LRFD Specifications for Structural Supports for Highway Signs, Luminaires, and Traffic Signals

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PREFACE

The first edition of *LRFD Specifications for Structural Supports for Highway Signs, Luminaires, and Traffic Signals* supersedes the sixth edition of the *Standard Specifications for Structural Supports for Highway Signs, Luminaires, and Traffic Signals*. It includes changes approved by the Highways Subcommittee on Bridges and Structures in 2014.

An abbreviated table of contents follows this preface. Detailed tables of contents precede each Section and each Appendix.

AASHTO Publications Staff

FOREWORD

The first edition of the *LRFD Specifications for Structural Supports for Highway Signs, Luminaires, and Traffic Signals* incorporates recent work performed under the National Cooperative Highway Research Program (NCHRP), specifically NCHRP 10-80, and other research efforts including state-sponsored activities. These Specifications address:

- Division I on design,
- Division II on fabrication, construction, and
- Division III on inspection, and asset management.

Where possible, these specifications incorporate other AASHTO documents, specifically, the AASHTO *LRFD Bridge Design Specifications*, AASHTO *LRFD Bridge Construction Specifications*, AASHTO *Manual for Bridge Evaluation*, and AASHTO *Standard Specifications for Transportation Materials and Methods of Sampling and Testing.*

The design specifications are founded upon the Sixth Edition of *Standard Specifications for Structural Supports for Highway Signs, Luminaires, and Traffic Signals* which incorporate a wealth of research, engineering practice, and long history of satisfactory performance for the vast majority of structures. Based upon NCHRP Report 796 (2014), LRFD calibration, and current research for both loads and resistances are incorporated. Resistances include several specifications associated with improved detailing for fatigue performance. Additionally, new sections on Fabrication, Construction, Inspection, and Asset Management are based upon best practices. These areas are evolving as agencies gain more experience with inspection and management of their ancillary structure inventories.

The design specifications provided in Division I are based on the LRFD methodology and are intended to address the usual structural supports. Requirements more stringent than those in the Specifications may be appropriate for atypical structural supports. The commentary is intended to provide background on some of the considerations contained in the Specifications; however, it does not provide a complete historical background or detailed discussions of the associated research studies. The Specifications and accompanying commentary do not replace sound engineering knowledge and judgment in design, fabrication, construction, inspection, or asset management.

AASHTO Highways Subcommittee on Bridges and Structures

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SECTION 1: INTRODUCTION

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SECTION 1:

INTRODUCTION

1.1—SCOPE

The provisions of these load and resistance factor design specifications for structural supports for highway signs, luminaires, and traffic signals, hereinafter referred to as the Specifications, are applicable to the structural design of supports for highway signs, luminaires, and traffic signals (LRFD Structural Supports). The types of supports addressed in these Specifications are discussed in Article 1.4. The Specifications are intended to serve as a standard and guide for design, fabrication, construction, inspection, and asset management.

C1.1

These Specifications are the result of National Cooperative Highway Research Program (NCHRP) Project 10-80 and the corresponding NCHRP Report 796. These Specifications are intended to replace the sixth edition, *Standard Specifications for Structural Supports for Highway Signs, Luminaires, and Traffic Signals* (2013).

At the discretion of the Owner, proprietary solutions may be considered. These solutions may address both new structures and the repair or rehabilitation of existing structures. Testing of proprietary solutions shall model actual conditions as closely as possible, and the test methods and results shall be published.

Where appropriate, the language and intent of the Specifications is kept the same as in the *AASHTO LRFD Bridge Design Specifications* and the *AASHTO LRFD Bridge Construction Specifications*. The following definitions are used:

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 structural design.

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

The design provisions of these Specifications employ the Load and Resistance Factor Design (LRFD) methodology. The factors have been developed from the theory of reliability based on current statistical knowledge of loads and structural performance, including materials properties.

Seismic design is not included in these Specifications, and such procedures should be prescribed by the Owner.

In most cases, wind combined with other load effects controls the structural design.

The commentary references other documents that provide suggestions for meeting the requirements and intent of these Specifications. However, those documents and the commentary are not intended to be a part of these Specifications.

The commentary discusses some provisions of the Specifications with emphasis given to the explanation of new or revised provisions that may be unfamiliar to the reader. The commentary is not intended to provide a complete historical background concerning the development of this or previous Specifications, nor is it intended to provide a detailed summary of the studies and research data reviewed in developing the provisions. References to some of the research data are provided, however, for those who wish to study the background material in depth. Not all references are cited.

1.2—DEFINITIONS

AA—Aluminum Association.

AASHTO—American Association of State Highway and Transportation Officials.

ACI—American Concrete Institute.

AISC—American Institute for Steel Construction.

Arm—A cantilevered member, either horizontal or sloped, which is typically attached to a pole.

ASCE—American Society for Civil Engineers.

ASD—Allowable stress design.

AWS—American Welding Society.

Bridge Support—Also known as span-type support; a horizontal or sloped member or truss supported by at least two vertical supports.

Cantilever—A member, either horizontal or vertical, supported at one end only.

CMS—Changeable message sign, a sign that displays a variable message.

Collapse—A major change in the geometry of the structure rendering it unfit for use.

Component—Either a discrete element of the structure or a combination of elements requiring individual design consideration.

Design—Proportioning and detailing the components and connections of a structure.

Designer—The person responsible for design of the structural support.

Ductility—Property of a component or connection that allows inelastic response.

DMS—Dynamic Message Sign, see *CMS.*

Engineer—Person responsible for the design of the structure or review of design-related field submittals such as erection plans, or both.

Evaluation—Determination of load-carrying capacity or remaining life of an existing structure.

Extreme Event Limit States—Limit states relating to events such as wind, earthquakes, and vehicle collision, with return periods in excess of the design life of the structure.

Factored Load—Nominal loads multiplied by the appropriate load factors specified for the load combination under consideration.

Factored Resistance—Nominal resistance multiplied by a resistance factor.

FHWA—U.S. Federal Highway Administration.

Force Effect—A deformation, stress, or stress resultant (i.e., axial force, shear force, torsional, or flexural moment) caused by applied loads or imposed deformations.

High-Level Lighting—Also known as high-mast lighting; lighting provided at heights greater than 55 ft, typically using four to twelve luminaires.

High-Level Luminaire Support—Truss-type or pole-type tower that provides lighting at heights greater than about 55 ft, typically using four to twelve luminaires.

High-Mast Lighting Tower (HMLT)—Another description for a pole-type high-level luminaire support.

Load Effect—Same as force effect.

Limit State—A condition beyond which the structure or component ceases to satisfy the provisions for which it was designed.

Load Factor—A statistically-based multiplier applied to force effects accounting primarily for the variability of loads, the lack of accuracy in analysis, and the probability of simultaneous occurrence of different loads. Related to the statistics of the resistance through the calibration process.

Load and Resistance Factor Design (LRFD)—A reliability-based design methodology in which force effects caused by factored loads are not permitted to exceed the factored resistance of the components.

LRFD Structural Supports—highway signs, luminaires, and traffic signals.

Luminaire—A complete lighting unit consisting of a lamp or lamps together with the parts designed to provide the light, to position and protect the lamps, and to connect the lamps to an electric power supply.

Mast Arm—A member used to hold a sign, signal head, or luminaire in an approximately horizontal position.

Mean Recurrence Interval (MRI)—The expected time period for the return of a wind speed that exceeds the basic wind speed. The annual probability of exceeding the basic wind in any one-year period is the reciprocal of this value.

Member—A component that is positioned between two physical joints of a structure.

Model—An idealization of a structure for the purpose of analysis.

Monotube—A support that is composed of a single tube.

Multiple-Load-Path Structure—A structure capable of supporting the specified loads following loss of a main load-carrying component or connection.

NCHRP—National Cooperative Highway Research Program.

NDS—2012. National Design Specification for Wood Construction.

Nominal Resistance—Resistance of a component or connection to force effects, as indicated by the dimensions specified in the contract documents and by permissible stresses, deformations, or specified strength of materials.

Overhead Sign—A sign mounted over a roadway or near, and elevated with respect to, a travel way.

Owner—The person or agency having jurisdiction for the design, construction, and maintenance of the structural support.

Pole—A vertical support that is often tall, relatively slender, and generally rounded or multisided.

Pole Top—A descriptive term indicating that an attachment is mounted at the top of a structural support, usually pertaining to one luminaire or traffic signal mounted at the top of a pole.

Rehabilitation—A process in which the resistance of the structure is either restored or increased.

Resistance Factor—A statistically-based multiplier applied to nominal resistance primarily accounting for variability of material properties, structural dimensions and workmanship, and uncertainty in the prediction of resistance. Related to the statistics of the loads through the calibration process.

Roadside Sign—A sign mounted beside the roadway on a single or multiple supports.

SEI—Structural Engineering Institute (within ASCE).

Service Life—The period of time that the structure is expected to be in operation.

Service Limit States—Limit states relating to stress, deformation, and concrete cracking under regular operating conditions.

Sign—A device conveying a specific message by means of words or symbols, erected for the purpose of regulating, warning, or guiding traffic.

Span Wire—A steel cable or strand extended between two poles, commonly used as a horizontal support for signs and traffic signals.

Strength Limit States—Limit states relating to strength and stability during the design life.

Structural Support—A system of members(s) used to resist load effects associated with self weight, attached signs, luminaires, traffic signals, and any other applicable loads (notably wind)

Structure—*See* Structural Support.

Traffic Signal—An electrically operated control device by which traffic is regulated, warned, or directed to take specific actions.

TRB—Transportation Research Board.

Truss—A structural system composed of framework that is often arranged in triangles.

VMS—Variable Message Sign, see *CMS*.

1.3—APPLICABLE SPECIFICATIONS

The following specification documents may be referenced for additional information on design, materials, fabrication, construction, and asset management:

- AASHTO *Standard Specifications for Highway Bridges*,
- AASHTO *LRFD Bridge Design Specifications,*
- AASHTO *LRFD Bridge Construction Specifications*,
- AASHTO *Standard Specifications for Transportation Materials and Methods of Sampling and Testing*,
- ASCE/SEI 7-10 *Minimum Design Loads for Building and Other Structures*,
- AASHTO *Manual for Bridge Evaluation*,
- *AISC Steel Construction Manual*,
- *ACI Building Code Requirements for Structural Concrete and Commentary*,
- ADM *Aluminum Design Manual*,
- AWS *Structural Welding Code—Steel*,
- AWS *Structural Welding Code—Aluminum*,
- *National Design Specifications (NDS) for Wood Construction*, and
- *Book of ASTM Standards*

C1.3

Other specifications may be appropriate, such as Owner-specific specifications, which may preclude or include these Specifications.

The references listed in the Specifications may not be the most current available. The more current literature might be the same or different (applicable or not applicable) to these Specifications. Caution is advised.

1.4—TYPES OF STRUCTURAL SUPPORTS

Structural supports are categorized as follows:

- Sign support structures,
- Luminaire support structures,
- Traffic signal support structures, and
- Combinations of the above structures.

1.4.1—Sign

Structural supports for signs include both overhead and roadside structures intended to support highway traffic signs.

C1.4.1

Typical overhead and roadside sign supports are shown in Figure C1.4.1-1. Overhead sign structures are generally of the bridge or cantilever type. It is also common to support signs on existing grade separation structures that span the traffic lanes.

1.4.2—Luminaire

Structural supports for luminaires include typical poles with luminaire arms, typical poles with luminaires mounted at pole top, and high-level luminaire supports (both truss and pole type).

C1.4.2

The illumination of roadways requires the use of poles, generally tubular pole shafts that support one to two luminaires and range in height from about 30 ft to 55 ft. High-level luminaire supports normally range in heights from 55 ft to 150 ft or higher and usually support four to twelve luminaires illuminating large areas. Typical luminaire supports and high-level supports are shown in Figure C1.4.2-1.

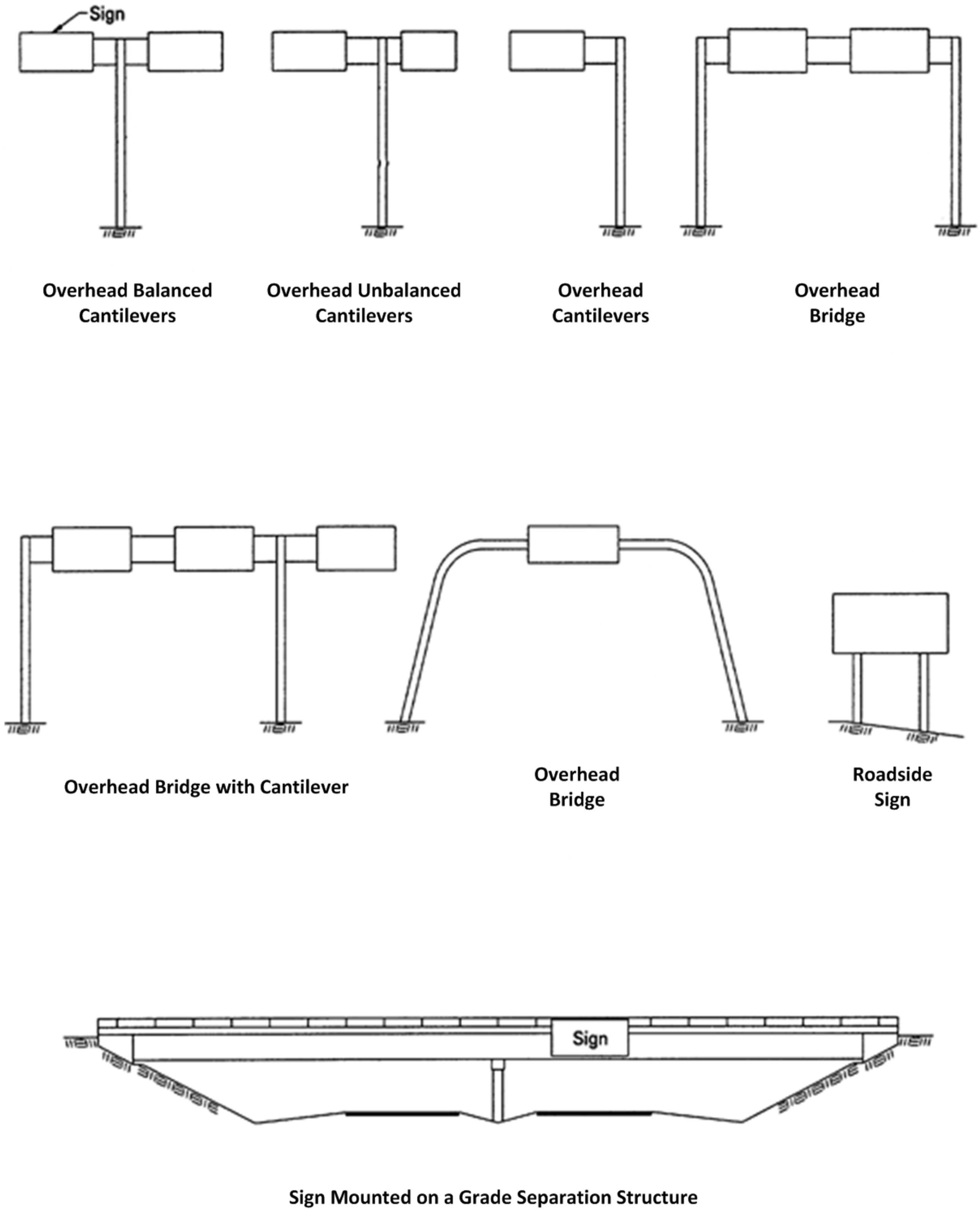


Figure C1.4.1-1—Sign Supports

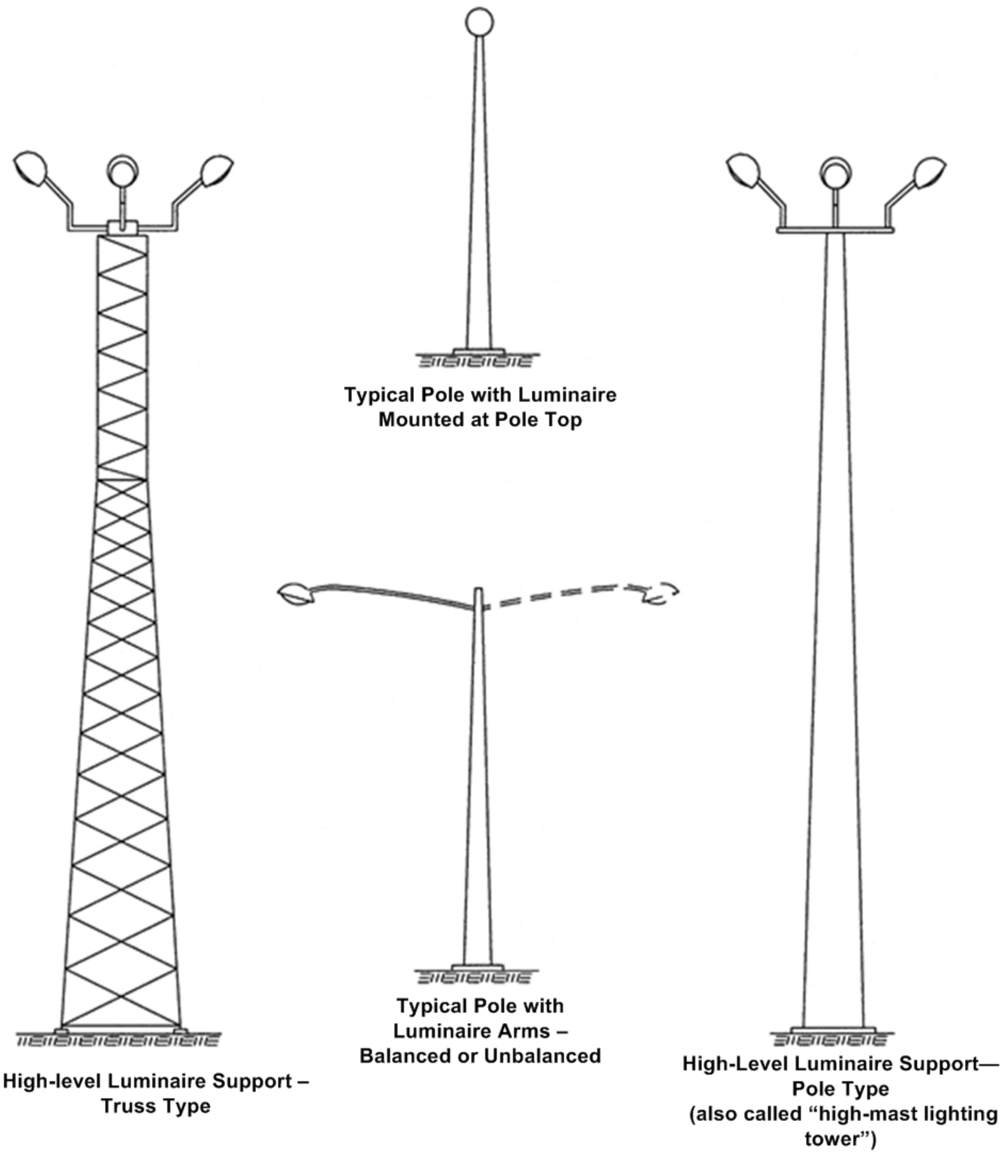


Figure C1.4.2-1—Luminaire Structural Supports

1.4.3—Traffic Signal

Structural supports for mounting traffic signals include pole top, cantilevered arms, bridge, and span wires.

C1.4.3

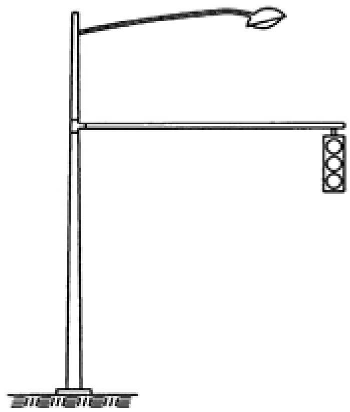
Typical traffic signal supports are shown in Figure C1.4.3-1.

1.4.4—Combination Structures

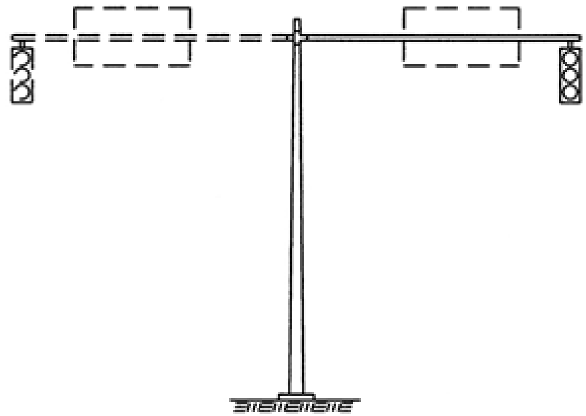
Combination structures include structural supports that combine any of the functions described in Articles 1.4.1, 1.4.2, and 1.4.3.

C1.4.4

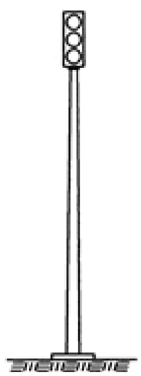
Generally, combination structures are composed of a luminaire support and a traffic signal support. Other structures may combine traffic signal or luminaire supports with those for utility lines.



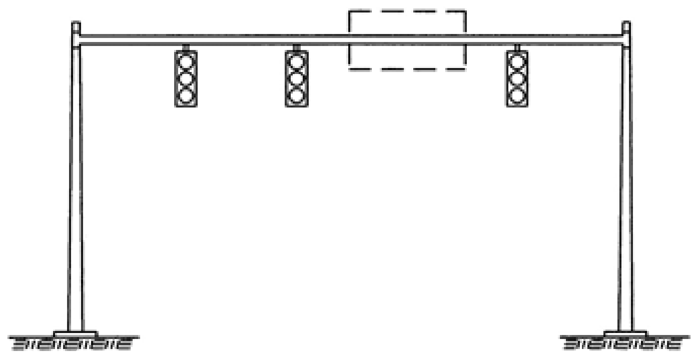
Combination Cantilever Arm Mounted Luminaires and Traffic Signals



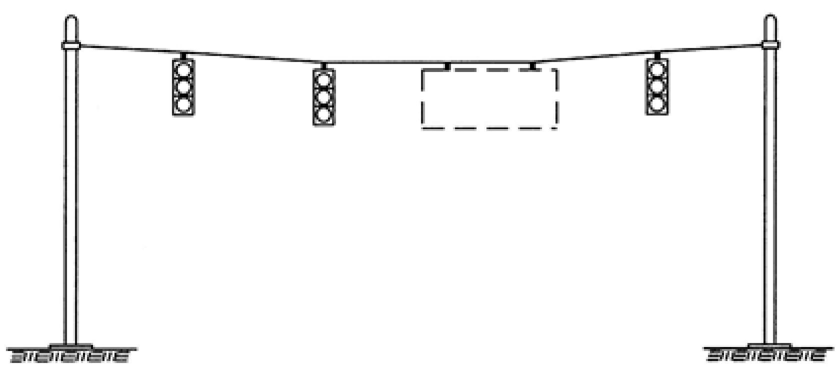
Cantilever Arm Mounted Traffic Signals (Balanced and Unbalanced)



Pole Top-Mounted Traffic Signals



Bridge Mounted Traffic Signals



Span Wire Mounted Traffic Signals

Figure C1.4.3-1—Traffic Signal Structural Supports

1.5—DESIGN PHILOSOPHY

1.5.1—General

Structures shall be designed for specified limit states to achieve the objectives of constructability, safety, and serviceability, with due regard to issues of inspectability, economy, and aesthetics.

Regardless of the type of analysis used, Eq. 1.5.2.1-1 shall be satisfied for all specified force effects and combinations thereof.

C1.5.1

The limit states specified herein are intended to provide for a buildable, serviceable structure capable of safely carrying design loads for a specified time.

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 structural engineering specifications and is permitted because the lower bound theorem insures safety. The lower bound theorem has two fundamental requirements: equilibrium is satisfied in the analysis and ductility is provided. (e.g., see Barker and Puckett, 2012)

1.5.2—Limit States

1.5.2.1—General

Each component and connection shall satisfy Eq. 1.5.2.1-1 for each limit state unless otherwise specified. All limit states shall be considered of equal importance.

$$\sum \gamma_i Q_i \leq \varphi R_n = R_r \qquad (1.5.2.1\text{-}1)$$

where:

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

φ = resistance factor: a statistically based multiplier applied to nominal resistance,

Q_i = force effect,

R_n = nominal resistance, and

R_r = factored resistance: φR_n.

C1.5.2.1

Eq. 1.5.2.1-1 is the basis of LRFD methodology. Assigning resistance factor $\varphi = 1.0$ to all service and fatigue limit states is a default, and may be overridden by provisions in other Sections.

Resistance factors for strength and extreme limit states are defined in the materials sections. The load factors are defined in Section 3 Loads. The resistances are prescribed in separate sections as specified in Sections 5, 6, 7, 8, 9, and 13.

1.5.2.2—Service Limit State

The service limit state shall be taken as restrictions on stress, deformation, and concrete cracking under service conditions.

C1.5.2.2

The service limit states are based upon experience and judgment and are not a formal calibration.

1.5.2.3—Fatigue Limit State

Fatigue limit state shall be used to ensure that the expected fatigue load effects remain below the constant amplitude fatigue limit resistance. Section 11 is focused upon the fatigue limit state for steel and aluminum structures.

C1.5.2.3

Recent research has primarily focused on the load effects and resistance associated with steel structures. These results are scaled to aluminum with a general factor. Only high-cycle fatigue is considered.

1.5.2.4—Strength Limit State

Strength limit state shall be used to ensure that strength and stability, both local and global, are provided to resist the specified statistically significant load combinations that a structure is expected to experience.

C1.5.2.4

The strength limit state considers stability or yielding of each structural element. If the resistance of any element, including splices and connections, is exceeded, it is assumed that the structural resistance has been exceeded.

The structural supports are often statically determinate and often cantilevered. As such they do not have significant inelastic reserve strength or opportunity for load redistribution to other components. The exception is overhead trusses and frames.

1.5.2.5—Extreme Limit State

The extreme event limit state shall be used to ensure the survival of a structure during a major wind event. For these Specifications, the wind with gravity combination is considered an extreme event.

C1.5.2.5

Extreme event limit states are considered to be unique occurrences whose return period may be significantly greater than the design life of the structure. ASCE/SEI 7-10 considers wind to have a load factor of 1.0; this is the same as seismic loads prescribed in that document. The wind speed maps in ASCE/SEI 7-10 provide maximum wind speeds greater than in the past. The wind load factor has changed from 1.6 in 2005 to 1.0 in 2010 for example. The overall wind load effect changes are relatively minor in most locales, however, in some coastal regions, the changes were significant.

1.6—REFERENCES

AASHTO. 2009. *AASHTO Transportation Glossary,* Fourth Edition. American Association of State Highway and Transportation Officials, Washington, DC.

AASHTO. 2013. *Standard Specifications for Structural Supports for Highway Signs*, *Luminaires and Traffic Signals*, Sixth Edition. American Association of State Highway and Transportation Officials. Washington, DC.

AASHTO. 2002. *AASHTO Standard Specifications for Highway Bridges*, 17th Edition. American Association of State Highway and Transportation Officials, Washington, DC.

AASHTO. 2014. *AASHTO LRFD Bridge Design Specifications,* Seventh Edition. American Association of State Highway and Transportation Officials, Washington, DC.

AASHTO. 2010. *AASHTO LRFD Bridge Construction Specifications,* Third Edition. American Association of State Highway and Transportation Officials, Washington, DC.

AASHTO. 2014. *Standard Specifications for Transportation Materials and Methods of Sampling and Testing,* 34th Edition. American Association of State Highway and Transportation Officials, Washington, DC.

American Concrete Institute. 2011. *ACI Building Code Requirements for Structural Concrete and Commentary*. Farmington, MI.

American Institute for Steel Construction. 2010. *Steel Construction Manual*. Chicago, IL.

ASCE/SEI 07–10. 2010. *Minimum Design Loads for Buildings and other Structures*. American Society of Civil Engineers, Reston, VA.

ASTM. 2012. *Book of ASTM Standards*. American Society for Testing and Materials, West Conshohocken, PA.

Aluminum Association. 2010. *Aluminum Design Manual*. Aluminum Association, Arlington, VA.

AWS. 2010. Structural Welding Code—Steel, D1.1/D1.1M, American Welding Society, Miami, FL.

AWS. 2008. Structural Welding Code—Aluminum, D1.2/D1.2M, American Welding Society, Miami, FL.

Puckett, J., Garlich, M., Barker, M., Nowak, A., Menzemer, C., 2014. Development and Calibration of AASHTO-LRFD Specifications for Structural Supports for Highway Signs, Luminaires, and Traffic Signals, *NCHRP Report 796*. Transportation Research Board, National Research Council, Washington, DC.

AWC. 2012. National Design Specification® (NDS®) for Wood Construction, American Wood Council, Leesburg, VA.

SECTION 2: GENERAL FEATURES OF DESIGN

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SECTION 2:

GENERAL FEATURES OF DESIGN

2.1—SCOPE

Minimum requirements are provided or referenced for aesthetics, clearances, constructibility, inspectability, and maintainability of structural supports. Guidelines for determining vertical and horizontal clearances, use of breakaway supports, use of guardrails, illumination of the roadway, sizes of signs, illumination and reflectorization of signs, and maintenance are provided in the following references:

- AASHTO *A Policy on Geometric Design of Highways and Streets*,
- *Manual on Uniform Traffic Control Devices* (MUTCD),
- AASHTO *Manual for Assessing Safety Hardware* (MASH),
- NCHRP 350 *Recommended Procedures for the Safety Performance Evaluation of Highway Features,*
- AASHTO *Roadside Design Guide*,
- AASHTO *Maintenance Manual for Roadways and Bridges*, and
- AASHTO *Roadway Lighting Design Guide*.

C2.1

This Section provides the Designer with information and references to determine the configuration, overall dimensions, and location of structural supports for highway signs, luminaires, and traffic signals. The information in this Section is broad in scope. No attempt has been made to establish rigid criteria in such areas as vertical heights of traffic signal and luminaire supports and levels of illumination. This Section provides references and considerations for the different aspects of design that should be considered in the preliminary stages of a project. In addition to the requirements provided within this Section, many Owners have specific requirements.

2.2—DEFINITIONS

Barrier—Longitudinal traffic barrier, usually rigid, used to shield roadside obstacles or non-traversable terrain features. It may occasionally be used to protect pedestrians from vehicle traffic.

Breakaway—Design feature that allows a sign, luminaire, or pole top-mounted traffic signal support to yield, fracture, or separate near ground level on vehicle impact.

Clearance—Horizontal or vertical dimension to an obstruction.

Clear Zone—An unobstructed, relatively flat area beyond the edge of the traveled way for the recovery of errant vehicles. The traveled way does not include shoulder or auxiliary lanes.

CMS—Abbreviation for Changeable Message Sign.

Curb—A vertical or sloping surface, generally along and defining the edge of a roadway or roadway shoulder.

DMS—Dynamic Message Sign, see *CMS.*

FHWA—U.S. Federal Highway Administration.

Gore—Center area immediately past the point where two roadways divide at an acute angle, usually where a ramp leaves a roadway.

Guardrail—Type of longitudinal barrier that may deflect upon impact.

Mounting Height—Minimum vertical distance to the bottom of a sign or traffic signal relative to the pavement surface.

Pedestal Pole—Relatively short pole supporting a traffic signal head attached directly to the pole.

Roadside—Area between the shoulder edge and the right-of-way limits, or the area between roadways of a divided highway.

Roadway—Highway or street.

Support Facility—Transportation systems that support the roadway, e.g., parking lots, rest areas, etc.

Traveled Way—Roadway width not including shoulder or auxiliary lanes.

User—Person using the roadway including motorists, bicyclists, or pedestrians.

VMS—Variable Message Sign, see *CMS*.

2.3—AESTHETICS

The structural support should complement its surroundings, be graceful yet functional in form, and present an appearance of adequate strength. The support should have a pleasing appearance that is consistent with the aesthetic effect of the highway's other physical features. Supports should have clean, simple lines, which will present minimum hazard to motorists, cyclists, or pedestrians.

Structural supports should be designed and located so as not to distract the user's attention or obstruct the view of the highway, the view of other signs, or important roadway features. The effect that signing or lighting installations have on the surrounding environment should be considered.

C2.3

The appearance of ordinary structural supports should consider aesthetics and function. Combination poles, which serve multiple functions for lighting, traffic control, and electrical power, should be considered to reduce the number of individual poles along the highway.

"The use of [a] bridge as a support for message or directional signing or lighting should be avoided wherever possible" (*AASHTO LRFD Bridge Design Specifications* (AASHTO 2014)). Tradeoffs may exist between bridge aesthetics and sign economy and functionality.

2.4—FUNCTIONAL REQUIREMENTS

2.4.1—Lighting Systems

The Designer should select the light source, luminaire distribution, mounting height, and luminaire overhang based on factors including the geometry and character of the roadway, the environment, proposed maintenance, economics, aesthetics, and overall lighting objectives.

C2.4.1

The AASHTO *Roadway Lighting Design Guide* (AASHTO 2005) provides information on the warranting conditions for use of lighting, level and uniformity of luminance, quality of light, location of poles, use of breakaway devices, high-mast poles, and maintenance. Additional information may be found in the *FHWA Lighting Handbook* (2012). Decisions on lighting may also be guided by crash statistics and use of the *Highway Safety Manual* (HSM) (AASHTO 2010).

Some communities limit the amount of surrounding illumination, and shielding may be required. The same average illumination can usually be obtained by more than one installation arrangement.

The function for various roadway users, including pedestrians and cyclists, may have differing requirements that should be considered.

2.4.1.1—Vertical Heights for Luminaire Supports

The height of the luminaire support should be determined by the Designer to meet a particular need within the situational constraints.

C2.4.1.1

Design attributes that should be considered in determining the height of a luminaire support include:

- Glare characteristics,
- Desired level of illumination and distribution of light,
- Photometric characteristics of a selected lamp and luminaire,
- Available space for placing the supports,
- Inspection capability,
- Maintenance capability (maximum attainable servicing height),
- Compliance with local ordinances and statutes, and
- Consideration of local customs and aesthetics.

Height restrictions may be imposed by various government agencies, such as the Federal Highway Administration (FHWA) with respect to breakaway devices and the Federal Aviation Administration for airspace considerations.

2.4.1.2—Illumination of the Roadway

The Designer should consider the quality of light and the level of illumination.

C2.4.1.2

Highway illumination is provided to improve driver nighttime visibility and to promote safer and more efficient use of special roadway facilities located at ramps, intersections, and potentially hazardous areas. Other roadway users, including pedestrians and cyclists, may have different safety requirements that should be considered.

The amount of illumination that should be provided over a roadway depends on visibility, visual comfort, light distribution, and geometry. Disability and discomfort glare, pavement glare, road location, and obstructions to visibility and traffic patterns are other factors that influence the level of illumination.

A luminaire installation should provide a visual environment that is conducive to safe and comfortable night driving.

2.4.2—Structural Supports for Signs and Traffic Signals

2.4.2.1—Vertical Clearances

Vertical clearance shall be provided of not less than 17 ft to the sign, light fixture, walkway, or sign bridge over the entire width of the pavement and shoulders unless the grade separation structures or other structures nearby have lesser vertical clearance. In cases of lesser clearance, the overhead sign support may be as low as 1 ft higher than the vertical clearance of other supports.

Additional guidance on vertical clearances may be found in the *Manual on Uniform Traffic Control Devices* (MUTCD). (FHWA 2009)

C2.4.2.1

The minimum clearance should include an allowance for possible future overlays.

The additional 1-ft vertical clearance is required so that high vehicles will strike the stronger overpass structures first, thereby lessening the chance of major collision damage to the structurally weaker overhead sign support or traffic signal support structures. A depiction of this clearance limit is illustrated in Figure C2.4.2.1-1.

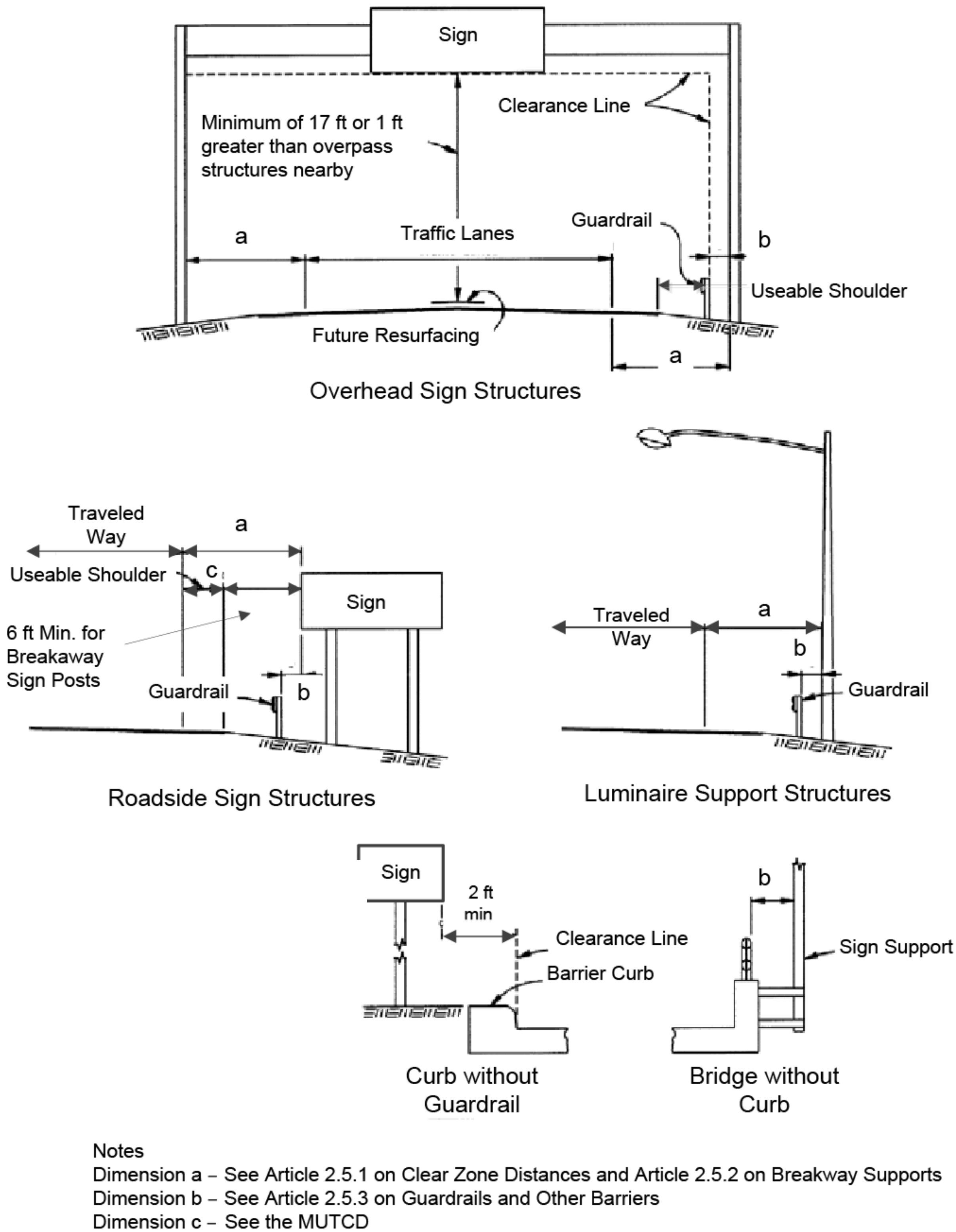

Figure C2.4.2.1-1—Location of Structural Supports

2.4.2.2—Size, Height, and Location of Signs

The MUTCD should be consulted for the sizes, heights, and placement of signs for any installation.

C2.4.2.2

The MUTCD includes information on signs for sizes, illumination and reflectorization, location, height, and lateral clearance.

2.4.2.3—Illumination and Retroreflectivity of Signs

Illumination and retroreflectivity of signs should conform with the provisions of the MUTCD.

Except where retroreflectivity is deemed adequate, all overhead sign installations should normally be illuminated. The lighting equipment should produce uniform illumination for the sign surface and the position of the lighting fixtures should not impair normal viewing of the sign or obstruct view of the roadway. Where internal illumination is used in conjunction with translucent materials, the colors of the sign should appear essentially the same by night and day.

Retroreflectivity levels are required to be maintained above minimum levels by use of a management or assessment method.

C2.4.2.3

The *Roadway Lighting Design Guide* provides additional information.

By an engineering study, headed or prismatic retroreflectivity sheeting could be used to eliminate the need for sign illumination.

2.4.2.4—Changeable Message Signs

The design of changeable message signs (CMS), enclosures, and connections to the support structure normally require additional considerations that are beyond the scope of these Specifications. The MUTCD should be consulted on size, height, and placement.

C2.4.2.4

CMS are composed of lamps or luminous elements that may be visible during the day as well as at night. The lamps and electronics are contained within an enclosure, which typically weighs significantly more than most sign panels.

The MUTCD includes information on the use and design of changeable message signs.

2.5—ROADSIDE REQUIREMENTS FOR STRUCTURAL SUPPORTS

Consideration shall be given to safe passage of vehicles adjacent to or under a structural support. The hazard to errant vehicles within the clear zone distance, defined in Article 2.5.1, should be minimized by locating obstacles a safe distance away from the traveled way. Roadside requirements and location of structural supports for highway signs, luminaires, and traffic signals should generally adhere to the principles given in Articles 2.5.1 through 2.5.9.

C2.5

Where possible, a single support should be used for dual purposes (e.g., signals and lighting). Consideration should also be given to locating luminaire supports to minimize the necessity of encroaching on the traveled way during routine maintenance.

2.5.1—Clear Zone Distance

Structural supports should be located in conformance with the clear zone concept as contained in Chapter 3, "Roadside Topography and Drainage Features," of the *Roadside Design Guide* (AASHTO 2011), or other clear zone policy accepted by FHWA. Where the practical limits of structure costs, type of structures, volume and design speed of through-traffic, and structure arrangement make conformance with the *Roadside Design Guide* impractical, the structural support should be provided with a breakaway device or protected by the use of a guardrail or other barrier.

C2.5.1

The clear zone, illustrated in Figure C2.4.2.1-1, is the roadside border area beyond the traveled way, available for safe use by errant vehicles. This area may consist of a shoulder, a recoverable slope, a non-recoverable slope, and/or a clear run-out area. The desired width is dependent on the traffic volumes and speeds and on the roadside geometry.

Suggested minimum clear zone distances are provided in the *Roadside Design Guide* and are dependent on average daily traffic, slope of roadside, and design vehicle speed. Additional discussions of clear zone distances and lateral placement of structural support is provided in the MUTCD and *A Policy on Geometric Design of Highways and Streets*

(AASHTO 2011). Decisions on appropriate clear zones may also be guided by the HSM.

2.5.2—Breakaway Supports

Breakaway supports should be used for luminaire and roadside sign supports when they cannot be placed outside the roadside clear zone or behind a guardrail. The requirements of Section 12, "Breakaway Supports," shall be satisfied. The requirements of Articles 2.5.2.1 and 2.5.2.2 should be met for the proper performance of the breakaway support.

Breakaway supports housing electrical components shall have the use of electrical disconnects considered for all new installations and for existing installations that experience frequent knockdown.

C2.5.2

Generally, breakaway supports should be provided whenever the support is exposed to traffic, even if beyond the clear zone on a traversable slope. The recommended clear zone distances included in the *Roadside Design Guide* accommodate only about 80 percent of errant vehicles. The use of breakaway supports beyond the clear zone will provide an added measure of safety for the remaining 20 percent.

2.5.2.1—Foundations

The top of foundations and projections of any rigidly attached anchor bolts or anchor supports should not extend above the ground level enough to increase the hazard or to interfere with the operation of a breakaway support.

C2.5.2.1

Foundations for breakaway supports located on slopes are likely to require special details to avoid creating a notch in the slope that could impede movement of the support when broken away or a projection of the foundation that could snag the undercarriage of an impacting vehicle. Foundations should be designed considering the breakaway stub height limitations of Section 12.

2.5.2.2—Impact Height

Breakaway supports should be located such that the location of impact of an errant vehicle's bumper is consistent with the maximum bumper height used in breakaway qualification tests.

C2.5.2.2

The *Manual for Assessing Hardware Safety* (MASH) (AASHTO 2009) provides guidance.

2.5.3—Guardrails and Other Barriers

The location of roadside sign and luminaire supports behind a guardrail should provide clearance between the back of the rail and the face of the support to ensure that the rail will deflect properly when struck by a vehicle. Continuity of the railing on rigid highway structures should not be interrupted by sign or luminaire supports.

The clearance between the edge of a sign panel, which could present a hazard if struck, and the back of a barrier should also take into consideration the deflection of the barrier. The edge of a sign shall not extend inside the face of the railing.

C2.5.3

Guardrails, as illustrated in Figure C2.4.2.1-1, are provided to shield motorists from fixed objects and to protect fixed objects, such as overhead sign supports. The *Roadside Design Guide* provides guidance.

The clearance between the back of the barrier and the face of the support may vary, depending on type of barrier system used. The *Roadside Design Guide* may be used to determine the proper clearance.

2.5.4—Roadside Sign and Luminaire Supports

Roadside sign and typical luminaire supports, within the clear zone distance specified in Article 2.5.1, should be designed with a breakaway feature acceptable under MASH, NCHRP 350, or protected with a guardrail or other barrier. Where viewing conditions are favorable, roadside sign and typical luminaire supports may be placed outside the clear zone distance.

C 2.5.4

Where there is a probability of being struck by errant vehicles, even supports outside the suggested clear zone should preferably be breakaway.

For many years, NCHRP 350 was the standard for the assessment and performance of highway safety features. *The AASHTO/FHWA Joint Implementation Plan* (2009) outlines details regarding the use of NCHRP 350 and MASH for design and existing systems.

2.5.5—Overhead Sign Supports and High-Level Lighting Supports

Overhead sign and high-level lighting structural supports should be placed outside the clear zone distance or protected with a proper guardrail or other barrier.

C2.5.5

Overhead sign and high-level lighting supports are considered fixed-base support systems that do not yield or break away on impact. The large mass of these support systems and the potential safety consequences of falling to the ground necessitate a fixed-base design. Fixed-base systems are rigid obstacles and should not be used in the clear zone area unless shielded by a barrier. In some cases, it may be cost effective to place overhead sign supports outside the clear zone with no barrier protection when the added cost of the greater span structure is compared with the long-term costs of guardrail and vegetation maintenance. Structures can sometimes be located in combination with traffic barriers protecting other hazards, such as culverts, bridge ends, and embankments.

2.5.6—Traffic Signal Supports

Traffic signal supports that are installed on high-speed facilities should be placed as far away from the roadway as practical. Shielding these supports should be considered if they are within the clear zone for that particular roadway.

C2.5.6

Traffic signal structural supports with mast arms or span wires normally are not provided with a breakaway device. However, pedestal pole traffic signal supports are appropriately designed to be breakaway devices. Pedestal poles should, if possible, be placed on breakaway supports because they are usually in close proximity to traffic lanes.

2.5.7—Gores

Where obstruction in the gore is unavoidable within the clear zone, protection should be provided by an adequate crash cushion or the structure should be provided with a breakaway device.

2.5.8—Urban Areas

For sign, luminaire, and traffic signal structures located in working urban areas, the minimum lateral clearance from a barrier curb to the support is 24 in. Where no curb exists, the horizontal clearance to the support should be as much as reasonably possible, but at least 24 in.

C2.5.8

The 24-in. offset is not an urban clear zone; rather it was established to avoid interference with truck mirrors, open doors, and so forth.

2.5.9—Joint-Use Supports

Where possible, consideration should be given to the joint usage of supports in urban areas.

C2.5.9

Preference should be given to joint usage to reduce the number of supports in urban areas. For example, a traffic sign and signal support can be combined with a lighting pole.

Care should be taken at the design stage to ensure that the critical load carrying members of the support are of sufficient capacity for all the likely uses made of the support. This could be achieved by indicating in design documents (the Owner’s records) limitations on use such as maximum EPA and EPA attachment eccentricity.

2.6—INTEGRATION OF STRUCTURAL SUPPORTS WITH ROADWAY AND BRIDGE DESIGN

2.6.1—Signs

Sign panels may be supported on existing or proposed grade separation structures. Although the minimum vertical clearance requirements for overhead signs do not apply in these cases, a minimal vertical clearance to avoid posting the bridge should be maintained. Frames and other attachments to an existing structure shall be designed to support the sign panel. The overhead sign should be located as near to the most advantageous position for traffic operation as possible, but where structurally adequate support details can be provided.

C2.6.1

Sign installation on grade separated structures is generally acceptable aesthetically when the sign panels do not extend below the girders or above the railing. The sign panel should be placed slightly above the minimum vertical clearance specified for the grade separation structure. Close liaison between structural, bridge and traffic engineers is essential for signs mounted on grade separation structures.

The placement of overhead signs must be considered in the preliminary design stages to avoid possibly restricting the driver's view of sign messages by other signs or structures. Signing is an integral part of the roadway environment and must be developed along with the roadway and bridge designs.

2.6.2—Luminaires

The location of luminaire supports should be coordinated with the function and location of other structures.

C2.6.2

The location of the luminaire supports should be coordinated with the location of the sign structures so that the driver's view of sign legends is not hampered. Attention should be given to correlating interchange and structure lighting with the lighting provided on the other sections of the roadway. Where practical, high-level lighting may be used to reduce the number of supports required, present fewer roadside obstacles, and improve safety for maintenance personnel.

2.7—FABRICATION, MATERIALS, AND DETAILING

Guidance for fabrication of support structures is provided in Section 14. Herein working drawings, connection details, specific geometric requirements, etc. are addressed.

C2.7

This section is new to the LRFD Structural Supports Specifications. Previously, guidance was included in a variety of locations within the material sections.

2.8—CONSTRUCTION

Guidance for construction of support structures is provided in Section 15. Erection procedures, anchorage installations, protective systems, etc. are addressed there.

C2.8

This section is new to the LRFD Structural Supports Specifications. Previously, guidance was included in a variety of locations within the material sections.

2.9—INSPECTION AND REPORTING

Guidance for construction of support structures is provided in Section 16. Inspection types, frequencies, planning, scheduling, and access issues are addressed and element-level definitions are provided there.

C2.9

This section is new to the LRFD Structural Supports Specifications. Previously, guidance was included in a variety of locations within the material sections.

2.10—MANAGEMENT

Guidance for asset management of support structures is provided in Section 17. Archives, replacement considerations, maintenance programs, etc. are addressed there.

C2.10

This section is new to the LRFD Structural Supports Specifications. Previously, guidance was included in a variety of locations within the material sections.

2.11—REFERENCES

AASHTO. 2007. *AASHTO Maintenance Manual for Roadways and Bridges*, Fourth Edition. American Association of State Highway and Transportation Officials, Washington, DC.

AASHTO. 2011. *A Policy on Geometric Design of Highways and Streets*, Sixth Edition. American Association of State Highway and Transportation Officials, Washington, DC.

AASHTO. 2010. *Highway Safety Manual*, First Edition. American Association of State Highway and Transportation Officials, Washington, DC.

AASHTO. 2005. *Roadway Lighting Design Guide*, Sixth Edition. American Association of State Highway and Transportation Officials, Washington, DC.

AASHTO. 2011. *Roadside Design Guide*, Fourth Edition. American Association of State Highway and Transportation Officials, Washington, DC.

AASHTO. 2014. *AASHTO LRFD Bridge Design Specifications,* Seventh Edition. American Association of State Highway and Transportation Officials, Washington, DC.

AASHTO/FHWA. 2009. *Joint Implementation Plan for the AASHTO Manual for Assessing Hardware*, American Association of State Highway and Transportation Officials, Washington, DC.

AASHTO. 2009. *Manual for Assessing Safety Hardware* (MASH). First Edition, American Association of State Highway and Transportation Officials, Washington, DC.

FHWA. 2009. *Manual on Uniform Traffic Control Devices*. Federal Highway Administration, U.S. Department of Transportation, Washington, DC.

FHWA. 2012. *FHWA Lighting Handbook*, FHWA-SA-11-22. Federal Highway Administration, U.S. Department of Transportation, Washington, DC.

Ross, H. E., D. L. Sicking, R. A. Zimmer, and J. D. Michie. 1993. *Recommended Procedures for the Safety Performance Evaluation of Highway Features*, NCHRP Report 350. Transportation Research Board, National Research Council, Washington, DC.

SECTION 3: LOADS

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SECTION 3:

LOADS

3.1—SCOPE

This Section specifies minimum requirements for loads and forces, the limits of their application, and load combinations that are used for the design or structural evaluation of supports for highway signs, luminaires, and traffic signals.

The operational risk category shall be used to establish the mean recurrence interval (MRI) for wind loads.

Fatigue-sensitive supports are addressed in Section 11.

C3.1

This Section includes consideration of dead, live, ice, and wind loads.

The Specification defines wind loads in terms of three-second (3-s) gust wind. Use of the 3-s gust wind speed map may result in increases or decreases to loads relative to the fastest-mile used in Specifications editions prior to 2001.

The risk associated with a failure may include factors such as average daily traffic and the likelihood of a structure falling onto a roadway or other important facility. The risk assessment may include the consequences of failure on a lifeline system, e.g., exit routes from hurricane-prone regions.

The MRI 300-yr, 700-yr, and 1700-yr interval wind speed maps are provided in Article 3.8.

Note that in previous editions of the Specifications, typically a 50-yr wind map was used with "safety factors" on resistance. In previous versions of ASCE/SEI 7 (2005 and before), a 50-yr wind map was available and adjustments were made for hurricane and other regions with important site-specific statistics. Additionally, the load factors associated with wind were typically 1.6 for the strength limit state. Beginning in 2010, ASCE/SEI 7-10 recalibrated the wind speed maps to increase hurricane and other regions and lower the load factor to be consistent with other extreme events such as earthquakes, i.e., to 1.0. Henceforth, the Specifications use the extreme limit state for non fatigue-related wind loads and this is consistent with ASCE/SEI 7-10.

3.2—DEFINITIONS

Allowable Stress Design (ASD)—A design approach where load effects are based upon load expected during the service life and nominal material strengths are decreased to provide a level of safety.

Basic Wind Speed, V—The 3-s gust wind speed at 33 ft above the ground associated with exposure C.

Design Wind Pressure, P_z—The pressure exerted on a member or attachment by wind. The pressure is calculated using appropriate design values for all variables in the wind pressure equation.

Directionality Factor, K_d—The maximum wind can come from any direction and the probability that the maximum drag coefficient is associated with the wind direction is reduced.

Drag Coefficient, C_d—A dimensionless coefficient that adjusts the effective velocity pressure for the effects of the geometry of the element, surface roughness, and the Reynolds number.

Effective Velocity Pressure, vp_z—The pressure exerted by the effects of the wind assuming that the drag coefficient, C_d, is equal to 1.0.

Effective Projected Area (EPA)—The equivalent C_d times the area projected area typically provided by a manufacturer. This may be used as an "area" with a unit value for C_d.

Fastest-Mile Wind Speed—The peak wind speed averaged over one mile of wind passing a point.

Gust Effect Factor, G—A dimensionless coefficient that adjusts the wind pressure to account for the dynamic interaction of the wind and the structure.

Height and Exposure Factor, K_z—A dimensionless coefficient that adjusts the magnitude of wind pressure referenced to a height above the ground of 33 ft for the variation of wind speed with height.

Lifeline Travelways—Travelways that are required to be open to all traffic after a major wind event and useable by emergency vehicles and for security, defense, economic, or secondary life safety. May be designated as critical for a local emergency plan (adapted from AASHTO, 2011).

Mean Recurrence Interval, MRI—The inverse of the probability of occurrence of a specific event in a 1-yr period.

Service Life—Period of time that the structure is expected to be in operation.

Solidity—The vertically projected area divided by the total enclosed elevation area for a truss or lattice structure.

Special Wind Region—A region where the magnitude of the local wind speeds is dramatically affected by local conditions. Wind speeds in these areas should be determined by consulting the authority having local jurisdiction or through the analysis of local meteorological conditions.

Three-Second Gust Wind Speed—The average wind speed measured over an interval of 3-s.

