WILL KEEP THE TUB GIRDER CENTERLINES RELATIVE TO THE HORIZONTAL CONTROL LINE.

FRAMING PLAN GEOMETRY

1... BASIC INFORMATION REQUIRED:
- STATIONS AT PC, SC, PT, ℄ BRG, ETC.
- AZIMUTHS , BEARINGS OR SKEW ANGLE OF ℄ PIER / ℄ BRG
- STATION AT CROSS SLOPE OR LANE WIDTH CHANGES.

2... SPACE CROSSFRAMES ALONG ℄ OF LONGEST GIRDER FOR MAX. SPACING.
3... LOCATE FIELD SPLICES (FS) RADIALLY TO EACH TUB. LOCATE ℄ FS FROM ℄ PIER.
4... CROSSFRAME LOCATION BASED ON DESIGN, TRY TO KEEP LATERAL BRACING AT AN ANGLE BETWEEN 30 DEGREES TO 45 DEGREES TO GIRDER WEB.

CROSS SECTION GEOMETRY

5... ROTATE TUB GIRDER WITH CROSS SLOPE.
6... MAINTAIN CONSTANT TRAPEZOIDAL SHAPE FOR ALL GIDERS ON A STRUCTURE. VARY DISTANCE BETWEEN TUBS IF DECK WIDTH FLARES.
7... MAINTAIN A CONSTANT DIMENSION FROM THE TOP OF DECK TO THE TOP OF THE WEB PLATE. SHOW THIS DIMENSION ON DESIGN DRAWINGS.
8... ALL TRANSVERSE GIRDER LOCATIONS SHALL BE LOCATED HORIZONTALLY. DO NOT CONTROL THE GIRDER LOCATIONS ALONG THE DECK.
9... USE A SINGLE BEARING ON CURVED AND/OR SKEWED STRUCTURES, TO ALLOW FOR TRANSVERSE ROTATION OF TUB. NOTE: BEARING DESIGN MUST ACCOMODATE THIS ROTATION.

DISCLAIMER NOTE
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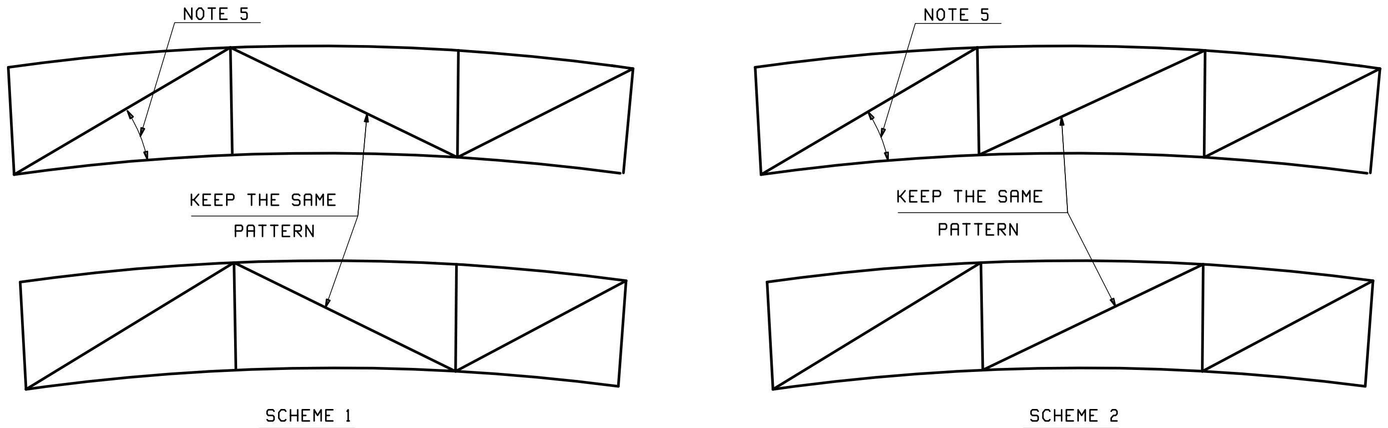
BASIC GEOMETRY - STEEL TUB GIRDERS

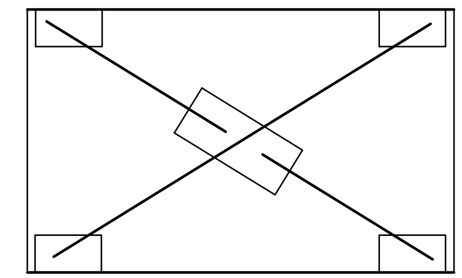
AASHTO/NSBA STEEL BRIDGE COLLABORATION

TASK GROUP 1, SUBTASK - GROUP 1.4
GUIDELINES FOR DESIGN DETAILS

PAGE NO. 113

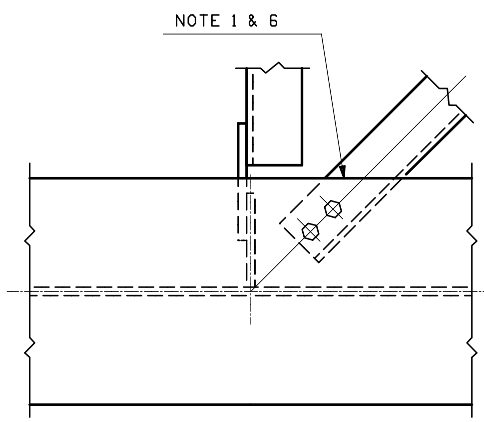


TYPICAL BRACING SCHEMES

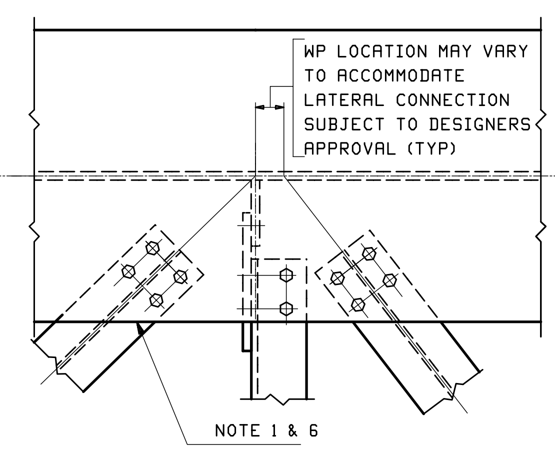


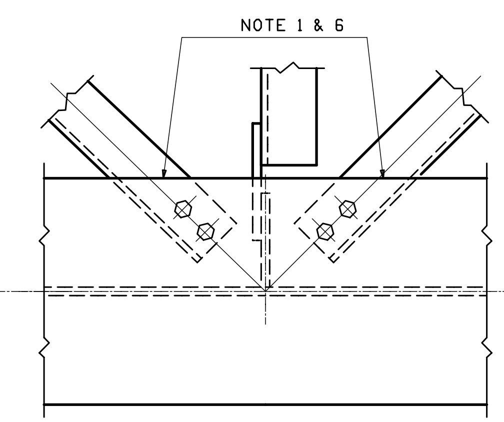
AVOID X BRACING

REQUIRES MORE PIECES, HOLES, BOLTS, AND UNNECESSSARILY STIFFENS GIRDER.

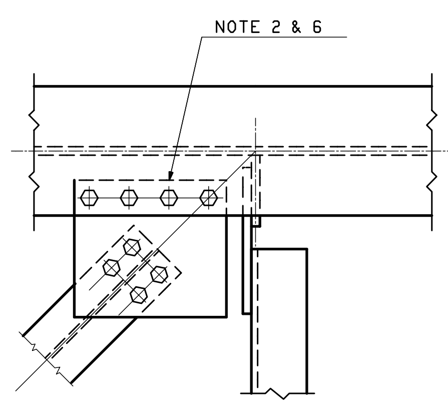


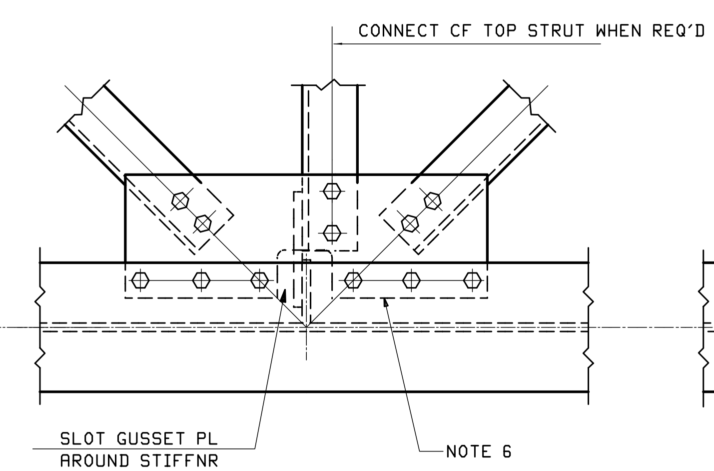
LATERAL BRACE BOLTED TO FLG
(PREFERED - NOTE 1)



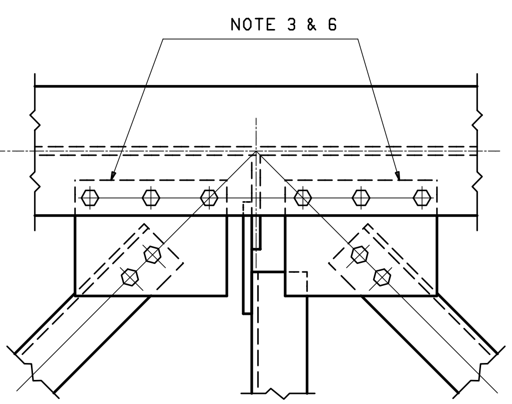


LATERAL BRACE BOLTED TO FLG
(PREFERED - NOTE 1)





LATERAL BRACE @ NARROW FLG



LATERAL BRACE & CF @ NARROW FLG

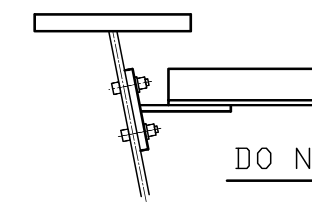
NOTES TO DESIGNER

1...WHEN POSSIBLE, BOLT LATERAL BRACING DIRECTLY TO FLANGES.

2...KEEP LATERAL GUSSET PLATES RECTANGULAR.

3...USE INDIVIDUAL GUSSET PLATES FOR EACH LATERAL BRACE WHERE POSSIBLE.

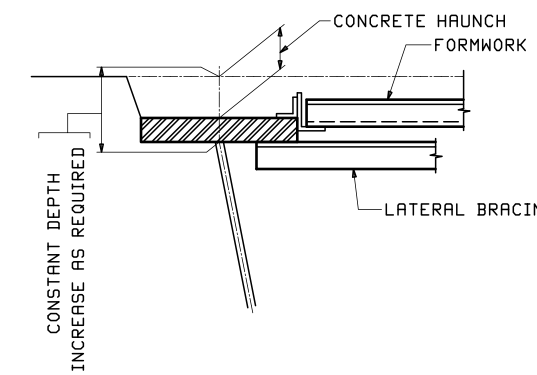
4...BRACING CONNECTIONS TO THE FLANGES ARE MORE ECONOMICAL, THAN CONNECTIONS TO THE WEBS, SINCE THEY INVOLVE FEWER COMPONENTS, AND BETTER FROM A DESIGN VIEWPOINT , SINCE IT PROVIDES A MORE DIRECT LOAD PATH.



DO NOT USE THIS DETAIL

5... ANGLE BETWEEN BRACING AND GIRDER SHOULD NOT BE LESS THAN 30° DEGREES, 45 DEGREES IS AN IDEAL ANGLE.

6... FILLS CAN BE USED TO DROP BRACING PLANE BELOW THE FORMWORK SUPPORTS. HOWEVER IT MAY BE MORE ECONOMICAL TO CUT FORMWORK AROUND THE BRACING MEMBERS. ANOTHER OPTION WOULD BE TO INCREASE THE HAUNCH DIMENSION AND RAISE THE FORMWORK SUPPORT ANGLES, SEE SKETCH:



OPTIONAL HAUNCH DETAIL

THIS DETAIL APPLIES TO STAY IN PLACE FORMS.
DESIGNER TO INVESTIGATE IF S.I.P FORMS CAN BE USED

7...FABRICATORS PREFER SCHEME 2 DUE TO MORE DUPLICATION OF DETAIL MATERIAL.

DISCLAIMER NOTE
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TUB GIRDER LATERAL BRACING

AASHTO/NSBA STEEL BRIDGE COLLABORATION

TASK GROUP 1, SUBTASK - GROUP 1.4
GUIDELINES FOR DESIGN DETAILS

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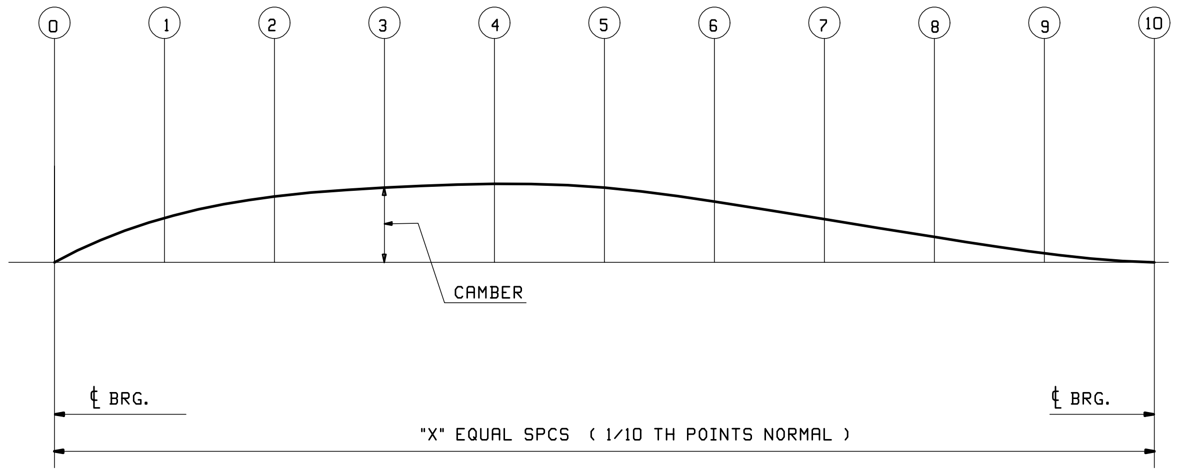
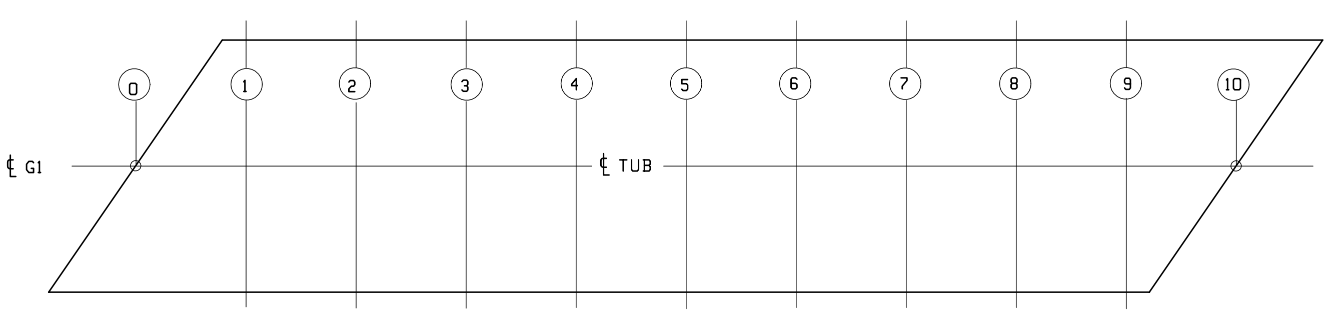


		VALUES SHOWN ARE IN INCHES (NOTE 1)										
LINE	CAMBER VALUES DUE TO DL	0	1	2	3	4	5	6	7	8	9	10
G1	STEEL DL	0										0
	DECK DL	0										0
	SUPERIMPOSED DL	0										0
	TOTAL DL	0										0

DO NOT PROVIDE GEOMETRIC CAMBER FOR TUB GIRDERS SINCE THIS INFORMATION IS NORMALLY GIVEN IN A VERTICAL PLANE AND A TUB IS NORMAL TO THE CROSS SLOPE. THE CAMBER DIAGRAM FOR EACH WEB IS BASED ON A MODIFIED CONICAL SHAPE IN THE PLANE OF THE WEB PLATE, WHICH IS NOT IN A VERTICAL PLANE.



TUB GIRDER CAMBER DIAGRAM

Show Camber Data Along ℄ Tub

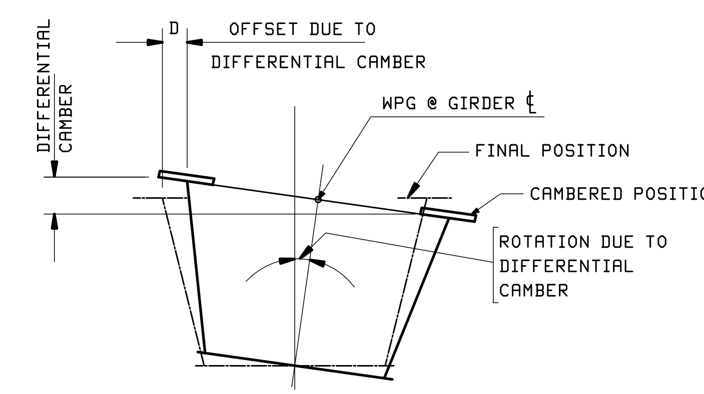
NOTES FOR DESIGN DRAWINGS:

1...CAMBERS CAN BE GIVEN IN FRACTIONS, DECIMAL OF A FOOT OR DECIMAL INCHES. STATE CLEARLY IF DIMENSIONS ARE IN FEET OR INCHES.

2...CAMBER INFORMATION IS SHOWN FOR A TUB GIRDER.

NOTES TO DESIGNER

1... AVOID HAVING DIFFERENT CAMBERS FOR EACH WEB BECAUSE OF PROBLEMS DUE TO DIFFERENTIAL CAMBERS.



DIFFERENTIAL CAMBER

PROBLEMS WHEN NEAR WEB AND FAR WEB HAVE DIFFERENT CAMBERS. THIS CAN BE AVOIDED IF THE TUB IS CAMBERED ALONG ITS ℄

TUB GIRDER CAMBER DIAGRAM

AASHTO/NSBA STEEL BRIDGE COLLABORATION

TASK GROUP 1, SUBTASK - GROUP 1.4
GUIDELINES FOR DESIGN DETAILS

DISCLAIMER NOTE

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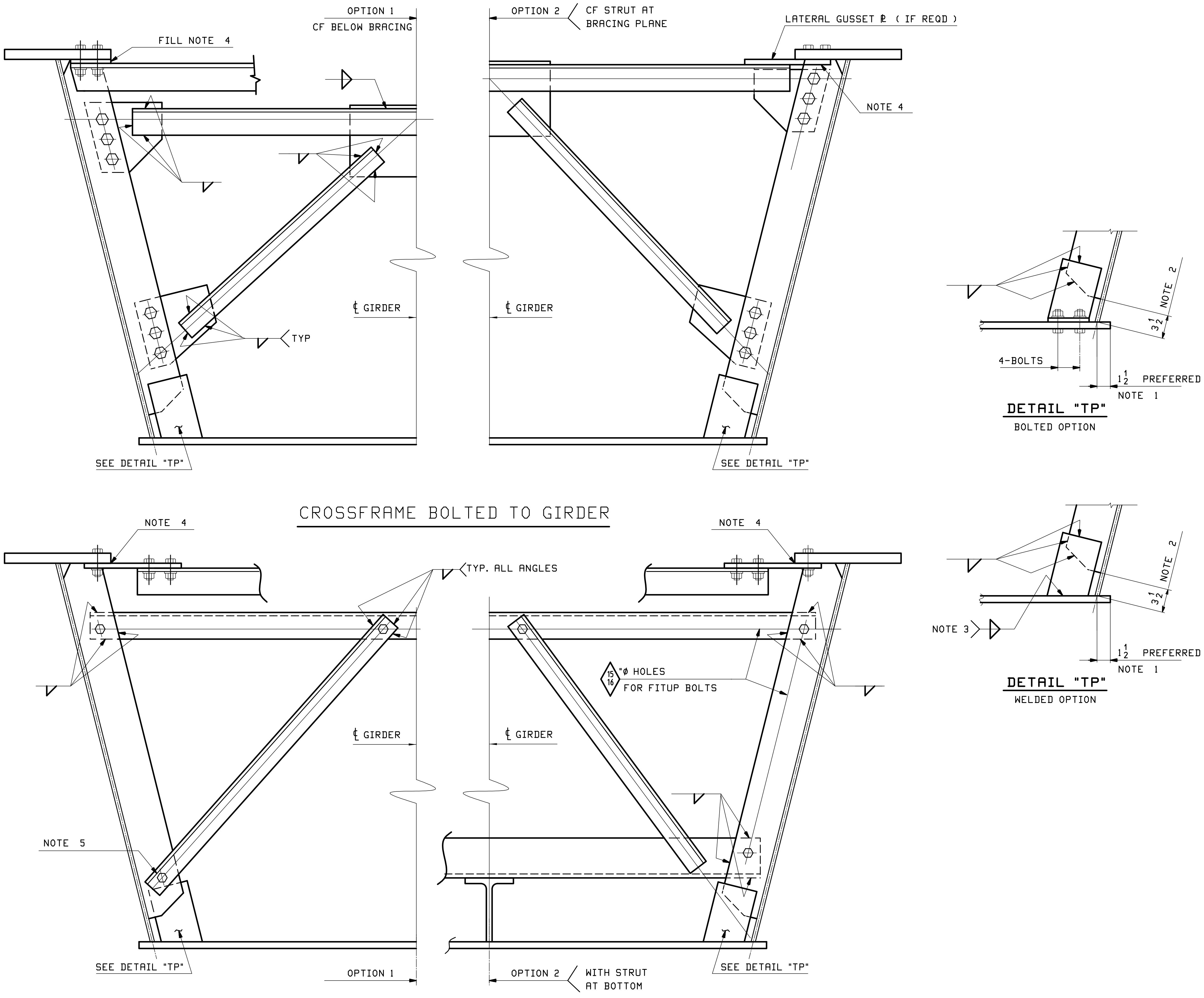

CROSSFRAME WELDED TO GIRDER

TYPICAL CROSSFRAME DETAILS FOR TUB GIRDER BRIDGES

DESIGNER MAY SHOW EITHER WELDED OR BOLTED CROSSFRAMES, BUT CONSIDER A FABRICATOR'S REQUEST FOR ALTERNATIVES.

1...THE BOTTOM FLANGE EXTENSION IS REQUIRED FOR FLUX SUPPORT AND THE WELDING MACHINE TRACKING. 1" MIN. MOST FABRICATOR'S PREFER $1\frac{1}{2}$"MIN.

2...A GAP OF $3\frac{1}{2}$" IS PREFERRED, DESIGNER TO CHECK 4 TO 6 tw REQUIREMENT. THE GAP AT THE BOTTOM ALLOWS THE WEB TO FLANGE WELDING TO BE MADE WITHOUT INTERRUPTIONS. HOWEVER THIS DETAIL MAY VARY DEPENDING ON THE FABRICATOR'S EQUIPMENT AND PROCEDURES. EXTENDING THE STIFFENER TO THE FLANGE WITHOUT THE USE OF TAB PLATES IS ACCEPTABLE.

3...WELD TO FLANGE WHEREVER FATIGUE STRESS RANGE PERMITS. IF REQUIRED FOR FATIGUE , USE A BOLTED TAB PLATE. SEE DETAIL "TP".

4...FILLS CAN BE USED TO DROP BRACING PLANE BELOW THE FORMWORK SUPPORTS. HOWEVER IT MAY BE MORE ECONOMICAL TO MODIFY FORMWORK AROUND THE BRACING MEMBERS.

5...ASSEMBLY BOLTS MUST NOT INTERFERE WITH WELDING.

USUAL ASSEMBLY SEQUENCE

A...WEB, TOP FLANGE AND STIFFENERS ARE USUALLY FABRICATED AS A SUB-ASSEMBLY PRIOR TO FITTING TO THE BOTTOM FLANGE.

B...THE CROSSFRAME IS BUILT IN A JIG AS A SUB-ASSEMBLY, FIT-UP AND WELDED. NOTE THAT ALL WELDING IS MADE FROM NEAR SIDE. BOLTED CROSSFRAMES ARE PREFERRED BY MOST FABRICATORS WHICH MINIMIZES ROLLING TUBS TO GET PROPER POSITION FOR WELDING.

C...THE CROSSFRAME SUB-ASSEMBLY IS THEN BOLTED TO THE WEB/TOP FLANGE SUB-ASSEMBLY WHICH WILL CONTROL SHAPE FOR THE FINAL GIRDER ASSEMBLY.

D...THE WEB/TOP FLANGE SUB-ASSEMBLY WITH THE CROSSFRAMES BOLTED IN PLACE IS THEN FITTED TO THE BOTTOM FLANGE PLATE WHICH HAS BEEN BLOCKED TO ITS CAMBERED SHAPE. THE WEB TO BOTTOM FLANGE PLATE WELDS ARE THEN MADE.

DISCLAIMER NOTE
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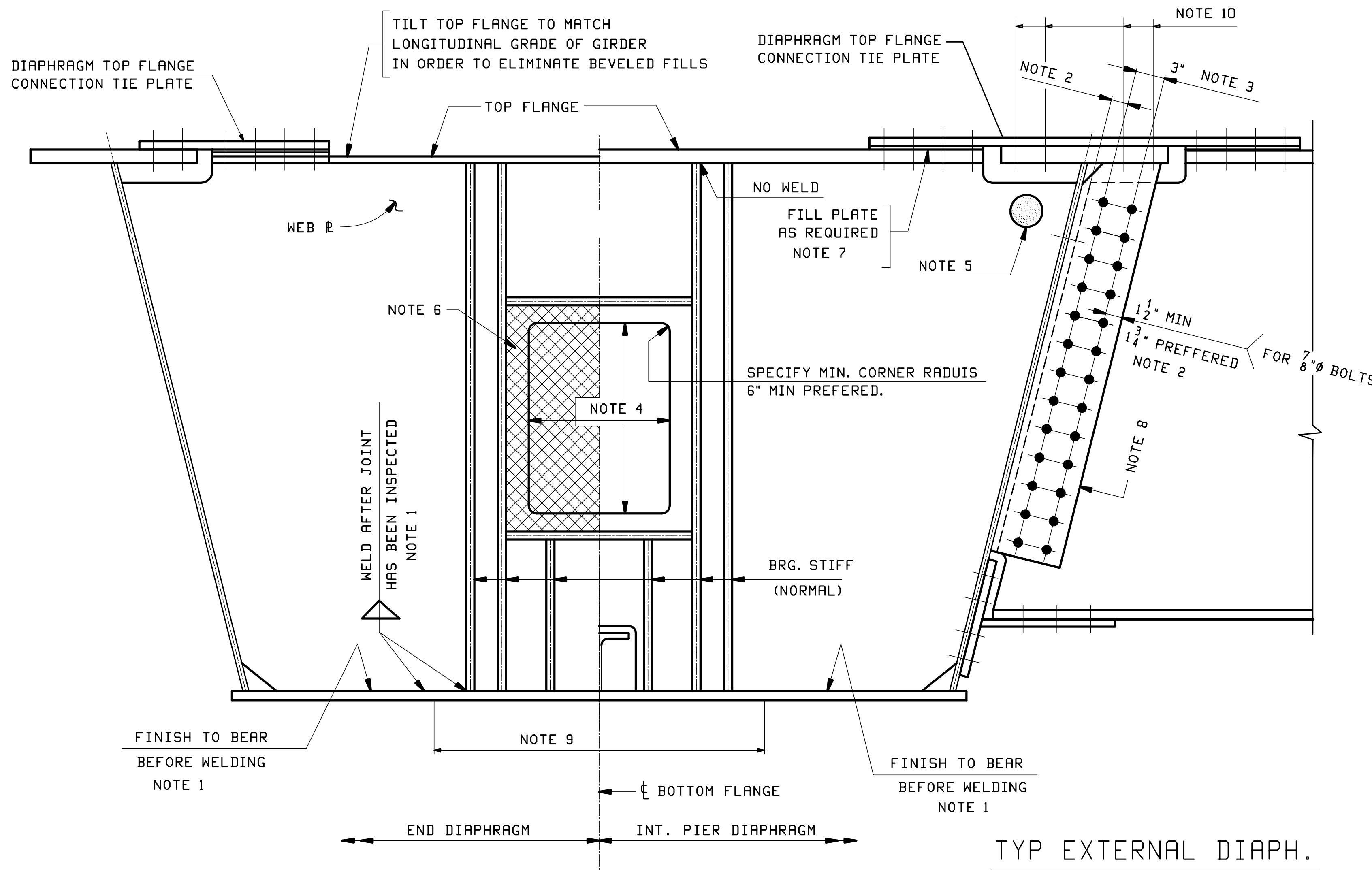
TYPICAL CROSSFRAME DETAILS
FOR TUB GIRDERS

AASHTO/NSBA STEEL BRIDGE COLLABORATION

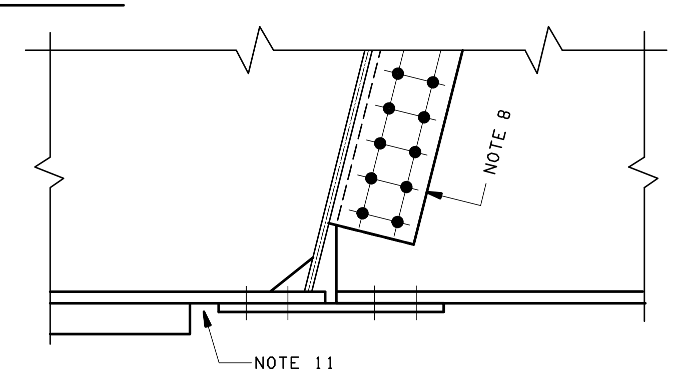
TASK GROUP 1, SUBTASK - GROUP 1.4
GUIDELINES FOR DESIGN DETAILS



TYP EXTERNAL DIAPH.

(OPTION 1)

BEARING DIAPHRAGMS



TYP EXTERNAL DIAPH.

(OPTION 2)

DISCLAIMER NOTE

INFORMATION SHOWN IS FOR CONCEPT ONLY. APPLICATION TO SPECIFIC STRUCTURES IS THE DESIGNER'S RESPONSIBILITY.

NOTES TO DESIGNERS

1...DETAIL DIAPHRAGM ASSEMBLY/BOTTOM FLANGE CONNECTION WITH FILLET WELDS AND FINISH TO BEAR SURFACES. AVOID FULL PENETRATION WELDS.

2...SINCE MOST FABRICATOR'S PREFER MORE THAN MINIMUM EDGE TO ALLOW FOR FABRICATION AND DRILLING TOLERANCES, PROVIDE ANOTHER $\frac{1}{8}$" TO $\frac{1}{4}$" MORE MATERIAL THAN THE MINIMUM REQUIRED.

3...ALLOW SUFFICIENT DISTANCE BETWEEN END DIAPHRAGMS AND BACK WALLS TO FACILITATE FIELD BOLTING (PREFERRED) OR PROVIDE ACCESS HOLES.

4...SIZE DIAPHRAGM ACCESS OPENINGS IN ACCORDANCE WITH STATE DESIGN GUIDELINES. 18" x 36" PREFERRED MINIMUM.

5...PROVIDE OPENINGS IN DIAPHRAGM TO FACILITATE RACEWAYS FOR MAINTENANCE LIGHTING CONDUIT. USE EXISTING CLIPS AT WEB TO FLANGE WELD IF POSSIBLE.

6...DIAPHRAGM ACCESS OPENINGS AT END OF UNITS SHALL BE COVERED BY SCREENED DOOR TO ALLOW FOR VENTILATION AND INSPECTION ACCESS AT PIERS WHILE PROHIBITING ANIMAL ACCESS. AT END DIAPHRAGMS, ADJACENT TO ABUTMENTS, AN ACCESS HATCH IN THE BOTTOM FLANGE SHOULD BE PROVIDED. SEE PAGE MO. 118 FOR DETAILS.

7...FILLS MAY BE REQUIRED FOR FIT UP ($\frac{1}{8}$" MIN) THICKER FILLS MAY REQUIRE DEVELOPING THE FILL BY EXTENDING PAST THE END OF THE TOP FLANGE CONNECTION TIE PLATE, DESIGNER TO CHECK AASHTO REQUIREMENTS.

8...CONNECTION CAN BE MADE WITH EITHER A CONNECTION ℙ OR A ANGLE. AVOID USING END PLATES WELDED TO THE DIAPHRAGM WEB.

9...FINISH BEARING CONTACT AREA TO BEAR. (PER 3.5.1.9 OF D1.5 BRIDGE WELDING CODE)

10...WHEN POSSIBLE, DO NOT CONNECT TIRE PLATE TO TOP FLANGE. IF BOLTING TIE PLATE TO GIRDER IS REQUIRED, DESIGNER SHOULD INVESTIGATE NET SECTION OF TOP FLANGE DUE TO THIS HIGHLY STRESSED AREA.

11...DESIGNER TO CHECK CLEARANCE TO SOLE PLATE.

GENERAL DETAILING FABRICATION METHODS:

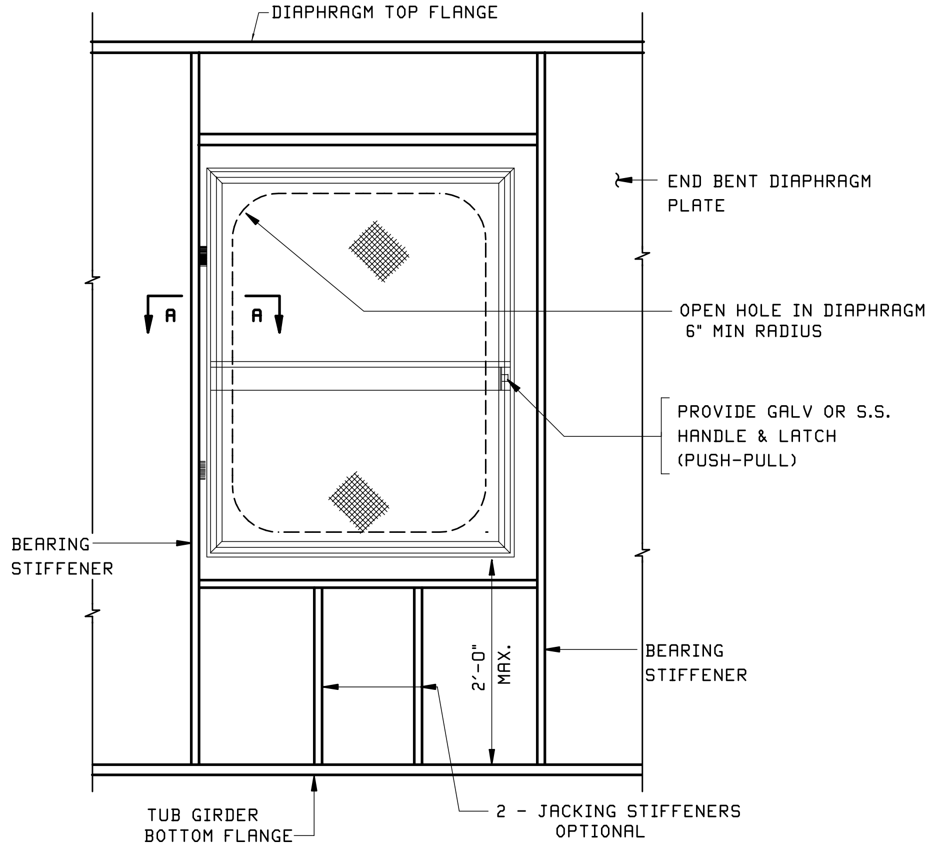
A...DETAIL DIAPHRAGM SO IT CAN BE SUB-ASSEMBLED, THEN FITTED TO BOTTOM FLANGE AND WEB ASSEMBLIES IN THE SHOP. KEEP STIFFENERS NORMAL TO BOTTOM OF FLANGE. IF BEARINGS ARE BOLTED TO FLANGE, THEN CHECK BOLT CLEARANCE TO STIFFENERS AND WELDS.

BEARING DIAPHRAGMS - TUB GIRDER BRIDGES

AASHTO/NSBA STEEL BRIDGE COLLABORATION

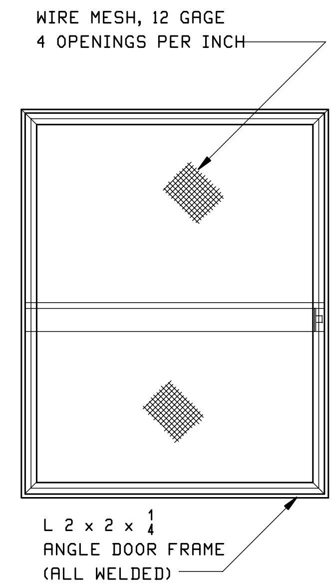
TASK GROUP 1, SUBTASK - GROUP 1.4
GUIDELINES FOR DESIGN DETAILS

PAGE NO. 117

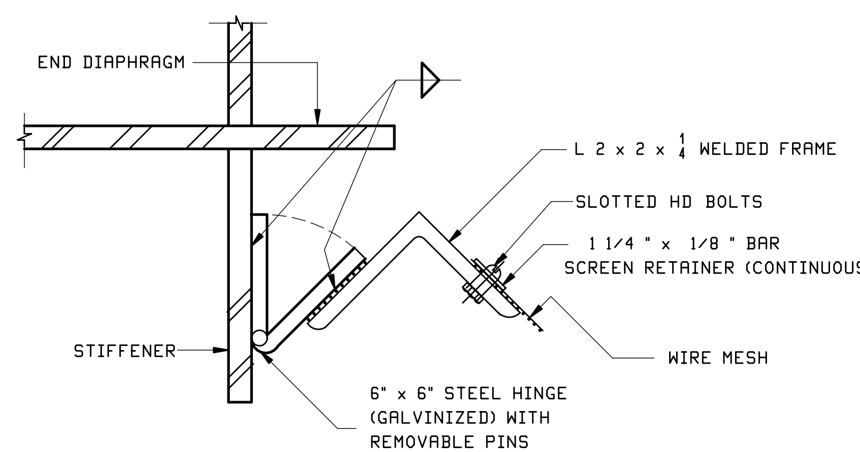


ELEVATION OF ACCESS DOOR

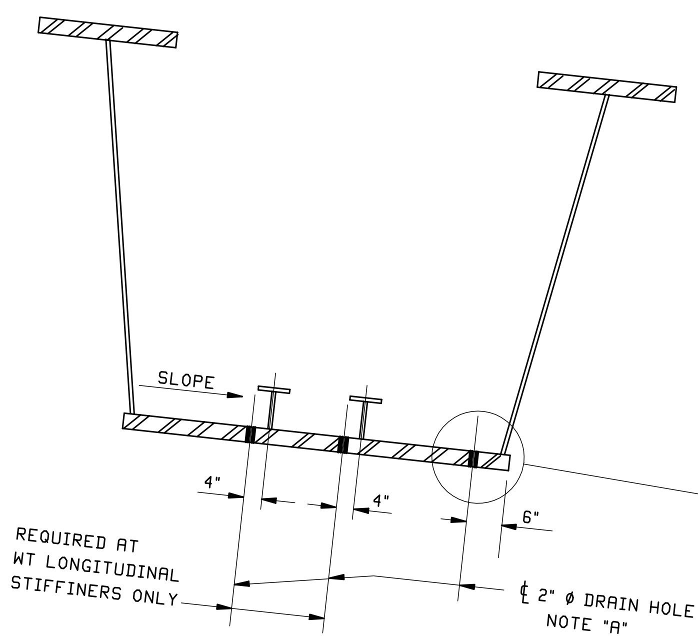
AT END BENT DIAPHRAGM OPENING



DOOR FRAME

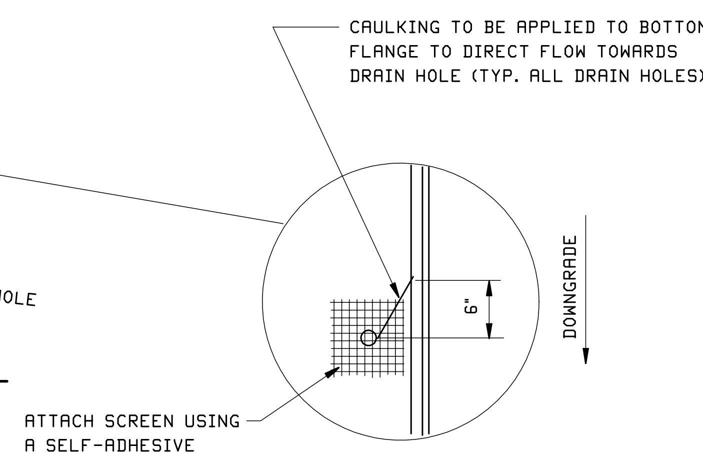


SECTION A-A



SECTION - GIRDER DRAIN HOLE DETAIL

PLACE DRAIN HOLES AT 50'-0" MAXIMUM SPACING,
AND 5'-0 MIN. FROM ℄ PIER OR BACKWALL.