3.3—NOTATION

b = overall width (ft) (3.9.4.2)
BL_n = basic load normal to the plane of the structure (3.9.3)
BL_t = basic load transverse to the plane of the structure (3.9.3)
C_d = drag coefficient (3.2) (3.8.1) (3.8.7) (C3.8.7) (3.9.1)
C_{dD} = drag coefficient for round cylinder of diameter D (3.8.7)
C_{dd} = drag coefficient for round cylinder of diameter d_o (3.8.7)
C_{dm} = drag coefficient for multisided section (3.8.7) (C3.8.7)
C_{dr} = drag coefficient for round section (3.8.7) (C3.8.7)
C_v = velocity conversion factor for the selected mean recurrence interval (3.8.7)
d = depth (diameter) of member (ft) (3.8.7)
D = major diameter of ellipse (ft) (3.8.7)
DL = dead load (lb) (3.9)
d_o = minor diameter of ellipse (ft) (3.8.7)
G = gust effect factor (3.2) (3.8.1) (3.8.2) (3.8.6) (C3.8.6)
Ice = ice load (lb) (3.7) (3.9)
K_d = directionality factor (3.2) (3.8.1) (3.8.2) (3.8.5)
K_z = height and exposure factor (3.2) (3.8.1) (3.8.2) (3.8.4)
L_{sign} = longer dimension of the attached sign (ft) (3.8.7)
n_c = normal component of wind force (lb) (3.9.3)
P_z = design wind pressure (psf) (3.2) (3.8.1)
r_c = ratio of corner radius to radius of inscribed circle (3.8.7) (C3.8.7)
r_m = ratio of corner radius to radius of inscribed circle where multisided section is considered multisided (3.8.7)
r_r = ratio of corner radius to radius of inscribed circle where multisided section is considered round (3.8.7)
r_s = ratio of corner radius to depth of square member (3.8.7)
t_c = transverse component of wind force (lb) (3.9.3) (C3.9.3)

V = basic wind speed, expressed as a 3-s gust wind speed, at 33 ft above the ground in open terrain (mph) (3.2) (3.8.1) (3.8.2) (3.8.7) (C3.8.7)

W_h = wind load on exposed horizontal support (lb) (3.9.2) (3.9.3) (3.9.4.2)

W_l = wind load on luminaires (lb) (3.9.2) (3.9.3) (3.9.4.2)

W_p = wind load on sign panel or traffic signal (lb) (3.9.2) (3.9.3) (3.9.4.2)

W_{sign} = shorter dimension of the attached sign (ft) (3.8.7)

W_v = wind load on exposed vertical supports (lb) (3.9.3)

z = height at which wind pressure is calculated (ft) (3.8.4)

z_g = constant for calculating the exposure factor and is a function of terrain (3.8.4)

α = constant for calculating the exposure factor and is a function of terrain (3.8.4)

3.4—LOAD FACTORS AND LOAD COMBINATIONS

The loads described in Articles 3.5 through 3.8 shall be combined into appropriate load combinations as required in Table 3.4-1. Each part of the structure shall be proportioned for the combination producing the maximum load effect.

The fatigue loads shall be computed in accordance with Section 11.

C3.4

This publication supersedes the AASHTO *Standard Specifications for Structural Supports for Highway Signs, Luminaires, and Traffic Signals* (2009). Beginning with this edition, the design philosophy is based on LRFD. The load and resistance factors are calibrated to provide a reliability index of approximately 3.0 for 300-yr MRI, 3.0 to 3.5 for 700-yr MRI, and 3.5-4.0 for 1700-yr MRI for main members.

These specifications use fatigue limit states I and II for infinite and finite life approaches, respectively. AASHTO LTS Design (Section 11) uses only the infinite life approach for design (fatigue limit state I). The evaluation of the fatigue limit may use a finite life approach to predict the remaining fatigue life for asset management purposes.

NCHRP Report 796 outlines the calibration. (Puckett, et al., 2014)

Table 3.4-1—Load Combinations and Load Factors

Load Combination Limit State	Description	Reference Articles	Permanent: Dead Components (DC)		Transient: Live Load (LL)	Transient: Wind (W)	Fatigue: Truck Gust (TrG)	Fatigue: Natural Wind Gust Vibration (NWG)	Fatigue: Vortex-Induced Vibration (VVW)	Fatigue: Combined Wind on High-level Towers	Fatigue: Galloping-Induced Vibration (GVW)
			Max/Min	Mean			Apply separately				
Strength I	Gravity	3.5, 3.6, and 3.7	1.25		1.6						
Extreme I	Wind	3.5, 3.8, 3.9	1.1/0.9			1.0[a]					
Service I	Translation	10.4		1.0		1.0[b]					
Service III	Crack control for Prestressed Concrete			1.0		1.00					
Fatigue I	Infinite-life	11.7		1.0			1.0	1.0	1.0	1.0	1.0
Fatigue II	Evaluation	17.5		1.0			1.0	1.0	1.0	1.0	1.0

a. Use Figures 3.8-1, 3.8-2, or 3.8-3 (for appropriate return period)

b. Use Figure 3.8-4 (service)

3.5—PERMANENT LOADS

The permanent load shall consist of the weight of the structural support, signs, luminaires, traffic signals, lowering devices, and any other appurtenances permanently attached to and supported by the structure. Temporary loads during inspection and maintenance shall also be considered as part of the permanent loads.

C3.5

In the these specifications, the terms permanent load or dead load may be used interchangably. Dead load is to include all permanently attached fixtures, including hoisting devices and walkways provided for servicing of luminaires or signs.

The points of application of the weights of the individual items may be their respective centers of gravity.

Manufacturers' data may be used for the weights of components.

3.6—LIVE LOADS

A live load consisting of a single load of 0.5 kips distributed over 2.0 ft transversely to the member shall be used for designing members for walkways and service platforms.

C3.6

The specified live load represents the weight of a person and equipment during servicing of the structure. Only the members of walkways and service platforms are designed for the live load. Any structural member designed for the combined loadings in Article 3.4 will be adequately proportioned for live load application. For OSHA-compliant agencies, additional requirements may apply.

Typically, live load will not control the design of the structural support.

3.7—ICE LOAD–ATMOSPHERIC ICING

Atmospheric ice load due to freezing rain or in-cloud icing may be applied around the surfaces of the structural supports, traffic signals, horizontal supports, and luminaires; but it may be considered only on one face of sign panels.

The Owner shall specify any special icing requirements that occur, including those in and near mountainous terrain, gorges, the Great Lakes, and Alaska.

C3.7

NCHRP Report 796 illustrates that ice and wind on ice does not practically control the critical load effect. To simplify these Specifications, these load combinations have been eliminated. (Puckett et al, 2014)

For extreme cases where the Owner indicates, either local conditions or the ice and coincident wind loads provided ASCE/SEI 7 may be used for guidance. (e.g. ASCE/SEI 7, 2010).

3.8—WIND LOAD

Wind load shall be based on the pressure of the wind acting horizontally on the supports, signs, luminaires, traffic signals, and other attachments computed in accordance with Articles 3.8.1 through 3.8.7, Eq. 3.8.1-1 using the appropriate mean recurrence interval basic wind speed as shown in Figures 3.8-1, 3.8-2, 3.8-3, and 3.8-4. The mean recurrence interval is determined with Table 3.8-1.

C3.8

The selection of the MRI accounts for the consequences of failure. A "typical" support could cross the travelway during a failure thereby creating a hazard for travelers (MRI = 700 yrs). The Owner should specify the ADT and Risk Category (or MRI).

All supports that could cross lifeline travelways are assigned a high risk category to consider the consquences of failure (MRI = 1700 yrs).

Supports that cannot cross the travelway are assigned a low risk and 300-yr MRI.

Table 3.8-1—Mean Recurrence Interval

	Risk Category		
Traffic Volume	Typical	High	Low
ADT≤100	300	1700	300
100<ADT≤1000	700	1700	300
1000<ADT≤10000	700	1700	300
ADT>10000	1700	1700	300
Typical: Failure could cross travelway			
High: Support failure could stop a lifeline travelway			
Low: Support failure could not cross travelway			
Roadside sign supports: use 10-yr MRI, see Figure 3.8-4.			

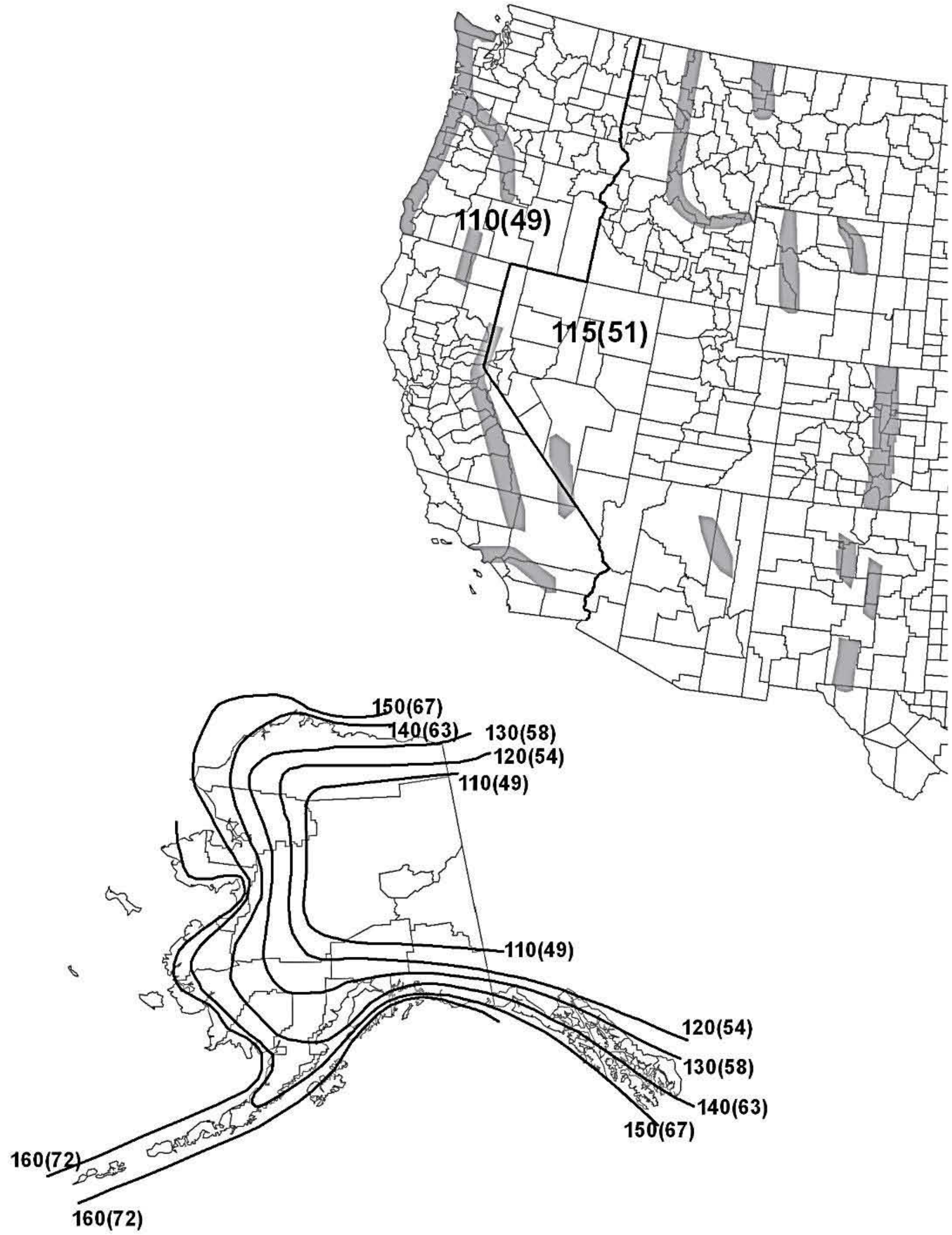


Figure 3.8-1a—700-Year MRI Basic Wind Speed, mph (m/s)–Western U.S. including Alaska (used with permission from ASCE)

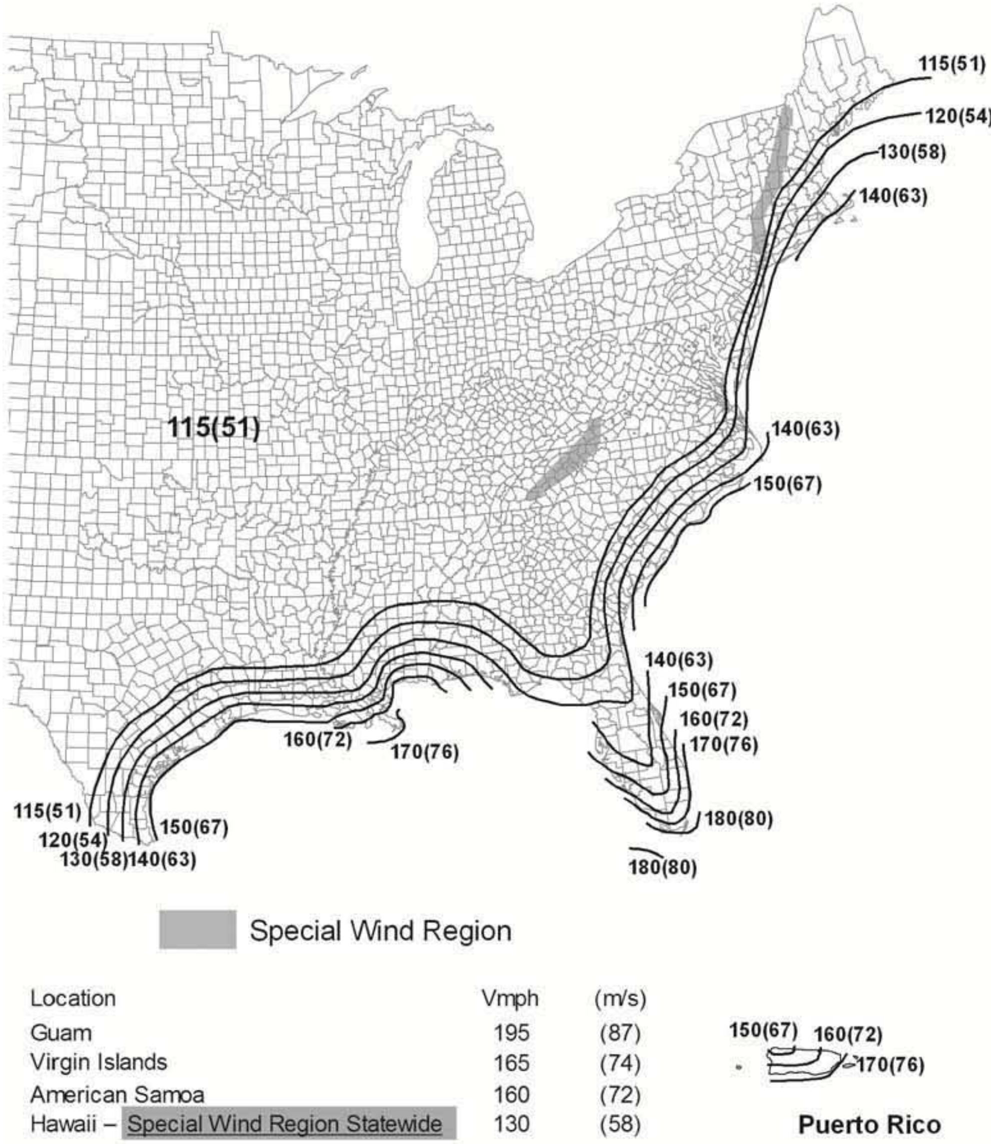


Figure 26.5-1A (*Continued*)

Figure 3.8-1b—700-Year MRI Basic Wind Speed, mph (m/s)—Eastern U.S. and Islands (used with permission from ASCE)

Notes:

1. Values are nominal design 3-s gust wind speeds in mph (m/s) at 33 ft above ground for Exposure C category,
2. Linear interpolation between wind contours is permitted.
3. Islands and coastal areas outside the last contour shall use the last wind speed contour of the coastal area.
4. Mountainous terrain, gorges, ocean promontories, and special wind regions shall be examined for unusual wind conditions.
5. Wind speeds correspond to approximately a 7 percent probability of exceedance in 50 yrs (Annual Exceedance Probability = 0.00143, MRI = 700 Yrs)

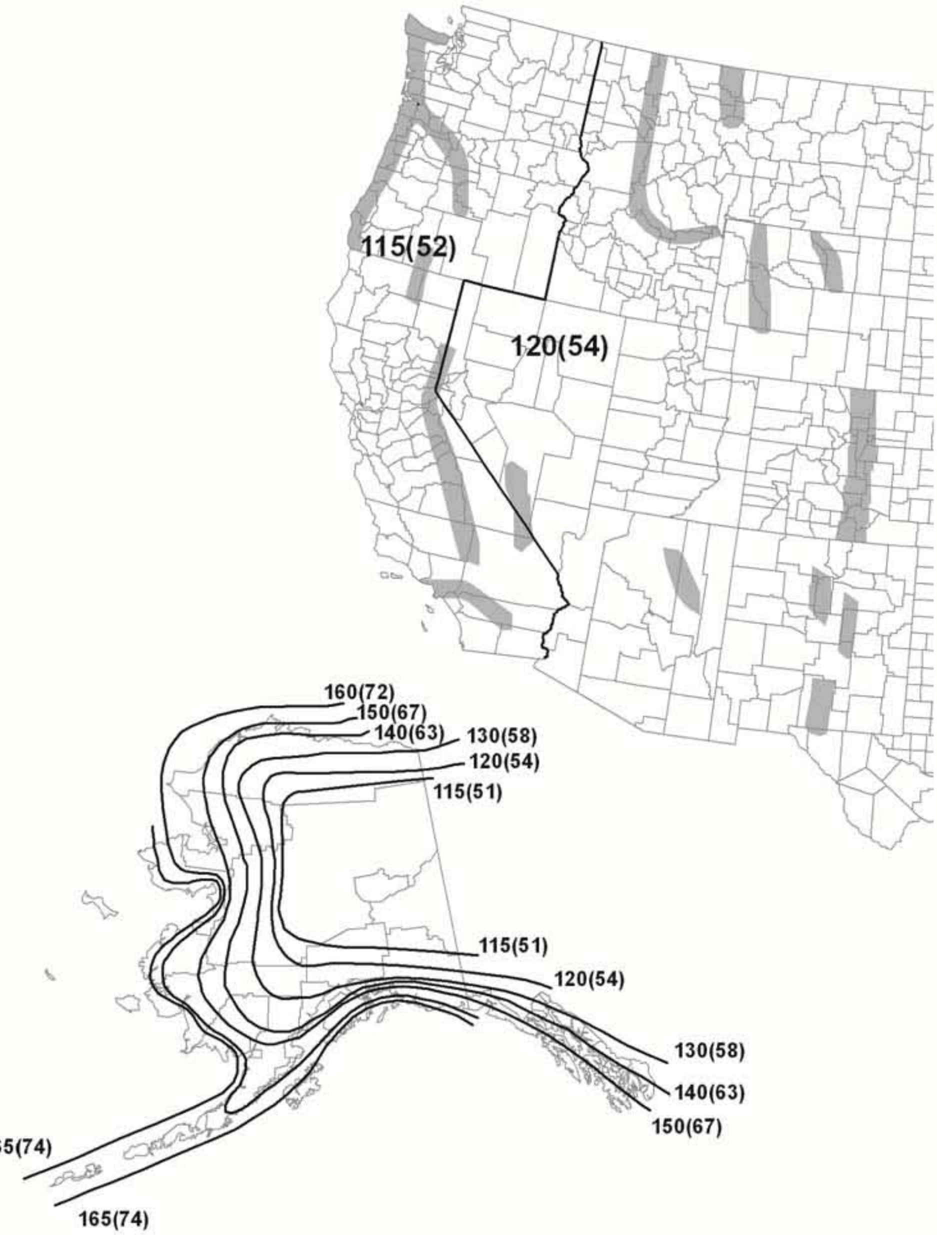


Figure 3.8-2a—1700-Year MRI Basic Wind Speed, mph (m/s)–Western U.S. including Alaska (used with permission from ASCE)

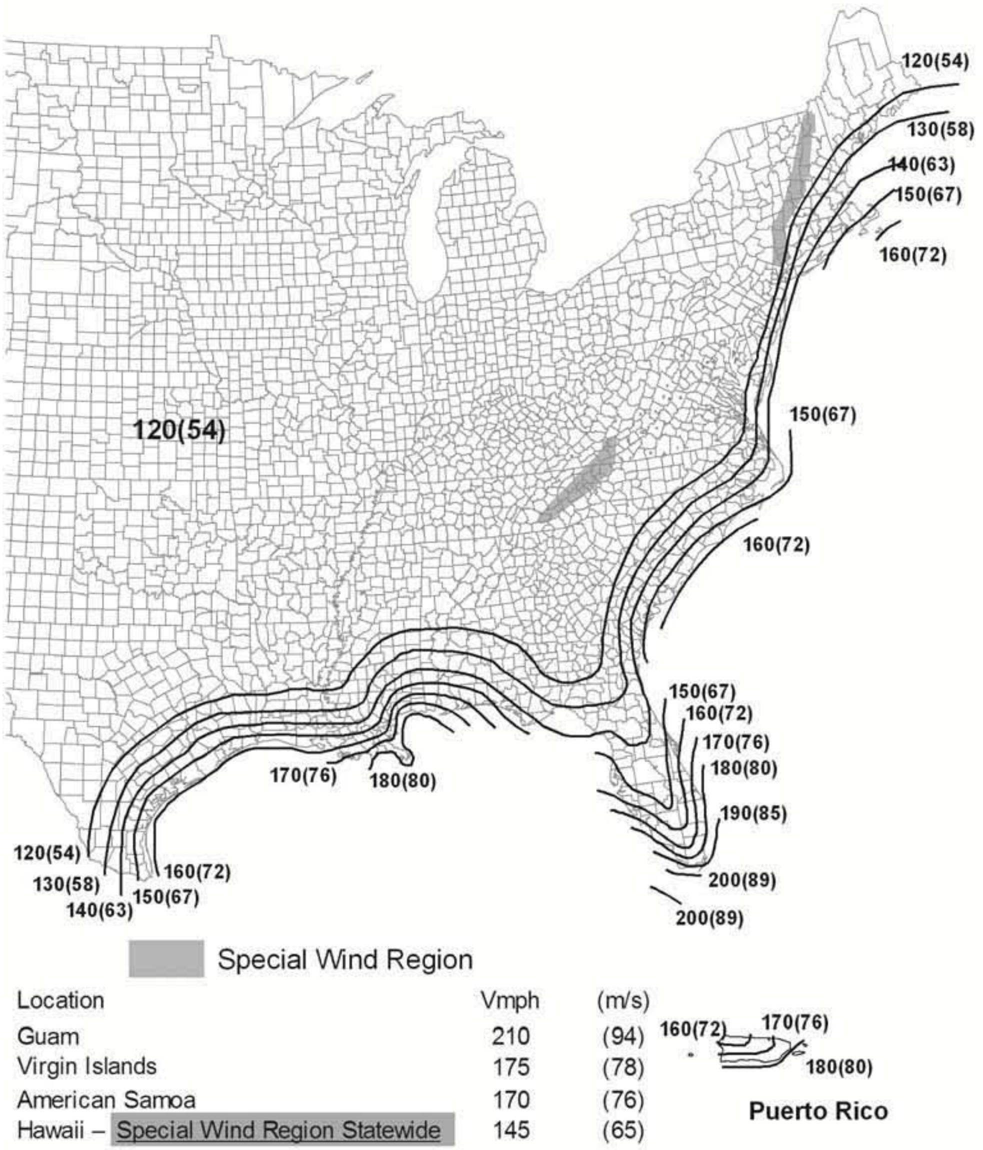


Figure 26.5-1B (*Continued*)

Figure 3.8-2b—1700-Year MRI Basic Wind Speed, mph (m/s)—Eastern U.S. and Islands (used with permission from ASCE)

Notes:

1. Values are nominal design 3-s gust wind speeds in mph (m/s) at 33 ft above ground for Exposure C category,
2. Linear interpolation between wind contours is permitted.
3. Islands and coastal areas outside the last contour shall use the last wind speed contour of the coastal area.
4. Mountainous terrain, gorges, ocean promontories, and special wind regions shall be examined for unusual wind conditions.
5. Wind speeds correspond to approximately a 3 percent probability of exceedance in 50 yrs (Annual Exceedance Probability = 0.000588, MRI = 1700 yrs)

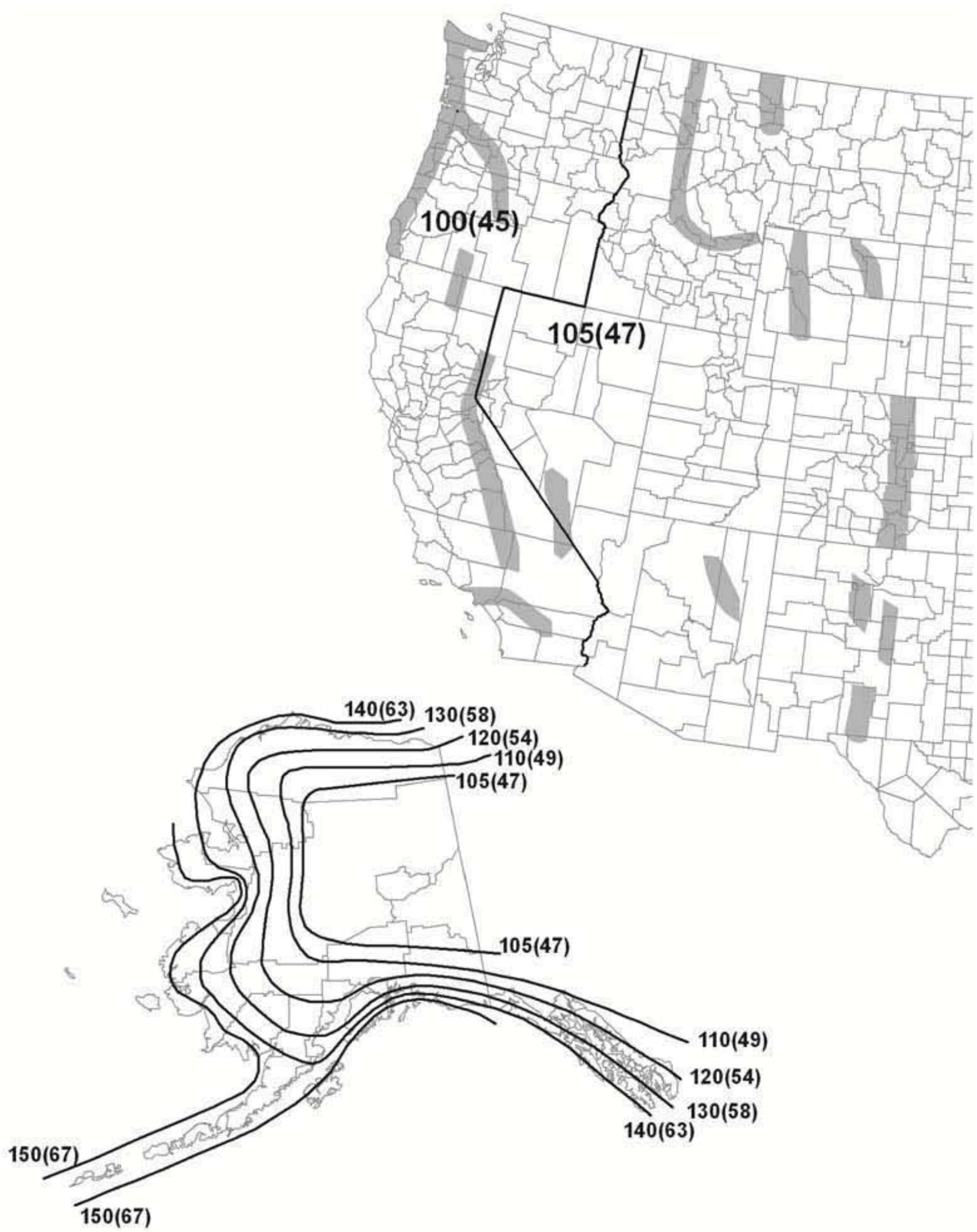


Figure 3.8-3a— 300-Year MRI Basic Wind Speed, mph (m/s)–Western U.S. including Alaska (used with permission from ASCE)

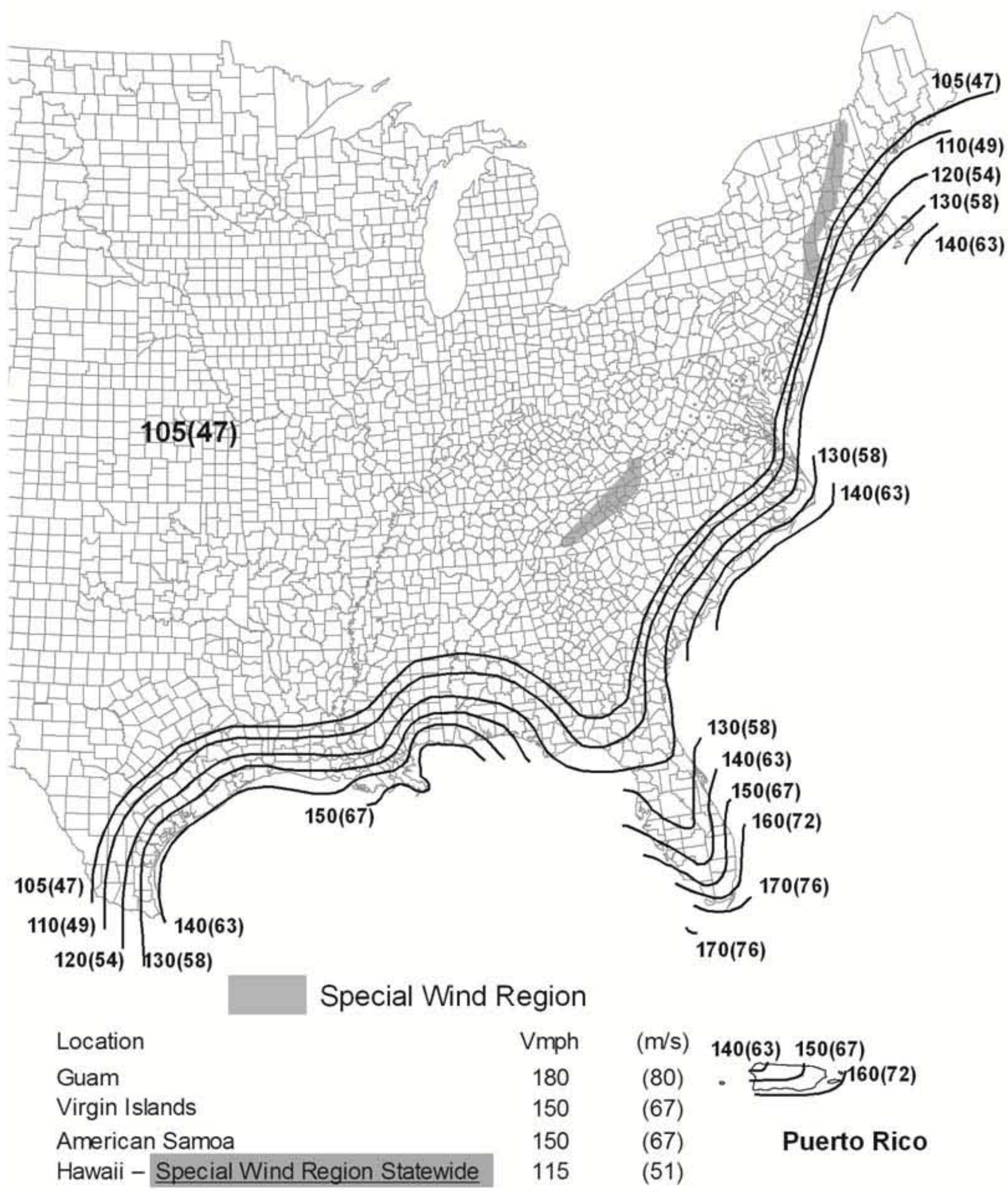


Figure 3.8-3b—300-Year MRI Basic Wind Speed, mph (m/s)—Eastern U.S. and Islands (used with permission from ASCE)

Notes:
1. Values are nominal design 3-s gust wind speeds in mph (m/s) at 33 ft above ground for Exposure C category,
2. Linear interpolation between wind contours is permitted.
3. Islands and coastal areas outside the last contour shall use the last wind speed contour of the coastal area.
4. Mountainous terrain, gorges, ocean promontories, and special wind regions shall be examined for unusual wind conditions.
5. Wind speeds correspond to approximately a 15 percent probability of exceedance in 50 yrs (Annual Exceedance Probability = 0.00333, MRI = 300 Yrs)

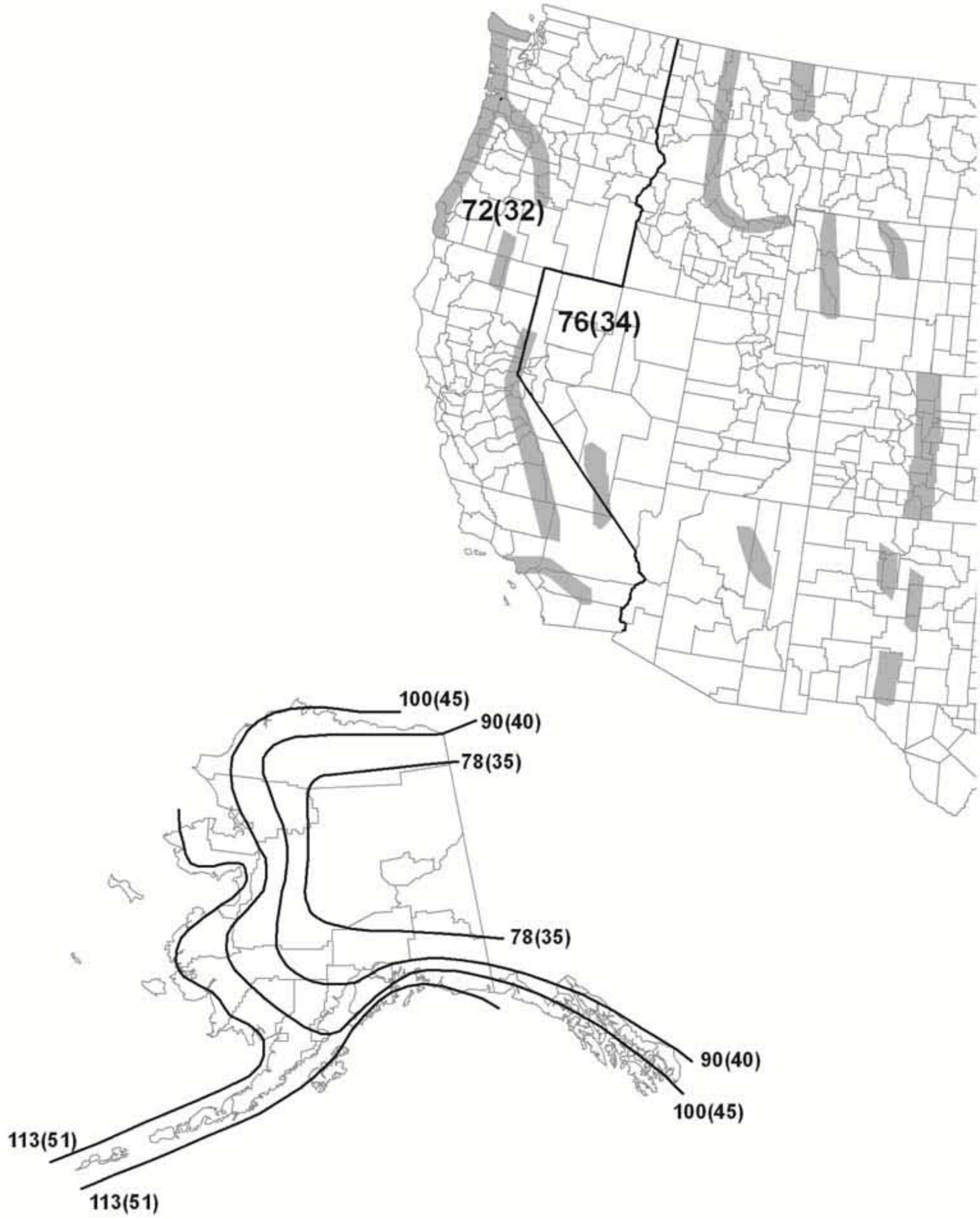


Figure 3.8-4a—10-Year MRI Gust Wind Speed, mph (m/s)–Western U.S. including Alaska (used with permission from ASCE)

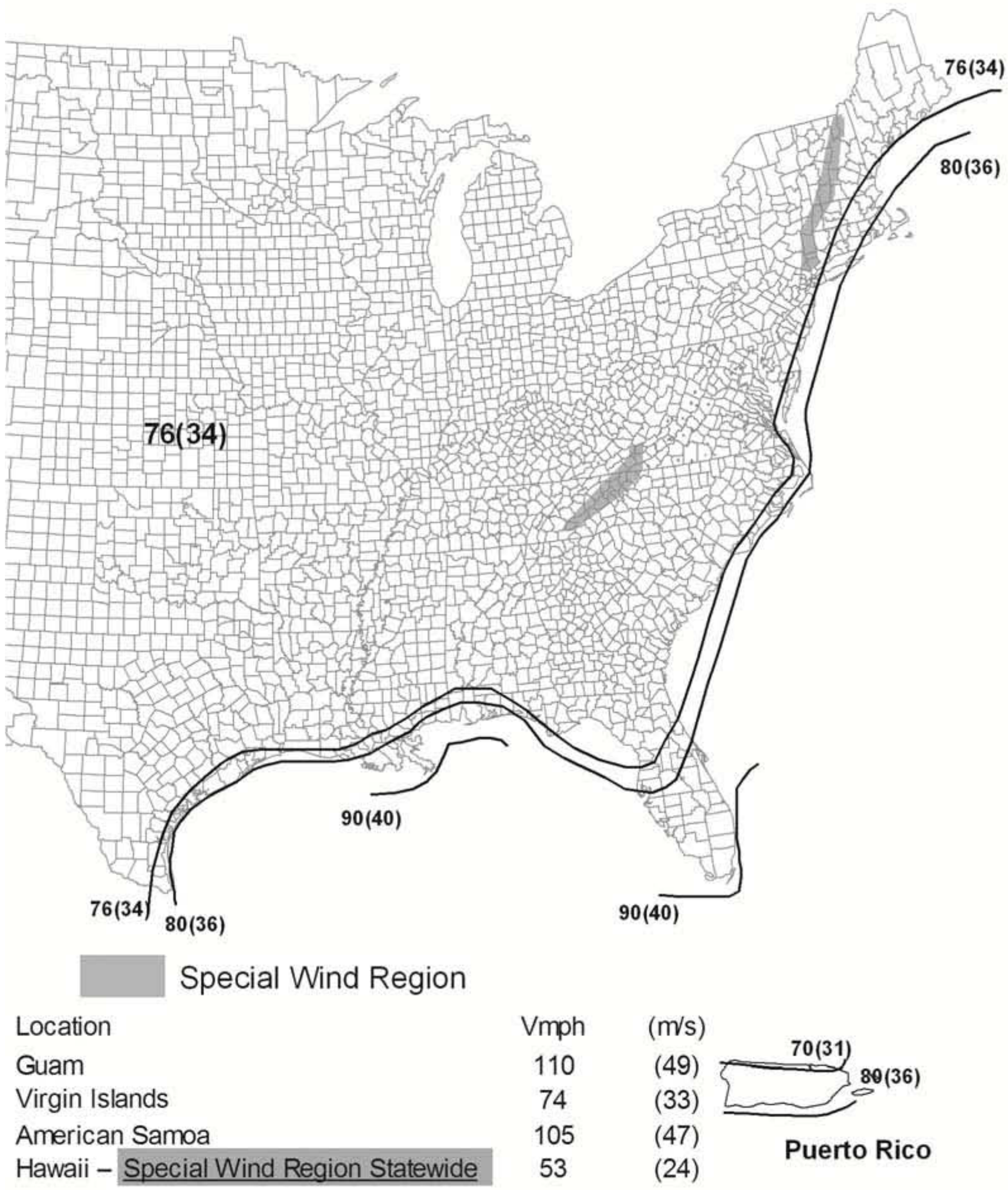


Figure 3.8-4b—10-Year MRI Gust Wind Speed, mph (m/s)—Eastern U.S. and Islands (with permission from ASCE)

Notes:

1. Values are nominal design 3-s gust wind speeds in mph (m/s) at 33 ft above ground for Exposure C category,
2. Linear interpolation between wind contours is permitted.
3. Islands and coastal areas outside the last contour shall use the last wind speed contour of the coastal area.
4. Mountainous terrain, gorges, ocean promontories, and special wind regions shall be examined for unusual wind conditions.

3.8.1—Wind Pressure Equation

The design wind pressure shall be computed using:

$$P_z = 0.00256 K_z K_d G V^2 C_d \text{ (psf)} \qquad (3.8.1\text{-}1)$$

where

V is the basic wind speed (mph),

K_z is the height and exposure factor defined in Article 3.8.4,

K_d is the directionality factor defined in Article 3.8.5,

G is the gust effect factor defined in Article 3.8.6, and

C_d is the drag coefficient defined in Article 3.8.7.

C3.8.1

The wind pressure equation is based on fundamental fluid-flow theory and formulations presented in ASCE/SEI 7-10. The risk category is considered in the velocity term in Article 3.8.

3.8.2—Basic Wind Speed

The basic wind speed V used in the determination of the design wind pressure for the Extreme I limit state shall be as given in Figures 3.8-1, 3.8-2, or 3.8-3. V is entered in mph. K_z, K_d, and G are dimensionless.

For service limit states, Figure 3.8-4 shall be used.

C3.8.2

The basic wind speeds in this Section are based on the 3-s gust wind speed maps presented in Figures 3.8-1, 3.8-2, and 3.8-3. The maps are based on peak gust data collected at 485 weather stations (Peterka, 1992; Peterka and Shahid, 1993) and from predictions of hurricane speeds on the United States Gulf and Atlantic Coasts (Batts et al, 1980; Georgiou et al, 1983; Vickery and Twisdale, 1993). The maps present the variation of 3-s gust wind speeds associated with a height of 33 ft for open terrain (Exposure C). See ASCE/SEI 7-10 for definitions of exposure categories.

Three-second gust wind speeds are used because most national weather service stations currently record and archive peak gust wind.

Service limit states use a MRI of 10 yrs.

3.8.2.1—Elevated Locations

For site conditions elevated considerably above the surrounding terrain, where the influence of ground on the wind is reduced, consideration must be given to using higher pressures at levels above 33 ft.

C3.8.2.1

It may be necessary in some cases to increase the basic wind speed to account for the effects of terrain. Although most situations will not require such an increase in wind speeds, ASCE/SEI 7 presents a rational procedure to increase the basic wind speed when a structure is located on a hill or escarpment.

The 0.00256 term in Equation 3.8.1-1 includes the density of air at standard conditions at sea level. The design wind pressure may be adjusted for the lower air density at higher elevation.

3.8.3—Special Wind Regions

The wind speed maps presented in Figures 3.8-1, 3.8-2, and 3.8-3 show several special wind regions. If the site is located in a special wind region, or if special local conditions exist in mountainous terrain and gorges, the selection of the basic wind speed should consider localized effects. Where records or experience indicate that wind speeds are higher than those reflected in Figures 3.8-1, 3.8-2, or 3.8-3, the basic wind speed should be increased using information approved by the Owner.

C3.8.3

If the wind speed is to be determined through the use of local meteorological data, ASCE/SEI 7 presents procedures for analyzing local meteorological data.Such increases in wind speed should be based on judgment and the analysis of regional meteorological data. In no case shall the basic wind speed be reduced below that presented in Figures 3.8-1, 3.8-2, or 3.8-3.

3.8.4—Height and Exposure Factor K_z

The height and exposure factor K_z shall be determined either from Table C3.8.4-1 or calculated using Eq. 3.8.4-1:

$$K_z = 2.0\left(\frac{z}{z_g}\right)^{\frac{2}{\alpha}} \qquad (3.8.4\text{-}1)$$

where z is the height above the ground at which the pressure is calculated or 16 ft, whichever is greater, and z_g and α are constants that vary with the exposure condition. Based on ASCE/SEI 7, α should be taken to be 9.5 and z_g should be taken to be 900 ft for Exposure C.

C3.8.4

K_z is a height and exposure factor that varies with height above the ground depending on the local exposure conditions and may be conservatively set to 1.0 for heights less than 33 ft. The variation is caused by the frictional drag offered by various types of terrain.

ASCE/SEI 7-10 defines acceptable wind design procedures using different terrain exposure conditions. For a specified set of conditions, the wind pressures associated with the different exposures increase as the exposure conditions progress from B to D, with exposure B resulting in the least pressure and exposure D resulting in the greatest pressure. Exposure C has been adopted for use in these Specifications as it represents open terrain with scattered obstructions. Exposure coefficients for other terrain conditions may be used per ASCE/SEI 7-10 with permission of the Owner.

Once the terrain exposure conditions are established, the height and exposure factor, K_z, is calculated using the relationship that is presented in ASCE/SEI 7-10.

Table C3.8.4-1 presents the variation of the height and exposure factor, K_z, as a function of height. The coefficient 2.01 in ASCE/SEI 7-10 is rounded to 2.0 in Eq. 3.8.4-1.

Table C3.8.4-1 —Height and Exposure Factors, K_z[a]

Height z, ft	K_Z
>15	0.84
20	0.90
30	0.98
40	1.04
50	1.09
60	1.13
70	1.17
80	1.20
90	1.23
100	1.26
110	1.28
120	1.31
130	1.33
140	1.35
150	1.37

[a] See Eq. 3.8.4-1 for calculation of K_z. (Exposure C)

3.8.5—Directionality Factor K_d

The directionality factor is defined in Table 3.8.5-1.

Table 3.8.5-1—Directionality Factors, K_d

Support Type	Directionality Factor
High-mast and Pole	
Round	0.95
Square	0.90
Octagonal	0.95
Dodecagonal	0.95
Hexdecagonal	0.95
Traffic Signal	0.85
Dynamic Message Sign	0.85
Overhead Frame/Truss	0.85
Support with horizontal arms or members supporting sign and/or signals	0.85

C3.8.5

The directionality factor accounts for two effects: the maximum wind can come from any direction and the probability that the maximum drag coefficient is associated with the wind direction is reduced. These values are consistent with those from ASCE 7-10 as based upon work by Ellingwood 1981 and Ellingwood et al, 1982.

3.8.6—Gust Effect Factor *G*

The gust effect factor, G, shall be taken as a minimum of 1.14.

C3.8.6

G is the gust effect factor and it adjusts the effective velocity pressure for the dynamic interaction of the structure with the gust of the wind.

Information presented in ASCE/SEI 7-10 states that if the fundamental frequency of a structure is less than one Hz or if the ratio of the height to least horizontal dimension is greater than 4, the structure should be designed as a wind-sensitive structure. Thus, virtually all structures addressed by these Specifications should be classified as wind-sensitive structures based on the height to least horizontal dimension ratio. It is not appropriate to use a nonwind-sensitive gust effect factor, G, for the design of sign, luminaire, and traffic signal structures. Special procedures are presented in the commentary of ASCE/SEI 7 for the calculation of the gust effect factor for wind-sensitive structures. The ASCE/SEI 7 calculation procedure requires reasonable estimates of critical factors such as the damping ratio and fundamental frequency of the structure. These factors are site and structure dependent. Relatively small errors in the estimation of these factors result in significant variations in the calculated gust effect factor. Therefore, even though sign, luminaire, and traffic signal support structures are wind sensitive, the benefits of using the ASCE/SEI 7 gust effect factor calculation procedure do not outweigh the complexities introduced by its use.

If the designer wishes to perform a more rigorous gust effect analysis, the procedures presented in ASCE/SEI 7 may be used with permission of the Owner.

3.8.7—Drag Coefficients C_d

The wind drag coefficient, C_d, shall be determined from Table 3.8.7-1.

C3.8.7

The wind drag coefficients in Table 3.8.7-1 were established based upon the work of several research projects as noted in the footnotes. Some coefficients are a strong function of Reynold's number. The term Vd is a simplified form of Reynolds using units convenient for LTS design. The algebraic form of these equations is somewhat different; however, the behavior is similar as illustrated in Figure C3.8.7-1 and C3.8.7-2 where different shapes and equations are shown.

The typical extreme event wind speed is 105 mph or greater. Therefore, for diameters 8-in or greater, the C_d is associated with the turbulent case, $Vd > 78$ mph-ft, and the C_d is a constant (rightmost column of Table 3.8.7-1). For the fatigue limit state, the wind speeds are on the order of 10 mph and the Vd will be low and the C_d will be the larger value in the leftmost column of Table 3.8.7-1. Between these extremes, the equations can the used.

This observation simplifies the load application where speed varies with height, etc. The reliability calibration used these bounds in determining the load and resistance factors. See NCHRP 796 (Puckett et al, 2014).

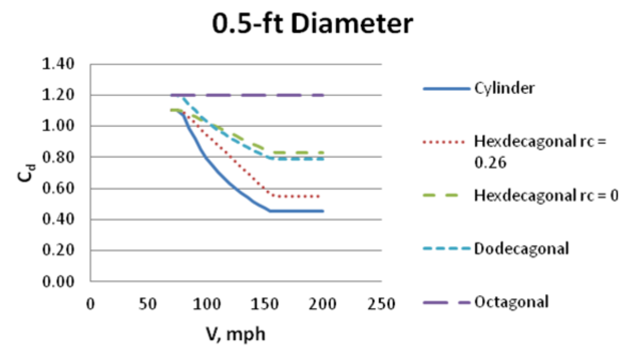


Figure C3.8.7-1—C_d for various shapes (6in., 0.5ft)

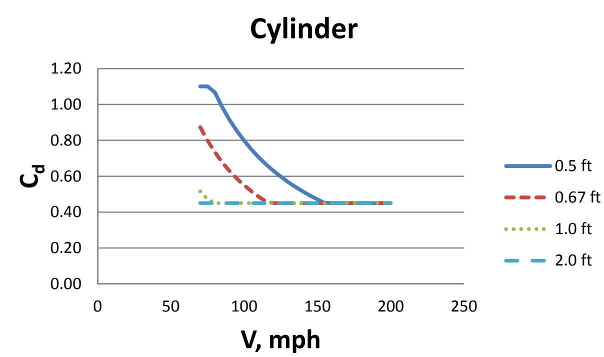
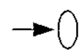
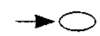


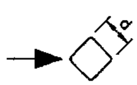
Figure C3.8.7-2—C_d for cylinder for various diameters

Table 3.8.7-1—Wind Drag Coefficients, C_d [a]

Sign Panel L_{sign}/W_{sign} = 1.0 2.0 5.0 10.0 15.0	 1.12 1.19 1.20 1.23 1.30	
Traffic Signals[b]	1.20	
Luminaires (with generally rounded surfaces)	0.50	
Luminaires (with rectangular flat side shapes)	1.20	
Elliptical Member ($D/d_o \le 2$)	Broadside Facing Wind $1.7\left(\frac{D}{d_o}-1\right)+C_{dD}\left(2-\frac{D}{d_o}\right)$ 	Narrow Side Facing Wind $C_{dd}\left[1-0.7\left(\frac{D}{d_o}-1\right)^{\frac{1}{4}}\right]$ 
Two Members or Trusses (one in front of other) (for widely separated trusses or trusses having small solidity ratios see note c)	1.20 (cylindrical) 2.00 (flat)	
Dynamic Message Signs (CMS)[g]	1.70	
Attachments	Drag coefficients for many attachments (cameras, luminaires, traffic signals, etc.) are often available from the manufacturer, and are typically provided in terms of effective projected area (EPA), which is the drag coefficient times the projected area. If the EPA is not provided, the drag coefficient shall be taken as 1.0.	

Continued on next page

Table 3.8.7-1—Wind Drag Coefficients, C_d [a]—*Continued*

Single Member or Truss Member	$Vd \le 39$ mph-ft	39 mph-ft < Vd < 78 mph-ft	$Vd \ge 78$ mph-ft
Cylindrical	1.10	$\dfrac{129}{(Vd)^{1.3}}$	0.45
Flat[d]	1.70	1.70	1.70
Hexdecagonal: 16-Sides $0 \le r_c < 0.26$	1.10	$1.37 + 1.08r_c - \dfrac{Vd}{145} - \dfrac{Vdr_c}{36}$	$0.83 - 1.08r_c$
Hexdecagonal: 16-Sides $r_c \ge 0.26$[e]	1.10	$0.55 + \dfrac{(78.2 - Vd)}{71}$	0.55
Dodecagonal[e]: 12-Sides	1.20	$\dfrac{10.8}{(Vd)^{0.6}}$	0.79
Octagonal[e]: 8-Sides	1.20	1.20	1.20
Square 	$2.0 - 6r_s$ [for $r_s < 0.125$] 1.25 [for $r_s \ge 0.125$]		
Diamond[f] 	1.70 [for $d = 0.33$ & 0.42] 1.90 [for $d \ge 0.50$]		

Notes:

a. Wind drag coefficients for members, sign panels, and other shapes not included in this table shall be established by wind tunnel tests (over an appropriate range of Reynolds numbers), in which comparative tests are made on similar shapes included in this table. Values reported in peer-reviewed publications based upon wind tunnel tests are acceptable. Reynolds Number $Re = (9200)V_{mph}d_{ft}$

b. Wind loads on free-swinging traffic signals may be modified based on experimental data or other criteria as agreed by the Owner (e.g., Marchman, 1971).

c. Data show that the drag coefficients for a truss with a very small solidity ratio are merely the sum of the drags on the individual members, which are essentially independent of one another. When two elements are placed in a line with the wind, the total drag depends on the spacing of the elements. If the spacing is zero or very small, the drag is the same as on a single element; however, if the spacing is infinite, the total force would be twice as much as on a single member. When considering pairs of trusses, the solidity ratio is of importance because the distance downstream in which shielding is effective depends on the size of the individual members. The effect of shielding decreases with smaller spacing as the solidity decreases. Further documentation may be found in *Transactions* (ASCE, 1961).

d. Flat members are those shapes that are essentially flat in elevation, including plates and angles.

e. Valid for members having a ratio of corner radius to distance between parallel faces equal to or greater than 0.125. For multisided cross-sections with a large corner radius, a transition value for C_d can be taken as:

If $r_c \le r_m$, then $C_d = C_{dm}$

If $r_m < r_c < r_r$, then $C_d = C_{dr} + (C_{dm} - C_{dr})[(r_r - r_c)/(r_r - r_m)]$

If $r_c \ge r_r$, then $C_d = C_{dr}$

where:

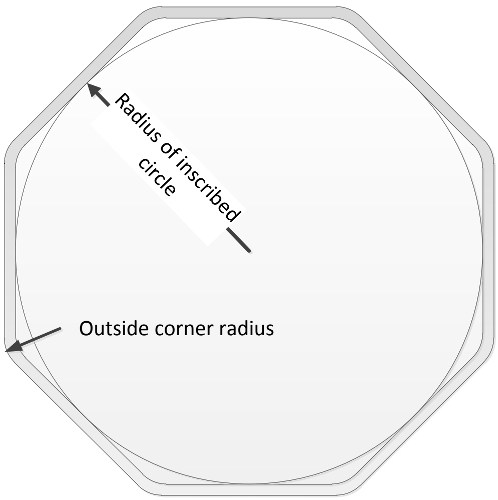
r_c = ratio of corner radius (outside) to radius of inscribed circle,

C_{dm} = drag coefficient for multisided section,

C_{dr} = drag coefficient for round section,

r_m = maximum ratio of corner radius to inscribed circle where the multisided section's drag coefficient is unchanged (see figure and table below), and

r_r = ratio of corner radius to radius of inscribed circle where multisided section is considered round (see figure and table below).



Shape	r_m	r_r
16-Sided, Hexdecagonal	0.26	0.63
12-Sided, Dodecagonal	0.50	0.75
8-Sided, Octagonal	0.75	1.00

f. The drag coefficient applies to the diamond's maximum projected area measured perpendicular to the indicated direction of wind.

g. A value of 1.7 is suggested for Dynamic Message Signs (DMS) until research efforts can provide accurate drag coefficients. This value may be used for both horizontal and vertical loads.

The validity of the drag coefficients (dimensionless) presented in Table 3.8.7-1 have been the subject of research (McDonald et al, 1995). Based on this work coupled with independent examinations of the information presented in Table 3.8.7-1, the drag coefficients were changed to account for 3-s gust wind speeds, except for square and diamond shapes.

Research concerning C_d values for dodecagonal shapes (12 sides) has been conducted at Iowa State University (James, 1971). The highest coefficients for the dodecagonal cylinder were measured for wind normal to a flat side. A circular cylinder was included in the testing program to allow a check on the test equipment, and boundary corrections were applied to the raw test data. All measured values for the dodecagonal cylinder with zero angle of incidence were higher than those measured for the circular cylinder. Lower shape coefficients might be justified for some velocities; however, this would require additional data for the dodecagons at lower Reynolds numbers and review of the factors specified for round cylinders.

The equation for the C_d value for an elliptical member with the narrow side facing the wind was empirically derived to fit wind tunnel test data.

The drag coefficients for hexadecagons (16 sided) include the effects that varying ratios of corner radius to cylinder radius have on the drag coefficient. The C_d values for C_vVd greater than 78 mph-ft were selected from information from wind tunnel tests on a number of hexadecagons with different ratios of corner radius to radius of inscribed circle (James, 1985).

These minimum C_d values vary linearly from 0.83 for a ratio of zero, to a value of 0.55 for ratios equal to or over 0.26. For consistency between maximum C_d values for cylinders and hexadecagons, the maximum C_d value for hexadecagons was selected to be the maximum value given for a cylinder. For a given ratio, values of C_d for Vd values between 39 mph-ft and 78 mph-ft vary linearly from 1.10 for Vd equal to 39 mph-ft to the minimum value at Vd equal to 78 mph-ft. The wind force resulting from use of these C_d values represents the total static force acting on the member, which would be the vector sum of the actual drag force and lift or side force.

If traffic signals or signs on span-wire pole structures are restrained from swinging in the wind, the full wind load must be applied. When agreed on between Owner and Designer, reduced forces may be used for free-swinging traffic signals when substantiated by research (Marchman, 1971; Marchman and Harrison, 1971). Wind loads on signs that are not restrained from swinging in the wind may be reduced with the consent of the Owner. Wind tunnel test results (Marchman and Harrison, 1971) indicate instability problems with traffic signals with certain hood configurations when not restrained from swinging. These instability problems should be considered when designing span-wire support structures. Orientation varies according to tests, but a value of 1.20 was shown to be conservative over a wide range. Values of C_d for square tubing, with the wind direction perpendicular to the side of the tube, have been revised to reflect the influence from the ratio of the corner radius to depth of member (James and Vogel, 1996).

A transition in the values of C_d for multisided cross-sections (hexdecagonal, dodecagonal, and octagonal) that approach round was developed in NCHRP 494, *Structural Supports for Highway Signs, Luminaires, and Traffic Signals* (Fouad et al., 2003) and incorporated as note *e* to Table 3.8.7-1. The method uses a linear equation to interpolate between the drag coefficient for round poles, C_{dr}, and the drag coefficient for multisided poles, C_{dm}, with respect to the variable r_c. If r_c is unknown, the section can conservatively be treated as multisided using the lowest reasonable value of r_c for the section.

When three members are used to form a triangular truss, the wind load shall be applied to all of the members. Even though all of the members are not in the same plane of reference, they may be seen in a normal elevation.

As provided in note *b* to Table 3.8.7-1, consideration may be given to modifying the forces applied to free-swinging traffic signals. Traffic signals are a combination of aerodynamic and non-aerodynamic shapes and open spaces; because of these factors, it is difficult to predict traffic signal wind loading, especially when the signal head is free to swing. At this time, it appears that the only practical means of predicting wind loading on free-swinging traffic signals is through wind-loading tests.

The fatigue pressures required in Section 11 for high-mast light towers were developed assuming the values provided for $Vd < 39$ mph-ft., i.e., C_d ranging from 1.10 to 1.20. Wind tunnel test results for these shapes may indicate a higher C_d for low wind speeds. However, because the fatigue pressures in Article 11.7.2 were developed (calibrated) using tabulated values here, these C_d values should be used. Additionally, an adjustment for air density is not permitted for fatigue.

3.9—DESIGN WIND LOADS ON STRUCTURES

Figures 3.9-1 through 3.9-3 depict application of loads on various types of structural supports.

3.9.1—Load Application

The wind loads acting horizontally on a structure shall be determined by the areas of the supports, signs, luminaires, signals, and other attachments and shall be applied to the surface area as viewed in normal elevation (vertical projection).

The effective projected area (EPA) is the projected area multiplied by the appropriate drag coefficient. If the EPA is provided for the luminaire or attachment, the design wind pressure (Article 3.8.1) shall be computed without incorporating the drag coefficient C_d.

C3.9.1

The largest projected area or the effective projected area may be used when calculating wind loads at various wind angles from the normal elevation for three-dimensional attachments such as traffic signals and luminaires.

When the effective projected area (EPA) is supplied by the manufacturer of the luminaire or other attachments, then validation of the EPA values (such as through wind tunnel tests or computation fluid dynamics) should be made available upon request of the Owner.

3.9.2—Design Loads for Horizontal Supports

Horizontal supports of luminaire support structures and all connecting hardware shall be designed for wind loads, W_l and W_h, applied at the centroids of pressure of the respective areas.

Horizontal supports of sign structures (cantilevered or bridge type) and traffic signal structures shall be designed for wind loads, W_h and W_p, applied normal to the support at the centroids of the respective areas.

Horizontal supports for span-wire pole structures shall be designed for wind loads, W_h and W_p, where W_h may be applied as a series of concentrated loads along the span wire.

3.9.3—Design Loads for Vertical Supports

The vertical supports for luminaire structures, traffic signal structures (excluding pole-top mounted traffic signals and pole-top luminaire structures), and sign support structures shall be designed for the effects of wind from any direction.

In lieu of a rigorous analysis, the effects of wind from any direction may be approximated by the combinations of normal and transverse wind loads acting simultaneously.

C3.9.3

These Specifications provide a simplified means to account for the effects of wind from any direction. Other more rigorous methods that appropriately account for the effects of wind from any direction may also be used.

These wind load cases are consistent with ASCE/SEI 7-10 Article 27.4. A rigorous analysis implies wind from many directions using the associated projected areas and drag coefficients. This level of refinement is typically

Table 3.9.3-1 defines three load cases: (1) full wind normal to the plane of the structure, (2) full wind transverse to the plane of the structure, and (3) 0.75 times the full wind in both directions applied simulataneously.

Table 3.9.3-1—Wind Load Cases

Load Case	Normal Component (n_c)	Transverse Component (t_c)
1	1.0 (BL_n)	0(BL_t)
2	0(BL_n)	1.0(BL_t)
3	0.75(BL_n)	0.75(BL_t)

The two basic loads (BL_n and BL_t) for structures with rigid horizontal supports shall be the effects from the wind loads, W_v, W_l, W_p, and W_h, applied at the centroid of the associated areas. BL_n is the basic load with W_v, W_l, W_p, and W_h applied normal to the plane of the structure. BL_t is the basic load with W_v, W_l, W_p, and W_h applied transverse to the plane of the structure. Non-symmetrical single roadside sign supports shall be designed for normal and transverse components (n_c, t_c).

Vertical supports for span-wire pole structures (provided only a single span wire is attached to each support) shall be designed for the full wind loads, W_v, W_p, and W_h, applied normal to the span wire, without the application of the transverse components (t_c).

unnecessary; however it may be automated with minimal effort required. For the simpler approach using load case 3, the projected areas and drag coefficents should be used for the two orthogonal directions.

The envelope of the three load cases is used to design components where coincident actions may be used for force interaction effects. Alternatively, the largest actions from any load case may be conservatively combined in most cases.

Article 3.9.3 is a different approach than used in previous versions of these specifications.

For structures with components oriented primarily in one plane, only the normal case requires analysis and other cases may be dismissed by engineering judgment. For structures with major components oriented in different planes, e.g. a traffic signal with two orthogonal arms, the normal and transverse loads may be significant.

For indeterminate frames, the loads may be distributed in proportion to the relative lateral stiffness and restraint conditions of the supports. A frame analysis is typical for such cases.

The transverse components (t_c) may be neglected for the design of typical span-wire structures (provided only a single wire is attached to each support), because the wind direction resulting in the maximum tension in the span wire should be normal to the span.

A possible exception would be if the combined projected areas of the sides of the attachments are significantly greater than their combined projected areas parallel to the span.

3.9.4—Unsymmetrical Wind Loading

To allow for unsymmetrical wind loading, the loading conditions stipulated in Articles 3.9.4.1 and 3.9.4.2 shall be used in conjunction with those of Articles 3.9.2 and 3.9.3 to compute the torsional load effects. The resulting torsional stresses shall be included with the wind load stresses for the fully loaded structure.

3.9.4.1—Overhead Cantilevered Supports

For vertical supports with balanced double cantilevers (i.e., equal torsional load effects from each arm), the normal wind load shall be applied to one arm only, neglecting the force on the other arm. For unbalanced double cantilevers, the normal wind loads shall be applied only to the arm that results in the larger torsional load effect.

When vertical supports have more than two arms, and the arms are mounted opposite or at diverging angles from one another, the wind load shall be applied to one arm when balanced or only to the arm that results in the largest torsional load effect when unbalanced.

3.9.4.2—Concentrically Mounted Supports

For high-level (pole or truss type) lighting structures, pole top mounted luminaire supports, pole top mounted traffic signals, or roadside signs with single supports, the torsional load effect for concentrically mounted attachments shall be calculated as the wind loads, W_p, W_l, and W_h, multiplied by 0.15b, where b is the width measured between the out-to-out extremities of the attachments.

For nonconcentrically mounted attachments, the torsional load effect shall be calculated using the net torque.

C3.9.4.2

A minimum 15 percent eccentricity is required for concentrically loaded support structures.

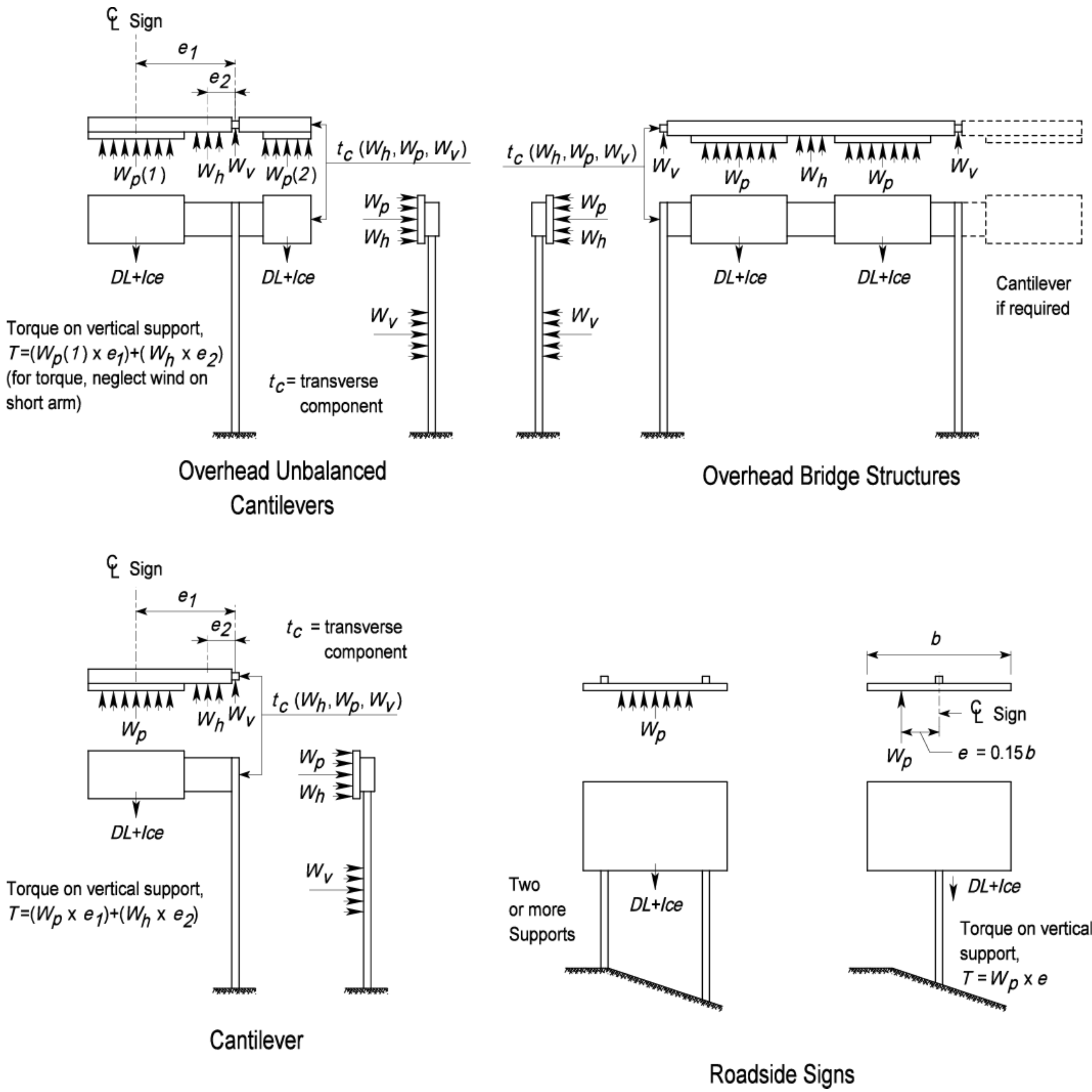


Figure 3.9-1—Loads on Sign Support Structures

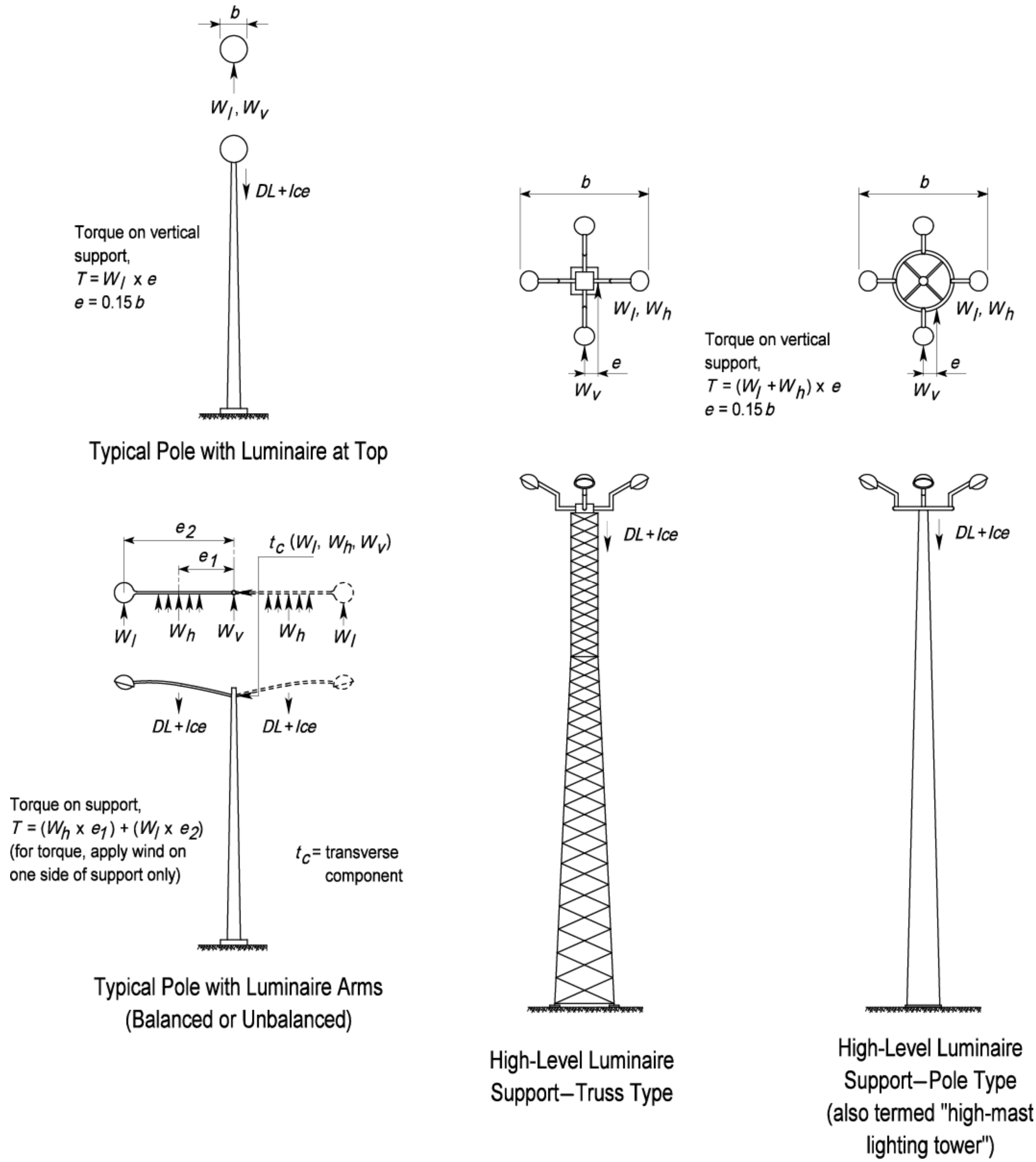


Figure 3.9-2—Loads on Luminaire Support Structures

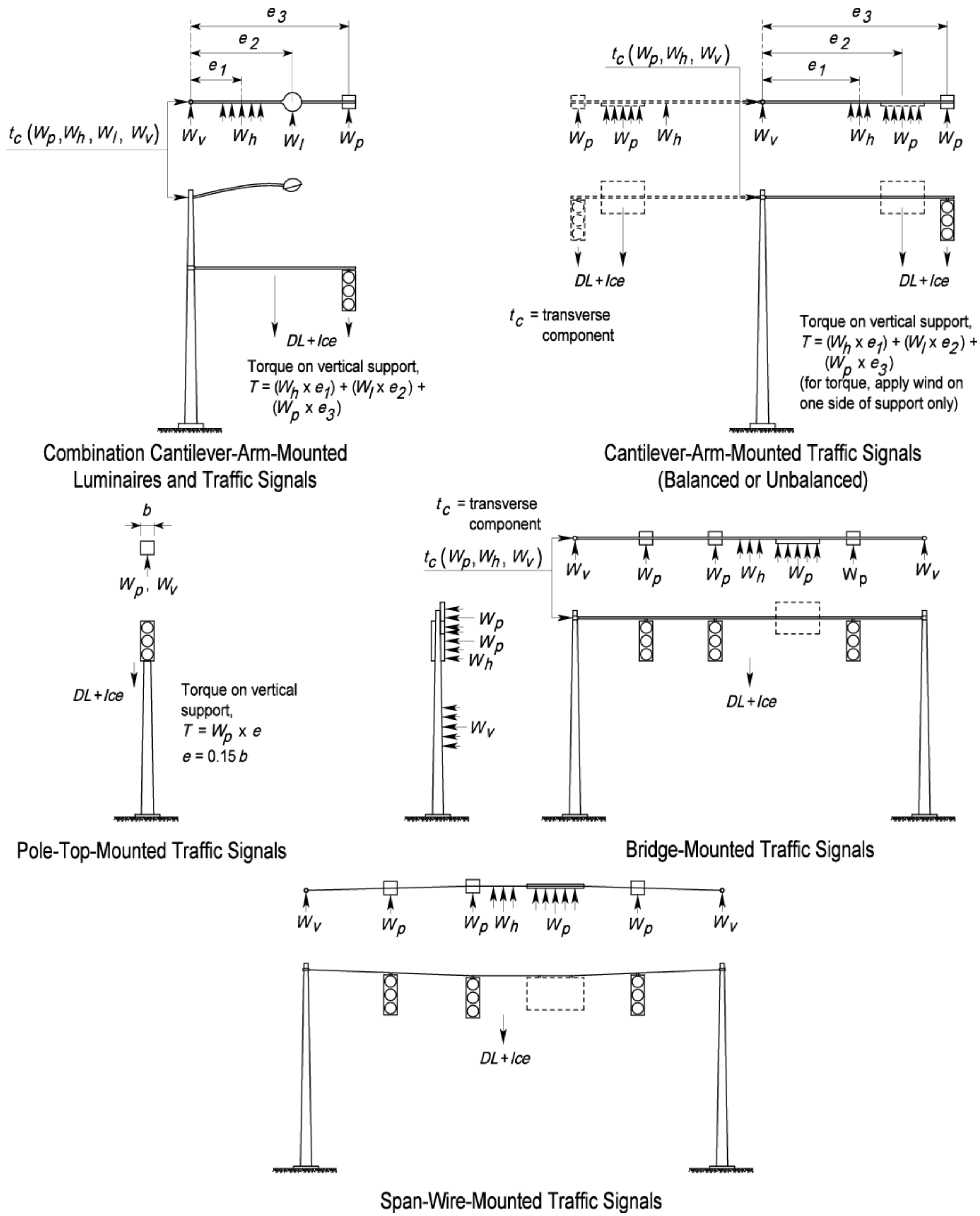


Figure 3.9-3—Loads on Traffic Signal Support Structures

3.10—REFERENCES

AASHTO. 2011. *Guide Specifications for LRFD Seismic Bridge Design*, Second Edition, American Association of State Highway and Transportation Officials, Washington, DC.

ASCE. 1961. *Wind Forces on Structures – Final Report of the Task Committee on Wind Forces*, Transactions of the American Society of Civil Engineers, Vol. 126, No. 2, February 1961, pp. 1124-1198. American Society of Civil Engineers, Reston, VA.

ASCE. 2010. *Minimum Design Loads for Buildings and Other Structures*, Report no. ASCE/SEI 7-10. American Society of Civil Engineers, Reston, VA.

Batts, M. E., M. R. Cordes, L. R. Russell, J. R. Shaver, and E. Simiu. 1980. *Hurricane Wind Speeds in the United States*, NBS Building Science Series 124. National Bureau of Standards, Washington, DC.

Ellingwood, B. 1981. *Wind and Snow Load Statistics for Probabilistic Design* Journal of the Structural Division, Vol. 107, No. 7, July 1981, pp. 1345-1350. American Society of Civil Engineers, Reston, VA.

Ellingwood, B., MacGregor, J. G., Galambos, T. V., and Corniess, C. A. 1982. *Probability-Based Load Criteria: Load Factors and Load Combinations,* Journal of the Structural Division, Vol. 108, No. 5, May 1982, pp. 978-997. American Society of Civil Engineers, Reston, VA.

Fouad, F.H., J.S. Davison, N. Delatte, E.A. Calvert , S.E. Chen, E. Nunez, and R. Abdalla. 2003. *Structural Supports for Highway Signs, Luminaires, and Traffic Signals*, NCHRP Report 494. Transportation Research Board, National Research Council, Washington DC.

Georgiou, P. N., A. G. Davenport, and B. J. Vickery. 1983. "Design Wind Speeds in Regions Dominated by Tropical Cyclones," *Journal of Wind Engineering and Industrial Aerodynamics*. Elsevier, Amsterdam, The Netherlands, Vol. 13, pp. 139–152.

James, W. D. 1971, Wind Tunnel Tests of a Two-Dimensional Round-Cornered Dodecagonal Cylinder, Meyer Manufacturing, Red Wing, MN.

James, W. D. 1985. Effects of Reynolds Number and Corner Radius on Two-Dimensional Flow Around Hexdecagonal Cylinders, AIAA-83-1705. In Proc., AIAA Sixth Computational Fluid Dynamics Conference, Danvers, MA, American Institute of Aeronautics and Astronautics, Reston, VA.

James, W. D., and J. M. Vogel. 1996. Variation of Section Drag Coefficient with Variation in Reynolds Number for Square Cylinders with Various Corner Radii, Valmont Industries, Valley, NE.

Krayer, W. R., and R. D. Marshall. 1992. "Gust Factors Applied to Hurricane Winds," *Bulletin of the American Meteorological Society.* American Meteorology Society, Washington, DC, Vol. 73, pp. 613–617.

Marchman, III, J. 1971. "Wind Loading on Free-Swinging Traffic Signals," *Transportation Engineering Journal*. American Society of Civil Engineers, New York, NY, Vol. 97, No., TE2, pp. 237–246. (May 1971)

Marchman III, J. F., and W. P. Harrison, Jr. 1971. *Wind Loading on Traffic Signals,* Report to Hapco Company, Abingdon, VA. Virginia Polytechnic Institute, Blacksburg, VA.

McDonald, J. R., K. C. Mehta, W. W. Oler, and N. Pulipaka. 1995. *Wind Load Effects on Signs, Luminaires, and Traffic Signals Structures*, Report No. 1303-F. Wind Engineering Research Center, Texas Tech University, Lubbock, TX.

Minor, R. C. 1972. *Wind Tunnel Tests of Elliptical and Round Tubes*. Hapco Company, Abingdon, VA.

Peterka, J. A. 1992. "Improved Extreme Wind Prediction for the United States," *Journal of Wind Engineering and Industrial Aerodynamics.* Elsevier, Amsterdam, The Netherlands, Vol. 41, pp. 533–541.

Peterka, J. A., and S. Shahid. 1993. "Extreme Gust Wind Speeds in the U.S.," *Proceedings, 7th U.S. National Conference on Wind Engineering* 2. University of California, Los Angeles, Los Angeles, CA, pp. 503–512.

Puckett, J., Garlich, M., Barker, M., Nowak, A., Menzemer, C., 2014. Development and Calibration of AASHTO-LRFD Specifications for Structural Supports for Highway Signs, Luminaires, and Traffic Signals, NCHRP Report 796. Transportation Research Board, National Research Council, Washington, DC.

Simiu, E. and R. H. Scanlan. 1986. *Wind Effects on Structures*, Second Edition. John Wiley & Sons, New York, NY.

Simiu, E., M. J. Changery, and J. J. Filliben. 1979. "Extreme Wind Speeds at 129 Stations in the Contiguous United States," *Building Science Series,* Report 118. National Bureau of Standards, Washington, DC.

Vickery, B. J., and L. A. Twisdale. 1993. "Prediction of Hurricane Wind Speeds in the U.S," *Proceedings, 7th National Conference on Wind Engineering* 2. University of California, Los Angeles, Los Angeles, CA.

SECTION 4: ANALYSIS AND DESIGN—GENERAL CONSIDERATIONS

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SECTION 4:

ANALYSIS AND DESIGN—GENERAL CONSIDERATIONS

4.1—SCOPE

This Section describes methods of analysis for the design of structural supports for highway signs, luminaires, and traffic signals. Other methods of analysis that are based on documented material characteristics and satisfy equilibrium and compatibility may also be used.

C4.1

The overall analysis philosophy for structural supports is stated in this Section. Additionally, approximate methods of analyses and simplified equations to assist the Designer are provided in appendices. Procedures for second-order analysis and calculating forces in span-wire structures are provided in Appendix A.

4.2—DEFINITIONS

Equilibrium—A state where the sums of forces and moments about any point in space are equal to zero.

Secondary Bending Moment—An increase in the bending moment resulting from axial load and the structure's lateral translation under load.

4.3—NOTATION

B_2	=	moment magnification factor for second-order effects (4.8.1) (C4.8.1)
D_P	=	factored weight of the pole (kips) (4.8.1)
E	=	modulus of elasticity (ksi) (4.8.1) (C4.8.1)
F_y	=	specified minimum yield stress (ksi) (4.8.1) (C4.8.1)
I_B	=	moment of inertia for the cross-section at the base of the pole (in.^{4}) (4.8.1)
I_T	=	moment of inertia for the cross-section at the top of the pole (in.^{4}) (4.8.1)
k	=	slenderness factor (4.8.1) (C4.8.1)
L	=	length of the pole (in.) (4.8.1) (C4.8.1)
P_T	=	factored vertical concentrated load at the top of the pole (kips) (4.8.1)
$P_{\text{equivalent}}$	=	equivalent axial load for a non-prismatic cantilever with a concentrated load at the tip (kips) (4.8.1)
$P_{\text{EulerBottom}}$	=	Euler buckling load based upon the bottom moment of inertia (kips) (4.8.1)
r	=	radius of gyration (in.) (4.8.1) (C4.8.1)

4.4—DESIGN METHOD

These Specifications follow a Load and Resistance Factor Design (LRFD) approach for the design of steel, aluminum, wood, fiber-reinforced composite, and prestressed concrete structural supports.

4.5—STRUCTURAL ANALYSIS

Structural analysis shall be used for determining the critical load effects (axial, shear, bending, torsion and combinations) for which the members and connections shall be designed. Methods of analysis that satisfy the requirements of equilibrium and compatibility and use the linear stress–strain relationship for the material may be used.

Analysis and design of structural supports shall conform to generally accepted engineering practices.

C4.5

The apparent inconsistency between linear structural analysis and plastic analysis of cross sections for resistance is supported by the lower-bound theorem. Details can be found in Barker and Puckett (2013).

4.6—DESIGN OF STRUCTURAL SUPPORTS

4.6.1—Vertical Cantilever Supports (Pole-Type)

Second-order effects in accordance with Article 4.8 shall be considered in the design of vertical pole-type cantilever supports.

C4.6.1

The approximate method outlined in Article 4.8 is suitable for hand computations. Alternatively, rigorous tabular method may be used. This method is readily programmed into a spreadsheet. See NCHRP Report 796 Examples (Puckett et al., 2014).

4.6.2—Horizontal Supports (Single-Member or Truss)

Horizontal supports (single-member or truss) shall be proportioned using the combined stress equations given in these Specifications.

4.6.3—Horizontal Supports (Span Wire and Connections)

The wire and its connection in span-wire structures shall be designed for the maximum tension force encountered.

C4.6.3

The procedures outlined in Appendix A may be used for span-wire structures.

4.7—ANALYSIS OF SPAN-WIRE STRUCTURES

The analysis of span-wire structures shall be performed using methods based on commonly accepted principles of mechanics. Tensions and deflections of span wires shall satisfy equilibrium and compatibility. Loads applied to span-wire structures shall be computed according to Section 3, "Loads." Gravity and wind loads induced by attachments (i.e., signs, signals, and accessories) may be applied as equivalent point loads. Gravity and wind loads induced by the span wire may be applied as concentrated loads.

Refined analytical methods based on large-deflection theory or finite element formulations should be considered in the analysis of complex span-wire configurations because the behavior of these structures may not be adequately addressed by using small-deflection theory.

C4.7

Because of the nonlinear relationship between geometry and forces in span wires, superposition principles should not be applied to combine the effects of different loads. Therefore, the analysis should be performed considering a single load case with all loads acting simultaneously.

Appendix A provides two methods to compute tensions on span wires. The simplified method, outlined in Appendix A is intended to consider the case of rigid vertical supports. A detailed method, outlined in the same appendix, is intended to consider the case of flexible vertical supports.

4.8—SECOND-ORDER EFFECTS

For vertical cantilever support structures (pole-type), the secondary bending moment caused by the axial load shall be accounted for by provisions of Article 4.8.1, unless a more detailed calculation is made in accordance with Article 4.8.2.

C4.8

When a member is subjected to axial compressive stresses acting simultaneously with bending stresses, a second-order moment equal to the product of the resulting eccentricity times the applied axial load is generated. To account for this effect on vertical pole-type members, two methods of evaluating bending stresses are presented: the simplified method outlined in Article 4.8.1, and the detailed method outlined in Article 4.8.2.

The simplified method is intended primarily for hand computations, whereas the detailed method is intended for situations where a refined analysis is desired and a computer is available. The simplified method is conservative with respect to the detailed method.

Second-order effects in horizontal members are usually negligible because the associated axial forces are small.

4.8.1—Simplified Method

In the combined resistance equations for steel and aluminum (in Article 5.12 and Article 6.12), the bending load effect shall be multiplied by the coefficient for amplification, B_2, to account for the secondary moment. The coefficient for amplification, B_2, may be taken as:

$$B_2 = \frac{1}{1 - \left[\dfrac{P_{equivalent}}{P_{Euler\,bottom}}\right]} \geq 1.0 \tag{4.8.1-1}$$

where

$$P_{equivalent} = \sqrt[3]{\frac{I_B}{I_T}} P_T + 0.38 D_P$$

$$P_{Euler\,bottom} = \frac{\pi^2 E I_B}{(kL)^2}$$

where:

P_T = factored vertical concentrated load at the top of the pole (kip),

D_P = factored weight of the pole (kip),

I_B = moment of inertia for the cross-section at the base of the pole (in.^{4}),

I_T = moment of inertia for the cross-section at the top of the pole (in.^{4}),

k = slenderness factor,

L = length of the pole (in.), and

r = radius of gyration (in.).

Eq. 4.8.1-1 is valid where:

$$2\pi \sqrt{\frac{E}{F_y}} \leq \frac{kL}{r}$$

If this limit is exceeded, analysis by a detailed method is required.

C4.8.1

The coefficient for amplification B_2 is included in the Specifications to be used mainly for vertical cantilever supports over 55 ft in height or where other conditions are such that secondary P-Δ effects are significant.

The term B_2 is traditionally used to represent moment magnification due to second-order load effects due to P-Δ. For example see, AISC, ACI, and/or *AASHTO LRFD Bridge Design Specifications*.

The term B_1 is traditionally used to represent moment magnification based upon axial softening and chord-slope deformation, i.e., non-sway. This type of magnification is small in the types of supports addressed in these specifications and can be ignored under the limiting slenderness required in this article.

Eq. 4.8.1-1 is limited to values of kL/r greater than or equal to:

$$2\pi \sqrt{\frac{E}{F_y}}$$

to ensure that the maximum axial stress is limited to $0.25F_y$, where F_y is the specified minimum yield strength and E is the modulus of elasticity. This requirement is intended to keep the axial stresses sufficiently low such that effects of residual stresses on the buckling behavior of the pole can be ignored. The radius of gyration, r, may be calculated at a distance of $0.50L$ for a tapered column. The slenderness factor, k, may be assigned 2.0 for this approximation.

4.8.2—Detailed Method

In lieu of the approximate procedure of Article 4.8.1, a second-order elastic analysis that is applicable to all materials covered by the Specifications may be performed considering the final deflected position of the vertical support. Limit state load factors shall be applied. All loads may be assumed to be applied simultaneously.

C4.8.2

As an alternative procedure, a more rigorous method of analysis is presented in NCHRP Report 796 (Puckett et al., 2014), whereby the member is analyzed considering the actual deflected shape. With this method, the coefficient of amplification, B_2, is taken as 1.0 because the secondary moment is included in the load effects directly from analysis. This method implies a nonlinear relationship between the applied loads and the resulting deflections.

4.9—REFERENCES

AASHTO. 2014. *AASHTO LRFD Bridge Design Specifications,* Seventh Edition. American Association of State Highway and Transportation Officials, Washington, DC.

ACI. 2014. *Building Code Requirements for Structural Concrete*, ACI 318-14. American Concrete Institute, Farmington Hills, MI.

AISC. 2010. Manual of Steel Construction, 14th Ed. American Institute of Steel Construction, Chicago, IL.

Barker and Puckett. 2013. *Design of Highway Bridges–An LRFD Approach*, third edition, Wiley, New York, NY.

Fouad, F. H., E. A. Calvert, and E. Nunez. 1998. *Structural Supports for Highway Signs, Luminaires, and Traffic Signals*, NCHRP Report 411. Transportation Research Board, National Research Council, Washington, DC.

Puckett, J., M. Garlich, M. Barker, A. Nowak, C. Menzemer, 2014. *Development and Calibration of AASHTO-LRFD Specifications for Structural Supports for Highway Signs, Luminaires, and Traffic Signals*, NCHRP Report 796. Transportation Research Board, National Research Council, Washington, DC.

SECTION 5: STEEL DESIGN

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SECTION 5

STEEL DESIGN

5.1—SCOPE

This Section specifies design provisions for steel structural supports. Fatigue-sensitive steel support structures are further addressed in Section 11. Additional design provisions not addressed in this Section shall be obtained from other references as noted.

Design provisions are provided for round and multi-sided tubular shapes, I-shaped sections, channels, plates, angles, and anchor bolts above the foundation. Anchorage requirements are specified in Section 15.

C5.1

Laminated structures may be used when the fabrication process is such that adequate shear transfer between the lamina can be achieved. Their use is subject to the approval of the Owner.

5.2—DEFINITIONS

Anchor Bolt—A bolt, stud, or threaded rod used to transmit loads from the attachment into the concrete support or foundation. The end cast in concrete shall be provided with a positive anchorage device, such as forged head, nut, hooked end, or attachment to an anchor plate to resist forces on the anchor bolt.

Anchorage—The process of attaching a structural member or support to the concrete structure by means of an embedment, taking into consideration those factors that determine the load capacity of the anchorage system.

Attachment—The structural support external to the surfaces of the embedment that transmits loads to the embedment.

Compact Section—A section capable of developing the plastic moment capacity.

Ductile Anchor Connection—A connection whose resistance is controlled by the strength of the steel anchorage rather than the strength of the concrete.

Ductile Anchor Failure—A ductile failure occurs when the anchor bolts are sufficiently embedded so that failure occurs by yielding of the steel anchor bolts.

Embedment—The portion of a steel component embedded in the concrete used to transmit applied loads from the attachment to the concrete support or foundation.

Headed Anchor—A headed bolt, a headed stud, or a threaded rod with an end nut.

High-Mast Lighting Tower (HMLT)—Pole-type tower that provides lighting at heights greater than 55 ft.

Lateral-Torsional Buckling (LTB)— The buckling mode of a flexural member involving deflection normal to the plane of bending that occurs simultaneously with twist about the shear center of the cross section.

Local Flange Buckling (LFB)—Section instability due to buckling of flange or other local part of the cross section.

Multi-sided Tube—A section with generally round characteristics having eight or more sides.

Noncompact Section—A section in which the moment capacity is not permitted to exceed its yield moment.

Rectangular Tube—A square or rectangular section (four sides). Resistance checks differ from multi-sided tubes.

Retrofit Anchor Bolt—An anchor that is installed into hardened concrete.

Slender Section—A section in which the moment capacity is governed by buckling prior to reaching its yield moment.

5.3—NOTATION

A_e = effective net area (in.^{2}) (5.9.2) (5.9.3)
A_{EFF} = effective area summation (in.^{2}) (5.10.2.3)
A_g = gross area (in.^{2}) (5.9.2) (5.9.3) (5.10.2.3) (5.11.2.1.1) (5.11.2.1.2) (5.12.1)
A_n = net area (in.^{2}) (5.9.3)
A_v = shear area (in.^{2}) (5.11.2) (5.11.2.1.1) (5.11.2.1.2) (5.11.2.2)
A_w = area of the web (in.^{2}) (5.11.2.2)
a_w = ratio of two times the web area in compression due to application of major axis bending moment alone to the area of the compression flange components (5.8.3.2.4)
B = ratio (5.8.4.4) (5.12.1)
B = moment magnification factor (5.12.1)
B_x = moment magnification factor for second order effects (x axis) (5.12.1) (5.12.2)
B_y = moment magnification factor for second order effects (y axis) (5.12.1) (5.12.2)
b = element width (in.) (5.7.2) (C5.7.2) (5.7.3) (5.8.2) (5.8.3.1.2) (5.8.3.1.3) (5.10.2.3) (5.11.2.2)
b_e = element effective width (in.) (5.10.2.3)
b_f = flange width of rolled beam (in.) (5.7.3) (C5.9.3)
b_l = longer leg width (in.) (5.10.2.4)
b_s = shorter leg width (in.) (5.10.2.4)
C_b = moment gradient coefficient (5.8.3.1.3) (5.8.3.2.4) (5.8.7.2)
C_t = the torsional constant (5.11.3) (C5.11.3)
C_v = shear buckling coefficient (5.11.2.2)
C_w = warping constant (in.^{6}) (5.8.3.1.3)
c = lateral-torsional buckling section coefficient (5.8.3.1.3)
D = inside diameter of round cross-section (in.) (5.6.3) (5.6.6.1) (5.7.2) (C5.7.2) (5.8.2) (5.10.2.2) (5.11.2.1.1) (5.11.3.1.1)
D = outside distance from flat side to flat side of multi-sided tubes (in.) (5.6.2) (C5.6.2) (5.6.3) (5.6.6.1) (5.7.2) (C5.7.2) (5.8.2)
d = full nominal depth for stems of tees (in.) (5.7.3) (5.8.4.3) (5.8.4.4) (5.8.7.1) (5.8.7.2) (C5.9.3) (5.11.2.2)
d = full nominal depth for webs of rolled or formed sections (in.) (5.7.3) (5.8.4.3) (5.8.4.4) (5.8.7.1) (5.8.7.2) (C5.9.3)
E = modulus of elasticity of steel, 29,000 (ksi) (5.7.2) (5.7.3) (5.8.2) (5.8.3.1.3) (5.8.3.2.4) (5.8.4.3) (5.8.4.4) (5.8.7.1) (5.8.7.2) (5.10.2.1) (5.10.2.2) (5.10.2.3) (5.11.2.1.1) (5.11.2.2) (5.11.3.1.1) (5.12.1)
F_{cr} = critical buckling stress (ksi) (5.8.3.1.1) (5.8.3.2.4) (5.8.4.3) (5.8.7.2) (5.10.2.1) (5.10.2.3)
F_e = Euler stress, calculated in the plane of bending (ksi) (5.10.2.1)
F_L = stress defined in Table 5.7.3-1 (ksi) (5.7.3)
F_{nt} = torsional resistance (ksi) (5.11.3) (5.11.3.1.1) (5.11.3.1.2) (5.11.3.2)
F_{nv} = nominal shear resistance (ksi) (5.11.2) (5.11.2.1.1) (5.11.2.1.2) (5.11.2.2) (5.11.3.2)
F_u = specified minimum fracture stress (ksi) (5.9.2)
F_y = specified minimum yield stress (ksi) (5.7.2) (5.7.3) (5.8.2) (5.8.3.1.1) (5.8.3.1.2) (5.8.3.1.3) (C5.8.3.1.3) (5.8.3.2.1) (5.8.3.2.2) (5.8.3.2.3) (5.8.3.2.4) (5.8.4.2) (5.8.5.1) (5.8.5.2) (5.8.7.1) (5.8.7.2) (5.9.2) (5.10.2.1) (5.10.2.2) (5.11.2.1.1) (5.11.2.1.2) (5.11.2.2) (5.11.3.1.2)
f = buckling stress (ksi) (5.10.2.3)
G = elastic shear modulus (ksi) (5.8.4.4)
g = transverse center-to-center spacing (gauge) between lines of fasteners (in.) (5.9.3)
H = height of backing ring at a groove-welded tube-to-transverse-plate connection (in.) (5.6.5) (C5.6.5)
h = clear distance between the flanges for webs of rolled or built-up sections (in.) (5.7.3) (5.11.2.2)
h_c = twice the distance from center of gravity to inside face of compression flange (in.) (5.7.3) (5.8.3.2.4)
h_o = distance between flange centroids (in.) (5.8.3.1.3) (5.8.3.2.4)
h_p = twice the distance from plastic neutral axis to inside face of compression flange (in.) (5.7.3)

I = moment of inertia of the bolt group (in.^{4}) (5.8.4.4)
I_y = out-of-plane moment of inertia (in.^{4}) (5.8.3.1.3) (5.8.3.2.1) (5.8.3.2.3) (5.8.4.4)
I_{yc} = out-of-plane moment of inertia of the compression flange (in.^{4}) (5.8.3.2.1) (5.8.3.2.3)
J = torsional constant (in.^{4}) (5.8.3.1.3) (5.8.3.2.4) (5.8.4.4)
K = effective length factor (5.10.2.1) (C5.10.2.1) (5.10.2.4) (5.10.3) (5.12.1)
k_c = slenderness ratio (5.7.3)
k_v = buckling factor (5.11.2.2)
L = distance between cross-sections braced against twist or lateral displacement of the compression flange. For cantilevers braced against twist only at the support, L may conservatively be taken as the actual length (in.). (5.9.4) (C5.10.2.1) (5.10.2.4) (5.10.3) (5.12.1)
L = length of column (in.) (5.9.3) (5.9.4) (C5.10.2.1) (5.10.3) (5.10.2.4) (5.11.3.1.1) (5.12.1)
L = length of connection in the direction of loading (in.) (5.9.3) (5.9.4) (5.10.2.1) (C5.10.2.1) (5.10.2.4) (5.10.3) (5.11.3.1.1) (5.12.1)
L_b = unbraced length of member (in.) (5.8.3.1.3) (5.8.3.2.4) (5.8.4.4.) (5.8.7.1) (5.8.7.2)
L_p = lateral-torsional buckling length limit (in.) (5.8.3.1.3) (C5.8.3.1.3) (5.8.3.2.4)
L_r = lateral-torsional buckling length limit (in.) (5.8.3.1.3) (5.8.3.2.4)
L_v = distance from maximum to zero shear force (in.) (5.11.2.1.1)
L_w = length of weld (in.) (5.9.3)
M_A = absolute value of the quarter-point moment in the unbraced segment (kip-in.) (5.8.3.1.3)
M_B = absolute value of the center-point moment in the unbraced segment (kip-in.) (5.8.3.1.3)
M_C = absolute value of the third quarter-point moment in the unbraced segment (kip-in.) (5.8.3.1.3)
M_{cr} = lateral-torsional buckling moment (5.8.4.4)
M_{max} = absolute value of the maximum moment in the unbraced segment (kip-in.) (5.8.3.1.3)
M_n = nominal flexural resistance (kip-in.) (5.8.1) (5.8.2) (5.8.3.1.1) (5.8.3.1.2) (5.8.3.1.3) (5.8.3.2.2) (5.8.3.2.3) (5.8.3.2.4) (5.8.4.1) (5.8.4.2) (5.8.4.3) (5.8.4.4) (5.8.5.1) (5.8.5.2) (5.8.7.1) (5.8.7.2)
M_{nx} = nominal flexural resistance about x axis (kip-in.) (5.12.2)
M_{ny} = nominal flexural resistance about y axis (kip-in.) (5.12.2)
M_p = plastic flexural resistance (kip-in.) (5.7.3) (5.8.2) (5.8.3.1.1) (5.8.3.1.2) (5.8.3.2.1) (5.8.3.2.3) (5.8.4.1) (5.8.4.2) (5.8.5.1) (5.8.5.2) (5.8.7.1) (5.8.7.2)
M_{px} = plastic flexural resistance about x axis (kip-in.) (5.12.2)
M_{py} = plastic flexural resistance about y axis (kip-in.) (5.12.2)
M_r = factored flexural resistance (kip-in.) (5.8.1) (5.8.3.2.4) (5.12.1)
M_{rx} = factored flexural resistance about x axis (kip-in.) (5.12.1) (5.12.2)
M_{ry} = factored flexural resistance about y axis (kip-in.) (5.12.1) (5.12.2)
M_u = factored bending moment (kip-in.) (5.12.1)
M_{ux} = factored bending moment about x axis (kip-in.) (5.12.1) (5.12.2)
M_{ux}^* = factored bending moment skewed diagonal loading about x axis (kip-in.) (5.12.1) (5.12.2)
M_{uy} = factored bending moment about y axis (kip-in.) (5.12.1) (5.12.2)
M_{uy}^* = factored bending moment from skewed diagonal loading about y axis (kip-in.) (5.12.1) (5.12.2)
M_y = yield moment (kip-in.) (5.7.3) (5.8.4.1) (5.8.4.2) (5.8.7.1) (5.8.7.2)
M_{yc} = yield moment based upon compression yielding (kip-in.) (5.8.3.2.1) (5.8.3.2.2) (5.8.3.2.4)
M_{yt} = yield moment based upon tension yielding (kip-in.) (5.8.3.2.3)
n = number of sides on a multi-sided section (5.6.2) (5.7.2) (C5.7.2)
P_e = Euler elastic buckling capacity (kip) (5.12.1)
P_{nc} = minimum nominal compressive strength (kip) (5.10.1) (5.10.2.1)
P_{nt} = minimum nominal compressive strength (kip) (5.12.1)
P_{nu} = minimum nominal rupture strength on the effective net area (kip) (5.9.1) (5.9.2)
P_{ny} = minimum nominal tensile strength from limit states of yield on gross sectional area (kip) (5.9.1) (5.9.2)

P_r = factored nominal axial resistance (kip) (5.12.1)

P_{rc} = minimum nominal compressive strength (kip) (5.10.1)

P_{rt} = minimum nominal tension strength (kip) (5.9.1)

P_u = factored axial load (kip) (5.12.1)

Q = local buckling adjustment factor (5.10.2.1) (C5.10.2.1) (5.10.2.2) (5.10.2.3)

R = root gap at a groove-welded tube-to-transverse-plate connection (in.) (5.6.5)

R_n = general nominal resistance (5.13)

R_{nr} = factored nominal resistance of cables (kip) (5.13)

R_{pc} = moment ratio for compression flange yielding (5.8.3.2.1) (5.8.3.2.2) (5.8.3.2.4)

R_{pt} = moment ratio for tension flange yielding (5.8.3.2.3)

R_{rt} = factored tensile resistance of cables (kip) (5.13)

r = governing radius of gyration (in.) (5.9.4) (5.10.2.1) (C5.10.2.1) (5.10.2.4) (5.10.3) (5.12.1) (5.13)

r_b = inside bend radius of a plate (in.) (C5.7.2)

r_t = radius of gyration of a section comprising the compression flange plus $^1/_3$ of the compression web area, taken about an axis in the plane of the web (in.) (5.8.3.2.4)

r_{ts} = radius of gyration on effective section (in.) (5.8.3.1.3)

r_x = radius of gyration about the axis parallel to the connected leg (in.) (5.10.2.4)

r_y = radius of gyration about the y-axis (in.) (5.8.3.1.3) (C5.8.3.1.3)

r_z = radius of gyration about the minor principle axis (in.) (5.10.2.4)

S_x = elastic section modulus (in.^{3}) (5.8.3.1.2) (5.8.3.1.3) (C5.8.3.1.3) (5.8.3.2.1) (5.8.3.2.4) (5.8.4.3) (5.8.7.2) (C5.12.2)

S_{xc} = elastic section modulus for outer compression fibers (in.^{3}) (5.7.3) (5.8.3.2.1) (5.8.3.2.2) (5.8.3.2.3) (5.8.3.2.4)

S_{xt} = elastic section modulus for outer tension fibers (in.^{3}) (5.7.3) (5.8.3.2.1) (5.8.3.2.2) (5.8.3.2.3)

S_y = elastic section modulus (in.^{3}) (5.8.5.1) (5.8.5.2)

s = longitudinal center-to-center spacing (pitch) of any two consecutive holes (in.)

T_n = nominal torsional resistance (kip-in.) (5.11.1) (5.11.3)

T_r = factored torsional resistance (kip-in.) (5.11.1)

T_u = factored torque (kip-in.) (5.12.1)

t = wall thickness or thickness of element (in.) (5.6.5) (C5.6.5) (5.6.6.1) (5.7.2) (C5.7.2) (5.7.3) (5.8.2) (5.8.3.1.2) (5.8.7.1) (5.8.7.2) (5.10.2.2) (5.10.2.3) (5.11.2.1.1) (5.11.2.2) (5.11.3.1.1)

t_w = thickness of web (in.) (5.7.3) (5.8.4.3) (5.11.2.2)

U = shear lag coefficient (5.9.3) (C5.9.3)

V_n = nominal shear resistance (kip) (5.11.1) (5.11.2)

V_r = factored shear resistance (kip) (5.11.1) (5.12.1)

V_{rx} = factored shear resistance parallel to the x axis (kip) (5.12.1)

V_{ry} = factored shear resistance parallel to the y axis (kip) (5.12.1)

V_u = factored shear (kips) (5.12.1)

V_{ux} = factored shear parallel to the x axis (5.12.1)

V_{uy} = factored shear parallel to the y axis (5.12.1)

W = clear width of reinforced and unreinforced holes and cutouts (in.) (5.6.6.1)

w = width of plate (distance between welds) (in.) (5.9.3)

x = connection eccentricity (in.) (5.9.3)

$\bar{x}$ = connection eccentricity (in.) (5.9.3)

Z = plastic section modulus (5.8.7.1)

Z_x = plastic section modulus about the x axis (in.^{3}) (5.8.2) (5.8.3.1.1) (5.8.3.1.3) (C5.8.3.1.3) (5.8.3.2.1) (5.8.4.3) (C5.12.2)

Z_y = plastic section modulus about the y axis (in.^{3}) (5.8.5.1)

λ = width–thickness ratio of the element (5.8.2) (5.8.3.1.2) (C5.8.3.2) (5.8.3.2.1) (5.8.3.2.2) (5.8.3.2.3) (5.8.4.2) (5.8.4.3) (5.8.5.2) (5.10.2.1) (5.10.2.2) (5.10.2.3) (5.12.2)

λ_{max} = maximum width–thickness ratio (C5.7.1) (5.7.2) (5.7.3) (C5.11.3.1.2) (5.12.2)

λ_p = width–thickness ratio at the compact limit (5.7.1) (C5.7.1) (5.7.2) (5.7.3) (5.8.2) (5.8.3.1.2) (5.8.3.2.1) (5.8.3.2.2) (5.8.3.2.3) (5.8.4.2) (5.8.4.3) (5.8.5.2)

λ_r = width–thickness ratio at the noncompact limit (5.7.1) (C5.7.1) (5.7.2) (5.7.3) (5.8.2) (5.8.3.1.2) (C5.8.3.2) (5.8.3.2.1) (5.8.3.2.3) (5.8.5.2) (5.10.2.1) (5.10.2.2) (5.10.2.3) (5.12.2)

θ = angle of the sound beam for ultrasonic inspection of groove welds (degrees) (5.5.3.2) (5.6.5)

ϕ_c = resistance factor for compression (5.5.3.2) (5.10.1) (5.12.1)

ϕ_f = resistance factor for flexure (5.5.3.2) (5.8.1) (5.12.2)

ϕ_{rt} = resistance factor for cables (5.13)

ϕ_T = resistance factor for torsion (5.5.3.2)

ϕ_t = resistance factor for bolt tension (5.11.1) (5.12.1)

ϕ_u = resistance factor for tension fracture (5.5.3.2) (5.9.1)

ϕ_v = resistance factor for shear (5.5.3.2) (5.11.1)

ϕ_y = resistance factor for tension yield (5.5.3.2) (5.9.1)

5.4—MATERIAL

Grades of steel listed in the *AASHTO LRFD Bridge Design Specifications* (LRFD Design) are applicable for welded structural supports for highway signs, luminaires, and traffic signals.

C5.4

Steel other than that listed may be used with permission from the Owner.

Typical steel materials used in structural supports for highway signs, luminaires, and traffic signals are:

- ASTM A595 Grade A, B, and C
- ASTM A572 Grade 42, 50, 55, 60, and 65
- ASTM A1011
- ASTM F1554 Grade 36, 55, and 105 Anchor Bolts

For steels not generally addressed by LRFD Design, but having a specified yield strength acceptable to the Owner, the LRFD limit state design criteria shall be derived by applying the general equations given in LRFD Design except as indicated by this Section.

Generally, the Specification indicated in this Section applies.

All steels greater than 0.5 in. in thickness, used for structural supports for highway signs, luminaires, and traffic signals, that are main load carrying tension members shall meet the current Charpy V-Notch impact requirements in LRFD Design.

Although the structural supports addressed by these Specifications are not subjected to high-impact loadings, steel members greater than 0.5 in. in thickness should meet a general notch toughness requirement to avoid brittle fracture. The non-fracture critical values may be used.

5.5—DESIGN LIMIT STATES

5.5.1—General

Structural components and connections shall be proportioned to satisfy the requirements at strength, extreme event, service, and fatigue limit states.

5.5.2—Service Limit State

General service requirements are provided in Section 10.

5.5.3—Strength Limit State

5.5.3.1—General

Strength and stability shall be considered using the applicable strength load combinations specified in Table 3.4-1.

C5.5.3.1

NCHRP project 10-80 developed specific LRFD load and resistance factors using ASCE/SEI 07-2010 loading.

5.5.3.2—Resistance Factors

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

- Flexure $\phi_f = 0.90$
- Shear $\phi_v = 0.90$
- Torsion $\phi_T = 0.95$
- Axial compression, $\phi_c = 0.90$
- Tension, fracture in net section $\phi_u = 0.75$
- Tension, yielding in gross section $\phi_y = 0.90$

C5.5.3.2

NCHRP Project Report 796 determined resistance factors specifically for signs, luminaires, and traffic signal supports and these may differ from other specifications. (Puckett et al., 2014)

5.5.4—Extreme Limit State

All applicable load combinations in Table 3.4-1 for the extreme event limit state shall be investigated.

The resistance factors for the extreme event shall be as defined in the strength limit state in Article 5.5.3.

C5.5.4

The ASCE 7-10 wind maps were generated using a wind load factor of unity. These specifications use a similar approach and therefore include wind in combination with other loads to be addressed as an extreme event.

5.5.5—Fatigue Limit State

Components and details shall be investigated for fatigue as specified in Section 11.