PLAN OF DRAIN HOLES

NOTES FOR DESIGN DRAWINGS:

1...DOOR MUST OPEN TOWARDS THE INSIDE OF THE STEEL TUB GIRDER.

2...COST OF SCREENED CLOSURE DOOR IS INCIDENTAL TO THE COST OF STRUCTURAL STEEL.

3...STRUCTURAL STEEL FABRICATOR SHALL SUBMIT SHOP DRAWINGS FOR APPROVAL.

4...ALL WORK SHOWN ON THIS SHEET SHALL BE SHOP FABRICATED AND MOUNTED PRIOR TO SHIPPING TO THE JOB SITE.

5... THESE DETAILS & MATERIAL SIZES ARE SUGGESTED AND ARE FOR A GUIDE ONLY, ENGINEER SHOULD CHECK WITH OWNER FOR POSSIBLE PREFERRED STANDARDS.

NOTE "A":
COVER VENT HOLES AND DRAIN HOLES WITH 20 GAGE GALVANIZED WELDED METAL SCREENING (1/4 " OPENINGS). ATTACH TO GIRDER WEBS/FLANGES, WITH AN OWNER APPROVED METHOD.

DISCLAIMER NOTE

INFORMATION SHOWN IS FOR CONCEPT ONLY. APPLICATION TO SPECIFIC STRUCTURES IS THE DESIGNER'S RESPONSIBILITY.

STEEL TUB SCREENING DETAILS

AASHTO/NSBA STEEL BRIDGE COLLABORATION

TASK GROUP 1, SUBTASK - GROUP 1.4
GUIDELINES FOR DESIGN DETAILS

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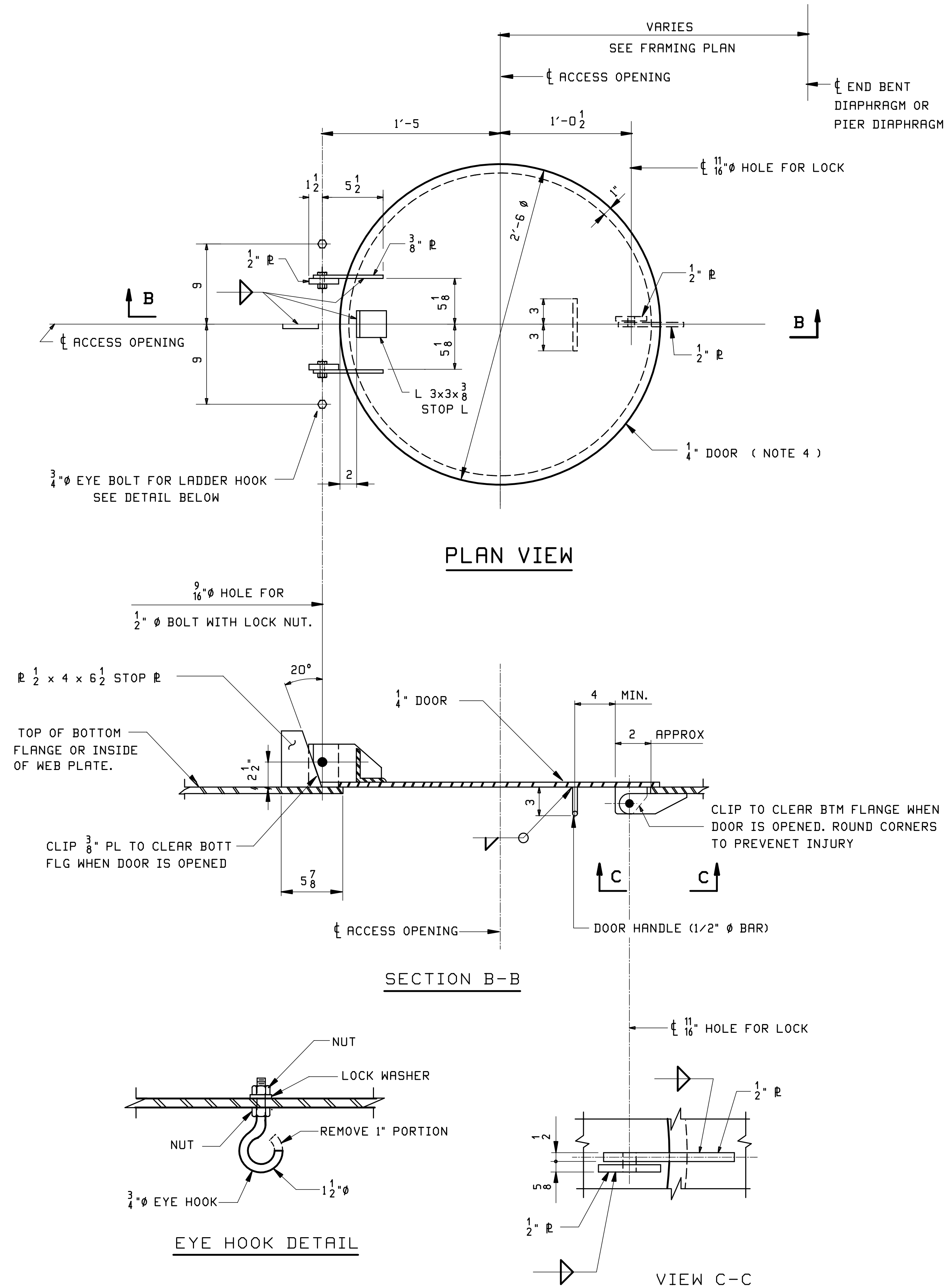

NOTE !!

THESE DETAILS & MATERIAL SIZES ARE SUGGESTED AND ARE FOR A GUIDE ONLY, ENGINEER SHOULD CHECK WITH OWNER FOR POSSIBLE PREFERRED STANDARDS.

THIS DETAIL COULD BE USED FOR A WEB OR FLANGE ACCESS DOOR.

NOTES FOR DESIGN DRAWINGS:

1...FOR ACCESS OPENING LOCATION SEE FRAMING PLAN.

2...ALL STRUCTURAL STEEL IN ACCESS HATCH SHALL BE ASTM A709 GRADE 36 AND SHALL BE GALVANIZED AFTER FABRICATION IN ACCORDANCE WITH ASTM A-123.

3...ALL EXPOSED EDGES OF PLATES AND OPENINGS SHALL BE GROUND SMOOTH.

NOTES FOR DESIGNERS

CHECK THE RESULTING CROSS-SECTION OF BOTTOM FLANGE TO DETERMINE IF A REINFORCING PLATE IS REQUIRED.

4...INVESTIGATE USE OF A PERFORATED PLATE, GRATING, OR OTHER OWNER PREFERRED MATERIAL.

5...DOOR MUST OPEN TOWARDS INSIDE OF THE STEEL TUB GIRDER.

DISCLAIMER NOTE

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ACCESS OPENING DETAILS

AASHTO/NSBA STEEL BRIDGE COLLABORATION

TASK GROUP 1, SUBTASK - GROUP 1.4
GUIDELINES FOR DESIGN DETAILS

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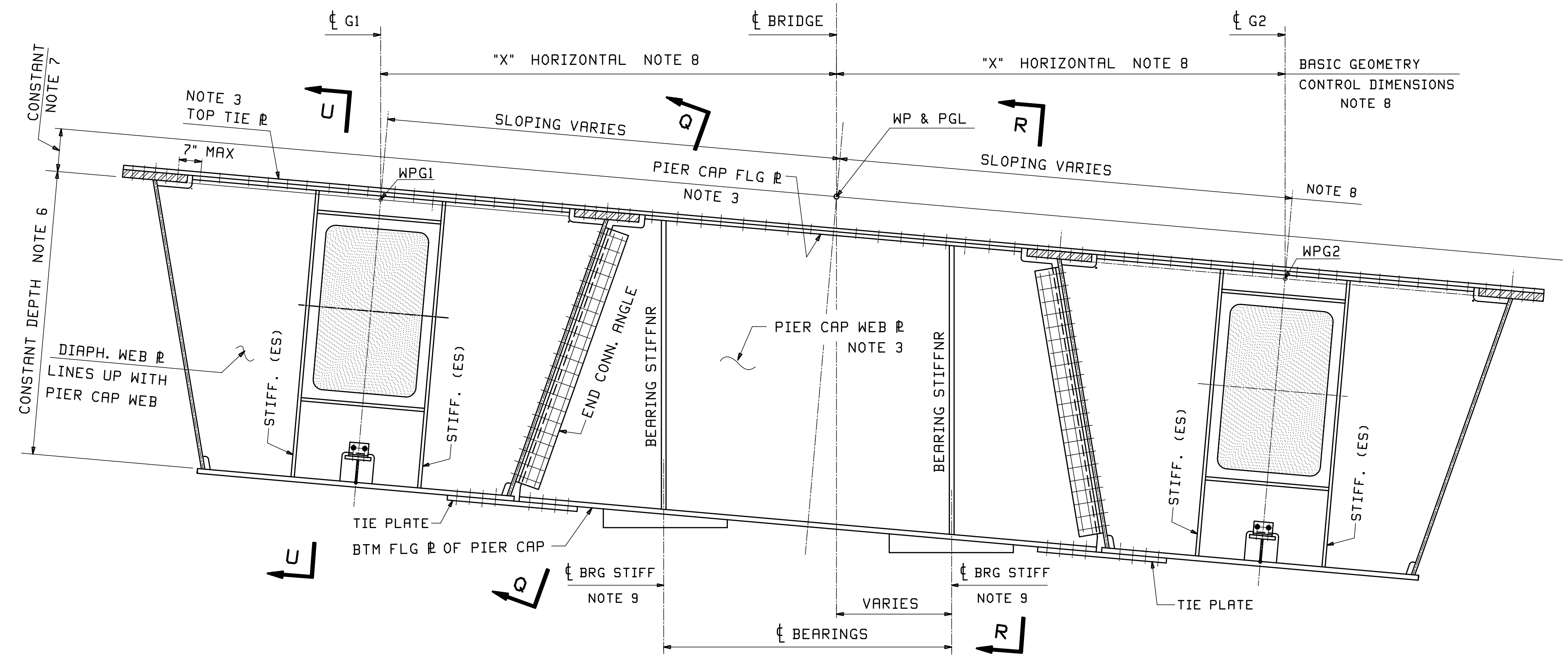
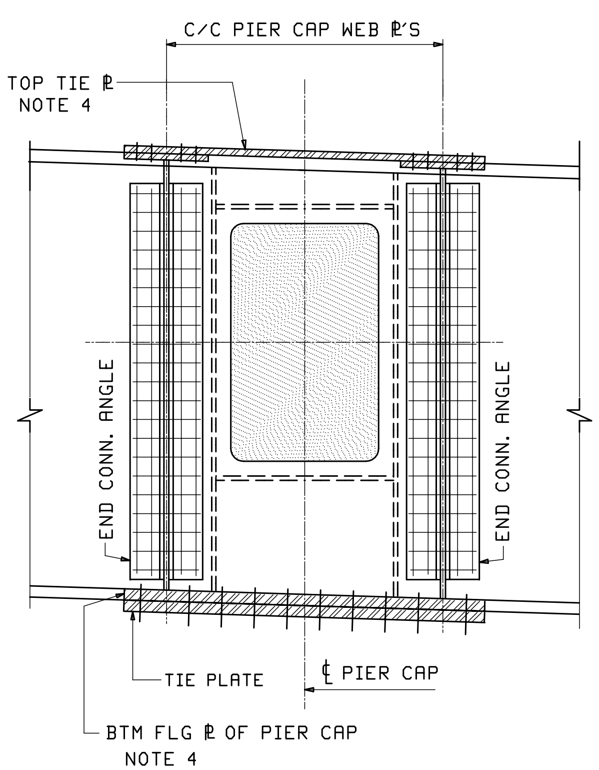
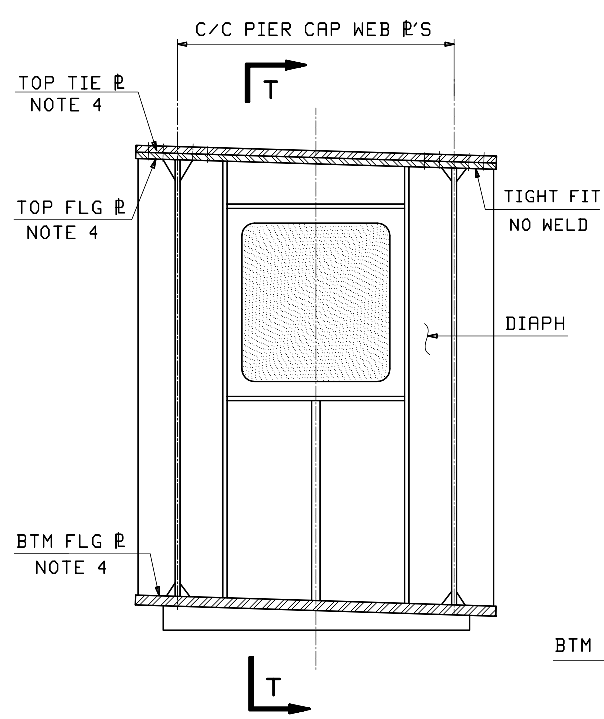


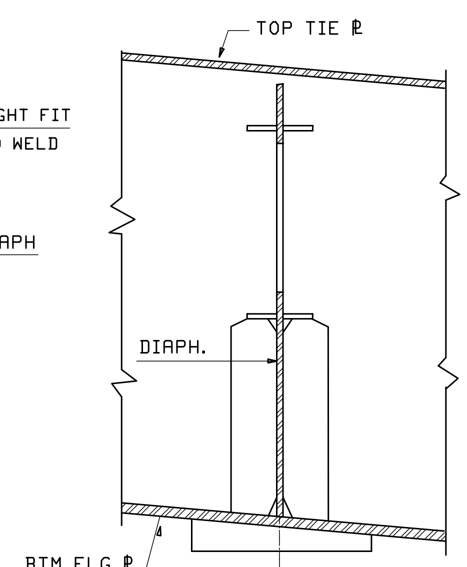
SECTION AT PIER CAP



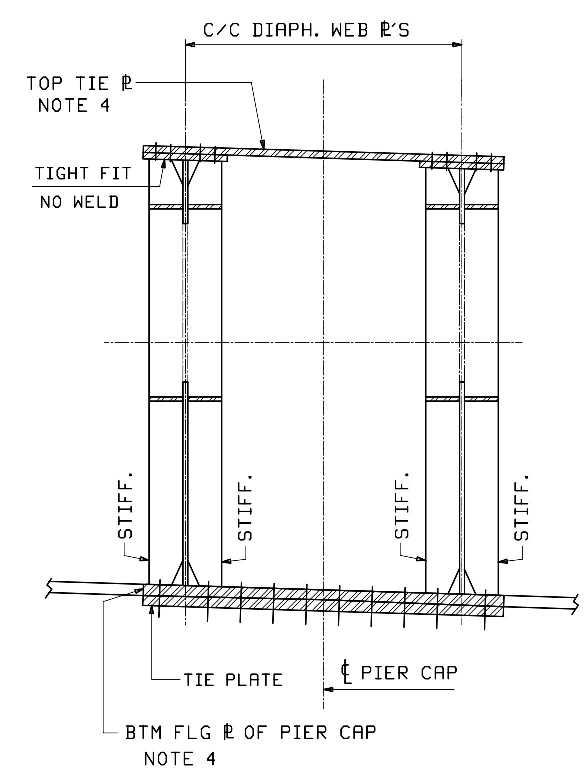
SECTION Q-Q



SECTION R-R



SECTION T-T



SECTION U-U

GENERAL NOTES

1...ALL STRUCTURAL STEEL SHALL BE ASTM A709 UNLESS OTHERWISE NOTED.

2...ALL BOLTED CONNECTIONS SHALL BE SLIP-CRITICAL.

3...TOP FLANGES AND WEBS OF PIER CAP AND DIAPHRAGM ARE FRACTURE CRITICAL AND SHALL MEET THE REQUIREMENTS OF CHAPTER 12 OF ANSI/AASHTO/AWS D1.5 BRIDGE WELDING CODE, FRACTURE CONTROL PLAN (FCP) FOR NONREDUNDANT MEMBERS.

4 ...SLOPE TOP AND BOTTOM FLANGES OF PIER CAP TO BE PARALLEL TO GRADE OF LONGITUDINAL GIRDER. THIS WILL ELIMINATE USING BEVELED FILL PLATES.

CROSS SECTION GEOMETRY

5... ROTATE TUB GIRDER WITH CROSS SLOPE.

6... MAINTAIN CONSTANT TRAPEZOIDAL SHAPE FOR ALL GIDERS ON A STRUCTURE. VARY DISTANCE BETWEEN TUBS IF DECK WIDTH FLARES.

7... MAINTAIN A CONSTANT DIMENSION FROM THE TOP OF DECK TO THE TOP OF THE WEB PLATE. SHOW THIS DIMENSION ON DESIGN DRAWINGS.

8... ALL TRANSVERSE GIRDER LOCATIONS SHALL BE LOCATED HORIZONTALLY. DO NOT CONTROL THE GIRDER LOCATIONS ALONG THE DECK.

9... BEARING STIFFERNERS MAY BE VERTICAL OR NORMAL TO FLANGES.

NOTES FO RDESIGNERS

A... AVOID FULL PENETRATION WELDS OF PLATES OR STIFFENERS TO FLANGES. USE FINISH TO BEAR WITH FILLET WELDS

B... SEE PAGE NO. 117 FOR ADDITIONAL APPLICABLE NOTES FOR ACCESS OPENINGS, BOLT SPACING, EDGE DISTANCES, WELDING , ETC.

C... SEE PAGE NO. 118 FOR DRAIN HOLES NOTES.

DISCLAIMER NOTE

INFORMATION SHOWN IS FOR CONCEPT ONLY. APPLICATION TO SPECIFIC STRUCTURES IS THE DESIGNER'S RESPONSIBILITY.

INTEGRAL PIER CAP STEEL BOX

AASHTO/NSBA STEEL BRIDGE COLLABORATION

TASK GROUP 1, SUBTASK - GROUP 1.4
GUIDELINES FOR DESIGN DETAILS

PAGE NO. 120

AASHTO/NSBA Steel Bridge Collaboration

G 4.2 - 2006

Recommendations for the Qualification of Structural Bolting Inspectors

AASHTO/NSBA Steel Bridge Collaboration

Preface

This document is a standard developed by the AASHTO/NSBA Steel Bridge Collaboration. The primary goal of the Collaboration is to achieve steel bridge design and construction of the highest quality and value through standardization of the design, fabrication, and erection processes. Each standard represents the consensus of a diverse group of professionals. A listing of those serving on the Committee which developed the *Guide Specification for Application of Coating Systems with Zinc-Rich Primers* will be included in future editions.

As consensus documents, the Collaboration standards represent the best available current approach to the processes they cover. It is intended that Owners adopt and implement Collaboration standards in their entirety to facilitate the achievement of standardization. It is understood, however, that local statutes or preferences may prevent full adoption of the document. In such cases Owners should adopt these documents with the exceptions they feel are necessary.

Disclaimer

TABLE OF CONTENTS

1. SCOPE

These guidelines have been developed by the AASHTO/NSBA Steel Bridge Collaboration to define essential factors involved in structural bolting and the qualification of personnel inspecting and monitoring those operations.

This document should be a guide in developing individual training and qualification programs. The recommended elements listed in the Body of Knowledge in Section 6 should be modified to meet specific needs.

2. REFERENCES

The following Industry Standards and Specifications should be used in the development of a compliant qualification and certification program:

- AASHTO/NSBA Steel Bridge Collaboration S2.1– Steel Bridge Fabrication Guide Specification
- AISC Manual of Steel Construction
- RCSC Specification for Structural Joints – Using ASTM A 325 or A 490 Bolts
- ASTM Standard Specifications as they apply to structural bolting: A 325, A 449, A 490, A 563, F 436, F 959, F 1852.
- FHWA-SA-91-031, "High-Strength Bolts for Bridges"

3. FUNCTIONS

3.1. Capabilities. A qualified Structural Bolting Inspector should be able to perform bolting inspections, supervise one or more inspectors, prepare inspection procedures, conduct audits of field bolting conditions and methods, and ensure that bolting operations conform to project requirements and applicable standards.

The Inspector should be able to identify deficiencies in structural bolting, verify rotational capacity testing and observe pre-installation testing.

3.2. Duties. Although the Inspector's duties are defined by each organization, a Bolting Inspector should be able perform at least the following duties:

3.2.1 Interpret drawings and other documents.

3.2.2 Interpret and accept rotational capacity test (ROCAP) reports, material test reports (MTRs) and manufacturers' certificates of compliance.

3.2.3 Verify fastener assemblies for correct components, including bolt head markings and manufacturer and supplier marks, nut markings and structural washer markings, supported by proper documentation.

3.2.4 Verify proper storage conditions.

3.2.5 Verify proper lubrication and surface condition of structural bolts, nuts and washers.

3.2.6 Identify the requirements for washers and ensure adherence.

3.2.7 Verify snug-tight conditions prior to final pretensioning.

3.2.8 Witness performance of pre-installation verification and verify reference marks on components & material before final pretensioning.

3.2.9 Verify the suitability and calibration of equipment used to perform structural bolting activities.

3.2.10 Verify that the knowledge of personnel supervising or performing bolting applications is adequate.

3.2.11 Determine whether bolts may be reused (i.e. loosened and retightened).

3.2.12 Determine and verify required minimum fastener pretension.

3.2.13 Prepare clear, concise reports and verify that pertinent records are maintained.

4. TRAINING/EXPERIENCE REQUIREMENTS

4.1. Recommendations. The Structural Bolting Inspector should:

4.1.1 Be a high school graduate, or hold a state approved high school equivalency diploma.

4.1.2 Possess at least one year experience in structural steel bridge fabrication, erection and/or inspection and be directly involved with structural bolting operations.

4.2. Structured Training. Training and qualification required by this document may be considered satisfied if the candidate Inspector provides documentation for successfully completing a minimum of 8 hours of training offered by an organization or individual recognized by the Owner. Such documentation should include evidence of satisfying the requirements listed in Section 5.

4.3. Experience Credit. Structural Bolting Inspectors not meeting the above experience guidelines should work under the supervision of a qualified Structural Bolting Inspector until such time as the appropriate experience and training is gained.

5. QUALIFICATION

5.1. Requirement. The requirements for qualifying a Structural Bolting Inspector under this Standard are as follows:

5.1.1 The Inspector should pass a written test of no less than 50 multiple choice and/or True/False questions with a minimum of 70% correct, proving a general understanding of the Body of Knowledge specified in Section 6 of this document.

5.1.2 The Inspector should demonstrate hands-on proficiency by completing or directing the completion of rotational capacity testing and pre-installation verification testing using a tension calibration device (e.g. Skidmore-Wilhelm) in the presence of an individual qualified in accordance with this standard.

5.1.3 The Inspector should remain qualified under this Standard provided he/she documents ongoing employment involving structural bolting operations, and performing inspection duties at least once every six months.

5.2. Documentation. The employer of the Inspector should maintain training records (including dates and curricula), completed examinations, evidence of initial demonstrated proficiency, qualification certificates issued, and documentation of inspections performed (dates, testing, types of assemblies).

5.2.1 Qualification certificates should contain the following information:

5.2.1.1 Name of the Inspector.

5.2.1.2 Statement indicating satisfactory completion of training.

5.2.1.3 Statement indicating that the Inspector's qualification is in accordance with this document or an acceptable industry standard.

5.2.1.4 Signature of party responsible for maintaining qualification.

5.2.1.5 Date of qualification.

6. BODY OF KNOWLEDGE

6.1. Bolted Connection Overview. This section delineates code requirements and proper methods for using bolted joints that the Inspector should understand and enforce. The Inspector should also understand the installation and inspection requirements for various types of high strength bolted joints.

6.1.1 Joint Types.

The types of bolted joints commonly used in bridge and highway construction.

6.1.1.1 Slip-Critical

The definition, load transfer method, surface and bolt requirements, typical usage, installation and inspection of slip-critical joints.

6.1.1.2 Snug-Tightened

The definition, installation requirements and inspection of snug tightened joints.

6.1.1.3 Pretensioned

The definition, installation requirements and inspection of pre-tensioned joints.

6.1.2 Load Transfer

The difference between various load transfer mechanisms for HS bolted joints.

6.1.2.1 Shear/Bearing

Bolts subjected to direct shear at bearing connections in bolted splices.

6.1.2.2 Direct Tension

Effects of direct tension on connections for hangers, prying action, etc.

6.1.2.3 Friction

The types of surface conditions required to obtain proper slip coefficients.

6.1.3 Bolt Holes and Slots

The acceptance/rejection quality standards for standard bolt holes, oversized holes and short and long slotted holes including dimensions, reaming, multi-ply alignment, out-of-round, quality (tears, burrs, etc). The acceptance/rejection criteria for bolt spacing and edge distance (see RCSC Table 3.1).

6.1.4 Snug Tightening

The definitions of "snug tight", for both rotational capacity testing and also for installation. Proper systematic tightening to bring an entire assembly to the snug tight condition.

6.1.5 Bolt Pretension

The principles and methods to tighten bolts in pretensioned and slip-critical joints (see RCSC Table 8.1, minimum bolt pretension values)

6.2. Bolting Materials and Usage. Be able to properly identify the bolting materials required for a project, ensure the materials' quality upon receipt and after storage, and select the proper materials and methods for installation.

6.2.1 Bolts

6.2.1.1 Bolt Types

Understand the differences between various types of bolts (hex head, Twist-off [TC] bolts, lock-pin and collar, etc.).

6.2.1.2 Material Grades

The differences and common uses for each grade and type of structural bolt (A325, A354, A490, A449, F1852, Type 1, Type 3, etc), including restricted uses and conditions.

6.2.1.3 Markings, Dimensions, etc.

The significance of head markings on structural bolts as well as common bolt dimensions and terminology (see RCSC Fig. C-2.1).

6.2.2 Nuts

6.2.2.1 Materials, Markings, Dimensions, etc.

The significance of markings on structural grade nuts as well as common nut dimensions and terminology (see RCSC Fig. C-2.1).

6.2.2.2 Suitability with Bolts

The acceptable combinations of nuts and bolts for structural bolting applications (see RCSC Table 2.1, see also ASTM A563 Table X1.1), and the qualification of bolt-nut assemblies.

6.2.3 Washers

6.2.3.1 Material, Dimensions, etc.

The standard materials and sizes of hardened steel washers for structural applications (see ASTM F436), and plate or beveled washers for slots or sloping surfaces.

6.2.3.2 Connection Requirements

The correct usage of hardened steel washers in applications with standard, slotted and oversized holes.

6.2.4 Direct Tension Indicators

6.2.4.1 Material, Dimensions, etc.

The configuration, interpretation and sizes of direct tension indicators (load indicating washers) used with high-strength bolts (see ASTM F959).

6.2.4.2 Connection Requirements

The correct installation and verification of direct tension indicators, including hardened washer requirements based on the turned element, and placement for standard, slotted, and oversized holes.

6.2.5 Bolt Length/Grip

The correct bolt length for given applications, identify stripping, excessive stickout, and bottoming out of nuts.

6.2.6 Reuse of Bolts

The situations in which bolts that have been pretensioned may be reused (black A 325 vs. coated A 325 & A 490) and know how to check for suitability for reuse.

6.2.7 Storage
Proper jobsite storage and protection.

6.2.7.1 Jobsite/Shop Storage
Proper lot segregation and storage procedures and identify and isolate defective items.

6.2.7.2 Lubrication
Proper lubrication of nuts and bolts (black as well as galvanized) and the proper methods and materials are used for re-lubrication (see 6.3.1).

6.2.8 Bolted Splices
The requirements for the use of alignment (drift) pins for splice alignment prior to snug tightening and pretensioning.

6.3. Inspection. Know and understand the specific inspection requirements as they apply to the following below listed installation methods.

6.3.1 Prior to Installation. How to identify the fasteners (i.e. grade, etc.), proper storage and lubrication, and know and perform proper pre-installation verification.

6.3.2 Rotational Capacity Testing (ROCAP). Witness the performance of rotational capacity testing for each bolt-nut-washer assembly lot, verifying the effectiveness of nut lubrication and the ability of bolts to resist stripping.

6.3.3 Snug Tight. Know and understand the definition of snug tight and systematic tightening, know how to identify a snug tightened condition and know to verify this condition prior to pretensioning.

6.3.4 After Snugging. The Inspector should know proper snug-tightening procedures and results, know how to apply systematic tightening and know bolt and nut match-marking.

6.3.5 During Pretensioning. The Inspector should know correct observation techniques for witnessing turning of bolts both with and without match-marking and know how to address loose bolts.

6.3.6 After Pretensioning. The Inspector should be able to inspect and verify match-marking and that proper torque has been applied (proper use of torque wrench) and should know the parameters for reuse of previously pretensioned bolts.

6.4. Installation Methods. Verify and inspect for various common fastener installation techniques.

6.4.1 Turn-of-Nut Installation. Be familiar with all critical aspects of the turn-of-nut installation technique.

6.4.1.1 Pre-Installation Verification.
Observe pre-installation verification and demonstrate the proper use of a tension calibration device (e.g. Skidmore-Wilhelm) for long bolts as well as the procedure for short-bolt testing. Determine the check torque for installed bolts.

6.4.1.2 Joint Pretensioning

Verify snug tight condition, know the match-mark system used on nuts & bolts and know the required pretension for the bolts used.

6.4.2 Twist-Off-Type "Tension-Control" Bolts Understand all aspects of the installation technique for twist-off type "tension-control" bolts (Torsion actually governs twist-off, but it's related to fastener tension by test results.).

6.4.2.1 Pre-Installation Verification

Verify proper tension at twist-off using a tension calibration device (e.g. Skidmore-Wilhelm) for long bolts or for short-bolt testing.

6.4.2.2 Joint Pretensioning

Verify the snug tight condition is achieved before any twist-offs occur and understand the need for systematic tightening.

6.4.2.3 Specific Inspection Aspects.

6.4.2.3.1.After Snugging

Verify the snug tight condition, with the twist-off spline intact, and verify the bolt grip is correct.

6.4.2.3.2.During Pretensioning

Observe and verify the correct operation of the wrench and determine that all bolts remain tight as pretensioning continues.

6.4.2.3.3.After Pretensioning

Observe and verify that all twist-off splines are sheared and apply check-torque to random bolts.

6.4.3 Direct Tension Indicators (Load Indicating Washers). Understand the installation of high-strength bolts using direct tension indicators (DTIs), also known as load indicating washers.

6.4.3.1 Pre-Installation Verification

Using a tension calibration device (e.g. Skidmore-Wilhelm), verify DTIs properly indicate pretension by closing gaps or other methods.

6.4.3.2 Joint Pretensioning

Verify proper bolt pretension by visual verification or by the use of a feeler gauge.

6.4.4 Calibrated Wrench Installation. Understand the installation of high-strength bolts using a calibrated torque wrench.

6.4.4.1 Pre-Installation Verification

Using a tension calibration device (e.g. Skidmore-Wilhelm) for long bolts or direct tension indicators for short-bolt testing, ascertain the torque required to fully pretension a bolt/nut from a snug condition. Know how to set the calibrated wrench to indicate when the correct torque is applied for both installation and also for post-installation check-torques.

6.4.4.2 Joint Pretensioning

Verify snug tight condition, know systematic tightening pattern used on nuts & bolts and be able to set the wrench properly for both installation and check torque values.

6.5. Time Management. It is not possible to closely monitor multiple bolting crews. Be aware of crew schedules and tell-tale indications of good or bad crew technique to determine how often to check each operation for efficient inspection.

6.6 Arbitration of Disputes. Be familiar with the correct procedures for arbitrating disputes (see RCSC Section 10).

AASHTO/NSBA Steel Bridge Collaboration

G 4.4 - 2006

Sample Owners
Quality Assurance Manual

AASHTO/NSBA Steel Bridge Collaboration

Preface

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Disclaimer

All data, specifications, suggested practices presented herein, are based on the best available information and delineated in accordance with recognized professional engineering principles and practices, and are published for general information only. Procedures and products, suggested or discussed, should not be used without first securing competent advice respecting their suitability for any given application.

Publication of the material herein is not to be construed as a warranty on the part of the American Association of State Highway and Transportation Officials (AASHTO) or the National Steel Bridge Alliance (NSBA) - or that of any person named herein - that these data and suggested practices are suitable for any general or particular use, or of freedom from infringement on any patent or patents. Further, any use of these data or suggested practices can only be made with the understanding that neither AASHTO nor NSBA makes any warranty of any kind respecting such use and the user assumes all liability arising therefrom.

Sample Quality Assurance Manual
Structural Steel Shop Inspection

Name of Organization

Address Information

Date: ___________________

Revision No: ___________________

Revision Date: ___________________

Table of Contents

(State of ___ Department of Transportation)
QUALITY ASSURANCE MANUAL

SECTION 1
SCOPE

This document serves as a sample quality assurance manual to be used as a guide for Bridge Owners or third party inspection agencies to develop their own quality assurance plans. This sample manual is based on the guidelines in AASHTO Document No: SBFQC-1 (AASHTO/NSBA Steel Bridge Collaboration Specification S4.1-2002) *Steel Bridge Fabrication QC/QA Guide Specification.* As written, this sample manual is intended to be used on projects where the AASHTO Document No: SBFQC-1 is being used for the quality control guidelines.

As a sample manual, this is intended to serve as a model, or base document, for creating a QA manual. When using this document to create a QA manual, care should be exercised to determine if additional sections or instructions are necessary. Similarly, if any sections are not applicable, these should be removed or altered as necessary.

Refer to AASHTO Document No: SBFQC-1 for a listing of applicable terms and definitions.

SECTION 2
SPECIFICATIONS AND DOCUMENTS

<<<The intent of this section is to recommend the documents that the inspector should be furnished with or acquire during the course of a project. This list should be modified as necessary to suit the owner's specific requirements >>>

2.1 Quality Assurance Inspectors (QAIs) shall obtain, or have available the following documentation at the fabrication shop:

- applicable current standard specifications, supplements, special provisions, special specifications, and addenda
- approved shop drawings with current revisions
- fabricator's Quality Control Plan (QCP), which is to include the company's Non-Destructive Evaluation (NDE) written practice.
- prefabrication meeting minutes, if any
- applicable AASHTO/AWS D1.5, Bridge Welding Code
- applicable American Welding Society (AWS) D1.1, *Structural Welding Code*
- applicable provisions of the AREMA Manual for Railway Engineering, if required for the project
- AWS A2.4, *Symbols for Welding and Nondestructive Testing*
- AWS A3.0, *Standard Welding Terms and Definitions*
- applicable ASTM or AASHTO specifications
- applicable coating test methods
- applicable SSPC Specifications
- metrication conversion tables, if required.
- Mill Test Reports (MTRs) for material used in fabrication
- list of qualified welders, welding operators, and tack welders
- approved welding procedure specifications (WPSs)
- approved welding procedure qualification records (PQRs)
- applicable pre-approved non-critical fracture critical material (FCM) repair procedures
- applicable approved repair procedures
- copy of approved nondestructive testing procedures, only when submission and approval of these testing procedures is required by the owners specification
- qualification documents for all certified welding inspectors (CWI) and NDE quality control (QC) personnel
- NDE reports for all work on this project that has been inspected and accepted by NDE
- project record log sheets.
- Copy of the approved coatings plan, when submission and approval of coatings plan is required by specification.

2.2 Familiarization with Requirements

The QAI shall become familiar with applicable portions of the Contract documents covering the work to be inspected. The QAI shall study the plans and specifications before fabrication commences to provide ample opportunity to coordinate with the Owner.

2.3 Use of Shop Drawings

The QAI should become familiar with the shop drawings. The QAI shall coordinate with the Owner regarding any discrepancies between the plans and specifications and the shop drawings. Fabrication should proceed only with approved shop drawings. However, if the Fabricator elects to proceed prior to receipt of approved shop drawings (performing work at their own risk), notify the Owner and, if directed proceed with QA functions using non-approved shop drawings. Later, verify conformance of fabrication with the approved drawings. The shop must submit revisions to the shop drawings to the Owner for approval to reflect changes in details and provide a permanent record of the as-built condition. The QAI should ensure that fabrication is in conformance with the latest revisions.

SECTION 3
INSPECTION EQUIPMENT

<<<The intent of this section is to recommend those tools and equipment necessary for proper inspection. . This list should be modified as necessary to suit the owner's specific requirements >>>

The QAI shall have/or have available the following equipment at the fabrication shop. Some equipment may not be applicable depending on the nature of fabrication:

- tape measure, 25 ft. (7 m), 1/32" or 1-mm increments.
- metal tape measure, 100 ft. (30 m), 1/8-inch or 0.01-ft (5-mm) increments
- pocket metal ruler(s), 1/32" or 1-mm increments
- flashlight and spare batteries
- camera; QAI to photograph only raw and fabricated materials produced for the owner.
- feeler gauges
- fillet weld gauges
- undercut gauge
- skewed fillet weld gauge
- bevel gauge
- micrometer
- mirror for examining restricted access areas (such as snipes)
- NDE tools, if applicable
- surface roughness gauges for machine and flame cutting
- (Ref. ANSI B41 or AWS C4.1-G)
- temperature-indicating crayons for 30°F (15°C) above and below desired temperatures or surface pyrometer
- sling psychrometer
- thermometers for determining air, paint and metal surface temperatures
- blast profile comparator or replica tape for direct measurements and a permanent record
- dry film thickness gauge
- tools for checking surface anomalies, coating adhesion, etc.
- surface profile comparator for media (sand, shot or grit) used and/or deformable replica
- tape and micrometer to check profile depth before coating.
- 10X Lens

SECTION 4
INSPECTOR QUALIFICATIONS

<<<The intent of this section is to specify the minimum recommended qualification and training requirements for QAIs-the basis for which is given in S4.1 . This list should be modified as necessary to suit the owner's specific requirements(these could include bolting training, etc) >>>

QAIs shall have the following minimum required knowledge, abilities, and experience:

4.1 Fabrication Inspection Qualifications.

A QAI performing welding inspection must be a Certified Welding Inspector (CWI) or equivalent, in accordance with the Bridge Welding Code. QAIs who are Certified Associate Welding Inspectors (CAWI) may work under the direct supervision of a CWI. QAIs who interpret and perform NDE must be certified in accordance with the applicable ASNT SNT-TC-1A requirements for each NDE method being used in accordance with the Bridge Welding Code.

4.2 Minimum Inspection Experience.

A QAI performing welding inspection should have the following minimum inspection experience:

Project Type	**Minimum Recommended Years of Experience***
Rolled beam bridges	1 year
Welded plate girders (I sections, box sections, etc.)	2 years
Complex structures, such as trusses, arches, cable-stayed bridges, and moveable bridges	3 years
Fracture critical (FC) members	3 years (required by D1.5)

* Experience in rolled beam bridge inspection will not be counted towards the experience needed for plate girders, complex structures, or fracture-critical members.

Inspectors who have less experience than that specified above should work under the guidance of an inspector having those qualifications. QAIs must be proficient with the typical fabrication inspection procedures described in this document.

4.3 Coatings Inspection Qualifications.

A QAI performing coatings inspection must be qualified to inspect coatings and coatings applications.