5.6—GENERAL DIMENSIONS AND DETAILS

5.6.1—Minimum Thickness of Materials

The minimum thickness of material for main supporting members of steel truss-type supports shall be 0.1793 in. For secondary members, such as bracing and truss webs, the minimum thickness shall be 0.125 in. The minimum thickness of material for all members of pole-type supports and truss-type luminaire arms shall be 0.125 in. These limits may be reduced no more than ten percent for material designated by gauge numbers.

Steel supports for small roadside signs may be less than 0.125 in. in thickness.

C5.6.1

Main members are those that are strictly necessary to ensure integrity of a structural system. Secondary members are those that are provided for redundancy of the system and stability of components and members. Minimum thickness requirements may be based on service considerations such as corrosion resistance as specified by the Owner. The thickness 0.1793 in. is associated with 7 gauge sheet steel material.

Supports without an external breakaway mechanism that have thicknesses less than 0.125 in. have shown good safety characteristics in that they readily fail under vehicle impact, with little damage to the vehicle or injury to the occupants. These thinner supports should be used on those installations considered to have a relatively short life expectancy such as small roadside signs.

5.6.2—Minimum Number of Sides

Tubular structures shall be of round or multi-sided cross-section. Cross-sections with concave external surface, such as “fluted” cross-sections, are not covered by the provisions for multi-sided cross-sections and

C5.6.2

Fatigue cracking in multi-sided tube-to-transverse-plate connections initiates at the bend corners and progresses towards the flat face between the corners. Research has demonstrated the existence of high stress shall not be used without approval of the Owner. Multi-sided tubular sections shall have a minimum number of sides as stated in the following equation, and a minimum internal bend radius of five times the tube wall thickness or 1 in., whichever is larger.

$$n \geq \sqrt{5D} \tag{5.6.2-1}$$

where:

D = outside distance from flat side to flat side of multi-sided tubes (in.), and
n = greater than or equal to the square root of $5D$ or 8, whichever is larger.

concentration at the bend corners, which caused crack initiation early during the fatigue tests in eight-sided tubes with sharp bend radii (Roy et al., 2011).

Compared to a round tube of similar size, welded connections in multi-sided tubes with fewer sides and internal bend radii less than 1 in. exhibited significantly less fatigue resistance. Increasing the number of sides and/or increasing the internal bend radius can improve fatigue performance of multi-sided sections (Roy et al., 2011).

The requirement of minimum number of sides for multi-sided tubes was derived considering a maximum $^1/_2$ in. radial distance between the multi-sided tube and its inscribed circle at a corner. The $^1/_2$ in. distance was chosen based on the performance of specimens that were fatigue tested in the laboratory. For flat-to-flat distance D of multi-sided cross-sections, the minimum number of sides is conservatively provided as:

D up to 13 in.

8 sides (octagonal)

D greater than 13 in. and up to 28 in.

12 sides (dodecagonal)

D greater than 28 in. and up to 50 in.

16 sides (hexadecagonal)

Although there is no clear consensus among the research community, multi-sided galvanized tubes employing very sharp bend radii can be susceptible to strainage embrittlement, cold-worked embrittlement, and hydrogen embrittlement leading to early fatigue cracking in service. A minimum bend radius of five times the tube wall thickness can mitigate the possibility of such embrittlement.

Square or rectangular sections are susceptible to early fatigue cracking leading to poor fatigue performance. These sections should not be used for highway sign, signal, and high-level luminaire support structures. (Dexter and Ricker, 2002)

Fluted cross sections and other complex hollow sections present a variety of issues that may affect the design and performance of structures. Owner approval of poles or mast arms using fluted and other complex hollow sections should require verification of the materials, design methods, detailing requirements, fabrication methods, and fatigue performance by analysis and testing of components and systems.

5.6.3—Transverse Plate Thickness

The base plate thickness shall be considered in the design of tube-to-transverse-plate connections. In addition, for arms or pole bases of supports that are designed according to Section 11, the minimum plate thickness shall be as provided in Table 5.6.3-1.

C5.6.3

Experimental and analytical studies (Koenigs et al., 2003; Hall, 2005; Warpinski, 2006; Ocel et al., 2006; Roy et al., 2011; Stam et al., 2011) demonstrated that the fatigue resistance of tube-to-transverse-plate connections is a function of the relative flexibility of the tube and the transverse plate. Transverse plate flexibility has a major impact on stress amplification in the tube wall adjacent

Table 5.6.3-1—Minimum Transverse Plate Thickness for Fatigue

Section Diameter or depth D, in.	Minimum Plate Thickness, in.
$D \leq 8$	1.5
$D > 8$	2.0

to the weld toe. Increasing the transverse plate thickness is the most cost-effective means of reducing the flexibility of the transverse plate and increasing the connection fatigue resistance. In-service fatigue cracking in tube-to-transverse-plate connections often occurred where relatively thin plates were used along with a few discrete fasteners.

Reducing the opening size in the transverse plate and/or increasing the number of fasteners are other cost-effective means of reducing the flexibility of the transverse plate and increasing the fatigue resistance of tube-to-transverse-plate connections. In laboratory tests (Roy et al., 2011), groove-welded tube-to-transverse-plate connections exhibited significantly better fatigue resistance compared to fillet-welded connections in identical structures, because a smaller opening could be used in the transverse plate.

Fluted cross sections and other complex hollow sections present a variety of issues that may affect the design and performance of structures. Owner approval of poles or mast arms using fluted and other complex hollow sections should require verification of the materials, design methods, detailing requirements, fabrication methods, and fatigue performance by analysis and testing of components and systems.

5.6.4—Stiffened Base Connection

In stiffened fillet-welded tube-to-transverse-plate connections (socket connections) only tapered stiffeners having a termination angle of 15 degrees on the tube shall be used.

The minimum height of stiffeners shall be 12 in. At least eight stiffeners shall be used equally spaced around the tube wall. The stiffener spacing shall not exceed 16 in.

When stiffened fillet-welded tube-to-transverse-plate connections are used, the minimum thickness of the tube wall shall be 0.25 in.

The ratio of the stiffener thickness to the tube-wall thicknesses shall not exceed 1.25.

Stiffeners having a transition radius shall not be used.

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

C5.6.4

In support structures employing larger diameter and thicker tubes, optimized stiffened tube-to-transverse-plate fillet-welded connections can provide a cost-effective design compared to an increased transverse plate thickness. Parametric studies (Roy et al., 2011) demonstrated that the fatigue performance of a stiffened connection is a function of the geometric parameters of the connection: the tube thickness, the transverse plate thickness, the stiffener shape and size (thickness, height, and angle), and the number of stiffeners (or stiffener spacing). A large stiffener thickness relative to the tube can attract more stress into the stiffeners and can increase distortion of the tube. By contrast, relatively thin stiffeners can reduce distortion of the tube but fail to sufficiently reduce the stress at the fillet-weld and can cause fatigue cracking through the throat of the stiffener-to-transverse plate weld.

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

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

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

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

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

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

5.6.5—Backing Ring

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

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

where:

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

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

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

t = the tube wall thickness (in.).

For tube-to-transverse-plate connection employing an external collar, the tube thickness for the above equation shall include the thickness of the collar and the tube.

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

C5.6.5

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

However, when the backing ring is welded to the tube at the top, this weld interferes with the ultrasonic inspection of the groove weld by allowing the sound wave to travel from the outside of the shaft through the weld into the backing ring. The sound wave then gets trapped in the backing ring and does not reach the groove weld. For a successful inspection, the weld at the top of the backing ring should be above the centerline of the probe. According to AWS D1.1, the ultrasonic beam should bounce at least once to the area of inspection, which creates a full “V” signal. From experience, a shallow beam angle such as 70 degrees produces the best results. Thus, for thicker tubes with a 45-degree bevel and a root gap, the probe placement gets higher and therefore the backing ring needs to be taller.

The backing ring heights for different tube thicknesses are tabulated in the following table for a root gap of $^1/_4$ in. and an angle of ultrasonic beam of 70°.

Table C5.6.5-1—Required Backing Ring Height

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

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

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

5.6.6—Holes and Cutouts

5.6.6.1—Unreinforced and Reinforced Holes and Cutouts

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

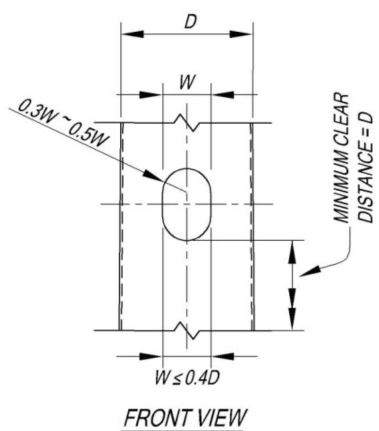

Figure 5.6.6.1-1—Details of Unreinforced Holes and Cutouts

C5.6.6.1

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

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

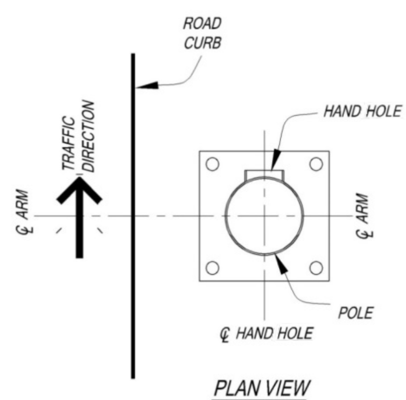

Figure C5.6.6.1-1—Recommended Orientation of a Hand Hole

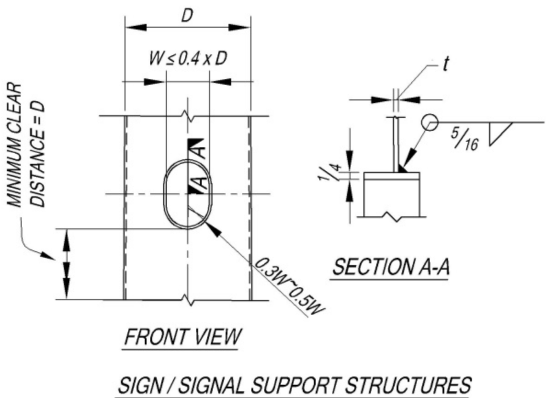

Figure 5.6.6.1-2—Details of Reinforced Holes and Cutouts

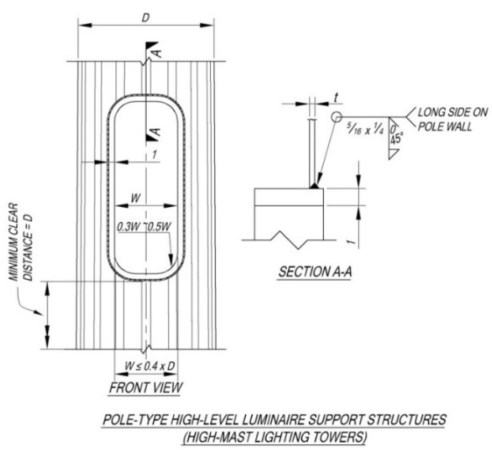

Figure 5.6.6.1-3—Details of Reinforced Holes and Cutouts for High-Mast Poles

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

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

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

5.6.7—Mast-Arm-to-Pole Connections

Mast-arm-to-pole connections employing fillet-welded gusseted box or ring-stiffened box have been shown to be effective and fatigue resistant. Connections validated with testing may be used and are encouraged.

C5.6.7

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

In-service fatigue cracking of these connections has been reported. Fatigue testing has shown the advantage of ring stiffeners that completely encircle a pole relative to a built-up box connection. For built-up box connections, it is recommended that the width of the box be at least the same as the diameter of the column (i.e., the sides of the box are tangent to the sides of the column).

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

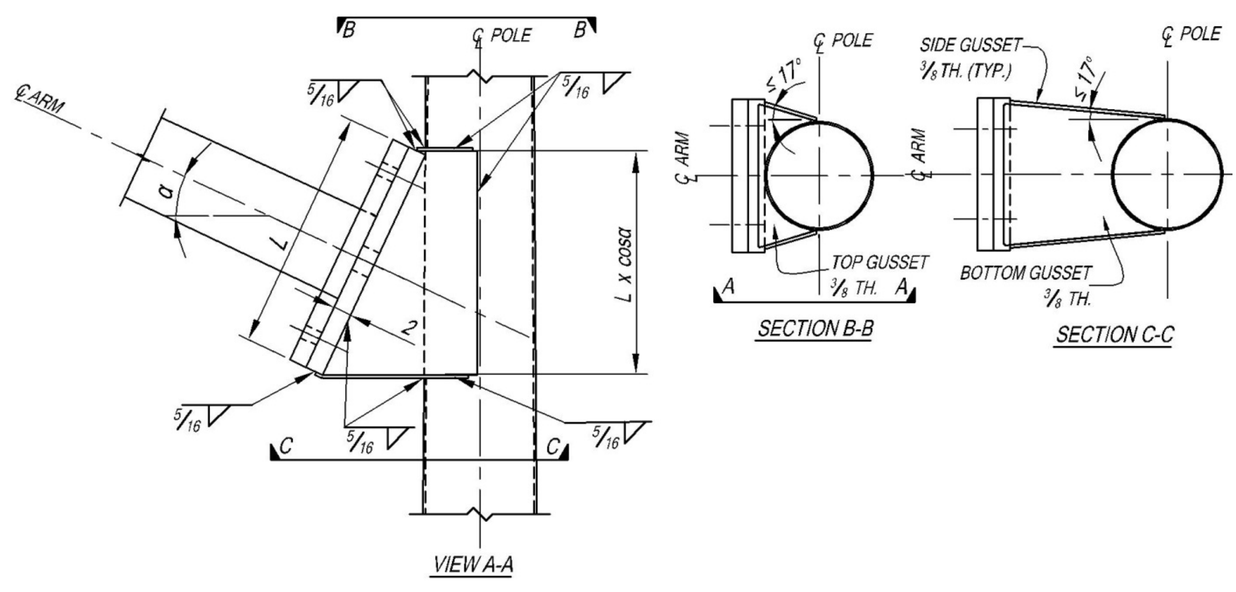

Figure C5.6.7-1—Details of Fillet-Welded Gusseted Box Connections

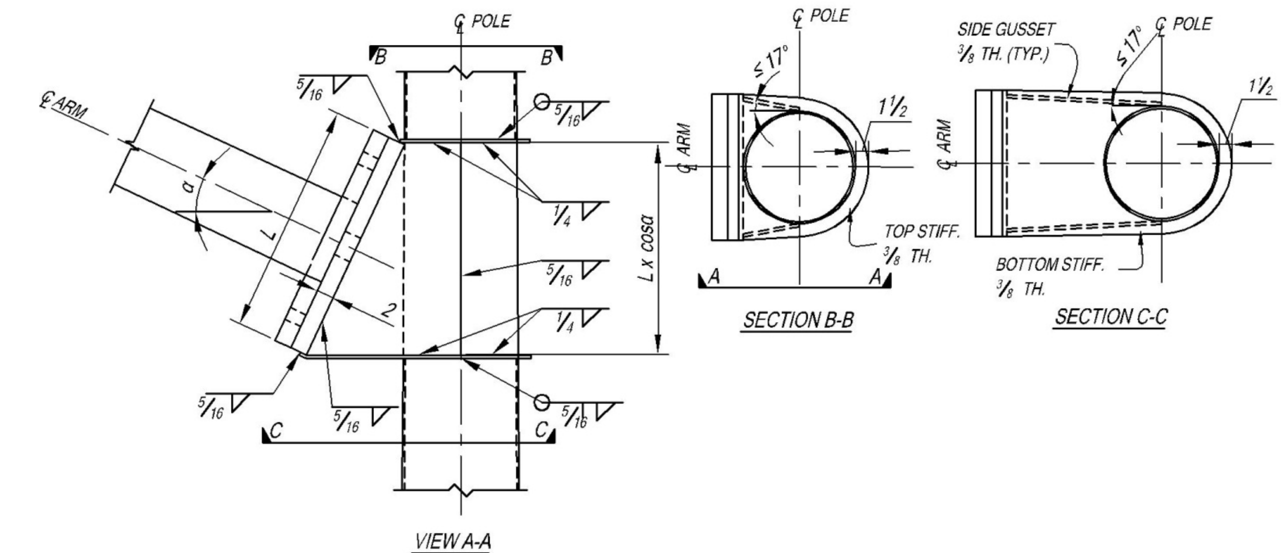

Figure C5.6.7-2—Details of Fillet-Welded Ring-Stiffened Box Connections

5.6.8—Slip Type Field Splice

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

C5.6.8

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

5.7—SECTION CLASSIFICATION FOR LOCAL BUCKLING

5.7.1—Classification of Steel Sections

Steel sections are classified as compact, noncompact, and slender element sections. For a section to qualify as compact or noncompact, the width–thickness ratios of compression elements must not exceed the applicable corresponding limiting λ_p or λ_r values given in Tables 5.7.2-1 and 5.7.3-1, respectively. If the width–thickness ratios of any compression element section exceed the noncompact limiting value, λ_r, the section is classified as a slender element section.

C5.7.1

Cross section elements with width–thickness ratios greater than the limits in Tables 5.7.2-1 and 5.7.3-1 may experience local buckling. Flexural members may be subject to local buckling when the width–thickness ratio exceeds λ_p. Members in compression may be subject to local buckling when the width–thickness ratio exceeds λ_r.

5.7.2—Width–Thickness Ratios for Round and Multi-sided Tubular Sections

The limiting diameter–thickness, *D/t*, ratios for round sections and width–thickness, *b/t*, ratios for multi-sided tubular sections are given in Table 5.7.2-1.

For multi-sided tubular sections, the effective width, *b*, is the inside distance between intersection points of the flat sides less:

C5.7.2

The equation for the element width of multi-sided tubular sections may be calculated as:

$$b = \tan\left(\frac{180}{n}\right)\left[D' - 2t - \text{minimum}\left(2r_b, 8t\right)\right] \quad \text{(C5.7.2-1)}$$

$$\tan\left(\frac{180}{n}\right) \quad (5.7.2\text{-}1)$$

times the minimum of the inside bend radius or 4*t*, on each side. If the bend radius is not known, the element width, *b*, may be calculated as the inside width between intersection points of the flat sides less:

$$(3t)\tan\left(\frac{180}{n}\right) \quad (5.7.2\text{-}2)$$

where:

D' = the outside distance from flat side to flat side of multi-sided tubes and 180/*n* is in degrees.

Table 5.7.2-1—Width–Thickness Ratios for Round and Multi-Sided Tubular Sections

Shape	Ratio	λ_P	λ_r	λ_{max}
Round	*D/t*	$0.07\frac{E}{F_y}$	$0.11\frac{E}{F_y}$ *(compression)* $0.31\frac{E}{F_y}$ *(flexure)*	$0.45\frac{E}{F_y}$
Hexadecagonal (16)	*b/t*	$1.12\sqrt{\frac{E}{F_y}}$	$1.26\sqrt{\frac{E}{F_y}}$	$2.14\sqrt{\frac{E}{F_y}}$
Dodecagonal (12)	*b/t*	$1.12\sqrt{\frac{E}{F_y}}$	$1.41\sqrt{\frac{E}{F_y}}$	$2.14\sqrt{\frac{E}{F_y}}$
Octagonal (8)	*b/t*	$1.12\sqrt{\frac{E}{F_y}}$	$1.53\sqrt{\frac{E}{F_y}}$	$2.14\sqrt{\frac{E}{F_y}}$
Flanges of Square/Rectangle	*b/t*	$1.12\sqrt{\frac{E}{F_y}}$	$1.53\sqrt{\frac{E}{F_y}}$	$2.14\sqrt{\frac{E}{F_y}}$

5.7.3—Width–Thickness Ratios for Compression Plate Elements

Limiting width–thickness ratios for nontubular shapes are given in Table 5.7.3-1.

Plate elements are considered unstiffened or stiffened, depending on whether the element is supported along one or two edges, parallel to the direction of the compression force, respectively.

For unstiffened elements, which are supported along one edge parallel to the direction of the compression force, the width shall be taken as:

- b = half the full nominal flange width, b_f, for flanges of I-shaped members and tees (in.),
- b = the full nominal dimension for legs of angles and flanges of channels (in.), and
- d = the full nominal depth for stems of tees (in.).

For stiffened elements, which are supported along two edges parallel to the direction of the compression force, the width shall be taken as:

C5.7.3

For cross sections element not listed in Table 5.7.3-1, the AISC *Manual of Steel Construction* (2011) should be used to determine the limiting width–thickness ratios. The maximum width–thickness ratio would be the applicable λ_r limit.

- h = the clear distance between flanges for webs of rolled or built-up sections (in.),
- d = the full nominal depth for webs of rolled or built-up sections (in.),
- h_c = twice the distance from center of gravity to inside face of compression flange (in.), and
- h_p = twice the distance from plastic neutral axis to inside face of compression flange (in.).

The unsupported width of such elements shall be taken as the distance between the nearest lines of fasteners or welds, or between the roots of the flanges in the case of rolled sections, or as otherwise specified in this Article.

Table 5.7.3-1—Width–Thickness Ratios for Nontubular Sections

Description	Width/Thickness	λ_P	$\lambda_r = \lambda_{max}$
Flexure of flanges of rolled I-shapes, tees, and channels	b/t	$0.38\sqrt{\frac{E}{F_y}}$	$1.0\sqrt{\frac{E}{F_y}}$
Uniform compression in I-shape flanges and channel flanges	b/t	N/A	$0.56\sqrt{\frac{E}{F_y}}$
Uniform compression of single angles and double angles and all other unstiffened elements	b/t	N/A	$0.45\sqrt{\frac{E}{F_y}}$
Uniform compression in stems of tees	d/t	N/A	$0.75\sqrt{\frac{E}{F_y}}$
Uniform compression of webs of I-shapes and all other stiffened elements	h/t_w, b/t	N/A	$1.49\sqrt{\frac{E}{F_y}}$
Flexure in legs of angles	b/t	$0.54\sqrt{\frac{E}{F_y}}$	$0.91\sqrt{\frac{E}{F_y}}$
Flexure in stems of tees	d/t	$0.84\sqrt{\frac{E}{F_y}}$	$1.03\sqrt{\frac{E}{F_y}}$
Flexure in webs of doubly symmetrical I-shapes	h/t_w	$3.76\sqrt{\frac{E}{F_y}}$	$5.70\sqrt{\frac{E}{F_y}}$
Flexure in webs of singly symmetrical I-shapes and built-up I-shapes	h_c/t_w	$\dfrac{\frac{h_c}{h_p}\sqrt{\frac{E}{F_y}}}{\left(0.54\frac{M_p}{M_y}-0.09\right)} \le \lambda_r$	$5.70\sqrt{\frac{E}{F_y}}$
Flexure in flanges of singly symmetrical I-shapes and built-up I-shapes	b/t	$0.38\sqrt{\frac{E}{F_y}}$	$0.95\sqrt{\frac{k_cE}{F_L}}$

Notes:

h_c = twice the distance from center of gravity to inside face of compression flange

h_p = twice the distance from plastic neutral axis to inside face of compression flange

$k_c = 4/\sqrt{h/t_w} \le 0.76$

$F_L = 0.7F_y$ when $S_{xt}/S_{xc} \ge 0.7$; $F_L = F_y\, S_{xt}/S_{xc} \ge 0.5$ when $S_{xt}/S_{xc} \le 0.7$

5.7.4—Slender Element Sections

Except as allowed for round and multi-sided tubular sections, compression plate elements that exceed the noncompact limit specified in Table 5.7.3-1 shall not be permitted.

5.8—COMPONENTS IN FLEXURE

5.8.1—General

The provisions of this article apply to flexure of rolled open, tubular, and built-up plate sections.

The factored flexural resistance, M_r, shall be:

$$M_r = \phi_f M_n \qquad (5.8.1\text{-}1)$$

where:

M_n = nominal flexural resistance, and

ϕ_f = resistance factor as specified in Article 5.5.3.2.

The nominal resistance is determined by Articles 5.8.2 through 5.8.7 as applicable.

C5.8.1

NCHRP Report 796 used previous research to develop LRFD nominal strength equations for typical sign, luminaire, and signal supports. Past allowable stress design equations were modified to limit state design equations for tubular members. AISC (2011) design equations have been incorporated for typical structures. For structures not addressed in these Specifications, other resources should be considered such as AISC (2011) and LRFD Design.

5.8.2—Nominal Bending Strength for Round and Multi-Sided Tubular Members

For round and multi-sided tubular members that have compact, noncompact, and slender element sections as defined in Table 5.7.2-1, the nominal bending strength, M_n, shall be computed according to Table 5.8.2-1.

The nominal bending strength for multi-sided tubes shall not exceed the nominal bending strength for round tubes of equivalent diameter. The equivalent diameter for a multi-sided tube shall be the outside distance between parallel sides.

Table 5.8.2-1—Nominal Bending Strength, M_n, for Tubular Members

Shape	Compact	NonCompact	Slender
	$\lambda \le \lambda_p$	$\lambda_p < \lambda \le \lambda_r$	$\lambda > \lambda_r$
Round	$M_n = M_p = Z_x F_y$	$M_n = M_p\left[0.77 + \dfrac{0.016(E/F_y)}{D/t}\right]$	$M_n = M_p\left[\dfrac{0.25(E/F_y)}{D/t}\right]$
Hexadecagonal	$M_n = M_p = Z_x F_y$	$M_n = M_p\left[2.59 - \dfrac{1.43(b/t)}{\sqrt{E/F_y}}\right]$	$M_n = M_p\left[1.12 - \dfrac{0.26(b/t)}{\sqrt{E/F_y}}\right]$
Dodocagonal	$M_n = M_p = Z_x F_y$	$M_n = M_p\left[1.77 - \dfrac{0.69(b/t)}{\sqrt{E/F_y}}\right]$	$M_n = M_p\left[1.15 - \dfrac{0.25(b/t)}{\sqrt{E/F_y}}\right]$
Octagonal	$M_n = M_p = Z_x F_y$	$M_n = M_p\left[1.50 - \dfrac{0.45(b/t)}{\sqrt{E/F_y}}\right]$	$M_n = M_p\left[1.14 - \dfrac{0.22(b/t)}{\sqrt{E/F_y}}\right]$
Square and Rectangular	$M_n = M_p = Z_x F_y$	$M_n = M_p\left[1.37 - \dfrac{0.33(b/t)}{\sqrt{E/F_y}}\right]$	$M_n = M_p\left[1.23 - \dfrac{0.23(b/t)}{\sqrt{E/F_y}}\right]$

5.8.3—Nominal Bending Strength for Flanged I-Shaped Members, Channels, and Rectangular Tubes

This article applies to singly or doubly symmetric beams loaded in the plane of symmetry. It also applies to channels loaded in a plane passing through the shear center parallel to the web or restrained against twisting at load points and points of support.

C5.8.3

Rectangular tubes are torsionally stiff; however, in an application where the member unbraced length is long, lateral-torsional buckling should be checked.

5.8.3.1—Doubly Symmetric I-Shape, Channel, and Rectangular Tubes Members Bent about Strong Axis with Compact Webs

The nominal bending strength shall be the lower value obtained according to the limit states of plastic moment, lateral-torsional buckling (LTB), and flange local buckling (FLB).

5.8.3.1.1—Plastic Moment

$$M_n = M_p = Z_x F_y \qquad (5.8.3.1.1\text{-}1)$$

where:

Z_x = plastic section modulus (in.^{3}), and

F_y = yield stress (ksi).

C5.8.3.1.1

Web buckling is not included because *d*/*t* ratios do not exceed λ_r. Elastic element buckling limits ($\lambda > \lambda_r$) do not need to be considered, except for rectangular tubes, because slender elements are not allowed.

5.8.3.1.2—Flange Local Buckling

If $\lambda \le \lambda_p$ FLB does not apply.

If $\lambda > \lambda_p$

$$M_n = M_p - \left(M_p - 0.7F_y S_x\right)\left(\frac{\lambda - \lambda_p}{\lambda_r - \lambda_p}\right) \qquad (5.8.3.1.2\text{-}1)$$

where:

S_x = section modulus, (in.^{3}),

F_y = yield stress, (ksi), and

λ_p and λ_r are defined in Table 5.7.3-1.

For slender rectangular tubes where $\lambda > \lambda_p$, M_n is determined from Table 5.8.2-1 using *b/t* of the flange.

5.8.3.1.3—Lateral-Torsional Buckling

If $L_b \le L_p$

$$M_n = M_p = Z_x F_y \quad (5.8.3.1.3\text{-}1)$$

If $L_p \le L_b \le L_r$

$$M_n = C_b \left[M_p - \left(M_p - 0.7 F_y S_x \right) \left(\frac{L_b - L_p}{L_r - L_p} \right) \right] \le M_p \quad (5.8.3.1.3\text{-}2)$$

If $L_b > L_r$

$$M_n = F_{cr} S_x \le M_p \quad (5.8.3.1.3\text{-}3)$$

$$F_{cr} = \frac{C_b \pi^2 E}{\left(\frac{L_b}{r_{ts}} \right)^2} \sqrt{1 + 0.078 \frac{Jc}{S_x h_o} \left(\frac{L_b}{r_{ts}} \right)^2} \quad (5.8.3.1.3\text{-}4)$$

$$L_p = 1.76 r_y \sqrt{\frac{E}{F_y}} \quad (5.8.3.1.3\text{-}5)$$

$$L_r = 1.95 r_{ts} \frac{E}{0.7 F_y} \sqrt{\frac{Jc}{S_x h_o} + \sqrt{\left(\frac{Jc}{S_x h_o} \right)^2 + 6.76 \left(\frac{0.7 F_y}{E} \right)^2}} \quad (5.8.3.1.3\text{-}6)$$

or conservatively, L_r can be taken as:

$$L_r = \pi r_{ts} \sqrt{\frac{E}{0.7 F_y}} \quad (5.8.3.1.3\text{-}7)$$

where:

$$r_{ts} = \sqrt{\frac{\sqrt{I_y C_w}}{S_x}} \quad (5.8.3.1.3\text{-}8)$$

C5.8.3.1.3

AISC (2011) does not provide formulas for lateral-torsional buckling resistance for HSS tubes. The reason is that for practical application in buildings, a deflection control will control before any LTB develops. However, for structural supports for signs, luminaires, and traffic signals, this may not be the case and the equations below may be used (see AISC (1993) Article F1.2a).

$$L_p = \frac{3750 r_y}{F_y Z_x} \sqrt{JA} \quad (C5.8.3.1.3\text{-}1)$$

$$L_r = \frac{57000 r_y \sqrt{JA}}{F_y S_x} \quad (C5.8.3.1.3\text{-}2)$$

$c = 1$ for doubly symmetric I-shapes

$$c = \frac{h_o}{2}\sqrt{\frac{I_y}{C_w}} \text{ for channels} \tag{5.8.3.1.3-9}$$

where:

L_b = length between points that are either braced against lateral displacement of compression flange or braced against twist of the cross section (in.),

L_p = the limiting laterally unbraced length for the limit state of yielding (in.),

L_r = the limiting laterally unbraced length for the limit state of inelastic LTB (in.),

h_o = the distance between flange centroids (in.),

C_w = warping constant (in.^{6}),

I_y = out-of-plane moment of inertia (in.^{4}),

S_x = elastic section modulus taken about the x axis (in.^{3}),

J = the torsional constant (in.^{4}),

F_y = yield stress (ksi),

Z_x = plastic section modulus taken about the x axis (in.^{3}), and

E = elastic modulus (ksi).

The moment gradient adjustment factor, C_b, is:

$$C_b = \frac{12.5M_{\max}}{2.5M_{\max} + 3M_A + 4M_B + 3M_C} \tag{5.8.3.1.3-10}$$

where:

C_b can conservatively be taken as 1.0 and should be 1.0 for cantilever beams, and

M_{max} = absolute value of the maximum moment in the unbraced segment (kip-in.),

M_A = absolute value of the moment in the one-quarter point of the unbraced segment (kip-in.),

M_B = absolute value of the moment in the center of the unbraced segment (kip-in.),

M_C = absolute value of the moment in the three-quarter point of the unbraced segment (kip-in.).

5.8.3.2—Other I-Shaped Members with Compact or Noncompact Webs and Compact or Noncompact Flanges

The nominal bending strength shall be the lower value obtained according to the limit states of compression flange yielding, compression flange local buckling, tension flange yielding, and lateral-torsional buckling.

5.8.3.2.1—Compression Flange Yielding

$$M_n = R_{pc} M_{yc} = R_{pc} F_y S_{xc} \quad (5.8.3.2.1\text{-}1)$$

If $\frac{I_{yc}}{I_y} \leq 0.23$, use $R_{pc} = 1.0$

For compact web, $\lambda \leq \lambda_p$

$$R_{pc} = \frac{M_p}{M_{yc}} \quad (5.8.3.2.1\text{-}2)$$

$$M_p = Z_x F_y \leq 1.65 S_{xc} F_y \quad (5.8.3.2.1\text{-}3)$$

For noncompact web, $\lambda_p < \lambda \leq \lambda_r$

$$R_{pc} = \left[\frac{M_p}{M_{yc}} - \left(\frac{M_p}{M_{yc}} - 1\right)\left(\frac{\lambda - \lambda_p}{\lambda_r - \lambda_p}\right)\right] \leq \frac{M_p}{M_{yc}} \quad (5.8.3.2.1\text{-}4)$$

where:

S_{xc} = section modulus for the compression flange, (in^3).

5.8.3.2.2—Compression Flange Local Buckling

If $\lambda \leq \lambda_p$ compression FLB buckling does not apply,

where $\lambda > \lambda_p$

$$M_n = \left[R_{pc} M_{yc} - \left(R_{pc} M_{yc} - F_L S_{xc}\right)\left(\frac{\lambda - \lambda_p}{\lambda_r - \lambda_p}\right)\right] \quad (5.8.3.2.2\text{-}1)$$

for:

$$\frac{S_{xt}}{S_{xc}} \geq 0.7, \quad F_L = 0.7 F_y$$
$$\frac{S_{xt}}{S_{xc}} < 0.7, \quad F_L = \frac{S_{xt}}{S_{xc}} F_y \geq 0.5 F_y \quad (5.8.3.2.2\text{-}2)$$

C5.8.3.2

Buckling limits ($\lambda > \lambda_r$) do not need to be considered because slender elements are not allowed.

5.8.3.2.3—Tension Flange Yielding

If $S_{xt} > S_{xc}$ tension flange yielding does not apply.

If $S_{xt} \leq S_{xc}$

$$M_n = R_{pt} M_{yt} = R_{pt} F_y S_{xt} \qquad (5.8.3.2.3\text{-}1)$$

For compact web, $\lambda \leq \lambda_p$

$$R_{pt} = \frac{M_p}{M_{yt}} \qquad (5.8.3.2.3\text{-}2)$$

For noncompact web, $\lambda > \lambda_p$

$$R_{pt} = \left[\frac{M_p}{M_{yt}} - \left(\frac{M_p}{M_{yt}} - 1 \right) \left(\frac{\lambda - \lambda_p}{\lambda_r - \lambda_p} \right) \right] \leq \frac{M_p}{M_{yt}} \qquad (5.8.3.2.3\text{-}3)$$

5.8.3.2.4—Lateral-Torsional Buckling

If $L_b \leq L_p$ LTB does not apply.

If $L_p \leq L_b \leq L_r$,

$$M_n = C_b \left[R_{pc} M_{yc} - \left(R_{pc} M_{yc} - F_L S_{xc} \right) \left(\frac{L_b - L_p}{L_r - L_p} \right) \right] \leq R_{pc} M_{yc}$$

(5.8.3.2.4-1)

If $L_b > L_r$

$$M_r = F_{cr} S_x \leq R_{pc} M_{yc} \qquad (5.8.3.2.4\text{-}2)$$

$$F_{cr} = \frac{C_b \pi^2 E}{\left(\frac{L_b}{r_t} \right)^2} \sqrt{1 + 0.078 \frac{J}{S_x h_o} \left(\frac{L_b}{r_t} \right)^2} \qquad (5.8.3.2.4\text{-}3)$$

$$L_p = 1.1 r_t \sqrt{\frac{E}{F_y}} \qquad (5.8.3.2.4\text{-}4)$$

$$J = 0 \text{ if } \frac{I_{yc}}{I_y} \leq 0.23 \qquad (5.8.3.2.4\text{-}5)$$

$$L_r = 1.95 r_t \frac{E}{F_L} \sqrt{\frac{J}{S_{xc} h_c} + \sqrt{\frac{J}{S_{xc} h_c} + 6.76 \left(\frac{F_L}{E} \right)^2}}$$

(5.8.3.2.4-6)

The effective radius of gyration for LTB, r_t, can be approximated by the radius of gyration of the compression flange plus one-third of the compression portion of the web:

$$r_t = \frac{b_{fc}}{\sqrt{12\left(1+\frac{1}{6}a_w\right)}} \qquad (5.8.3.2.4\text{-}7)$$

where:

a_w = ratio of two times the web area in compression due to application of major axis bending moment alone to the area of the compression flange components.

5.8.4—Tees and Double Angles Loaded in Plane of Symmetry

The nominal bending strength shall be the lowest value obtained according to limit states of plastic moment, flange local buckling, stem local buckling, and lateral-torsional buckling.

5.8.4.1—Plastic Moment

$$M_n = M_p \qquad (5.8.4.1\text{-}1)$$

$M_p = Z_x F_y \leq 1.6 M_y$ for stems in tension

$M_n = M_y$ for stems in compression

5.8.4.2—Flange Local Buckling

If $\lambda \leq \lambda_p$ or flange in tension, then FLB does not apply.

If $\lambda > \lambda_p$ and flange in compression

$$M_n = M_p - \left(M_p - 0.7F_y S_{xc}\right)\left(\frac{\lambda - \lambda_p}{\lambda \ - \lambda_p}\right) \leq 1.6M_y$$

(5.8.4.2-1)

5.8.4.3—Stem Local Buckling

$$M_n = F_{cr} S_x \qquad (5.8.4.3\text{-}1)$$

For $\lambda < \lambda_p$ or stem in tension

$F_{cr} = F_y$

For $\lambda \geq \lambda_p$ and stem in compression

$$F_{cr} = \left[2.55 - 1.84\frac{d}{t_w}\sqrt{\frac{F_y}{E}}\right]F_y \qquad (5.8.4.3\text{-}2)$$

5.8.4.4—Lateral-Torsional Buckling

$$M_n = M_{cr} = \frac{\pi\sqrt{EI_yGJ}}{L_b}\left[B+\sqrt{1+B^2}\right] \tag{5.8.4.4-1}$$

where:

B $= 2.3\left(\frac{d}{L_b}\right)\sqrt{\frac{I_y}{J}}$ when stem in tension, and

B $= -2.3\left(\frac{d}{L_b}\right)\sqrt{\frac{I_y}{J}}$ when stem in compression and within the unbraced length.

5.8.5—I-Shaped, Channels, Tees, and Double Angles Bent about the Minor Axis

The nominal bending strength shall be the lower value obtained according to the limit states of plastic moment and flange local buckling.

5.8.5.1—Plastic Moment

$$M_n = M_p = Z_yF_y \leq 1.6S_yF_y \tag{5.8.5.1-1}$$

5.8.5.2—Flange Local Buckling

If $\lambda \leq \lambda_p$ FLB does not apply.

If $\lambda > \lambda_p$

$$M_n = M_p - \left(M_p - 0.7F_yS_y\right)\left(\frac{\lambda-\lambda_p}{\lambda_r-\lambda_p}\right) \tag{5.8.5.2-1}$$

5.8.6—Single Angles

Single angles subject to flexure should be avoided unless loaded through the shear center. Single angles under flexure are subject to biaxial bending and torsion.

C5.8.6

If single angles are used in flexure, other resources such as AISC (2011) or LRFD Design should be consulted.

5.8.7—Round and Rectangular Bars

The nominal bending strength shall be the lower value obtained according to plastic moment and lateral-torsional buckling.

5.8.7.1—Plastic Moment

For round and rectangular bars bent about the minor axis, and rectangular bars with $\frac{L_bd}{t^2} \leq \frac{0.08E}{F_y}$ bent about the major axis:

$$M_n = M_p = F_yZ \leq 1.6M_y \tag{5.8.7.1-1}$$

5.8.7.2—Lateral-Torsional Buckling for Rectangular Bars Bent about the Major Axis

If $\frac{0.08E}{F_y} < \frac{L_b d}{t^2} \leq \frac{1.9E}{F_y}$

$$M_n = C_b\left[1.52 - 0.274\left(\frac{L_b d}{t^2}\right)\frac{F_y}{t}\right]M_y \leq M_p \quad (5.8.7.2\text{-}1)$$

If $\frac{L_b d}{t^2} > \frac{1.9E}{F_y}$

$$M_n = F_{cr}S_x \leq M_p \quad (5.8.7.2\text{-}2)$$

$$F_{cr} = \frac{1.9EC_b}{\frac{L_b d}{t^2}} \quad (5.8.7.2\text{-}3)$$

where:

t = the section width (in.), and

d = the section depth (in.).

C5.8.7.2

For rectangular bars bent about the minor axis and for round bars, lateral-torsional buckling does not apply.

5.9—COMPONENTS IN TENSION

5.9.1—General

The provisions of this Article apply to tension of rolled open, tubular, and built-up plate sections.

The factored tensile resistance, P_{rt}, shall be

$$P_{rt} = \min\left[\phi_y P_{ny}, \phi_u P_{nu}\right] \quad (5.9.1\text{-}1)$$

where:

ϕ_y and ϕ_u = resistance factor as specified in Article 5.5.3.2

P_{ny} = minimum nominal tensile strength from the limit states of yield on the gross sectional area, and

P_{nu} = rupture strength on the effective net area.

C5.9.1

AISC (2011) design equations were incorporated for typical sign, luminaire, and signal supports. For members and limit states not addressed in these Specifications, other resources should be considered such as AISC (2011) and LRFD Design.

5.9.2—Nominal Tensile Strength

The nominal tensile strength for yield on the gross section shall be:

$$P_{ny} = A_g F_y \quad (5.9.2\text{-}1)$$

C5.9.2

The definitions and limits on the effective net area are based on AISC (2011).

where:

A_g = the gross section area (in.^{2}), and

F_y = yield strength (ksi).

The tensile strength for the fracture on the effective net area shall be:

$$P_{nu} = A_e F_u \tag{5.9.2-2}$$

where:

A_e = net effective area (in.^{2}), and

F_u = tensile strength (ksi).

5.9.3—Effective Net Area

The effective net area, A_e, shall be taken equal to the net area, A_n, when the load is transmitted directly to each of the cross-sectional elements by bolts.

The net area, A_n, shall be calculated as the sum of the individual net areas along the potential critical section. When calculating, A_n, the width deducted for the bolt hole shall be taken as $^1/_{16}$ in. greater than the nominal dimension of the hole.

For a chain of holes across a part in any diagonal or zigzag line (staggered holes), the net width of the part shall be determined by deducting from the gross width the sum of the hole deductions for all the holes in the chain and adding, for each gauge space in the chain, the quantity $s^2/4g$.

For members without holes, the net area, A_n, is equal to the gross area, A_g.

When the load is transmitted through some but not all of the cross-sectional elements, shear lag shall be considered. The effective net area, A_e, shall be computed as:

$$A_e = UA_n \tag{5.9.3-1}$$

where:

U = shear-lag reduction coefficient

For tension members, except plates and hollow structural shapes, connected by fasteners or longitudinal welds or with longitudinal welds in combination with transverse welds:

$$U = \left(1 - \frac{\bar{x}}{L}\right) \leq 0.9 \tag{5.9.3-2}$$

where:

$\bar{x}$ = connection eccentricity, defined as the distance from the connection plane, or face of the member,

C5.9.3

In lieu of the calculated value for U, the following values may be used for bolted connections:

U = 0.80 (single or double angles with four or more bolts per line in the direction of load)

U = 0.60 (single or double angles with three bolts per line in the direction of load)

U = 0.90 (with three or more bolts per line in the direction of load and $b_f \geq {}^2/_3 d$)

U = 0.85 (flange connection I-shaped or tees with three or more bolts per line in the direction of load and $b_f < {}^2/_3 d$)

to the centroid of the section resisting the connection force, (in.)

L = length of connection in the direction of loading (in.), and

Larger values of U are permitted to be used when justified by tests or other rational criteria.

For members connected by only transverse welds:

$$U = \frac{Area\ of\ Directly\ Connected\ Elements}{A_g}$$

For plate members connected by longitudinal welds along both edges:

for $L_w \geq 2w$, $U = 1.00$ (5.9.3-3)

for $2w > L_w \geq 1.5w$, then $U = 0.87$ (5.9.3-4)

for $1.5w > L_w \geq w$, then $U = 0.75$ (5.9.3-5)

where:

L_w = length of longitudinal weld (in.), and

w = plate width (distance between welds) (in.).

The effective net area, A_e, shall not be taken greater than 85 percent of the gross area, A_g, for the design of connecting elements such as splice plates, gusset plates, and connecting plates.

5.9.4—Slenderness Limit

For trusses, L/r shall not exceed 240 for members in tension.

5.10—COMPONENTS IN COMPRESSION

5.10.1—General

The provisions of this article apply to compression of rolled open, tubular, and built-up plate sections.

The factored compressive resistance, P_{rc}, shall be:

$$P_{rc} = \phi_c P_{nc} \quad (5.10.1\text{-}1)$$

where:

ϕ_c = resistance factor as specified in Article 5.5.3.2, and

P_{nc} = minimum nominal compressive strength defined in Article 5.10.2.

C5.10.1

AISC (2011) design equations have been incorporated for typical sign, luminaire, and signal supports. For members and limit states not addressed in these Specifications, other resources should be considered such as AISC (2011) and LRFD Design.

5.10.2—Nominal Compressive Strength

5.10.2.1—Flexural Buckling

The nominal compressive strength shall be calculated as follows:

$$P_{nc} = A_g F_{cr} \quad (5.10.2.1\text{-}1)$$

when $\frac{KL}{r} \leq 4.71\sqrt{\frac{E}{QF_y}}$

$$F_{cr} = Q\left(0.658\right)^{\left(\frac{QF_y}{F_e}\right)} F_y \quad (5.10.2.1\text{-}2)$$

when $\frac{KL}{r} > 4.71\sqrt{\frac{E}{QF_y}}$

$$F_{cr} = 0.877 F_e \quad (5.10.2.1\text{-}3)$$

$$F_e = \frac{\pi^2 E}{\left(\frac{KL}{r}\right)^2} \quad (5.10.2.1\text{-}4)$$

If all section elements $\lambda \leq \lambda_r$

$$Q = 1.0 \quad (5.10.2.1\text{-}5)$$

5.10.2.2—Round Tube with Slender Elements

If $\lambda > \lambda_r$

$$Q = 0.67 + \frac{0.038}{\left(\frac{D}{t}\right)}\left(\frac{E}{F_y}\right) \leq 1.0 \quad (5.10.2.2\text{-}1)$$

5.10.2.3—Multi-Sided and Square and Rectangular Tubes with Slender Elements

If $\lambda > \lambda_r$

$$Q = \frac{A_{EFF}}{A_g} \quad (5.10.2.3\text{-}1)$$

A_{EFF} is calculated from the sum of parts using effective widths, b_e

$$b_e = 1.92t\sqrt{\frac{E}{f}}\left[1 - \frac{0.34}{\left(\frac{b}{t}\right)}\sqrt{\frac{E}{f}}\right] \leq b \quad (5.10.2.3\text{-}2)$$

$f = F_{cr}$ using $Q = 1.0$.

C5.10.2.1

The radius of gyration, r, may be calculated at a distance of $0.50L$ for a tapered column. This value is conservative for all tapered light poles.

For a cantilever column, the effective length factor, K, may be taken as 2.1. Equations using Q are for members with slender elements (round and multi-sided tubular members). For other members, Q = 1 because slender elements are not allowed.

5.10.2.4—Single Angles

Single angles in trusses with the ratio of long-leg-width–short-leg-width less than 1.7 may be designed as axially loaded members with the following column slenderness ratios, *KL/r*.

For equal-leg angles or unequal-leg angles connected through the longer leg:

$$\frac{KL}{r} = 72 + 0.75\left(\frac{L}{r_x}\right)$$
$$\text{when } \frac{L}{r_x} \leq 80 \qquad (5.10.2.4\text{-}1)$$

$$\frac{KL}{r} = 32 + 1.25\left(\frac{L}{r_x}\right) \qquad (5.10.2.4\text{-}2)$$
$$\text{when } \frac{L}{r_x} > 80$$

For unequal-leg angles connected through the shorter leg, *KL/r*, from the applicable equation shall be increased by adding the amount:

$$4\left[\left(\frac{b_l}{b_s}\right)^2 - 1\right],$$

but *KL/r* shall not be taken less than $0.95KL/r_z$ where:

r_x = radius of gyration about the axis parallel to the connected leg (in.),

r_z = radius of gyration about the minor principle axis (in.),

b_l = longer leg width, (in.), and

b_s = shorter leg width, (in.).

C5.10.2.4

If the longer-to-shorter-leg width ratio exceeds 1.7, the angle should be designed as a beam-column considering bending affects. The slenderness ratios indirectly account for bending in the angle due to eccentricity of the loading. The slenderness equations are from AISC (2011).

5.10.2.5—Torsional Buckling

For non-doubly symmetric shapes and some doubly symmetric shapes, torsional and flexural-torsional buckling may need to be considered.

C5.10.2.5

Because torsional column buckling is not a common problem with sign and luminaire and signal supports members, strength equations are not included here. If torsional buckling is of concern, design equations of AISC (2011) should be applied.

5.10.3—Slenderness Limit

For trusses, *KL*/*r* shall not exceed 140 for members in compression.

5.11—COMPONENTS IN DIRECT SHEAR AND TORSION

5.11.1—General

The provisions of this article apply to direct shear and torsion of rolled open, tubular, and built-up plate sections.

The factored direct shear resistance, V_r, shall be:

$$V_r = \phi_v V_n \tag{5.11.1-1}$$

And the factored torsional shear resistance, T_r, shall be:

$$T_r = \phi_t T_n \tag{5.11.1-2}$$

where:

V_r = nominal direct shear capacity,

T_r = nominal torsion capacity, and

ϕ_v and ϕ_t = the resistance factor as specified in Article 5.5.3.2.

C5.11.1

Previous allowable stress design equations for typical sign, luminaire, and signal support structures were modified to limit state design equations for tubular members. AISC (2011) design equations have been incorporated for other typical structures. For structures not addressed in these Specifications, other resources should be considered such as AISC (2011) and LRFD Design.

Herein, direct shear is the shear force created by a change/gradient in bending moment.

5.11.2—Nominal Direct Shear Strength

The nominal direct shear strength due to shear shall be:

$$V_n = A_v F_{nv} \tag{5.11.2-1}$$

where:

F_{nv} = nominal shear stress capacity (ksi), and

A_v = shear area (in.^{2}), as defined in Articles 5.11.2.1 and 5.11.2.2.

5.11.2.1—Nominal Shear Stress Capacity for Tubular Members

5.11.2.1.1—Round Tubular Members

The nominal shear stress capacity for round tubular shapes shall be the greater of:

$$F_{nv} = \frac{1.60E}{\sqrt{\frac{L_v}{D}}\left(\frac{D}{t}\right)^{5/4}} \tag{5.11.2.1.1-1}$$

or

C5.11.2.1.1

AISC (2011) equations for nominal direct shear strength are incorporated for round tubes.

$$F_{nv} = \frac{0.78E}{\left(\frac{D}{t}\right)^{3/2}} \qquad (5.11.2.1.1\text{-}2)$$

but shall not exceed $0.6F_y$,

where:

L_v = distance from the maximum to zero shear force, and

$$A_v = \frac{A_g}{2} \qquad (5.11.2.1.1\text{-}3)$$

5.11.2.1.2—Multi-Sided Tubular Members

The nominal direct shear stress capacity for multi-sided non-square and rectangular tubular shapes shall be:

$$F_{nv} = 0.6F_y \qquad (5.11.2.1.2\text{-}1)$$

$$A_v = \frac{A_g}{2} \qquad (5.11.2.1.2\text{-}2)$$

C5.11.2.1.2

Previous editions of these specifications have shown that multi-sided tubes will not buckle under shear with width–thickness ratios limited to λ_{max}.

5.11.2.2—Nominal Direct Shear Strength for I-Shapes; Channels; Tees; and Square and Rectangular, and Double Angle Shapes

The nominal shear stress capacity shall be:

$$F_{nv} = 0.6F_yC_v \qquad (5.11.2.2\text{-}1)$$

If $\frac{b}{t} \le 1.10\sqrt{\frac{k_vE}{F_y}}$

$$C_v = 1.0 \qquad (5.11.2.2\text{-}2)$$

If $1.10\sqrt{\frac{k_vE}{F_y}} < \frac{b}{t} \le 1.37\sqrt{\frac{k_vE}{F_y}}$

$$C_v = \frac{1.10\sqrt{\frac{k_vE}{F_y}}}{\frac{b}{t}} \qquad (5.11.2.2\text{-}3)$$

If $\frac{b}{t} > 1.37\sqrt{\frac{k_vE}{F_y}}$

$$C_v = \frac{1.51k_vE}{\left(\frac{b}{t}\right)^2 F_y} \qquad (5.11.2.2\text{-}4)$$

For I-shapes, channels and tees:

$$A_v = A_w = dt_w \qquad (5.11.2.2\text{-}5)$$

$$k_v = 5 \qquad (5.11.2.2\text{-}6)$$

$$\frac{b}{t} = \frac{h}{t_w} \qquad (5.11.2.2\text{-}7)$$

For double angles:

$$A_v = 2bt \qquad (5.11.2.2\text{-}8)$$

$$k_v = 1.2 \qquad (5.11.2.2\text{-}9)$$

For square and rectangular shapes:

$$A_v = 2ht \qquad (5.11.2.2\text{-}10)$$

$$k_v = 5 \qquad (5.11.2.2\text{-}11)$$

$$\frac{b}{t} = \frac{h}{t} \qquad (5.11.2.2\text{-}12)$$

5.11.3—Nominal Torsion Strength

The nominal torsion strength due to torsion shall be:

$$T_n = C_t F_{nt} \qquad (5.11.3\text{-}1)$$

where:

T_n = nominal torsion strength, and

C_t = the torsional constant.

5.11.3.1—Nominal Torsion Strength for Tubular Members

5.11.3.1.1—Round Tubular Members

The nominal torsion stress capacities for round tubular shapes shall be the greater of:

$$F_{nt} = \frac{1.23E}{\sqrt{\frac{L}{D}}\left(\frac{D}{t}\right)^{5/4}} \qquad (5.11.3.1.1\text{-}1)$$

and

$$F_{nt} = \frac{0.6E}{\left(\frac{D}{t}\right)^{3/2}} \qquad (5.11.3.1.1\text{-}2)$$

but shall not exceed $0.6F_y$.

C5.11.3

Values for C_t are provided for different shapes in Appendix B.

5.11.3.1.2—Multi-Sided Tubular Members

The nominal torsion stress capacity for multi-sided non-square and rectangular tubular shapes shall be:

$$F_{nt} = 0.6F_y \quad (5.11.3.1.2\text{-}1)$$

C.5.11.3.1.2

Previous editions of these specifications have shown that multi-sided tubes will not buckle with width-to-thickness ratios limited to λ_{max}.

5.11.3.2—I-Shapes; Channels; Tees; and Square and Rectangular, and Angle Shapes

For torsion on open I-shape, channel, tee, and angle sections, AISC Design Guide 9 (1997) may be used to develop an appropriate nominal torsional capacity.

For square and rectangular shapes

$$F_{nt} = F_{nv} \quad (5.11.3.2\text{-}1)$$

5.12—COMBINED FORCES

C5.12

5.12.1—Combined Force Interaction Requirements

Members subjected to combined bending, axial compression or tension, shear, and torsion shall be proportioned to meet the following:

$$\frac{P_u}{P_r} + \frac{BM_u}{M_r} + \left(\frac{V_u}{V_r} + \frac{T_u}{T_r}\right)^2 \leq 1.0 \quad (5.12.1\text{-}1)$$

If $\frac{T_u}{T_r} \leq 0.20$ torsional and shear effects can be ignored, and when:

$$\frac{P_u}{P_r} \geq 0.20$$

$$\frac{P_u}{P_r} + \frac{8}{9}\frac{BM_u}{M_r} \leq 1.0 \quad (5.12.1\text{-}2)$$

when $\frac{P_u}{P_r} < 0.20$

$$\frac{P_u}{2P_r} + \frac{BM_u}{M_r} \leq 1.0 \quad (5.12.1\text{-}3)$$

For round and multi-sided tubular members,

$$M_u = \sqrt{M_{ux}^2 + M_{uy}^2} \quad (5.12.1\text{-}4)$$

and

$$V_u = \sqrt{V_{ux}^2 + V_{uy}^2} \quad (5.12.1\text{-}5)$$

AISC (2011) design equations were incorporated for typical sign, luminaire, and signal supports. For members and limit states not addressed in these specifications, other resources should be considered such as AISC (2011) and LRFD Design.

For structural supports for signs, luminaires, and traffic signals, direct shear is typically small and therefore only torsional effects are checked to determine which interaction equation to use.

For members with biaxial bending about geometric or principal axes, the term $\frac{BM_u}{M_r}$ may be expanded to:

$$B_x \frac{M_{ux}}{M_{rx}} + B_y \frac{M_{uy}}{M_{ry}} \quad (5.12.1\text{-}6)$$

$$\frac{V_u}{V_r} = \text{the greater of } \frac{V_{ux}}{V_{rx}} \text{ or } \frac{V_{uy}}{V_{ry}} \quad (5.12.1\text{-}7)$$

when member is in tension:

$$P_r = \phi_t P_{nt} \quad (5.12.1\text{-}8)$$

when member is in compression:

$$P_r = \phi_c P_{nc} \quad (5.12.1\text{-}9)$$

Moment Magnifier B:

For prismatic members:

Compression: $$B = \frac{1}{1 - \frac{P_u}{P_e}} \quad (5.12.1\text{-}10)$$

where:

$$P_e = \frac{\pi^2 E A_g}{\left(\frac{KL}{r}\right)^2} \quad (5.12.1\text{-}11)$$

Tension:

$$B = 1.0 \quad (5.12.1\text{-}12)$$

For non-prismatic members, Tension:

$$B = 1.0 \quad (5.12.1\text{-}13)$$

Compression: B shall be computed according to Section 4.

This includes square and rectangular tubes and other nontubular shapes.

5.12.2—Bending of Square and Rectangular Tubes

Square and rectangular tubes shall meet the design requirements of Article 5.12.1 for bending about the geometric axes. In addition, this section applies to tubes bent about a skewed (diagonal) axis. The following interaction equation shall be satisfied:

$$\left(\frac{B_x M_{ux}^*}{M_{rx}}\right)^\alpha + \left(\frac{B_y M_{uy}^*}{M_{ry}}\right)^\alpha \le 1 \quad (5.12.2\text{-}1)$$

where for tubes with $\lambda \le \lambda_r$

C5.12.2

NCHRP Report 494, *Supports for Highway Signs, Luminaries, and Traffic Signals* (Fouad et al., 2003) compared theoretical diagonal bending to experimental tests. The interaction increase in nominal strength is justified for tubes bent about the diagonal for sections with limited width–thickness ratios. Although the diagonal strength properties are significantly less than the primary axis properties, tests show additional strength compared with current strength predictions. For compact sections, the reserve strength is 33 percent higher for bending about a diagonal axis ($Z_x/S_x = 1.5$)

$\alpha = 1.60, M_{rx} = \phi_f M_{px}, M_{ry} = \phi_f M_{py}$

and, for tubes with $\lambda_r < \lambda \leq \lambda_{max}$

$\alpha = 1.00, M_{rx} = \phi_f M_{nx}$ and $M_{ry} = \phi_f M_{ny}$

M^*_{ux}, M^*_{uy} = factored moments from skewed diagonal loading.

than about the principal axes ($Z_x/S_x = 1.13$), where Z_x and S_x are the plastic and elastic section moduli, respectively.

5.13—CABLES AND CONNECTIONS

The provisions of this Article apply to cables and their connections.

The factored tensile resistance, R_{rt}, shall be

$$R_{rt} = \phi_{rt} R_{nr} \tag{5.13-1}$$

where:

ϕ_{rt} is the resistance factor as specified in Article 5.5.3.2.

For horizontal supports (wire and connections) of span-wire pole structures, the resistance of the cable or connection is the nominal breaking strength of the cable or connection.

C5.13

Typically manufacturers' data may be used for the resistances.

5.14—WELDED CONNECTIONS

Welding design and fabrication shall be in accordance with the latest edition of the *AWS Structural Welding Code—Steel* (2010) and *AWS Structural Welding Code—Reinforcing Steel* (2011).

Fatigue considerations are provided in Section 11.

C5.14

Hybrid laser arc welding (HLAW) is categorized in AWS D1.1 as "Other Welding Processes." Process variables are to be agreed upon by the Fabricator and Owner. Fabrication guidance is provided in Division II.

5.15—BOLTED CONNECTIONS

Design of bolted connections shall be in accordance with the current LRFD Design.

Fatigue considerations are provided in Section 11.

5.16—ANCHOR BOLT CONNECTIONS

This Article provides the minimum requirements for design of steel anchor bolts used to transmit loads from attachments into concrete supports or foundations by means of tension, bearing, and shear. A minimum of eight anchor bolts shall be used to connect high-mast lighting towers.

C5.16

Figure C5.16-1 shows a typical steel-to-concrete double-nut connection. Figure C5.16-2 shows a typical single-nut connection. Installation considerations are provided in Division III.

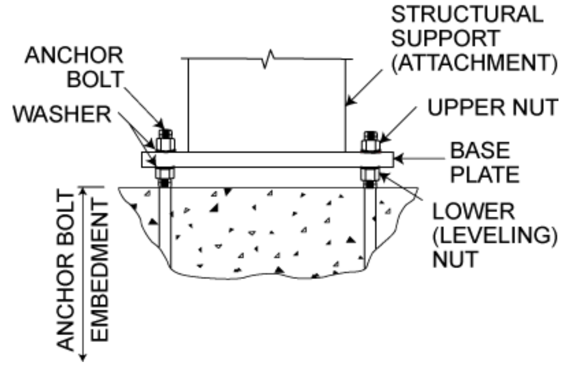

Figure C5.16-1—Typical Double-Nut Connection

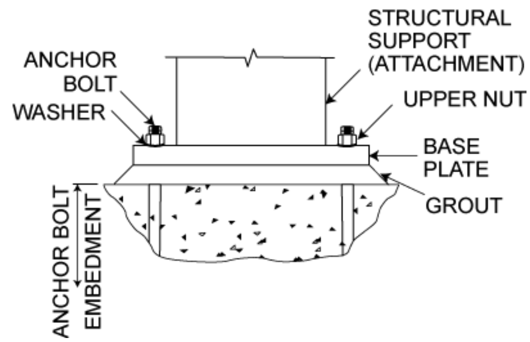

Figure C5.16-2. Typical Single-Nut Connection

5.16.1—Anchor Bolt Types

Cast-in-place anchor bolts shall be used in new construction.

The following requirements shall apply:

- Anchor bolts may be headed through the use of a preformed bolt head or by other means, such as a nut, flat washer, or plate;
- Hooked anchor bolts with a yield strength not exceeding 55 ksi may be used; and
- Deformed reinforcing bars may be used as anchor bolts.

C5.16.1

The ring-shaped base plate of a high-level (pole-type) luminaire support has low bending stiffness. The number of anchor bolts and the geometry of the base plate determine the stiffness of the base plate. Research, both fatigue tests and analytical studies, indicates that using less than 12 bolts can result in a reduction in fatigue performance in some connections. The fatigue strength of the butt-welded connection detail with an external collar reinforcement shown in Detail 4.8 of Table 11-9.3.1-1 is less sensitive to the number of anchor bolts and as few as 8 bolts can be used with this detail. However, due to the field problems in properly tightening the anchor bolts, the use of 12 bolts is recommended to provide adequate anchorage stiffness when fatigue is controlling the design of the luminaire support connection.

Research (Jirsa et al., 1984) has shown that headed cast-in-place anchor bolts perform significantly better than hooked anchor bolts, regarding possible pull-out prior to development of full tensile strength. Caution should be exercised when using deformed reinforcing bars as anchor bolts, because no fatigue test data are available on threaded reinforcing bar. The ductility of deformed reinforcing bars, as measured by elongation, can be significantly less than most other anchor bolts.

Anchor bolts with hooks make it impossible to perform a proper ultrasound inspection.

5.16.2—Anchor Bolt Materials

Anchor bolt material not otherwise specified shall conform to the requirements of ASTM F1554.

For hooked smooth bars, the yield strength shall not exceed 55 ksi.

Reinforcing bar material used for anchor bolts shall conform to ASTM A615 or ASTM A706. The yield strength shall not exceed 80 ksi.

Typical anchor bolt design material properties are provided in Table 5.16.2-1.

C5.16.2

Steel with yield strengths greater than 120 ksi have been found to be susceptible to stress corrosion in most anchorage environments [ACI 349–90 (1995)]. Galvanized steel with tensile strengths greater than 160 ksi are more susceptible to hydrogen embrittlement.

Threaded reinforcing bars (ASTM A706) may be used for anchor bolts. Reinforcing bars conforming to ASTM A615 have been used in the past. However, because of possible low toughness, ASTM A615 reinforcing bars should not be used for nonredundant, fatigue susceptible support structures such as cantilevers and high-mast luminaries. Anchor bolts conforming to ASTM F1554 usually have satisfactory fracture toughness. Charpy V-notch impact testing is not required for anchor bolt material.

Table 5.16.2-1—Typical Anchor Bolt Material

Material Specification	Yield Strength, ksi	Minimum Tensile Strength, ksi
ASTM F1554 Bolts	36	58
ASTM F1554 Bolts	55	75
ASTM F1554 Bolts	105	125
ASTM A706 Bars	60	80

Note: ASTM A615 bars are not recommended for anchor bolts when subject to fatigue.

5.16.3—Design Basis

The anchor bolts and their anchorage shall be designed to transmit loads from the attachment into the concrete support or foundation by means of tension, bearing, and shear, or any combination thereof.

The design of the anchor bolt and its anchorage shall ensure transfer of load from anchor to concrete. The anchorage system shall be proportioned such that the load in the steel portion of the anchorage will reach its minimum tensile strength prior to failure of the concrete.

C5.16.3

AISC (2006), *Design Guide 1: Base Plate and Anchor Rod Design* may be used. Concrete anchorages may be designed using ACI 318-11, Appendix D. Resistance factors shall be as specified in ACI 318-11. Loads shall be determined from this Specification.

A ductile connection to concrete fails by yielding of the steel anchor. A nonductile failure will occur by a brittle fracture of the concrete in tension or by the anchor slipping in the concrete without the steel yielding. All anchor bolts should be designed for a ductile steel failure prior to any sudden loss of capacity of the anchorages resulting from a brittle failure of the concrete.

5.17—REFERENCES

AASHTO. 2002. Standard Specifications for Highway Bridges, 17th Edition, HB-17. American Association of State Highway and Transportation Officials, Washington, DC.

AASHTO. 1990. Standard Specification for Steel Anchor Bolts, M 314. American Association of State Highway and Transportation Officials, Washington, DC. Available individually in downloadable form; also in Standard Specifications for Transportation Materials and Methods of Sampling and Testing.

AASHTO. 2010. Zinc Coating (Hot-Dip) on Iron and Steel Hardware, M 232M/M 232. American Association of State Highway and Transportation Officials, Washington, DC. Available individually in downloadable form; also in Standard Specifications for Transportation Materials and Methods of Sampling and Testing.

AASHTO. 2014. Standard Specification for Zinc (Hot-Dip Galvanized) Coatings on Iron and Steel Products, M 111M/M 111. American Association of State Highway and Transportation Officials, Washington, DC. Available individually in downloadable form; also in Standard Specifications for Transportation Materials and Methods of Sampling and Testing.

AASHTO. 2014. *AASHTO LRFD Bridge Design Specifications,* 7th Edition, American Association of State Highway and Transportation Officials, Washington, DC.

ACI. 1991. *State-of-the-Art Report on Anchorage to Concrete,* ACI 355.1R-91. American Concrete Institute, Farmington Hills, MI.

ACI. 2011. *Building Code Requirements for Structural Concrete,* ACI 318-11. American Concrete Institute, Farmington Hills, MI.

AISC. 1989. *Manual of Steel Construction—Allowable Stress Design,* 9th Edition. American Institute of Steel Construction, Chicago, IL.

AISC. 2011. *Manual of Steel Construction—Load and Resistance Factor Design, Vol. 1,* 14th Edition. American Institute of Steel Construction, Chicago, IL.

AISC. 1997. *Design Guide 9: Torsional Analysis of Structural Steel Members,* Seaburg, P.A. and Carter, C.J., American Institute of Steel Construction, Chicago, IL.

AISC. 2006. *Design Guide 1: Base Plate and Anchor Rod Design*, Fisher, J.M, and Kloiber, L.A., 2nd Edition. American Institute of Steel Construction, Chicago, IL.

ASCE. 2010. *Minimum Design Loads for Buildings and Other Structures*, Report No. ASCE/SEI 7-10. American Society of Civil Engineers, Reston, VA.

AISC. 2011. *Manual of Steel Construction—Load and Resistance Factor Design,* 14th Edition. American Institute of Steel Construction, Chicago, IL.

ASCE. 2011. *Design of Steel Transmission Pole Structures,* Report No. ASCE/SEI 48-11. American Society of Civil Engineers, Reston, VA.

ASTM. 2007. Standard Specification for Anchor Bolts, Steel, 36, 55, and 105-ksi Yield Strength, F1554-07a, *Annual Book of ASTM Standards*. American Society for Testing Materials, West Conshohocken, PA.

ASTM. 2014. “Standard Specification for Deformed and Plain Low-Alloy Steel Bars for Concrete Reinforcement" ASTM A706/A706M-14, *Annual Book of ASTM Standards*. American Society for Testing Materials, West Conshohocken, PA.

ASTM. 2013. “Standard Specification for High-Strength Low-Alloy Columbium-Vanadium Structural Steel,” A572/A572M-13a, *Annual Book of ASTM Standards*. American Society for Testing Materials, West Conshohocken, PA.

ASTM. 2014. “Standard Specification for Deformed and Plain Carbon-Steel Bars for Concrete Reinforcement,” A615/A615M-14, *Annual Book of ASTM Standards*. American Society for Testing Materials, West Conshohocken, PA.

ASTM. 2014. “Standard Specification for Steel, Sheet and Strip, Hot-Rolled, Carbon, Structural, High-Strength Low-Alloy, High-Strength Low-Alloy with Improved Formability, and Ultra-High Strength,” A1011/A1011M-14, *Annual Book of ASTM Standards*. American Society for Testing Materials, West Conshohocken, PA.

ASTM. 2014. “Standard Specification for Steel Tubes, Low-Carbon or High-Strength Low-Alloy, Tapered for Structural Use,” A595/A595M-14, *Annual Book of ASTM Standards*. American Society for Testing Materials, West Conshohocken, PA.

AWS. 2010. *Structural Welding Code—Steel*, ANSI/AWS D1.1. American Welding Society, Miami, FL.

AWS. 2011. *Structural Welding Code—Reinforcing Steel*, ANSI/AWS D1.4. American Welding Society, Miami, FL.

Dexter, R., and M. Ricker. 2002. *Fatigue-Resistant Design of Cantilever Signal, Sign, and Light Supports*, NCHRP Report 469. Transportation Research Board, National Research Council, Washington, DC.

Fouad, F. H., J. S. Davison, N. Delatte, E. A. Calvert, S. E. Chen, E. Nunez, and R. Abdalla. 2003. "Structural Supports for Highway Signs, Luminaires, and Traffic Signals," NCHRP Report 494. Transportation Research Board, National Research Council, Washington DC.

Hall, J. H. 2005. The Effect of Baseplate Flexibility on the Fatigue Performance of Welded Socket Connections in Cantilevered Sign Structures. Master’s thesis, Graduate School of Engineering, Lehigh University, Bethlehem, PA.

James, R. W., P. B. Keating, R. W. Bolton, F. C. Benson, D. E. Bray, R. C. Abraham, and J. B. Hodge. 1997. *Tightening Procedures for Large-Diameter Anchor Bolts*. Report No. FHWA/TX-98/1472-IF. Texas Transportation Institute, Texas Department of Transportation, Austin, Texas.

Jirsa, J.O., et al. 1984. *Strength and Behavior of Bolt Installations Anchored in Concrete Piers*, Research Report No. 305-1F. Center for Transportation Research, Texas Department of Transportation, Austin, TX.

Kaczinski, M. R., R. J. Dexter, and J. P. Van Dien. 1998. *Fatigue-Resistant Design of Cantilevered Signal, Sign and Light Supports*, NCHRP Report 412. Transportation Research Board, National Research Council, Washington, DC.

Koenigs, M. T., T. A. Botros, D. Freytag, and K. H. Frank. 2003. *Fatigue Strength of Signal Mast Arm Connections,* Research Report 4178-2. Texas Department of Transportation, Austin, TX.

Ocel, J. M., R. J. Dexter, and J. F. Hajjar. 2006. *Fatigue-Resistant Design for Overhead Signs, Mast-Arm Signal Poles, and Lighting Standards*, Report No. MN/RC-2006-07. Minnesota Department of Transportation, St. Paul, Minnesota.

Puckett, J., M. Garlich, M. Barker, A. Nowak, and C. Menzemer. 2014. *Development and Calibration of AASHTO LRFD Specifications for Structural Supports for Highway Signs, Luminaires, and Traffic Signals*, NCHRP Report 796. Transportation Research Board, National Research Council, Washington, DC.

Roy, S., Y. C. Park, R. Sause, J. W. Fisher, and E.K. Kaufmann. 2011. *Cost Effective Connection Details for Highway Sign, Luminaire, and Traffic Signal Structures*. NCHRP Web Only Document 176 (Final Report for NCHRP Project 10-70). Transportation Research Board, National Research Council, Washington, DC.

Schilling, C. G. 1965. “Buckling Strength of Circular Tubes,” *Journal of the Structural Division* 19, No. ST5 (October 1965). American Society of Civil Engineers, New York, NY, pp. 325–348.

Stam, A., N. Richman, C. Pool, C Rios, T. Anderson, and K. Frank. 2011. *Fatigue Life of Steel Base Plate to Pole Connections for Traffic Structures*, Research Report No. FHWA/TX-11/9-1526-1. Center for Transportation Research, Texas Department of Transportation, Austin, TX.

Warpinski, M. K. 2006. The Effect of Base Connection Geometry on the Fatigue Performance of Welded Socket Connections in Multi-Sided High-Mast Lighting Towers. Master’s thesis, Graduate School of Engineering, Lehigh University, Bethlehem, PA.

SECTION 6: ALUMINUM DESIGN

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6.1—SCOPE

This Section provides the design provisions for structural supports for highway signs, luminaires, and traffic signals as described in Article 1.4 that are fabricated using aluminum alloys.

All design provisions specified herein are based on the Load and Resistance Factor Design (LRFD) method. The design resistance given in terms of stress or strength for a particular limit state shall be computed using the nominal resistance stress (F_{nr}) or nominal strength (R_n) determined from the given formula or by analysis, and multiplying it by its respective resistance factor (ϕ). The computed design resistance shall equal or exceed the required stress or strength computed using LRFD factored load combinations,

$$\phi R_n \geq \sum_i \gamma_i Q_i \qquad (6.1\text{-}1)$$

where:

γ_i = load factors, and
Q_i = load effects.

LRFD resistance factors are specified for each of the limit states addressed herein and load factors are defined in Article 3.4.

C6.1

Specifications are given in terms of a design stress when possible. For example, the limit states for member tension, member flexure, member buckling, and local buckling are specified in terms of design stress. The limit states associated with connections are given in terms of design strength because the geometric factors of the connections cannot easily be decoupled from the design analysis. Attention must be given to each specification to ensure the correct application.

All LRFD resistance factors are taken directly from the *Aluminum Design Manual* (ADM, 2010). It is not the intent of this section to merely duplicate the ADM. However, because the ADM is recognized as the controlling design guideline, duplication is inherent. Provisions and commentary taken from the ADM have been tailored to the extent necessary to facilitate the design of structural supports for highway signs, luminaires, and traffic signals. If additional design details are required, the ADM may be referenced.

6.2—DEFINITIONS

Aluminum—Aluminum or an aluminum alloy.

Available strength—Design resistance.

Beam—A structural member that has the primary function of resisting bending moments.

Bearing-type connection—A bolted connection where shear forces are transmitted by the bolt bearing against the connection elements.

Block shear rupture—In a connection, the limit state of tension fracture or yielding along one path and shear yielding or fracture along another path.

Bolt—A headed and externally threaded mechanical device designed for insertion through holes in assembled parts to mate with a nut and normally intended to be tightened or released by turning that nut.

Buckling—The limit state of a sudden change in the geometry of a structure or any of its elements under a critical compressive loading condition.

Column—A structural member that has the primary function of resisting a compressive axial force.

Design load—The applied load determined in accordance with load combinations.

Design resistance—The resistance factor multiplied by the nominal resistance, ϕR_n.

Design stress—The design force effect divided by the appropriate section property, such as section modulus or cross section area.

Effective length—The length of an otherwise identical column with the same strength when analyzed with pinned end conditions.

Effective length factor—Ratio between the effective length and the unbraced length of the member.

Effective net area—Net area modified to account for the effect of shear lag.

Elastic analysis—Structural analysis based on the assumption that the structure returns to its original geometry on removal of the load.

Element—A component of a shape's cross section. Elements are connected to other elements only along their longitudinal edges. Elements addressed by the Specification include flat elements, described by their width b and thickness t, and curved elements, described by their mid-thickness radius R_b and thickness t. An Aluminum Association standard I-beam, for example, consists of five flat elements—a web element and two elements in each flange.

Factored load—The product of a load factor and the nominal load.

Fastener—Generic term for a weld, bolt, rivet, or other connecting device.

Fatigue—The limit state of crack initiation and growth resulting from repeated application of loads.

Filler metal—Metal to be added in making a welded joint.

Fillet weld—Weld of generally triangular cross section made between intersecting surfaces of elements.

Flexural buckling—A buckling mode in which a compression member deflects laterally without twist or change in cross-sectional shape.

Flexural-torsional buckling—A buckling mode in which a compression member bends and twists simultaneously.

Gauge—Transverse center-to-center spacing of fasteners.

Gauge—A traditional term for referring to the thickness of a wrought product. The use of thickness, rather than gauge, is now the preferred description of this dimension.

Geometric axis—Axis parallel to a web, flange, or angle leg.

Grip—Distance between the bolt head and the nut.

Lateral-torsional buckling—The buckling mode of a flexural member involving deflection normal to the plane of bending that occurs simultaneously with twist about the shear center of the cross-section.

Limit state—A condition in which a structure or component becomes unfit for service and is judged either to be no longer useful for its intended function (serviceability limit state) or to have reached its ultimate load-carrying capacity (strength limit state).

Load effect—Forces, stresses, and deformations produced in a structural component by the applied loads.

Load factor—A factor that accounts for deviations of the nominal load from the actual load, for uncertainties in the analysis that transforms the load into a load effect, and for the probability that more than one extreme load will occur simultaneously.

Local buckling—The limit state of buckling of a compression element within a cross section.

Lockbolt—A two-piece fastener consisting of a pin (bolt) and collar. The softer, smooth bore collar is mechanically swaged (reduced or tapered by squeezing) onto the pin and into either zero pitch, annular lock grooves, or special thread-form grooves in a tension–tension installation method. Hydraulic or pneumatic installation tools provide the tension and swaging action.

Longitudinal centroidal axis—Axis through the centroid of a member along its length.

LRFD (*load and resistance factor design*)—A method of proportioning structural components such that the design strength equals or exceeds the required resistance of the component under the action of the LRFD load combinations.

LRFD load combination—A load combination intended for superimposed load effects.

Member—An individual, discrete component (such as a beam or column) of a larger structure.

Member buckling—Flexural, torsional, or flexural-torsional buckling of the overall member.

Net area—Gross area reduced to account for removed material.

Nominal dimension—Designated or theoretical dimension, as in the tables of section properties.

Nominal load—Magnitude of the load specified by the applicable building code.

Nominal resistance—Resistance of a structure or component (without the resistance factor applied) available to resist load effects, as determined in accordance with these Specifications.

Pitch—Longitudinal center-to-center spacing of fasteners; center-to-center spacing of bolt threads along the axis of a bolt.

Post-buckling strength—The load or force that can be resisted by an element, member, or frame after initial elastic buckling has occurred.

Pull-out—The tensile load required to pull a screw out of a threaded part.

Resistance factor—A factor that accounts for unavoidable deviations of the actual strength from the nominal strength, its variability, and for the manner and consequences of failure.

Rivet—A headed and unthreaded mechanical device used to assemble two or more components by an applied force which deforms the protrouding end to develop a completed mechanical joint.

Rod—A solid wrought product that is long in relation to its circular cross section, with a diameter not less than 0.375 in.

Screw—A headed and externally threaded fastener held in place by threading into one of the connected parts.

Screw chase—A groove parallel to the longitudinal axis of an extrusion, intended to retain a screw whose axis is perpendicular to the longitudinal axis of the extrusion.

Screw slot—A semi-hollow in an extrusion intended to retain a screw parallel to the axis of the extrusion.

Self-drilling screw—A screw that drills and taps its own hole as it is being driven.

Service load combination—Load combinations under which serviceability limit states are evaluated.

Slip-critical connection—A bolted connection designed to resist movement by friction on the faying surface of the connection under the clamping forces of the bolts.

Stiffener—A structural element attached or integral to a member to distribute load, transfer shear, or prevent buckling.

Structural component—Member, connector, or connecting element or assemblage.

Structure—An object, including but not limited to buildings, walls, fences, towers, bridges, railings, signs, and luminaires, designed to support loads.

Tapping screw—A screw that threads a preformed hole as it is being driven.

Thread-cutting screw—A tapping screw that is installed into a preformed hole, with internal mating threads formed as a result of cutting out the material being tapped to form the relief area of the threaded shank.

Thread-forming screw—A tapping screw that is installed into a preformed hole, with internal mating threads formed as a result of cold flow of the material being tapped into the relief area of the threaded shank.

Torsional buckling—A buckling mode in which a compression member twists about its shear center axis.

Tubular shape—A hollow shape that resists lateral-torsional buckling primarily by torsional resistance rather than warping resistance; that is, for which C_w is much less than $0.038JL_b^2$.

Unbraced length—The length of a member between brace points or between a brace point and a cantilever's free end, measured between the longitudinal centroidal axes of the bracing members. For columns, brace points are points at which lateral translation is restrained for flexural buckling or twisting is restrained for torsional buckling. For beams, brace points are points at which the compression flange is restrained against lateral deflection or the cross section is restrained against twisting.

Weld-affected zone—Metal within 1 in. of the centerline of a weld.

6.3—NOTATION

A_b = nominal cross sectional area of a bolt in the unthreaded body (in.^{2}) (6.5.1)
$A_{bearing}$ = bearing surface/area (6.9.2)
A_e = effective net cross sectional area (in.^{2}) (6.6.6) (C6.13.2)
A_f = area of the member farther than $\frac{2}{3}c$ from the neutral axis (C6.13.4)
A_g = gross cross sectional area (in.^{2}) (6.5.1) (6.7.2.1) (6.7.2.2) (6.7.3) (C6.13.5) (C6.13.7)
A_{gt} = gross area in tension (in^2) (6.5.1) (6.14.10)
A_{gv} = gross area in shear (in^2) (6.5.1) (6.14.10)
A_i = the net cross sectional area of a local element "i" of a larger cross section (in.^{2}) (6.7.2.1)
A_n = the net cross sectional area (in.^{2}) (6.5.1) (6.6.5) (6.6.6)
A_{nt} = net area in tension (in^2) (6.5.1) (6.14.10)
A_s = cross sectional area of an intermediate stiffener (in.^{2}) (6.5.1)
A_{we} = effective area of groove weld (6.5.1) (6.14.8)
A_{wz} = the portion of area of a cross-section lying within 1 in. of the center of weld (in.^{2}) (6.5.1) (C6.13.2) (C6.13.4) (C6.13.5) (C6.13.7)
A_{wzc} = effective area of groove weld (6.5.1)
a = nominal individual outside width of an element of an angle, (in.) (6.6.5)
B = buckling constant interpreted, with following subscript (6.5.2) (6.12.2):
c–compression in columns/members/elements (6.5.1) (6.5.2)
p–compression in flat plates (6.5.1) (6.5.2)
t–compression in round tubes (6.5.1) (6.5.2)
tb–flexure in round tubes (6.5.1) (6.5.2)
br–flexure in rectangular bars (6.5.1) (6.5.2)
s–shear in flat plates (6.5.1) (6.5.2)
b = effective width, usually of a subject element (in.) (6.5.1)

b = the clear height of the web for unstiffened webs, (in.) (6.10.1) (6.11.2)

b = nominal individual outside width of an element of an angle, (in.) (6.6.5)

C = torsional shear constant (6.11.2)

C = buckling constant interpreted, with following subscript (6.5.1) (6.5.2):

c–compression in columns/members/elements (6.5.1) (6.5.2)

p–compression in flat plates (6.5.1) (6.5.2)

t–compression in round tubes (6.5.1) (6.5.2)

tb–flexure in round tubes (6.5.2)

br–flexure in rectangular bars (6.5.1) (6.5.2)

s–shear in flat plates (6.5.1) (6.5.2)

C = torsional shear constant that relates shear stress to torsion for hollow tubes (6.5.1)

C_b = coefficient that accounts for the moment gradient along the beam's length (6.5.1) (6.8.2) (C6.8.2) (6.8.6)

C_c = slenderness limit for axial compression

C_w = warping constant (6.2)

c = distance from the neutral axis to the extreme fiber (in.) (C6.13.4)

c_c = distance from the neutral axis to the element extreme fiber with the greatest compressive stress (in.) (6.5.1)

c_{cf} = distance from the neutral axis to the centerline of the compression flange (in.) (6.5.1) (6.8.5)

c_{cw} = distance from the neutral axis to the web group's extreme compression fiber (in.) (6.5.1) (6.8.5)

c_o = distance from the neutral axis to extreme fiber of the element (in.) (6.5.1)

c_{tf} = distance from the neutral axis to the extreme fiber of the tension flange (in.) (6.5.1) (6.8.5)

c_{tw} = distance from the neutral axis to the web group's extreme tension fiber (in.) (6.5.1) (6.8.5)

D = nominal diameter of fastener (in.) (6.5.1) (6.14.2) (6.14.3) (6.14.6)

D = nominal diameter of a solid rod (in.) (6.5.1) (6.9.1) (6.11.3)

D = buckling slope constant (ksi), with following subscript (6.5.1) (6.5.2):

c–compression in columns/members/elements (6.5.1) (6.5.2)

p–compression in flat plates (6.5.1) (6.5.2)

t–compression in round tubes (6.5.1) (6.5.2)

tb–flexure in round tubes (6.5.1) (6.5.2)

br–flexure in rectangular bars (6.5.1) (6.5.2)

s–shear in flat plates (6.5.1) (6.5.2)

D_h = nominal diameter of a fastener hole (in.) (6.5.1) (6.9.2) (6.14.7)

D_{rivet} = nominal rivet diameter (6.14.7)

d = full depth of the section or beam (in.) (6.5.1) (6.8.3) (6.11.2)

d_{cs} = depth of the countersink (6.9.2)

d_e = edge distance (in.) (6.5.1) (6.9.1) (6.9.2) (6.14.7)

d_{Leff} = the effective length of a weld (in.) (6.5.1)

d_t = the effective throat depth of a fillet weld (in.) (6.5.1) (6.14.9)

E = modulus of elasticity in compression (ksi) (6.5.1) (6.5.2) (6.7.2) (6.7.3) (6.8.6)

F_b = stress corresponding to the nominal flexural strength (ksi) (6.5.1) (6.8.5) (6.12.5)

F_{bo} = stress corresponding to the flexural compression strength for an element if no part of the cross section were weld-affected (ksi) (6.5.1) (6.13.4) (6.13.7)

F_{bw} = stress corresponding to the flexural compression strength for an element if the entire cross section were weld-affected (ksi) (6.5.1) (6.13.4) (6.13.7)

F_{bx} = stress corresponding to the nominal flexural strength about the major axis "x" (ksi) (6.5.1)

F_{by} = stress corresponding to the nominal flexural strength about the minor axis "y" (ksi) (6.5.1)

F_c = compressive stress corresponding to the nominal uniform compressive strength (ksi) (6.5.1) (6.7.2.2) (6.7.3) (6.8.5)

F_{ci} = compressive stress corresponding to the nominal compressive strength of local element "i" (ksi) (6.5.1) (6.7.2.1)

F_{co} = stress corresponding to the uniform compression strength for an element if no part of the cross section were weld-affected (ksi) (6.5.1) (6.13.3) (6.13.6)

F_{cw} = stress corresponding to the uniform compression strength for an element if the entire cross section were weld-affected (ksi) (6.5.1) (6.13.3) (6.13.6)

F_{cy} = nominal compressive yield strength (ksi) (6.5.1) (6.5.2) (6.7.2.1) (C6.7.2) (6.12.5)

F_e = compressive stress corresponding to nominal elastic buckling strength (ksi) (6.5.1) (6.7.2) (6.7.3) (6.8.6)

F_n = nominal shear strength of an A325 bolt (ksi) (6.5.1)

F_s = shear stress corresponding to nominal shear strength (ksi) (6.5.1) (6.11.1) (6.11.2)

F_{so} = stress corresponding to the shear strength for an element if no part of the cross section were weld-affected (ksi) (6.5.1) (6.13.8)

F_{su} = nominal shear ultimate strength (ksi) (6.5.1) (6.14.10)

F_{suw} = nominal shear ultimate strength of weld-affected material (ksi) (6.5.1) (6.14.8)

F_{sw} = stress corresponding to the welded shear strength for 1. the filler metal of a fillet weld, or 2. an element assuming the entire cross section were weld-affected (ksi) (6.5.1) (6.13.8) (6.14.9)

F_{sy} = nominal shear yield strength (ksi) (6.5.1) (6.5.2) (6.11.3) (6.14.10)

F_t = tensile stress corresponding to the nominal tensile strength (ksi) (6.5.1)

F_{to} = un-welded yield strength (ksi) (6.13.5) (6.13.7)

F_{tu} = nominal tensile ultimate strength (ksi) (6.5.1) (6.13.2) (6.13.5) (6.13.7) (6.14.10)

F_{tuw} = nominal tensile ultimate strength of weld-affected material (ksi) (6.5.1) (6.13.2) (6.13.5) (6.13.7) (6.14.8)

F_{ty} = nominal tensile yield strength (ksi) (6.5.1) (6.13.2) (C6.13.5) (6.13.7) (6.14.10)

F_{tw} = adjusted welded yield strength (ksi) (6.13.5) (6.13.7)

F_{tyw} = nominal tensile yield strength of weld-affected material (ksi) (6.5.1) (6.13.2) (6.13.5) (6.13.7)

f_{bx} = bending stress about the major axis "x" corresponding to a specific flexural loading (ksi) (6.5.1)

f_{by} = bending stress about the minor axis "y" corresponding to a specific flexural loading (ksi) (6.5.1)

f_c = compressive stress developed by a specific axial loading (ksi) (6.5.1)

f_s = shear stress developed by a specific torsion or transverse shear loading (ksi) (6.5.1)

ft = axial tension stress (ksi) (6.5.1) (6.8.5)

G = shear modulus of elasticity (ksi) (6.4)

G_f = bolt grip the total thickness of parts being fastened (in.) (6.14.6)

h_{sc} = hole factor for slip-critical bolted connections (6.5.1) (6.14.6)

I_{cy} = moment of inertia of the compression flange about its y-axis (in.^{4}) (6.8.2)

I_f = moment of inertia of the flange group about the cross section's neutral axis consisting of the flat elements in uniform compression and flat elements in uniform tension and their edge or intermediate stiffeners (in.^{4}) (6.5.1) (6.8.5)

I_w = moment of inertia of the web group about the cross section's neutral axis consisting of the flat elements in flexure and their intermediate stiffeners (in.^{4}) (6.5.1) (6.8.5)

I_y = moment of inertia of a member about its y-axis (in.^{4}) (6.5.1) (6.8.2) (6.8.3)

J = torsion constant (in.^{4}) (6.5.1) (6.8.3) (6.11.1)

k = effective length factor by rational analysis. k shall be taken larger than or equal to unity unless rational analysis justifies a smaller value (6.5.1) (6.7.1) (6.7.3) (6.7.4)

k_1 = coefficient for determining λ_2 slenderness limit for elements tolerating post-buckling stress (6.5.1) (6.5.2) (C6.7.2.2)

k_2 = coefficient used to determine post-buckling stress (6.5.1) (6.5.2) (C6.7.2.2)

k_t = tension alloy coefficient (6.5.1)

L = unsupported length in plane of bending. Bracing points are the points at which the compression flange is restrained against lateral movement or twisting (in.) (6.5.1) (6.6.7) (6.7.1) (6.7.3) (6.7.4)

L_b = length of the beam between brace points or between a brace point and the free end of a cantilever beam (in.) (6.5.1) (6.8.3) (6.8.6)

L_c = length of a connection in the direction of the load measured from the center of fasteners or the end of welds (in.) (6.6.6)

L_v = length of tube from zero shear force to maximum shear force (in.) (6.5.1) (6.10.2) (6.11.1)

Lw = overall weld length (in) (6.14.9)

L_{we} = effective weld length for tension and compression (6.5.1) (6.14.8) (6.14.9)

M_A = absolute value of the moment at the quarter point of the unbraced segment (in.-kip) (6.8.2)

M_B = absolute value of the moment at the midpoint of the unbraced segment (in.-kip) (6.8.2)

M_C = absolute value of the moment at the three-quarter point of the unbraced segment (in.-kip) (6.8.2)

M_{max} = absolute value of the maximum moment in the unbraced segment (in.-kip) (6.8.2)

M_n = the nominal moment strength (in.-kip) (6.5.1) (6.8.6)

M_{nc} = the nominal moment strength of the element group of a section that is subject to compression (in.-kip) (6.5.1) (6.8.5)

M_{no} = the nominal lateral-torsional buckling strength if no part of the cross section was weld-affected (in.-kip) (6.13.4)

M_{nw} = nominal lateral-torsional buckling strength if the entire cross section were weld-affected (in.-kip) (6.13.4)

m = coefficient for elements in flexure, supported on two edges (6.5.1)

N_s = number of slip/shear planes in the connection (6.5.1)

n = thread count per inch (6.5.1) (6.7.2.1)

P = an absolute axial load of column/member (kip) (6.12)

P_c = axial compressive load of column/member (kip) (6.5.1)

P_n = nominal axial compressive resistance of a column/member (kip) (6.5.1) (6.7.2.1) (6.7.2.2) (6.7.3)

Q_i = load effects (6.1)

r = radius of gyration of the member/column/element about the axis of buckling (in.) (6.5.1) (6.6.7) (6.7.1) (6.7.3) (6.7.4)

$\mathrm{r_s}$ = radius of gyration of stiffener about the stiffened elements mid-thickness (in) (6.7.2)

r_y = radius of gyration about the y axis (in) (6.8.3) (C6.8.6)

r_{ye} = effective radius of gyration about the section y-axis for lateral-torsional buckling (in.). See ADM, Part I, Section F.2.2 for definition (6.5.1) (6.8.3) (6.8.6) (C6.8.6)

R = for torsion and shear, outside radius of round tube or maximum outside radius of oval tube (in.) (6.11.1)

R_b = mid-thickness radius of curvature of curved plate and tubular beam element (in.) (6.5.1) (6.10.2) (6.11.1) (6.13.3) (6.13.4)

R_n = nominal resistance (kip) (6.1) (6.5.1) (6.14.4) (6.14.5) (6.14.6) (6.14.8) (6.14.9) (6.14.10)

S = section modulus (in.^{3}) (6.5.1) (6.13.4)

S_c = section modulus of a beam on the compression side of the neutral axis (in.^{3}) (6.5.1) (6.8.6)

S_{eq} = equivalent slenderness ratio (6.7.2)

S_w = a fillet weld with equal-sized legs (6.14.8) (6.14.9)

S_x = elastic section modulus (in^3) (C6.12.5)

s = the longitudinal center-to-center spacing (pitch) of any two consecutive holes (in.) (6.6.5) (6.14.7)

T = strength of round or oval tubes in torsion (kip-in) (6.11.1)

t = thickness, usually of a subject element (in.) (6.5.1) (6.6.5) (6.7.2) (6.9.2) (6.10.1) (6.10.2) (6.11.2) (6.13.3) (6.13.4)

t_i = thickness of the i^{th} element

T_b = minimum fastener tension (kip) (6.5.1) (6.14.5)

T_n = nominal torsion resistance (in.-kip) (6.5.1) (6.11.2) (6.11.3)

v = Poisson's ratio (6.4)

w_i = least net width of the i^{th} element (6.6.5)

$\bar{x}$ = Eccentricity of the connection in the x-axis direction (in.) (6.6.6)

$\bar{y}$ = Eccentricity of the connection in the y-axis direction (in.) (6.6.6)

Z_x = plastic section modulus (in^3) (C6.12.5)

α = the ratio of the weld zone area (the portion of area of a cross section lying within 1 in. of the center of weld in.^{2}) to the gross area (Table 6.5.1-13) (C6.13.5) (C6.13.7)

α = exponent for square tubes under combined (Table 6.5.1-12)

α = multiplier for bearing strength as a way to account for countersinks (6.9.2)

β = ratio of weld zone area to effective area (6.5.1) (C6.13.2)

Δ = an eccentricity generally associated with an axial load connection resulting in second-order moment affects (6.12)

δ = a lateral deflection/deformation generally associated with an axial loaded member resulting in second-order moment effects (6.5.1) (6.12)

κ = ratio of weld zone area to member area farther than 2c/3 away from neutral axis (6.5.1)

λ = slenderness ratio of a column/member/element used to determine compression limit states (6.5.1) (6.7.2) (6.10.1) (6.10 2) (6.11.1) (6.11.2) (6.12.5)

λ_{eq} = slenderness ratio derived using analytically determined parameters (6.5.1) (6.7.1) (C6.10.1)

λ_1 = slenderness ratio that demarcates yielding from inelastic buckling of the subject column/member/element (6.5.1) (6.7.1) (6.12.5)

λ_2 = slenderness ratio that demarcates inelastic buckling from elastic buckling of the subject column/member/element (6.5.1) (6.7.1) (6.7.2) (6.7.2.2) (C6.7.2.2) (6.12.5)

μ = mean slip coefficient (6.5.1)

ρ_{st} = stiffener effectiveness ratio (6.7.2)

ϕ = resistance factor (6.1) (6.5.1) (6.8.5) (6.8.6) (6.12.5) (6.14.4) (6.14.5) (6.14.6) (6.14.8) (6.14.9) (6.14.10)

ϕ_b = resistance factor for flexure (6.5.1) (6.8.5) (6.8.6)

ϕ_c = resistance factor for compression (6.5.1) (6.7.2) (6.7.2.1) (6.7.2.2) (6.7.3)

ϕ_t = resistance factor for tension (6.5.1) (6.11.2) (6.11.3)

ϕ_v = resistance factor for shear (6.5.1)

ϕ_T = resistance factor for torsion (6.5.1)

χ = ratio of weld zone area to member area farther than 2c/3 away from neutral axis (6.5.1)

6.4—MATERIAL AND MATERIAL PROPERTIES

Mechanical properties given in Articles 6.4.1 through 6.4.3 shall apply to materials used at operating temperatures $\leq 200^{\circ}F$. Alloys 5083, 5086, and 5456 shall not be used at operating temperatures $>150^{\circ}F$.

Unless otherwise noted:

- Poisson's ratio may be taken to be $v = 0.33$,
- Shear modulus $G = \dfrac{3E}{8}$ (ksi),
- Density $\rho = 0.10\left(\dfrac{\text{lbs}}{\text{in.}^3}\right)$,
- Shear yield strength F_{sy} =0.6$_{ty}$,
- Shear ultimate strength F_{su} =0.6$_{tu}$, and
- Bearing strength shall be computed as functions of F_{tu} as specified in the bearing formulas given in Table 6.5.1-10.

Values for the modulus of elasticity given in Table 6.4.1-1 are the result of compressive strength testing and represent typical values. The tensile modulus is approximately 2 percent less than the compressive modulus. A modulus 100 ksi less than those given shall be used for deflection computations.

6.4.1—Wrought Aluminum Alloys

The mechanical properties for non-welded wrought aluminum alloys used as structural members are given in Table 6.4.1-1. Specifications and properties of welded wrought aluminum alloys are given in Table 6.4.2-1.

Applicable ASTM specification designations are:

- A153/A153M "Standard Specification for Zinc Coating (Hot-Dip) on Iron and Steel Hardware,"
- A325 "Standard Specification for Structural Bolts, Steel, Heat Treated, 120/105 ksi Minimum Tensile Strength,"
- A563 "Standard Specification for Carbons and Alloy Steel Nuts,"
- B26/B26M "Standard Specification for Aluminum Alloy Sand Castings,"
- B108 /B108M "Standard Specification for Aluminum-Alloy Permanent Mold Castings,"
- B209 "Standard Specification for Aluminum and Aluminum-Alloy Sheet and Plate,"
- B210 "Standard Specification for Aluminum and Aluminum-Alloy Drawn Seamless Tubes,"
- B211 "Standard Specification for Aluminum and Aluminum-Alloy Rolled or Cold-Finished Bar, Rod, and Wire,"

C6.4

The aluminum products specifically included in this section are well suited for application to structural supports for highway signs, luminaires, and traffic signals because of their associated properties and are most commonly selected. For aluminum alloys not found in Tables 6.4.1-1 and 6.4.2-1, reference should be made to Part I-Specifications for Aluminum Structures of the ADM.

Minimum specified values and minimum expected values are both used in the tables. Minimum specified values are given for tensile yield and tensile ultimate strengths; they are statistically determined from at least 100 tests and 10 lots material. Minimum expected values are given for compressive yield and shear ultimate strengths; they are not guaranteed.

- B221 "Standard Specification for Aluminum and Aluminum-Alloy Extruded Bars, Rods, Wire, Profiles, and Tubes,"
- B241/B241M "Standard Specification for Aluminum and Aluminum-Alloy Seamless Pipe and Seamless Extruded Tube,"
- B308/B308M "Standard Specification for Aluminum-Alloy 6061-T6 Standard Structural Profiles,"
- B429/B429M "Standard Specification for Aluminum-Alloy Extruded Structural Pipe and Tube,"
- B618/B618M "Standard Specification for Aluminum-Alloy Investment Castings,"
- B695 "Standard Specification for Coatings of Zinc Mechanically Deposited on Iron and Steel,"
- B928/B928M "Standard Specification for High Magnesium Aluminum-Alloy Sheet and Plate for Marine Service and Similar Environments,"
- F593 "Standard Specification for Stainless Steel Bolts, Hex Cap Screws, and Studs,"
- F594 "Standard Specification for Stainless Steel Nuts," and
- F1554 "Standard Specification for Anchor Bolts, Steel, 36, 55, and 105-ksi Yield Strength."

Table 6.4.1-1 Minimum Mechanical Properties for Select Aluminum Alloys

Alloy and Temper	Product	Thickness Range[(a)] (in.)	Tension F_{tu}[(b)] (ksi)	Tension F_{ty}[(b)] (ksi)	Compression F_{cy}[(b)] (ksi)	Shear F_{su}[(b)] (ksi)	Modulus of Elasticity E[(b)] (ksi)
5083-H111	Extrusions	Up through 0.500	40	24	21	24	10400
5083-H111	Extrusions	0.501 and over	40	24	21	23	10400
5083-H116, H321	Sheet and Plate	0.188–1.500	44	31	26	26	10400
5083-H116, H321	Plate	1.501–3.00	41	29	24	24	10400
5086-H34	Sheet and Plate Drawn Tube	All	44	34	32	26	10400
5456-H116	Sheet and Plate	0.188–1.25	46	33	27	27	10400
5456-H116	Plate	1.251–1.50	44	31	25	25	10400
5456-H321	Sheet and Plate	0.188–0.499	46	33	27	27	10400
5456-H321	Plate	0.500–1.500	44	31	25	25	10400
5456-H116, H321	Plate	1.501–3.00	41	29	25	25	10400
6005-T5	Extrusions	Up through 1.00	38	35	35	24	10100
6005A – T61	Extrusions	Up through 1.00	38	35	35	24	10100
6061-T6, T651	Sheet and Plate	0.01–4.00	42	35	35	27	10100
6061-T6, T6510, T6511	Extrusions	All	38	35	35	24	10100
6061-T6, T651	Rod and Bar	Up through 8.00	42	35	35	25	10100
6061-T6	Drawn Tube	0.025–0.500	42	35	35	27	10100
6061-T6	Pipe	All	38	35	35	24	10100
6063-T5	Extrusions	Up through 0.500	22	16	16	13	10100
6063-T5	Extrusions	0.501 to 1.00	21	15	15	12	10100
6063-T6	Extrusions and Pipe	All	30	25	25	19	10100
6105-T5	Extrusions	Up through 0.500	38	35	35	24	10100
6351-T5	Extrusions	Up through 1.00	38	35	35	24	10100

Notes:

(a) Most product and thickness ranges are taken from *Aluminum Standards and Data* (Aluminum Association, 2006).

(b) See Article 6.4

6.4.2—Cast Alloys

The mechanical properties of non-welded cast aluminum materials used in structural systems are given in Table 6.4.2-1. Specifications and properties of welded cast aluminum alloys are addressed in Article 6.4.1. Young's modulus of elasticity, E, shall be taken to be:

E = 10000 ksi

The compressive yield strength shall be taken to be equal to the tensile yield strength

$F_{cy} = F_{ty}$

unless otherwise specified.

Applicable ASTM specifications are:

- B26, Aluminum Alloy Sand Castings, and
- B108, Aluminum Alloy Permanent Mold Castings.

Dimensional tolerances shall conform to ASTM B26 and B108.

The owner shall require the casting producer to report tensile yield strengths. The owner shall require that the tensile ultimate and tensile yield strengths of specimens cut from sand castings to meet the values specified in Table 6.4.2-1.

Radiographic inspection shall be in accordance with ASTM B26 Grade C or ASTM B108 Grade C criterion. Quantity and acceptance criteria shall be as given in Part 1, Table A.3.7 of the ADM.

C6.4.2

Tensile properties given in Table 6.4.2-1 assume samples are taken from castings and not separately cast bars.

Modulus tests of aluminum cast alloys are generally done in compression. Because tensile strength is often 2 percent less than the compression strength; an "average" value of 10000 ksi is specified.

Table 6.4.2-1 represents 75 percent of the strength values found in ASTM B26 and ASTM B108 which are the values specified for use by Section A.3.3 of the ADM.

Table 6.4.2-1—Minimum Mechanical Properties for Select Welded Aluminum Alloys

Alloy and Temper	Product	Thickness Range (in.)	Tension		Compression	Shear
			F_{tuw} $^{(a)}$ (ksi)	F_{tyw} (ksi)	F_{cyw} (ksi)	F_{suw} (ksi)
5083-All	Extrusions	All	39	16	16	24
5083-All	Sheet and Plate	0.188–1.500	40	18	18	24
5083-All	Plate	1.501–3.00	39	17	17	24
5086-All	All	All	35	14	14	21
5456-All	Sheet and Plate	0.188–1.500	42	19	19	26
5456-All	Plate	1.501–3.00	41	18	17	25
6005-T5	Extrusions	Up through 1.00	24	13	13	15
6005A-T61	Extrusions	Up through 1.00	24	13	13	15
6061-T6, T651, T6510, T6511[(a)]	All		24	15	15	15
6061-T6, T651, T6510, T6511[(b)]	All	Over 0.0375	24	11	11	15
6063-T5, T52, T6	Extrusions	All	17	8	8	11
6351-T5, T6[(a)]	Extrusions	All	24	15	15	15
6351-T5, T6[(b)]	Extrusions	Over 0.0375	24	11	11	15

Notes:

(a) When welded with 5183, 5356, or 556 filler alloy regardless of thickness, and when welded with 4043, 5554, or 5654 alloy filler for thickness < 0.375 in.

(b) When welded with 4043, 5554, or 5654 alloy filler.

Table 6.4.2-2 Nominal Strength of Select Aluminum Alloy Castings

Alloy	Temper	Process	F_{tu} (ksi)	F_{ty} (ksi)	Notes
356.0	T6	Sand	22.5	15.0	*a, d*
356.0	T7	Sand	23.3	—	*a, b, d*
A356.0	T6	Sand	25.5	18.0	*a, d*
A444.0	T4	Permanent Mold	15.0	—	*a, b, c, d*
356.0	T6	Permanent Mold	24.8	16.5	*a, d*
356.0	T71	Permanent Mold	18.8	—	*a, b, d*
A356.0	T61	Permanent Mold	28.5	19.5	*a, c, d*

(a) Values represent 75 percent of those given in ASTM B26 and ASTM B108.
(b) Yield strength measured and specified by contract or purchase order.
(c) Properties apply to sections less than 2 in. thick except for A444.0 where a maximum thickness of 0.75 in. applies.
(d) 75 percent values reflect specimens removed from castings and not separately cast bars.

6.4.3—Aluminum Filler Alloys

Filler alloys covered by this Specification conform to AWS A.5.10. Mechanical properties for filler metals shall be as given in Table 6.4.3-1. A filler alloy compatibility chart is provided in Table 6.4.3-2.

Table 6.4.3-1 Nominal Strengths of Aluminum Alloy Filler Metals

Filler Alloy	F_{tu} (ksi)	F_{su} (ksi)
4043	24	11.5
5183	40	21
5356	35	17
5554	31	17
5556	42	20

Table 6.4.3-2 Filler Alloy Compatibility

Base Metal	5052	5456	6005 6061 6063	356.0 A356.0 A444.0
5052	5183/5356	5356	5183/5356	4043
5083	5183	5356	5356	4043/5356
5456		5356	5356	4043
6005, 6061, 6063			4043	4043
356.0, A356.0, A444.0				4043

6.5—LRFD SUMMARY DESIGN REQUIREMENTS

6.5.1—Design Stress Formulas

Tables 6.5.1-1 through 6.5.1-14 provide a comprehensive, but not necessarily complete, summary of design stress and strength formulas. Each formula is given with its governing design limit state identified. These formulas shall be used for the design of structural supports for highway signs, luminaires, and traffic signals that are fabricated using any and all aluminum alloys. Limit state definitions and descriptions are provided in Articles 6.6 through 6.14.