4.4 Documented training in materials preparation, coatings application, and inspection is suggested for the QC and QA coatings inspectors. Recommended training includes one or more of the following:

- American Institute of Steel Construction (AISC) – Application and Inspection of Sophisticated Coatings
- National Association of Corrosion Engineers (NACE) - International Coating Inspector training and Certification Program Session I: Coating Inspection Training
- Society for Protective Coatings (SSPC) - C-1 Fundamentals of Protective Coatings for Industrial Structures
- National Highway Institute (NHI) - Bridge Coating Inspection Course No. 13079
- Other training programs that are considered acceptable by the Engineer

SECTION 5
RECORDS AND REPORTING

<<<The intent of this section is to recommend the records and reporting required to be kept by QAIs. This section should be modified as necessary to suit the owner's specific requirements >>>

The QAI shall maintain neat and orderly records for each project. Documentation of the status of fabrication and acceptability of members shall be performed on the forms included in Appendix A. *<<<note: specific form sand directions for completing shall be inserted by owner >>>*

In addition to completion of the necessary forms, the QAI shall maintain a narrative report for each project, as directed by the Engineer. The narrative report should either be legibly hand-written in a permanently bound book, or be maintained in an electronic log with automatic date and time recording. Record the Fabricator's activity on the work inspected, including both positive and negative comments, information provided to the Fabricator and any agreements made. Make entries as soon possible after the events or conversations.

Obtain copies of Fabricator generated records such as NDE reports, inspection reports, etc. for the project files and submission to the Engineer as directed.

Furnish a written report on a weekly basis to the Engineer as directed. Include those forms and documents required by the Engineer. Number the reports consecutively until completion of the work, with the last report noted "final".

Make notes, letters, faxes, reports, and memoranda clear and brief, and keep them on file. Sign on-site correspondence as its originator.

The QAI should maintain verbal communication with the Engineer or his representative as directed. The QAI shall also maintain good relations with the Fabricator's Quality Control Inspector (QCI). Ensure that both the Engineer and QCI are timely apprised of the QAI's findings including nonconformances. Written documentation is not a substitute for appropriate dialogue with the Fabricator, but should provide a record of important discussions.

SECTION 6
GENERAL INSTRUCTIONS

<<<The intent of this section is to recommend the general requirements for QAI activities. This list should be modified as necessary to suit the owner's specific requirements >>>

6.1 Responsibilities of the QAI

Verify that production quality and fabrication processes satisfy contract requirements, including the QCP. Verify that the fabricator has QCI performing inspection functions during all fabrication operations. Perform QA inspections in accordance with this manual and other instructions by the Engineer. Determine extent and frequency of inspection based on the Engineer's direction.

Accept materials that satisfy the Contract requirements. Do not waive items that are contractual obligations of the Fabricator and do not accept material that does not conform to the Contract requirements without the written approval of the Engineer.

Do not direct the Fabricator's personnel. Do not provide suggestions on how to fabricate material. However, the QAI should advise the Fabricator if any operation would, in their opinion, result in noncompliance with the Contract. Direct all official communications to the Fabricator's quality control. Do not convey directives or personal judgments about overall shop quality or concerns about employee competence to production personnel.

Do not divulge a fabricator's proprietary information to another fabricator. Do not publish, copy or distribute any proprietary information, documents, or forms received from the Fabricator for any purpose other than the contractual needs of the Owner.

6.2 Role of the QAI

Perform verification tests, measurements, inspection, or observations to assure that fabricated items conform to the Contract requirements. Although the QAI does not perform QC work, some QA activities may duplicate a portion of QC activity for verification.

If there are questions about a requirement or level of quality, contact the Engineer and, if directed, alert the Fabricator.

Conduct consistent inspections based on the Contract requirements while providing guidance to the Fabricator concerning interpretation of the plan details and specification mandates. Obtain assistance from the Engineer as needed.

Be familiar with the QCP to better understand the QC operations of the shop. Verify that the shop is conducting operations in accordance with their QCP.

6.3 Interaction with the Fabricator Quality Control Inspector

Verify the effectiveness of the QCI's evaluation of the work.

Perform verification inspection after the QCI has completed inspection and testing in accordance with the QCP. However, serious problems noted at any time or stage of fabrication must be immediately pointed out to the QCI. Notify the Engineer if there are any unresolved problems.

Though QA inspection may include all aspects of fabrication, the QAI must not supersede QC, which is the responsibility of the Fabricator. If QC is not accomplishing its role, the Engineer and Fabricator must determine the necessary corrections.

6.4 Interaction with the Engineer

If the Fabricator's inquiries involve design questions, material substitutions, alternate fabrication methods, or items that are beyond the authority of the QAI, refer them to the Engineer.

6.5 Interpretation of the Contract

Review Contract requirements. If conflicts arise regarding their interpretation or adequacy, seek guidance from the Engineer. Inform the QCI of the results of this discussion.

6.6 Fabrication Observation

Establish a proactive pattern of regular and frequent observations during the progress of work to verify satisfactory workmanship without delaying production or missing critical operations.

Coordinate verifications with the QCI and accomplish them with minimal additional material handling by the Fabricator and with as little interference with the work in process as possible.

Though there are not designated points during fabrication when the suitability of materials must be checked, problems should be discovered and addressed as early as possible.

6.7 Nonconforming Materials and Workmanship

A nonconformance is defined as a fabrication error or alteration in the work that does not meet project specifications. Some minor nonconformances can be remedied as provided for in the welding code and project specifications. A typical example of this would include cosmetic weld repairs. Other nonconformances may be more serious and cannot be remedied through simple repair as allowed in the applicable welding code. These types of nonconformances render the affected component unacceptable until such time as the issue is referred to the Engineer for disposition. Typical examples of these nonconformances include, but are not limited to, mis-located holes, incorrect material, and final dimensions not in accordance with approved drawings, un-authorized welds, welding without approved welding procedures, and overheating of members. When in doubt regarding the proper disposition method, the QAI should obtain clarification from the Engineer/Owner as necessary.

Bring all nonconformance issues to the attention of the Fabricator immediately upon discovery. However, do not direct corrective action. If the Fabricator fails to take corrective action, or continues to operate in an unacceptable manner, immediately notify the Engineer. Verbal notification of nonconformance issues to the fabricator is sometimes sufficient; however serious specification noncompliance issues should always be conveyed in writing to the fabricator and the Engineer.

For significant problems, the Fabricator must submit a written proposal concerning the issue, providing documentation of the situation and proposed actions to address the issue. The Fabricator may write directly to the Engineer, Contractor, or both, as directed, and in all cases send a copy to the QAI.

When the Engineer’s approval is required for a repair, the inspector shall review and confirm the Fabricator’s proposed methods of repair and description of the existing material conditions. Seek guidance from the Engineer for clarification when necessary. Follow up to verify that all required corrections and applicable NDE have been accomplished. All nonconformances shall be properly resolved before the members can be considered for final acceptance.

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SECTION 7
INSPECTION PROCEDURES

<<<The intent of this section is to specify the inspection requirements for QAI activities. This list should be modified as necessary to suit the owner's specific requirements >>>

7.1 Mill Test Reports

7.1.1 Verify use of proper materials by reviewing a copy of the MTRs when the material arrives and by monitoring heat numbers during fabrication until the material is joined into a piece-marked item.

7.1.2 Obtain MTRs from the Fabricator in accordance with the Department's customary practice, including the number of copies and when and to whom MTRs should be submitted.

7.1.3 Verify the following information on MTRs:
- product description (specifications, grade, H or P testing frequency)
- chemistry
- physical test results, including Charpy V-Notch when applicable
- applicable "Buy America" certification requirements
- heat number
- certification signature (Quality Control Department and Notary, when required)

7.1.4 Do not accept material if the Fabricator cannot furnish appropriate certifications to establish compliance with the required material properties and "Buy America" requirements, if required in the Contract.

7.1.5 Maintain a record of heat number identification for main members.

7.1.6 Accept structural steel based on MTRs. Miscellaneous hardware or other associated products may be accepted based on certifications of compliance.

7.1.7 If the Department requires additional independent physical and/or chemical tests of the material's properties, then these tests must be performed as soon as practical, and prior to fabrication. If the independent tests indicate noncompliance, do not allow use of the material unless an agreement is reached between the Department and the Fabricator as to its acceptability. Bring such noncompliance to the attention of the QCI for evaluation and disposition.

7.1.8 Verify compliance of MTRs with the requirements of the relevant ASTM or AASHTO specification.

7.2 Inspection of Raw Materials

7.2.1 Verify that the requirements of ASTM A 6 (AASHTO M 160) or ASTM A20 as applicable, which cover the common requirements for hot-rolled plates, shapes, sheet piling and bars, are applied, as applicable, for material acceptance inspection and repairing certain surface defects.

7.2.2 Check materials for surface defects and discontinuities, both initially and as material is being worked. Check rolled sections and steel castings for dimensions, straightness, twist, fins, scabs, and rolling defects, prior to fabrication.

7.2.3 Grade of material shall be in accordance with the shop dwg./project requirements. No unauthorized substitutions of material (size or grade) are allowed without the Engineer's approval.

7.3 Material Cutting Inspection

7.3.1 Check that methods employed for material cutting are allowed by the contract documents specifications. Monitor steel plate during cutting for internal defects or other problems. Check that cutting methods do not produce unacceptable gouges or surface roughness. Check that internal defects or gouges are evaluated and repaired in accordance with the applicable code requirements. Verify that heat numbers are being transferred to cut members

7.4 Fit-up and Welding Inspection

7.4.1 Review the consumable manufacturer's certificate of conformance maintained by the Fabricator for all consumables used. If required by the Department, obtain copies for the project file.

7.4.2 If the Department maintains a list of approved electrodes, verify that all consumables used by the Fabricator are on the list.

7.4.3 Monitor these criteria before welding begins:

- Approved Shop drawings clearly indicate the details of welded joints by welding symbols or sketches. Missing or inappropriate weld details are unacceptable and shall be referred to the Fabricator and Engineer for disposition. Approved drawings must be corrected and approved prior to final acceptance.

- appropriate equipment in acceptable condition and periodically calibrated per QCP
- proper functioning of drying and baking ovens
- All welders, welding operators, and tack welders are qualified in accordance with the contract documents.
- Appropriate welding procedure specifications (WPS) for all detailed joints have been submitted and approved by the Engineer

7.4.4 Monitor these consumable-handling criteria (randomly audit):
- storage, condition, and exposure times of welding consumables
- re-drying and recycling limits.

7.4.5 Monitor these criteria during welding operations (randomly audit):
- joint details, including root face and opening, bevel angle, and alignment of parts are within appropriate welding code and WPS tolerances
- proper application of extension tabs (run-on and run-off)
- cleanliness of surfaces to be welded
- proper condition and storage of welding consumables
- size, quality and location of tack welds.
- the following of approved welding procedure specifications (WPSs) including amperage, voltage, speed of travel, electrode extension, shielding gas flow rate, and preheat, interpass, and/or post-heat temperatures within applicable welding code and WPS tolerances
- workmanship of individual welders
- use of proper repair procedures for fabrication errors, including, when required, the Engineer’s approval
- weld starts and stops, securing and removing run-on and run-off tabs, stopping short of snipes or plate edges, and ending without craters
- For stud welding, ensure test studs are being performed and materials are acceptable

7.4.6 Monitor these final weld quality criteria:
- size, profile, and contour of fillet and groove welds
- defects in welds or parent metal only as permitted by contract documents code/specifications
- accurate interpretation by QCIs for the acceptance or rejection of welds
- cleaning and backgouging of welds, including thorough removal of unsound metal and gouging contamination (copper, carbon). Check proper profile of back-gouged weld for compliance.

- Overgrinding of weld or adjacent base metal areas so as to reduce material/weld throat.
- Stud welds exhibit full 360 degree flash or arc welded studs are visually acceptable

7.5 Nondestructive Testing

7.5.1 Review and approve the personnel qualification documentation of those performing NDE for the Fabricator. Assure that all NDE is being scheduled by QCI so that QAI witnessing of the NDE operation is possible.

7.5.2 Periodically witness NDE, review the test results, and verify that reports are complete and legible and completed in a timely manner.

7.5.3 For radiographic testing (RT), conduct the following activities:

- Interpret test results in accordance with the contract documents.
- Verify that final edges may be properly interpreted. (If the plate will be cut after RT, the final edge may be within the plate on the RT film.)
- Verify proper application of edge blocks if necessary (plate edge is final edge in structure).
- Verify that each radiograph represents a unique section or piece by comparing punch marks or other approved methods of marking the work and corresponding marks on the film.
- Verify that the entire specified area is tested.

7.5.4 Periodically observe or conduct, if necessary, ultrasonic testing (UT) to verify the Fabricator's NDE results. The QAI will determine intervals for observation of verification testing unless otherwise directed by the Engineer.

- Verify the calibration of equipment, including horizontal and vertical linearity checks.

7.5.5 For magnetic particle testing (MT), conduct the following activities:

- Periodically observe the application and interpret the results of MT performed on primary members to verify that they satisfy *Bridge Welding Code* requirements.

- On ancillary, secondary or miscellaneous items, periodically observe and interpret MT when required by the contract documents.

- Observe MT when needed to verify visual findings.

- Observe and interpret MT applied to evaluate removal of defects and welded shop repairs for base metal and deficient welds.

- Assure E 709 compliance including assurance that proper lighting levels are maintained during testing.

7.5.6 For liquid penetrant testing (PT), periodically observe technique and interpret results.

7.6 General Visual Inspection

During fabrication the QAI should monitor and spot-check that the work performed by the fabricator meets the contract requirements, including, as a minimum, the following:
- straightness
- no unauthorized corrections made by welding or manual thermal cutting
- size and quality of punches and dies
- proper setup and securing of drilling or reaming templates
- bolt hole location, edge distance, and diameter
- cylindrical and perpendicular bolt holes
- absence of burrs, tears, and chips in bolt holes
- thickness of plates, clearances, fitup accuracy, alignment of holes, and proper size of sections at field connections
- shop assembly of girders or other parts required for reaming or drilling of field splice holes: positioning, securing, match marking members and splice plates, splice plate orientation (flange splice plates' rolling direction parallel to flanges), fills in assembly, and all plies in contact when assembled
- flatness of flanges at bearing areas
- bearing plates and bearing assemblies, including rockers and shoes for structural steel and expansion joints
- proper surface finish and protection of machined surfaces
- contact condition of milled bearing surfaces
- camber blocking during girder assembly, prior to drilling and QAI acceptance for disassembly
- records of final sweep or camber
- inspection and installation of fasteners in the shop

- location of stiffeners and connection plates
- match-marking of assembled members
- preparation of match-mark diagrams
- control and use of heat and/or pressure to obtain or correct sweep and camber in accordance with shop's QCP, and avoidance of buckles, twists, kinks or other defects
- legibility and position of erection and shipping marks
- no unacceptable twists, bends, kinks, or sweep in finished members
- proper number of pieces
- small parts properly packaged or otherwise secured against loss or damage in transit
- loose pieces fastened in place for shipment
- application of rust-preventive material when required, and covering to prevent contamination of painted surfaces.
- No cutting apart of welded members without the Engineers approval.

7.7 Dimensional Inspection

7.7.1 Periodically observe laydowns and shop assembly.

7.7.2 Verify the Fabricator's geometry control methods and measurements. For full or partial shop assemblies, receive the QCI's signed reports of measurements for the Department's records. If requested by the Department, photographs should also be included in the QAI's report.

7.8 Bolting Inspection

7.8.1 Verify the acceptability of fastener components by reviewing MTRs and test reports (including rotational capacity test reports)

7.8.1 If fasteners to be tested by the Department are sampled at the Fabricator's facility, witness and document sampling of components for fastener assemblies in accordance with the Department's practice.

7.8.2 Verify that all fasteners are properly stored and segregated.

7.8.3 When fasteners are installed in the shop, ensure that installation and verification testing procedures are properly followed.

7.8.4 Witness the rotational capacity and verification testing for shop-installed high-strength fasteners.

7.8.5 Assure current calibration and functionality of torque wrenches and bolt tension-indicating devices.

7.9 Coating Inspection

7.9.1 When shop sampling is performed:

- Coordinate coating sampling in the shop with the Fabricator.
- Conduct sampling as early as possible.
- Witness sampling, including mixing or stirring if required for uniformity.
- Ensure that the required samples are delivered to the Owner in suitable containers.

7.9.2 If sampling is not required, check the owner-maintained list of pre-tested, pre-approved coatings to ensure that the actual batches or lots of the paint to be used are acceptable.

7.9.3 If the paint manufacturer sends coating samples directly to the Department for testing and the batches are approved prior to shipment to the Fabricator or jobsite, verify that the batch numbers received correspond to the approved list, and, if applicable, that approval stamps are present.

7.9.4 Prior to coating application, verify the following:

- Coating containers are properly marked with a batch number.
- Batches have been properly strained and mixed (note when pot life initiates).

7.9.5 When sampling is required, do not accept coated girders until the Department's lab accepts the coating.

7.9.6 For coating application inspection, verify the following:

- The shop is checking and documenting environmental conditions and coating is being applied and cured within acceptable conditions.
- proper cleaning and surface preparation of base metal prior to coating application of coating in accordance with manufacturer and/or contract requirements
- adequate curing of each coat as demonstrated by the prescribed test and, when multi-coat systems are used, prior to the application of subsequent coats
- thickness of coating, wet or dry, as specified for each system and type
- sufficient drying of coating prior to loading for shipment
- absence of dry spray, runs, sags and other defects
- proper coating of inaccessible and limited access areas
- proper treatment of faying surfaces.

7.10 Fracture Critical Members

7.10.1 When fabrication is to occur on members designated as Fracture Critical Members (FCMs), the QAI shall become familiar with the requirements of the AASHTO/AWS Fracture Control Plan (Chapter 12 of the Bridge Welding Code).

7.10.2 Check that the base metal complies with the additional requirements for FCMs including fine-grain practice, prohibition of mill repairs, and toughness requirements.

7.10.3 Check that the Fabricator has complied with the more stringent purchasing, storing, and handling requirements for consumables as required by the Fracture Control Plan.

7.10.4 Check that the fabricator complies with the additional fabrication requirements of the Fracture Control Plan including preheating requirements, tack welding limitations, and straightening/cambering/curving requirements.

7.10.5 Confirm that repair welding conforms to the Fracture Control Plan. “Noncritical Repairs” may be preapproved by the Engineer. “Critical Repairs” shall be approved by the Engineer prior to beginning the repair and shall be documented giving details of the type of discontinuity, location, and extent of repair. Verify that all discontinuities to be repaired are covered by the repair procedure. All repair welding shall be monitored and inspected by the QCI and the QAI.

7.10.6 Confirm that the repair is properly made in accordance with the approved repair procedure including additional requirements such as proper preheat, postheat, and nondestructive testing.

7.10.7 Verify that the Fabricator’s NDT Level III is certified by ASNT.

SECTION 8
MATERIAL STORAGE

<<<The intent of this section is to recommend the activities required by the QAI as related to Material Storage during the course of a project. This list should be modified as necessary to suit the owner's specific requirements >>>

8.1 The project schedule may require that completed members be stored at the fabrication shop or other location for a period of time before shipment to the jobsite. Check that the completed members are stored in a manner that will not cause distortion or damage. Check that lifting devices do not damage the material or the coating. The fabricator is responsible to repair any storage damage prior to shipment to the project.

SECTION 9
LOADING AND SHIPPING

<<<The intent of this section is to recommend activities required by the QAI as related to Loading of material during the course of a project. This list should be modified as necessary to suit the owner's specific requirements >>>

9.1 When all work is complete, conduct a final visual examination of the work.

9.2 The QCI will provide copies of reports covering the materials to be shipped. Verify that all data are correct.

9.3 Randomly observe handling and loading of the work to verify that the methods and supports used will prevent significant damage during shipping. Check that damage to coatings during the storage and loading process are properly repaired as appropriate.

SECTION 10
FINAL ACCEPTANCE

<<<The intent of this section is to recommend the requirements of the QAI during the final acceptance of material during the course of a project. This list should be modified as necessary to suit the owner's specific requirements >>>

10.1 When fabrication is complete and the inspection results demonstrate that all contract documents have been satisfied, the materials are conditionally accepted. Confirm that all nonconformances have been properly resolved.

10.2 Affix the approval stamp or shipping tag on the fabricated piece (or a group of parts bundled or contained together), if required, during preparation of a member/component for shipping, indicating that a representative of the Owner has inspected and accepted the work. Presence of this stamp does not relieve the Contractor of responsibility for proper loading, shipping, final fit, and acceptable final condition of the member or component.

10.3 If required, notify jobsite personnel or the owner of the shipment. Submit final report to owner as directed.

10.4 If the fabricator elects to ship material that does not meet the contract documents or has unresolved nonconformances, immediately contact the Engineer to notify. Do not affix an approval stamp or tag to the fabricated component(s).

10.5 If the material is being shipped to a secondary processor such as a galvanizer or coater, then final acceptance/stamping may occur at that location. Coordinate with the Engineer to confirm the method for inspection at the secondary location.

APPENDIX A
INSPECTION FORMS

<<<The intent of this section is to include the inspection forms routinely required during the course of a project. This list should be modified as necessary to suit the owner's specific requirements >>>

Bearing Status Record

Fabricator Name:

Contract Number	Project Number

Type								
Piece Mark								
Approved Drawing								
	Init.	Date	Init.	Date	Init.	Date	Init.	Date
Approved Welding Procedure								
Approved Welders								
Mill Certs								
Cutting								
Machining Complete								
Welding Complete								
Sample Test PTFE, Fabric Pad, Rubber								
NDT Complete								
Blast Clean								
Paint Complete								
Friction/Proof Load Test								
Final Dimensions Check								
Shipping								
Weight (lb)								

Remarks:

:

Expansion Dams Status Record Form

Fabricator Name:

Contract Number	Project Number

Type											
Piece Mark											
Approved Drawings											
		Init.	Date	Init.	Date	Init.	Date	Init.	Date	Init.	Date
Welders Qualifications											
Approved Welding Procedure											
Mill Certs											
Cutting	Angles										
	Divisor										
	Pedestals										
	Curb Parapet										
Welding Complete											
Stud Welding Complete											
Bend Test											
Dimension Check	Length										
	Elevation										
	Lug Depth										
	Pedestals										
	Contours										
Seal	Certs										
	Insp.(Car Seal #)										
	Sample										
	Approved										
Adhesive	Sample										
	Approved										
Blast Clean											
Installation of Seal											
Depth of Seal											
Joint Opening											
Cleaning of Seal											
NDT	Dye Penetrant										
	Magnetic Particle										
	Ultrasonic										
Painting Complete											
Galvanizing											
Final Approval											
Shipping											
Weight (lb)											

Remarks:

Fabrication Status Record For Secondary Items & Misc.

Fabricator Name:

Contract Number	Project Number

Item								
Type								
Piece Mark								
Quantity								
Approved Drawing								
	Init.	Date	Init.	Date	Init.	Date	Init.	Date
Mill Certs								
Material Cut								
Holes Drilled								
Assembly/Weld Complete								
Blast Clean								
Paint / Galvanizing Complete								
Final Check								
Shipping								
Weight (lb)								

Remarks:

Girder Fabrication Project Status Record

Fabricator Name:

Piece Mark										
Approved Drawing										
Welders Qualifications										
Approved Weld Procedure										
Mill Certs										
	Init.	Date	Init.	Date	Init.	Date	Init.	Date	Init.	Date
Top Flange Cut										
Top Flange Welded										
Top Flange RT										
Bottom Flange Cut										
Bottom Flange Welded										
Bottom Flange RT										
Web Cut										
Web Welded										
Web RT										
Web to Flange Fit										
Web to Flange Welded										
Stiffeners Fit										
Stiffeners Welded										
Fit Others										
Weld Others										
Weld Dimensions Checked										
Bearing Area Flatness										
NDT – **Magnetic Particle**										
NDT – **Ultrasonic**										
Length Check										
Web Flatness										
Flange Tilt										
Camber Check										
Sweep Check										
Blast Clean										
Pick up Repair's										
Paint – **1st Coat**										
Paint – **2nd Coat**										
Paint – **3rd Coat**										
Final Approval										
Shipment										
Weight (lb)										

Remarks:

Steel Sign Structure - Pole Project Status Record

Fabricator Name:

Contract Number	Project Number

Item										
Type										
Piece Mark										
Approved Drawing										
	Init.	Date	Init.	Date	Init.	Date	Init.	Date	Init.	Date
Welders Qualifications										
Approved Weld Procedure										
Mill Certs										
Cutting										
Drilling										
Welding Complete										
Weld Dimensions Checked										
NDT – **Dye Penetrant**										
NDT – **Magnetic Particle**										
NDT – **Radiographic**										
NDT – **Ultrasonic**										
Galvanizing										
Storage										
Final Approval										
Shipping										
Weight (lb)										

Remarks:

Stringer Fabrication Project Status Record

Fabricator Name:

Contract Number	Project Number

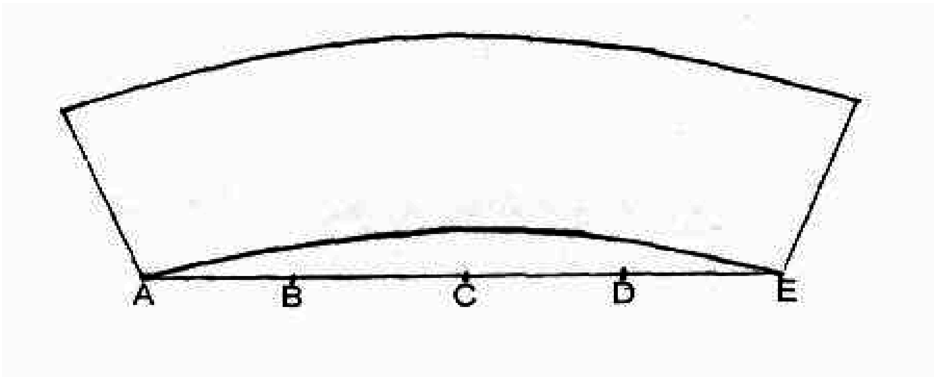
Piece Mark										
Approved Drawing										
Welders Qualifications										
Approved Weld Procedure										
Mill Certs										
	Init.	Date	Init.	Date	Init.	Date	Init.	Date	Init.	Date
Heat Camber										
Cover Plate Fit T/B										
Cover Plate Welded T/B										
Stiffeners Welded										
Weld Dimensions Checked										
Attach Bearings										
Bearing Protection										
NDT – **Radiography**										
NDT – **Magnetic Particle**										
NDT – **Ultrasonic**										
Length Check										
Web Flatness										
Flange Tilt										
Camber Check										
Sweep Check										
Blast Clean										
Pick up Repair's										
Paint – **1st Coat**										
Paint – **2nd Coat**										
Paint – **3rd Coat**										
Paint – **4th Coat**										
Final Approval										
Shipment										
Weight (lb)										

Remarks:

CAMBER DIAGRAM

Fabricator Name:

Contract Number	Project Number



Item: Piece Mark: Length: Tolerance:

Heat Corrected: Yes No Max. Temp.: Heat Pattern:

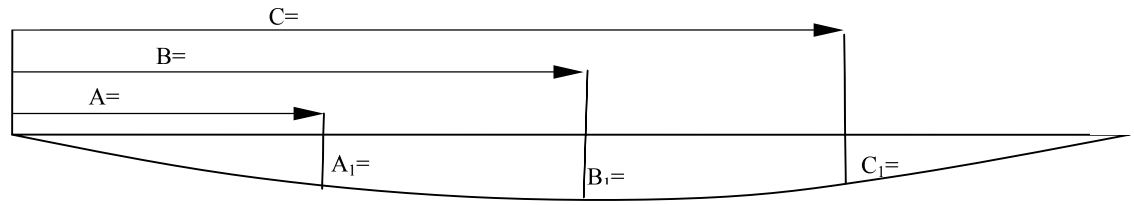
Panel Points	B	C	D
Location (From Left End N.S)			
Required Camber			
Camber Before Heat Correction			
Final Camber			
Heat Location (From Left End N.S)			

Inspector Name:

Inspector Signature:

SWEEP DIAGRAM

Fabricator Name:	
Contract Number	**Project Number**



Item: Piece Mark : Length: Tolerance:

Heat Corrected: Yes No Max. Temp.: Heat Pattern:

Top Flange

Distance from Left	Required Sweep	Sweep Before Heat Correction	Final Sweep	Heat Location (from left)

Bottom Flange

Distance from Left	Required Sweep	Sweep Before Heat Correction	Final Sweep	Heat Location (from left)

Material Certification Review Summary Form

Fabricator Name:

Contract Number	Project Number

P.O. #	Mill / Supplier	Heat or Lot #	Materials Description	Insp. Initial	Date Accept.

Remarks:

GIRDER HEAT NUMBER RECORD

Fabricator Name:

Contract Number	Project Number

Piece Mark	Top Flange	Web	Bottom Flange

Remarks:

Rolled Beam-Stringer Heat Number Record

Fabricator Name:

Contract Number	Project Number

Piece Mark	Beam Size	Beam Length	Beam Heat #	Cover Plate Heat #	
				Top	Bottom

Remarks:

Quality Control Repair Notice Summary

Fabricator Name:

Contract Number	Project Number

QCRN	Date	Piece Mark	Location/ Type of Defect	Date Submitted	Date Procedure Approved	Date Repaired	MT ACC	UT ACC	PT ACC	RT ACC	Final ACC

Remarks:

Bolt Torquing Verification

Fabricator Name:

Contract Number	Project Number

Bolt Grade: Dia.: Inches Length: Inches Galvanized:

Supplier		
	Bolt:	
	Nut:	
	Washer:	

Installation Process Used

Status Indicator	Attribute
	1. Verify – Bolt Marking
	2. Check – Surface condition of bolts, nuts and washers
	3. Check – Storage of bolts, nuts and washers
	4. Check – Faying Surfaces of Joints
	5. Observe – Calibration/Testing procedures
	6. Verify – Selected procedure is properly applied
	7. Verify – Procedure used provides the tension values given in table
	8. Monitor – Installation of fasteners verifying that the selected procedure demonstrated in the initial testing to supply the specified tension is properly applied

If Unsatisfactory, Explain:

\# of Bolts in connection: # of Bolts verified: Inspection lot numbers:

Inspection Procedure (Once per day when installing H.S. Bolts):
Three bolts of the same grade, size, and condition shall be placed individually in a device calibrated to measure bolt tension, such as Skidmore-Wilhelm. Tighten each bolt in the calibrated device to the specified tension. Using the inspection wrench turn the nut and additional 5 degree (approx. 1” at a 12” radius). Record the torque values below:

Tension device Ser. No.: Calibration Date: Cal. Due Date:

Bolt 1:

Bolt 2:

Bolt 3:

Average Torque Value:

Test Status:

Inspector Name:

Inspector Signature:

Galvanizing Check List

Galvanizer’s Name:

Contract Number	Project Number

Type of Inspection (Select one): In-Process Inspection Final Inspection

Painted after Galvanizing: Yes No

Item: Type:

Piece Mark: Number of Pieces:

Item	Status	Remarks
Material free from damage:		
Material free from grease or other coatings:		
Galvanizing procedure reviewed:		
Quenching performed:		
Chromate treatment allowed:		
Cutting / Welding performed by galvanizer:		
Material properly handled after coating:		
Material Stored Properly:		
Visual Appearance acceptable:		
Thickness acceptable (use DFT form):		
Adhesion visually acceptable:		
All welds visually checked for cracks:		
Material free from distortion:		
Repairs performed per ASTM A780:		
Final approval / Ship:		

Inspector Name:

Inspector Signature:

NDT Inspection Report

Fabricator Name:

Contract Number	Project Number

Date	Piece Mark	NDT Type	Observations	Accepted	ASNT Operators	Witnessed By

Paint Inspection Report

Fabricator Name: Report Date:
Contract Number: Project Number:

Piece Mark:

Environmental Conditions for Surface Preparation

Date: Time:
Dr Bulb Temp.: Wet Bulb Temp.:
Relative Humidity: Dew Point:
* Surface Temp.: Inspector:
Application:
Surface finish actual: Surface Profile Actual:
Inspector: Date:

Environmental Conditions for Prime Coat

Date: Time:
Dr Bulb Temp.: Wet Bulb Temp.:
Relative Humidity: Dew Point:
* Surface Temp.: Inspector:
Application:
Coating Product: Batch # A Batch # B
Start Date Time
Stop Date Time
Actual DFT Low: High Avg.
Inspector: Date:

Environmental Conditions for Intermediate Coat

Date: Time:
Dr Bulb Temp.: Wet Bulb Temp.:
Relative Humidity: Dew Point:
* Surface Temp.: Inspector:
Application:
Coating Product: Batch # A Batch # B
Start Date Time
Stop Date Time
Actual DFT Low: High Avg.
Inspector Date:

Environmental Conditions for Final Coat

Date: Time:
Dr Bulb Temp.: Wet Bulb Temp.:
Relative Humidity: Dew Point:
* Surface Temp.: Inspector:
Application:
Coating Product: Batch # A Batch # B
Start Date Time
Stop Date Time
Actual DFT Low: High Avg.
Inspector: Date:

* (must be 5 degrees above dew point)

Project Narrative

Fabricator Name:

Contract Number	Project Number	Date

Inspector Name:

Inspector Signature:

AASHTO/NSBA Steel Bridge Collaboration
S 8.1 - 2006

Guide for Application of Coating Systems With Zinc-Rich Primers to Steel Bridges

AASHTO/NSBA Steel Bridge Collaboration

Preface

This document is a standard developed by the AASHTO/NSBA Steel Bridge Collaboration. The primary goal of the Collaboration is to achieve steel bridge design and construction of the highest quality and value through standardization of the design, fabrication, and erection processes. Each standard represents the consensus of a diverse group of professionals. A listing of those serving on the Committee which developed the *Guide for Application of Coating Systems with Zinc-Rich Primers to Steel Bridges* will be included in future editions.

As consensus documents, the Collaboration standards represent the best available current approach to the processes they cover. It is intended that Owners adopt and implement Collaboration standards in their entirety to facilitate the achievement of standardization. It is understood, however, that local statutes or preferences may prevent full adoption of the document. In such cases Owners should adopt these documents with the exceptions they feel are necessary.

This document establishes and defines the functions, operations, requirements, and activities needed to achieve consistent quality in steel bridge painting. It is based on a cooperative approach to achieving quality, where both the Owner's and Contractor's representatives work together with a clear understanding of their roles and responsibilities, resulting in steel bridges completed in an efficient manner while meeting all contractual requirements.

Disclaimer

All data, specifications, suggested practices presented herein, are based on the best available information and delineated in accordance with recognized professional engineering principles and practices, and are published for general information only. Procedures and products, suggested or discussed, should not be used without first securing competent advice respecting their suitability for any given application.

Publication of the material herein is not to be construed as a warranty on the part of the American Association of State Highway and Transportation Officials (AASHTO) or the National Steel Bridge Alliance (NSBA) - or that of any person named herein - that these data and suggested practices are suitable for any general or particular use, or of freedom from infringement on any patent or patents. Further, any use of these data or suggested practices can only be made with the understanding that neither AASHTO nor NSBA makes any warranty of any kind respecting such use and the user assumes all liability arising therefrom.

Table of Contents

Introduction

This guide represents a consensus on best industry practice for shop application of zinc rich primers to previously uncoated bridge steel and includes the proper preparation of the steel surfaces and the mixing, application, and cure of coatings. To simplify the application parameters for a system based on zinc-rich primers on new steel bridges, a series of charts have been developed. These charts provide a convenient summary listing the detailed requirements for surface preparation, environmental conditions, coating application, curing, and verification testing. The guide addresses a three-coat system consisting of primer, intermediate coat, and topcoat, but is appropriate for application of a two-coat system or primer only.

Environmental, containment, and safety issues referenced herein are outside the scope of this guide and should be addressed in other documents.

This guide uses mandatory (imperative language) to assist owners and specifiers with development of contract language.

Section 1
Definitions

1.1 Applicator ("Shop")
Person or contractor who applies a coating.

1.2 Breaking the Corner (Corner Chamfering)
A process by which a sharp corner is flattened by passing a grinder or other suitable device along the corner, normally in a single pass.

1.3 Checking
The fine cracking that develops in paint films during prolonged curing and/or weathering that does not penetrate to the underlying surface. ASTM D 660 is the "Standard Test Method for Evaluating Degree of Checking of Exterior Paints."

1.4 Conformance Certification
A verification issued by the coating manufacturer confirming that a particular batch of product was produced in accordance with the manufacturer's standard. This standard of performance for the product must have previously been approved or accepted by the Owner.

1.5 Corner
The intersection of two surfaces.

1.6 Edge
An exposed, through-thickness surface of a plate or rolled shape, for example the as-rolled side face of a beam flange, channel flange or angle leg, or as a result of thermal cutting, sawing, or shearing. Edges may be planar or rounded, and either perpendicular or skewed to adjacent faces.

1.7 Edge Grinding (Edge Conditioning)
Very shallow grinding or other pre-blast cleaning preparation of thermal cut edges (TCEs) to remove a thin, hardened layer left by resolidification. It does not include grinding required by the D1.5 Bridge Welding Code or ASTM A 6 to remove cutting, handling or material anomalies.

1.8 Fastener
A mechanical device used to attach two or more items together; e.g., a bolt, nut and washer.

1.9 Faying Surfaces
Contacting surfaces where joints in steel structures are fastened together, formed, for example, by fastening devices.

1.10 Limited Access Areas
Partially or completely enclosed surfaces, the majority of which are not visible without the use of special devices such as mirrors, and not readily accessible for coating by routine methods.

1.11 Mud Cracking
A coating defect resembling the irregular cracking of drying mud that typically arises during the curing of a relatively inflexible coating applied too thickly.

1.12 Qualified Product
A coating product which has been approved based on testing to a Federal, State, or regional agency, or a test protocol (e.g., AASHTO, NEPCOAT, NTPEP, or DOT protocol).

1.13 Sharp
An acute corner or prominence that is able or appears to be able to cut human flesh.

1.14 Spot Priming
Application of primer paint to localized spots where the substrate is bare or where additional protection if needed because of damage to or deterioration of a former coat.

1.15 Stripe Coat

A coat of paint applied only to edges or to welds on steel structures before or after a full coat is applied to the entire surface. The stripe coat is intended to give those areas the required dry film thickness (DFT) to resist corrosion.

1.16 Visible Coating Defects

Imperfections that may be detected by examination without magnification. These include runs, sags, lifting, chipping, cracking, spalling, flaking, mudcracking, pinholing, and checking.

1.17 Weld Spatter, Tight

Beads of metal produced during the welding process with adequate thermal energy to adhere on metal adjacent to the weld area. The droplets retain their individual shape but have sufficient fusion to resist removal by hand scraping with a putty knife, per SSPC-SP 2.

Section 2
Reference Standards

Unless otherwise noted in the contract, the latest edition of the following standards and regulations in effect at the time of Contract letting form a part of this guide. A copy of applicable reference standards shall be available at the painting facility.