Table 6.5.1-1—Stress Formulas: Member Axial Tension

Eq. # (Article)	Limit State			Design Stress
Eq. 6.5.1-1 (6.6.2) (6.13.2 item 1a)	Axial Tension (any tension member)	Gross Section	Yielding	$\phi F_t = \phi_t F_{ty}, \quad \phi_t = 0.9$
Eq. 6.5.1-2 (6.6.4)		Net Effective Area	Rupture	$\phi F_t = \phi_t \frac{F_{tu}}{k_t}, \quad \phi_t = 0.75$
Eq. 6.5.1-3 (6.8.3 item 1a)	Elements in Uniform Tension	Gross Area	Yielding	$\phi F_t = \phi_t F_{ty}, \quad \phi_t = 0.9$
Eq. 6.5.1-4 (6.8.3 item 1b)			Rupture	$\phi F_t = \phi_t \frac{F_{tu}}{k_t}, \quad \phi_t = 0.75$

$k_t \equiv$ tension coefficient for alloy and temper (reference Article 6.6.2 and Table 6.6.2-1)

Table 6.5.1-2—Stress Formulas: Member/Element Flexural Tension

Eq. # (Article)	Limit State			Design Stress
Eq. 6.5.1-5 (6.8.3 item 2a)	Flexure tension	Elements in Flexure	Yielding	$\phi F_t = \phi_t 1.30 F_{ty}, \quad \phi_t = 0.9$
Eq. 6.5.1-6 (6.8.3 item 2b)			Rupture	$\phi F_t = \phi_t \frac{1.42 F_{tu}}{k_t}, \quad \phi_t = 0.75$
Eq. 6.5.1-7 (6.8.3 item 2c)		Pipes and Round Tubes	Yielding	$\phi F_t = \phi_t 1.17 F_{ty}, \quad \phi_t = 0.9$
Eq. 6.5.1-8 (6.8.3 item 2d)			Rupture	$\phi F_t = \phi_t \frac{1.24 F_{tu}}{k_t}, \quad \phi_t = 0.75$
Eq. 6.5.1-9 (6.8.2) (6.8.2)		Rectangular Bars and Rods	Yielding	$\phi F_t = \phi_t 1.30 F_{ty}, \quad \phi_t = 0.9$
Eq. 6.5.1-10 (6.8.2) (6.8.2)			Rupture	$\phi F_t = \phi_t \frac{1.42 F_{tu}}{k_t}, \quad \phi_t = 0.75$

$k_t \equiv$ tension coefficient for alloy and temper (reference Article 6.6.2 and Table 6.6.2-1)

Table 6.5.1-3—Stress Formulas: Member Axial Compression in Columns

Eq. # (Article)	Limit State	Design Stress: λ_2	Design Stress: $\lambda < \lambda_2$ (Yielding/Inelastic)	Design Stress: $\lambda \geq \lambda_2$ (Elastic)
Eq. 6.5.1-11 (6.7.1) (6.13.3)	Member Axial Compression Through the Centroidal Axis $\lambda = \frac{k\,L}{r}$, $\phi_c = 0.90$	C_c	$\phi F_c = \phi_c\left(0.85\left(B_c - D_c\lambda\right)\right)$ *if* $\leq \phi_c\ F_{cy}$ *else* $\phi F_c = \phi_c\ F_{cy}$	$\phi F_c = \phi_c \frac{0.85\ \pi^2 E}{\lambda^2}$
Eq. 6.5.1-12 (6.7.2.1)	Weighted Average of Local Element Buckling Stresses	$\phi P_c = \phi_c \sum_{i=1}^{n} F_{ci} A_i + \phi_c F_{cy}\left(A_g - \sum_{i=1}^{n} A_i\right)$, $\phi_c = 0.9$		
Eq. 6.5.1-13 (6.7.2.2)	Alternative Weighted Average of Local Element Buckling Stresses	$\phi P_c = \phi_c F_c A_g$, $\phi_c = 0.9$		
Eq. 6.5.1-14 (6.7.3)	Interaction Between Member Buckling and Local Buckling	*If* $F_e < F_c$, $\phi P_n = \phi_c \left[\frac{0.85\pi^2 E}{\left(kL/r\right)^2}\right]^{\frac{1}{3}} F_e^{\frac{2}{3}} A_g$, $\phi_c = 0.90$		

Table 6.5.1-4—Stress Formulas: Member Flexural Compression

Eq. # (Article)	Limit State	Design Stress				
		λ_1	λ_2	$\lambda \le \lambda_1$ (Yielding)	$\lambda_1 < \lambda < \lambda_2$ (Inelastic)	$\lambda \ge \lambda_2$ (Elastic/ Post-Buckling)
Eq. 6.5.1-15 (6.8.3 item 1)	Lateral-Torsional Buckling (open shapes) $\lambda = \frac{L_b}{r_{ye}\sqrt{C_b}}, \quad \phi_b = 0.9$ $r_{ye} \equiv$ see Article F.2.1 in ADM		$1.2\,C_c$		$\phi F_b = \phi_b \left(B_c - \frac{D_c \lambda}{1.2} \right)$	$\phi F_b = \phi_b \frac{\pi^2 E}{(1.2\,\lambda)^2}$
Eq. 6.5.1-16 (6.8.3 item 3a)	Lateral-Torsional Buckling (closed shapes) $\lambda = \frac{2 L_b S_c}{C_b \sqrt{I_y J}}, \quad \phi_b = 0.9$ see Article F.3.1 in ADM		$\left(\frac{C_c}{1.6} \right)^2$		$\phi F_b = \phi_b \left(B_c - 1.6\, D_c \sqrt{\lambda} \right)$	$\phi F_b = \phi_b \frac{\pi^2 E}{2.56\,\lambda}$
Eq. 6.5.1-17 (6.8.3 item 4a)	Lateral-Torsional Buckling (rectangular bars) $\lambda = \frac{d}{t}\sqrt{\frac{L_b}{C_b d}}, \quad \phi_b = 0.9$ see Article F.4.2 in ADM		$\frac{C_{br}}{2.3}$		$\phi F_b = \phi_b \left(B_{br} - 2.3\, D_{br} \lambda \right)$	$\phi F_b = \phi_b \frac{\pi^2 E}{5.29\,\lambda^2}$
Eq. 6.5.1-18 (6.8.3 item 6a)	Local Buckling of Single Angles if a leg tip is a point of maximum compression $\lambda = \frac{b}{t}, \quad \phi_b = 0.9$ see Article F.5 in ADM	$\frac{B_{br} - 1.3\, F_{cy}}{4.0\, D_{br}}$	$\frac{C_{br}}{4.0}$	$\phi F_b = \phi_b 1.3\, F_{cy}$	$\phi F_b = \phi_b \left(B_{br} - 4.0\, D_{br} \lambda \right)$	$\phi F_b = \phi_b \frac{\pi^2 E}{(4.0\,\lambda)^2}$
Eq. 6.5.1-19 (6.8.3 item 6b)	Local Buckling of Single Angles if a leg is in uniform compression $\lambda = \frac{b}{t}, \quad \phi_b = 0.9$ see Article F.5 in ADM	$\frac{B_p - F_{cy}}{5.0\, D_p}$	$\frac{C_p}{5.0}$	$\phi F_b = \phi_b F_{cy}$	$\phi F_b = \phi_b \left(B_p - 5.0\, D_p \lambda \right)$	$\phi F_b = \phi_b \frac{\pi^2 E}{(5.0\,\lambda)^2}$

Table 6.5.1-5—Stress Formulas: Member Flexural Compression

Eq. # (Article)	Limit State		Design Stress/Strength
Eq. 6.5.1-20 (6.8.3 item 2b)	Flexural Compressive Yielding Stress $\phi F_b = \phi \dfrac{M_n}{S}$	Pipes and Round Tubes	$\phi F_b = \phi_b 1.17 F_{cy}, \quad \phi_b = 0.9$
Eq. 6.5.1-21 (6.8.3 item 4b) (6.8.3 item 5a)	Flexural Compressive Yielding Stress $\phi F_b = \phi \dfrac{M_n}{S}$	Solid Rectangular Bars and Rods	$\phi F_b = \phi_b 1.30 F_{cy}, \quad \phi_b = 0.9$
Eq. 6.5.1-22 (6.8.5 item 1)	Weighted Average Compressive Flexural Member Strength	Beam Elements	$\phi M_{nc} = \phi_b \left(\dfrac{F_c I_f}{c_{cf}} + \dfrac{F_b I_w}{c_{cw}} \right), \quad \phi_b = 0.9$
Eq. 6.5.1-23 (6.8.5 item 2)	Weighted Average Tensile Flexural Member Strength	Beam Elements	$\phi M_{nt} = \phi_b \left(\dfrac{F_t I_f}{c_{tf}} + \dfrac{F_b I_w}{c_{tw}} \right), \quad \phi_b = 0.9$
Eq. 6.5.1-24 (6.8.6 item 2)	Interaction between Local Buckling and Lateral-Torsional Buckling	Open Sections	$\phi M_n \le \phi_b \left[\dfrac{\pi^2 E}{\left(\dfrac{L_b}{1.2 r_{ye} \sqrt{C_b}} \right)^2} \right]^{\frac{1}{3}} F_e^{\frac{2}{3}} S_c, \quad \phi_b = 0.9$

Table 6.5.1-6—Stress Formulas: Local Buckling—Flat Elements under Uniform Compression

Eq. # (Article)	Limit State	λ_1	λ_2	Design Stress: $\lambda \le \lambda_1$ (Yielding)	Design Stress: $\lambda_1 < \lambda < \lambda_2$ (Inelastic)	Design Stress: $\lambda \ge \lambda_2$ (Elastic/ Post-Buckling)
Eq. 6.5.1-25 (6.7.2) (6.8.4 item 3)	Flat Elements Supported on One Edge (for columns whose buckling axis is not an axis of symmetry) $\lambda = \frac{b}{t}, \quad \phi_c = 0.90$	$\frac{B_p - F_{cy}}{5.0\,D_p}$	$\frac{C_p}{5.0}$	$\phi F_c = \phi_c F_{cy}$	$\phi F_c = \phi_c \left(B_p - 5.0\,D_p \lambda\right)$	$\phi F_c = \phi_c \frac{\pi^2 E}{(5.0\,\lambda)^2}$
Eq. 6.5.1-26 (6.7.2.2) (6.8.4 item 3)	Flat Elements Supported on One Edge (for all other buckling axis conditions beyond Equation 6.5.1-17, for both columns and beams) $\lambda = \frac{b}{t}, \quad \phi_c = 0.90$	$\frac{B_p - F_{cy}}{5.0\,D_p}$	$\frac{k_1 B_p}{5.0\,D_p}$	$\phi F_c = \phi_c F_{cy}$	$\phi F_c = \phi_c \left(B_p - 5.0\,D_p \lambda\right)$	$\phi F_c = \phi_c \frac{k_2 \sqrt{B_p E}}{5.0\,\lambda}$
Eq. 6.5.1-27 (6.7.2.2) (6.8.4 item 3)	Flat Elements Supported on Both Edges $\lambda = \frac{b}{t}, \quad \phi_c = 0.90$	$\frac{B_p - F_{cy}}{1.6\,D_p}$	$\frac{k_1 B_p}{1.6\,D_p}$	$\phi F_c = \phi_c F_{cy}$	$\phi F_c = \phi_c \left(B_p - 1.6\,D_p \lambda\right)$	$\phi F_c = \phi_c \frac{k_2 \sqrt{B_p E}}{1.6\,\lambda}$
Eq. 6.5.1-28 (6.7.2.2) (6.8.4 item 3)	Flat Elements Supported on Both Edges With an Intermediate Stiffener $\lambda = 4.62\left(\frac{b}{t}\right)\sqrt{\frac{1+\frac{A_s}{bt}}{1+\sqrt{1+\frac{10.67 I_o}{bt^3}}}}, \quad \phi_c = 0.90$	$\frac{B_c - F_{cy}}{D_c}$	C_c	$\phi F_c = \phi_c F_{cy}$	$\phi F_c = \phi_c \left(B_c - D_c \lambda\right)$	$\phi F_c = \phi_c \frac{\pi^2 E}{\lambda^2}$
Eq. 6.5.1-29 (6.7.2.2) (6.8.4 item 3)	Flat Elements—Alternative Method $\lambda_{eq} = \pi\sqrt{\frac{E}{F_e}}, \quad \phi_c = 0.90$ F_e is the elastic local buckling stress of the cross section via analysis	$\frac{B_p - F_{cy}}{D_p}$	$\frac{k_1 B_p}{D_p}$	$\phi F_c = \phi_c F_{cy}$	$\phi F_c = \phi_c \left(B_p - D_p \lambda_{eq}\right)$	$\phi F_c = \phi_c \frac{k_2 \sqrt{B_p E}}{\lambda_{eq}}$

Table 6.5.1-7—Stress Formulas: Local Buckling—Flat Elements under Flexural (Non-Uniform) Compression

Eq. # (Article)	Limit State	Design Stress λ_1	λ_2	$\lambda \le \lambda_1$ (Yielding)	$\lambda_1 < \lambda < \lambda_2$ (Inelastic)	$\lambda \ge \lambda_2$ (Elastic/Post-Buckling)
Eq. 6.5.1-30 (6.8.4 item 4)	Flat Elements Supported on Both Edges and Flat Elements Supported on the Compression Edge with the Tension Edge Free $\lambda = \frac{b}{t}$, $\phi_c = 0.90$	$\frac{B_{br} - 1.3\,F_{cy}}{m\,D_{br}}$	$\frac{k_1\,B_{br}}{m\,D_{br}}$	$\phi F_c = \phi_c 1.3\,F_{cy}$	$\phi F_c = \phi_c\left(B_{br} - m\,D_{br}\lambda\right)$	$\phi F_c = \phi_c \frac{k_2\sqrt{B_{br}E}}{m\,\lambda}$
Eq. 6.5.1-31 (6.8.4 item 4)	Flat Elements Supported on the Tension Edge with the Compression Edge Free $\lambda = \frac{b}{t}$, $\phi_c = 0.90$	$\frac{B_{br} - 1.3\,F_{cy}}{3.5\,D_{br}}$	$\frac{C_{br}}{3.5}$	$\phi F_c = \phi_c 1.3\,F_{cy}$	$\phi F_c = \phi_c\left(B_{br} - 3.5\,D_{br}\lambda\right)$	$\phi F_c = \phi_c \frac{\pi^2 E}{(3.5\,\lambda)^2}$
Eq. 6.5.1-32 (6.8.4 item 4)	Flat Elements Supported on Both Edges with a Longitudinal Stiffener [see stiffener "I" requirements in ADM] $\lambda = \frac{b}{t}$, $\phi_c = 0.90$	$\frac{B_{br} - 1.3\,F_{cy}}{0.29\,D_{br}}$	$\frac{k_1\,B_{br}}{0.29\,D_{br}}$	$\phi F_c = \phi_c 1.3\,F_{cy}$	$\phi F_c = \phi_c\left(B_{br} - 0.29\,D_{br}\lambda\right)$	$\phi F_c = \phi_c \frac{k_2\sqrt{B_{br}E}}{0.29\,\lambda}$
Eq. 6.5.1-33 (6.8.4 item 4)	Flat Elements – Alternative Method $\lambda_{eq} = \pi\sqrt{\frac{E}{F_e}}$, $\phi_c = 0.90$ F_e is the elastic local buckling stress of the cross section via analysis	$\frac{B_{br} - 1.3\,F_{cy}}{D_{br}}$	$\frac{k_1\,B_{br}}{D_{br}}$	$\phi F_c = \phi_c 1.3\,F_{cy}$	$\phi F_c = \phi_c\left(B_{br} - D_{br}\lambda_{eq}\right)$	$\phi F_c = \phi_c \frac{k_2\sqrt{B_{br}E}}{\lambda_{eq}}$

When using Eq. 6.5.1-30, $m = 1.15 + \frac{c_o}{2c_c}$ *for* $-1 < \frac{c_o}{c_c} < 1$, $m = \frac{1.3}{\left(1 - \frac{c_o}{c_c}\right)}$ *for* $\frac{c_o}{c_c} < -1$, $m = 0.65$ *for* $c_c = -c_o$

$c_c \equiv$ distance from neutral axis to the element extreme fiber with the greatest compression stress

$c_o \equiv$ distance from neutral axis to other extreme fiber of the element

Table 6.5.1-8—Stress Formulas: Local Buckling—Curved Elements/Round and Oval Tubes

Eq. # (Article)	Limit State	Design Stress λ_1	λ_2	$\lambda \le \lambda_1$	$\lambda_1 < \lambda < \lambda_2$	$\lambda \ge \lambda_2$
Eq. 6.5.1-34 (6.7.2) (6.13.3 Item 1d) (6.13.4 Item 1d)	Flexure—Uniform Compression Gross Section (curved elements supported on both edges) $\lambda = \frac{R_b}{t}$, $\phi_b = 0.90$	$\left(\frac{B_t - F_{cy}}{D_t}\right)^2$	C_t	$\phi F_b = \phi_b F_{cy}$ (Yielding)	$\phi F_b = \phi_b \left(B_t - D_t\sqrt{\lambda}\right)$ (Inelastic)	$\phi F_b = \phi_b \frac{\pi^2 E}{16\lambda\left(1+\frac{\sqrt{\lambda}}{35}\right)^2}$ (Elastic)
Eq. 6.5.1-35 (6.7.2) (6.8.2) (6.8.3)	Local Buckling Flexural Strength for Pipes and Round Tubes $\lambda = \frac{R_b}{t}$, $\phi_b = 0.90$	$\left(\frac{B_{tb} - B_t}{D_{tb} - D_t}\right)^2$	C_t	$\phi F_b = \phi_b \left(B_{tb} - D_{tb}\sqrt{\lambda}\right)$ (Upper Inelastic)	$\phi F_b = \phi_b \left(B_t - D_t\sqrt{\lambda}\right)$ (Lower Inelastic)	$\phi F_b = \phi_b \frac{\pi^2 E}{16\lambda\left(1+\frac{\sqrt{\lambda}}{35}\right)^2}$ (Elastic)

$R_b \equiv$ radius of curved elements taken at the mid-thickness

Table 6.5.1-9—Stress Formulas: Member Shear

Eq. # (Article)	Limit State	Design Stress λ_1	λ_2	$\lambda \le \lambda_1$ (Yielding)	$\lambda_1 < \lambda < \lambda_2$ (Inelastic)	$\lambda \ge \lambda_2$ (Elastic)
Eq. 6.5.1-36 (6.10.1)	Member Flat Webs Supported on Both Edges $\lambda = \frac{b}{t}$, $\phi_v = 0.90$	$\frac{B_s - F_{sy}}{1.25 D_s}$	$\frac{C_s}{1.25}$	$\phi F_s = \phi_v F_{sy}$	$\phi F_s = \phi_v \left(B_s - 1.25 D_s \lambda\right)$	$\phi F_s = \phi_v \frac{\pi^2 E}{(1.25\lambda)^2}$
Eq. 6.5.1-37 (6.10.2)	Round or Oval Tubes $\lambda = 2.9\left(\frac{R_b}{t}\right)^{\frac{5}{8}}\left(\frac{L_v}{R_b}\right)^{\frac{1}{4}}$, $\phi_v = 0.9$	$\frac{1.3 B_s - F_{sy}}{1.63 D_s}$	$\frac{C_s}{1.25}$	$\phi F_s = \phi_v F_{sy}$	$\phi F_s = \phi_v \left(1.3 B_s - 1.63 D_s \lambda\right)$	$\phi F_s = \phi_v \frac{1.3\pi^2 E}{(1.25\lambda)^2}$

Table 6.5.1-10—Stress Formulas: Bearing

Eq. # (Article)	Limit State	Design Stress
Eq. 6.5.1-38 (6.9.1.1)	Bolt in a Round Hole	$\phi\left(\frac{d_e}{D}F_{tu}\right)\leq\phi\left(2F_{tu}\right),\quad \phi=0.75$
Eq. 6.5.1-39 (6.9.1.2)	Bolt in a Slotted Hole	$\phi\left(1.33F_{tu}\right),\quad \phi=0.75$
Eq. 6.5.1-40 (6.9.1.3)	Rivet in a Round Hole	$\phi\left(\frac{d_e}{D_h}F_{tu}\right)\leq\phi\left(2F_{tu}\right),\quad \phi=0.75$
Eq. 6.5.1-41 (6.9.1.4)	Pin in a Round Hole	$\phi\left(\frac{d_e}{1.5D}F_{tu}\right)\leq\phi\left(1.33F_{tu}\right),\quad \phi=0.75$
Eq. 6.5.1-42 (6.9.1.5)	Screw in a Round Hole	$\phi\left(\frac{d_e}{D}F_{tu}\right)\leq\phi\left(2F_{tu}\right),\quad \phi=0.5$
Eq. 6.5.1-43 (6.9.3)	Flat Surfaces	$\phi\left(1.33F_{tu}\right),\quad \phi=0.75$

Table 6.5.1-11—Stress Formulas: Member Torsion

Eq. # (Article)	Limit State	Design Stress/Design Strength				
		λ_1	λ_2	$\lambda\leq\lambda_1$ (Yielding)	$\lambda_1<\lambda<\lambda_2$ (Inelastic)	$\lambda\geq\lambda_2$ (Elastic)
Eq. 6.5.1-44 (6.11.1)	Round or Oval Tubes $\lambda=2.9\left(\frac{R_b}{t}\right)^{\frac{5}{8}}\left(\frac{L_v}{R_b}\right)^{\frac{1}{4}},\quad \phi_T=0.90$	$\frac{B_s-F_{sy}}{1.25D_s}$	$\frac{C_s}{1.25}$	$\phi_T F_{sy}$	$\phi_T\left(B_s-1.25D_s\lambda\right)$	$\phi_T\frac{\pi^2E}{\left(1.25\lambda\right)^2}$
Eq. 6.5.1-45 (6.11.2)	Rectangular Tubes with Flat Sides Supported on Both Edges $\lambda=\frac{b}{t},\quad \phi_T=0.90$	$\frac{B_s-F_{sy}}{1.25D_s}$	$\frac{C_s}{1.25}$	$\phi_T F_{sy}$	$\phi_T\left(B_s-1.25D_s\lambda\right)$	$\phi_T\frac{\pi^2E}{\left(1.25\lambda\right)^2}$
Eq. 6.5.1-46 (6.11.2)	Rectangular Tube Member Resistance	$\phi_T T_n=\phi_T F_s C,\quad \phi_T=0.90$				
Eq. 6.5.1-47 (6.11.3)	Rod Member Resistance	$\phi_T T_n=\phi_T\left(0.196F_{sy}D^3\right),\quad \phi_T=0.90$				

Table 6.5.1-12—Stress Formulas: Member with Combined Forces

Eq. # (Article)	Limit State	Design Stress
Eq. 6.5.1-48 (6.12.2)	Vertical Cantilever Pole Subject to Axial Compression, Uni/Biaxial Flexure, and Shear (except square tubes subject to biaxial bending) $\phi_c = 0.9;\quad \phi_b = 0.9;\quad \phi_v = 0.9$	$\frac{f_c}{\phi_c F_c} + B_{2x}\frac{f_{bx}}{\phi_b F_{bx}} + B_{2y}\frac{f_{by}}{\phi_b F_{by}} + \left(\frac{f_s}{\phi_v F_s}\right)^2 \leq 1.0$
Eq. 6.5.1-49 (6.12.3)	Members Subject to Axial Compression, Uni/Biaxial Flexure, and Shear (except vertical cantilever poles and square tubes subject to biaxial bending) $\phi_c = 0.9;\quad \phi_b = 0.9;\quad \phi_v = 0.9$	$\frac{f_c}{\phi_c F_c} + \frac{B_x f_{bx}}{\phi_b F_{bx}} + \frac{B_y f_{by}}{\phi_b F_{by}} + \left(\frac{f_s}{\phi_v F_s}\right)^2 \leq 1.0$
Eq. 6.5.1-50 (6.12.4)	Members Subject to Axial Tension, Uni/Biaxial Flexure, and Shear $\phi_t = 0.9;\quad \phi_b = 0.9;\quad \phi_v = 0.9$	$\frac{f_t}{\phi_t F_t} + \frac{f_{bx}}{\phi_b F_{bx}} + \frac{f_{by}}{\phi_b F_{by}} + \left(\frac{f_s}{\phi_v F_s}\right)^2 \leq 1.0$
Eq. 6.5.1-51 (6.12.5)	Square Tubes Subject to Axial Compression, Biaxial Bending, and Shear $\phi_c = 0.9;\quad \phi_b = 0.9;\quad \phi_v = 0.9$	$\frac{f_c}{\phi_c F_c} + \left(\frac{B_x f_{bx}}{\phi_b F_{bx}}\right)^{\propto} + \left(\frac{B_y f_{by}}{\phi_b F_{by}}\right)^{\propto} + \left(\frac{f_s}{\phi_v F_s}\right)^2 \leq 1.0$

$\alpha =$ See Article 6.12.5

Table 6.5.1-13—Stress Formulas: Welded Members and Elements

Eq. # (Article)	Limit State			Design Stress
Eq. 6.5.1-52 (6.13.2 item 1b)	Member Uniform Tension (longitudinal welds) $\phi_t = 0.9$	Yielding	Gross Section Area	$\phi_t \left(F_{ty}(1-\alpha)+F_{tyw}\,\alpha\right)$
Eq. 6.5.1-53 (6.13.2 item 2)	Member Uniform Tension (longitudinal and transverse welds) $\phi_t = 0.75$	Rupture	Net Effective Area	$\phi_t \left(\dfrac{F_{tu}(1-\beta)}{k_t}+F_{tuw}\,\beta\right)$
Eq. 6.5.1-54 (6.13.3 item 2)	Member Uniform Compression (longitudinal welds) $\phi_c = 0.9$	Gross Section Area		$\phi_c \left(F_{co}(1-\alpha)+F_{cw}\alpha\right)$
Eq. 6.5.1-55 (6.13.4 item 2)	Member Flexural (longitudinal welds) $\phi_b = 0.9$			$\phi_b \left(F_{bo}(1-\chi)+F_{bw}\,\chi\right)$
Eq. 6.5.1-56 (6.13.6)	Element Uniform Compression (all welds) $\phi_c = 0.9$	Gross Section Area		$\phi_c \left(F_{co}(1-\alpha)+F_{cw}\alpha\right)$
Eq. 6.5.1-57 (6.13.7 item 2)	Element Flexural Compression (all welds) $\phi_c = 0.9$	Gross Section Area		$\phi_c \left(F_{bo}(1-\delta)+F_{bw}\,\delta\right)$
Eq. 6.5.1-58 (6.13.5 item 1)	Element Uniform Tension (longitudinal and transverse welds) $\phi_t = 0.9$	Yielding		$\phi_t \left(F_{ty}(1-\alpha)+F_{tyw}\,\alpha\right)$
Eq. 6.5.1-59 (6.13.5 item 2)	Element Uniform Tension (longitudinal and transverse welds) $\phi_t = 0.75$	Rupture		$\phi_t \left(\dfrac{F_{tu}(1-\alpha)}{k_t}+F_{tuw}\,\alpha\right)$
Eq. 6.5.1-60 (6.13.7 item 1a)	Element Flexural Tension (longitudinal and transverse welds) $\phi_t = 0.9$	Yielding		$\phi_t\, 1.30\left(F_{ty}(1-\alpha)+F_{tyw}\,\alpha\right)$
Eq. 6.5.1-61 (6.13.7 item 1b)	Element Flexural Tension (longitudinal and transverse welds) $\phi_t = 0.9$	Rupture		$\phi_t\, 1.42\left(\dfrac{F_{tu}(1-\alpha)}{k_t}+F_{tuw}\,\alpha\right)$
Eq. 6.5.1-62 (6.13.8)	Shear (all welds) $\phi_v = 0.9$	Gross Section Area		$\phi_v \left(F_{so}(1-\alpha)+F_{sw}\alpha\right)$

When using the design stress formulas, $\alpha = \dfrac{A_{wz}}{A_g}$; $\beta = \dfrac{A_{wz}}{A_e}$; $\chi = \dfrac{A_{wz}}{A_f}$; $\delta = \dfrac{A_{wzc}}{A_{gc}}$

Table 6.5.1-14—Strength Formulas: Connections

Eq. # (Article)	Limit State		Design Strength
Eq. 6.5.1-63 (6.14.4 item 1)	Aluminum Bolt Tension Strength	Tensile Rupture	$\phi R_n = \phi F_{tu} \left(\pi \frac{\left(D - \frac{1.191}{n}\right)^2}{4} \right), \quad \phi = 0.65$
Eq. 6.5.1-64 (6.14.4 item 1)	ASTM A325 Steel Bolt Tension Strength	Tensile Rupture	$\phi R_n = \phi F_{tu} (0.75 A_b), \quad \phi = 0.75$
Eq. 6.5.1-65 (6.14.4 item 2) (6.14.6)	Aluminum Bolt Shear Strength (see Article 6.14.6 for long grip bolted connections)	Threads within Shear Plane	$\phi R_n = \phi F_{su} \left(\pi \frac{\left(D - \frac{1.191}{n}\right)^2}{4} \right), \quad \phi = 0.65$
Eq. 6.5.1-66 (6.14.3) (6.14.6)	Aluminum Bolt Shear Strength (see Article 6.14.6 for long grip bolted connections)	Threads outside of Shear Plane	$\phi R_n = \phi F_{su} \left(\pi \frac{D^2}{4} \right), \quad \phi = 0.65$
Eq. 6.5.1-67 (6.14.4 item 2) (6.14.5 item 1)	ASTM A325 Bolt Shear Strength (slip critical connection)	Threads within Shear Plane	$\phi \mathrm{R_n} = \phi \mathrm{F_n A_b}, \quad \phi = 0.75, \quad \mathrm{F_n} = 48\ \mathrm{ksi}$
Eq. 6.5.1-68 (6.14.4 item 3) (6.14.5 item 1)	ASTM A325 Bolt Shear Strength (slip critical connection)	Threads outside of Shear Plane	$\phi \mathrm{R_n} = \phi \mathrm{F_n A_b}, \quad \phi = 0.75, \quad \mathrm{F_n} = 60\ \mathrm{ksi}$
Eq. 6.5.1-69 (6.14.5 item 3)	Bolted Slip Resistance	Strength Based	$\phi R_n = \phi 1.13 \mu\, h_{sc}\, T_b\, N_s, \quad \phi = 0.85$ $\mu,\ h_{sc},\ T_b \equiv$ see Article 6.14.5
Eq. 6.5.1-70 (6.14.5 item 2)	Bolted Slip Resistance	Serviceability Based	$\phi R_n = \phi 1.13 \mu\, h_{sc}\, T_b\, N_s, \quad \phi = 1.0$ $\mu,\ h_{sc},\ T_b \equiv$ see Article 6.14.5
Eq. 6.5.1-71 (6.14.5)	Rivet Shear Strength		$\phi R_n = \phi F_{su} \left(\pi \frac{D_h^{\ 2}}{4} \right), \quad \phi = 0.65$
Eq. 6.5.1-72 (6.14.8)	Groove Weld	Tensile or Compression Strength	$\phi R_n = \phi (F_{tuw}\, A_{we}), \quad \phi = 0.75$ $F_{tuw},\ A_{we} \equiv$ see Article 6.14.8
Eq. 6.5.1-73 (6.14.8)	Groove Weld	Shear Strength	$\phi R_n = \phi (F_{suw}\, A_{we}), \quad \phi = 0.75$ $F_{suw},\ A_{we} \equiv$ see Article 6.14.8

Continued on next page

Table 6.5.1-14—Strength Formulas: Connections—*Continued*

Eq. # (Article)	Limit State		Design Strength
Eq. 6.5.1-74 (6.14.9)	Fillet Weld	All Load Conditions	$\phi R_n = \phi(F_{sw} L_{we}), \quad \phi = 0.75$ $F_{sw,} L_{we} \equiv$ see Article 6.14.10
Eq. 6.5.1-75 (6.14.10 item 1)	Block Shear	Bolted Connection	If $F_{tu} A_{nt} \geq F_{su} A_{nt}$ then $\phi R_n = \phi(F_{sy} A_{gv} + F_{tu} A_{nt}), \quad \phi = 0.75$ else, if $F_{su} A_{nt} > F_{tu} A_{nt}$ then $\phi R_n = \phi(F_{su} A_{nv} + F_{ty} A_{gt}), \quad \phi = 0.75$
Eq. 6.5.1-76 (6.14.10 item 2)	Block Shear	Welded Connection	If $F_{tu} A_{gt} \geq F_{su} A_{gt}$ then $\phi R_n = \phi(F_{sy} A_{gv} + F_{tu} A_{gt}), \quad \phi = 0.75$ else, if $F_{su} A_{gt} > F_{tu} A_{gt}$ then $\phi R_n = \phi(F_{su} A_{gv} + F_{ty} A_{gt}), \quad \phi = 0.75$

$D \equiv$ nominal bolt diameter; $A_b \equiv$ nominal bolt cross sectional area; $N_s \equiv$ number of slip/shear planes in the connection; $D_h \equiv$ nominal diameter of a fastener hole; $d_{Leff}, d_t \equiv$ see Article 6.14.8; $n \equiv$ thread count per in.

6.5.2—Buckling Constant Formulas

Buckling constants, B, C, and D, for the design stress formulas of Table 6.5.1-2 through Table 6.5.1-11 shall be determined using Table 6.5.2-1 and Table 6.5.2-2 as required by temper and weld conditions. *Post-buckling constants*, k_1 and k_2, are likewise provided in Table 6.5.2-1 and Table 6.5.2-2 as required by temper and weld conditions.

Table 6.5.2-1—Formulas for Buckling Constants for Products Whose Temper Designation Begins with -O, -H, -T1, -T2, -T3, or –T4, and Weld-Affected Zones of All Tempers

Types of Members and Stress	**Intercept** (ksi)	**Slope** (ksi)	**Intersection**
Compression in Columns and Beam Flanges	$B_c = F_{cy}\left[1+\left(\frac{F_{cy}}{1000\kappa}\right)^{\frac{1}{2}}\right]$	$D_c = \frac{B_c}{20}\left(\frac{6B_c}{E}\right)^{\frac{1}{2}}$	$C_c = \frac{2B_c}{3D_c}$
Axial (Uniform) Compression in Flat Elements	$B_p = F_{cy}\left[1+\left(\frac{F_{cy}}{440\kappa}\right)^{\frac{1}{3}}\right]$	$D_p = \frac{B_p}{20}\left(\frac{6B_p}{E}\right)^{\frac{1}{2}}$	$C_p = \frac{2B_p}{3D_p}$
Axial (Uniform) Compression in Curved Elements/Round Tubes	$B_t = F_{cy}\left[1+\left(\frac{F_{cy}}{6500\kappa}\right)^{\frac{1}{5}}\right]$	$D_t = \frac{B_t}{3.7}\left(\frac{B_t}{E}\right)^{\frac{1}{3}}$	C_t = see note b
Bending Compression in Flat Elements	$B_{br} = 1.3F_{cy}\left[1+\left(\frac{F_{cy}}{340\kappa}\right)^{\frac{1}{3}}\right]$	$D_{br} = \frac{B_{br}}{20}\left(\frac{6B_{br}}{E}\right)^{\frac{1}{2}}$	$C_{br} = \frac{2B_{br}}{3D_{br}}$
Bending Compression in Curved Elements/Round Tubes	$B_{tb} = 1.5F_{cy}\left[1+\left(\frac{F_{cy}}{6500\kappa}\right)^{\frac{1}{5}}\right]$	$D_{tb} = \frac{B_{tb}}{2.7}\left(\frac{B_{tb}}{E}\right)^{\frac{1}{3}}$	$C_{tb} = \left(\frac{B_{tb}-B_t}{D_{tb}-D_t}\right)^2$
Shear in Flat Elements	$B_s = F_{sy}\left[1+\left(\frac{F_{sy}}{240\kappa}\right)^{\frac{1}{3}}\right]$	$D_s = \frac{B_s}{20}\left(\frac{6B_s}{E}\right)^{\frac{1}{2}}$	$C_s = \frac{2B_s}{3D_s}$
Post Buckling Constants (flat elements in compression)	$k_1 = 0.50, \quad k_2 = 2.04$		
Post Buckling Constants (flat elements in flexure)	$k_1 = 0.50, \quad k_2 = 2.04$		

a. $\kappa = 1.0$ ksi

b. C_t shall be determined using a plot of curves of limit state stress based on elastic and inelastic buckling or by trial-and-error solution.

Table 6.5.2-2—Formulas for Buckling Constants for Products Whose Temper Designation Begins with T5, T6, T7, T8, or T9

Types of Members and Stress	Intercept (ksi)	Slope (ksi)	Intersection
Compression in Columns and Beam Flanges	$B_c = F_{cy}\left[1+\left(\frac{F_{cy}}{2250\kappa}\right)^{\frac{1}{2}}\right]$	$D_c = \frac{B_c}{10}\left(\frac{B_c}{E}\right)^{\frac{1}{2}}$	$C_c = 0.41\frac{B_c}{D_c}$
Axial (Uniform) Compression in Flat Elements	$B_p = F_{cy}\left[1+\left(\frac{F_{cy}}{1500\kappa}\right)^{\frac{1}{3}}\right]$	$D_p = \frac{B_p}{10}\left(\frac{B_p}{E}\right)^{\frac{1}{2}}$	$C_p = 0.41\frac{B_p}{D_p}$
Axial (Uniform) Compression in Curved Elements/Round Tubes	$B_t = F_{cy}\left[1+\left(\frac{F_{cy}}{50{,}000\kappa}\right)^{\frac{1}{5}}\right]$	$D_t = \frac{B_t}{4.5}\left(\frac{B_t}{E}\right)^{\frac{1}{3}}$	C_t = see note b
Bending Compression in Flat Elements	$B_{br} = 1.3F_{cy}\left[1+\left(\frac{F_{cy}}{340\kappa}\right)^{\frac{1}{3}}\right]$	$D_{br} = \frac{B_{br}}{20}\left(\frac{6B_{br}}{E}\right)^{\frac{1}{2}}$	$C_{br} = \frac{2B_{br}}{3D_{br}}$
Bending Compression in Curved Elements/Round Tubes	$B_{tb} = 1.5F_{cy}\left[1+\left(\frac{F_{cy}}{50{,}000\kappa}\right)^{\frac{1}{5}}\right]$	$D_{tb} = \frac{B_{tb}}{2.7}\left(\frac{B_{tb}}{E}\right)^{\frac{1}{3}}$	$C_{tb} = \left(\frac{B_{tb}-B_t}{D_{tb}-D_t}\right)^2$
Shear in Flat Elements	$B_s = F_{sy}\left[1+\left(\frac{F_{sy}}{800\kappa}\right)^{\frac{1}{3}}\right]$	$D_s = \frac{B_s}{10}\left(\frac{B_s}{E}\right)^{\frac{1}{2}}$	$C_s = 0.41\frac{B_s}{D_s}$
Post Buckling Constants (flat elements in compression)	$k_1 = 0.35, \quad k_2 = 2.27$		
Post Buckling Constants (flat elements in flexure)	$k_1 = 0.50, \quad k_2 = 2.04$		

a. $\kappa = 1.0$ ksi

b. C_t shall be determined using a plot of curves of limit state stress based on elastic and inelastic buckling or by trial-and-error solution.

6.6—TENSION DESIGN STRESS— AXIAL LOADING

6.6.1—Limit States

Tension design strength for members subjected to axial loading through their centroid shall be the lesser of:

- the factored yield stress applied to the gross section of the member,
- the factored ultimate or fracture stress applied to the net section of the member.

See Figure 6.6.1-1.

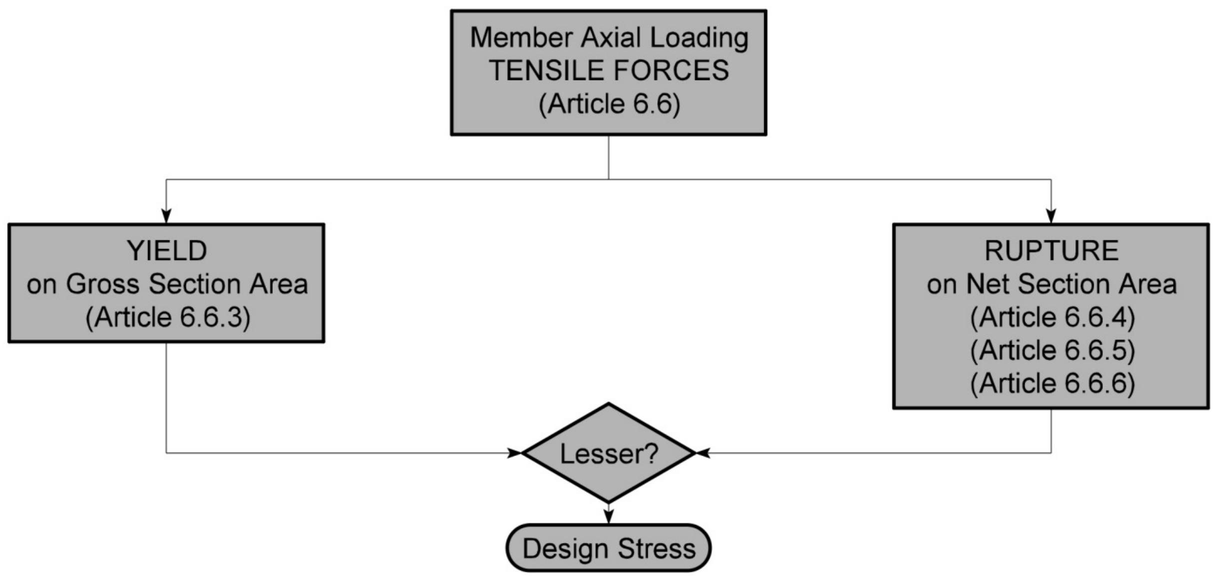

Figure 6.6.1-1—Logical Design Flow for Tension Design Stress—Axial Loading

6.6.2—Tension Coefficient

The tension coefficient, k_t, is utilized in tension formulas within Table 6.5.1-1.

C6.6.2

Alloys-tempers with

$$\frac{notch - strength}{yield - strength} < 1.0$$

will rupture at a notch before yielding (Kaufman, 2001). Such alloys-tempers require a reduction in tensile ultimate strength used for design. The tension coefficient, k_t, provides the means for this compensation.

For alloys-tempers not found in Table 6.4.2-1 and Table 6.4.2-2, see Table A.3.1 of the ADM.

Table 6.6.2-1—Tension Coefficient k_t

Alloy and Temper	Unwelded	Weld-Affected Zone
6005-T5, 6105-T5	1.25	—
All Others Listed in Table 6.4.1-1 and Table 6.4.2-2	1.0	1.0

6.6.3—Design Stress in the Gross Area

The design stress for member tension governed by nominal yield stress applied to the gross section area of the member is given in Eq. 6.5.1-1.

6.6.4—Design Stress in the Net Area

The design stress for member tension governed by nominal ultimate stress applied to the net section area of the member is given in Eq. 6.5.1-2.

C6.6.4

The net section area generally exists over a short length of the overall length of the member. The elongation of the member resulting from yielding across the net section is small. Yielding on the net section area is not considered a governing limit state.

6.6.5—Net Area

Net Section Area, A_n, of the member shall be computed as a function of all section elements. Specifically,

$$A_n = \sum_{i=1 \rightarrow all} t_i \, w_i \qquad (6.6.5\text{-}1)$$

where:

t_i = thickness of the i^{th} element
w_i = least net width of the i^{th} element

The least net width of the i^{th} element shall be computed by first identifying all chains of holes across its width, and for each separate chain subtracting the adjusted nominal hole diameters of drilled or reamed holes from the nominal gross width of the element.

- For a chain of in-line holes across the width of the element, the net element width shall be taken as the nominal element gross width and subtracting the adjusted nominal hole diameter for each of the holes of the chain.
- For a chain of staggered holes across the width of the element, the net element width shall be taken as the nominal element gross width and subtracting the adjusted nominal hole diameter for each of the holes in the chain and adding for each gauge space in the chain, the quantity $\left(\frac{s^2}{4g}\right)$

where:

s = the longitudinal center-to-center spacing (pitch) of any two consecutive holes (in.)

g = the center-to-center spacing (gauge) between fastener gauge lines (in.)

The least net width shall be the smallest net element width of all chains of holes.

For angles with holes in each of the opposing legs, the two elements can be treated as one for net section area computation. The nominal width of the combined elements shall be considered to be the flattened width of the angle defined to be the sum of the two element outside widths minus the element thickness,

$$w = a + b - t \qquad (6.6.5\text{-}2)$$

C6.6.5

The *adjusted nominal hole diameter* for net element width computation shall be taken as the nominal hole diameter for the drilled hole or reamed hole and adding $^1/_{32}$ in.

The gauge between holes in opposing legs shall be the sum of gauges between the holes and the common edge of the elements minus the element thickness,

$$g = g_a + g_b - t \quad (6.6.5\text{-}3)$$

where:

w = nominal width of the combined angle elements for computation of net width, (in.)

a, b = nominal individual outside widths of the two elements of the angle, (in.)

t = nominal thickness of the angle elements, (in.)

Gauge, g, and pitch, s, for the chain of holes spread across opposite legs of angles shall be computed as normal using the flattened width geometry as described above.

Weld metal in plug or slot welds shall not be included in the net area calculation.

6.6.6—Effective Net Area

The Effective Net Area, A_e, for angles, channels, tees, zees, and I-shaped sections shall be computed for members in tension.

- If tension is transmitted directly to all of the cross section elements of the member by fasteners or welds, the effective net area is equal to the net area.

$$A_e = A_n \quad (6.6.6\text{-}1)$$

- If tension is transmitted by fasteners or welds through some but not all of the cross-sectional elements of the member, the effective net area is computed as follows:

$$A_e = A_n \left(1 - \frac{\bar{x}}{L_c}\right)\left(1 - \frac{\bar{y}}{L_c}\right) \quad (6.6.6\text{-}2)$$

where:

L_c = length of the connection in the direction of the load, measured from the center of fasteners or the end welds. If $L_c = 0$, the net effective area is the net area of the connected elements (in.)

$\bar{x}$ = eccentricity of the connection in the x-axis direction (in.)

$\bar{y}$ = eccentricity of the connection in the y-axis direction (in.)

C6.6.6

Effective net area computation is a means to account for non-uniform stress distribution across the section of a member in tension that is the result of the connection geometry. Non-uniform stress distribution across the section at the connection for angles, tees, zees, and I-shaped sections is generally the by-product of some but not all of their section elements being utilized in the connection. This results in the effective section area in tension being less than the net section area (May and Menzemer, 2005).

For this equation to be valid:

$$\bar{x} \text{ and } \bar{y} < L_c \tag{6.6.6-3}$$

6.6.7—Slenderness Limit

For truss members in tension, the slenderness ratio shall be limited as follows:

$$\frac{L}{r} \leq 200 \tag{6.6.7-1}$$

where:

L = the length of the truss member in tension (in.)

r = the least radius of gyration of the truss member in tension (in.)

6.7—COMPRESSION DESIGN STRESS—AXIAL LOADING

The design compressive resistance of a member subjected to axial compression shall be the lesser of the factored nominal stress dictated by limit states for:

a) member buckling,

b) local buckling, and

c) interaction between member and local buckling.

See Figure 6.7-1.

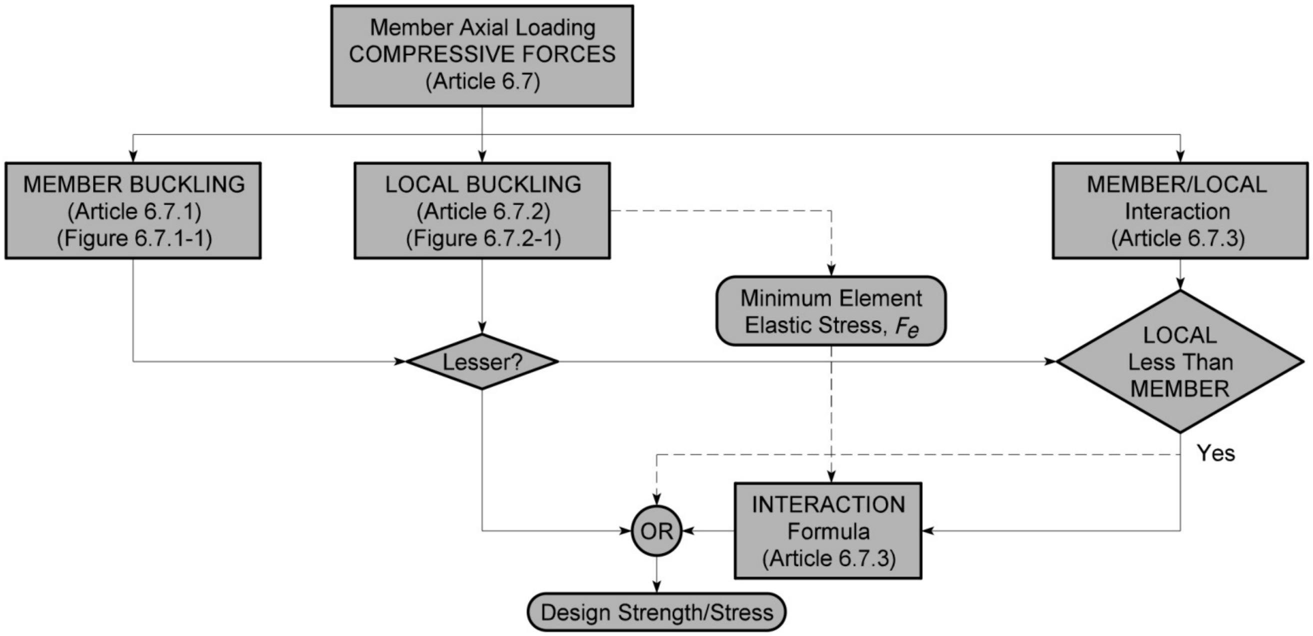

Figure 6.7-1—Logical Design Flow for Compression Design Stress—Axial Loading

6.7.1—Member Buckling

The design compressive stress governed by member buckling is given by Eq. 6.5.1-11 and diagrammed in Figure 6.7.1-1.

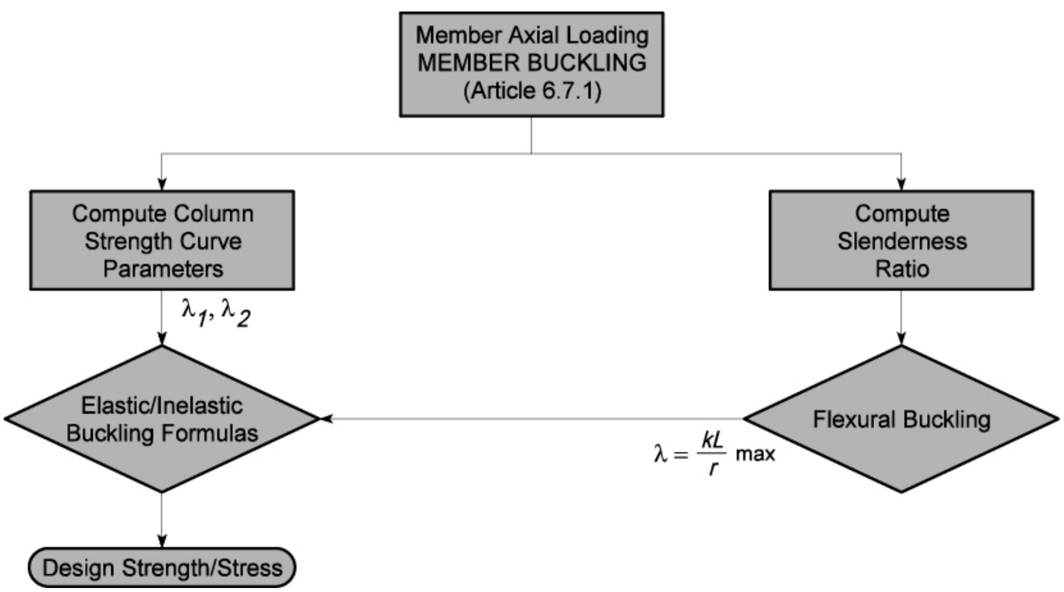

Figure 6.7.1-1—Logical Design Flow—Member Buckling

6.7.2—Local Buckling Stress

Local buckling limit state stresses shall be evaluated for all section elements of structural members that are subjected to compressive forces from member axial loading.

Limit states and associated design stress formulas are identified and tabularized separately for flat elements (Table 6.5.1-6) and for curved elements (Table 6.5.1-8).

The local buckling design strength shall be determined by either the weighted average method (Article 6.7.2.1) or the alternative method (Article 6.7.2.2).

6.7.2.1—Weighted Average Method

Weighted average design strength shall be computed as:

$$\phi_c P_n = \phi_c \sum_{i=1}^{n} F_{ci} A_i + \phi_c F_{cy} \left(A_g - \sum_{i=1}^{n} A_i \right), \quad \phi_c = 0.90 \tag{6.7.2.1-1}$$

where:

P_n = nominal axial compressive/local-buckling strength for the member subjected to axial load (kip)

F_{ci} = local compressive buckling stress of element i computed as described above (ksi)

F_{cy} = nominal compressive yield strength (ksi)

A_i, A_g = area of element i and member gross section area respectively (in.^{2})

C6.7.2

The local buckling strength of a member has been shown to equal the sum of the local buckling strength of each of a compression member's elements by Crockett (1942). In general, the portion of a cross section near the intersection of elements does not buckle and may be assumed to carry a stress of F_{cy} at the member's calculated maximum strength.

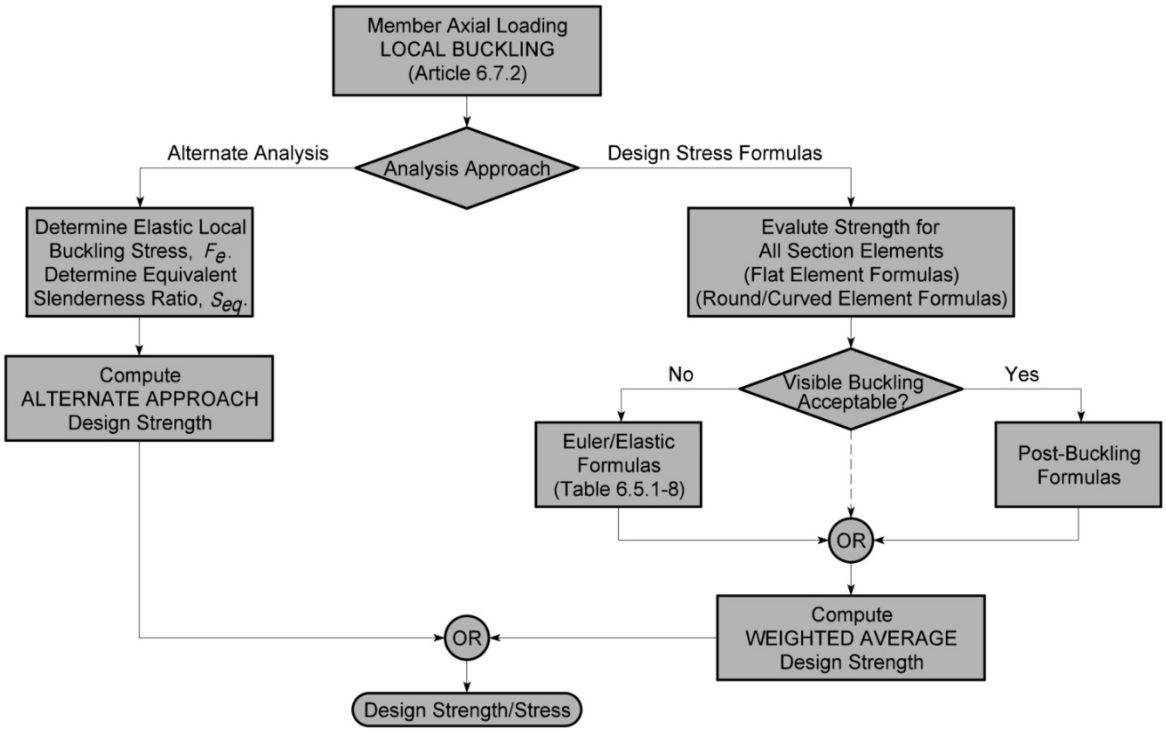

Figure 6.7.2-1—Logical Design Flow—Local Buckling

Table 6.7.2-1—Alternative Local Buckling Stress Formulas

Equations of Table 6.5.1-6 (Post-Buckling Design Stress)	**Optional (Euler Elastic Buckling Design Stress)**
Flat elements supported on one edge subjected to uniform compression: Eq. 6.5.1-26, $\left(\frac{b}{t} \geq \lambda_2\right)$	$\phi_c \frac{\pi^2 E}{(5.0\,\lambda)^2}, \quad \phi_c = 0.9$
Flat elements supported on two edges subjected to uniform compression: Eq. 6.5.1-27, $\left(\frac{b}{t} \geq \lambda_2\right)$	$\phi_c \frac{\pi^2 E}{(1.6\,\lambda)^2}, \quad \phi_c = 0.9$
Flat elements supported on one edge and with a stiffener on the other edge subjected to uniform compression:	$\phi_c \left[(1-\rho_{st})\frac{\pi^2 E}{(5.0\,\lambda)^2} + \rho_{st}\frac{\pi^2 E}{(1.6\,\lambda)^2}\right], \quad \phi_c = 0.9$

$$S_e = 1.28\sqrt{\frac{E}{F_y}}$$

$\rho_{st} = 1.0$ for $b/t \leq S_e/3$ where $S_e = 1.28\sqrt{\frac{E}{F_y}}$

$$\rho_{st} = \frac{r_s}{9t\left(\frac{b/t}{S_e} - \frac{1}{3}\right)}$$ for $S_e < b/t \leq S_e$

$$\rho_{st} = \frac{r_s}{1.5t\left(\frac{b/t}{S_e} + 3\right)}$$ for $S_e < b/t < 2S_e$

6.7.2.2—Alternative Method

Alternate design strength shall be computed as:

$$\phi_c P_n = \phi_c F_c A_g \qquad (6.7.2.2\text{-}1)$$

where:

P_n = nominal axial compressive/local-buckling strength for the member subjected to axial load (kip)

F_c = local compressive buckling stress given by Table 6.5.1-3 (ksi)

A_g = member gross section area (in.^{2})

The design stress formulas given in Eq. 6.5.1-26, Eq. 6.5.1-27, and Eq. 6.5.1-29 are post-buckling tolerant allowing localized buckling for the condition:

local slenderness ratio $\geq \lambda_2$.

For those cases where visible local buckling is unacceptable, the Euler based critical design stress formulas given in Table 6.7.2-1 shall be used in lieu of the post buckling-design stress formula.

C6.7.2.2

The critical design stress, F_{cr} , calculated for cases where:

local slenderness ratio $\geq \lambda_2$

using Eq. 6.5.1-26, Eq. 6.5.1-27, and Eq. 6.5.1-29 results in a post-buckling design stress. The post-buckling design stress can be significantly higher than design stress based on elastic buckling behavior of the element alone. Post-buckling coefficients, k_1 and k_2, reflect differences in the shape of stress strain curves for natural and artificially aged tempers and element loading conditions.