2.1 American Association of State Highway and Transportation Officials (AASHTO)

2.1.1 AASHTO M 160, Standard Specification for General Requirements for Rolled Steel Plates, Shapes, Steel Pilings, and Bars for Structural Use

2.1.2 AASHTO M 300, Standard Specification for Inorganic Zinc-Rich Primer

2.1.3 AASHTO R 31, Standard Practice for Evaluation of Coating Systems with Zinc-Rich Primers

2.2 American Society for Testing and Materials (ASTM)

2.2.1 ASTM A 6, Specification for General Requirements for Rolled Structural Steel Bars, Plates, Shapes, and Sheet Piling

2.2.2 ASTM D 3359, Standard Test Methods for Measuring Adhesion by Tape Test

2.2.3 ASTM D 4138, Standard Test Method for Measurement of Dry Paint Thickness of Protective Coating Systems by Destructive Means

2.2.4 ASTM D 4285, Standard Test Method for Indicating Oil or Water in Compressed Air

2.2.5 ASTM D 4414, Standard Practice for Measurement of Wet Film Thickness by Notch Gages

2.2.6 ASTM D 4417, Standard Test Methods for Field Measurement of Surface Profile of Blast Cleaned Steel

2.2.7 Standard Test Method for Pull-Off Strength of Coatings Using Portable Adhesion Testers

2.2.8 ASTM D 4752, Standard Test Method for Measuring MEK Resistance of Ethyl Silicate Zinc-Rich Primers by Solvent Rub

2.3 SSPC: The Society for Protective Coatings

2.3.1 SSPC-Paint 29, Zinc Dust Sacrificial Primer, Performance-Based

2.3.2 SSPC-Paint 30, Weld-Through Inorganic Zinc Primer

2.3.3 SSPC-PS 12.00, Guide to Zinc-Rich Coating Systems

2.3.4 SSPC-QP 1, Standard Procedure for Evaluating Painting Contractors (Field Application to Complex Industrial Structures SSPC-AB 1, Mineral and Slag Abrasives

2.3.5 SSPC-AB 2, Cleanliness of Recycled Ferrous Metallic Abrasives

2.3.6 SSPC-AB 3, Ferrous Metallic Abrasive

2.3.7 SSPC Guide 13, Guide for the Identification and Use of Industrial Coating Material in Computerized Product Databases

2.3.8 SSPC-PA 1, Shop, Field, and Maintenance Painting of Steel

2.3.9 SSPC-PA 2, Measurement of Dry Coating Thickness with Magnetic Gages

2.3.10 SSPC-Paint 20, Zinc-Rich Coating, Type I–Inorganic, and Type II–Organic)

2.3.11 SSPC-QP 3, Standard Procedure for Evaluating Qualifications of Shop Painting Contractors

2.3.12 SSPC-SP 1, Solvent Cleaning

2.3.13 SSPC-SP 2, Hand Tool Cleaning

2.3.14 SSPC-SP 3, Power Tool Cleaning

2.3.15 SSPC-SP 10/NACE No. 2, Near-White Blast Cleaning

2.3.16 SSPC-SP 11, Power Tool Cleaning to Bare Metal

2.3.17 SSPC-SP 15, Commercial Grade Power Tool Cleaning

2.3.18 SSPC-SP COM, Surface Preparation and Abrasives Commentary, SSPC Painting Manual, Volume 2, "Systems and Specifications"

2.3.19 SSPC-Guide 15, Field Methods for Retrieval and Analysis of Soluble Salts on Steel and Other Nonporous Substrates

2.3.20 SSPC-VIS 1, Guide and Reference Photographs for Steel Surfaces Prepared by Dry Abrasive Blast Cleaning

2.4 Research Council on Structural Connections (RCSC)

Specification for Structural Joints Using ASTM A325 or A490 Bolts, Section 5(b), endorsed by the Research Council on Structural Connections.

2.5 American Institute for Steel Construction (AISC)

2.5.1 Sophisticated Paint Endorsement (SPE)

2.6 Related Reference Documents

2.6.1 Applicable Ordinances and Regulations

2.6.2 Equipment and Coating Manufacturer's Published Instructions and Product Data Sheets (PDS).

Section 3
General

3.1 Qualification

When the contract requires painting more than 1,500 square feet of steel surface, the organization(s) performing coating application must demonstrate qualification by obtaining SSPC QP 1 certification for field painting or either SSPC-QP 3 certification or the AISC Sophisticated Paint Endorsement (SPE) for shop painting. This qualification must be maintained throughout the painting portion of the project. If it expires or is revoked for any reason, notify the owner, who may require that a qualified organization complete the coating portion of the project.

3.2 Quality Control

Conduct and document quality control inspection of the cleaning and painting operations including, at a minimum, measurements of ambient conditions, surface profile, surface cleanliness, coating material acceptability, dry film thicknesses, and visual inspection for coating continuity and defects. Record the data and make it available for the Owner's review.

3.3 Written Procedures

Maintain written standard procedures, submitted to the Owner upon request, covering such items as verifying and maintaining paint manufacturer data, measuring and recording dry film thickness and cure time, protection and treatment of faying surfaces, and other information needed to successfully document and apply all coats of paint.

3.4 Miscellaneous

Surfaces to be painted and the coating system to be used shall be as indicated on plans and/or contract documents. Unless otherwise noted, paint is not required on flange surfaces that will be embedded in concrete, or inside bolt holes, although overspray is permitted on flange surfaces and inside bolt holes.

3.5 Protective Coating System

3.5.1 Use only Owner-approved coating systems meeting the applicable slip, creep, and other contract requirements.

3.5.2 Coating products suitable for use shall meet the test requirements of AASHTO M 300, AASHTO R 31, SSPC-Paint 20 (Type II – Inorganic, Type I B or Type IC), SSPC-Paint 29 (Type II – Inorganic, Type I B or Type IC) or SSPC-Paint 30, or other acceptable qualified product testing protocols meeting owner acceptable criteria. SSPC-PS Guide 12.00 provides additional information about selecting coating materials.

3.5.3 All coatings shall be supplied by the same coating manufacturer and designated by the coating manufacturer for use as a recommended system, unless otherwise approved by the Owner.

3.5.4 Coating thickness shall be as specified by the Owner and within the manufacturer's published allowable thickness ranges. Coating thickness ranges on faying surfaces are limited to the maximum allowable thickness defined in the test certification according to RCSC (see Section 2.4) for the given product (see Commentary Ssection C3.5.4).

3.6 Coating Repairs

Submit repair procedures in accordance with the manufacturer's written recommendations for the Owner's approval. Repairs to the final coat must result in an acceptable, uniform gloss and color on visible surfaces. The Owner shall have final authority concerning the coating's uniformity and acceptable appearance.

3.7 Paint Storage

Paint shall be stored in accordance with SSPC-QP 3 and manufacturer's Product Data Sheet. (PDS).

3.8 Steel Storage After Painting

Handle steel members with care to minimize damage to or contamination of the coating. Handle large members with synthetic slings, padded chains and lifting clamps, or other non-injurious methods, and store them on padded or otherwise protected blocking. Small assemblies may be bundled utilizing cushioners (e.g., cardboard, template paper, carpeting, etc.) or other means to avoid or minimize metal-to-metal contact of painted areas. Paint must be dry-to-handle in accordance with the coating manufacturer's PDS before lifting or placing on supports to avoid paint damage or foreign material adhering to painted surfaces.

3.9 Final Acceptance

After all shop coats have been applied, repaired as required and cured, the Owner may conditionally accept the coating. Final coating acceptance will be given at the jobsite. Shop coatings will be evaluated upon arrival for "fabricate and deliver" contracts. After steel is erected, damaged coatings are repaired, and field bolted or welded connections are fully coated, assessment of the entire paint system for "fabricate and erect" contracts or the field applied coat(s) for erection and field painting contracts will be completed.

Section 4
Material Acceptance

4.1 Paint

4.1.1 Verify that all paint materials satisfy composition and testing requirements, are in conformance with the owner-approved Qualified Products List or other applicable requirements, and will not exceed the manufacturer's specified shelf life before use.

4.1.2 Instruct the coatings supplier to provide batch samples of all coating material components to the Owner as required or requested.

4.1.3 Materials will be rejected if the material arrives at the application site in other than original, unopened containers; if a container has a break in the lid seal or a puncture; or if the coating materials have begun to polymerize, solidify, gel, or deteriorate in any manner.

4.2 Abrasive Media Condition

Verify that abrasive cleaning material meets the requirements of SSPC-AB 1, "Mineral and Slag Abrasives"; SSPC-AB 2, "Cleanliness of Recycled Ferrous Metallic Abrasives"; or SSPC-AB 3, "Ferrous Metallic Abrasive." The condition and cleanliness of recycled abrasives shall be confirmed in accordance with the applicator's approved quality control program as per SSPC-QP 3 and/or AISC Sophisticated Paint Endorsement.

Section 5
Surface Preparation

5.1 Material Anomalies

Corner Condition – Remove all sharp corners prior to painting with either organic or inorganic zinc-rich primer by creating a small chamfer. With inorganic zinc rich primer, corners less acute than the definition of "sharp" need no further treatment prior to final cleaning (profiling) and painting. Sharp corners may usually be removed by a single pass with a grinder. For organic zinc-rich primer-based systems, all corners resulting from sawing, burning, or shearing operations must be broken or a primer stripe coat shall be applied to those corners.

5.1.1 Preparation of Thermal Cut Edges – Before blast cleaning, condition (by lightly grinding) thermal cut edges (TCEs) to be painted, if conditioning is necessary to achieve proper blast cleaning profile.

5.1.2 Base Metal Surface Irregularities – Remove all visually evident surface defects in accordance with ASTM A 6 or AASHTO M 160 prior to blast cleaning steel. When material defects exposed by blast cleaning are removed, restore the blast profile either by blast cleaning or by using mechanical tools in accordance with SSPC-SP 11 or SP 15.

5.1.3 Weld Irregularities or Spatter – Remove or repair all sharp weld prominences, and all heavy, sharp, or loose weld spatter. Occasional individual particles of rounded tight weld spatter may remain, but widespread, sharp, or clustered particles of tight weld spatter must be removed.

5.2 Pre-Cleaning

Remove all oil, grease, and other adherent deleterious substances, including bolt lubricant and excess dye, from areas to be painted, in accordance with SSPC-SP 1, prior to abrasive blast cleaning.

5.3 Abrasive Blast Cleaning

5.3.1 Abrasive blast clean the entire surface to be coated in accordance with the cleanliness and profile required by the manufacturer's PDS and the owner's specifications.

5.3.2 If the material for the project is heavily rusted or pitted, or as directed by the Owner, measure the non-visible contaminant and remove to a level in accordance with owner's specifications, or as required by Table 5.2, Row 4.

5.3.3 Assess the profile per ASTM D 4417, Method C (Replica Tape).

5.4 Bolts (Fasteners)

5.4.1 When bolts are to be installed and final-tightened before priming, prepare them as necessary so that after the steel is abrasive blast cleaned, exposed bolt surfaces will satisfy the requirements outlined in Table 5.1.

5.4.2 Black bolts, nuts and washers, including flat faces of nuts and bolt heads facing adjacent material, may require spot blast cleaning or other surface preparation before general blast cleaning in order to assure that the proper surface profile to obtain adhesion of the primer, has been achieved.

5.4.3 If the zinc coating on mechanically or hot-dip galvanized bolts is damaged during abrasive blast cleaning or tightening, it may be left "as is" if the entire coating system (including the zinc-rich primer) will be applied over the fasteners.

5.4.4 Obtain the identity of solvents and methods needed to remove the lubricant. Supply to the General Contractor, shop and field painters, the Owner, and other interested parties the information concerning the lubricant removal and the cleanliness necessary for intermediate coat adhesion. Perform periodic evaluation to ensure that lubricant and excess dye are adequately removed.

5.4.5 When zinc-coated twist-off tension-control or lock-pin-and-collar fasteners are used, completely seal the sheared end of each fastener with non-silicone-type sealing compound conforming to the provisions in Federal Specification TT-S-230, Type II.

Apply the sealant to the non-rusted surface on the same day that the bolt is installed. The sealant shall be compatible with the subsequently applied coating.

Table 5.1 Surface Preparation Requirements for Fasteners

Item	**Fasteners Installed Prior to Cleaning & Primer Application**		**Fasteners Installed After Primer Application**	
	Coating System	**Surface Prep.**	**Coating System**	**Surface Prep.**
Black Iron Bolts	OZ or IOZ	Section 5.4.1	OZ, IOZ	SSPC-SP 1, 10
			Other Specified	SSPC-SP 1 and as required to achieve the proper degree of cleaning specified
Galvanized (Mechanical or Hot Dip)	OZ or IOZ	SSPC-SP 1	Intermediate Coat	SSPC-SP 1, 2, 3, and/or 12

OZ= Organic Zinc-Rich Coating IOZ= Inorganic Zinc-Rich Coating

5.5 Summary

Table 5.2 summarizes the requirements for pre-cleaning, cleaning, profile, and surface cleanliness of structural steel.

Table 5.2 Surface Preparation Summary Table

Requirement	Basis for Acceptance	Minimum	Maximum	Frequency/ Extent
1. Pre-Clean	Visual	SSPC-SP 1	N/A	100%
2. Degree of Cleaning	Visual	PDS	N/A	100%
3. Profile (ASTM D 4417)	Test	PDS	PDS	once every shift for automated operations and twice per shift per nozzle , and whenever the operating media mix is changed
4. Surface Cleanliness (SSPC-Guide 15)[2]	Test	N/A	per owner requirements	Varies[1]
6. Fasteners	Test Black Iron Mechanically or Hot Dip Galvanized	 SP 1, SP 10 SP 1	 SP 1, SP 10 SP 1	 100% 100%

1 Only needed on heavily rusted or pitted steel as described in SSPC-VIS 1 Conditions C and D.

2 The test methods indicated in SSPC-Guide 15 vary in their sensitivity and the particular test method used should be selected by the Owner.

Section 6
Paint Application

6.1 General

6.1.1 Apply coatings in accordance with the Contract requirements, SSPC-PA 1, and the manufacturer's instructions.

6.1.2 Conduct and document an on-going quality control inspection of the materials, prepared surfaces, and the prime, intermediate, and topcoat painting per Tables 6.1 to 6.3 (presented at the end of Section 6).

6.1.3 Record the daily storage temperature range for coating materials and verify conformance with the coating manufacturer's product data sheet. Use inventory control to ensure that components are used within the shelf life prescribed by the manufacturer. Record the coating batch numbers from the mixed components, the amount and type of thinner used, and the date applied in the application log.

Verify that the coating has been applied to provide a continuous, uniform film of the specified thickness; is well bonded to the metal or previously applied coating; is free of laps, streaks, sags, or other visually evident defects; and was applied within the manufacturer's specified pot life.

6.1.4 Areas that fail any required test(s) shall be subject to the nonconformance disposition procedure outlined in Section 3.6.

6.1.5 Tables 6.1, 6.2, and 6.3 show information related to ambient conditions; surface cleanliness; and mixing, application, and cure. The charts show the specific source of relevant control information, as well as minimum and maximum tolerances. Also indicated are inspection frequency requirements.

6.2 Coatings on Faying Surfaces

Coatings or coating systems applied to contact surfaces of bolted connections between primary members shall satisfy RCSC requirements for the specified slip friction coefficient (see Section 2.4) and the temperature-adjusted cure time. Prior to bolting, verify that the coating on faying surfaces is properly cured in accordance with the manufacturer's requirements. Verify that the dry film thicknesses and the temperature-adjusted cure time for slip critical bolted faying surfaces are within the range previously validated by testing of the coating or coating system.

6.3 Intermediate and Topcoat

The color of the topcoat shall be as specified in the contract documents. If an intermediate coat is used, its color shall contrast with both the primer and final coat, as approved by the Owner. Coating materials used to apply piecemarks shall be compatible with the existing and any subsequent coats or the piece marks shall be removed as a contaminant.

Table 6.1 Zinc-Rich Prime Coat Inspection

Requirement	Basis for Acceptance	Minimum	Maximum	Frequency/Extent
1. Current Painter Qualification Verified	Applicator QC Plan	As Required by SSPC/ AISC	N/A	Every painter/project
2. Ambient Temperature	Product Data Sheet	As Required	As Required	Every 4 hours[1]
3. Dew Point & Relative Humidity	Product Data Sheet	As Required	As Required	Every 4 hours[1]
4. Surface Temperature	Product Data Sheet	As Required	As Required	Every 4 hours[1]
5. Primer Component Batch Number	Owner Approved Batch Numbers[2]	N/A	N/A	Every Paint Kit
6. Verification of Surface Cleanliness	SSPC-SP 10	N/A	24 hrs and before Flash Rust	Examine visually and with cloth w/in 1 hour prior to priming
7. Date and Time	N/A	N/A	N/A	Each Lot of Work[3]
8. Piece Mark or Bundle	N/A	N/A	N/A	Each Lot of Work[3]
9. Temperature of Mixed Primer	Product Data Sheet	As Required	As Required	When mixing components[4]
10. Proper Mixing and Straining	Product Data Sheet	As Required	N/A	Every Pot Mix[4]
11. Primer Induction Time[5]	Product Data Sheet	N/A	N/A	Every Pot Mix[4]
12. Primer Pot Life[5]	Product Data Sheet	N/A	As Required	Every Pot Mix[4]
13. Primer Stripe Coat[6,7]	Product Data Sheet	N/A	N/A	N/A
14. Primer Recoat Window Time	Product Data Sheet	As Required	Acceptable[7]	Each lot of work
15. Primer Cure Time	Product Data Sheet	As Required	As Required	Each Lot of Work
16. Primer Cure Test[6]	Product Data Sheet or Owner Spec	As Required	As Required	Daily or Once / Shift
17. Dry Film Thickness	Product Data Sheet or Owner Spec	As Required	As Required	SSPC-PA 2
18. Visual Inspection	SSPC-PA 1	No defects	N/A	100%
19. Primer Coat Evaluation and Repair[9]	SSPC PA 1 & Approved Procedure	As Required	N/A	Visual, 100% of each element
20. Primer Recoat Time	Product Data Sheet	As Required	Acceptable[6]	Each lot of work

Footnotes follow Table 6.3.

Table 6.2 Intermediate Coat Inspection

Requirement	Basis for Acceptance	Minimum	Maximum	Frequency/Extent
1. Current Painter Qualification Verified	Applicator QC Plan	As required by SSPC/ AISC	N/A	Every painter/project
2. Ambient Temperature	Product Data Sheet	As Required	As Required	Every 4 hours[1]
3. Dew Point & Relative Humidity	Product Data Sheet	As Required	As Required	Every 4 hours[1]
4. Surface Temperature	Product Data Sheet	As Required	As Required	Every 4 hours[1]
5. Intermediate Coat Component Batch Number	Owner Approved Batch Numbers[2]	N/A	N/A	Every Paint Kit
6. Primer Coat Evaluation and Repair[6]	SSPC PA 1 & Approved Procedure	As Required	N/A	Visual, 100% of each element
7. Primer Recoat Time	Product Data Sheet	Full Cure[6]	Acceptable[6]	Each Lot of Work[3]
8. Verification of Primer Surface Cleanliness	SSPC-SP 1	As Required	N/A	Initial and Every 4 Hours of Painting
9. Date and Time	N/A	N/A	N/A	Each Lot of Work[3]
10. Piece Mark or Bundle	N/A	N/A	N/A	Each Lot of Work[3]
11. Temperature of Mixed Inter. Coat	Product Data Sheet	As Required	As Required	When mixing components[4]
12. Intermediate Coat Mixing &/or Straining	Product Data Sheet	As Required	N/A	Every Pot Mix[4]
13. Intermediate Coat Induction Time[5]	Product Data Sheet	N/A	N/A	Every Pot Mix[4]
14. Intermediate Coat Pot Life[5]	Product Data Sheet	N/A	As Required	Every Pot Mix[4]
15. Intermediate Coat DFT	Owner spec or PDS[8]	As Required	As Required	SSPC-PA 2
16. Visual Inspection	SSPC-PA 1	No Defects	N/A	100%
17. Intermediate Coat Evaluation and Repair[9]	SSPC PA 1 & Approved Procedure	As Required	N/A	Visual, 100% of each element
18. Intermediate Coat Recoat Time	Product Data Sheet	Full Cure	As Required	Each Lot of Work[3]

Footnotes follow Table 6.3.

Table 6.3 Top Coat Inspection

Requirement	Basis for Acceptance	Minimum	Maximum	Frequency/Extent
1. Current Painter Qualification Verified	Applicator QC Plan	As Required by SSPC/ AISC	N/A	Every painter/project
2. Ambient Temperature	Product Data Sheet	As Required	As Required	Every 4 hours[1]
3. Dew Point & Relative Humidity	Product Data Sheet	As Required	As Required	Every 4 hours[1]
4. Surface Temperature	Product Data Sheet	As Required	As Required	Every 4 hours[1]
5. Top Coat Component Batch Number	Owner Approved Batch Numbers[2]	N/A	N/A	Every Paint Kit
6. Intermediate Coat Evaluation and Repair[9]	SSPC PA 1 & Approved Procedure	As Required	N/A	Visual, 100% of each element
7. Intermediate Coat Recoat Time	Product Data Sheet	Full Cure	Acceptable	Each Lot of Work
8. Verification of Int. Coat Surface Cleanliness	SSPC-SP 1	As Required	N/A	Initial and Every 4 Hours of Painting
9. Date and Time	N/A	N/A	N/A	Each Lot of Work[3]
10. Piece Mark or Bundle	N/A	N/A	N/A	Each Lot of Work[3]
11. Temperature of Mixed Top Coat	Product Data Sheet	As Required	As Required	When mixing components[4]
12. Top Coat Mixing &/or Straining	Product Data Sheet	As Required	N/A	Every Pot Mix[4]
13. Top Coat Induction Time[5]	Product Data Sheet	N/A	N/A	Every Pot Mix
14. Top Coat Pot Life[5]	Product Data Sheet	N/A	As Required	Every Pot Mix[4]
15. Topcoat Dry Time	Product Data Sheet	Full Cure	N/A	Each Lot of Work[3]
16. Top Coat DFT	Owner Spec or PDS[8]	As Required	As Required	SSPC-PA 2
17. Visual Inspection	SSPC-PA 1	No Defects	N/A	100%
18. Paint System Final Evaluation and Repair[9]	SSPC PA 1 & Approved Procedure	As Required	N/A	Visual, 100% of each element

1 Based on weather conditions, more or less frequent testing may be stipulated by the Owner

2 Owner-approval may be based on sampling and testing or the manufacturer's certification acceptance that batch compositions conform to previously approved standards. Primer for faying (contact) surfaces of high strength bolted connections (for slip-critical, frictional transfer of load) must satisfy RCSC requirements for a Class B rating, based on certified tests by the coating manufacturer or applicator.

3 All items in each lot that were coated with the same batch of paint must be identified. Small items may be identified by bundle or shipping container number.

4 Every pot mixed should be verified. QC must document acceptance by initials or signature, and form must include time/date and batch component numbers.

5 Upon addition of the activator. Does not apply to single or multiple component coatings not utilizing a catalyst or other activator that reacts to modify the mixed coating and/or limit pot life.

6 Cure testing of primer is required prior to shop application of intermediate or stripe coat, or if adding extensive organic zinc primer to increase initial DFT. (Not required for spot priming.) See manufacturer's thinning requirements if repriming previously coated surface. For field-applied intermediate coat, the cure test is not required, but adequate primer cure must be verified before handling and shipping. For inorganic zinc, if the PDS does not have a specified cure test, then ASTM D 4752 shall apply.

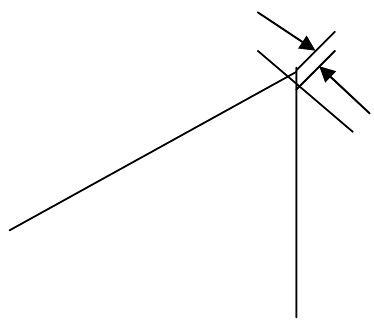
7 Stripe coat(s) may be applied either before or after the full coat to ensure adequate coverage on outside corners, edges and other areas specified in the applicator's coating plan. Note that striping IOZ is not normally recommended.

8 Based on difference between average DFT of coats applied and previous readings for similar areas. For example, webs, stiffeners, flanges, cross frames, and bearings may each have different averages due to application patterns.

9 All repairs shall meet the requirements of Section 3.6. The recoat window shall follow the guidelines of this guide, the coating manufacturer's recommendations, and good painting practice as dictated by SSPC.

COMMENTARY

C1.3 The breaking (flattening) of a sharp corner is sketched below:



Care should be taken to ensure that no new sharp edges are raised by the grinding.

Extensive testing has proven shop grinding of corners is unnecessary for improving coating coverage and corrosion protection when using ethyl silicate inorganic zinc-rich primer systems with minimum zinc loading of 83%. Limited testing has also shown that while organic zinc-rich materials sometimes tend to drawn thin on square corners, they cover well on corners that are merely flattened ("breaking the edge"). Therefore, when organic zinc-rich materials are applied to corners that have been "cut" (sawed, burned, or sheared), the corners must either be "broken" or stripe coated per SSPC-PA 1, Section 6.6, "Striping."

C1.7 Edge grinding can be used to remove martensite, a hardened form of steel that may occur due to rapid resolidification following thermal cutting. This layer is typically very thin, about 0.01" to 0.02" (0.25 to 0.50 mm) thick, and is dependent upon the steel's chemistry and thickness. Light grinding is generally sufficient to remove this material, and is only necessary if the hardness interferes with achieving the desired profile during blast cleaning. The presence of martensite and/or the small grooves normally left by thermal cutting are not a fatigue or stress concentration problem.

C3.5.4 DFTs in excess of those permitted, although constituting a specification nonconformance, should be considered on a case-by-case basis. Occasional excursions, especially at inside corners and other areas prone to high DFT due to pattern overlap, may be acceptable if the coating is well adhered and free of the deficiencies listed in Section 6.1.3. Attempting to reduce DFT in an otherwise acceptable coating may result in significant problems or extensive, unnecessary rework. The applicator should be notified of the problem and amend procedures to avoid recurrences. QC and QA inspectors should document the deficiencies noted, any corrections required, and resulting changes in the applicator's practices, consistent with the resolution of any other nonconformance item.

C3.8 Naturally, while members may be moved from the paint "skids" when the coating is "dry to handle," the coating must be thoroughly cured prior to bundling, storing and shipping to minimize damage from the handling during these processes.

C3.9 Although the Owner (through the QA Inspector) may accept the shop-painted fabricated items before shipment to the job site, final acceptance of the paint system by the Owner on "fabricate and erect" or field painting contracts will typically occur following erection of the structure, after all field coats and paint repairs have been completed. The Owner may elect to check the coated member at the fabrication/paint shop prior to shipping and approve the item, waiving field inspection and approval, especially on Type 1 (three shop coat) contracts, for the shop painting requirement of "fabricate and deliver" contracts, and on pieces with limited access after erection, and prior to field coating.

C4.1.1 The issues surrounding the testing and selection of actual coating materials are not addressed in this document because AASHTO maintains a testing protocol and database of testing results as part of AASHTO NTPEP (National Transportation Product Evaluation Program) initiative.

C5.1.1 Some corner "softening" (flattening) occurs during blast cleaning; however, blast cleaning alone will not sufficiently break a sharp corner. Corners in the as-rolled condition are not normally in need of any treatment. See further discussion in Section C1.3.

C5.1.3 While the removal of all weld spatter is recommended, it is recognized that absolute compliance would present difficulties, often leading to unnecessary rework. As a practical matter, occasional tightly adhered spatter may remain as long as paint coverage and adhesion are not adversely affected.

C5.4 The extra cleaning specified in Section 5.4.2 may be required because of surface hardness, limited blast media access, etc.

C5.4.4 Removal of the lubricant wax and non-absorbed dye (blue for mechanically galvanized and green for hot dip galvanized) has been easily accomplished with an alkaline household cleaner such as ammonia. Care should be taken to ensure that no residue from the cleaner remains after cleaning that may affect subsequent coating adhesion; typically ammonia will evaporate by the time the coating is applied. Any dye coloring remaining on galvanized nuts after weathering or the required surface preparation is not believed to be detrimental to subsequent coating performance or appearance. A white cloth wipe test with no color transfer can be used to confirm that all lubricant and non-absorbed dye has been removed, leaving only the residual "stain" on the surface.

C6.3. Under extreme exposure conditions, the Owner may specify stripe coating for intermediate and/or topcoat.

C Table 6.1, Line 13 and footnote 7 Stripe coating, when specified, is normally performed as outlined in SSPC-PA 1 subsection 6.6, "Striping," and in accordance with the coating manufacturer's recommendations. The purpose of the stripe coat is to assure that adequate coating thickness is deposited on designated surfaces. Apply the stripe coat either before or after the full prime coat, by spray application or by brush or roller in accordance with the coating manufacturer's recommendation. The stripe coat can be applied as allowed by the coating manufacturer's recommendation.

C Table 6.3, Line 18 The adhesion test confirms that the coating system achieves satisfactory adhesion. The test is destructive, but the value inherent in assuring system adhesion outweighs the cost of performing and repairing occasional adhesion test sites.

Due to the destructive nature of adhesion testing, alternate approaches are permitted to satisfy the requirement. For example, if a steel coupon is affixed to a member and cleaned and painted along with the member, the coupon can be the location of the adhesion test, avoiding coating damage and repairs on the permanent member.

Alternately, by attaching a dolly to the coated member and applying only enough pull on the dolly to reach a predetermined stress, and then removing the test equipment and leaving the dolly in place, the test does not have to be destructive. The dolly can be coated the same color as the structure.

2006 AASHTO BRIDGE COMMITTEE AGENDA ITEM: 21

SUBJECT: LRFD Bridge Design Specifications: Section 3, Various Articles

TECHNICAL COMMITTEE: T-15 Foundations

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

☒ **DESIGN SPEC** ☐ **CONSTRUCTION SPEC** ☐ **MOVABLE SPEC**
☐ **LRFR MANUAL** ☐ **OTHER**

☒ **US VERSION** ☐ **SI VERSION** ☐ **BOTH**

DATE PREPARED: 12/30/05
DATE REVISED:

AGENDA ITEM:

Item #1

In Article 3.2 "Definitions", add the following definition:

Apparent Earth Pressure – Lateral pressure distribution for anchored walls constructed from the top down.

Item #2

In Article 3.3.1 "Notations", add the following notation:

AEP = apparent earth pressure for anchored walls (ksf) (3.4.1)

Item #3

In Article 3.4.1, Table 3.4.1-2, after the second bullet under "EH: Horizontal Earth Pressure" add a third bullet and associated new load factors as follows:

	Maximum	Minimum
• AEP for anchored walls	1.35	N/A

Item #4

In Article 3.11.5.7, modify the title to read as follows:

"3.11.5.7 Apparent Earth Pressures~~s~~ (AEP) For Anchored Walls"

OTHER AFFECTED ARTICLES:

Section 11 (proposed as a separate agenda item).

BACKGROUND:

Currently, the LRFD specifications require that the load factor EH for active earth pressure (γ_p = 1.5), a load factor derived from calibration by fitting to allowable stress design (ASD) for design of gravity walls subjected to active earth pressure, be applied to the earth pressure for anchored walls. The apparent earth pressure used to design anchored walls was empirically derived to represent an upper bound (and therefore very conservative) envelope of pressures measured behind anchored walls, a much different situation than is the case for gravity walls subjected to active earth pressure. The current LRFD specifications do not differentiate between active earth pressure and apparent earth pressure with regard to application of load factors. Hence, a definition and notation have been added to Articles 3.2 and 3.3.1 to define apparent earth pressure separate from active earth pressure.

Furthermore, past allowable stress design practice for anchored walls as defined in the AASHTO Standard Specifications (2002) and FHWA manuals on anchored wall design (e.g., Sabatini, et al., 1999) required a total safety factor of 1.33 for determination of the number of ground anchors needed to resist the apparent earth pressure and 2.5 for the design of the soldier piles or shafts for vertical resistance in compression. The safety factor of 1.33 was derived from the practice of proof testing every anchor to 1.33 times the design load to the anchor calculated from the apparent earth pressure, AEP. As the current LRFD specifications, Section 11, Table 11.5.6-1, requires a resistance factor of 1.0 for the determination of the number and resistance of anchors required to resist the AEP, calibration by fitting to ASD implies that a load factor of 1.33 should be used. Similarly, for compression resistance design of the vertical wall elements (i.e., drilled in place soldier piles or shafts), since the current specifications require a resistance factor of 0.5 to 0.6 for shafts in sand or in an intermediate geomaterial (IGM), on average this implies a load factor of 1.35 should be used to be consistent with past allowable stress design practice. Therefore, for these two limit states, the currently required load factor to be applied to the AEP for anchored walls is more conservative than past practice.

Since the load factor for active earth pressure in the current LRFD specifications was originally derived using calibration by fitting to ASD, the fact that the design of anchored walls is more conservative than what would be obtained from past ASD practice is the result of forcing the use of a load factor that is not applicable to anchored wall design, not because the requirement for a more conservative design could be justified through statistical data and reliability theory.

Using statistical data gathered by Flaate (1966) and O'Rourke (1975) for anchored wall AEP, and statistical data for anchor pullout resistance gathered by D'Appolonia (1999), and using a resistance factor of 1.0 for anchor pullout as specified in the current LRFD specifications, calibrations conducted using reliability theory (Monte Carlo Method as described by Allen, et al., 2005) indicate that a load factor of 1.1 to 1.35, depending on the material in which the anchor is embedded, is adequate to achieve a target reliability index β of 3.5. Similarly, for vertical compression resistance of the vertical wall elements, using statistical data provided in Paikowsky, et al., 2004) for shafts embedded in mixed soils, using the currently specified resistance factor in Section 10 for drilled shafts in sand of 0.55, a load factor of 1.35 is needed to obtain the desired target reliability index.

Therefore, based on these analyses, it is recommended that a new load factor for AEP for anchored walls of 1.35 be added to Table 3.4.1-2, also noting that this recommended load factor will be more consistent with past design practice for anchored walls.

Note that the new load factor of 1.35 in combination with the resistance factor of 0.9 for flexural capacity of vertical wall elements in Table 11.5.6-1 will result in a more flexible beam for the vertical wall elements in anchored walls. To be consistent with past ASD practice (i.e., $0.55F_y$), a resistance factor of 0.75 would be needed in combination with the proposed load factor. To evaluate how much of a problem this is from the standpoint of reliability theory, reliability theory calibrations using the Monte Carlo Method were carried out. Beam flexural resistance statistics from Nowak (1999) in combination with the AEP statistics cited above were used. The results of these analyses indicate that a resistance factor of approximately 1.0 should be used in combination with the proposed load factor of 1.35 to achieve the target reliability index of 3.5. Therefore, statistically, it is justified to leave the resistance factor for beam flexural resistance at 0.9 as shown in Table 11.5.6-1.

ANTICIPATED EFFECT ON BRIDGES:

None

REFERENCES:

Allen, T, Nowak, A, and Bathurst, R. (2005). *Calibration to Determine Load and Resistance Factors for Geotechnical and Structural Design.* TRB Circular E-C079, 83 pp.

AASHTO (2002). *Standard Specifications for Highway Bridges.* American Association of State Highway and Transportation Officials, Seventeenth Edition, Washington, D.C., USA.

D'Appolonia, 1999, *Developing New AASHTO LRFD Specifications for Retaining Walls*, Report for NCHRP Project 20-7, Task 88, 63 pp.

Paikowsky, S. G., with contributions from Birgisson, B., McVay, M., Nguyen, T., Kuo, C., Baecher, G., Ayyab, B., Stenersen, K., O'Malley, K., Chernauskas, L., and O'Neill, M. (2004). *Load and Resistance Factor Design (LRFD) for Deep Foundations*. NCHRP (Final) Report 507, Transportation Research Board, Washington, DC, 126 pp.

Flaate, K.S., 1966, *Stresses and Movements in Connection with Braced Cuts in Sand and Clay*, Ph.D Dissertation, University of Illinois, Urbana, 265 pp.

O'Rourke, T.D., 1975, A study of two Braced Excavations in Sand and Interbedded Stiff Clay, Ph.D Dissertation, University of Illinois, Urbana, 255 pp.

Nowak, A. S. (1999). *Calibration of LRFD Bridge Design Code*. NCHRP Report 368, Transportation Research Board, Washington, DC.

Sabatini, P.J., Pass, D.G., and Bachus, R. C., 1999, Geotechnical Engineering Circular No. 4 – Ground Anchors and Anchored Systems, Federal Highway Administration, Report No. FHWA-SA-99-015, NTIS, Springfield, Virginia, 281 pp.

OTHER:

None

2006 AASHTO BRIDGE COMMITTEE AGENDA ITEM: 22

SUBJECT: LRFD Bridge Design Specifications: Section 11, Article 11.5.6

TECHNICAL COMMITTEE: T-15 Foundations

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

☒ **DESIGN SPEC** ☐ **CONSTRUCTION SPEC** ☐ **MOVABLE SPEC**
☐ **LRFR MANUAL** ☐ **OTHER**

☒ **US VERSION** ☐ **SI VERSION** ☐ **BOTH**

DATE PREPARED: 12/30/05
DATE REVISED:

AGENDA ITEM:

Item #1

In Article 11.5.6, Table 11.5.6-1, 3rd row from the top of the table, change "Bearing resistance of vertical elements" to "Axial compressive resistance of vertical elements".

Item #2

In Article 11.5.6, Table 11.5.6-1, 4th row from the top of the table, change the resistance factor for passive resistance of vertical elements from "1.0" to "0.75".

Item #3

In Article 11.5.6, Table 11.5.6-1, Note (2), revise as follows:

(2)Apply where proof test(s) are conducted on every production anchor to a load of 1.0 or greater times the factored ~~design~~ load on the anchor.

OTHER AFFECTED ARTICLES:

Section 3 (proposed as a separate agenda item) and Article 11.9.4.2 (proposed as a separate agenda item).

BACKGROUND:

The proposed change in terminology will make the terminology used this table consistent with the recently rewritten and approved Section 10.

The resistance factor change is necessary to accommodate the change in the load factor to 1.35 as recommended in the proposed agenda item for Article 3.4.1. Calibration by fitting to allowable stress design practice to design anchored walls for passive resistance results in a resistance factor of 0.75 to 0.9, corresponding to a safety factor of 2.0 to 1.5, respectively. Using statistical passive resistance data for discrete elements buried in sand from Chen and Kulhawy (1994), apparent earth pressure statistical data from Flaate (1966) to represent the load, and reliability theory using the Monte Carlo Method, a resistance factor of 0.75 provides a reliability index, β, of approximately 3.5, which is consistent with the design of the rest of the wall elements. In any case, the resistance factor of 1.0 for passive resistance of the vertical wall elements currently provided in this table is clearly unconservative and needs to be decreased. Therefore, a resistance factor of 0.75 is recommended to be consistent with past design practice and the limited reliability analyses conducted as described above.

The changes proposed for Note (2) below Table 11.5.6-1 make the wording in the note consistent with the wording proposed for Article11.9.4.2 in a separate agenda item.

ANTICIPATED EFFECT ON BRIDGES:

None

REFERENCES:

Chen, Y.-J., and Kulhawy, F.H., 1994, *Case History Evaluation of the Behavior of Drilled Shafts Under Axial and Lateral Loading*, Report TR-104601, Electric Power Research Institute, Palo Alto, 392 pp.

Flaate, K.S., 1966, *Stresses and Movements in Connection with Braced Cuts in Sand and Clay*, Ph.D Dissertation, University of Illinois, Urbana, 265 pp.

OTHER:

None

2006 AASHTO BRIDGE COMMITTEE AGENDA ITEM: 23

SUBJECT: LRFD Bridge Design Specifications: Section 11, Articles 11.9.4.2 and 11.9.8.1

TECHNICAL COMMITTEE: T-15 Foundations

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

☒ **DESIGN SPEC** ☐ **CONSTRUCTION SPEC** ☐ **MOVABLE SPEC**
☐ **LRFR MANUAL** ☐ **OTHER**

☒ **US VERSION** ☐ **SI VERSION** ☐ **BOTH**

DATE PREPARED: 12/30/05
DATE REVISED:

AGENDA ITEM:

Item #1

In Article 11.9.4.2, remainder of paragraph after the list of variables for Equation 11.9.4.2-1, revise to read as follows:

For preliminary design, the resistance of anchors may either be based on the results of anchor pullout load tests; estimated based on a review of geologic and boring data, soil and rock samples, laboratory testing and previous experience; or estimated using published soil/rock-grout bond guidelines. For final design, the contract documents ~~shall~~ may require ~~that verification~~ preproduction tests ~~or pullout tests~~ such as pullout tests or extended creep tests on sacrificial anchors ~~in each soil unit~~ be conducted to establish anchor lengths and capacities that are consistent with the contractor's chosen method of anchor installation. ~~and then conduct~~ Either performance or proof tests shall be conducted on every production anchor to 1.0 or greater times the factored ~~design~~ load to verify capacity.

Item #2

In Article C11.9.4.2, third paragraph, revise the first sentence as follows:

The resistance factors in Table 11.5.6-1, in combination with the load factor *EH* for ~~horizontal active~~ apparent earth pressure for anchored walls (Table 3.4.1-2), are consistent with what would be required based on allowable stress design, for preliminary design of anchors for pullout (see *Sabatini, et al., 1999*).

Item #3

In Article C11.9.4.2, third paragraph, revise the third sentence as follows:

Use of the resistance factors in Table 11.5.6-1 and the load factor for ~~horizontal active~~ apparent earth pressure for anchored walls in Table 3.4.1-2, with the values of presumptive ultimate unit bond stresses other than the minimum values in Tables C1 through C3 could result in unconservative designs unless the Engineer has previous experience with the particular soil or rock unit in which the bond zone will be established.

Item #4

In Article C11.9.4.2, delete the fifth and sixth paragraphs and replace them as follows:

~~Additional verification testing of the anchor zone soil or rock units should be considered if the preliminary~~

~~anchor design using unit bond stresses provided in the tables above indicate that anchored walls are marginally infeasible. This may occur due to lack of adequate room laterally to accommodate the estimated anchor length within the available right of way or easement.~~

~~Every anchor should be performance tested, proof tested, and/or creep tested during construction to permit evaluation of its expected short-term and long-term load-carrying capacity.~~

See Article 11.9.8.1 for guidance on anchor testing.

Item #5

At the end of Article 11.9.8.1, add the following new paragraph:

At the end of the testing of each production anchor, the anchor should be locked off to take up slack in the anchored wall system to reduce post-construction wall deformation. The lock-off load should be determined and applied as described in *AASHTO LRFD Bridge Construction Specifications*, Article 6.5.5.6.

Item #6

In Article C11.9.8.1, modify the first paragraph as follows and add a new second paragraph as follows:

Common anchor load tests include ~~performance and/or creep~~ pullout tests performed on ~~a selected number of anchors~~ sacrificial preproduction anchors, and creep, performance, and proof tests performed on ~~all other~~ the production anchors. None of the~~se~~ production anchor tests determine the actual ultimate anchor load capacity.~~;~~ ~~t~~The production anchor test results only provide an indication of serviceability under a specified load. Performance tests consist of incremental loading and unloading of anchors to verify sufficient capacity to resist the test load, verify the free length and evaluate the permanent set of the anchor. Proof tests, usually performed on each production anchor, consist of a single loading and unloading cycle to verify sufficient capacity to resist the test load and prestress the anchor. Creep tests, recommended for cohesive soils with a plasticity index greater than 20 percent or a liquid limit greater than 50 percent, and highly weathered, soft rocks, consist of incremental, maintained loading of anchors to assess the potential for loss of anchor bond capacity due to ground creep.

Pullout tests should be considered in the following circumstances:

- If the preliminary anchor design using unit bond stresses provided in the tables above indicate that anchored walls are marginally infeasible, requiring that a more accurate estimate of anchor capacity be obtained during wall design. This may occur due to lack of adequate room laterally to accommodate the estimated anchor length within the available right of way or easement;

- If the anticipated anchor installation method or soil/rock conditions are significantly different than those assumed to develop the presumptive values in Tables C11.9.4.2-1 through C11.4.9.2-3 and inadequate site specific experience is available to make a reasonably accurate estimate of the soil/rock-grout anchor bond stresses.

Item #7

At the end of Article C11.9.8.1, add the following paragraphs:

Note that the test details provided in the *AASHTO LRFD Bridge Construction Specifications*, Article 6.5.5, at least with regard to the magnitude of the incremental test loads, were developed for allowable stress design. These incremental test loads should be divided by the load factor for apparent earth pressure for anchored walls provided in Table 3.4.1-2 when testing to factored anchor loads.

Typically, the anchor lock-off load is equal to 80 to 100 percent of the nominal (unfactored) anchor load to insure that the slack in the anchored wall system is adequately taken up so that post-construction wall deformation is minimized. However, a minimum lock-off load of 50 percent is necessary to properly engage strand anchor head wedges.

OTHER AFFECTED ARTICLES:

Section 3 (proposed as a separate agenda item).

BACKGROUND:

Regarding the changes to anchor testing requirements provided in Article 11.9.4.2, the article as previously written could be misunderstood to indicate that anchor pullout tests or verification tests, both of which must be conducted on sacrificial pre-production anchors, must be conducted in all cases. This is not consistent with current national practice. Furthermore, the term "verification test" is not found in available national guidance documents on anchor design and testing. The proposed changes reflect the anchor testing guidance found in national guidance documents. The changes to Article C11.9.4.2 were necessary to be consistent with the changes in Article 11.9.4.2. Furthermore, the paragraphs deleted conflicted with provisions in Article C11.9.8.1. A reference to Article C11.9.8.1 was added to Article C11.9.4.2 to insure consistency between these articles.

The changes to the third paragraph in Article C11.9.4.2 as proposed makes reference to the load factor for design of anchored walls consistent with the proposed new load factor for apparent earth pressure provided in a separate 2006 agenda item.

Regarding the addition to Article 11.9.8.1 on the application of anchor lock-off loads, national practice is to lock off anchors after testing is complete to take up slack in the anchored wall system. The current specifications provide no guidance on this issue.

Regarding the changes to Article C11.9.8.1, the wording appears to imply that pullout tests should be conducted on some of the anchors in a wall, which is not consistent with national practice. The proposed changes are consistent with changes made to Article C11.9.4.2 as described above. Additional guidance is also provided as to when pullout tests should be considered. Additional guidance on the determination of lock-off loads is also provided, as the current specifications only refer to the contract documents for guidance on the magnitude of the lock of load needed.

ANTICIPATED EFFECT ON BRIDGES:

None

REFERENCES:

Sabatini, P.J., Pass, D.G., and Bachus, R. C., 1999, Geotechnical Engineering Circular No. 4 – Ground Anchors and Anchored Systems, Federal Highway Administration, Report No. FHWA-SA-99-015, NTIS, Springfield, Virginia, 281 pp.

OTHER:

None

2006 AASHTO BRIDGE COMMITTEE AGENDA ITEM: 24 (REVISION 1)

SUBJECT: LRFD Bridge Design Specifications: Section 8, Wood Structures

TECHNICAL COMMITTEE: T-16 Timber Structures

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

☒ **DESIGN SPEC** ☐ **CONSTRUCTION SPEC** ☐ **MOVABLE SPEC**
☐ **LRFR MANUAL** ☐ **OTHER**

☒ **US VERSION** ☐ **SI VERSION** ☐ **BOTH**

DATE PREPARED: 1-17-06
DATE REVISED: 5-10-06

AGENDA ITEM:

Section 8 – Updated Section **(REVISION 1 TO ATTACHMENT - PAGE 33)**

Existing tables that were replaced with new tables (note: only the title has been underlined):

8.4.1.1.4-1
8.4.1.1.4-2
8.4.1.2.3-1
8.4.1.2.3-2

New tables (note: only the title has been underlined):

8.4.4.4-2
8.4.4.6-1
8.4.4.6-2
8.4.4.7-1
8.4.4.8-2
8.4.4.9-1

No existing tables were deleted without a replacement.

OTHER AFFECTED ARTICLES:

None

BACKGROUND:

The wood industry introduced the new edition of the National Design Specification (NDS 2005) with new values of properties of wood components. Traditionally, the AASHTO LRFD has referred to the specification adopted by the wood industry. Therefore, there is an urgent need to update the AASHTO LRFD Code and make it consistent with NDS 2005. The new edition of NDS (2005) includes major changes compared to the previous editions, in particular, the strength of wood is specified in terms of allowable stress.

ANTICIPATED EFFECT ON BRIDGES:

The proposed changes will affect the bridge design procedures (new tables and formulas), but no changes in the final results are expected.

REFERENCES:

AF&PA. 2005. *National Design Specification® (NDS®) for Wood Construction.* American Forest & Paper Association, Washington DC.

AITC. 2001. *Standard Appearance Grades for Structural Glued Laminated Timber.* AITC 110-2001. American Institute of Timber Construction. Centennial, CO.

AITC. 1996. *Standard Specifications for Structural Glued Laminated Timber of Hardwood Species.* AITC 119-96. American Institute of Timber Construction. Centennial, CO.

AITC. 2004. *Standard Specifications for Structural Glued Laminated Timber of Softwood Species.* AITC 117-2004. American Institute of Timber Construction. Centennial, CO.

AITC. 2002. *Structural Glued Laminated Timber.* ANSI/AITC A190.1. American Institute of Timber Construction. Centennial, CO.

OTHER:

None

AASHTO LRFD Code, Section 8

WOOD STRUCTURES
(Proposed Revisions 4-18-06)

8.1 SCOPE

This section specifies design requirements for structural components made of sawn lumber products, stressed wood, glued laminated timber, wood piles, and mechanical connections.

8.2 DEFINITIONS

~~*Base Resistance*~~ *Reference Design Value* – The allowable stress value or modulus of elasticity specified in the NDS ~~resistance for wet use conditions and two-month load duration for wood products treated with preservatives.~~

Beams and Stringers (B&S) – Beams and stringers are rectangular pieces that are 5.0 or more in. (nominal) thick, with a depth more than 2.0 in. (nominal) greater than the thickness. B&S are graded primarily for use as beams, with loads applied to the narrow face.

Bent – A type of pier consisting of two or more columns or column-like components connected at their top ends by a cap, strut, or other component holding them in their correct positions.

Cap – A sawn lumber or glulam component place horizontally on an abutment or pier to distribute the live load and dead load of the superstructure. Also, a metal, wood, or mastic cover to protect exposed wood end grain from wetting.

Combination Symbol – A product designation used by the structural glued laminated timber industry, see ~~Ritter (1990)~~ AITC 117-2004.

Crib – A structure consisting of a foundation grillage and a framework providing compartments that are filled with gravel, stones, or other material satisfactory for supporting the structure to be placed thereon.

Decking – A subcategory of dimension lumber, graded primarily for use with the wide face placed flatwise.

Delamination – Adhesive failure causing the ~~The~~ separation of laminations.

Diaphragm – Blocking between two main longitudinal beams consisting of solid lumber, glued laminated timber, or steel X-bracing.

Dimension Lumber – Lumber with a nominal thickness of from 2.0 up to but not including 5.0 in. and having a nominal width of 2.0 in. or more.

Dowel – A relatively short length of round metal bar used to interconnect or attach two wood components in a manner to minimize movement and displacement.

Dressed Lumber –Lumber that has been surfaced by a planing machine on one or more sides or edges.

Dry – The condition of having relatively low moisture content, i.e., not more than 19 percent for sawn lumber and 16 percent for glued laminated timber.

Frame Bent – A type of framed timber superstructure.

Structural Glued Laminated Timber (glulam) – An engineered, stress-rated product of a timber laminating plant comprised of assemblies of specially selected and prepared wood laminations securely bonded together with adhesives. The grain of all laminations is approximately parallel longitudinally. Glued laminated timber is permitted to be comprised of pieces end joined to form any length, of pieces placed or bonded edge to edge to make any width, or of pieces bent to curved form during bonding.

Grade – The designation of the material quality of a manufactured piece of wood.

Grade Mark – The identification of lumber with symbols or lettering to certify its quality or grade.

Grain – The direction, size arrangement, appearance, or quality of the fibers in wood or lumber.

Green Wood – A freshly sawn or undried wood. Wood that has become completely wet after immersion in water would not be considered green but may be said to be in the green condition.

Hardwood – Generally one of the botanical groups of trees that have broad leaves or the wood produced by such trees. The term has no reference to the actual hardness of the wood.

Horizontally Laminated Timber – Laminated wood in which the laminations are arranged with their wider dimension approximately perpendicular to the direction of applied transverse loads.

Laminate – A product made by bonding two or more layers (laminations) of material or materials.

Laminated Wood – An assembly made by bonding layers of veneer or lumber with an adhesive, nails, or stressing to provide a structural continuum so that the grain of all laminations is essentially parallel.

Laminating – The process of bonding lamination together with adhesive, including the preparation of the laminations, preparation and spreading of adhesive, assembly of laminations in packages, application of pressure, and curing.

Lamination – A full width and full length layer contained in a component bonded together with adhesive. The layer itself may be composed of one or several wood pieces in width or length.

Machine Evaluated Lumber (MEL) – Mechanically graded lumber certified as meeting the criteria of a specific commercial grading system.

Machine Stress Rated (MSR) Lumber – ~~A grade of structural lumber determined by measuring the stiffness of each piece by use of a grading machine~~ Mechanically graded lumber certified as meeting the criteria of a specific commercial grading system.

Mechanically Graded Lumber – Solid sawn lumber graded by mechanical evaluation in addition to visual examination.

Modulus of Rupture (MOR) – The maximum stress at the extreme fiber in bending, calculated from the maximum bending moment on the basis of an assumed stress distribution.

Moisture Content ~~(MC)~~ – An indication of the amount of water contained in the wood, usually expressed as a percentage of the weight of the oven dry wood.

NDS – National Design ~~Standards~~ Specification by the ~~National Forest Products~~ American Forest & Paper Association.

NELMA – Grading rules by Northeastern Lumber Manufacturers Association.

NLGA – Grading rules by National Lumber Grades Authority.

Net Size – The size used in design to calculate the resistance of a component. Net size is close to the actual dry size.

NSLB – Grading rules by Northern Softwood Lumber Bureau.

Oil-Borne Preservative – A preservative that is introduced into wood in the form of an oil-based solution.

Plank – A broad board, usually more than 1.0 in. thick laid with its wide dimension horizontal and used as a bearing surface or riding surface.

Posts and Timber (P&T) – Posts and timbers pieces with a square or nearly square cross-section, 5.0 by 5.0 in. (nominal) and larger, with the width not more than 2.0 in. (nominal) greater than the thickness. Lumber in the P&T size classification is graded primarily for resisting axial loads.

Preservative – Any substance that is effective in preventing the development and action of wood-decaying fungi, borers of various kinds, and harmful insects.

Rough~~-~~Sawn Lumber – Lumber that has not been dressed but that has been sawn, edged, and trimmed.

Sawn Lumber – The product of a sawmill not further manufactured other than by sawing, resawing, passing lengthwise through a standard planing mill, drying, and cross-cutting to length.

Softwood – Generally, one of the conifers or the wood produced by such trees. The term has no reference to the actual hardness of the wood.

~~Specified Resistance~~ *Adjusted Design Value* – Reference design value multiplied by applicable adjustment factors ~~Nominal resistance adjusted by time effect factor, size effect factor, wet service factor, and deck factor.~~

SPIB – Grading rules by Southern Pine Inspection Bureau.

Stress Grades – Lumber grades having assigned working stress and modulus of elasticity in accordance with accepted principles of resistance grading.

Structural Lumber – ~~A l~~ Lumber that has been graded and assigned design values based on standardized procedures to ensure acceptable reliability~~. is intended for use where predictable material properties are required~~.

Sawn Timbers – Lumber that is nominally 5.0 in. or more in least dimension.

Vertically Laminated Timber – Laminated wood in which the laminations are arranged with their wider dimension approximately parallel to the direction of load.

Visually ~~Stress~~ Graded Lumber – Structural lumber graded solely by visual examination.

Waterborne Preservative – A preservative that is introduced into wood in the form of a water-based solution.

WCLIB – Grading rules by West Coast Lumber Inspection Bureau.

Wet-Use – Use conditions where the moisture content of the wood in service exceeds 16 percent for glulam and 19 percent for sawn lumber.

WWPA – Grading rules by Western Wood Products Association.

8.3 NOTATION

A	=	parameter for beam stability (8.6.2)
A_b	=	bearing area (in.^{2}) (8.8.3)
A_g	=	gross cross-sectional area of the component (in.^{2}) (8.8.2)
A_n	=	net cross-sectional area of the component (in.^{2}) (8.9)
a	=	coefficient (8.4.4.~~2~~5)
B	=	parameter for compression (8.8.2)
b	=	width of the glued laminated timber component, thickness of lumber component (see Figure 1) (in.) (8.4.4.~~2~~5)
C_b	=	bearing factor (8.8.3)

C_c	=	curvature factor (8.4.1.2)
~~C_D~~C_d	=	deck factor (8.4.4.~~1~~8)
C_F	=	~~modification~~ size factor ~~for size effects~~ (8.4.4.~~1~~4)
C_{fu}	=	flat use factor (8.4.4.6)
C_i	=	incising factor (8.4.4.7)
C_{KF}	=	format conversion factor (8.4.4.2)
~~C_s~~C_L	=	beam stability factor (8.6.2)
C_M	=	wet service factor (8.4.4.~~1~~3)
C_P	=	column stability factor (8.8.2)
C_V	=	volume factor (8.4.4.5)
C_λ	=	time effect factor (8.4.4.9)
d	=	depth of the beams or stringers or width of the dimension lumber component (8.4.4.3) or glulam depth (8.4.4.4) as shown in Figure 1 (in.) ~~(8.4.4.2)~~
E	=	~~specified~~ adjusted modulus of elasticity (ksi) (8.4.4.1)
E_o	=	~~base~~ reference modulus of elasticity (ksi) (8.4.1.1.4)
~~E_x~~	~~=~~	~~modulus of elasticity parallel to grain (ksi) (8.4.1.2.3)~~
~~E_y~~	~~=~~	~~modulus of elasticity perpendicular to grain (ksi) (8.14.1.2.3)~~
F	=	~~nominal resistance~~ adjusted design value (ksi) (8.4.4.1)
F_b	=	~~specified resistance~~ adjusted design value in flexure (ksi) (~~8.4.1.2.3~~8.4.4.1)
F_{bo}	=	~~base resistance~~ reference design value of wood in flexure (ksi) (8.4.1.1.4)
F_c	=	~~specified resistance~~ adjusted design value of wood in compression parallel to grain (ksi) (8.4.4.1)
F_{co}	=	~~base resistance~~ reference design value of wood in compression parallel to grain (ksi) (8.4.1.1.4)
F_{cp}	=	~~specified resistance~~ adjusted design value of wood in compression perpendicular to grain (ksi) (8.4.4.1)
F_{cpo}	=	~~base resistance~~ reference design value of wood in compression perpendicular to grain (ksi) (8.4.1.1.4)
F_o	=	~~base resistance~~ reference design value (ksi) (8.4.4.1)
F_t	=	~~specified resistance~~ adjusted design value of wood in tension (ksi) (8.4.4.1)
F_{to}	=	~~base resistance~~ reference design value of wood in tension (ksi) (8.4.1.1.4)
F_v	=	~~specified resistance~~ adjusted design value of wood in shear (ksi) (8.4.4.1)
F_{vo}	=	~~base resistance~~ reference design value of wood in shear (ksi) (~~8.4.4.1~~8.4.1.1.4)
~~f~~	~~=~~	~~stress level used in grade designation (ksi) (8.4.1.1.4)~~
G	=	specific gravity (8.4.1.1.4)
K	=	effective buckling length factor (8.8.2)
L	=	length (ft.) (8.4.4.~~2~~5)
L_e	=	effective length (in.) (8.6.2)
L_u	=	laterally unsupported length of the component (in.) (8.6.2)
M_n	=	nominal flexural resistance (kip-in.) (8.6~~.1~~)
M_r	=	factored flexural resistance, ϕM_n (kip-in.) (8.6~~.1~~)
M_u	=	factored moment (kip-in.) (8.10~~.1~~)
P_n	=	nominal compression or tension resistance (kip) (8.8~~.1~~, 8.9)
P_r	=	factored axial resistance (kip) (8.8~~.1~~, 8.9)
P_u	=	factored axial load (kip) (8.10~~.1~~)
S	=	section modulus (in.^{3}) (8.6.2)
V_n	=	nominal shear resistance (kip) (8.7)
V_r	=	factored shear resistance, ϕV_n (kip) (8.7)
ϕ	=	resistance factor (8.5.2.2)

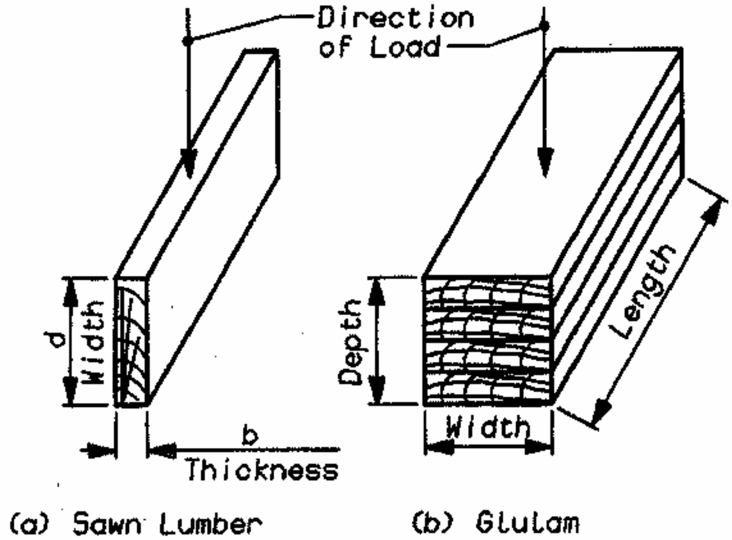


Figure 8.3-1 Dimensions as Defined for Various Types of Wood Products.

8.4 MATERIALS

8.4.1 Wood Products

Nominal resistance for wood products shall be based on specified size and conditions of use with respect to moisture content and time effect. To obtain nominal resistance and stiffness values for design, the ~~base~~ reference design values specified in Tables 8.4.1.1.4-1, 8.4.1.1.4-2, 8.4.1.1.4-3, 8.4.1.2.3-1, 8.4.1.2.3-2, and 8.4.1.3-1 shall be adjusted for actual conditions of use in accordance with Article 8.4.4.

C8.4.1

~~Base~~ Reference design values are based on ~~wet~~dry-use conditions, with the wood moisture content not exceeding 19 percent for sawn lumber and 16 percent for structural glued laminated timber. ~~Base~~ Reference design values are applied to material preservatively treated in accordance with AASHTO M133.

Reference design values have been taken from the National Design Specification (NDS) for Wood Construction. The NDS publishes reference values for allowable stress design (ASD) and provides format conversion factors for use of these values with the load and resistance factor design (LRFD) methodology. To facilitate the direct use of the values developed by the wood products industry and included in the NDS, the same format has been adopted for AASHTO LRFD design.

8.4.1.1 Sawn Lumber

8.4.1.1.1 General

Sawn lumber shall comply with the requirements of AASHTO M168.

When solid sawn beams and stringers are used as continuous or cantilevered beams, the grading provisions applicable to the middle third of the length shall be applied to at least the middle two-thirds of the length of pieces to be used as two-span continuous beams and to the entire length of pieces to be used over three or more spans or as cantilevered beams.

8.4.1.1.2 Dimensions

Structural calculations shall be based on the actual net dimensions for the anticipated use conditions.

Dimensions stated for dressed lumber shall be the nominal dimensions. Net dimensions for dressed lumber shall be taken as 0.5 in. less than nominal, except that the net width of dimension lumber exceeding 6.0 in. shall be taken as 0.75 in. less than nominal.

For rough-sawn, full sawn, or special sizes, the actual dimensions and moisture content used in design shall be indicated on the contract documents.

C8.4.1.1.2

These net dimensions depend on the type of surfacing, whether dressed, rough-sawn, or full-sawn.

The designer should specify surface requirements on the plans. Rough-sawn lumber is typically 0.125 in. larger than standard dry dressed sizes, associated with the "F_{bo}" value in Table 8.4.1.1.4-2, and full-sawn lumber, which is not widely used, ~~and~~ is cut to the same dimensions as the nominal size. In both of the latter cases, thickness and width dimensions are variable, depending on the sawmill equipment. Therefore, it is impractical to use rough-sawn or full-sawn lumber in a structure that requires close dimensional tolerances.

For more accurate dimensions, surfacing can be specified on one side (S1S), two sides (S2S), one edge (S1E), two edge (S2E), combinations of sides and edges (S1S1E, S2S1E, S1S2E) or all sides (S4S).

8.4.1.1.3 Moisture Content

The moisture content of dimension lumber ~~4.0 in. or less in nominal thickness~~ shall not be greater than 19 percent at the time of installation.

8.4.1.1.4 ~~Base Resistance~~ Reference Design Values ~~and Modulus of Elasticity~~

~~Base resistance~~ Reference design values ~~and modulus of elasticity~~ for visually graded sawn lumber shall be as specified in Table 1.

~~Base resistance~~ Reference design values ~~and modulus of elasticity~~ for mechanically graded dimension lumber shall be as specified in Table 2.

Unless otherwise indicated, ~~base resistance~~ reference design value in flexure for dimension lumber and posts and timbers shall apply to material where the load is applied to either the narrow or wide face. ~~base resistance~~ Reference design value in flexure for decking grades shall apply only with the load applied to the wide face. ~~Base resistance in flexure for beam and stringer grades shall apply only with the load applied to the narrow face.~~

Values for specific gravity, G, shear parallel to grain, F_v, and compression perpendicular to grain, F_{cpo}, for mechanically graded dimension lumber shall be taken as specified in Table 3. For species or species groups not given in Table 3, the G, F_{vo} and F_{cpo} values for visually graded lumber may be used.

~~Base resistance~~Reference design values for lumber grades not given in Table 1 and Table 2 shall be obtained from the National Design Specification® (NDS®) for Wood Construction.

Where the "E_o" or "F_{to}" values shown on a grade stamp differ from Table 2 values associated with the "F_{bo}" on the grade stamp, the values on the stamp shall be used in design, and the "F_{co}" value associated

C8.4.1.1.4

~~NDS allowable stresses are provided for ten-year load duration and dry use. Factors listed inthis article transform allowable stresses to the lower 5th percentile of the ultimate stress for two-month load duration and wet-use.~~

In calculating design values in Table 2, the natural gain in strength and stiffness that occurs as lumber dries has been taken into consideration as well as the reduction in size that occurs when unseasoned with the "F_{bo}" value in Table 2 shall be used.

For machine evaluated lumber (MEL) commercial grades M-17, M-20 and M-27, F_{co}, requires qualification and quality control shall be required.

Reference design values specified in Table 2 shall be taken as applicable to lumber that will be used under dry conditions. For 2.0 in. to 4.0 in. thick lumber, the dry dressed sizes shall be used regardless of the moisture content at the time of manufacture or use.

lumber shrinks. The gain in load carrying capacity due to increased strength and stiffness resulting from drying more than offsets the design effect of size reductions due to shrinkage.

For any given bending design value, F_{bo}, the modulus of elasticity, E_o, and tension parallel to grain, F_{to}, design value may vary depending upon species, timber source or other variables. The "E_o" and "F_{to}" values included in the "F_{bo}-E_o" grade designations in Table 2 are those usually associated with each "F_{bo}" level. Grade stamps may show higher or lower values if machine rating indicates the assignment is appropriate.

Higher G values may be claimed when (a) specifically assigned by the rules writing agency or (b) when qualified by test, quality controlled for G and provided for on the grade stamp. When a different G value is provided on the grade stamp, higher F_{vo} and F_{cpo} design values may be calculated in accordance with the grading rule requirements.

8.4.1.2 Structural Glued Laminated Timber (Glulam)

8.4.1.2.1 General

Structural ~~Glued~~ glued laminated timber shall be manufactured using wet-use adhesives and shall comply with the requirements of ~~AASHTO M 168~~ ANSI/AITC A190.1-2002. Glued laminated timber may be manufactured from any lumber species, provided that it meets the requirements of ~~AASHTO M 168~~ ANSI/AITC A190.1 and is treatable with wood preservatives in accordance with the requirements of Article 8.4.3.

~~Wet-use adhesives shall conform with the requirements of ASTM D2559 *Standard Specification for Adhesives for Structural Laminated Wood Products for Use Under Exterior (Wet-Use) Exposure Conditions.*~~

The contract documents shall require that each piece of glued laminated timber be distinctively marked and provided with a Certificate of Conformance by an accredited ~~qualified~~ inspection and testing agency, indicating that the requirements of ~~AASHTO M 168~~ ANSI/AITC A190.1 have been met and that straight or slightly cambered bending members have been stamped TOP on the top at both ends so that the ~~natural~~ camber, if any, shall be positioned opposite to the direction of applied loads.

Industrial appearance grade, as defined in AITC ~~#~~-110-2001 *Standard Appearance Grades for Structural Glued Laminated Timber* shall be used, unless otherwise specified.

C8.4.1.2.1

When wet-use adhesives are used, the bond between the laminations, which is stronger than the wood, will be maintained under all exposure conditions. Dry-use adhesives will deteriorate under wet conditions. For bridge applications, it is not possible to ensure that all areas of the components will remain dry. ANSI/AITC A190.1-2002 requires the use of wet-use adhesives for the manufacture of structural glued laminated timber.

~~Glued laminated timber can be manufactured to virtually any shape or size. The most efficient and economical design generally results when standard sizes are used. Acceptable manufacturing tolerances are given in AASHTO M 168.~~

Structural ~~G~~glued laminated timber is ~~normally~~ available in ~~three~~ four standard appearance grades: framing, industrial, architectural, and premium. Architectural and premium grades are typically planed or sanded, and exposed irregularities are filled with a wood filler that may crack and dislodge under exterior exposure conditions. Framing grade is surfaced hit-or-miss to produce a timber with the same net width as standard lumber for concealed applications where matching the width of framing lumber is important.

Framing grade is not typically used for bridge applications. In addition to the four standard appearance grades, certain manufacturers will use special surfacing techniques to achieve a desired look, such as a rough sawn look. Individual manufacturers should be contacted for details.

8.4.1.2.2 Dimensions

Dimensions stated for glued laminated timber shall be taken as the actual net dimensions.

In design, structural calculations shall be based on the actual net dimensions. Net width of structural glued laminated timber shall be as specified in Table 1 or other dimensions as agreed upon by buyer and seller.

Table 8.4.1.2.2-1 Net Dimensions of Glued Laminated Timber.

Nominal width of laminations ~~Dimension~~ (in.)	Western Species Net Finished Dimension (in.)	Southern Pine Net Finished Dimension (in.)
3	2 1/8 or 2 1/2	2 1/8 or 2 1/2
4	3 1/8	3.0 or 3 1/8
6	5 1/8	5.0 or 5 1/8
8	6 3/4	6 3/4
10	8 3/4	8 1/2
12	10 3/4	10 1/2
14	12 1/4	12.0
16	14 1/4	14.0

~~Net thickness of a lamination shall be determined using the dry dimensions for sawn lumber as specified in Article 8.4.1.1.2.~~ The total glulam net depth shall be taken as the product of the thickness of the laminations and the number of laminations.

C8.4.1.2.2

Structural glued laminated timber can be manufactured to virtually any shape or size. The most efficient and economical design generally results when standard sizes are used. Acceptable manufacturing tolerances are given in ANSI/AITC A190.1-2002.

The use of standard sizes constitutes good practice and is recommended whenever possible. Nonstandard sizes should only be specified after consultation with the laminator.

Southern Pine timbers are typically manufactured from 1.375 in. thick laminations, while timbers made from Western Species and Hardwoods are commonly manufactured from 1.5 in. thick laminations. Curved members may be manufactured from thinner laminations depending on the radius of curvature. Radii of curvature of less than 27.0 ft. 6.0 in. normally require the use of thinner laminations.

Table 8.4.1.1.4-1 Reference Design Values for Visually Graded Sawn Lumber.

Species and commercial grade	Size classification	Design Values (ksi) Bending	Tension parallel to grain	Shear parallel to grain	Compression perpendicular to grain	Compression parallel to grain	Modulus of Elasticity	Grading Rules Agency
		F_{bo}	F_{to}	F_{vo}	F_{cpo}	F_{co}	E_o	
DOUGLAS FIR-LARCH								
Select Structural	Dimension 2 in. & wider	1.50	1.00	0.18	0.625	1.70	1,900	WCLIB WWPA
No. 1 & Btr		1.20	0.80	0.18	0.625	1.55	1,800	
No. 1		1.00	0.675	0.18	0.625	1.50	1,700	
No. 2		0.90	0.575	0.18	0.625	1.35	1,600	
Dense Select Structural	Beams and Stringers	1.90	1.10	0.17	0.73	1.30	1,700	WCLIB
Select Structural		1.60	0.95	0.17	0.625	1.10	1,600	
Dense No. 1		1.55	0.775	0.17	0.73	1.10	1,700	
No. 1		1.35	0.675	0.17	0.625	0.92	1,600	
No. 2		0.875	0.425	0.17	0.625	0.60	1,300	
Dense Select Structural	Posts and Timbers	1.75	1.15	0.17	0.73	1.35	1,700	
Select Structural		1.50	1.00	0.17	0.625	1.15	1,600	
Dense No. 1		1.40	0.95	0.17	0.73	1,20	1,700	
No. 1		1.20	0.825	0.17	0.625	1.00	1,600	
No. 2		0.75	0.475	0.17	0.625	0.70	1,300	
Dense Select Structural	Beams and Stringers	1.90	1.10	0.17	0.73	1.30	1,700	WWPA
Select Structural		1.60	0.95	0.17	0.625	1.10	1,600	
Dense No. 1		1.55	0.775	0.17	0.73	1.10	1,700	
No. 1		1.35	0.675	0.17	0.625	0.925	1,600	
No. 2 Dense		1.00	0.50	0.17	0.73	0.70	1,400	
No. 2		0.875	0.425	0.17	0.625	0.60	1,300	
Dense Select Structural	Posts and Timbers	1.75	1.15	0.17	0.73	1.35	1,700	
Select Structural		1,50	1.00	0.17	0.625	1.15	1,600	
Dense No. 1		1.40	0.95	0.17	0.73	1.20	1,700	
No. 1		1.20	0.825	0.17	0.625	1.00	1,600	
No. 2 Dense		0.85	0.55	0.17	0.73	0.825	1,400	
No. 2		0.75	0.475	0.17	0.625	0.70	1,300	
EASTERN SOFTWOODS								
Select Structural	Dimension 2 in. & wider	1.25	0.575	0.14	0.335	1.20	1,200	NELMA NSLB
No. 1		0.775	0.35	0.14	0.335	1.00	1,100	
No. 2		0.575	0.275	0.14	0.335	0.825	1,100	
HEM-FIR								
Select Structural	Dimension 2 in. & wider	1.40	0.925	0.15	0.405	1.50	1,600	WCLIB WWPA
No. 1 & Btr		1.10	0.725	0.15	0.405	1.35	1,500	
No. 1		0.975	0.625	0.15	0.405	1.35	1,500	
No. 2		0.85	0.525	0.15	0.405	1.30	1,300	
Select Structural	Beams and Stringers	1.30	0.75	0.14	0.405	0.925	1,300	
No.1		1.05	0.525	0.14	0.405	0.75	1,300	
No.2		0.675	0.35	0.14	0.405	0.50	1,100	
Select Structural	Posts and Timbers	1.20	0.80	0.14	0.405	0.975	1,300	
No.1		0.975	0.65	0.14	0.405	0.85	1,300	
No.2		0.575	0.375	0.14	0.405	0.575	1,100	

Species and commercial grade	Size classification	Design Values (ksi) Bending F_{bo}	Tension parallel to grain F_{to}	Shear parallel to grain F_{vo}	Compression perpendicular to grain F_{cpo}	Compression parallel to grain F_{co}	Modulus of Elasticity E_o	Grading Rules Agency
MIXED SOUTHERN PINE								
Select Structural	Dimension 2 in. – 4 in. wide	2.05	1.20	0.175	0.565	1.80	1,600	SPIB
No.1		1.45	0.875	0.175	0.565	1.65	1,500	
No.2		1.30	0.775	0.175	0.565	1.65	1,400	
Select Structural	Dimension 5 in. – 6 in. wide	1.85	1.10	0.175	0.565	1.70	1,600	
No.1		1.30	0.75	0.175	0.565	1.55	1,500	
No.2		1.15	0.675	0.175	0.565	1.55	1,400	
Select Structural	Dimension 8 in. wide	1.75	1.00	0.175	0.565	1.60	1,600	
No.1		1.20	0.70	0.175	0.565	1.45	1,500	
No.2		1.05	0.625	0.175	0.565	1.45	1,400	
Select Structural	Dimension 10 in. wide	1.50	0.875	0.175	0.565	1.60	1,600	
No.1		1.05	0.60	0.175	0.565	1.45	1,500	
No.2		0.925	0.55	0.175	0.565	1.45	1,400	
Select Structural	Dimension 12 in. wide	1.40	0.825	0.175	0.565	1.55	1,600	
No.1		0.975	0.575	0.175	0.565	1.40	1,500	
No.2		0.875	0.525	0.175	0.565	1.40	1,400	
Select Structural	5 in.x5 in. and Larger	1.50	1.00	0.165	0.375	0.90	1,300	
No.1		1.35	0.90	0.165	0.375	0.80	1,300	
No.2		0.85	0.55	0.165	0.375	0.525	1,000	
NORTHERN RED OAK								
Select Structural	Dimension 2 in. & wider	1.40	0.80	0.22	0.885	1.15	1,400,	NELMA
No. 1		1.00	0.575	0.22	0.885	0.925	1,400	
No. 2		0.975	0.575	0.22	0.885	0.725	1,300	
Select Structural	Beams and Stringers	1.60	0.95	0.205	0.885	0.95	1,300	
No.1		1.35	0.675	0.205	0.885	0.80	1,300	
No.2		0.875	0.425	0.205	0.885	0.50	1,000	
Select Structural	Posts and Timbers	1.50	1.00	0.205	0.885	1.00	1,300	
No.1		1.20	0.80	0.205	0.885	0.875	1,300	
No.2		0.70	0.475	0.205	0.885	0.40	1,000	
RED MAPLE								
Select Structural	Dimension 2 in. & wider	1.30	0.75	0.21	0.615	1.10	1,700	NELMA
No. 1		0.925	0.55	0.21	0.615	0.90	1,600	
No. 2		0.90	0.525	0.21	0.615	0.70	1,500	
Select Structural	Beams and Stringers	1.50	0.875	0.195	0.615	0.90	1,500	
No.1		1.25	0.625	0.195	0.615	0.75	1,500	
No.2		0.80	0.40	0.195	0.615	0.475	1,200	
Select Structural	Posts and Timbers	1.40	0.925	0.195	0.615	0.95	1,500	
No.1		1.15	0.75	0.195	0.615	0.825	1,500	
No.2		0.65	0.425	0.195	0.615	0.375	1,200	

Species and commercial grade	Size classification	Design Values (ksi) Bending	Tension parallel to grain	Shear parallel to grain	Compression perpendicular to grain	Compression parallel to grain	Modulus of Elasticity	Grading Rules Agency
		F_{bo}	F_{to}	F_{vo}	F_{cpo}	F_{co}	E_o	
RED OAK								
Select Structural	Dimension 2 in. & wider	1.15	0.675	0.17	0.82	1.00	1,400	NELMA
No. 1		0.825	0.50	0.17	0.82	0.825	1,300	
No. 2		0.80	0.475	0.17	0.82	0.625	1,200	
Select Structural	Beams and Stringers	1.35	0.80	0.155	0.82	0.825	1,200	
No.1		1.15	0.55	0.155	0.82	0.70	1,200	
No.2		0.725	0.375	0.155	0.82	0.45	1,000	
Select Structural	Posts and Timbers	1.25	0.85	0.155	0.82	0.875	1,200	
No.1		1.00	0.675	0.155	0.82	0.775	1,200	
No.2		0.575	0.40	0.155	0.82	0.35	1,000	
SOUTHERN PINE								
Select Structural	Dimension 2 in. – 4 in. wide	2.85	1.60	0.175	0.565	2.10	1,800	SPIB
No.1		1.85	1.05	0.175	0.565	1.85	1,700	
No.2		1.50	0.825	0.175	0.565	1.65	1,600	
Select Structural	Dimension 5 in. – 6 in. wide	2.55	1.40	0.175	0.565	2.00	1,800	
No.1		1.65	0.90	0.175	0.565	1.75	1,700	
No.2		1.25	0.725	0.175	0.565	1.60	1,600	
Select Structural	Dimension 8 in. wide	2.30	1.30	0.175	0.565	1.90	1,800	
No.1		1.50	0.825	0.175	0.565	1.65	1,700	
No.2		1.20	0.65	0.175	0.565	1.55	1,600	
Select Structural	Dimension 10 in. wide	2.05	1.10	0.175	0.565	1.85	1,800	
No.1		1.30	0.725	0.175	0.565	1.60	1,700	
No.2		1.05	0.575	0.175	0.565	1.50	1,600	
Select Structural	Dimension 12 in. wide	1.90	1.05	0.175	0.565	1.80	1,800	
No.1		1.25	0.675	0.175	0.565	1.60	1,700	
No.2		0.975	0.55	0.175	0.565	1.45	1,600	
Select Structural	5 in. x 5 in. and Larger	1.50	1.00	0.165	0.375	0.95	1,500	
No. 1		1.35	0.90	0.165	0.375	0.825	1,500	
No. 2		0.85	0.55	0.165	0.375	0.525	1,200	
SPRUCE-PINE-FIR								
Select Structural	Dimension 2 in. & wider	1.25	0.70	0.135	0.425	1.40	1,500	NLGA
No. 1/ No. 2		0.875	0.45	0.135	0.425	1.15	1,400	
Select Structural	Beams and Stringers	1.10	0.65	0.125	0.425	0.775	1,300	
No.1		0.90	0.45	0.125	0.425	0.625	1,300	
No.2		0.60	0.30	0.125	0.425	0.425	1,000	
Select Structural	Posts and Timbers	1.05	0.70	0.125	0.425	0.80	1,300	
No.1		0.85	0.55	0.125	0.425	0.70	1,300	
No.2		0.50	0.325	0.125	0.425	0.50	1,000	

Species and commercial grade	Size classification	Design Values (ksi)						Grading Rules Agency
		Bending	Tension parallel to grain	Shear parallel to grain	Compression perpendicular to grain	Compression parallel to grain	Modulus of Elasticity	
		F_{bo}	F_{to}	F_{vo}	F_{cpo}	F_{co}	E_o	
SPRUCE-PINE-FIR (SOUTH)								
Select Structural	Dimension 2 in. & wider	1.30	0.575	0.135	0.335	1.20	1,300	NELMA NSLB WCLIB WWPA
No. 1		0.875	0.40	0.135	0.335	1.05	1,200	
No. 2		0.775	0.35	0.135	0.335	1.00	1,100	
Select Structural	Beams and Stringers	1.05	0.625	0.125	0.335	0.675	1,200	
No.1		0.90	0.45	0.125	0.335	0.55	1,200	
No.2		0.575	0.30	0.125	0.335	0.375	1,000	
Select Structural	Posts and Timbers	1.00	0.675	0.125	0.335	0.70	1,200	
No.1		0.80	0.55	0.125	0.335	0.625	1,200	
No.2		0.475	0.325	0.125	0.335	0.425	1,000	
YELLOW POPLAR								
Select Structural	Dimension 2 in. & wider	1.00	0.575	0.145	0.42	0.90	1,500	NSLB
No. 1		0.725	0.425	0.145	0.42	0.725	1,400	
No. 2		0.70	0.40	0.145	0.42	0.575	1,300	

Table 8.4.1.1.4-2 Reference Design Values for Mechanically Graded Dimension Lumber.