The ADM utilizes an average stress across the entire element width for calculating the post-buckling strength rather than an effective width. This generally results in somewhat simpler strength calculations (Galambos, 1987). For such cases where post-buckling design stress is specified, local buckling may be visible even though an adequate margin is provided by the formula against ultimate member failure.

6.7.3—Interaction between Member Buckling and Local Buckling

When the local buckling design stress is less than the member buckling design stress, the member design strength shall be computed as a function of both. Specifically,

if $F_e < F_c$,

then:

$$\phi_c P_n = \phi_c \left[\frac{0.85\pi^2 E}{(kL/r)^2} \right]^{\frac{1}{3}} F_e^{\frac{2}{3}} A_g , \qquad (6.7.3\text{-}1)$$

where:

P_n = nominal axial compressive/local-buckling strength for the member subjected to axial load, (kip)

F_e = the smallest nominal elastic buckling stress among the elements of the member's section (Article 6.7.2) (ksi),

F_c = local compressive buckling stress (Article 6.7.1) (ksi),

A_g = gross section area (in.^{2})

C6.7.3

When the elements of a column or beam buckle without overall buckling of the component, the stiffness and strength of the column or beam is reduced. In this case, the member fails at a stress that is between the local and global buckling strengths. The interaction equation provides an intermediate strength and has been shown to agree well with test data (Sharp, 1970).

6.7.4—Slenderness Limit

The slenderness limit for truss members subjected to axial compression shall not exceed 120.

$$\frac{kL}{r} \leq 120 \tag{6.7.4-1}$$

6.8—FLEXURAL DESIGN STRESS

6.8.1—Limit States

The design flexural strength of a member subjected to bending shall be identified by the following conditions:

- The member is loaded in a plane parallel to a principal axis that passes through the shear center, and
- The member is restrained against rotation about its longitudinal axis at load points and supports.

The design flexural strength shall be the lesser of the factored nominal stress for the following limit states:

1. Member lateral-torsional buckling,
2. Member yielding and rupture,
3. Local elements in tension,
4. Local elements in compression,
5. Weighted average flexural member strength, and
6. Interaction between local buckling and lateral-torsional buckling.

See Figure 6.8.1-1 and Figure 6.8.1-2.

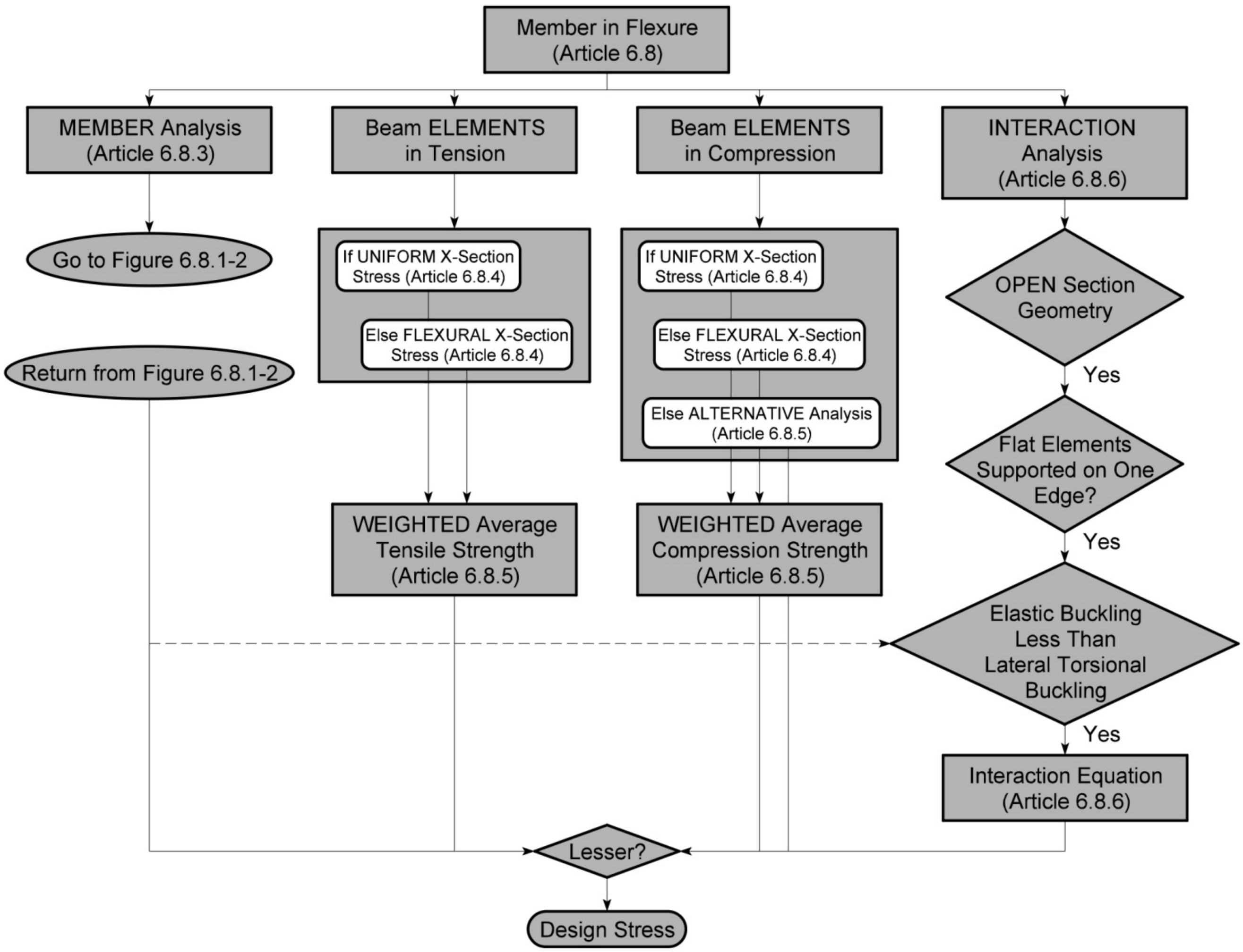

Figure 6.8.1-1—Logical Design Flow for Flexural Design Stress

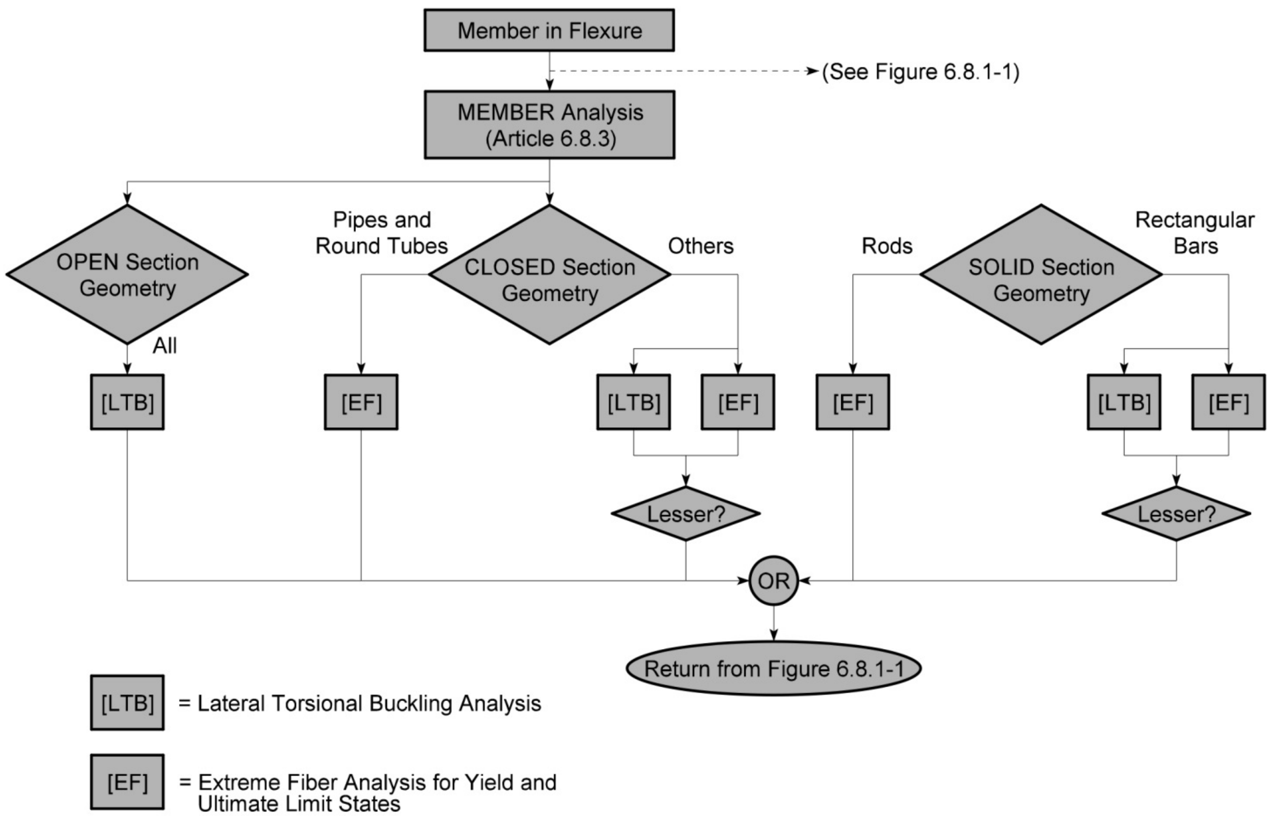

Figure 6.8.1-2—Logical Design Flow for Flexural Design Stress—Member Analysis

6.8.2—Bending Coefficient, C_b

The bending coefficient is determined by:

1. Members supported on both ends with uniform bending moment:

   $C_b = 1$

2. Cantilevers: reference 4), 5), and 6) below.
3. Doubly-symmetric shapes between brace points:

$$C_b = \frac{12.5M_{max}}{2.5M_{max} + 3M_A + 4M_B + 3M_C} \quad (6.8.2\text{-}1)$$

where:

M_{max} = absolute value of the maximum moment in the unbraced segment (in.-kip),

M_A = absolute value of the moment at the quarter point of the unbraced segment (in.-kip),

M_B = absolute value of the moment at the midpoint of the unbraced segment (in.-kip),

C6.8.2

C_b is applied to segments for beams between brace points. In general, lateral-torsional buckling strengths are based on a uniform moment over the unbraced length. If the actual moment over the unbraced length varies, the actual lateral-torsional buckling strength may be greater than the predicted strength. C_b attempts to account for this strength increase.

Reference ADM, Part I, Section F.1 for all other member and moment conditions.

M_C = absolute value of the moment at the three-quarter point of the unbraced segment (in.-kip).

4. Doubly-symmetric shapes unbraced at the free end with a concentrated load at the free end through the centroid:

$$C_b = 1.3$$

5. Doubly-symmetric shapes unbraced at the free end with a uniform transverse load through the centroid:

$$C_b = 2.1$$

6. For singly-symmetric shapes between brace points with $\frac{I_{cy}}{I_y} \leq 0.1 \quad or \quad \frac{I_{cy}}{I_y} \geq 0.9$,

$$C_b = 1$$

7. For singly-symmetric shapes between brace points with $0.1 < \frac{I_{cy}}{I_y} < 0.9$, use Eq. 6.8.2-1.

6.8.3—Members in Flexure

1. Open Shapes:

The design stress for lateral-torsional buckling (LTB) for open shapes is given by Eq. 6.5.1-15. The effective radius of gyration, r_{ye}, (in.) shall be taken as:

a. $$r_{ye} = \frac{1}{1.7}\sqrt{\frac{I_y d}{S_c}\sqrt{1 + 0.152\frac{J}{I_y}\left(\frac{L_b}{d}\right)^2}} \quad (6.8.3\text{-}1)$$

for shapes symmetric about the bending axis and between brace points of beams subject to end moment only or subject to transverse loads applied at the beam's neutral axis or at brace points.

b. $$r_{ye} = \frac{1}{1.7}\sqrt{\frac{I_y d}{S_c}\left(\pm 0.5 + \sqrt{1.25 + 0.152\frac{J}{I_y}\left(\frac{L_b}{d}\right)^2}\right)}$$

(6.8.3-2)

for shapes symmetric about the bending axis and between brace points of beams subject to transverse loads applied at the top or bottom flange, where the load is free to move laterally with the beam if the beam buckles. 0.5 is negative when the load acts toward the shear center and positive when the acting away from the shear center.

c. $r_{ye} = r_y$ for shapes symmetric about the bending axis but not fitting conditions a or b.

d. For singly-symmetric shapes that are asymmetric about the bending axis, or for other shapes asymmetric about the bending axis, see the ADM, Part I, Sections F.2.2.2 and F.2.2.3 respectively.

2. Closed Shapes—Pipes and Round Tubes:

   a. The design stress for pipes and round tubes for the limit state of local buckling is given by Eq. 6.5.1-35.

   b. The design stress for pipes and round tubes for the limit state of flexural compressive yielding is given by Eq. 6.5.1-20.

   c. The design stress for pipes and round tubes for the limit state of flexural tensile yielding is given by Eq. 6.5.1-7.

   d. The design stress for pipes and round tubes for the limit state of flexural tensile rupture is given by Eq. 6.5.1-8.

3. Closed Shapes—All except Pipes and Round Tubes:

   a) The design stress for closed shapes subject to lateral-torsional buckling is given by Eq. 6.5.1-16.

   b) The member design stress for other than pipes and round tubes for the limit states of flexural tensile yield, flexural tensile rupture, and flexural compressive yield shall be determined by appropriate element evaluation.

4. Solid Shapes—Rectangular Bars

   a. The design stress for solid rectangular bars with major axis bending subject to lateral-torsional buckling is given by Eq. 6.5.1-17.

   b. The design stress for rectangular bars for the limit state of flexural compressive yielding is given by Eq. 6.5.1-21.

   c. The design stress for rectangular bars for the limit state of flexural tensile yielding is given by Eq. 6.5.1-9.

d. The design stress for rectangular bars for the limit state of flexural tensile rupture is given by Eq. 6.5.1-10.

5. Solid Shapes—Rods

   a. The design stress for rods for the limit state of flexural compressive yielding is given by Eq. 6.5.1-21.

   b. The design stress for rods for the limit state of flexural tensile yielding is given by Eq. 6.5.1-9.

   c. The design stress for rods for the limit state of flexural tensile rupture is given by Eq. 6.5.1-10.

6. Single Angles:

   a. The design stress for single angles if a leg tip is a point of maximum compression is given by Eq. 6.5.1-18.

   b. The design stress for single angles if a leg is in uniform compression is given by Eq. 6.5.1-19.

6.8.4—Beam Elements Local Buckling

Local buckling limit state stresses shall be evaluated for all section elements of structural members that are subjected to compressive and tensile forces from flexural loading.

1. Beam Elements under Uniform Tension

   a. The design stress for the limit state of tensile yielding is given by Eq. 6.5.1-3.

   b. The design stress for the limit state of tensile rupture is given by Eq. 6.5.1-4.

2. Beam Elements in Flexure Tension (Non-Uniform)

   a. The design stress for the limit state of tensile yielding is given by Eq. 6.5.1-5.

   b. The design stress for the limit state of tensile rupture is given by Eq. 6.5.1-6.

3. Flat Beam Elements under Uniform Compression: The design stress for beam elements subjected to uniform compression is given by the appropriate Equations in Table 6.5.1-6.

4. Flat Beam Elements in Flexure Compression (Non-Uniform): The design stress for beam elements

subjected to flexural compression is given by the appropriate Equations in Table 6.5.1-7.

6.8.5—Weighted Average Flexural Member Strength

Using the element design stresses determined in Article 6.5.1, the member flexural strength shall be computed as the lesser of:

1. The weighted average of the beam elements design stress for elements subjected to compressive forces, given here and by Eq. 6.5.1-22:

$$\phi M_{nc} = \phi_b \left(\frac{F_c\, I_f}{c_{cf}} + \frac{F_b\, I_w}{c_{cw}} \right) \tag{6.8.5-1}$$

where:

F_c = local buckling stress of the flat elements in uniform compression (ksi)

F_b = local buckling stress of the flat elements in flexural compression (ksi)

c_{cf} = distance from the centerline of the compression flange to the cross section's neutral axis (in.)

c_{cw} = distance from the extreme compression fiber to the cross section's neutral axis (in.)

I_f = moment of inertia of the flange group about the cross section's neutral axis consisting of the flat elements in uniform compression and the flat elements in uniform tension and their edge or intermediate stiffeners (in.^{4})

I_w = moment of inertia of the web group about the cross section's neutral axis consisting of the flat elements in flexure and their intermediate stiffeners (in^4)

2. The weighted average of the beam elements design stress for elements subjected to tensile forces, given here and in Eq. 6.5.1-23:

$$\phi M_{nt} = \phi_b \left(\frac{F_t\, I_f}{c_{tf}} + \frac{F_b\, I_w}{c_{tw}} \right) \tag{6.8.5-2}$$

where:

F_t = tensile stress for the flat elements in uniform tension (ksi)

F_b = tensile stress of the flat elements in flexural tension (ksi)

c_{tf} = distance from the extreme tension fiber to the cross section's neutral axis,(in.)

C6.8.5

The weighted average flexural strength is taken as the lesser of the bending strength based on buckling of the compressed elements or the bending strength based on the elements in tension. This approach is based on the work of Jombock and Clark (1968) on formed sheet beams, and was improved at a later date by Kim (2003).

c_{tw} = distance from the web group's extreme tension fiber to the cross section's neutral axis (in.)

I_f and I_w are as defined above (in.^{4})

6.8.6—Interaction—Local Buckling and Lateral–Torsional Buckling (LTB)

Interaction design strength shall be investigated for open shape members given the following conditions are true:

1. Flanges are flat elements supported on one edge.
2. The flange's elastic buckling design stress determined in Article 6.8.4 is less than the LTB determined in accordance with Article 6.8.3.

If these conditions are true, the member flexural design strength shall be limited to:

$$\phi M_n \le \phi_b \left[\frac{\pi^2 E}{\left(\dfrac{L_b}{1.2 r_{ye} \sqrt{C_b}} \right)^2} \right]^{\frac{1}{3}} F_e^{\frac{2}{3}} S_c, \quad \phi_b = 0.9 \tag{6.8.6-1}$$

C6.8.6

When the elements of a column or beam buckle without overall buckling of the component, the stiffness and strength of the column or beam is reduced. In this case, the member fails at a stress that is between the local and global buckling strengths. The interaction equation provides an intermediate strength and has been shown to agree well with test data (Sharp, 1970).

Using r_y yields a very conservative estimate for moderate and high slenderness ratios. Sharp (1993) showed that using an effective slenderness ratio, r_{ye}, is more accurate.

6.9—BEARING DESIGN STRESS

Bearing design stress shall be determined for bolts (or pins) in round or slotted holes, rivets in round holes, screws in round holes, and for bearing on flat surfaces.

6.9.1—Bolts, Pins, and Screws—Bearing

Bearing design stress for *bolts*, *pins*, and *screws* are a function of the nominal fastener diameter, D, to edge distance, d_e, ratio, $\frac{d_e}{D}$, where d_e shall be measured from the center of the bolt to the edge of the component being fastened in the direction of the force. The thickness of the connected element is taken to be t. The bearing area shall then be defined to be $D \cdot t$, or the nominal bolt diameter times the connected element thickness.

C6.9.1

The limit state for bolt bearing is reached when elongation of the fastener hole is excessive (Menzemer et al., 2001). When the bearing stress given by Eq. 6.5.1-38 is achieved and no rupture occurs, the hole may demonstrate excessive elongation. If hole elongation is not desirable, then the design should provide for higher bearing design stress.

The specified bearing design stress assumes adequate shear strength of the bolt, pin, or rivet.

The bearing design stress for a slotted bolt/pin shall be determined to prevent overstressing the material between the slot and the edge of the component being fastened.

6.9.1.1—Bolt in Round Hole

For a bolt in a round hole, the bearing design stress shall be determined using Eq. 6.5.1-38.

6.9.1.2—Bolt in Slotted Hole

For a bolt in a slotted hole, the bearing design stress shall be given by Eq. 6.5.1-39.

6.9.1.3—Rivet in Round Hole

For a rivet in a round hole, the bearing design stress shall be determined using Eq. 6.5.1-41.

6.9.1.4—Pin in Round Hole

For a pin in a round hole, the bearing design stress shall be determined using Eq. 6.5.1-41.

6.9.1.5—Screw in Round Hole

For a screw in a round hole, the bearing design stress shall be determined using Eq. 6.5.1-42.

6.9.2—Rivets—Bearing

For a rivet in a round hole, the bearing design stress shall be determined using Eq. 6.5.1-40 and is a function of the nominal hole diameter, D_h, to edge distance, d_e, ratio, $\frac{d_e}{D_h}$,

where:

d_e shall be measured from the center of the rivet to the edge of the component being fastened in the direction of the force. The bearing surface/area, A_{bearing}, for material with nominal thickness, t, shall be defined as:

$$A_{\text{bearing}} = D_h \alpha t$$

For ordinary holes: $\alpha = 1.0$

For holes countersunk to a depth, d_{cs}: $\alpha = 1 - \frac{d_{cs}}{2}$

C6.9.2

Rivet-hole sizing assumes the rivet completely fills the hole.

6.9.3—Flat Surfaces—Bearing

The bearing design stress for flat surfaces in contact shall be determined using Eq. 6.5.1-43. The bearing design stress shall be applied to the projected bearing area.

6.10—SHEAR DESIGN STRESS

6.10.1—Flat Webs Supported on Both Edges—Shear

The shear design stress for flat webs supported on both edges subjected to shear in the plane of the web is given by Eq. 6.5.1-36.

C6.10.1

For stiffened webs refer to the ADM, Part I, Section G.2 for the definition of λ.

The web slenderness ratio shall be taken as $\lambda = \frac{b}{t}$

where:

b = the clear height of the web for unstiffened webs (in.)

t = the web thickness (in.)

6.10.2—Round and Oval Tubes—Shear

The shear design stress for round or oval tubes is given by Eq. 6.5.1-37. The tube slenderness ratio shall be taken as:

$$\lambda = 2.9\left(\frac{R_b}{t}\right)^{\frac{5}{8}}\left(\frac{L_v}{R_b}\right)^{\frac{1}{4}} \qquad (6.10.2\text{-}1)$$

where:

R_b = mid-thickness radius of a round tube, or maximum mid-thickness of an oval tube (in.)

L_v = length of tube from maximum to zero shear force, (in.)

t = thickness of the tube (in.)

6.11—TORSION DESIGN STRESS

6.11.1—Round and Oval Tubes—Torsion

The torsional design stress for round or oval tubes for the limit state of torsional yielding and torsional buckling is given by Eq. 6.5.1-44. The tube slenderness ratio shall be taken as:

$$\lambda = 2.9\left(\frac{R_b}{t}\right)^{\frac{5}{8}}\left(\frac{L_v}{R_b}\right)^{\frac{1}{4}} \qquad (6.11.1\text{-}1)$$

where:

R_b = mid-thickness radius of a round tube, or maximum mid-thickness of an oval tube (in.)

L_v = length of tube from maximum to zero shear force (in.)

t = thickness of the tube (in.)

The torsional design strength shall be computed as:

$$T = \frac{F_s J}{R} \qquad (6.11.1\text{-}2)$$

where:

R = outside diameter of the tube (in.)

J = torsion constant of the tube (in.^{4})

6.11.2—Rectangular Tubes—Torsion

The torsional design stress for rectangular tubes for the limit states of torsional yielding and torsional buckling is given by Eq. 6.5.1-45. The member design stress for the rectangular tube shall be governed by the side of the rectangular tube with the highest slenderness ratio.

The side slenderness ratio shall be taken as $\lambda = \dfrac{b}{t}$

where:

b = clear width of the side for unstiffened sides (in.)

t = side thickness (in.)

The torsion design strength for the rectangular tube as a member is given by Eq. 6.5.1-46.

$$\phi_T T_n = \phi_T F_s C, \quad \phi_T = 0.90$$

C = torsional shear constant which may be conservatively taken as:

$$C = 2(b-t)(d-t)t - 4.5(4-\pi)t^3 \qquad (6.11.2\text{-}1)$$

where:

b = width of the tube (in.)

d = depth of the tube (in.)

C6.11.2

For stiffened sides refer to ADM, Part I, Section G.2 for the definition of λ.

6.11.3—Rods—Torsion

The torsional design strength for rods for the limit state of torsional yielding is given by Eq. 6.5.1-47 where:

F_{sy} = shear yield strength (see Article 6.4) (ksi) and

d = diameter of the rod (in).

6.12—DESIGN STRESSES FOR COMBINED FORCES

Members subjected to combined bending, axial tension or compression, shear, and torsion shall be proportioned to meet the limitations of Articles 6.12.2 through 6.12.5 as applicable. Additional bending due to second order effects of axial compressive loading acting through the deformation, P–δ, and/or lateral deflection of the joints of a member, P–Δ, shall be considered.

6.12.1—Second-Order Load Amplification Factor

The modified second-order load amplification factor, B_2, is provided in Article 4.8.

6.12.2—Axial Compression, Flexure, and Shear–Vertical Cantilever Pole Supports

Vertical cantilever poles subjected to axial compression, bending, and shear shall be governed by Eq. 6.5.1-48.

The amplification factor, B_2, shall be calculated in accordance with Article 4.8 to account for the second order bending effects.

6.12.3—Axial Compression, Flexure, and Shear

Members (except for vertical cantilever pole-type supports and square tubes subject to biaxial bending) subjected to axial compression, bending, shear, and torsion shall be governed by Eq. 6.5.1-49.

6.12.4—Axial Tension, Flexure, and Shear

Members subjected to axial tension, bending, and shear shall be governed by Eq. 6.5.1-50.

6.12.5—Square Tubes Subject to Axial Compression, Biaxial Bending, and Shear

Square tubes used for vertical cantilever supports subjected to biaxial bending shall be governed by Eq. 6.5.1-51.

For tubes with element slenderness $\lambda \leq \lambda_1$, the design stress shall be taken as $\phi F_b = \phi F_{cy}$ with α=1.6. For tube elements with intermediate slenderness $\lambda_1 < \lambda \leq \lambda_2$, the design stress, ϕF_b, shall be determined using Table 6.5.1-12 with α=1.

C6.12.5

Menzemer et. al. (2009) compared theoretical diagonal bending to experimental results. The interaction increase in the design stress is justified for square tubes bent about a diagonal axis for sections with limited width-thickness ratios. For compact sections, the reserve strength is 33 percent higher for bending about a diagonal axis $\left(\frac{Z_x}{S_x} = 1.5\right)$ than about the principal axes $\left(\frac{Z_x}{S_x} = 1.13\right)$.

6.12.6—Crippling and Bending of Flat Webs

The design formula for the condition of combined flexural bending and concentrated loads connected transverse to the axis of the member is given in the ADM Section J.8.3 and shall be used when the concentrated load is applied at a distance of one-half or more of the member depth from the member end.

6.13—WELDED MEMBERS

Nominal strengths for the most commonly used welded alloys are specified in Table 6.4.2-2.

1. For lighting poles fabricated from 6005 aluminum that are less than or equal to 0.25 in. thick, welded in the T1 temper with 4043 filler, and subsequently artificially aged to the T5 temper after welding, mechanical properties of the base metal within 1.0 in. of the weld shall

C6.13

The post-weld heat-treated strength of 6063 and 6005 light poles was studied in the early 1970s by the National Electric Manufacturers Association (NEMA); The Aluminum Association (AA); and aluminum pole manufacturers Hapco, IKG and P&K. The testing to support this was done by the Spun Aluminum Pole group of NEMA in the late 1960s. Results from the studies of the tensile yield and ultimate strengths of 6063-T4 welded with 4043 and post-weld aged to a T6 condition and 6005-T1 welded with 4043 and subsequently aged to be taken as 85 percent of the unwelded 6005-T5.

2. For lighting poles fabricated from 6063 aluminum that are less than or equal to 0.375 in. thick, welded in the T4 temper and subsequently artificially aged to the T6 temper after welding, mechanical properties of the base metal within 1.0 in. of the weld shall be taken as 85 percent of the un-welded 6063-T6.

The strength of welded aluminum alloy castings shall be those established using the AWS D1.2 weld procedure qualification test.

Nominal strengths of aluminum alloy filler metals are given in Table 6.4.3-1. Filler/alloy compatibility is given in Table 6.4.3-2.

a T5 temper demonstrated that base metal within 1 in. of the weld possessed strengths that could be taken to be equal to 85 percent of the unwelded 6063-T6 and 6005-T5 base metal. Specimens included extruded 6063 poles through 0.375 in. thick and 6005 poles through 0.25 in. thick. Unfortunately, the data were not documented in the open literature, although The Aluminum Association accepted the results and has made use of them in specifications for aluminum structures.

The strength of welded castings are not universally established and accepted; therefore AWS D1.2 testing is specified.

6.13.1—Weld-Affected Zone

The weld-affected zone, or heat-affected zone, shall be taken to extend 1 in. from the center of the weld.

6.13.2—Welded Member in Axial Tension

1. For *tensile yielding*:

   a. For transverse welds, the member can be assumed unwelded; the design stress shall be governed by nominal yield strengths of the member as given by Eq. 6.5.1-1 and shall be applied to the gross section area of the member.

   b. For longitudinal welds, the design stress shall be governed by the area-weighted average of nominal yield strengths of the unwelded, F_t, and the weld-affected, F_{tyw}, zones as given by Eq. 6.5.1-52 and shall be applied to the gross section area of the member.

2. For *tensile rupture*:

The design stress shall be governed by the area-weighted average of nominal ultimate tensile strength of the unwelded, F_{tu}, and weld-affected, F_{tuw}, zones as given by Eq. 6.5.1-53 and shall be applied to the effective net section. (See Article 6.6.5.)

C6.13.2

Note that for members with transverse welds, the welded-zone-area-to-net-effective-area ratio is:

$$\beta = \frac{A_{wz}}{A_e} = 1.0 \text{ for tensile rupture.}$$

6.13.3—Welded Member in Axial Compression

1. For *compression members with transverse welds:*

   a. If the member is supported at both ends with a transverse weld <0.05*L* from either end, then assume the entire cross section is not weld-affected and use Eq. 6.5.1-11.

C6.13.3

b. If the member is supported at both ends with a transverse weld $\geq 0.05L$ from the either end, then assume that the entire cross section is weld-affected and use Eq. 6.5.1-11.

c. If the member is supported at only one end with a transverse weld, then assume that the entire cross section is weld-affected and use Eq. 6.5.1-11.

d. If the member is a tube with a circumferential weld, then the local buckling formula given in Eq. 6.5.1-34 shall be applied only if $\frac{R_b}{t} \leq 20$.

If the constraints of Article 6.13.3 Item 1d are not satisfied, then a detailed strength analysis is required.

2. For *compression members with longitudinal welds*: The member buckling design stress shall be governed by the area-weighted average of nominal compressive strengths of the unwelded, F_{co}, and the weld-affected, F_{cw}, zones of the member as given by Eq. 6.5.1-54.

   The nominal compression strength, F_{co}, for the member shall be computed according to Article 6.5.1 using buckling constants for unwelded metal. Likewise, the nominal critical compression strength, F_{cw}, for the member shall be computed according to Article 6.5.1 using buckling constants for the weld-affected metal.

6.13.4—Welded Flexural Members

C6.13.4

1. For *flexural members with transverse welds*:

   a. If the member is supported at both ends with a transverse weld $< 0.05L$ from either end, then assume that the entire cross section is not weld-affected.

   b. If the member is supported at both ends with a transverse weld $\geq 0.05L$ from either end, then assume that the entire cross section is weld-affected and proceed as per Article 6.5.1.

   c. If the member is supported at only one end with a transverse weld, then assume that the entire cross section is weld-affected and proceed as per Article 6.5.1.

   d. If the member is a tube with a circumferential weld, then the local buckling formula given in Eq. 6.5.1-34 shall be applied only if $\frac{R_b}{t} \leq 20$.

If the constraints of Article 6.13.4 Item 1d are not satisfied, then a detailed strength analysis is required.

2. For *longitudinal welds subjected to flexure*:

The member flexural design stress shall be governed by the area-weighted average of nominal flexural strengths of the unwelded, F_{bo}, and weld-affected, F_{bw}, zones of the member as given by Eq. 6.5.1-55.

The area weighting factors of Table 6.5.1-13 are A_{wz}, the area of the weld-affected zone; and A_f, the area of the member farther than $\frac{2}{3}c$ from the neutral axis, where c

The nominal flexural strength $F_{bo} = \dfrac{M_{no}}{S}$ for the member shall be computed according to Article 6.5.1 using buckling constants and mechanical properties for unwelded metal. Likewise, the nominal flexural strength $F_{bw} = \dfrac{M_{nw}}{S}$ for the member shall be computed according to Article 6.8 using buckling constants and mechanical properties for the weld-affected metal.

is the distance from the neutral axis to the extreme compression fiber.

6.13.5—Welded Elements in Uniform Tension

For welded elements subjected to uniform tensile stress:

1. For *tensile yielding*:

The design tensile yield stress shall be governed by the area-weighted average of nominal tensile yield strengths of the unwelded , $F_{to} = F_{ty}$, and the weld-affected , $F_{tw} = F_{tyw}$, zones of the member as given by Eq. 6.5.1-58.

2. For *tensile rupture*:

The design tensile rupture stress shall be governed by the area-weighted average of nominal tensile rupture strengths of the unwelded , $F_{to} = F_{tu}$, and the weld-affected, $F_{tw} = F_{tuw}$, zones of the member as given by Eq. 6.5.1-59.

C6.13.5

Note that for elements with transverse welds, the welded-zone-area-to-gross-area ratio is:

$$\alpha = \frac{A_{wz}}{A_g} = 1.0$$

6.13.6—Welded Elements in Uniform Compression

For *welded elements subjected to uniform compression*:

The element design stress shall be governed by the area-weighted average of nominal uniform compressive strengths of the unwelded, F_{co}, and the longitudinally weld-affected, F_{cw}, zones of the element as given by Eq. 6.5.1-56. A transverse weld in the uniform compression zone shall not govern design yield strength.

The nominal uniform compression strength, $F_{co,}$ for the element shall be computed according to Article 6.7.2 using buckling constants and mechanical properties for unwelded metal. Likewise, the nominal uniform compression strength, F_{cw}, for the element shall likewise be computed according to Article 6.7.2 using buckling constants and mechanical properties for the weld-affected metal.

6.13.7—Welded Elements in Flexure

1. For *welded elements in flexural tensile stress*:

a. For *tensile yielding*:

The design tensile yield stress shall be governed by the area-weighted average of nominal tensile yield strengths of the unwelded, $F_{to} = 1.30\ F_{ty}$, and the weld-affected, $F_{tw} = 1.30\ F_{tyw}$, zones of the member as given by Eq. 6.5.1-60.

b. For *tensile rupture*:

The design tensile rupture stress shall be governed by the area-weighted average of nominal tensile rupture strengths of the unwelded, $F_{to} = 1.42\ F_{tu}$, and the weld-affected, $F_{tw} = 1.42\ F_{tuw}$, zones of the member as given by Eq. 6.5.1-61.

2. For *elements subjected to flexural compression stress*:

The element design stress shall be governed by the area-weighted average of nominal flexural compressive strengths of the unwelded, F_{bo}, and weld-affected, F_{bw}, zones of the element as given by Eq. 6.5.1-57.

The nominal flexural compressive strength, F_{bo}, for the element shall be computed according to Article 6.5.1 using buckling constants and material properties for unwelded metal. Likewise, the nominal flexural compression strength, F_{bw}, for the element shall be computed according to Article 6.8.3 using buckling constants and material properties for the weld-affected metal.

The resulting member weighted average design flexural strength based on welded elements shall be determined as usual according to Article 6.8.

C6.13.7

Note that for elements with transverse welds, the welded-zone-area-to-gross-area ratio is:

$$\alpha = \frac{A_{wz}}{A_g} = 1.0$$

Local buckling strength of elements subjected to flexure is not affected by welds in the tension zone of the element; therefore, the area-weighted average computation includes only the compression portion of the element.

6.13.8—Welded Member Subjected to Shear

The shear design stress shall be governed by the area-weighted average of nominal shear strengths of the unwelded and weld-affected zones of the section as given by Eq. 6.5.1-62.

The nominal shear strength, F_{so}, shall be computed using Article 6.5.1 using mechanical properties for unwelded metal. Likewise, the nominal shear strength, F_{sw}, shall be computed using Article 6.10 using mechanical properties for the weld-affected metal.

6.14—CONNECTIONS

The design strength of connections shall be determined as specified herein and in accordance with the ADM Chapters B and J.

Fasteners shall not be considered to share load in combinations with welds.

When the line of action of the resultant force does not coincide with the center of gravity of the fastener or weld group, the effect of the eccentricity shall be accounted for.

6.14.1—Bolt Material

Aluminum bolt material shall meet ASTM F486 and be 6061-T6 or 7075-T3. Nuts shall meet ASTM F467. Nuts for bolts larger than $^1/_4$ in. shall be 6061-T6 or 6262-T9. Flat washers shall be Alclad 2024-T4. Spring lock washers shall be 7075-T6.

Carbon steel bolts, nuts, and washers shall be galvanized by hot dip meeting ASTM A153 or by mechanical means meeting ASTM B695. Galvanized fasteners and nuts shall be lubricated in accordance with ASTM A563. ASTM A490 bolts shall not be used.

Stainless steel bolts, nuts, and washers shall be ASTM 300 series. Bolts shall meet ASTM F593, A193, or A320. Nuts shall meet ASTM F594 or A194.

6.14.2—Holes and Slots

The nominal diameter of a bolt hole shall not be more than $^1/_{16}$ in. greater than the nominal diameter of the fastener unless slip critical joints are used.

The width of slots for bolted connections shall not be more than $^1/_{16}$ in. wider than the nominal diameter of the bolt. If the nominal length of the slot is more than 2.5*D* (*D* = nominal bolt diameter) or the edge distance is less than 2*D*, the edge distance perpendicular to the length of the slot and slot length shall be sized to avoid overstressing the material along the slot. Bearing load connections should be made so that the action of the load is perpendicular to the slot. Slip critical connections can be made with the load action at any orientation to the slot.

C6.14.2

To avoid overstressing the material along the slot, the designer, at minimum, should check bearing, rupture, and beam action deformation on the edge side of the slot when the force action is perpendicular to the slot.

6.14.3—Minimum Spacing and Edge Distance of Bolts

The distance between bolt centers shall not be less than 2.5*D* (*D* = nominal bolt diameter).

The distance from the center of a bolt to the edge of a part shall not be less than 1.5*D*. For slotted holes, see Article 6.14.2.

See Article 6.9.1.2 for design bearing strength.

6.14.4—Bolted Connections

1. For *bolt tension,* the design strength of aluminum bolts, ϕR_n, is given in Eq. 6.5.1-63. The design strength for ASTM A325 steel bolts is given in Eq. 6.5.1-64. The given strength formulas are for the limit state of tensile rupture.

2. For *bolt shear with the threads within the shear plane,* the design strengths, ϕR_n, are given in Eq. 6.5.1-65 (aluminum) and Eq. 6.5.1-67 (ASTM A325 steel).

3. For *bolt shear with the threads outside of the shear plane,* the design strengths, ϕR_n, are given in Eq. 6.5.1-66 (aluminum) and Eq. 6.5.1-68 (ASTM A325 steel).

4. For *bolt bearing* see Article 6.9.

6.14.5—Slip-Critical Bolted Connections

Slip-critical bolted connections between aluminum members or between aluminum and steel members shall comply with the Research Council on Structural Connections (RCSC) *Specification for Structural Joints Using ASTM A325 or A490 Bolts* (2009) (RSRC Specification) except as noted herein and in the ADM Chapter J.

For slip-critical connections, the aluminum alloys used shall have tensile yield strengths of at least 15 ksi. Galvanized ASTM A325 bolts and nuts meeting ASTM A563 Grade DH or ASTM A194 Grade 2H, and galvanized washers meeting ASTM F436 shall be used. Washers shall be used under bolt heads and nuts.

Holes for slip-critical connections shall be standard holes, oversize holes, short-slotted holes, or long-slotted holes. Nominal dimensions for each hole type shall not exceed those found in the RCSC Specification. For a long slotted hole, see ADM Sections J.3.2 and J.3.8.5.

1. *Bolt Design Strength*—The A325 galvanized bolt shear design strength, ϕR_n, is given in Eq. 6.5.1-67 and Eq. 6.5.1-68 in accordance with ADM Section J.3.8.4.

2. *Slip Resistance*—Strength Limit State: When the connection is made with oversized holes or slots

C6.14.5

Slip coefficients for all conditions other than what is noted must be determined according to the RCSC Specification, Appendix A.

parallel to the direction of the load, the design strength, ϕR_n, shall be determined using Eq. 6.5.1-69.

3. *Slip Resistance*—Serviceability Limit State: When connections are made with standard holes or slotted holes transverse to the direction of the load, the design strength, ϕR_n, shall be determined using Eq. 6.5.1-70.

4. Bolt bearing design stress shall be determined according to Article 6.9.

The mean slip coefficient is $\mu = 0.5$ for aluminum alloys abrasion blasted with coal slag to an average substrate profile of 0.002 in. that are in contact with similar aluminum surfaces or steel surfaces coated with zinc-rich paint to a maximum dry film thickness of 0.004 in. The hole factor shall be defined as:

h_{sc} = 1.0 for standard holes,

h_{sc} = 0.85 for oversized and short-slotted holes,

h_{sc} = 0.7 for long-slotted holes oriented perpendicular to the direction of the load, and

h_{sc} = 0.6 for long-slotted holes oriented parallel to the direction of the load.

The fastener tension, T_b, shall be determined in accordance to RCSC Specification Table 8.1.

6.14.6—Long Grip Bolts

The bolt grip, G_f, is defined as the total thickness of parts being fastened. If for an aluminum bolt with diameter D:

$$G_f \geq 4.5D \tag{6.14.6-1}$$

the aluminum bolt shear design strength, ϕR_n (given by Eq. 6.5.1-65 or Eq. 6.5.1-66), shall be reduced by the following factor:

$$Factor = \frac{1}{\left[\frac{1}{2} + \frac{G_f}{9D}\right]} \tag{6.14.6-2}$$

6.14.7—Rivets

Rivet Material: Rivets shall be 300 series stainless steel or aluminum alloys conforming to ASTM B316. Carbon steel rivets shall not be used unless aluminum is joined to carbon steel and the structure is protected against corrosion as per Article 14.5.11.

Holes for Rivets: The finished diameter of holes, D_h, for cold driven rivets with nominal rivet diameter, D_{rivet}, shall be:

$$D_h \leq 1.04\,D_{rivet} \tag{6.14.7-1}$$

Minimum Rivet Spacing: The minimum spacing between centers, s, of rivets with nominal rivet diameter, D_{rivet}, shall be:

$$s \geq 3.0\,D_{rivet} \tag{6.14.7-2}$$

Minimum Rivet Edge Distance: The distance from the center of the rivet to an edge of a connected part, d_e, shall not be:

$$d_e \geq 1.5 D_{rivet} \tag{6.14.7-3}$$

Aluminum rivets shall not be used to carry tensile loads.

Rivet Shear Strength: The rivet design shear strength is given in Eq. 6.5.1-71.

Rivet Bearing Strength: The rivet design bearing strength is addressed in Article 6.9.2.

6.14.8—Groove-Weld Connections

Groove weld penetration shall be as defined in the ADM Section J.2.1.

The size, S_w, of a completed joint penetration groove weld is the thickness of the thinner part joined. The size, S_w, of a partial joint penetration groove weld is the depth of preparation for all V and bevel groove welds with an inclined angle >45 degrees. The size, S_w, of partial joint penetration groove welds is the depth of preparation for all J and U groove welds.

The *effective weld length*, L_{we}, for tension and compression is the length of the weld perpendicular to the direction of tensile or compressive stress. The effective weld length, L_{we}, for shear is the length of the weld parallel to the direction of the shear stress.

The *effective area,* A_{we}, of the groove weld shall be computed as:

$$A_{we} = S_w\,L_{we} \tag{6.14.8-1}$$

Groove Weld Design Strength: The groove weld tensile or compressive design strength as given in Eq. 6.5.1-72 is:

$$\phi R_n = \phi\left(F_{tuw}\,A_{we}\right), \quad \phi = 0.75 \tag{6.14.8-2}$$

where:

$F_{tuw} \equiv$ the least of the welded tensile ultimate strengths of the base metals and the filler.

Groove Weld Shear Strength: The groove weld shear design strength as given in Eq. 6.5.1-73 is:

$$\phi R_n = \phi\left(F_{suw} A_{we}\right), \quad \phi = 0.75 \qquad (6.14.8\text{-}3)$$

where:

$F_{suw} \equiv$ the least of the welded shear ultimate strengths of the base metals and the filler.

6.14.9—Fillet-Weld Connections

The *effective throat depth*, d_t, for a fillet weld with equal-sized legs, S_w, shall be taken as:

$$d_t = \frac{\sqrt{2}}{2} S_w \qquad (6.14.9\text{-}1)$$

Otherwise, $d_t \equiv$ the shortest distance from the root of the weld to the face of the weld.

The *effective length*, L_{we}, of a weld shall be taken as the overall length of the weld, L_w, (including the boxing):

$$L_{we} = L_w \qquad (6.14.9\text{-}2)$$

If however,

$$L_{we} \leq 4\, S_w, \quad then \quad S_w = \frac{L_{we}}{4} \qquad (6.14.9\text{-}3)$$

Intermittent Fillet Welds: The length of segments of intermittent fillet welds shall be:

$$L_w \geq 1.5\ in. \qquad (6.14.9\text{-}4)$$

The maximum length of an end-loaded fillet weld shall be:

$$L_{we} = 100\, S_w \qquad (6.14.9\text{-}5)$$

Fillet Weld Design Strength: Stress on a fillet weld shall be considered to be shear for any direction of applied load. The fillet weld design strength, ϕR_n, shall be as given in Eq. 6.5.1-74 is:

$$\phi R_n = \phi\left(F_{sw} L_{we}\right), \quad \phi = 0.75 \qquad (6.14.9\text{-}6)$$

where $F_{sw} \equiv$ the smallest of the following:

1. The product of the shear strength of the filler metal and the effective throat depth, d_t.

2. The product of the weld-affected ultimate shear strength of the base metal and the fillet size, S_w, for longitudinally loaded welds.
3. The product of the weld-affected base metal tensile strength and the fillet size, S_w, for transversely loaded fillet welds.

6.14.10—Block Shear

The design strength, ϕR_n, of an element in block shear shall be governed by considering two block shear failure modes as follows:

1. For bolted connection elements with shear on some segment(s) of the failure path and tension on the remaining segment(s), use the following formula given in Eq. 6.5.1-75:

   If $F_{tu}\ A_{nt} \geq F_{su}\ A_{nv}$

   then:

   $$\phi R_n = \phi\left(F_{sy}\ A_{gv} + F_{tu}\ A_{nt}\right) \quad (6.14.10\text{-}1)$$

   else, if $F_{su}\ A_{nv} > F_{tu}\ A_{nt}$

   then:

   $$\phi R_n = \phi\left(F_{su}\ A_{nv} + F_{ty}\ A_{gt}\right) \quad (6.14.10\text{-}2)$$

   ϕ=0.75

2. For welded connection elements with shear on some segment(s) of the failure path and tension on the remaining segment(s), use the following formula given in Eq. 6.5.1-76:

   If $F_{tu}A_{gt} \geq F_{su}A_{gv}$

   then:

   $$\phi R_n = \phi\left(F_{sy}A_{gv} + F_{tu}A_{gt}\right) \quad (6.14.10\text{-}3)$$

   else, If $F_{su}A_{gv} > F_{tu}A_{gt}$

   then:

   $$\phi R_n = \phi\left(F_{su}A_{gv} + F_{ty}A_{gt}\right) \quad (6.14.10\text{-}4)$$

   ϕ = 0.75

   where:

$A_{nt} \equiv$ the net area in tension

$A_{gt} \equiv$ the gross area in tension

C6.14.10

The block shear strength evaluation of aluminum connections elements is based on bolted test results from 6061 and 5083 series alloys (Menzemer et al. 1999a and 1999b).

A_{nv} ≡ the net area in shear

A_{gv} ≡ the gross area in shear

6.14.11—Flanges and Webs with Concentrated Forces

When connections are made that result in concentrated loads being applied transverse to the axis of a structural member, the stability of the member's web shall be verified to prevent local buckling. The web behavior is more restrained when the point load is applied away from the ends of the member; consequently there are separate design formulas for when a concentrated transverse load is located near or away from the end of the member. Both design formulas are given in ADM Section J.8.1.

6.15—REFERENCES

Aluminum Association. 2006. *Aluminum Standards and Data*. Aluminum Association, Washington, DC.

Aluminum Association. 2010. *Aluminum Design Manual 2010*. Aluminum Association, Washington, DC.

ASTM. 2007. "Standard Specification for Anchor Bolts, Steel, 36, 55, and 105-ksi Yield Strength," F1554-07a, *Annual Book of ASTM Standards*. American Society for Testing Materials, West Conshohocken, PA.

ASTM. 2009. "Standard Specification for Coatings of Zinc Mechanically Deposited on Iron and Steel," B4-04 (2009), *Annual Book of ASTM Standards*. American Society for Testing Materials, West Conshohocken, PA.

ASTM. 2009. "Standard Specification for Stainless Steel Nuts," F594-09, *Annual Book of ASTM Standards*. American Society for Testing Materials, West Conshohocken, PA.

ASTM. 2009. "Standard Specification for Zinc Coating (Hot-Dip) on Iron and Steel Hardware," A153/A153M-09, *Annual Book of ASTM Standards*. American Society for Testing Materials, West Conshohocken, PA.

ASTM. 2010. "Standard Specification for Aluminum-Alloy 6061-T6 Standard Structural Profiles," B308/B308M-10, *Annual Book of ASTM Standards*. American Society for Testing Materials, West Conshohocken, PA.

ASTM. 2010. "Standard Specification for Aluminum-Alloy Extruded Structural Pipe and Tube," B429/B6M-10, *Annual Book of ASTM Standards*. American Society for Testing Materials, West Conshohocken, PA.

ASTM. 2010. "Standard Specification for Aluminum and Aluminum-Alloy Rivet and Cold-Heading Wire and Rods," ASTM B316/B316M-10, *Annual Book of ASTM Standards*. American Society for Testing Materials, West Conshohocken, PA.

ASTM. 2011. "Standard Practice for Preparation of Use and Care Booklets for Vacuum Cleaners," F486-01(2011), *Annual Book of ASTM Standards*. American Society for Testing Materials, West Conshohocken, PA.

ASTM. 2011. "Standard Specification for Hardened Steel Washers," F436-11, *Annual Book of ASTM Standards*. American Society for Testing Materials, West Conshohocken, PA.

ASTM. 2012. "Standard Specification for Aluminum and Aluminum-Alloy Drawn Seamless Tubes," B210-12, *Annual Book of ASTM Standards*. American Society for Testing Materials, West Conshohocken, PA.

ASTM. 2012. "Standard Specification for Aluminum and Aluminum-Alloy Rolled or Cold Finished Bar, Rod, and Wire," B211-12, *Annual Book of ASTM Standards*. American Society for Testing Materials, West Conshohocken, PA.

ASTM. 2012. "Standard Specification for Aluminum and Aluminum-Alloy Seamless Pipe and Seamless Extruded Tube," B241/B241M-12, *Annual Book of ASTM Standards*. American Society for Testing Materials, West Conshohocken, PA.

ASTM. 2012. "Standard Specification for Structural Bolts, Steel, Heat Treated, 120/105 ksi Minimum Tensile Strength," A325-12, *Annual Book of ASTM Standards*. American Society for Testing Materials, West Conshohocken, PA.

ASTM. 2013. "Standard Specification for Nonferrous Nuts for General Use," F467-13$^{\varepsilon 2}$, *Annual Book of ASTM Standards*. American Society for Testing Materials, West Conshohocken, PA.

ASTM. 2013. "Standard Specification for Stainless Steel Bolts, Hex Cap Screws, and Studs," F593-13a, *Annual Book of ASTM Standards*. American Society for Testing Materials, West Conshohocken, PA.

ASTM. 2014. "Standard Specification for Alloy-Steel and Stainless Steel Bolting for High Temperature or High Pressure Service and Other Special Purpose Applications," A193/A193M-14a, *Annual Book of ASTM Standards*. American Society for Testing Materials, West Conshohocken, PA.

ASTM. 2014. "Standard Specification for Alloy-Steel and Stainless Steel Bolting for Low-Temperature Service," A320/A320M-14, *Annual Book of ASTM Standards*. American Society for Testing Materials, West Conshohocken, PA.

ASTM. 2014. "Standard Specification for Aluminum-Alloy Investment Castings," B618/B618M-14, *Annual Book of ASTM Standards*. American Society for Testing Materials, West Conshohocken, PA.

ASTM. 2014. "Standard Specification for Aluminum-Alloy Permanent Mold Castings," B108/B108M-14, *Annual Book of ASTM Standards*. American Society for Testing Materials, West Conshohocken, PA.

ASTM. 2014. "Standard Specification for Aluminum-Alloy Sand Castings," B26/B26M-14, *Annual Book of ASTM Standards*. American Society for Testing Materials, West Conshohocken, PA.

ASTM. 2014. "Standard Specification for Aluminum and Aluminum-Alloy Extruded Bars, Rods, Wire, Profiles, and Tubes," B221-14, *Annual Book of ASTM Standards*. American Society for Testing Materials, West Conshohocken, PA.

ASTM. 2014. "Standard Specification for Aluminum and Aluminum-Alloy Sheet and Plate," B209-14, *Annual Book of ASTM Standards*. American Society for Testing Materials, West Conshohocken, PA.

ASTM. 2014. "Standard Specification for Carbon and Alloy Steel Nuts," A563-07a(2014), *Annual Book of ASTM Standards*. American Society for Testing Materials, West Conshohocken, PA.

ASTM. 2014. "Standard Specification for Carbon and Alloy Steel Nuts for Bolts for High Pressure or High Temperature Service, or Both," A194/A194M-14a, *Annual Book of ASTM Standards*. American Society for Testing Materials, West Conshohocken, PA.

ASTM. 2014. "Standard Specification for High Magnesium Aluminum-Alloy Sheet and Plate for Marine Service and Similar Environments," B928/B928M-14a, *Annual Book of ASTM Standards*. American Society for Testing Materials, West Conshohocken, PA.

ASTM. 2014. "Standard Specification for Structural Bolts, Alloy Steel, Heat Treated, 150 ksi Minimum Tensile Strength," A490-14a, *Annual Book of ASTM Standards*. American Society for Testing Materials, West Conshohocken, PA.

AWS. 2008. *Structural Welding Code—Aluminum*, D1.2/D1.2M. American Welding Society, Miami, FL.

Crockett, H. 1942. "Predicting Stiffener and Stiffened Panel Stresses," *Journal of the Aeronautical Sciences*, Vol. 9, November, 1942, p. 501. American Institute of Aeronautics and Astronautics, Reston, VA.

Galambos, T. 1987. *Guide to Stability Design Criteria for Metal Structures*, Fourth edition. Wiley Interscience, New York, NY.

Jombock, J. R., and J. W. Clark. "Bending Strength of Aluminum Formed Sheet Members," *Journal of the Structural Division*, ASCE, Vol. 94, No. ST2, February 1968. American Society of Civil Engineers, Reston, VA.

Kaufman, J. G. 2001, *Fracture Resistance of Aluminum Alloys*, ASM International, Materials Park, OH.

Kim, Y. 2003. *Behavior and Design of Aluminum Members in Bending*. PhD Dissertation, Cornell University, Ithaca, NY.

May, J., and C. Menzemer. "Strength of Bolted Aluminum Alloy Tension Members," *Journal of Structural Engineering*, Vol. 131, No. 7, July 2005, pp. 1125–1134.

Menzemer, C., L. Fei, and T. Srivatsan. “Design Criteria for Bolted Connection Elements in Aluminum Alloy 6061”, *Journal of Mechanical Design, Transactions of the ASME*, Vol. 121, September 1999, pp. 348-358.

Menzemer, C., L. Fei, and T. Srivatsan. “Mechanical Response and Failure of Bolted Connection Elements in Aluminum Alloy 5083,” *Journal of Materials Engineering and Performance*, Volume 8, No. 2, April 1999, pp. 211-218.