Commercial grade	Size classification	Design Values (ksi)				Grading Rules Agency
		Bending	Tension parallel to grain	Compression parallel to grain	Modulus of Elasticity	
		F_{bo}	F_{to}	F_{co}	E_o	
MACHINE STRESS RATED (MSR) LUMBER						
900f-1.0E	2 in. and less in thickness 2 in. and wider	0.90	0.35	1.05	1,000	WCLIB, WWPA, NELMA, NSLB
1200f-1.2E		1.20	0.60	1.40	1,200	NLGA, WCLIB, WWPA, NELMA, NSLB
1250f-1.4E		1.25	0.80	1.475	1,400	WCLIB, WWPA
1350f-1.3E		1.35	0.75	1.60	1,300	NLGA, WCLIB, WWPA, NELMA, NSLB
1400f-1.2E		1.40	0.80	1.60	1,200	NLGA, WWPA
1450f-1.3E		1.45	0.80	1.625	1,300	NLGA, WCLIB, WWPA, NELMA, NSLB
1450f-1.5E		1.45	0.875	1.625	1,500	WCLIB, WWPA
1500f-1.4E		1.50	0.90	1.65	1,400	NLGA, WCLIB, WWPA, NELMA, NSLB
1600f-1.4E		1.60	0.95	1.675	1,400	NLGA, WWPA
1650f-1.3E		1.65	1.02	1.70	1,300	NLGA, WWPA
1650f-1.5E		1.65	1.02	1.70	1,500	NLGA,SPIB,WCLIB,WWPA,NELMA,NSLB
1650f-1.6E-1075ft		1.65	1.075	1.70	1,600	WCLIB, WWPA
1650f-1.6E		1.65	1.175	1.70	1,600	WCLIB, WWPA
1650f-1.8E		1.65	1.02	1.75	1,800	WCLIB, WWPA
1700f-1.6E		1.70	1.175	1.725	1,600	WCLIB, WWPA
1750f-2.0E		1.75	1.125	1.725	2,000	WCLIB, WWPA
1800f-1.5E		1.80	1.30	1.75	1,500	NLGA, WWPA
1800f-1.6E		1.80	1.175	1.75	1,600	NLGA,SPIB,WCLIB,WWPA,NELMA,NSLB
1800f-1.8E		1.80	1.20	1.75	1,800	WCLIB, WWPA
1950f-1.5E		1.95	1.375	1.80	1,500	SPIB, WWPA
1950f-1.7E		1.95	1.375	1.80	1,700	NLGA,SPIB,WCLIB,WWPA,NELMA,NSLB
2000f-1.6E		2.00	1.30	1.825	1,600	NLGA, WWPA
2100f-1.8E		2.10	1.575	1.875	1,800	NLGA,SPIB,WCLIB,WWPA,NELMA,NSLB
2250f-1.7E		2.25	1.75	1.925	1,700	NLGA, WWPA
2250f-1.8E		2.25	1.75	1.925	1,800	NLGA, WCLIB, WWPA
2250f-1.9E		2.25	1.75	1.925	1,900	NLGA,SPIB,WCLIB,WWPA,NELMA,NSLB
2250f-2.0E-1600ft		2.25	1.60	1.925	2,000	WCLIB, WWPA
2250f-2.0E		2.25	1.75	1.925	2,000	WCLIB, WWPA
2400f-1.8E		2.40	1.925	1.975	1,800	NLGA, WWPA
2400f-2.0E		2.40	1.925	1.975	2,000	NLGA,SPIB,WCLIB,WWPA,NELMA,NSLB
2500f-2.2E		2.50	1.75	2.00	2,200	WCLIB, WWPA
2500f-2.2E-1925ft		2.50	1.925	2.00	2,200	WCLIB, WWPA
2550f-2.1E		2.55	2.05	2.025	2,100	NLGA,SPIB,WCLIB,WWPA,NELMA,NSLB
2700f-2.0E		2.70	1.80	2.10	2,000	WCLIB, WWPA
2700f-2.2E		2.70	2.15	2.10	2,200	NLGA,SPIB,WCLIB,WWPA,NELMA,NSLB
2850f-2.3E		2.85	2.30	2.150	2,300	NLGA,SPIB,WCLIB,WWPA,NELMA,NSLB
3000f-2.4E		3.00	2.40	2.20	2,400	NLGA, SPIB
MACHINE EVALUATED LUMBER (MEL)						
M-5	2 in. and less in thickness 2 in. and wider	0.90	0.500	1.05	1.100	SPIB
M-6		1.10	0.600	1.30	1.000	SPIB
M-7		1.20	0.650	1.40	1.100	SPIB

Commercial grade	Size classification	Design Values (ksi)				Grading Rules Agency
		Bending	Tension parallel to grain	Compression parallel to grain	Modulus of Elasticity	
		F_{bo}	F_{to}	F_{co}	E_o	
M-8	2 in. and less in thickness 2 in. and wider	1.30	0.70	1.50	1,300	SPIB
M-9		1.40	0.80	1.60	1,400	SPIB
M-10		1.40	0.80	1.60	1,200	NLGA, SPIB
M-11		1.55	0.85	1.675	1,500	NLGA, SPIB
M-12		1.60	0.85	1.675	1,600	NLGA, SPIB
M-13		1.60	0.95	1.675	1,400	NLGA, SPIB
M-14		1.80	1.00	1.75	1,700	NLGA, SPIB
M-15		1.80	1.10	1.75	1,500	NLGA, SPIB
M-16		1.80	1.30	1.75	1,500	SPIB
M-17		1.95	1.30	2.05	1,700	SPIB
M-18		2.00	1.20	1.825	1,800	NLGA, SPIB
M-19		2.00	1.30	1.825	1,600	NLGA, SPIB
M-20		2.00	1.60	2.10	1,900	SPIB
M-21		2.30	1.40	1.95	1,900	NLGA, SPIB
M-22		2.35	1.50	1.95	1,700	NLGA, SPIB
M-23		2.40	1.90	1.975	1,800	NLGA, SPIB
M-24		2.70	1.80	2.10	1,900	NLGA, SPIB
M-25		2.75	2.00	2.10	2,200	NLGA, SPIB
M-26		2.80	1.80	2.15	2,000	NLGA, SPIB
M-27		3.00	2.00	2.40	2,100	SPIB
M-28		2.20	1.60	1.90	1,700	SPIB
M-29		1.55	0.85	1.65	1,700	SPIB
M-30		2.05	1.05	1.85	1,700	SPIB
M-31		2.85	1.60	2.15	1,900	SPIB

Table 8.4.1.1.4-3 Reference Design Values of Specific Gravity, *G*, Shear, F_{vo}, and Compression Perpendicular to Grain, F_{cpo}, for Mechanically Graded Dimension Lumber.

Species	Modulus of Elasticity E (ksi)	Specific Gravity	Design Values (ksi)		Grading Rules Agency
			Shear parallel to grain	Compression perpendicular to grain	
		G	F_{vo}	F_{cpo}	
Douglas Fir-Larch	1,000 and higher	0.50	0.180	0.625	WCLIB, WWPA
	2,000	0.51	0.180	0.670	WCLIB, WWPA
	2,100	0.52	0.180	0.690	
	2,200	0.53	0.180	0.715	
	2,300	0.54	0.185	0.735	
	2,400	0.55	0.185	0.760	
Hem-Fir	1,000 and higher	0.43	0.150	0.405	WCLIB, WWPA
	1,600	0.44	0.155	0.510	WCLIB, WWPA
	1,700	0.45	0.160	0.535	
	1,800	0.46	0.160	0.555	
	1,900	0.47	0.165	0.580	
	2,000	0.48	0.170	0.600	
	2,100	0.49	0.170	0.625	
	2,200	0.50	0.175	0.645	
	2,300	0.51	0.190	0.670	
	2,400	0.52	0.190	0.690	
Southern Pine	1,000 and higher	0.55	0.175	0.565	SPIB
	1,800 and higher	0.57	0.190	0.805	SPIB
	1,200 and higher	0.42	0.135	0.425	NLGA
Spruce-Pine-Fir	1,800-1,900	0.46	0.160	0.525	NLGA
	2,000 and higher	0.50	0.170	0.615	
	1,000 and higher	0.36	0.135	0.335	NELMA, NSLB, WCLIB, WWPA
	1,200-1,900	0.42	0.150	0.465	NELMA, NSLB
Spruce-Pine-Fir (S)	1,200-1,700	0.42	0.150	0.465	WWPA
	1,800-1,900	0.46	0.160	0.555	
	2,000 and higher	0.50	0.175	0.645	NELMA, NSLB, WWPA

8.4.1.2.3 ~~Base Resistance~~ Reference Design Values ~~and Modulus of Elasticity~~

Grade combinations for structural glued laminated timber shall be as provided in AITC 117-2004 ~~Design~~, Standard Specifications for Structural Glued Laminated Timber of Softwood Species, or AITC 119-96, Standard Specifications for ~~Hardwood~~ Structural Glued Laminated Timber of Hardwood Species.

~~Base Resistance~~ Reference Design Values ~~and modulus of elasticity~~ for structural glued laminated timber shall be as specified in Tables 1 and 2:

- Table 1 contains design values for timbers with layups optimized to resist bending loads applied perpendicular to the wide face of the laminations (bending about the x-x axis). Design values are also included, however, for axial loads and bending loads applied parallel to the wide faces of the laminations. The design values in Table 1 are applicable to timbers with 4 or more laminations.

- Table 2 contains design values for timbers with uniform-grade layups. These layups are intended primarily for timbers loaded axially or in bending due to loads applied parallel to the wide faces of the laminations (bending about the y-y axis). Design values are also included, however, for bending due to loads applied perpendicular to the wide faces of the laminations. The design values in Table 2 are applicable to timbers with 2 or more laminations.

In Table 1, the tabulated design values, F_{bx},, for bending about the X-X axis (F_{bx}), require the use of special tension laminations. If these special tension laminations are omitted, value shall be multiplied by 0.75 for members greater than or equal to 15 in. in depth or by 0.85 for members less than 15 in. in depth.

In Table 1, the design value for shear, F_{vx}, shall be decreased by multiplying by a factor of 0.72 for non-prismatic members, notched members, and for all members subject to impact or cyclic loading. The reduced design value shall be used for design of members at connections that transfer shear by mechanical fasteners. The reduced design value shall also be used for determination of design values for radial tension and torsion. Design values, F_{vy}, shall be used for timbers with laminations made from a single piece of lumber across the width or multiple pieces that have been edge bonded. For timber manufactured from multiple piece laminations (across width) that are not

C8.4.1.2.3

The combinations in Table 1 are applicable to members consisting of 4 or more laminations and are intended primarily for members stressed in bending due to loads applied perpendicular to the wide faces of the laminations. However, design values are tabulated for loading both perpendicular and parallel to the wide faces of the laminations. The combinations and design values applicable to members loaded primarily axially or parallel to the wide faces of the laminations, are specified in Table 2. Design values for members of 2 or 3 laminations, are specified in Table 2.

edge-bonded, in addition to other reduction, design value shall be multiplied by 0.4 for members with 5, 7, or 9 laminations or by 0.5 for all other members. If combination 24F-V4 contain lumber with wane, then, in addition, the design value for shear parallel to grain, F_{VX}, shall be multiplied by 0.67 if wane is allowed on both sides. If wane is limited to one side, F_{vx} shall be multiplied by 0.83.

In Table 2, for members with 2 or 3 laminations, the shear design value for transverse loads parallel to the wide faces of the laminations, F_{vy}, shall be reduced by multiplying by a factor of 0.84 or 0.95, respectively. For members with 5, 7, or 9 laminations, in addition, F_{vy}, shall be multiplied by 0.4 for members manufactured from multiple piece laminations (across width) that are not edge bonded. The shear design value, F_{vy}, shall be multiplied by 0.5 for all other members manufactured from multiple piece laminations with unbonded edge joints.

In Table 2, the design value for shear, F_{vx}, shall be decreased by multiplying by a factor of 0.72 for nonprismatic members, notched members, and for all members subject to impact or cyclic loading. The reduced design value shall be used for design of members at connections that transfer shear by mechanical fasteners. The reduced design value shall also be used for determination of design values for radial tension and torsion.

In Table 2, the tabulated design values shall apply to timbers without special tension laminations. If special tension laminations are used, for members to 15 in. deep the design value for bending, F_{bx}, may be increased by multiplying by 1.18. For members greater than 15 in. deep and without special tension laminations, the bending design value, F_{bx}, shall be reduced by multiplying by a factor of 0.88.

~~Base Resistance~~ Reference design values ~~and modulus of elasticity~~ for combinations ~~symbols~~ not given in Table 1 or Table 2 shall be obtained from ~~the design values specified in the~~ AITC 117-~~2001~~ 2004. ~~Design, *Standard Specifications for Structural Glued Laminated Timber of Softwood Species.*~~

Table 8.4.1.2.3-1 Reference Design Values, ksi, for Structural Glued Laminated Softwood Timber Combinations (Members stressed primarily in bending).

		Bending About X-X Axis (Loaded Perpendicular to Wide Faces of Laminations)						Bending About Y-Y Axis (Loaded Parallel to Wide Faces of Laminations)				Axially Loaded			Fasteners	
		Extreme Fiber in Bending		Compression Perpendicular to Grain		Shear Parallel to Grain (Horizontal)	Modulus of Elasticity	Extreme Fiber in Bending	Compression Perpendicular to Grain	Shear Parallel to Grain (Horizontal)	Modulus of Elasticity	Tension Parallel to Grain	Compression Parallel to Grain	Modulus of Elasticity	Specific Gravity for Fastener Design	
		Tension Zone Stressed in Tension	Compression Zone Stressed in Tension	Tension Face	Compression Face										Top or Bottom Face	Side Face
Combination Symbol	Species Outer/ Core	F_{bxo}^{+}	F_{bxo}^{-}	F_{cpo}		F_{vxo}	E_{xo}	F_{byo}	F_{cpo}	F_{vyo}	E_{yo}	F_{to}	F_{co}	$E_{o\ axial}$	G_o	
20F-1.5E		2	1.1	0.425		0.21	1,500	0.8	0.315	0.185	1,200	0.725	0.925	1,300	0.42	
20F-V3	DF/DF	2.000	1.450	0.650	0.560	0.265	1,600	1.450	0.560	0.230	1,500	0.975	1.550	1,600	0.5	0.5
20F-V7	DF/DF	2.000	2.000	0.650	0.650	0.265	1,600	1.450	0.560	0.230	1,600	1.000	1.600	1,600	0.5	0.5
20F-V9	HF/HF	2.000	2.000	0.500	0.500	0.215	1,500	1.350	0.375	0.190	1,400	0.975	1.400	1,500	0.43	0.43
20F-V12	AC/AC	2.000	1.400	0.560	0.560	0.265	1,500	1.250	0.470	0.230	1,400	0.900	1.500	1,400	0.46	0.46
20F-V13	AC/AC	2.000	2.000	0.560	0.560	0.265	1,500	1.250	0.470	0.230	1,400	0.925	1.550	1,500	0.46	0.46
20F-V2	SP/SP	2.000	1.550	0.740	0.650	0.300	1,500	1.450	0.650	0.260	1,400	0.975	1.350	1,500	0.55	0.55
20F-V3	SP/SP	2.000	1.450	0.650	0.650	0.300	1,500	1.750	0.650	0.260	1,400	1.050	1.400	1,500	0.55	0.55
20F-V5	SP/SP	2.000	2.000	0.740	0.740	0.300	1,600	1.450	0.650	0.260	1,400	1.050	1.500	1,500	0.55	0.55
24F-1.7E		2.400	1.450	0.500		0.210	1,700	1.050	0.315	0.185	1,200	0.775	1.000	1,400	0.42	
24F-V5	DF/HF	2.400	1.600	0.650	0.650	0.215	1,700	1.200	0.375	0.190	1,500	1.150	1.450	1,600	0.5	0.43
24F-V10	DF/HF	2.400	2.400	0.650	0.650	0.215	1,800	1.450	0.375	0.190	1,500	1.100	1.550	1,600	0.5	0.43
24F-V1	SP/SP	2.400	1.750	0.740	0.650	0.300	1,700	1.450	0.650	0.260	1,500	1.100	1.550	1,600	0.55	0.55
24F-V4	SP/SP	2.400	1.450	0.740	0.650	0.210	1,700	1.050	0.470	0.185	1,300	0.875	1.000	1,500	0.55	0.43
24F-V5	SP/SP	2.400	2.400	0.740	0.740	0.300	1,700	1.750	0.650	0.260	1,500	1.150	1.650	1,600	0.55	0.55
24F-1.8E		2.400	1.450	0.650		0.265	1,800	1.450	0.560	0.230	1,600	1.1	1.600	1,700	0.5	
24F-V4	DF/DF	2.400	1.850	0.650	0.650	0.265	1,800	1.450	0.560	0.230	1,600	1.100	1.650	1,700	0.5	0.5
24F-V8	DF/DF	2.400	2.400	0.650	0.650	0.265	1,800	1.450	0.560	0.230	1,600	1.100	1.650	1,700	0.5	0.5
24F-V3	SP/SP	2.400	1.950	0.740	0.740	0.300	1,800	1.750	0.650	0.260	1,600	1.150	1.650	1,700	0.55	0.55
26F-1.9E		2.600	1.950	0.650		0.265	1,900	1.600	0.560	0.230	1,600	1.15	1.600	1,700	0.5	
26F-V1	DF/DF	2.600	1.950	0.650	0.650	0.265	2,000	1.750	0.560	0.230	1,800	1.300	1.850	1,900	0.5	0.5
26F-V2	DF/DF	2.600	2.600	0.650	0.650	0.265	2,000	1.750	0.560	0.230	1,800	1.300	1.850	1,900	0.5	0.5
26F-V2	SP/SP	2.600	2.100	0.740	0.740	0.300	1,900	2.200	0.740	0.260	1,800	1.250	1.650	1,900	0.55	0.55
26F-V4	SP/SP	2.600	2.600	0.740	0.740	0.300	1,900	2.100	0.650	0.260	1,800	1.200	1.600	1,900	0.55	0.55

Table 8.4.1.2.3-2 Reference Design Values, ksi, for Structural Glued Laminated Softwood Timber (Members stressed primarily in axial tension and compression).

			All Loading		Axially Loaded			Bending about Y-Y Axis				Bending About X-X Axis	
								Loaded Parallel to Wide Faces of Laminations				Loaded Perpendicular to Wide Faces of Laminations	
					Tension Parallel to Grain	Compression Parallel to Grain		Bending			Shear Parallel to Grain	Bending	Shear Parallel to Grain
Identification Number	Species	Grade	Modulus of Elasticity E_o	Compression Perpendicular to Grain F_{cpo}	2 or More Lami-nations F_{to}	4 or More Lami-nations F_{cpo}	2 or 3 Lami-nations F_{cpo}	4 or More Lami-nations F_{byo}	3 Lami-nations F_{byo}	2 Lami-nations F_{byo}	F_{vyo}	2 Lami-nations to 15 in. Deep F_{bxo}	F_{vxo}
Visually Graded Western Species													
1	DF	L3	1,500	0.560	0.900	1.550	1.200	1.450	1.250	1.000	0.230	1.250	0.265
2	DF	L2	1,600	0.560	1.250	1.950	1.600	1.800	1.600	1.300	0.230	1.700	0.265
3	DF	L2D	1,900	0.650	1.450	2.300	1.850	2.100	1.850	1.550	0.230	2.000	0.265
5	DF	L1	2,000	0.650	1.600	2.400	2.100	2.400	2.100	1.800	0.230	2.200	0.265
14	HF	L3	1,300	0.375	0.800	1.100	0.975	1.200	1.050	0.850	0.190	1.100	0.215
15	HF	L2	1,400	0.375	1.050	1.350	1.300	1.500	1.350	1.100	0.190	1.450	0.215
16	HF	L1	1,600	0.375	1.200	1.500	1.450	1.750	1.550	1.300	0.190	1.600	0.215
17	HF	L1D	1,700	0.500	1.400	1.750	1.700	2.000	1.850	1.550	0.190	1.900	0.215
69	AC	L3	1,200	0.470	0.725	1.150	1.100	1.100	0.975	0.775	0.230	1.000	0.265
70	AC	L2	1,300	0.470	0.975	1.450	1.450	1.400	1.250	1.000	0.230	1.350	0.265
71	AC	L1D	1,600	0.560	1.250	1.900	1.900	1.850	1.650	1.400	0.230	1.700	0.265
Visually Graded Southern Pine													
47	SP	N2M14	1,400	0.650	1.200	1.900	1.150	1.750	1.550	1.300	0.260	1.400	0.300
48	SP	N2D14	1,700	0.740	1.400	2.200	1.350	2.000	1.800	1.500	0.260	1.600	0.300
49	SP	N1M16	1,700	0.650	1.350	2.100	1.450	1.950	1.750	1.500	0.260	1.800	0.300
50	SP	N1D14	1,900	0.740	1.550	2.300	1.700	2.300	2.100	1.750	0.260	2.100	0.300

8.4.1.3 Piles

Wood piles shall comply with the requirements of AASHTO M 168.

~~Base Resistance~~ Reference design values ~~and modulus of elasticity~~ for round wood piles shall be as specified in Table 1.

C8.4.1.3

The ~~unmodified~~ ~~Base Resistance~~ reference design values ~~and modulus of elasticity~~ for wood piles are based on wet-use conditions.

Table 8.4.1.3-1 ~~Base Resistance~~ Reference Design Values ~~and Modulus of Elasticity~~ for Piles, ksi.

Species	F_{co}	F_{bo}	F_{cpo}	F_{vo}	E_o
Pacific Coast Douglas-Fir[1]	~~3.00~~1.25	~~6.22~~2.45	~~0.48~~0.23	~~0.33~~0.115	1500
Red Oak[2]	~~2.64~~1.10	~~6.22~~2.45	~~0.73~~0.35	~~0.39~~0.135	1250
Red Pine[3]	~~2.22~~0.90	~~4.83~~1.90	~~0.32~~0.155	~~0.24~~0.085	1280
Southern Pine[4]	~~2.70~~1.20	~~5.59~~2.40	~~0.52~~0.25	~~0.30~~0.11	~~1400~~1500

[1] Pacific Coast Douglas-Fir reference strengths apply to this species as defined in ASTM Standard D 1760-~~86~~01. For connection design use Douglas Fir-Larch reference design values.
[2] Red Oak reference strengths apply to Northern and Southern Red Oak.
[3] Red Pine reference strengths apply to Red Pine grown in the U.S. For connection design ~~to~~ use Northern Pine reference design values.
[4] Southern Pine reference strengths apply to Loblolly, Longleaf, Shortleaf, and Slash Pine.

8.4.2 Metal Fasteners and Hardware

8.4.2.1 General

Structural metal, including shapes, plates, bars, and welded assemblies shall comply with the applicable material requirements of Section 6.

8.4.2.2 Minimum Requirements

8.4.2.2.1 Fasteners

Bolts and lag screws shall comply with the dimensional and material quality requirements of ANSI/ASME B18.2.1, *Square and Hex Bolts an Screws – Inch Series*. Strengths for low-carbon steel bolts, Grade 1 through Grade 8, shall be as specified in Society of Automotive Engineers Specification SAE-429, *Mechanical and Material Requirements for Externally Threaded Fasteners.* Bolt and lag screw grades not given in SAE-429 shall have a minimum yield strength of 33.0 ksi.

8.4.2.2.2 Prestressing Bars

Prestressing bars shall comply with the requirements of AASHTO M 275 (ASTM A 722), *Standard Specification for Uncoated High-Strength Steel Bar for Prestressing Concrete,* and the applicable provisions of Section 5.

8.4.2.2.3 Split Ring Connectors

Split ring connectors shall be manufactured from hot-rolled carbon steel complying with the requirements of Society of Automotive Engineers Specification SAE-1010. Each circular ring shall be cut through in one place in its circumference to form a tongue and slot.

8.4.2.2.4 Shear Plate Connectors

Shear plate connectors shall be manufactured from pressed steel, light gage steel, or malleable iron. Pressed steel connectors shall be manufactured from hot-rolled carbon steel meeting Society of automotive Engineers Specification SAE-1010. Malleable iron connectors shall be manufactured in accordance with ASTM A 47, Grade 32510.

Each shear plate shall be a circle with a flange around the edge, extending at right angles to the plate face from one face only.

8.4.2.2.5 Nails and Spikes

Nails and spikes shall be manufactured from common steel wire or high carbon steel wire that is heat-treated and tempered. When used in withdrawal-type connections, the shank of the nail or spike shall be annularly or helically threaded.

8.4.2.2.6 Drift Pins and Bolts

Drift pins and drift bolts shall have a minimum flexural yield strength of 30.0 ksi.

8.4.2.2.7 Spike Grids

Spike grids shall conform to the requirements of ASTM A 47, Grade 32510, for malleable iron casting.

8.4.2.2.8 Toothed Metal Plate Connectors

Metal plate connectors shall be manufactured from galvanized sheet steel that complies with the requirements of ASTM A 653, Grade A, or better, with the following minimum mechanical properties:

Yield Point	33.0 ksi
Ultimate Strength	45.0 ksi
Elongation in 2.0 in.	20%

8.4.2.3 Corrosion Protection

8.4.2.3.1 Metallic Coating

Except as permitted by this section, all steel hardware for wood components shall be galvanized in accordance with AASHTO M 232 (ASTM A 153) or cadmium plated in accordance with AASHTO M 299 (ASTM B 696).

Except as otherwise permitted, all steel components, timber connectors, and castings other than malleable iron shall be galvanized in accordance with AASHTO M111 (ASTM A 123).

C8.4.2.3.1

Galvanized nuts should be retapped to allow for the increased diameter of the bolt due to galvanizing.

Protection for the high-strength bars used in stress-laminated decks should be clearly specified. Standard hot-dip galvanizing can adversely affect the properties of high-strength post-tensioning materials. A lower temperature galvanizing is possible with some high-strength bars. The manufacturer of the bars should be consulted on this issue

8.4.2.3.2 Alternative Coating

Alternative corrosion protection coatings may be used when the demonstrated performance of the coating is sufficient to provide adequate protection for the intended exposure condition during the design life of the bridge. When epoxy coatings are used minimum coating requirements shall comply with AASHTO M 284.

Heat-treated alloy components and fastenings shall be protected by an approved alternative protective treatment that does not adversely affect the mechanical properties of the material.

8.4.3 Preservative Treatment

8.4.3.1 Requirements for Treatment

All wood used for permanent applications shall be pressure impregnated with wood preservative in accordance with the requirements of AASHTO M 133.

Insofar as is practicable, all wood components should be designed and detailed to be cut, drilled, and otherwise fabricated prior to pressure treatment with wood preservatives. When cutting, boring, or other fabrication is necessary after preservative treatment, exposed, untreated wood shall be specified to be treated in accordance with AASHTO M 133

8.4.3.2 Treatment Chemicals

Unless otherwise approved, all structural components that are not subject to direct pedestrian contact shall be treated with oil-borne preservatives. Pedestrian railings and non-structural components that are subject to direct pedestrian contact shall be treated with water-borne preservatives or oil-borne preservatives in light petroleum solvent.

C8.4.3.2

The oil-borne preservative treatments have proven to provide adequate protection against wood attacking organisms. In addition, the oil provides a water repellant coating that reduces surface effects cause by cyclic moisture conditions. Water-borne preservative treatments do not provide the water repellency of the oil-borne treatment, and components frequently split and check, leading to poor field performance and reduced service life.

Direct pedestrian contact is considered to be contact that can be made while the pedestrian is situated anywhere in the access route provided for pedestrian traffic.

Treating of glued laminated timbers with water-borne preservatives after gluing is not recommended. Use of water-borne preservatives for glued laminated timber after gluing may result in excessive warping, checking, or splitting of the components due to post-treatment redrying.

8.4.3.3 Inspection and Marking

Preservative treated wood shall be tested and inspected in accordance with the requirements of AASTHO M 133. Where size permits, each piece of treated wood that meets treatment requirements shall be legibly stamped, branded, or tagged to indicate the name of the treater and the specification symbol or specification requirement to which the treatment conforms.

When requested, a certification indicating test results and the identification of the inspection agency shall be provided.

8.4.3.4 Fire Retardant Treatment

Fire retardant treatments shall not be applied unless it is demonstrated that they are compatible with the preservative treatment used, and the usable resistance and stiffness are reduced as recommended by the product manufacturer and applicator.

C8.4.3.4

Use of fire retardant treatments is not recommended because the large sizes of timber components typically used in bridge construction have inherent fire resistance characteristics. The pressure impregnation of wood products with fire resistant chemicals is known to cause certain resistance and stiffness losses in the wood. These resistance and stiffness losses vary with specific resistance characteristic, i.e., bending resistance, tension parallel to grain resistance, etc., treatment process, wood species and type of wood products, i.e. solid sawn, glued laminated, or other.

8.4.4 ~~Modification~~ Adjustment Factors for ~~Resistance~~ Reference Design Values ~~and Modulus of Elasticity~~

8.4.4.1 General

~~Nominal resistance and~~ Adjusted design values ~~modulus of elasticity values~~ shall be obtained by adjusting ~~base~~ reference design values by applicable ~~modification~~ adjustment factors in accordance with the following equations:

~~$F = F_o C_F C_M C_D C_T$ (8.4.4.1-1)~~

~~$E = E_o C_M$ (8.4.4.1-2)~~

$F_b = F_{bo}\, C_{KF}\, C_M\, (C_F$ or $C_v)\, C_{fu}\, C_i\, C_d\, C_\lambda$ (8.4.4.1-1)

$F_v = F_{vo} C_{KF} C_M C_i C_\lambda$ (8.4.4.1-2)

$F_t = F_{to} C_{KF} C_M C_F C_i C_\lambda$ (8.4.4.1-3)

$F_c = F_{co} C_{KF} C_M C_F C_i C_\lambda$ (8.4.4.1-4)

$F_{cp} = F_{cpo} C_{KF} C_M C_i C_\lambda$ (8.4.4.1-5)

$E = E_o C_M C_i$ (8.4.4.1-6)

where:

F	=	applicable adjusted design values ~~nominal resistance~~ F_b, F_v, F_t, F_c, or F_{cp} (ksi)
F_o	=	reference design values ~~base resistance~~ F_{bo}, F_{vo}, F_{to}, F_{co}, or F_{cpo} specified in Article 8.4(ksi)
E	=	~~base~~ adjusted modulus of elasticity (ksi)
E_o	=	~~base~~ reference modulus of elasticity specified in Article 8.4.~~1.1.4~~ (ksi)
C_{KF}	=	format conversion factor specified in Article 8.4.4.2
C_M	=	wet service factor specified in Article 8.4.4.3
C_F	=	size ~~effect~~ factor for visually-graded dimension lumber and sawn timbers specified in Article 8.4.4.~~2~~4
C_V	=	volume factor for structural glued laminated timber specified in Article 8.4.4.5
C_{fu}	=	flat-use factor specified in Article 8.4.4.6
~~C_M~~	~~=~~	~~wet service factor specified in Article 8.4.4.3~~
C_i	=	incising factor specified in Article 8.4.4.7
~~C_D~~ C_d	=	deck factor specified in Article 8.4.4.~~4~~8
~~C_T~~ C_λ	=	time effect factor specified in Article 8.4.4.~~5~~9

8.4.4.2 Format Conversion Factor, C_{KF}

The reference design values in Table 1 and 2 and reference design values specified in the NDS ® shall be multiplied by a format conversion factor, C_{KF}, for use with load and resistance factor design (LRFD). $C_{KF} = 2.5/\phi$, except for compression perpendicular to grain which shall be obtained by multiplying the allowable stress by a format conversion factor of $C_{KF} = 2.1/\phi$.

C8.4.4.2

The conversion factors were derived so that LRFD design will result in same size member as the allowable stress design (ASD) specified in NDS. For example, a rectangular component in flexure has to satisfy:

$1.25\ M_{DL} + 1.75\ M_{LL} < \phi\ S\ F_{bo}\ C_{KF}\ C_M\ (C_F \text{ or } C_v)\ C_{fu}\ C_i\ C_d\ C_\lambda\ C_L$ or (C8.4.4.2-1)

$$(1.25\ M_{DL} + 1.75\ M_{LL}) / (\phi C_{KF}\ C_{\lambda}) \leq S\ F_{bo}\ C_M\ (C_F \text{ or } C_v)\ C_{fu}\ C_i\ C_d\ C_L \quad \text{(C8.4.4.2-2)}$$

where:

M_{DL} = moment due to dead load

M_{LL} = moment due to live load

On the other hand, the allowable stress design (ASD) has to satisfy:

$$M_{DL} + M_{LL} \leq S\ F_{bo}\ C_M\ (C_F \text{ or } C_v)\ C_{fu}\ C_i\ C_d\ C_D\ C_L \text{ or}$$
$$(M_{DL} + M_{LL}) / (C_D) \leq S\ F_{bo}\ C_M\ (C_F \text{ or } C_v)\ C_{fu}\ C_i\ C_d\ C_L \quad \text{(C8.4.4.2-3)}$$

Therefore:

$$(1.25\ M_{DL} + 1.75\ M_{LL}) / (\phi C_{KF}\ C_{\lambda}) = (M_{DL} + M_{LL}) / (C_D) \quad \text{(C8.4.4.2-4)}$$

$$C_{KF} = [(1.25\ M_{DL} + 1.75\ M_{LL})(C_D)] / [(M_{DL} + M_{LL})(\phi C_{\lambda})] \quad \text{(C8.4.4.2-5)}$$

The format conversion factor is calculated assuming the ratio of M_{DL} and M_{LL} is 1:10, ϕ = 0.85, C_{λ} = 0.8, and C_D = 1.15.

8.4.4.3 Wet Service Factor, C_M

The dry use ~~resistance~~ reference design values specified in Tables 8.4.1.1.4-1 and 8.4.1.1.4-2 shall be adjusted for moisture content using the wet service factor, C_M, specified below:

- For sawn lumber with an in-service moisture content of 19 percent or less, C_M shall be taken as 1.0.
- For glued laminated timber with an in-service moisture content of 16 percent or less, C_M shall be taken as 1.0.
- Otherwise, C_M shall be taken as specified in Tables 1 and 2 for sawn lumber and glued laminated timber, respectively.

~~In applying Tables 1 and 2, the values of F_{xx} shall be taken as the size-adjusted resistances.~~

Reference design values for Southern Pine and Mixed Southern Pine timbers 5 in. x5 in. and larger shall be taken to apply to wet or dry use.

C8.4.4.3

An analysis of in-service moisture content should be based on regional, geographic, and climatological conditions. In the absence of such analysis, wet-use conditions should be assumed.

Reduction for wet-use is not required.

Table 8.4.4.3-1 Wet Service Factor for Sawn Lumber, C_M.

Nominal Thickness	$F_{bo}C_F \le$ ~~2.92~~1.15 ksi	$F_{bo}C_F >$ ~~2.92~~1.15 ksi	F_{to}	$F_{co}C_F \le$ ~~1.80~~0.75 ksi	$F_{co}C_F >$ ~~1.80~~0.75 ksi	F_{vo}	F_{cpo}	E_o
4 in. or less	1.00	0.85	1.00	1.00	0.80	0.97	0.67	0.90
over 4.0 in.	1.00	1.00	1.00	0.91	0.91	1.00	0.67	1.00

Table 8.4.4.3-2 Wet Service Factor for Glued Laminated Timber, C_M.

F_{bo}	F_{vo}	F_{to}	F_{co}	F_{cpo}	E_o
0.80	0.875	0.80	0.73	0.53	0.833

8.4.4.~~2~~4 Size ~~Effect~~ Factor, C_F, for Sawn Lumber

The ~~S~~size ~~effect~~ factor, C_F, shall be 1.0 unless specified otherwise herein.

For ~~sawn~~ visually-graded dimension lumber of all species except Southern Pine and Mixed Southern Pine, C_F shall be as specified in Table 1 ~~without modification~~.

~~For flat-wise use, the size effect factor, C_F, shall be taken as follows:~~

~~for 4×6 C_F = 1.10~~

~~for 4×8 C_F = 1.15~~

~~for 4×10 C_F = 1.25~~

~~for 4×12 C_F = 1.50~~

Reference design values for Southern Pine and Mixed Southern Pine dimension lumber have been size-adjusted; no further adjustment for size shall be applied.

For Southern Pine and Mixed Southern Pine dimension lumber wider than 12.0 in., the tabulated bending, compression, and tension parallel to grain design values, for the 12.0 in. depth, shall be multiplied by the size factor, C_F = 0.9.

C8.4.4.4

C_F does not apply to mechanically-graded lumber (MSR, MEL) or to structural glued laminated timber.

Tabulated design values for visually-graded lumber of Southern Pine and Mixed Southern Pine species groups have already been adjusted for size. Further adjustment by the size factor is not permitted.

Table 8.4.4.~~2~~4-1 Size ~~Effect~~ Factor, C_F, for Sawn Dimension Lumber.

GRADE	WIDTH (in.)	F_{bo}		F_{to}	F_{co}	ALL OTHER PROPERTIES
		Thickness				
		2.0 in. and 3.0 in.	4.0 in.	All	All	All
	Structural Light Framing – 2.0 in. x 2.0 in. through 4.0 in. x 4.0 in. Structural Joists and Planks – 2.0 in. x 5.0 in. through 4.0 in. x 16.0 in.					
Sel. Str No. 1 No. 2	≤ 4	1.5~~4~~	1.5~~4~~	1.5~~4~~	1.15~~6~~	
	5	1.4~~0~~	1.4~~0~~	1.4~~0~~	1.1~~3~~	
	6	1.3~~0~~	1.3~~0~~	1.3~~0~~	1.1~~0~~	
	8	1.2~~19~~	1.3~~0~~	1.2~~19~~	1.05~~6~~	1.00
	10	1.1~~09~~	1.2~~0~~	1.1~~09~~	1.0~~3~~	
	12	1.0~~0~~	1.1~~0~~	1.0~~0~~	1.0~~0~~	
	14 & wider	0.9~~3~~	1.0~~2~~	0.9~~3~~	0.9~~8~~	
	~~16~~	~~0.88~~	~~0.97~~	~~0.88~~	~~0.96~~	

For sawn *beams and stringers* with loads applied to the narrow face and *posts and timbers with loads applied to either face*, ~~and for vertically laminated, glued laminated timber with loads applied parallel to the wide face of the laminations~~ F_{bo} shall be adjusted by C_F determined as:

- If $d \le 12.0$ in., then

$$C_F = 1.0 \qquad (8.4.4.\cancel{2}4\text{-}1)$$

- If $d > 12.0$ in., then

$$C_F = \left(\frac{12}{d}\right)^{\frac{1}{9}} \qquad (8.4.4.\cancel{2}4\text{-}2)$$

where:

d = net width as shown in Figure 8.3-1

For *beams and stringers* with loads applied to the wide face, F_{bo} shall be adjusted by C_F as specified in Table 2.

Table 8.4.4.~~3~~4-2 Size Factor, C_F, for Beams and Stringers with loads applied to the wide face.

Grade	F_{bo}	E_o	Other Properties
SS	0.86	1.00	1.00
No. 1	0.74	0.90	1.00
No. 2	1.00	1.00	1.00

~~The directions of length, width, and thickness are shown in Figure 8.3-1.~~

8.4.4.~~4~~5 Volume Factor, C_V, (Glulam)

For ~~glued~~ horizontally laminated ~~timber~~ glulam, with loads applied perpendicular to the wide face of the laminations, F_{bo} shall be reduced by ~~C_F~~C_V, given below, when the depth, width, or length of a glued laminated timber exceeds 12.0 in., 5.125 in., or 21.0 ft., respectively:

$$C_V = \left[\left(\frac{12.0}{d}\right)\left(\frac{5.125}{b}\right)\left(\frac{21}{L}\right)\right]^a \leq 1.0 \qquad (8.4.4.~~2~~5\text{-}~~1~~3)$$

where:

d = depth of the component (in.)

b = width of the component (in.) For layups with multiple piece laminations (across the width) b = width of widest piece. Therefore: ~~but need not be taken as greater than~~ b ≤ 10.75 in. ~~(in.)~~

L = length of the component measured between points of contraflexure (ft.)

a = 0.05 for Southern Pine and 0.10 for all other species.

The volume factor, C_V, shall not be applied simultaneously with the beam stability factor, C_L, therefore, the lesser of these factors shall apply.

8.4.4.6 Flat-Use Factor, C_{fu}

When dimension lumber graded as *Structural Light Framing* or *Structural Joists and Planks* is used flatwise (load applied to the wide face), the bending reference design value shall be multiplied by the flat use factor specified in Table 1.

The flat-use factor shall not apply to dimension lumber graded as *Decking.*

C8.4.4.6

Design values for flexure of dimension lumber adjusted by the size factor, C_F, are based on edgewise use (load applied to the narrow face). When dimension lumber is used flatwise (load applied to the wide face), the bending reference design value should also be multiplied by the flat use factor specified in Table 1.

Design values for dimension lumber graded as *Decking* are based on flatwise use. Further adjustment by the flat-use factor is not permitted.

Table 8.4.4.6-1 Flat-Use Factor, C_{fu}, for Dimension Lumber.

Width (in.)	Thickness (in.)	
	2 and 3	4
2 and 3	1.0	--
4	1.1	1.0
5	1.1	1.05
6	1.15	1.05
8	1.15	1.05
10 and wider	1.2	1.1

Reference design values for flexure of vertically laminated glulam (loads applied parallel to wide faces of laminations) shall be multiplied by the flat use factors specified in Table 2 when the member dimension parallel to wide faces of laminations is less than 12.0 in.

Table 8.4.4.6-2 Flat-Use Factor, C_{fu}, for Glulam.

Member dimension parallel to wide faces of laminations (in.)	C_{fu}
10 3/4 or 10 1/2	1.01
8 3/4 or 8 1/2	1.04
6 3/4	1.07
5 1/8 or 5	1.10
3 1/8 or 3	1.16
2 1/2 or 2 1/8	1.19

8.4.4.7 Incising Factor, C_i

Reference design values for dimension lumber shall be multiplied by the incising factor specified in Table 1 when members are incised parallel to grain a maximum depth of 0.4 in., a maximum length of 3/8 in., and a density of incisions up to 1100/ft^2. Incising factors shall be determined by test or by calculation using reduced section properties for incising patterns exceeding these limits.

Table 8.4.4.7-1 Incising Factor for Dimension Lumber.

Design Value	C_i
E_o	0.95
F_{bo}, F_{to}, F_{co}, F_{vo}	0.80
F_{cpo}	1.00

8.4.4.~~4~~8 Deck Factor, C_d

Unless specified otherwise in this article, the deck factor, ~~C_D~~C_d, shall be equal to 1.0.

For stressed wood, nail-laminated, and spike-laminated decks constructed of solid sawn lumber 2.0 in. to 4.0 in. thick, F_{bo} may be adjusted by ~~C_D~~ C_d as specified in Table 1.

C8.4.4.~~4~~8

Mechanically laminated decks made of stressed wood, spike laminated, or nail-laminated solid sawn lumber exhibit an increased resistance in bending. The resistance of mechanically laminated solid sawn lumber decks is calculated by multiplying F_{bo} in Table 8.4.1.1.4-1 by the deck factor.

Deck factor is used instead of the repetitive member factor that is used in NDS.

Table 8.4.4.8-1 Deck Factor for Stressed Wood and Laminated Decks.

Deck Type	Lumber Grade	~~C_D~~C_d
Stressed Wood	Select Structural No. 1 or No. 2	1.30 1.50
Spike-Laminated or Nail-Laminated	All	1.15

For planks 4x6 in, 4x8 in, 4x10 in and 4x12 in., used in plank decks with the load applied to the wide face of planks, F_{bo} may be adjusted by C_d as specified in Table 2.

Table 8.4.4.8-2 Deck Factor for Plank Decks.

Size (in.)	C_d
4x6	1.10
4x8	1.15
4x10	1.25
4x12	1.50

The deck factors for planks in plank decks shall not be applied cumulatively with the flat use factor, C_{fu}, specified in Article 8.4.4.6.

The specified deck factors for planks in plank decks are based test results comparing the modulus of rupture (MOR) for plank specimens with load applied in narrow face and wide face (Stankiewicz and Nowak 1997). These deck factors can be applied cumulatively with the size factor, C_F, specified in Article 8.4.4.4.

8.4.4.~~5~~ 9 Time Effect Factor, C_λ

The time effect factor, ~~C_T~~ C_λ ~~for live load~~ shall be chosen to correspond to the appropriate strength limit state as specified in Table 1.~~taken as 0.8.~~

Table 8.4.4.9-1 Time Effect Factor.

Limit State	**C_λ**
Strength I	0.8
Strength II	1.0
Strength III	1.0
Strength IV	0.6
Extreme Event I	1.0

C8.4.4.9

NDS and AITC 117-2004 reference design values (based on 10-year loading) multiplied by the format conversion factors specified in Article 8.4.4.2, transform allowable stress values to strength level stress values based on 10 minute loading. It is assumed that a cumulative duration of bridge live load is two months and the corresponding time effect factor for Strength I is 0.8. A cumulative duration of live load in Strength II is shorter and the corresponding time effect factor for Strength II is 1.0. Resistance of wood subjected to long-duration loads is reduced. Load combination IV consists of permanent loads, including dead load and earth pressure.

8.5 LIMIT STATES

8.5.1 Service Limit State

The provisions of Article 2.5.2.6.2 should be considered.

8.5.2 Strength Limit State

8.5.2.1 General

Factored resistance shall be the product of nominal resistance determined in accordance with

Article 8.6, 8.7, 8.8, and 8.9 and the resistance factor as specified in Article 8.5.2.2.

8.5.2.2 Resistance Factors

Resistance factors, ϕ, shall be as given below.~~, except for load combination IV, which is specified in Article 8.5.2.3.~~

- Flexure ....$\phi = 0.85$
- Shear ....$\phi = 0.75$
- Compression Parallel to Grain............$\phi = 0.90$
- Compression Perpendicular to Grain...$\phi = 0.90$
- Tension Parallel to Grain ....$\phi = 0.80$
- Resistance During Pile Driving...........$\phi = 1.15$
- Connections ....$\phi = 0.65$

C8.5.2.2

In the case of timber pile foundations, the resistance factor may be raised to 1.0 when, in the judgment of the Engineer, a sufficient number of piles is used in foundation element to consider it to be highly redundant. This is indicated to be a judgment issue because there are no generally accepted quantitative guidelines at this writing.

For timber piles, the resistance factor to be applied when determining the maximum allowable driving resistance accounts for the short duration of the load induced by the pile driving hammer.

~~**8.5.2.3 Modified Resistance Factors**~~

~~For Strength Load Combination IV, Table 3.4.1-1, resistance factors shall be multiplied by 0.75.~~

~~**C8.5.2.3**~~

~~Resistance of wood subjected to long-duration loads is reduced. Load Combination IV consists of permanent loads, including dead load and earth pressure.~~

8.5.2.~~4~~.3 Stability

The structure as a whole or its components shall be proportioned to resist sliding, overturning, uplift, and buckling.

8.5.3 Extreme Event Limit State

For extreme event limit state, the resistance factor shall be taken as 1.0.

8.6 COMPONENTS IN FLEXURE

8.6.1 General

The factored resistance, M_r, shall be taken as:

$$M_r = \phi M_n \qquad (8.6.1\text{-}1)$$

where:

M_n = nominal resistance specified herein

ϕ = resistance factor specified in Article 8.5.2

8.6.2 Rectangular Section

The nominal resistance, M_n, of a rectangular component in flexure shall be determined from:

$$M_n = F_b S C_L \qquad (8.6.2\text{-}1)$$

C8.6.2

If lateral support is provided to prevent rotation at the points of bearing, but no other lateral support is provided throughout the bending component length, the unsupported length, L_u, is the distance between such points of intermediate lateral support.

in which:

$$\cancel{C_s = \frac{1+A}{1.9} - \sqrt{\frac{(1+A)^2}{3.61} - \frac{A}{0.95}} \le C_F}$$

$$C_L = \frac{1+A}{1.9} - \sqrt{\frac{(1+A)^2}{3.61} - \frac{A}{0.95}} \qquad (8.6.2\text{-}2)$$

- ~~For visually graded, sawn lumber:~~

$$\cancel{A = \frac{0.438Eb^2}{L_e dF_b}}$$

$$A = \frac{F_{bE}}{F_b} \qquad (8.6.2\text{-}3)$$

$$F_{bE} = \frac{K_{bE}E}{R_B{}^2} \qquad (8.6.2\text{-}4)$$

$$R_b = \sqrt{\frac{L_e d}{b^2}} \le 50 \qquad (8.6.2\text{-}5)$$

- ~~For glued laminated timber and mechanically graded lumber:~~

$$\cancel{A = \frac{0.609Eb^2}{L_e dF_b}} \qquad \cancel{(8.6.2\text{-}4)}$$

where:

K_{bE}	=	0.76 for visually graded lumber
K_{bE}	=	0.98 for MEL lumber
K_{bE}	=	1.06 for MSR lumber
K_{bE}	=	1.10 for glulam
F_b	=	~~specified~~ adjusted ~~resistance~~ design value in flexure specified in Article 8.4.4 (ksi)
E	=	~~specified~~ adjusted modulus of elasticity specified in Article 8.4.4 (ksi)
~~C_F~~	~~=~~	~~size effect factor specified in Article 8.4.4.2~~
C_L	=	beam stability factor
d	=	net depth specified in Article 8.4.1.1.2 (in.)
b	=	net width, as specified in Article 8.4.1.1.2 (in.)
L_e	=	effective unbraced length (in.)
S	=	section modulus (in.^{3})

Where the depth of a flexural component does not exceed its width, or where lateral movement of the compression zone is prevented by continuous support and where points of bearing have lateral support to prevent rotation, the stability factor, ~~C_s~~ C_L = 1.0. For other conditions, the beam stability factor shall be determined in accordance with the provisions specified herein.

The beam stability factor shall not be applied simultaneous with the volume factor for structural glued laminated timber, therefore, the lesser of these factors shall apply. ~~For the purpose of this article, C_F shall not be taken less than 1.0.~~

The effective unbraced length, L_e, may be determined as:

- If $L_u/d < 7$, then $L_e = 2.06\ L_u$
- If $7 \leq L_u/d \leq 14.3$, then $L_e = 1.63\ L_u + 3d$
- If $L_u/d > 14.3$, then $L_e = 1.84\ L_u$

where:

L_u = distance between point of lateral and rotational support (in.)

d = net depth specified in Article 8.4.1.1.2 (in.)

~~In all cases, the following shall apply:~~

$$\cancel{\sqrt{\frac{L_e d}{b^2}} \leq 50} \qquad \cancel{(8.6.2\text{-}5)}$$

If lateral support is provided to prevent rotation at the points of bearing, but no other lateral support is provided throughout the bending component length, the unsupported length, L_u, is the distance between such points of intermediate lateral support.

8.6.3 Circular Section

The nominal resistance, M_n, of a circular component in flexure shall be taken as:

$$\cancel{M_n = 1.18 F_b S}$$

$$M_n = F_b S \qquad (8.6.3\text{-}1)$$

8.7 COMPONENTS UNDER SHEAR

~~Shear need not be considered in the design of wood decks.~~

~~For components other than decks, s~~Shear shall be investigated at a distance away from the face of support equal to the depth of the component. When calculating the maximum design shear, the live load shall be placed so as to produce the maximum shear at a distance from the support equal to the lesser of either

C8.7

The critical section is between one and three depths from the support.

The critical shear in flexural components is horizontal shear acting parallel to the grain of the component. The resistance of bending components in shear perpendicular to grain need not be investigated.

Note that Eq. 4.6.2.2.2a-1 requires a special distribution factor in the calculation of the live load three times the depth, *d*, of the components or one-quarter of the span *L*.

The factored shear resistance, V_n, of a component of rectangular cross-section shall be calculated from:

$$V_r = \phi V_n \qquad (8.7\text{-}1)$$

in which:

$$V_n = \frac{F_v bd}{1.5} \qquad (8.7\text{-}2)$$

where:

ϕ = resistance factor specified in Article 8.5.2

F_v = ~~specified resistance~~ adjusted design value of wood in shear, specified in Article 8.4.1 (ksi)

8.8 COMPONENTS IN COMPRESSION

8.8.1 General

The factored resistance in compression, P_r, shall be taken as:

$$P_r = \phi P_n \qquad (8.8.1\text{-}1)$$

where:

P_n = nominal resistance as specified in Article 8.8.2 and 8.8.3 (kip)

ϕ = resistance factor specified in Article 8.5.2

8.8.2 Compression Parallel to Grain

~~For the purposes of this article, C_F shall not be taken to be less than 1.0.~~

Where components are not adequately braced, the nominal stress shall be modified by the column stability factor, C_P. If the component is adequately braced, C_P shall be taken as 1.0.

The nominal resistance, P_n, of a component in the compression parallel to grain shall be taken as:

$$\cancel{P_n = F_c A C_P} \; \underline{P_n = F_c A_g C_P} \qquad (8.8.2\text{-}1)$$

in which:

force effect when investigating shear parallel to the grain.

- ~~For sawn lumber:~~

$$C_P = \frac{1+B}{1.6} - \sqrt{\frac{(1+B)^2}{2.56} - \frac{B}{0.80}} \leq C_F \qquad \text{(8.8.2-2)}$$

- ~~For round piles:~~

$$C_P = \frac{1+B}{1.7} - \sqrt{\frac{(1+B)^2}{2.89} - \frac{B}{0.85}} \qquad \text{(8.8.2-3)}$$

~~For glued laminated timber and mechanically graded timber:~~

$$C_P = \frac{1+B}{1.8} - \sqrt{\frac{(1+B)^2}{3.24} - \frac{B}{0.9}} \qquad \text{(8.8.2-4)}$$

$$C_P = \frac{1+B}{c} - \sqrt{\frac{(1+B)^2}{c} - \frac{B}{c}} \qquad \text{(8.8.2-2)}$$

$$B = \frac{F_{cE}}{F_c} \qquad \text{(8.8.2-3)}$$

$$F_{cE} = \frac{K_{cE} E d^2}{L_e^2} \qquad \text{(8.8.2-4)}$$

where:

c = 0.8 for sawn lumber
c = 0.85 for round timber piles
c = 0.9 for glulam
K_{cE} = 0.52 for visually graded lumber
K_{cE} = 0.67 for MEL lumber
K_{cE} = 0.73 for MSR lumber
K_{cE} = 0.76 for glulam and round piles

- ~~For visually graded lumber:~~

- $$B = \frac{0.3Ed^2}{L_e F_b} \qquad \text{(8.8.2-5)}$$

- ~~For glued laminated timber and mechanically graded timber:~~

$$B = \frac{0.418Ed^2}{L_e F_b} \qquad \text{(8.8.2-6)}$$

~~where:~~

F_c = ~~nominal resistance~~ adjusted design value in compression parallel to the grain specified in Article 8.4.4 (ksi)
L_e = effective length taken as KL (in.)
~~K = factor specified in Article 4.6.2.5~~
~~B = parameter for compression~~
~~A_g = gross cross-sectional area of the component (in.^{2})~~

8.8.3 Compression Perpendicular to Grain

The nominal resistance, P_n, of a component in compression perpendicular to the grain shall be taken as:

$$P_n = F_{cp} A_b C_b \quad (8.8.3\text{-}1)$$

where:

F_{cp} = ~~nominal resistance~~ adjusted design value in compression perpendicular to grain, as specified in Article 8.4.4 (ksi)

A_b = bearing area (in.^{2})

C_b = bearing ~~modification~~ adjustment factor specified in Table 1

When the bearing area is in a location of high flexural stress or is closer than 3.0 in. from the end of the component, C_b shall be taken as 1.0. In all other cases, C_b shall be as specified in Table 1.

Table 8.8.3-1 ~~Modification~~ Adjustment Factors for Bearing.

	Length of Bearing measured along the grain, in.						
C_b	0.5	1.0	1.5	2.0	3.0	4.0	≥6.0
	1.75	1.38	1.25	1.19	1.13	1.10	1.00

8.9 COMPONENTS IN TENSION PARALLEL TO GRAIN

The factored resistance of wood in tension shall be taken as:

$$P_r = \phi P_n \quad (8.9\text{-}1)$$

in which:

$$P_n = F_t A_n \quad (8.9\text{-}2)$$

where:

F_t = ~~specified resistance~~ adjusted design value of wood in tension specified in Article 8.4.4 (ksi)

A_n = smallest net cross sectional area of the component (in.^{2})

ϕ = resistance factor specified in Article 8.5.2

8.10 COMPONENTS IN COMBINED FLEXURE AND AXIAL LOADING

8.10.1 Components in Combined Flexure and Tension

Components subjected to combined flexure and tension shall satisfy:

$$\cancel{\frac{P_u}{P_r}+\frac{M_u}{M_r}\le 1}\ \frac{P_u}{P_r}+\frac{M_u}{M_r^{*}}\le 1.0 \qquad (8.10.1\text{-}1)$$

and

$$\frac{M_u-\frac{d}{6}P_u}{M_r^{**}}\le 1.0 \qquad (8.10.1\text{-}2)$$

where:

P_u = factored tensile load (kip)

P_r = factored tensile resistance calculated as specified in Article 8.9 (kip)

M_u = factored flexural moment (kip-in.)

M_r* = F_bS

M_r** = factored flexural resistance adjusted by all applicable adjustment factors except C_V.

~~M_r = factored flexural resistance as specified in Article 8.6 (kip-in.)~~

C8.10.1

Satisfying Eq. 1 ensures that stress interaction on the tension face of the bending member does not cause beam rupture. M_r^* in this formula does not include modification by the beam stability factor, C_L.

Eq. 2 is applied to ensure that the bending/tension member does not fail due to lateral buckling of the compression face.

8.10.2 Components in Combined Flexure and Compression Parallel to Grain

Components subjected to flexure and compression parallel to grain shall satisfy:

$$\cancel{\left(\frac{P_u}{P_r}\right)^2 + \frac{M_u}{M_r} \leq 1}$$

$$\left(\frac{P_u}{P_r}\right)^2 + \frac{M_u}{M_r\left(1 - \frac{P_u}{F_{cE}A_g}\right)} \leq 1.0 \qquad (8.10.2\text{-}1)$$

where:

P_u = factored compression load (kip)

P_r = factored compressive resistance calculated as specified in Article 8.8 (kip)

M_u = factored flexural moment (kip-in.)

M_r = factored flexural resistance calculated as specified in Article 8.6 (kip-in.)

E_{cE} = Euler buckling stress as defined in Equation 8.8.2-4

A_g = gross cross-sectional area

8.11 BRACING REQUIREMENTS

8.11.1 General

Where bracing is required, it shall prevent both lateral and rotational deformation.

C8.11.1

In detailing of the diaphragms, the potential for shrinkage and expansion of the beam and the diaphragm should be considered. Rigidly connected steel angle framing may cause splitting of the beam and diaphragm as the wood attempts to swell and shrink under the effects of cyclic moisture.

8.11.2 Sawn Wood Beams

Beams shall be transversely braced to prevent lateral displacement and rotation of the beams and to transmit lateral forces to the bearings. Transverse bracing shall be provided at the supports for all span lengths and at intermediate locations for spans longer than 20.0 ft. The spacing of intermediate bracing shall be based on lateral stability and load transfer requirements but shall not exceed 25.0 ft. The depth of transverse bracing shall not be less than three-fourths the depth of the stringers or girders.

Transverse bracing should consist of solid wood blocking or fabricated steel shapes. Wood blocking shall be bolted to stringers with steel angles or suspended in steel saddles that are nailed to the blocks an stringer sides. Blocking shall be positively connected

C8.11.2

The effectiveness of the transverse bracing directly affects the long-term durability of the system. The bracing facilitates erection, improves load distribution, and reduces relative movements of the stringers and girders, thereby reducing deck deformations. Excessive deformation can lead to mechanical deterioration of the system.

Bracing should be accurately framed to provide full bearing against stringer sides. Wood cross-frames or blocking that are toe-nailed to stringers have been found to be ineffective and should not be used.

to the beams.

Transverse bracing at supports may be placed within a distance from the center of bearing equal to the stringer or girder depth.

8.11.3 Glued Laminated Timber Girders

Transverse bracing should consist of fabricated steel shapes or solid wood diaphragms.

Girders shall be attached to supports with steel shoes or angles that are bolted through the girder and into or through the support.

C8.11.3

Bracing should be placed tight against the girders and perpendicular to the longitudinal girder axis.

8.11.4 Bracing of Trusses

Wood trusses shall be provided with a rigid system of lateral bracing in the plane of the loaded chord. Lateral bracing in the plane of the unloaded chord and rigid portal and sway bracing shall be provided in all trusses having sufficient headroom. Outrigger bracing connected to extensions of the floorbeams shall be used for bracing through-trusses having insufficient headroom for a top chord lateral bracing system.

C8.11.4

Bracing is used to provide resistance to lateral forces, to hold the trusses plumb and true, and to hold the compression elements in line.

8.12 CAMBER REQUIREMENTS

8.12.1 Glued Laminated Timber Girders

Glued laminated timber girders shall be cambered a minimum of two times the dead load deflection at the service limit state.

C8.12.1

The initial camber offsets the effects of dead load deflection and long-term creep deflection.

8.12.2 Trusses

Trusses shall be cambered to sufficiently offset the deflection due to dead load, shrinkage, and creep.

C8.12.2

Camber should be determined by considering both elastic deformations due to applied loads, and inelastic deformations such as those caused by joint slippage, creep of the timber components, or shrinkage due to moisture changes in the wood components.

8.12.3 Stress Laminated Timber Deck Bridge

Deck bridges shall be cambered for three times the dead load deflection at the service limit state.

8.13 CONNECTION DESIGN

The design of timber connections using mechanical fasteners including, wood screws, nails, bolts, lag screws, drift bolts, drift pins, shear plates, split rings, and timber rivets shall be in accordance with the 2005 NDS®.

References:

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2006 AASHTO BRIDGE COMMITTEE AGENDA ITEM: 25

SUBJECT: Revision to Section 2 of the Bridge Welding Code

TECHNICAL COMMITTEE: T-17 Welding

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

☐ **DESIGN SPEC** ☐ **CONSTRUCTION SPEC** ☐ **MOVABLE SPEC**
☐ **LRFR MANUAL** ☒ **OTHER Bridge Welding Code**

☐ **US VERSION** ☐ **SI VERSION** ☒ **BOTH**

DATE PREPARED: 1/13/06
DATE REVISED:

AGENDA ITEM:

2.4 General

~~In general, stress concentrations should be avoided. This may be accomplished by sizing parts and organizing components to minimize constraint against ductile behavior, and avoiding unnecessary concentrations of welds, particularly where there are short unwelded portions of base metal between welds. Welds should not be larger than necessary. Welds should be sized to carry required loads at appropriate design stresses. Excess weld metal increases residual stress, and, when carried to extreme, may result in unacceptable distortion, cracks or lamellar tears. The organization of parts in weld assemblies and details of welded joints shall afford ample access for the deposition of all required weld passes.~~

<u>Weld connections shall be designed and detailed to satisfy the strength, stiffness, flexibility, and fatigue requirements of the AASHTO or other applicable design specifications.</u>

OTHER AFFECTED ARTICLES:

Commentary Section C2.4

BACKGROUND:

Improved language related to designed weld connections.

ANTICIPATED EFFECT ON BRIDGES:

None

REFERENCES:

None

OTHER:

None

2006 AASHTO BRIDGE COMMITTEE AGENDA ITEM: 26 (REVISION 1)

SUBJECT: Revision to Section 2 Commentary of the Bridge Welding Code

TECHNICAL COMMITTEE: T-17 Welding

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

☐ **DESIGN SPEC** ☐ **CONSTRUCTION SPEC** ☐ **MOVABLE SPEC**
☐ **LRFR MANUAL** ☒ **OTHER Bridge Welding Code**

☐ **US VERSION** ☐ **SI VERSION** ☒ **BOTH**

DATE PREPARED: 1/13/06
DATE REVISED: 5/23/06

AGENDA ITEM:

C2.4 Because bridges are dynamically loaded structures, fatigue is a very important design consideration. Fatigue is the result of repeated local plastic (inelastic) deformation. With enough cycles of such deformations, fatigue cracks can initiate and propagate. In some cases, stresses resulting from applied loads are elastic in a global sense, but localized areas may exist where the stresses concentrate or combine with internal stresses and exceed the yield strength of the metal. In welded construction, two factors can cause this to occur; stress concentrations and residual stresses from welding.

Stress concentrations can occur due to geometric changes in a member and discontinuities in the base metal or weldment. There are many examples of geometric changes, some of which include changes in width and/or thickness of flange plates, welded cover plates, and even reinforcement of a groove weld joining plates of similar size. Discontinuities can include laminations and nicks in base metal, as well as, cracks, porosity or slag inclusions in welds. The orientation of an imperfection can determine how severely it affects the member. This ~~in~~ is why base metal laminations parallel to the direction of applied stress are acceptable within certain limitations.

Residual stresses also affect fatigue performance. Residual stresses exist in any welded structure due to shrinkage during cooling of the weld. This can be a particular concern in welding highly restrained members. Even though the stress resulting from applied loads to a welded member may be within the elastic range, the added effect of the residual stresses may result in an inelastic level at the welds.

AASHTO Design specifications require that both the static strength, and the fatigue strength, be considered in design. The allowable fatigue stress ranges are based upon full scale testing of as-welded components that replicate typical bridge details. These allowable stress ranges incorporate the effects of geometric stress concentrations, allowable discontinuities, and residual stresses. Welded connections and connection details must comply with these design parameters.

OTHER AFFECTED ARTICLES:

Section 2.4

BACKGROUND:

Add commentary language for Section 2.4 related to designed weld connections.

ANTICIPATED EFFECT ON BRIDGES:

None

REFERENCES:

None

OTHER:

None

2006 AASHTO BRIDGE COMMITTEE AGENDA ITEM: 27

SUBJECT: Revision to Section 5 of the Bridge Welding Code

TECHNICAL COMMITTEE: T-17 Welding

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

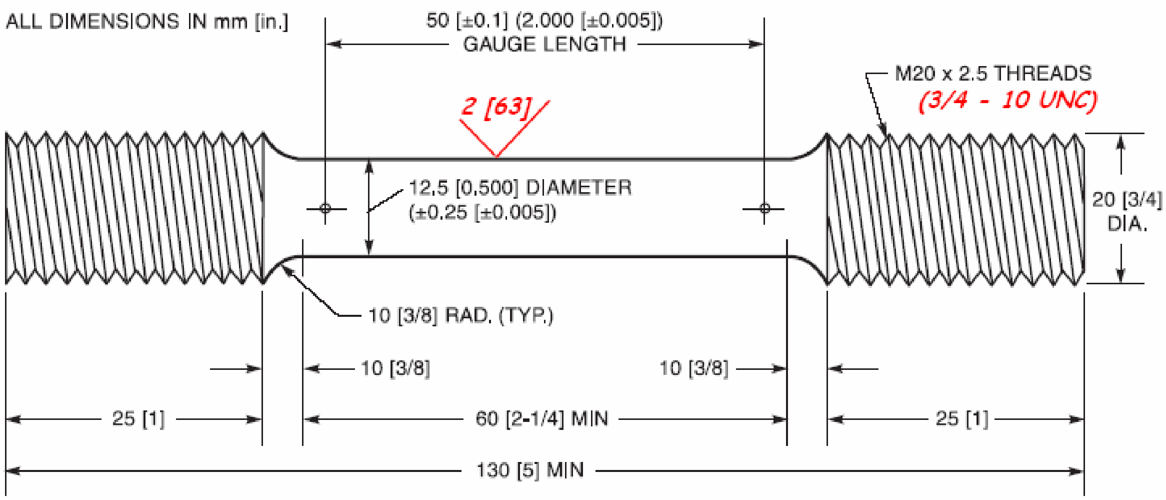
☐ **DESIGN SPEC** ☐ **CONSTRUCTION SPEC** ☐ **MOVABLE SPEC**
☐ **LRFR MANUAL** ☒ **OTHER Bridge Welding Code**

☐ **US VERSION** ☐ **SI VERSION** ☒ **BOTH**

DATE PREPARED: 1/13/06
DATE REVISED:

AGENDA ITEM:

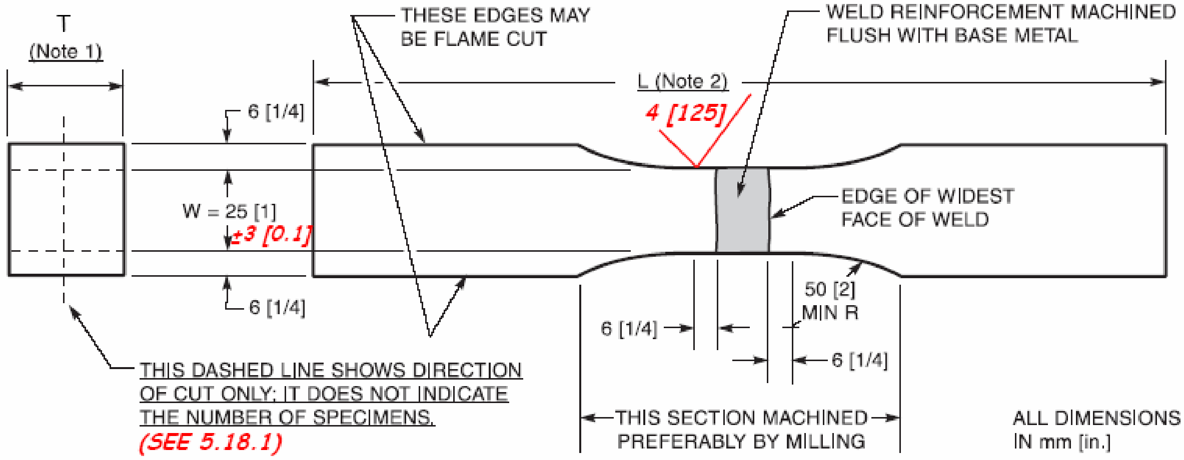
Figures 5.9 through 5.16:



General Notes:

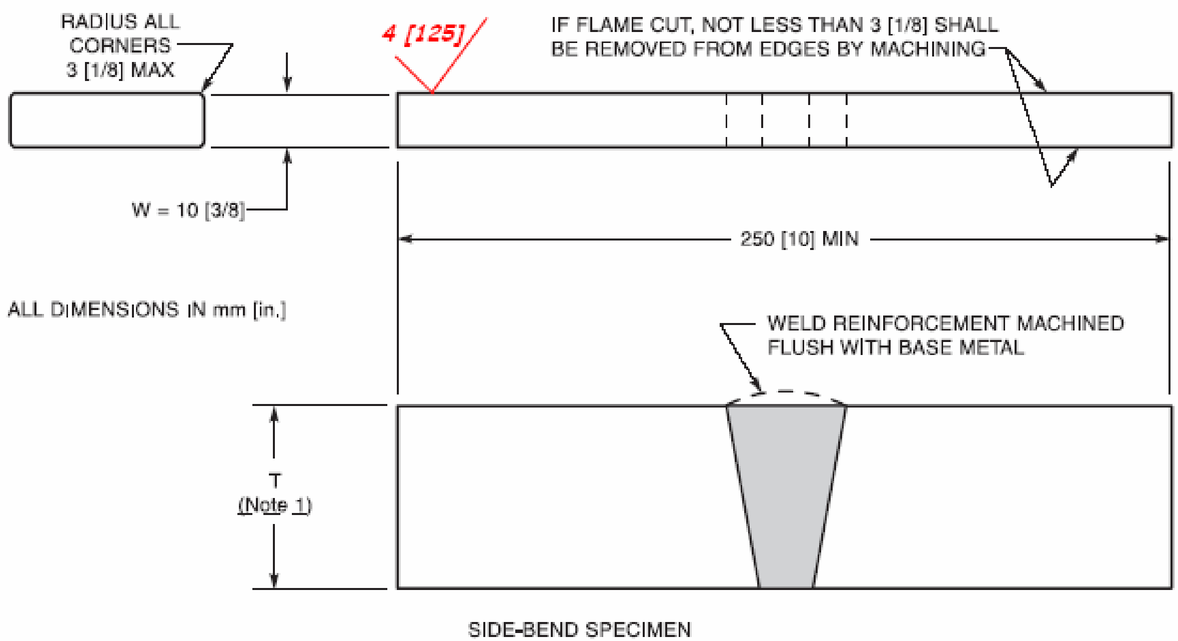
- The reduced section may have a gradual taper from the ends toward the center with the ends not more than 0.1 mm [0.005 in.] larger in diameter than the center.
- The all-weld metal tension specimen shall be taken from the center of the thickness of the weld, and from the center of the width of the weld at this location.

Figure 5.9—Standard Round All-Weld-Metal Tension Specimen (see 5.16.3)



Notes:
1. T depends on the thickness of test plate shown in Figure 5.1 or Figure 5.3; see 5.6.
2. L shall be the overall length of the test specimen. The length shall be sufficient to facilitate gripping in the testing apparatus. When practicable, the specimen should extend into the grips a distance greater than or equal to 2/3 the length of the grip.
3. Weld reinforcement and steel backing, if any, shall be removed flush with the surface of the specimen.

Figure 5.10—Reduced Section Tension Specimen (see 5.16.3)



Note:
1. T depends on the thickness of test plate shown in Figures 5.1, 5.2, and 5.3; see 5.6.
If T > 40 mm [1-1/2 in.], see AWS B4.0, Figure A5, notes 1 & 2 for guidance on cutting the specimen into strips between 20 and 40 mm [3/4 and 1-1/2 in.] wide.

Figure 5.11—Side-Bend Specimen (see 5.16.3)

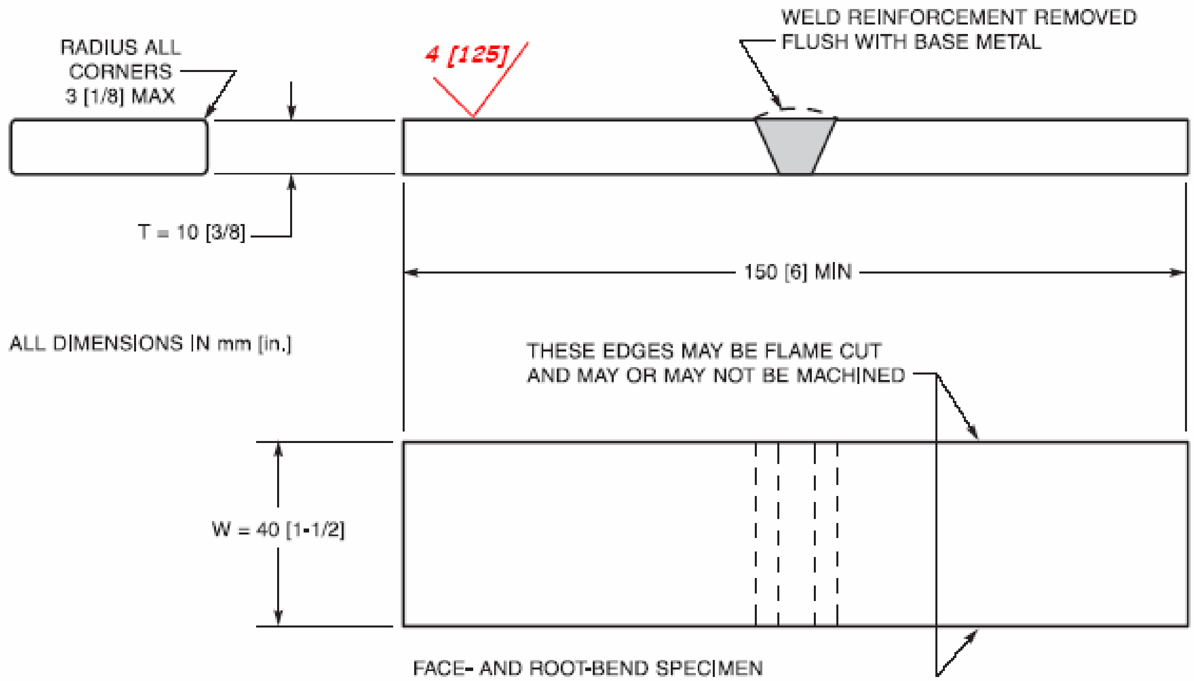
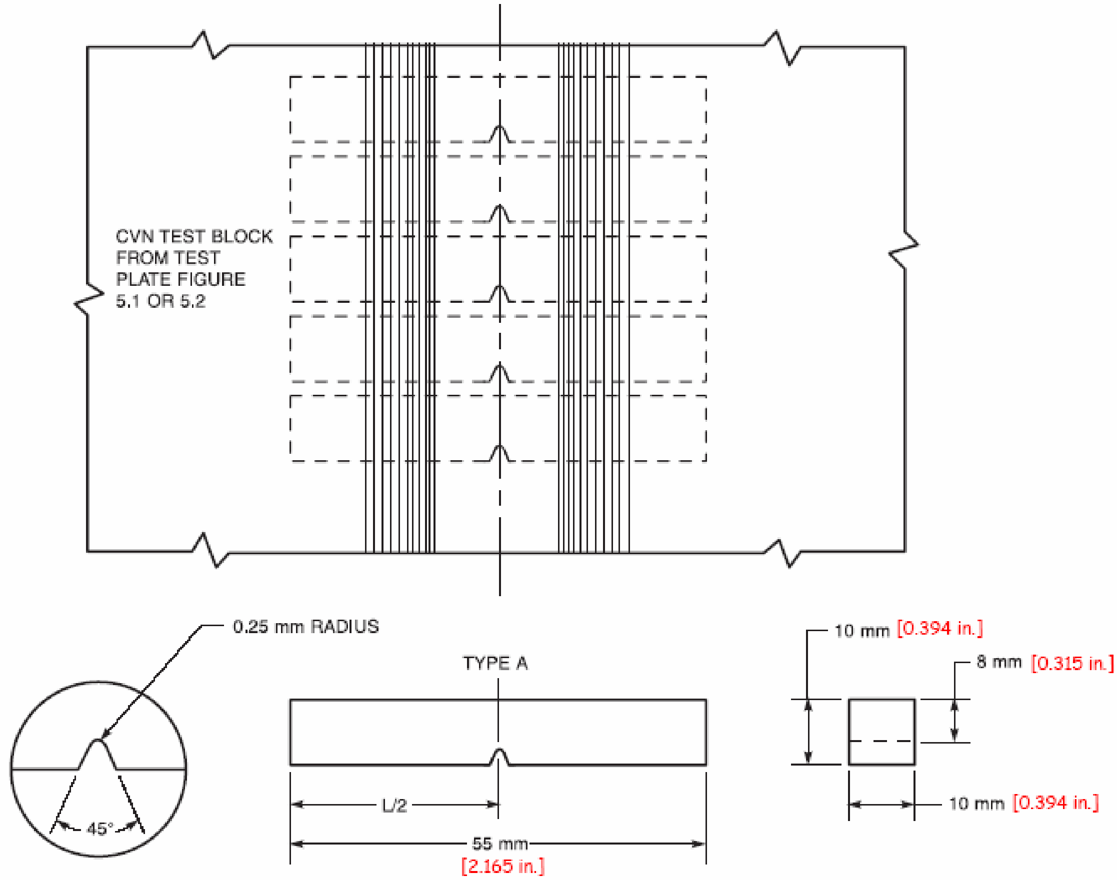


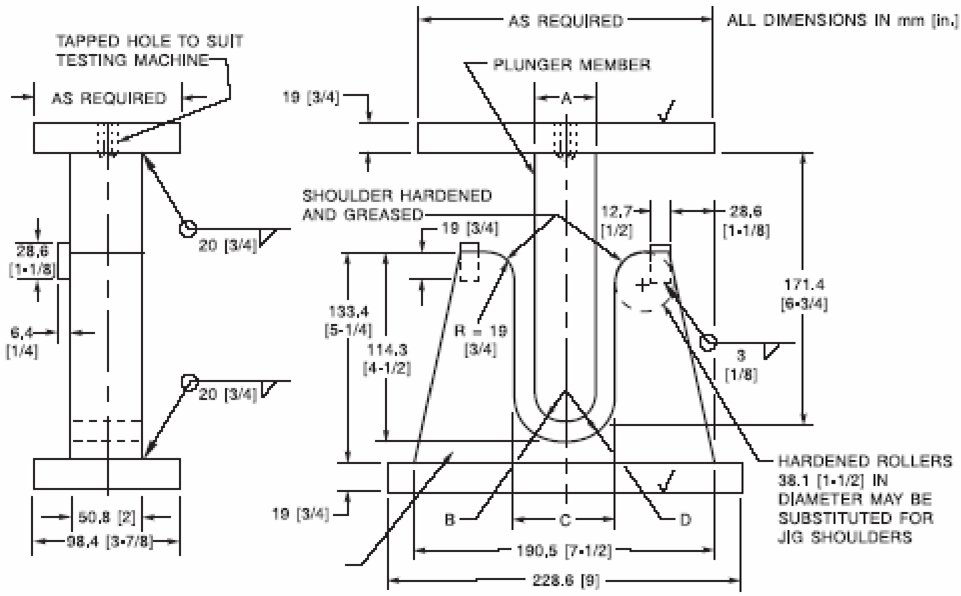
Figure 5.12—Face- and Root-Bend Specimen (see 5.16.3)



Notch length to edge	90 ± 2°
Note—Allowable variations shall be as follows:	
Adjacent sides ~~shall be at~~	90° ± 10 min
Cross section dimensions	~~±0.025 mm~~ ±0.075 mm [±0.003 in.]
Length of specimen (L)	±0, −2.5 mm [−0.100 in.]
Centering of notch (L/2)	±1 mm [±0.04 in.]: When an end-centering device is necessary to center the specimen in the anvil (see ~~11.2.2~~ 8.3.2, ASTM E 23, *Standard Methods for Notched Bar Impact Testing of Metallic Materials*), it shall be necessary that the notch be accurately centered to ensure compliance with ~~4.2.1~~ A1.10.2 (ASTM E 23).
Angle of notch	±1°
Radius of notch	±0.025 mm [±0.001 in.]
Depth ~~Dimensions to bottom~~ of notch: ~~Type A specimen~~	2 ± 0.025 mm [±0.001 in.]
Finish requirements	2 µm [63 µin] on notched surface and opposite face; 4 µm [125 µin] on other two surfaces

General Note: Five test specimens shown, eight required for ESW and EGW.

Figure 5.13—CVN Test Specimen—Type A (see 5.16.4)



Minimum Specified Base Metal Yield Strength, MPa [ksi]	A mm [in.]	B mm [in.]	C mm [in.]	D mm [in.]
345 [50] and under	38.1 [1-1/2]	19.0 [3/4]	60.3 [2-3/8]	30.2 [1-3/16]
Over 345 [50] to 620 [90]	50.8 [2]	25.4 [1]	73.0 [2-7/8]	36.6 [1-7/16]
620 [90] and over	63.5 [2-1/2]	31.8 [1-1/4]	85.7 [3-3/8]	42.9 [1-11/16]

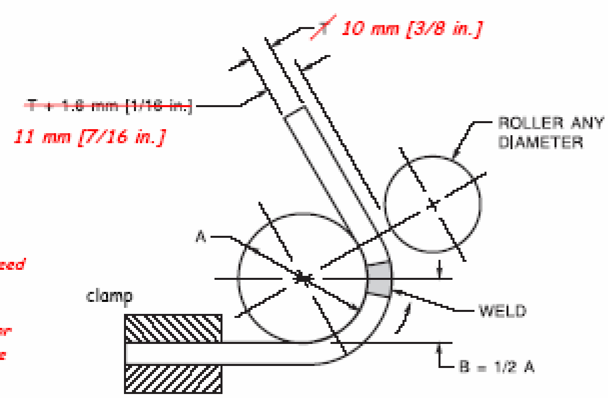
Notes:

1. ~~General Note:~~ Plunger and interior die surfaces shall be machine-finished.
2. *The diameter A of the plunger shall equal or exceed the weld face width (after machining). If this requirement cannot be met, see AWS B4.0M or B4.0 for guidance on adjusting the specimen thickness and fixture dimensions.*

Figure 5.14—Guided Bend Test Jig (see 5.18.3)

Notes:

1. *Minimum roller length shall be 50 mm [2 in.].*
2. *Diameter A shall equal or exceed the weld face width (after machining). If this requirement cannot be met, see AWS B4.0M or B4.0 for guidance on adjusting the specimen thickness and fixture dimensions.*



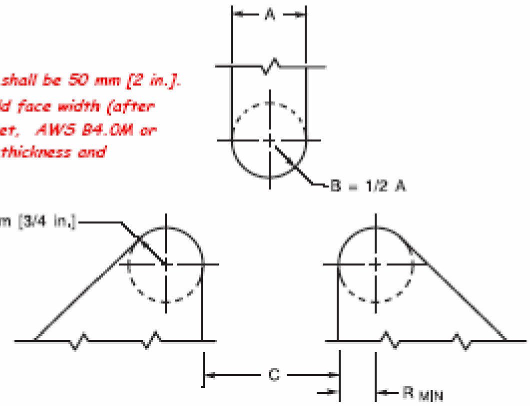
Minimum Specified Base Metal Yield Strength, MPa [ksi]	A mm [in.]	B mm [in.]
345 [50] and under	38.1 [1-1/2]	19.0 [3/4]
Over 345 [50] to 620 [90]	50.8 [2]	25.4 [1]
620 [90] and over	63.5 [2-1/2]	31.8 [1-1/4]

Figure 5.15—Alternate Wraparound Guided Bend Test Jig (see 5.18.3)

Notes:

1. Minimum roller length (or shoulder width) shall be 50 mm [2 in.].

2. Diameter A shall equal or exceed the weld face width (after machining). If this requirement cannot be met, AWS B4.0M or B4.0 for guidance on adjusting the specimen thickness and fixture dimensions.



Minimum Specified Base Metal Yield Strength, MPa [ksi]	A mm [in.]	B mm [in.]	C mm [in.]
345 [50] and under	38.1 [1-1/2]	19.0 [3/4]	60.3 [2-3/8]
Over 345 [50] to 620 [90]	50.8 [2]	25.4 [1]	73.0 [2-7/8]
620 [90] and over	63.5 [2-1/2]	31.8 [1-1/4]	85.7 [3-3/8]

Figure 5.16—Alternate Roller-Equipped Guided Bend Test Jig for Bottom Ejection of Test Specimen (see 5.18.3)

OTHER AFFECTED ARTICLES:

None

BACKGROUND:

Figures from AWS B4.0 are reproduced in D1.5 but are missing some provisions. Also there are inconsistencies in the labeling of Figs. 5.11, 5.12, and 5.15 ("T" in 5.15 is "T" in 5.12 but "W" in 5.11).

ANTICIPATED EFFECT ON BRIDGES:

None

REFERENCES:

None

OTHER:

None

2006 AASHTO BRIDGE COMMITTEE AGENDA ITEM: 28

SUBJECT: Revision to Section 6 of the Bridge Welding Code

TECHNICAL COMMITTEE: T-17 Welding

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

☐ **DESIGN SPEC** ☐ **CONSTRUCTION SPEC** ☐ **MOVABLE SPEC**
☐ **LRFR MANUAL** ☒ **OTHER Bridge Welding Code**

☐ **US VERSION** ☐ **SI VERSION** ☒ **BOTH**

DATE PREPARED: 1/13/06
DATE REVISED:

AGENDA ITEM:

6.1.3.4 Personnel Qualification. Personnel performing NDT shall be ~~qualified~~ certified in conformance with the American Society for Nondestructive Testing's (ASNT) *Recommended Practice No. SNT-TC-1A*, or an equivalent satisfactory to the engineer. Certification of Level I and Level II individuals shall be performed by the ASNT Central Certification Program (ACCP), or a Level III individual who has been certified by (1) ASNT, or (2) has the education, training, experience, and has successfully passed the written examinations prescribed in *ASNT SNT-TC-1A*. Individuals who perform NDT shall be ~~qualified for~~ certified as:
(1) NDT Level II, or
(2) NDT Level I working under the direct supervision of an individual qualified for NDT Level II or NDT Level III.
(3) NDT Level III and qualified per ASNT SNT-TC-1A as a Level II for NDT performed.

OTHER AFFECTED ARTICLES:

None

BACKGROUND:

Develop language for NDT certification such that Sections 6 and 12 are consistent.

ANTICIPATED EFFECT ON BRIDGES:

None

REFERENCES:

None

OTHER:

None

2006 AASHTO BRIDGE COMMITTEE AGENDA ITEM: 30

SUBJECT: Revision to Section 12 of the Bridge Welding Code

TECHNICAL COMMITTEE: T-17 Welding

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

☐ **DESIGN SPEC** ☐ **CONSTRUCTION SPEC** ☐ **MOVABLE SPEC**
☐ **LRFR MANUAL** ☒ **OTHER Bridge Welding Code**

☐ **US VERSION** ☐ **SI VERSION** ☒ **BOTH**

DATE PREPARED: 1/13/06
DATE REVISED:

AGENDA ITEM:

12.16.1.2 NDT Technicians. NDT technicians shall be ~~qualified~~ certified to ~~ASNT~~ Level II, or ~~III~~ certified to Level III and qualified to perform as a Level II in conformance with ASNT's *Recommended Practice SNT-TC-1A*. Level II technicians shall be supervised by an individual ~~qualified~~ certified to Level III. Level III individuals shall possess a currently valid ASNT Level III certificate. The Engineer may accept alternative qualifications which are deemed equivalent.

OTHER AFFECTED ARTICLES:

None

BACKGROUND:

Develop language for NDT certification such that Sections 6 and 12 are consistent.

ANTICIPATED EFFECT ON BRIDGES:

None

REFERENCES:

None

OTHER:

None

2006 AASHTO BRIDGE COMMITTEE AGENDA ITEM: 29

SUBJECT: Revision to Section 6 Commentary of the Bridge Welding Code

TECHNICAL COMMITTEE: T-17 Welding

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

☐ **DESIGN SPEC** ☐ **CONSTRUCTION SPEC** ☐ **MOVABLE SPEC**
☐ **LRFR MANUAL** ☒ **OTHER Bridge Welding Code**

☐ **US VERSION** ☐ **SI VERSION** ☒ **BOTH**

DATE PREPARED: 1/13/06
DATE REVISED:

AGENDA ITEM:

C6.1.3.4 ASNT Personnel Qualification. The code requires NDT personnel ~~shall~~ be ~~qualified~~ certified under a written practice developed in general conformance with ASNT's *Recommended Practice No. SNT-TC-1A, Personnel Qualification and Certification in Nondestructive Testing*, or an equivalent program. Other programs may include the AWS NDE Certification Program and the ANSI/ASNT CP-189, *ASNT Standard for Qualification and Certification of Nondestructive Testing Personnel.*

An NDT Level I individual has the skills to perform specific calibrations, specific NDT, and with prior written approval of the NDT Level III, perform specific interpretations and evaluations for acceptance or rejection and document the results, while under the direct ~~and frequent~~ supervision of a Level II.

An NDT Level II individual has the skills and knowledge to set up and calibrate equipment, to conduct tests, and to interpret, evaluate and document results in conformance with procedures approved by an NDT Level III. The Level II ~~shall~~ should be thoroughly familiar with the scope and limitations of the method to which certified, and should be capable of directing the work of NDT Level I personnel. The NDT Level II ~~shall~~ should be able to organize and report NDT results. The Level II will monitor, and approve the results of Level I personnel on at least a daily basis. The Level II is responsible for providing guidance to Level I and reviewing and signing all test reports generated by the Level I.

An ASNT NDT Level III individual has the skills and knowledge to establish techniques; to interpret codes, standards, and specifications; designate the particular technique to be used; and verify the accuracy of procedures. The individual should also have general familiarity with the other NDT methods. The NDT Level III is responsible for conducting or directing the training and examining of NDT personnel in the methods for which the NDT Level III ~~shall~~ will be qualified.

The Level III ~~shall~~ should have ~~taken written and practical tests as required~~ passed the basic and method examinations prescribed by SNT-TC-1A. The SNT-TC-1A tests required for Level III certification may be administered by ASNT or an independent third party deemed acceptable by the Engineer.

The code requires that NDT of non-FCM materials be performed by Level II technicians, or by Level I technicians only when working under the direct supervision of a Level II. Inspection by a Level III ~~shall~~ may not be recognized, as the Level III may not perform actual testing regularly enough to maintain the special skills required to set up or to conduct the tests, nor is a hands-on "practical" examination required for certification, unless certified under ASNT's ACCP testing. An ASNT Level III may conduct tests if that person ~~also~~ has passed a practical examination and holds a Level II certification. For Fracture Critical Members, under 12.16.1.2, testing of Fracture Critical Members ~~shall~~ should be done by either a ~~qualified~~ certified Level II under the supervision of a qualified Level III, or by a Level III certified by ASNT and qualified as a Level II, unless the Engineer accepts other forms of qualification.

OTHER AFFECTED ARTICLES:

Section 2.4

BACKGROUND:

Develop language for NDT certification such that Sections 6 and 12 are consistent.

ANTICIPATED EFFECT ON BRIDGES:

None

REFERENCES:

None

OTHER:

None

2006 AASHTO BRIDGE COMMITTEE AGENDA ITEM: 31

SUBJECT: Revision to Section 12 Commentary of the Bridge Welding Code

TECHNICAL COMMITTEE: T-17 Welding

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

☐ **DESIGN SPEC** ☐ **CONSTRUCTION SPEC** ☐ **MOVABLE SPEC**
☐ **LRFR MANUAL** ☒ **OTHER Bridge Welding Code**

☐ **US VERSION** ☐ **SI VERSION** ☒ **BOTH**

DATE PREPARED: 1/13/06
DATE REVISED:

AGENDA ITEM:

C12.16.1.2 NDT Technicians. The code requires ~~P~~personnel performing NDT ~~shall~~ be ~~qualified~~ certified as an NDT Level II or Level III that has passed a practical examination and is also qualified as a Level II, in conformance with ASNT Recommended Practice SNT-TC-1A. These requirements are more ~~restrictive~~ stringent than is required for NDT of non-fracture critical members by allowing the testing to be performed only by individuals ~~qualified~~ certified as NDT Level II and working under the supervision of ~~an~~ a certified ~~NDT~~ ASNT Level III, or ~~qualified~~ by a certified ~~as NDT~~ Level III qualified as a Level II. To ensure the capability of the Level III persons, they ~~shall~~ are required by code to be certified through ASNT testing or ~~the~~ testing equivalent as determined by the Engineer. The term "under the supervision" means that the NDT Level III person will be available, as necessary, and will personally oversee and independently verify the NDT Level II technician's work on a periodic basis.

OTHER AFFECTED ARTICLES:

None

BACKGROUND:

Develop language for NDT certification such that Sections 6 and 12 are consistent.

ANTICIPATED EFFECT ON BRIDGES:

None

REFERENCES:

None

OTHER:

None

2006 AASHTO BRIDGE COMMITTEE (REVISION 3) – 5/25/06

SUBJECT: LRFD Bridge Design Specifications

Editorial revisions and additions to various articles of the AASHTO LRFD Bridge Design Specifications

2006 EDITORIAL CHANGES - DESIGN

Location of Change	US/SI/ Both	Current Text	Proposed Text
Article C5.8.2.8, 2nd paragraph,	Both	…and Bruce (*to be published*) has indicated…	…and Bruce (~~*to be published*~~ *2003*) has indicated…
Article 5.8.3.4.1, 1st sentence	Both	… is less than $3d_y$…	… is less than $3d_v$…
Article C5.10.11.4.2, 2nd paragraph, 1st sentence – Note: Move this paragraph across from the 2nd paragraph of specification	Both	…reinforcements in piers, which are short in …	…reinforcements in piers, which ~~are~~ have a horizontal dimension that is short in …
Article 5.11.2.5.1, 2nd paragraph	Both	…critical section, shall satisfy:	…critical section, shall satisfy the lesser of:
Equations 5.13.2.4.2-1 through 5.13.2.4.2-4	Both	=	$\leq$
Article C5.14.2.5, 6th paragraph	SI	6,000 psi	41 MPa
Section 5, References	Both	Bruce, R. N., H. G. Russell, and J. J. Roller. To be published. "Fatigue and Shear Behavior of HPC Bulb-Tee Girders." Louisiana Transportation Research Center, Baton Rouge, LA.	Bruce, R. N., H. G. Russell, and J. J. Roller. ~~To be published~~ 2003. "Fatigue and Shear Behavior of HPC Bulb-Tee Girders," Interim Report - Louisiana Transportation Research Center, Report No. FHWA/LA. 03/382, Baton Rouge, LA, 58 pp.
Section 5, References	Both	---	AASHTO. 1989. *Guide Specification for Design and Construction of Segmental Concrete Bridges*, American Association of State Highway and Transportation Officials, Washington, DC.
Article 6.6.2, 2nd paragraph, 1st bullet	Both	• Splice plates and filler plates in bolted splices;	• Splice plates and filler plates in bolted splices connected in double shear;
Article C6.7.4.2, 2nd paragraph	Both	Intermediate diaphragms or cross-frames should be spaced as nearly uniform as practical to ensure that the flange resistance equations are appropriate.	Intermediate diaphragms or cross-frames should be provided at nearly uniform spacing in most cases, for efficiency of the structural design, for constructibility, and/or to allow the use of simplified methods of analysis for calculation of

			flange lateral bending stresses, such as those discussed in Articles C4.6.1.2.4b, C4.6.2.7.1 and C6.10.3.4. Closer spacings may be necessary adjacent to interior piers, in the vicinity of skewed supports, and in some cases, near midspan.
Article 6.10.1.5, 2nd bullet	US	• For permanent loads applied to composite sections: the stiffness properties of the long-term sections: the stiffness properties of the long-term composite section, assuming the concrete deck to be effective over the entire span length.	• For permanent loads applied to composite sections: ~~the stiffness properties of the long-term sections:~~ the stiffness properties of the long-term composite section, assuming the concrete deck to be effective over the entire span length.
Article C6.10.1.6, 10th paragraph	Both	…as specified in Article A6.3.3.	…as specified in Article A6.3.3. The elastic buckling stress is the appropriate stress for use in Eqs. 4 and 5 to estimate the elastic second-order amplification of the flange lateral bending stresses.
Article C6.10.1.9.1, 3rd paragraph	Both	$D_c \neq D$	$D_c \neq 0.5D$
Article C6.10.1.10.2, 10th paragraph	Both	The previous Specifications defined sections as compact or noncompact and did not explicitly distinguish between a noncompact and a slender web. It should be noted that these web classifications apply only to composite sections in negative flexure and noncomposite sections.	The previous Specifications defined sections as compact or noncompact and did not explicitly distinguish between a noncompact and a slender web. ~~It should be noted that these web classifications apply only to composite sections in negative flexure and noncomposite sections.~~ The classification of webs as compact, noncompact, or slender in these Specifications apply to composite sections in negative flexure and noncomposite sections. These classifications are consistent with those in AISC (*2005*). For composite sections in positive flexure, these Specifications still classify the entire cross-section as compact or noncompact based on the criteria in Article 6.10.6.2.2. The Article 6.10.6.2.2 classification includes consideration of the web slenderness as well as other cross-section characteristics.
Article 6.10.8.2.3, where list – Note: also revise the Notation list	Both	f_0 = stress without… largest compression at this point, or the smallest…	f_0 = stress without… largest compression at this point in the flange under consideration, or the smallest…
Article C6.10.10.4.2, 4th paragraph, 1st sentence	Both	Eq. 6	Eq. 8
Article 6.10.11.1.1, 4th paragraph	US	…less than $4t_w$ or more than the lesser of $6t_w$ and 4.0 in.	…less than $4t_w$ ~~or more than~~ , but not exceed the lesser of $6t_w$ and 4.0 in.
Article 6.10.11.1.1, 4th paragraph	SI	…less than $4t_w$ or more than the lesser of $6t_w$ and 100 mm.	…less than $4t_w$ ~~or more than~~ , but not exceed the lesser of $6t_w$ and 100 mm.

Equation 6.10.11.1.3-3	Both	$= \quad 2.5\left(\frac{D}{d_o/\boldsymbol{D}}\right)^2 - 2.0 \geq 0.5$	$J = \frac{2.5}{(d_o / D)^2} - 2.0 \geq 0.5$
Article C6.10.11.3.1, 6th paragraph, 3rd sentence	Both	Thus, full nominal yielding of the stiffeners is not permitted at the strength limit state as an upper bound.	Thus, full nominal yielding of the stiffeners is not permitted at the strength limit state and when checking constructibility as an upper bound.
Article C6.10.11.3.1, 6th paragraph, 5th sentence	Both	To account for…stiffener stress at the strength limit state in hybrid members,…	To account for…stiffener stress ~~at the strength limit state~~ in hybrid members,…
Article C6.10.11.3.1, 6th paragraph, 6th sentence	Both	It is recommended that the value of R_h applied to the flanges at the strength limit state at the section under consideration be applied in Eq. 1.	~~It is recommended that the value of R_h applied to the flanges at the strength limit state at the section under consideration be applied in Eq. 1.~~ For the strength limit state and constructibility checks, the corresponding value of R_h at the section under consideration should be applied in Eq. 1.
Article 6.10.11.3.3, where list	Both	r = radius of gyration… If F_{yw} is smaller than F_{ys}, the strip of the web included in the effective section shall be reduced by the ratio F_{yw}/F_{ys}.	r = radius of gyration… ~~If F_{yw} is smaller than F_{ys}, the strip of the web included in the effective section shall be reduced by the ratio F_{yw}/F_{ys}.~~
Article C6.10.11.3.3, 1st paragraph, last sentence	Both	…column section by F_{yw}/F_{ys}.	…column section by F_{yw}/F_{ys} in the calculation of the moment of inertia of the longitudinal stiffener.
Article 6.11.1, 1st paragraph, 1st sentence	Both	The provisions of this article apply to flexure of straight or horizontally curved steel single or multiple closed-box or tub sections symmetrical about the vertical axis in the plane of the web in simple or continuous bridges of moderate length.	The provisions of this article apply to flexure of straight or horizontally curved steel single or multiple closed-box or tub sections ~~symmetrical about the vertical axis in the plane of the web~~ in simple or continuous bridges of moderate length.
Article 6.11.8.2.2, R_2 definition	Both	R_2 = constant which when… is equal to F_{yr}	R_2 = constant which when… is equal to ~~F_{yr}~~ R_bF_{yr}
Article 6.13.1, 6th paragraph	Both	Unless expressly permitted by…	Unless ~~expressly~~ otherwise permitted by…
Article 6.14.2.8, 5th paragraph, variable (two revisions needed)	Both	F_u	F_y
Section 6, References	Both	See ATTACHMENT A	
Article CA6.2.1, 4th paragraph, 2nd sentence	Both	For a compact web section, the web plastification factors are equivalent to the shape factors.	For a compact web section, the web plastification factors are equivalent to the cross-section shape factors.
Article CA6.2.2, 1st paragraph, 2nd sentence	Both	As $2D_c/t_w$ approaches the noncompact web limit λ_{rw}, R_{pc} and R_{pt} approach values equal to 1.0 and the maximum potential flexural resistance expressed within the subsequent limit state equations is limited to the yield moment capacity M_y.	As $2D_c/t_w$ approaches the noncompact web limit λ_{rw}, R_{pc} and R_{pt} approach values equal to ~~1.0~~ R_h and the maximum potential flexural resistance expressed within the subsequent limit state equations ~~is limited to the yield moment capacity M_y~~ approaches a limiting value of R_hM_y.

Article CA6.2.2, 3rd paragraph, 2nd sentence	Both	In cases where D_c/D > 0.5, D_{cp}/D is typically smaller than D_c/D; therefore, $\lambda_{pw(D_c)}$ is smaller than $\lambda_{pw(D_{cp})}$.	In cases where D_c/D > 0.5, D_{cp}/D is typically ~~smaller~~ larger than D_c/D; therefore, $\lambda_{pw(D_c)}$ is smaller than $\lambda_{pw(D_{cp})}$.
Article CA6.2.2, 3rd paragraph, 3rd sentence	Both	However, when D_c/D < 0.5, D_{cp}/D is typically larger than D_c/D and $\lambda_{pw(D_c)}$ is larger than $\lambda_{pw(D_{cp})}$.	However, when D_c/D < 0.5, D_{cp}/D is typically ~~larger~~ smaller than D_c/D and $\lambda_{pw(D_c)}$ is larger than $\lambda_{pw(D_{cp})}$.
Article D6.1, 1st paragraph, 1st sentence	Both	The plastic moment, M_p, shall be calculated as the first moment of plastic forces about the plastic neutral axis.	The plastic moment, M_p, shall be calculated as the ~~first moment of~~ moment of the plastic forces about the plastic neutral axis.
Article D6.1, 3rd bullet	Both	• Calculating M_p. Equations for the five cases most likely to occur in practice are given in Table 1.	• Calculating M_p. Equations for the ~~five cases most likely to occur in practice~~ various potential locations of the plastic neutral axis (PNA) are given in Table 1.
Article D6.1, 7th paragraph, last sentence	Both	Conditions should be checked in the order listed below.	~~Conditions~~ The condition should be checked in the order listed ~~below~~ in Tables 1 and 2.
Table D6.1-1, Case VI, Condition	Both	$P_t + P_w + P_c + P_{rb} \ge \left(\frac{C_{rb}}{t_s}\right) P_s + P_{rt}$	$P_t + P_w + P_c + P_{rb} + P_{rt} \ge \left(\frac{C_{rt}}{t_s}\right) P_s$
Table D6.1-1, Case VII, $\bar{Y}$ AND M_p	Both	$\bar{Y} = (t_s)\left[\frac{P_{rb} + P_c + P_w + P_t - P_{rt}}{P_s}\right]$	$\bar{Y} = (t_s)\left[\frac{P_{rb} + P_c + P_w + P_t + P_{rt}}{P_s}\right]$
Table D6.1-1, Note: change uppercase C to lowercase c throughout table	Both	C	c
Figure underneath Table D6.1-1	Both	---	Change the label underneath the rightmost sketch underneath Table D6.1-1 to read ‘CASE V’. Add a dimension to this particular sketch showing ‘c_{rt}’ going from the top of the deck to the top level of reinforcing (represented by the top dashed line in the figure).
Article CD6.5.1, 1st paragraph, 2nd sentence	Both	…is greater than 1.7.	…is ~~greater than~~ less than or equal to 1.7.
Equation 10.6.2.4.3-2	Both	$S_c = \left[\frac{H_c}{1 + e_o}\right]\left[\left(C_c \log\left(\frac{\sigma'_f}{\sigma'_p}\right)\right)\right]$	$S_c = \left[\frac{H_c}{1+ e_o}\right]\left[\left(C_c \log \frac{\sigma'_f}{\sigma'_p}\right)\right]$
Equation 10.6.2.4.3-3	Both	$S_c = \left[\frac{H_c}{1 + e_o}\right]\left[C_c \log\left(\frac{\sigma'_f}{\sigma'_{pc}}\right)\right]$	$S_c = \left[\frac{H_c}{1+ e_o}\right]\left[C_c \log\left(\frac{\sigma'_f}{\sigma'_{pc}}\right)\right]$
Equation 10.6.2.4.3-4	Both	$S_c = H_c\left[C_{r\varepsilon} \log\left(\frac{\sigma'_p}{\sigma'_o}\right) + C_{c\varepsilon} \log\left(\frac{\sigma'_f}{\sigma'_p}\right)\right]$	$S_c = H_c\left[C_{r\varepsilon} \log\left(\frac{\sigma'_p}{\sigma'_o}\right) + C_{c\varepsilon} \log\left(\frac{\sigma'_f}{\sigma'_p}\right)\right]$

Equation 10.6.3.1.2a-1	US	$q_n = cN_{cm} + g\gamma D_f N_{qm} C_{wq} + 0.5 g\gamma \quad BN_{\gamma m} C_{w\gamma}$	$q_n = cN_{cm} + \gamma D_f N_{qm} C_{wq} + 0.5\gamma \quad BN_{\gamma m} C_{w\gamma}$
Article 10.6.3.1.2a, where list	US	g = gravitational acceleration (ft./sec.^{2})	~~g = gravitational acceleration (ft./sec.^{2})~~
Article 12.8.4.2 – Note: also revise the Notation list	US	$V_{DL} = [H_2(S)\ A_T]\ \gamma_s/2$	$V_{DL} = [H_2(S)\text{-}A_T]\ \gamma_s/2$
Article 12.8.4.2 – Note: also revise the Notation list	SI	$V_{DL} = g\left[H_2(S)A_T\right]\gamma_S/\left(2\times10^9\right)$	$V_{DL} = g\left[H_2(S)-A_T\right]\gamma_S/\left(2\times10^9\right)$
Article 12.10.4.2.4a, where list	SI	A_s = area of reinforcement per length of pipe, b (mm^2/mm)	A_s = area of reinforcement per length of pipe~~, b~~ (mm^2/mm)
Article C12.10.4.2.4a, 1st & 2nd paragraphs – Note: Delete commentary article number	SI	The required area of steel, A_s, as determined by Eq. 1, should be distributed over a unit length of the pipe, b, which is typically taken as 300 mm. The factored actions should also be consistent with the selected unit width.	~~The required area of steel, A_s, as determined by Eq. 1, should be distributed over a unit length of the pipe, b, which is typically taken as 300 mm.~~ ~~The factored actions should also be consistent with the selected unit width.~~
Equation 12.10.4.2.5-3	US	$\rho = \frac{A_s}{\phi bd} \le 0.02$	$\rho = \frac{A_s}{bd} \le 0.02$
Equation 12.10.4.2.5-3	SI	$\rho = \frac{A_s}{\phi d} \le 0.02$	$\rho = \frac{A_s}{d} \le 0.02$
Equation 12.10.4.2.5-9	Both	$M_{nu} = M_u - N_u \left[\frac{4(h-d)}{8}\right]$	$M_{nu} = M_u - N_u \left[\frac{(4h-d)}{8}\right]$
Article 12.11.2.1, 1st paragraph	Both	Distribution of wheel loads…	Loads and load combinations specified in Table 3.4.1-1 shall apply. Live load shall be considered as specified in Article 3.6.1.3. Distribution of wheel loads…

SUBJECT: LRFD Bridge Construction Specifications

Editorial revisions and additions to various articles of the AASHTO LRFD Construction Specifications

2006 EDITORIAL CHANGES - CONSTRUCTION

<table>
<tr><th>Location of Change</th><th>Current Text</th><th>Proposed Text</th></tr>
<tr><td>Table 8.2.2-1, 2nd column</td><td>

Minimum Cement Content
lb/yd^3
800
800
676
676
860
860
736
860
—^{c}
—^{c}

</td><td>

Minimum Cement Content
lb/yd^3
<u>611</u>
<u>611</u>
<u>517</u>
<u>517</u>
<u>658</u>
<u>658</u>
<u>564</u>
<u>658</u>
—^{c}
—^{c}

</td></tr>
</table>

ATTACHMENT A

The following revisions should be made to the Reference citations and listings in Section 4 and Section 6 of the AASHTO LRFD Specifications. These revisions are provided in the following list as a convenience with these Interim Specifications (in lieu of printing numerous replacement pages), and will be updated accordingly when the Fourth Edition of the AASHTO LRFD Specifications is eventually issued.

LRFD Section 4 Reference List Updates

1. Throughout Articles 4.5 and 4.6, change 'Galambos (1988)' to 'Galambos (1998)'.

2. Revise the reference listing for 'Galambos, T.V. 1988…' as follows:

   "Galambos, T. V., ed. ~~1988~~1998. *Guide to Stability Design for Metal Structures*. ~~4th~~ 5th ed."

3. In Article C4.6.3.7, change 'Podolny and Scalzi 1976' to 'Podolny and Scalzi 1986'.

4. Revise the reference listing for 'Podolny, …' as follows:

   "Podolny, W., and J. B. Scalzi. ~~1976~~1986"

LRFD Section 6 Reference List Updates

1. In Articles C6.4.3.3, C6.4.3.5 and C6.4.4, change "*AASHTO LRFD Bridge Construction Specifications (1998)*" to "*AASHTO LRFD Bridge Construction Specifications (2004)*" and revise the first reference listing in the Section 6 reference list to read as follows (and move to the proper location in the list):

   "AASHTO. 2004. *LRFD Bridge Construction Specifications and Interim Specifications*. Second Edition. American Association of State Highway and Transportation Officials, Washington, D.C."

2. In Article C6.7.5.3 (in the line immediately above Figure C6.7.5.3-1), remove the reference "Shkurti and Price 2004", and also remove this reference listing from the Section 6 reference list

3. In Article C6.10.1.3, remove the reference "Wright 2005", and also remove this reference listing from the Section 6 reference list

4. In Article C.6.10.1.10.1, change "Schilling and Frost (1964)" to "Frost and Schilling (1964)". Reverse the order of the names in the Section 6 reference list and place in the correct alphabetical order in the list.

5. At the end of the second paragraph of Article C6.10.2.2, change "White (2004)" to "White et al. (2004)". Add the following listing to the Section 6 reference list:

   "White, D.W., Barker, M. and Azizinamini, A. (2004). "Shear Strength and Moment-Shear Interaction in Transversely-Stiffened Steel I-Girders," Structural Engineering, Mechanics and Materials Report No. 27, School of Civil and Environmental Engineering, Georgia Institute of Technology, Atlanta, GA."

4. In the 6th paragraph of Article C6.10.3.2.1, change "White (2004)" to "White and Grubb (2005)"

5. In the second paragraph of Article C6.10.7.1.2, change 'Yakel and Azizinamini (*2004*)' to 'Yakel and Azizinamini (*2005*)', and replace the corresponding reference listing in the Section 6 reference list with the following:

"Yakel, A., and A. Azizinamini. 2005. "Improved Moment Strength Prediction of Composite Steel Plate Girders in Positive Bending." *Journal of Bridge Engineering*, American Society of Civil Engineers, Reston, VA, January/February."

6. In the first paragraph of Article C6.10.8.1.1, change "White (2004)" to "White and Grubb (2005)"

7. At the end of the 3rd paragraph of Article C6.10.8.2.3, change "White et al. (2001)" to "White (2004)."

8. At the end of the next-to-last paragraph of Article C6.10.9.1, change "and White (2004)" to ", White et al. (2001), White and Barker (2004), White et al. (2004) and Jung and White (2006)." Add the following to the Section 6 reference list:

   "White, D.W. and Barker, M. (2004). "Shear Resistance of Transversely-Stiffened Steel I-Girders," Structural Engineering, Mechanics and Materials Report No. 26, School of Civil and Environmental Engineering, Georgia Institute of Technology, Atlanta, GA, 105 pp."

   "Jung, S.K. and D.W. White (2006). "Shear Strength of Horizontally Curved Steel I-Girders – Finite Element Studies," *Journal of Constructional Steel Research*, 62(4), 329-342."

9. In Article C6.10.9.3.2, change all the occurrences of "White (2004)" to "White et al. (2004)"

10. At the end of the first paragraph of Article C6.11.1, change "Wolchuk (1990)" to "Wolchuck (1997)" and update the corresponding reference listing in the Section 6 reference list to the following:

    "Wolchuck, R. 1997. "Steel-Plate-Deck Bridges and Steel Box Girder Bridges." *Structural Engineering Handbook*, 4th ed., E.H. Gaylord, Jr., C.N. Gaylord, and J.E. Stallmeyer, eds., McGraw-Hill, New York, NY, pp. 19-1 to 19-31."

11. In the second paragraph of Article C6.11.8.2.2, change "Johnston (1966)" to "Galambos (1998)" and remove the citation for "Johnston, B.G. 1966" from the Section 6 reference list

12. In the REFERENCES citation for ASCE (1968), change "AASHTO" to "AASHO" and change "Beam" to "Beams"

13. Correct the REFERENCES citation for Cooper (1967) as follows:

    "Cooper, P.B. 1967. "Strength of Longitudinally Stiffened Plate Girders." *Journal of the Structural Division*, American Society of Civil Engineers, New York, NY, Vol. 93, No. ST2, pp. 419-451."

14. In the REFERENCES citation for Culver, change the date from "1983" to "1972."

15. In the third paragraph of Article CA6.1.1, change "White (2004)" to "White and Grubb (2005)"

16. In the second paragraph of Article CA6.2.1, change '(*White and Barth 1998*)' to '(*White and Barth 1998, Barth et al 2005*)', and add the following reference listing to the Section 6 reference list:

    "Barth, K.E., D.W. White, J.E. Righman, and L. Yang. (2005). "Evaluation of Web Compactness Limits for Singly and Doubly Symmetric Steel I-Girders." *Journal of Constructional Steel Research*, Elsevier, Vol. 61 (10)"

17. In Article CA6.3.3, change all occurrences of "White (2004)" to "White and Jung (2003)". Add the following citation to the REFERENCES section:

    "White, D.W. and Jung, S.-K (2003). "Simplified Lateral-Torsional Buckling Equations for Singly-Symmetric I-Section Members," Structural Engineering, Mechanics and Materials Report No. 24b, School of Civil and Environmental Engineering, Georgia Institute of Technology, Atlanta, GA."

18. Update the reference listing (shown below) in the Section 6 reference list as follows:

"Barth, K.E., B.A. Hartnagel, D.W. White, and M.G. Barker. 2004. "Improved Simplified Inelastic Design of Steel I-Girder Bridges." *Journal of Bridge Engineering*, American Society of Civil Engineers, Reston, VA, Vol. 9, No. 3."

19. In Article CD6.5.3, change "Elgaaly (1991)" to "Elgaaly and Salkar (1991)"

20. Change all occurrences of AISC (*1999*) to AISC (*2005*) within the Commentary to Section 6, *with the exception of the following*:

    All occurrences in Article C6.8.2.2

    All occurrences in Article C6.9.1

    All occurrences in Article C6.10.1.10.2.

    All occurrences in Article C6.12.2.2.1.

    The second occurrence in Article C6.12.2.3.1

21. Add the following reference to AISC (2005) within the Section 6 reference list [do not remove the reference to AISC (1999)]:

    AISC. 2005. *Specification for Structural Steel Buildings*. ANSI/AISC 360-05, American Institute of Steel Construction, Chicago, IL, March 9, 2005.

22. Replace the reference to White, D.W. 2004 in the REFERENCES list by

    "White, D.W. (2004). "Unified Flexural Resistance Equations for Stability Design of Steel I-Section Members – Overview," Structural Engineering, Mechanics and Materials Report No. 24a, School of Civil and Environmental Engineering, Georgia Institute of Technology, Atlanta, GA, 39 pp."

23. In the Section 6 reference listing for "Ollgaard, J.G., R.G. Slutter, and J.W. Fisher", 'American Iron and Steel Institute, Washington, DC' should be 'American Institute of Steel Construction, Chicago, IL'.

24. In the Section 6 reference listing for "Hardash, S., and R. Bjorhovde", 'American Iron and Steel Institute, Washington, DC' should be 'American Institute of Steel Construction, Chicago, IL'.

25. In the Section 6 reference listing for "Polyzois, D., and K.H. Frank", 'American Iron and Steel Institute, Washington, DC' should be 'American Institute of Steel Construction, Chicago, IL'.

26. In the Section 6 reference listing for "Sheikh-Ibrahim, F.I. 2002", add 'American Institute of Steel Construction, Chicago, IL' after the words '*AISC Engineering Journal*'.