Menzemer, C., Ortiz, R., Iasconne, R., and Srivatsan, T., “An Investigation of the Bearing Strength of Three Aluminum Alloys”, Material Science and Engineering: A, Vol. 327, Issue 2, April 2002, pp. 203-212.

Sharp, M. L., “Strength of Beams or Columns with Buckled Elements,” *Journal of the Structural Division*, ASCE, Technical Notes, May 1970, p. 1011. American Society of Civil Engineers, Reston, VA.

Sharp, M. L. 1993. *Behavior and Design of Aluminum Structures*. McGraw-Hill, New York, NY.

SECTION 7: PRESTRESSED CONCRETE DESIGN

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SECTION 7:

PRESTRESSED CONCRETE DESIGN

7.1—SCOPE

This Section specifies design provisions for prestressed concrete members. Additional required design provisions, such as those noted in the commentary, shall be obtained from the *AASHTO LRFD Bridge Design Specifications* (*LRFD Design*).

The Section assumes the primary application is concrete pole supports that are tension controlled. For simplification, compression controlled elements are not addressed. In such cases, *LRFD Design* shall be used to establish resistances.

7.2—DEFINITIONS

Cracking Moment—A bending moment that produces a tensile stress greater than the sum of induced compression plus the tensile strength of the concrete resulting in cracks on the tension face.

Development Length—Length of embedded tendon required to develop the design strength of prestressing tendons at a critical section.

Effective Prestress—Stress remaining in prestressing tendons after all losses have occurred, excluding effects of dead load and superimposed load.

Post-Tensioning—Method of prestressing in which tendons are tensioned after concrete has hardened.

Precast Concrete—Structural concrete element cast elsewhere than its final position.

Prestressed Concrete—Structural concrete in which internal stresses have been introduced to reduce potential tensile stresses in concrete resulting from loads.

Pretensioning—Method of prestressing in which tendons are tensioned before concrete is placed.

Reinforcement—Steel material, including reinforcing bar and excluding prestressing tendons.

Spiral Reinforcement—Continuously wound reinforcement in the form of a cylindrical helix.

Strength, Design—Nominal strength multiplied by a resistance factor.

Strength, Nominal—Strength of a member or cross-section before application of any resistance factors.

Strength, Required—Strength of a member or cross-section required to resist factored loads.

Tendon—Steel element, such as wire, bar, or strand, or a bundle of such elements, used to impart a prestress to concrete.

Transfer—Act of transferring stress in prestressing tendons from jacks or pretensioning bed to concrete member.

Transfer Length—tendon length required to full prestress.

7.3—NOTATION

A_v = area of shear reinforcement (in.^{2}) (7.6.3.3.3)
B_2 = coefficient for amplification (7.6.3.1.1)
b = width of the compression face (average if varying) (in.) (7.6.3.4.2)
b_w = width of the web (in.) (7.6.3.3.2)
c = depth of the neutral axis (in.) associated with the nominal moment resistance, M_n (7.6.4.2)
d = the distance from the extreme compression fiber to the centroid of longitudinal tension reinforcement (in.) associated with the nominal moment resistance, M_n (in.) (7.6.3.3.2) (7.6.3.3.3) (7.6.3.4.2) (7.6.4.2)
d_b = nominal diameter of pretensioning strand (in.) (7.6.5.1)
E_c = modulus of elasticity of concrete (ksi) (7.4.2.2)
E_s = modulus of elasticity of mild steel (ksi) (7.4.3.2)
E_p = modulus of elasticity of prestressing tendons (ksi) (7.4.4.2)
F_t = tensile strength of concrete (ksi) (7.6.3.3.2) (7.6.3.4.2)
f_r = modulus of rupture of concrete (ksi) (7.4.2.4)
f_{r1} = modulus of rupture of concrete average value used for service limit state (ksi) (7.4.2.4)
f_{r2} = modulus of rupture of concrete average value used for strength limit state, used for M_{cr} (7.4.2.4) (7.6.4.1)
f'_c = specified 28-day design compressive strength of concrete (ksi) (7.6.2.2) (7.6.3.4.2) (7.4.2.1) (7.4.2.2) (C7.4.2.4) (7.6.3.3.2) (7.6.3.4.2)
f'_{ci} = compressive strength of concrete at time of initial prestress release (ksi) (7.6.2.2)
$\sqrt{f'_c}$ = square root of specified compressive strength of concrete (ksi) (7.4.2.4) (C7.4.2.4) (7.6.2.2) (7.6.3.3.2) (C7.6.3.3.2) (7.6.3.4.2)
$\sqrt{f'_{ci}}$ = square root of compressive strength of concrete at time of initial prestress (ksi) (7.6.2.2)
f_{pc} = effective compressive stress in concrete due to prestress (ksi) (7.6.3.3.2) (7.6.3.4.2) (7.6.4.1)
f_{ps} = stress in prestressed reinforcement at nominal strength (ksi) (7.6.5.1)
f_{pu} = specified tensile strength of prestressing tendon (ksi) (7.4.4.1) (7.6.2.3)
f_{py} = specified yield strength of prestressing tendon (ksi) (7.4.4.1) (7.6.2.3)
f_{se} = effective stress in prestressed reinforcement (after allowance for all prestress losses) (ksi) (7.6.5.1)
f_y = specified yield strength of nonprestressed reinforcement (ksi) (7.6.3.3.3)
I = moment of inertia of the cross-section (in.^{4}) (7.6.3.3.2) (7.6.4.1)
J = polar moment of inertia (in.^{4}) (7.6.3.4.2)
K_1 = aggregate factor for concrete modulus of elasticity (7.4.2.2)
L_d = development length of prestressing strand (in.) (7.6.5.1)
M_{cr} = cracking moment (kip-in.) (7.6.4.1)
$M_{handling}$ = moment due to handling loads (kip-in.) (7.6.3.1.1)
M_n = nominal moment strength of a section (kip-in.) (7.6.3.2.1) (7.6.3.2.2) (7.6.4.2)
M_r = flexural resistance (kip-in.) (7.6.3.1.1) (7.6.3.2.1) (7.6.4.1)
M_u = factored moment at section (kip-in.) (7.6.3.1.1) (7.6.3.3.2)
Q = moment of area above the centroid (in.^{3}) (7.6.3.3.2)
r_o = outside radius of section (in.) (7.6.3.4.2)
s = spacing between shear reinforcement (in.) (7.6.3.3.3)
S = section modulus, (in.^{3}) (7.6.4.1)
T_c = shear resisted by the concrete (kip) (7.6.3.4.1)
T_n = nominal torsional strength at a section (kip-in.) (7.6.3.4.1) (7.6.3.4.2) (7.6.3.5)
T_r = torsional resistance (kip-in.) (7.6.3.1.3) (7.6.3.4.1)
T_s = shear resisted by the reinforcement (kip) (7.6.3.4.1)

T_u = factored torsional moment (kip -in.) (7.6.3.1.3) (7.6.3.4.1) (7.6.3.5)
t = effective wall thickness (in.) (for shear resistance, this is the total section thickness at the neutral axis) (7.6.3.3.2)
V_c = nominal shear strength provided by the concrete (kip) (7.6.3.3.1) (7.6.3.3.2)
V_n = nominal shear strength (kip) (7.6.3.3.1) (7.6.3.3.2) (7.6.3.5)
V_r = shear resistance (kip) (7.6.3.1.2) (7.6.3.3.1)
V_s = nominal shear strength provided by reinforcement (kip) (7.6.3.3.1) (C7.6.3.3.1) (7.6.3.3.3)
V_u = factored shear force (kip) (7.6.3.1.2) (7.6.3.3.2) (7.6.3.5)
w_c = unit weight of concrete (kcf) (7.4.2.2)
x = shorter overall dimension of rectangular part of cross-section (in.) (C7.4.2.4) (7.6.3.4.2)
y = longer overall dimension of rectangular part of cross-section (in.) (7.6.3.4.2)
$\gamma_{handling}$ = load factor for handling (7.6.3.1.1)
η = factor, as defined in Article 7.6.3.4 (7.6.3.4.2)
ϕ = resistance factor (7.5.3.2) (7.6.3.5)
ϕ_a = axial resistance factor (7.5.3.2)
ϕ_f = resistance factor for flexure (7.5.3.2) (7.6.3.2.1)
ϕ_v = resistance factor for shear (7.6.3.3.1) (7.5.3.2)
ϕ_t = resistance factor for torsion (7.6.3.4.1) (7.5.3.2)
λ = lightweight concrete adjustment factor (7.4.2.4) (C7.4.2.4) (7.6.3.3.2) (C7.6.3.3.2) (7.6.3.4.2)

7.4—MATERIALS

7.4.1—General

Designs should be based on the material properties cited herein and on the use of materials that conform to the standards for the grades of construction materials as specified in *AASHTO LRFD Bridge Construction Specifications* (2010).

When other grades or types of materials are used, their properties, including statistical variability, shall be established prior to design. The minimum acceptable properties and test procedures for such materials shall be specified in the contract documents. The contract documents shall define the grades or properties of all materials to be used.

C7.4.1

According to *AASHTO LRFD Bridge Construction Specifications* (2010), all materials and tests must conform to the appropriate standards included in the *AASHTO Standard Specifications for Transportation Materials and Methods of Sampling and Testing* (2014) and/or the standards of the American Society for Testing and Materials. Occasionally, it may be appropriate to use materials other than those included in the *AASHTO LRFD Bridge Construction Specifications*; for example, when concretes are modified to obtain very high strengths or other properties through the introduction of special materials, such as:

- Silica fume,
- Cements other than Portland or blended hydraulic cements,
- Proprietary high early strength cements,
- Ground granulated blast-furnace slag, and
- Other types of cementitious and/or pozzolanic materials.

In these cases, the specified properties of such materials should be measured using the testing procedures defined in the contract documents.

7.4.2—Normal and Lightweight Concrete

7.4.2.1—General

For each component, the specified compressive strength, f'_c, or the class of concrete shall be shown in the contract documents.

Design concrete strengths above 10.0 ksi for normal weight concrete shall be used only when allowed by the Owner. Specified concrete with strengths below 2.4 ksi shall not be used in structural applications.

The specified compressive strength for prestressed concrete shall not be less than 4.0 ksi. For lightweight structural concrete, air dry unit weight, strength, and any other properties required for the application shall be approved by the Owner and specified in the contract documents.

C7.4.2.1

The evaluation of the strength of the concrete used in the work should be based on test cylinders produced, tested, and evaluated in accordance with Section 8 of the *AASHTO LRFD Bridge Construction Specifications* (2010).

It is common practice that the specified strength be attained 28 days after placement. Other maturity ages may be used for design and specified for components that will receive loads at times appreciably different than 28 days after placement.

7.4.2.2—Modulus of Elasticity

In the absence of measured data, the modulus of elasticity, E_c, for concretes with unit weights between 0.090 and 0.155 kcf and specified compressive strengths up to 15.0 ksi may be taken as:

$$E_c = 33000 K_1 w_c^{1.5} \sqrt{f'_c} \tag{7.4.2.2-1}$$

where:

K_1 = correction factor for source of aggregate to be taken as 1.0 unless determined by physical test, and as approved by the authority of jurisdiction

w_c = unit weight of concrete (kcf)

f'_c = specified compressive strength of concrete (ksi)

7.4.2.3—Poisson's Ratio

Unless determined by physical tests, Poisson's ratio may be assumed as 0.2. For components expected to be subject to cracking, the effect of Poisson's ratio may be neglected.

C7.4.2.3

This is a ratio between the lateral and axial strains of an axially and/or flexurally loaded structural element.

7.4.2.4—Modulus of Rupture

Unless determined by physical tests, the modulus of rupture, f_r in ksi, for specified concrete strengths up to 15.0 ksi, may be taken as:

For normal-weight concrete:

when used to calculate the cracking moment of a member in Article 7.5.2 $\qquad f_{r1} = 0.24\lambda\sqrt{f'_c}$

when used to calculate the minimum reinforcement in Article 7.6.3.1 $\qquad f_{r2} = 0.37\lambda\sqrt{f'_c}$

C7.4.2.4

Concrete tensile and shear resistance formulae often have $\sqrt{f'_c}$ which is a nonhomogenous term. If f'_c is entered in psi then the coefficients on the left column of Table C7.4.2.4-1 are used. For units of ksi, then the coefficients in the right column are used. Similarly, Table 7.6.2.2-1 is provided for convenience as designers use both systems depending upon the design specifications being used.

where:

For normal weight concrete λ 1.0

For sand-lightweight concrete λ 0.85

For all-lightweight concrete λ 0.75

When physical tests are used to determine modulus of rupture, the tests shall be performed in accordance with AASHTO T 97 and shall be performed on concrete using the same proportions and materials as specified for the structure.

Table C7.4.2.4-1 Typical Coefficients x for shear resistance equations

$x\sqrt{f_c'}$, psi	$x\sqrt{f_c'}$, ksi
0.6	0.02
2	0.06
3	0.10
4	0.13
5	0.16
6	0.19
6.7	0.21
7.5	0.24
8	0.25
10	0.32
12	0.38
15	0.47

Data show that most modulus of rupture values are between $0.24\sqrt{f_c'}$ and $0.37\sqrt{f_c'}$ (ACI 1992; Walker and Bloem 1960; Khan, Cook, and Mitchell 1996). It is appropriate to use the lower bound value when considering service load cracking. The purpose of the minimum reinforcement in Article 7.6.3.1 is to assure that the nominal moment capacity of the member is at least 20 percent greater than the cracking moment.

Because the actual modulus of rupture could be as much as 50 percent greater than $0.24\sqrt{f_c'}$ the 20 percent margin of safety could be lost. Using an upper bound is more appropriate in this situation.

The properties of higher strength concretes are particularly sensitive to the constitutive materials. If test results are to be used in design, it is imperative that tests be made using concrete with not only the same mix proportions, but also the same materials as the concrete used in the structure.

The light weight concrete factor is used to generally adjust the concrete tensile strength for flexure, shear, and torsion computations. In essence, wherever the square root of f_c' is used, the resulting value is adjusted by λ.

7.4.2.5—Direct Tensile Strength

Direct tensile strength may be determined by either using ASTM C900, or the split tensile strength method in accordance with AASHTO T 198 (ASTM C496).

C7.4.2.5

For normal-weight concrete with specified compressive strengths up to 10 ksi, the direct tensile strength may be estimated as $0.23\lambda\sqrt{f_c'}$.

7.4.3—Reinforcing Steel

7.4.3.1—General

Reinforcing bars, deformed wire, cold-drawn wire, welded plain wire fabric, and welded deformed wire fabric shall conform to the material standards as specified in Article 9.2 of the *AASHTO LRFD Bridge Construction Specifications*.

Reinforcement shall be deformed, except that plain bars or plain wire may be used for spirals, hoops, and wire fabric.

The nominal yield strength shall be the minimum as specified for the grade of steel selected, except that yield strengths in excess of 75.0 ksi shall not be used for design purposes. The yield strength or grade of the bars or wires shall be shown in the contract documents. Bars with yield strengths less than 60.0 ksi shall be used only with the approval of the Owner.

Where ductility is to be assured or where welding is required, steel conforming to the requirements of ASTM A706, "Low Alloy Steel Deformed Bars for Concrete Reinforcement," should be specified.

7.4.3.2—Modulus of Elasticity

The modulus of elasticity, E_s, of steel reinforcing shall be assumed as 29,000 ksi.

7.4.4—Prestressing Steel

7.4.4.1—General

Uncoated, low-relaxation, seven-wire strand, or uncoated plain or deformed, high-strength bars, shall conform to the following materials standards, as specified for use in *AASHTO LRFD Bridge Construction Specifications*:

- AASHTO M 203/M 203M (ASTM A416/A416M), or
- AASHTO M 275/M 275M (ASTM A722/A722M).

Tensile and yield strengths for these steels are specified in Table 7.4.4.1-1.

Table 7.4.4.1-1—Properties of Prestressing Strand and Bar

Material	Grade or Type	Diameter (in.)	Tensile Strength, f_{pu} (ksi)	Yield Strength, f_{py} (ksi)
Strand	270 ksi	1/4 to 0.6 3/8 to 0.6 3/8 to 0.6	270	90% of f_{pu} for low-relaxation strand
Bar	Type 1, Plain Type 2, Deformed	3/4 to 1-3/8 5/8 to 1-3/8	150 150	85% of f_{pu} 80% of f_{pu}

Where complete prestressing details are included in the contract documents, the size and grade or type of steel shall be shown. If the plans indicate only the prestressing forces and locations of application, the choice of size and type of steel shall be left to the manufacturer subject to the Owner's approval.

7.4.4.2—Modulus of Elasticty

If more precise data are not available, the modulus of elasticity for prestressing steels, based on nominal cross-sectional area, may be taken as:

- for strand: $E_p = 28{,}500$ ksi, and
- for bar: $E_p = 30{,}000$ ksi.

7.5—DESIGN LIMIT STATES

7.5.1—General

Support components shall be proportioned to satisfy the requirements at all appropriate service, fatigue, strength, and extreme event limit states. Load combinations are provided in Table 3.4-1.

Prestressed concrete structural components shall be investigated for stresses and deformations for each stage that may be critical during construction, stressing, handling, transportation, and erection as well as during the service life.

Stress concentrations due to prestressing or other loads and to restraints or imposed deformations shall be considered.

7.5.2—Service Limit State

Actions to be considered at the service limit state shall be cracking, concrete stresses, and tendon stresses.

Load effects during casting and release shall have unit load factors. Load effects during service shall use the Service III load combination specified in Table 3.4-1. The cracking stress shall be taken as the modulus of rupture specified in Article 7.4.2.4.

Resistance factors shall be unity for the service limit state.

7.5.3—Strength Limit State

7.5.3.1—General

Strength and stability shall be considered using the applicable strength load combinations specified in Table 3.4-1.

Factored resistance shall be the product of nominal resistance as determined in accordance with the applicable provisions of Articles 7.5.3.

7.5.3.2—Resistance Factors

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

- For flexure and axial $\phi_f = \phi_a = 1.0$
- For shear and torsion $\phi_v = \phi_t = 0.90$ for normal weight concrete, and
- For shear $\phi_v = \phi_t = 0.70$ for light weight concrete

7.5.4—Extreme Event Limit State

All applicable load combinations in Table 3.4-1 for the extreme event limit state shall be investigated.

The resistance factors for the extreme event shall be as defined in the strength limit state in Article 7.5.3.2.

C7.5.4

The ASCE/SEI 7-10 wind maps were generated assuming the wind load factor was unity, the same as for seismic loads in that document. These specifications use a similar approach and therefore include wind in combination with other loads to be addressed as an extreme event. This is consistent with *LRFD Design* where seismic loads are considered in the extreme event limit state.

7.5.5—Fatigue Limit State

Fatigue in the concrete or the reinforcement does not need to be investigated.

C7.5.5

The fatigue loads as primarily determined for steel structures are small compared to the fatigue resistance of prestressed concrete members. Prestressed concrete does not have fatigue-prone details such as welds, etc. that have a relatively low fatigue resistance compared with concrete.

From a practical perspective, fatigue could be checked but it likely will not control.

7.6—DESIGN

7.6.1—General

This Article provides the basis for the section design to meet all applicable limit states.

7.6.2—Service Limit State

7.6.2.1—General

The service limit state attempts to limit cracking and overstress during manufacture. During manufacture, all load combination factors may be assumed to be unity. During service, the load combation factors shall be defined by Service III in Table 3.4-1.

7.6.2.2—Concrete Stresses

Service level stresses in concrete shall be in accordance with Table 7.6.2.2-1.

Table 7.6.2.2-1—Service-Level Stresses in Concrete

Load Condition	Service-Level Compressive Stress (ksi)	Service-Level Tensile Stress (ksi)
Pretensioned members at prestress transfer (before time-dependent prestress losses)	$0.60f'_{ci}$	$0.095\lambda\sqrt{f'_{ci}}$
Post-tensioned members at prestress transfer (before time-dependent prestress losses)	$0.55f'_{ci}$	$0.095\lambda\sqrt{f'_{ci}}$
Service I: Dead load only with unit load factors (after allowance for all prestress losses)	$0.45f'_{c}$	0
Service III load combination (after allowance for all prestress losses)	n/a	$0.095\lambda\sqrt{f'_{c}}$

7.6.2.3—Prestressed Tendon Stresses

Tensile stress in prestressing tendons shall not exceed the following:

- Due to tendon jacking force: $0.94f_{py}$, but not greater than the lesser of $0.80f_{pu}$ and the maximum value recommended by the manufacturer of prestressing tendons or anchorages;
- Immediately after prestress transfer: $0.82f_{py}$, but not greater than $0.74f_{pu}$; and
- Post-tensioning tendons, at anchorages and couplers, immediately after tendon anchorage: $0.70f_{pu}$.

C7.6.2.3

The allowable stresses for prestressing tendons are in accordance with *LRFD Design* for use with low-relaxation wire and strands, ordinary tendons (i.e., wire, strands, and bars), and bar tendons. Specifications for prestressing strand and wire are given in ASTM A416 and A421, respectively.

Low-relaxation wire and strand tendons meeting the requirements of ASTM A416 and A421 are commonly used for prestressed concrete poles. Low-relaxation tendons are recognized for their higher yield strength and reduced prestress losses.

7.6.2.4—Loss of Prestress

Loss of prestress shall be considered in the design of pretensioned and post-tensioned members.

C7.6.2.4

A detailed analysis of losses is not necessary except for unusual situations where deflections could become critical. Lump-sum estimate of losses may be used if supported by research data. Depending on the materials used, total prestress loss is usually between 15 percent and 25 percent. Reasonably accurate methods for calculation of prestress loss, such as that prescribed in *LRFD Design*, Article 5.9.5, “Loss of Prestress,” may be used to estimate of losses in lieu of using lump-sum estimates.

7.6.3—Strength and Extreme Limit States

7.6.3.1—General

7.6.3.1.1—Flexure

The combined load effects for the strength and extreme limit state shall be less than the resistance. The strength and extreme limit states are treated similiarly with the exception of the load factors defined in Table 3.4-1.

The flexural resistance shall be greater than the factored flexural load effects, including second order effects estimated with the amplification factor B_2 provided in Section 4.

C7.6.3.1.1

This Article generally considers the member to be a pole that primarily experiences bending, shear, and torsion.

$$M_u \leq M_r \tag{7.6.3.1.1-1}$$

where:

M_u = the factored load effect (Table 3.4-1) scaled by B_2,

M_r = the bending resistance,

For handling loads,

$$M_u = \gamma_{handling} M_{handling} \tag{7.6.3.1.1-2}$$

where:

$M_{handling}$ = the maximum moment expected during handling and erection of the member under its own weight and any attachments, and

$\gamma_{handling}$ = load factor for handling = 1.5.

7.6.3.1.2—Shear

The factored shear resistance, V_r, shall be greater than the load effect, V_u:

$$V_u \leq V_r \tag{7.6.3.1.2-1}$$

where:

V_u = the factored load effect (Table 3.4-1), and

V_r = the shear resistance.

7.6.3.1.3—Torsion

The factored torsional resistance, T_r, shall be greater than the load effect, T_u:

$$T_u \leq T_r \tag{7.6.3.1.3-1}$$

where:

T_u = the factored load effect (Table 3.4-1), and

T_r = the torsional resistance.

7.6.3.2—Flexural Resistance

7.6.3.2.1—General

The provisions of this article apply to flexure of prestressed concrete members. The factored flexural resistance, M_r, shall be:

$$M_r = \phi_f M_n \tag{7.6.3.2.1-1}$$

where:

M_n = the nominal bending resistance, and

ϕ_f = the resistance factor as specified in Article 7.5.3.

7.6.3.2.2—Nominal Flexural Resistance

Equations for the nominal flexural resistance, M_n, are given in *LRFD Design* for rectangular prestressed or partially prestressed members. For other cross-sections, M_n may be determined by strain compatibility analysis based on the assumptions specified in Article 5.7.2 of *LRFD Design*.

C7.6.3.2.2

Where the applicability of analysis methods is uncertain, ultimate strength may be determined by approved tests on full-scale sections.

7.6.3.3—Shear Resistance

7.6.3.3.1—General

The provisions of this article apply to shear of prestressed concrete members. The shear resistance, V_r, shall be:

$$V_r = \phi_v V_n \quad (7.6.3.3.1\text{-}1)$$

where:

V_n = the nominal shear resistance, and

ϕ_v = the resistance factor as specified in Article 7.5.3.2.

$$V_n = V_c + V_s \quad (7.6.3.3.1\text{-}2)$$

where:

V_s = the shear resisted by the reinforcement, and

V_c = the shear resisted by the concrete.

C7.6.3.3.1

Several models for computing shear resistance are available in *LRFD Design* and these may be used in lieu the model provided here. For concrete poles, the flexural effects is typically much larger than are the shear effects, so shear likely does not control the size of the section.

Where there is no transverse reinforcement, the shear resisted by the reinforcement V_s is zero.

7.6.3.3.2—Nominal Shear Resistance—Concrete

For square and rectangular prestressed concrete members with effective prestress force not less than 40 percent of the tensile strength of the flexural reinforcement, V_c (kips) may be computed as:

$$V_c = \left(0.02\lambda\sqrt{f'_c} + 0.7\left(\frac{V_u d}{M_u}\right)\right) b_w d \quad (7.6.3.3.2\text{-}1)$$

However, V_c need not be less than:

$$0.06\lambda\sqrt{f'_c}\, b_w d \quad (7.6.3.3.2\text{-}2)$$

nor shall it be greater than:

$$0.16\lambda\sqrt{f'_c}\, b_w d \quad (7.6.3.3.2\text{-}3)$$

where:

λ = light weight concrete factors, per Article 7.4.2.4,

f_c' = specified 28-day design compressive strength of concrete (ksi),

b_w = width of the web (in.), and

d = the distance from the extreme compression fiber to the centroids of longitudinal tension reinforcement (in.).

The quantity $V_u d/M_u$ shall not be greater than 1.0 where M_u is the factored moment occurring simultaneously with V_u at the section considered. The quantity, d, in the term $V_u d/M_u$ shall be the distance from the extreme compression fiber to the centroid of prestressed reinforcement.

For hollow circular prestressed members, V_c (kips) may be computed as:

$$V_c = \frac{\sqrt{F_t^2 + F_t f_{pc}}}{\frac{Q}{2It}} \qquad (7.6.3.3.2\text{-}4)$$

where:

F_t = $0.13\lambda\sqrt{f_c'}$, ksi

f_{pc} = the effective compressive stress in concrete due to prestress (ksi)

Q = the moment of area above (or below) the neutral axis (in^3),

I = the moment of inertia (in.^{4}), and

t = the effective wall thickness (in.).

The nominal shear strength of concrete is calculated based on elastic analysis and the assumption that cracking will occur when the principal stress reaches $0.13\lambda\sqrt{f_c'}$.

7.6.3.3.3—Nominal Shear Resistance—Steel

The shear force, V_s, contributed by the transverse steel may be computed as:

$$V_s = \frac{A_v f_y d}{s} \qquad (7.6.3.3.3\text{-}1)$$

where:

d = the diameter of the shear steel for round poles.

Design yield strength of shear reinforcement shall not exceed 60 ksi.

7.6.3.4—Torsional Resistance

7.6.3.4.1—General

The provisions of this Article apply to torsion of prestressed concrete members. The torsional resistance, T_r, shall be:

$$T_r = \phi_t T_n \quad (7.6.3.4.1\text{-}1)$$

where:

T_u = the factored load effect,

T_r = the torsional resistance, and

ϕ_t = the resistance factor as specified in Article 7.5.3.2.

$$T_n = T_c + T_s \quad (7.6.3.4.1\text{-}2)$$

where:

T_s = the shear resisted by the reinforcement, and

T_c = the shear resisted by the concrete.

7.6.3.4.2—Nominal Torionsal Resistance—Concrete

The nominal torionsal resistance, T_n (kip-in) , for a square or rectangular cross-section may be calculated as:

$$T_n = 0.19\lambda\sqrt{f_c'}\sqrt{1+\frac{10 f_{pc}}{f_c'}}\sum \eta x^2 y \quad (7.6.3.4.2\text{-}1)$$

where:

$$\eta = \frac{0.35}{0.75+\dfrac{b}{d}} \quad (7.6.3.4.2\text{-}2)$$

x = the shorter overall dimension of rectangular part of the cross section (in.),

y = the longer overall dimension of rectangular part of the cross section (in.),

λ = light weight concrete factors (see Article 7.4.2.4),

f_c' = specified 28-day design compressive strength of concrete (ksi),

b = width of the compression face (average if varying) (in.), and

d = the distance from the extreme compression fiber to the centroids of longitudinal tension reinforcement (in.).

The nominal torionsal resistance, T_n (kip-in) , for circular cross-sections is given by:

$$T_n = \frac{J}{r_o}\sqrt{F_t^2 + F_t f_{pc}} \tag{7.6.3.4.2-3}$$

where:

$$F_t = 0.13\lambda\sqrt{f_c^{'}} \tag{7.6.3.4.2-4}$$

J = the polar moment of inertia (in^4),

r_o = outer radius of the section (in.), and

f_{pc} = the effective compressive stress in concrete due to prestress (ksi)

7.6.3.4.3—Nominal Torionsal Resistance—Steel

LRFD Design may be used for sections with transverse steel reinforcement.

7.6.3.5—Combined Shear and Torsion Resistance

For members subjected to shear and torsion, the following interaction equation may be used to determine the strength of the member:

$$\left(\frac{V_u}{\phi V_n}\right)^2 + \left(\frac{T_u}{\phi T_n}\right)^2 \leq 1.0 \tag{7.6.3.5-1}$$

7.6.4—Reinforcement Limits

C7.6.4

The reinforcement should be sufficient such that the failure mode is ductile in regions of critical load effect.

7.6.4.1—Minimum Reinforcement Limits

A flexural component, the amount of prestressed and nonprestressed tensile reinforcement shall be adequate to develop a factored flexural resistance, M_r, at least equal to the lesser of:

- 1.2 times the cracking moment, M_{cr} (kip-in) , determined on the basis of elastic stress distribution and the modulus of rupture, f_{r2}, of the concrete as specified in Article 7.4.2.4 where M_{cr} may be taken as:

  $$M_{cr} = S\left(f_{r2} + f_{pc}\right) \tag{7.6.4.1-1}$$

  where:

  I = section modulus to the tension face (in^3),

  f_{r2} = modulus of rupture (see Article 7.4.2.4) (ksi), and

f_{pc} = effective compressive stress in concrete due to prestress (ksi).

or

- 1.33 times the moment required by the extreme event limit state.

7.6.4.2—Maximum Reinforcement Limits

The plastic neutral axis depth, c (in.), when the section reaches M_n should be less than or equal to 0.42 times the depth to the extreme tension steel fiber, d (in.); otherwise, an adjustment in the resistance factor shall be made.

C7.6.4.2

This article limits the maximum reinforcement such that the section steel yields prior to a compression failure. This behavior is accomplished by limiting the depth of the plastic neutral axis to 42 percent of the effective section depth. See Article 7.7.3.3 of *LRFD Design* for details for adjustments in the resistance factor should this requirement not be satisfied. This requirement is for sections where the load effect is critical.

7.6.5—Development of Prestressing Strand

7.6.5.1—Development Length

Three- and seven-wire pretensioning strand shall be bonded beyond the critical section for a development length, L_d, of not less than:

$$L_d = \left(f_{ps} - \frac{2}{3} f_{se} \right) d_b \qquad (7.6.5.1\text{-}1)$$

where:

L_d = development length of prestressing strand (in.)
f_{ps} = stress in prestressed reinforcement at nominal strength (ksi)
f_{se} = effective stress in prestressed reinforcement (after allowance for all prestress losses)
d_b = nominal diameter of pretensioning strand (in.)

7.6.5.2—Transfer Length

The transfer length of the prestressing tendons shall be considered at ends of members. The prestress force shall be assumed to vary linearly from zero at end of tendon to a maximum at a distance from end of tendon equal to the transfer length, assumed to be 60 diameters for strand and 100 diameters for single wire.

7.7—REFERENCES

AASHTO. 2002. *Standard Specifications for Highway Bridges*, 17th Edition, HB-17. American Association of State Highway and Transportation Officials, Washington, DC.

AASHTO. 2010. *AASHTO LRFD Bridge Construction Specifications,* Third Edition. American Association of State Highway and Transportation Officials, Washington, DC.

AASHTO. 2014. *AASHTO LRFD Bridge Design Specifications*, Seventh Edition, American Association of State Highway and Transportation Officials, Washington, DC.

AASHTO. 2014. *Standard Specifications for Transportation Materials and Methods of Sampling and Testing*, 34th Edition. American Association of State Highway and Transportation Officials, Washington, DC.

ACI. 2011. *Building Code Requirements for Structural Concrete*, ACI 318-2011. American Concrete Institute, Farmington Hills, MI.

ASCE/PCI Joint Committee on Concrete Poles. 1997. "Guide for the Design of Prestressed Concrete Poles—Final Draft," *PCI Journal.* American Society of Civil Engineers and Precast/Prestressed Concrete Institute, Chicago, IL, November/ December 1997.

ASTM. 2009. "Standard Specification for Low-Alloy Steel Deformed and Plain Bars for Concrete Reinforcement," A706/A706M-09b, *Annual Book of ASTM Standards*. American Society for Testing Materials, West Conshohocken, PA.

ASTM. 2012. "Standard Specification for Uncoated High-Strength Steel Bars for Prestressing Concrete," A722/A722M-12, *Annual Book of ASTM Standards*. American Society for Testing Materials, West Conshohocken, PA.

ASTM. 2010. "Standard Specification for Uncoated Stress-Relieved Steel Wire for Prestressed Concrete," A421/A421M-10, *Annual Book of ASTM Standards*. American Society for Testing Materials, West Conshohocken, PA.

ASTM. 2011. "Standard Test Method for Splitting Tensile Strength of Cylindrical Concrete Specimens," C496/C496M-11, *Annual Book of ASTM Standards*. American Society for Testing Materials, West Conshohocken, PA.

ASTM. 2012. "Standard Specification for Steel Strand, Uncoated Seven-Wire for Prestressed Concrete," A416/A416M-12a, *Annual Book of ASTM Standards*. American Society for Testing Materials, West Conshohocken, PA.

ASTM. 2014. "Standard Test Method for Pullout Strength of Hardened Concrete," C900-14, *Annual Book of ASTM Standards*. American Society for Testing Materials, West Conshohocken, PA.

Khan, A. A., W. D. Cook, and D. Mitchell,, "Tensile Strength of Low, Medium, and High-Strength Concretes at Early Ages," *ACI Materials Journal*, Vol. 93, No. 5, September-October 1996, pp. 487-493.

PCI. 2010. *PCI Design Handbook—Precast and Prestressed Concrete*, Seventh Edition. Precast/Prestressed Concrete Institute, Chicago, IL.

Walker, S. and D. L. Bloem, "Effect of Aggregate Size on Properties of Concrete," *Journal of the American Concrete Institute*, Vol. 57, No. 3, September 1960, pp. 283-98

SECTION 8: FIBER-REINFORCED COMPOSITES DESIGN

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SECTION 8:

FIBER-REINFORCED COMPOSITES DESIGN

8.1—SCOPE

This Section specifies design and testing provisions for fiber-reinforced polymer (FRP) structural supports for luminaires and signs. The provisions of this Section apply only to fiber-reinforced polymer (FRP) composites. Other structural composites may be used if approved by the Owner.

C8.1

Applications of FRPs are expanding as the experience in using these materials increases. Reinforced plastic composites have been successfully employed in major structural applications. This Section provides guidelines for the design of structural supports manufactured from FRP. This Section may be expanded in the future to include other composite materials.

8.2—DEFINITIONS

Fiber-Reinforced Polymer (FRP)—A composite material in which the polymer resin matrix is reinforced with high-strength fibers, most commonly glass.

Glassfiber-Reinforced Polymer (GFRP)—A composite material in which the polymer resin matrix is reinforced with glass fibers.

Surface Veil—A surfacing mat used in the outer surrounding layer of an FRP pole to produce a smooth surface and to protect the underlying material from weathering degradation.

Weathering Resistance—A property of the FRP material that resists degradation caused by environmental factors such as outdoor sunlight (UV) and wind-borne particulates. Degradation is evidenced by exposed fibers, cracks, crazes, or checks in the surface.

8.3—NOTATION

b	=	effective width of the side of a polygonal section (in.) (8.6.3.2) (C8.6.3.2) (8.6.3.3) (C8.6.3.5)
b_f	=	width of the flange of an I- or W- shape sections (in.) (8.6.3.3)
B_2	=	moment magnification factor due to second-order geometric effects (8.6.4.1) (C8.6.4.1)
C_1	=	lateral buckling constant (8.6.3.3)
D	=	outer diameter (in.) (8.6.3.2) (C8.6.3.2) (8.6.3.5)
d	=	depth of bending member (in.) (8.6.3.3)
E_b	=	modulus of elasticity in bending (ksi) (8.6.3.1) (8.6.3.3)
E_c	=	modulus of elasticity in compression (ksi) (8.6.3.1) (8.6.3.4) (8.6.4.1)
E_t	=	modulus of elasticity in tension (ksi) (8.6.3.1)
E_1	=	modulus of elasticity in bending in the longitudinal direction of the member (ksi) (8.6.3.2) (C8.6.3.2) (8.6.3.5) (C8.6.3.5) (C8.6.3.7)
E_2	=	modulus of elasticity in bending in the transverse direction of the member (ksi) (8.6.3.2) (C8.6.3.2) (8.6.3.5) (C8.6.3.7)
F_a	=	compression stress (ksi) (8.6.4.1)
F_{au}	=	ultimate compression stress (ksi) (8.6.3.1) (8.6.3.4) (8.6.3.5)
F_b	=	bending stress (ksi) (8.6.4.1) (C8.6.4.1)
F_{br}	=	bending resistance (ksi) (8.6.3.2) (8.6.3.3) (8.6.4.1) (C8.6.4.1) (8.6.4.2) (C8.6.4.2)
F_{brx}	=	bending resistance about the x-axis (ksi) (C8.6.4.1) (C8.6.4.2)
F_{bry}	=	bending resistance about the y-axis (ksi) (C8.6.4.1) (C8.6.4.2)
F_{bu}	=	ultimate bending stress (ksi) (8.6.3.1) (8.6.3.2) (8.6.3.3)
F_{bx}	=	bending stress about the major axis (ksi) (C8.6.4.1)
F_{by}	=	bending stress about the minor axis (ksi) (C8.6.4.1)

F_{cr} = compression resistance (ksi) (8.6.3.4) (8.6.3.5) (8.6.3.7) (8.6.4.1)
F'_e = Euler stress in compression (ksi) (8.6.4.1) (C8.6.4.1)
F'_{ex} = Euler stress factor in compression in the x-direction (ksi) (C8.6.4.1)
F'_{ey} = Euler stress factor in compression in the y-direction (ksi) (C8.6.4.1)
F_{tr} = tension resistance (ksi) (8.6.3.6) (8.6.4.2)
F_{tu} = ultimate tension stress (ksi) (8.6.3.1) (8.6.3.6) (8.6.3.7)
F_{vr} = shear resistance (ksi) (8.6.3.7)
F_{vu} = ultimate shear stress (ksi) (8.6.3.1)
f_a = factored compressive stress (ksi) (C8.6.4.1)
f_b = factored bending stress (ksi) (8.6.4.2) (C8.6.4.2)
f_{bx} = factored bending stress about the major axis (ksi) (C8.6.4.2)
f_{by} = factored bending stress about the minor axis (ksi) (C8.6.4.2)
f_t = factored tension stress (ksi) (8.6.4.2)
f_v = factored shear stress (ksi) (C8.6.3.7)
G = in-plane shear modulus (ksi) (8.6.3.2) (8.6.3.3) (8.6.3.7)
h = pole height above ground (ft) (8.6.2.1)
I_2 = moment of inertia about the centroidal weak axis (in.^{4}) (8.6.3.3)
J = torsional constant (in.^{4}) (8.6.3.3)
K_1 = orthotropy factor (8.6.3.2) (C8.6.3.2) (8.6.3.5) (C8.6.3.5)
k = effective buckling length factor (C8.6.2.1) (8.6.3.3) (8.6.3.4)
k_b = effective buckling length factor in the plane of bending (8.6.4.1)
L = unbraced length of member (in.) (8.6.3.3) (8.6.3.4) (8.6.4.1)
$M_{average}$ = average flexural resistance (kip-in.) (8.6.2.1)
M_n = nominal flexural resistance (kip-in.) (8.6.2.1)
M_r = factored flexural resistance (kip-in.) (8.6.2.1)
M_{xc} = lateral torsional strength (lb-in.) (8.6.3.3)
P_e = load to produce a moment at ground line equal to the design moment resultant (lb) (8.6.2.1)
P_{e2} = Euler load to produce buckling in the weak direction of the member (lb) (8.6.3.3)
R_1 = distance from center to external face for round tubular sections or radius of circle inscribed through apexes for polygonal sections (in.) (8.6.3.7)
r = radius of gyration (in.) (8.6.3.4)
r_b = radius of gyration in the plane of bending (in.) (8.6.4.1)
S_1 = section modulus about the strong axis (in.^{3}) (8.6.3.3)
t = wall thickness (in.) (8.6.3.2) (C8.6.3.2) (C8.6.3.3) (8.6.3.5) (8.6.3.7)
t_f = flange thickness of W- and I-shape beams (in.) (8.6.3.3)
V_R = coefficient of variation from the test data (C8.6.2.1)
λ_R = bias (C8.6.2.1)
ϕ = resistance factor (8.6.4.2)
ϕ_b = resistance factor for bending (8.6.2.1) (C8.6.2.1) (8.6.3.1) (8.6.3.2) (8.6.3.3) (C8.6.4.1) (8.6.4.2)
ϕ_c = resistance factor for compression (8.6.3.1) (8.6.3.4) (8.6.3.5) (8.6.4.1)
ϕ_t = resistance factor for tension (8.6.3.1) (8.6.3.6) (8.6.4.2)
ϕ_v = resistance factor for shear (8.6.3.1) (8.6.3.7)
μ = constant relating Poisson's ratio of an orthotropic material (8.6.3.2) (C8.6.3.2) (8.6.3.5) (C8.6.3.5) (C8.6.3.7)
μ_{12} = Poisson's ratio in the longitudinal direction of the member (8.6.3.1) (8.6.3.2) (8.6.3.7)

8.4—MATERIAL—FIBER-REINFORCED POLYMER (FRP)

FRP shall be composed of two principal constituents: polymer resin and glass fiber reinforcement.

C8.4

These Specifications cover primarily FRP composites, which are the most widely used composites for civil engineering applications requiring structural reliability.

FRP is composed of two principal constituents, namely polymer resin and fiber reinforcement. The FRP composite possesses superior properties not available to each constituent alone. The fiber reinforcement, which is significantly higher in stiffness and strength than the polymer resin, constitutes the main load-carrying element of the composite.

From a practical standpoint, FRP may be considered an elastic material, which exhibits a stress–strain behavior that is linear up to failure. The material does not yield or exhibit a permanent set due to transient overloads.

8.4.1—Polymer Resins

Unsaturated polymer shall be used as the principal matrix resin.

C8.4.1

The unsaturated polymer resin is a cross-linked thermosetting plastic that is hard and brittle and fractures on impact. However, when reinforced with high-strength fibers (usually but not limited to glass fibers), it develops reliable structural qualities.

8.4.2—Fiber Reinforcement

Reinforcement for FRP composites shall be E-, C-, or S-glass. Fiber may take any of the following forms:

- Continuous strands (i.e., rovings and yarns)
- Mats
- Chopped strands
- Milled fibers
- Fabrics

C8.4.2

The high-strength fibers reinforce the polymer resin and provide the strength and stiffness required for structural purposes. Three types of glass are commonly used as fiber reinforcement: E-, C-, and S-glass. E-glass, or electrical grade, is for general purpose structural uses, as well as for good heat resistance and high electrical properties. C-glass, or chemical grade, is best for resistance to chemical corrosion. S-glass, or high silica, is a special glass for high heat resistance and also has enhanced structural properties. Of the three types, E-glass is the most common in engineering applications.

8.5—MANUFACTURING METHODS

FRP members shall be manufactured by generally accepted methods that ensure high quality, good performance, and reliable mechanical properties of the members produced. (See Table 8.5-1.)

Table 8.5-1—Typical Ranges for Mechanical Properties of FRP Laminates

Process	Percent of Glass Fiber by Weight	Tensile Strength (ksi)	Tensile Modulus (10^3 ksi)	Flexural Strength (ksi)	Compressive Strength (ksi)
Filament Winding (glass-polymer)	40–75	35–50	2500–3600	30–50	30–45
Pultrusion (glass mat-polymer)	40–80	60–150	4000–6000	100–150	30–70
Pultrusion (glass mat and roving polymer)	30–55	7–35	800–2500	10–30	15–40
Centrifugal Casting	40–45	30–40	2200–2500	30–40	25–35
Hand Layup	40–75	30–40	800–2500	10–30	15–40

Note: These values are provided for information only and should not be used for design.

Source: Values for filament winding and pultrusion are from the *Structural Plastics Design Manual* (ASCE, 1985).

8.6—METHOD OF DESIGN

The design of FRP members shall be based on the LRFD approach.

The structural design of FRP members shall be performed either by full-scale testing as specified in Article 8.6.2 or by calculation using the resistance specified in Article 8.6.3. If testing is not performed, the design calculations provided shall be verified by documented test results on similar structures as approved by the Owner.

8.6.1—Design Assumptions

Design of FRP members shall be based on linear elastic theory.

C8.6.1

FRP materials are generally anisotropic. The mechanical properties vary according to the fiber content, orientation of the fibers, and the mechanical characteristics of the fibers and the resin. Depending on the particular placement of the fibers, the material may be considered orthotropic or isotropic.

Theories are available in the literature to predict the overall behavior of FRP based on the properties of its constituents (i.e., resin and glass). For practical design, however, the behavior of resin and glass is viewed at a gross scale to provide overall properties of the cross-section.

Although FRP qualifies as a viscoelastic material, which is temperature- and time-dependent, its behavior can be considered as linearly elastic that obeys Hooke’s law. A basic assumption is that “plane sections remain plane after bending.” This is primarily because of the linear stress–strain behavior up to failure.

8.6.2—Testing

Full-scale structural testing shall be used to verify the strength and deflection of FRP members.

The bending test criterion for FRP poles is summarized in Article 8.6.1. Other tests that may be required are given in Article 8.6.2. The test methods shall be approved by the Owner.

C8.6.2

Because FRP poles are usually round tubular tapered members whose performance is dependent upon the composition of the material and the manufacturing procedure, testing is required to determine the bending and torsional strength, as well as the weathering resistance of FRP poles.

Cracking and early failure can occur at hand holes during bending of poles, and early pole attachment failures can occur at cast shoe bases (i.e., pole-to-tube junction). These items can also be checked through testing.

8.6.2.1—Bending Strength and Stiffness of FRP Poles

The flexural strength of FRP poles may be determined in accordance to ASTM D4923 procedure, except for the following requirements.

- The flexural resistance, M_n, of the pole shall be determined by a point load applied 1 ft from the top. A minimum of ten poles shall be tested to determine the average, $M_{average}$, and associated coefficient of variation, *COV*.
- The pole shall have stiffness sufficient to carry a load, P_e, applied 1 ft from the top with a maximum deflection at the top no greater than 15 percent of the pole height, h, above ground/support. The equivalent point load shall be computed as:

$$P_e = \frac{M_{average}}{(h-1.0)} \tag{8.6.2.1-1}$$

- The bending resistance shall be computed as:

$$M_r = \phi_b M_n = \phi M_{average} \tag{8.6.2.1-2}$$

where

$\phi_b = 0.67$

For poles with mast arms, the slope at the top of the pole resulting from the dead load moment of the arm and luminaire shall not exceed 0.35 in./ft.

The resistance factor may be determined using the test data. The test-based resistance factor shall be approved by the Owner. It shall not be greater than 0.9.

C8.6.2.1

ASTM D4923 was withdrawn in 2010 because it was not updated per ASTM regulations. Historical version may be referenced. ASTM D1036-99 (2012) Standard Test methods of Static Test of Wood Pole could be adapted with care with consent of the Owner.

Tests may be conducted to study the effect of some particular characteristic and in such cases the selection of test specimens shall be made in such a manner as to ensure that the range of the characteristic under study has been adequately sampled, including a statically valid sample size.

Masmoudi et al (2008) provides research that may be applicable for modeling and testing FRP poles.

The flexural resistance factor can be based upon the test data, as:

$$\phi_b = \lambda_R (1 - kV_R) \tag{C8.6.2-1}$$

where:

λ_R = bias (assigned 1.1)

k = coefficient estimated as 2.0, and

V_R = coefficient of variation from the test data.

A resistance factor of 0.67 is associated with a V_R = 0.20. The bias is set typically to other materials. The minimum acceptable coefficient of variation is 0.20 for the tests.

8.6.2.2—Other Tests

The following additional tests shall be performed as required by the Owner:

a. Torsional strength per ASTM D4923,

b. Fatigue strength per ASTM D4923,

c. Weathering resistance per ASTM G154,

d. Adhesion of coatings,

e. Color change from UV exposure, and

f. Fatigue strength of connections.

8.6.3—Resistance

Resistance may be computed by provisions of Articles 8.6.3.2 through 8.6.3.7 if the design calculations provided are verified by documented test results on similar structures and as approved by the Owner.

C8.6.3

Resistance equations presented in Article 8.6.3 are intended for normal loading conditions and are obtained from various sources (Johnson, 1985). If the applied load is long term or cyclic, or if elevated temperatures and exposure to aggressive environments are expected, reductions in the resistance should be considered.

Resistance equations may also be obtained from the Manufacturer provided that adequate supporting documentation including test data are made available.

8.6.3.1—Determination of Mechanical Properties of FRP

The strength of FRP laminates shall be determined by testing using flat sheet samples in accordance with the ASTM standards listed in Table 8.6.3.1-1. Samples shall be manufactured in the same manner as that proposed for the structural member.

For structural members where the fiber orientation changes along the member, sheet samples shall be taken at locations of critical stresses.

C8.6.3.1

Because the mechanical properties of the FRP material could vary significantly depending on the particular composition and the manufacturing process, the test samples must be representative of the actual conditions in the final product.

The proposed resistance factors shown in Table 8.6.3.1-1 are maximum values based on common industry practice. Other values may be used when agreed on by the Owner and the Manufacturer.

Table 8.6.3.1-1 Standard Tests for Determining the Mechanical Properties of FRP

Property	Standard Test	Resistance factor
Bending Strength, F_{bu} (ksi)	ASTM D790	$\phi_b = 0.67$
Modulus of Elasticity in Bending, E_b (ksi)	ASTM D790	—
Tensile Strength, F_{tu} (ksi)	ASTM D638	$\phi_t = 0.80$
Modulus of Elasticity in Tension, E_t (ksi)	ASTM D638	—
Compressive Strength, F_{au} (ksi)	ASTM D695	$\phi_c = 0.55$
Modulus of Elasticity in Compression, E_c (ksi)	ASTM D695	—
Shear Strength, F_{vu} (ksi)	ASTM D732	$\phi_v = 0.55$
Poisson's ratio in the longitudinal direction, υ_{12}	ASTM D3039	—

8.6.3.2—Bending Resistance for Tubular Sections

The bending resistance for tubular sections may be calculated as follows:

- For round tubular sections:

$$F_{br} = \phi_b \frac{0.75 E_1 K_1}{\left(\dfrac{D}{t}\right)\sqrt{\mu}} = \phi_b F_{bu} \tag{8.6.3.2-1}$$

where:

$$K_1 = 1.414\left[\left(1 + \mu_{12}\left(\frac{E_2}{E_1}\right)^{\frac{1}{2}}\right)\left(\frac{E_2}{E_1}\right)^{\frac{1}{2}}\left(\frac{G}{E_1}\right)\right]^{\frac{1}{2}} \tag{8.6.3.2-2}$$

C8.6.3.2

For thin-walled FRP sections, local buckling is a major parameter that controls the strength of the member in bending. The bending stress is defined as a function of the critical buckling stress of the section. Equations to obtain the critical buckling stress are based on the plate theory for orthotropic elements, and they are expressed in terms of the aspect ratio, *b/t*, of the plate or the aspect ratio, *D/t*, of the cylinder. For polygonal sections, the critical buckling stress is determined for a long plate with simply supported long edges. Because there is some edge restraint at the intersection between sides of the polygon, the assumption of simply supported long edges leads to conservative values for the critical buckling stress.

- For polygonal sections (hexadecagonal, dodecagonal, octagonal, and square tubular sections):

$$F_{br} = \phi_b \frac{3.27 E_1 K_1}{\left(\frac{b}{t}\right)^2 \mu} = \phi_b F_{bu} \tag{8.6.3.2-3}$$

where:

$$K_1 = 0.5\left[\left(\frac{E_2}{E_1}\right)^{\frac{1}{2}} + \mu_{12}\left(\frac{E_2}{E_1}\right) + \left(\frac{2G\mu}{E_1}\right)\right] \tag{8.6.3.2-4}$$

for:

$$\mu = 1 - \mu_{12}^2\left(\frac{E_2}{E_1}\right) \tag{8.6.3.2-5}$$

According to Johnson (1985), it has been shown that the critical compressive stress caused by bending is 30 percent higher than the critical compressive stress caused by axial compressive loads for round tubular sections. Therefore, the critical buckling stress for a round tubular member under bending Eq. 8.6.3.2-1 is taken as 1.3 times the critical buckling stress for a round tubular member under axial compression Eq. 8.6.3.5-1.

Equations 8.6.3.2-1 and 8.6.3.2-3 may be used for planar isotropic materials by setting $E_1 = E_2$ in the equations for K_1 and μ.

8.6.3.3—Bending Resistance for W and I Sections

The bending resistance for W and I sections may be calculated as follows:

- For laterally supported W and I shapes:

$$F_{br} = \phi_b \frac{0.4 E_b}{\left(\frac{b_f}{t_f}\right)^2} = \phi_b F_{bu} \tag{8.6.3.3-1}$$

- For unsupported W and I shapes:

$$F_{br} = \phi_b \frac{C_1}{S_1}\sqrt{M_{xc}^2 + \left(\frac{dP_{e2}}{2}\right)^2} = \phi_b F_{bu} \tag{8.6.3.3-2}$$

where:

$$M_{xc} = \frac{\pi}{kL}\sqrt{E_b I_2 G J} \tag{8.6.3.3-3}$$

$$P_{e2} = \frac{\pi^2 E_b I_2}{\left(kL\right)^2} \tag{8.6.3.3-4}$$

for cantilever members, $C_1 = 1.0$, and $k = 2.1$.

C8.6.3.3

Bending resistances for W and I shapes are provided for isotropic materials. For thin-walled FRP sections, local buckling is the parameter controlling the strength of the member in bending; therefore, the critical bending stress is defined as the critical buckling stress of the section. The equations to obtain the critical buckling stress are based on the plate theory for isotropic elements, and they are expressed in terms of the aspect ratio, b/t, of the plate.

Barbero and Raftoyiannis (1993) have proposed a general equation for the bending stress of pultruded W and I beams based on the equations developed for the bending stresses of wood members.

8.6.3.4—Compression Resistance—Flexural Buckling

The compression resistance considering flexural buckling may be calculated as follows:

- For:

$$\frac{kL}{r} < \sqrt{\frac{2\pi^2 E_c}{F_{au}}} \tag{8.6.3.4-1}$$

$$F_{cr} = \phi_c \left[\frac{F_{au}}{\pi} - \frac{F_{au}^{\frac{3}{2}}}{2\sqrt{2\pi^2 E_c}} \left(\frac{kL}{r} \right) \right] \tag{8.6.3.4-2}$$

- For:

$$\frac{kL}{r} \geq \sqrt{\frac{2\pi^2 E_c}{F_{au}}} \tag{8.6.3.4-3}$$

$$F_{cr} = \phi_c \frac{\pi^2 E_c}{\left(\frac{kL}{r} \right)^2} \tag{8.6.3.4-4}$$

C8.6.3.4

Compression design of FRP members is generally controlled by buckling. Flexural buckling and local buckling should be considered. Equations for the compression stress for flexural buckling are taken from the *Structural Plastics Design Manual*, and they are applicable to isotropic and orthotropic materials.

8.6.3.5—Compression Resistance—Local Buckling

The compressive resistance considering local buckling may be calculated as follows:

- For round tubular sections:

$$F_{cr} = \phi_c \frac{0.57 E_1 K_1}{\left(\frac{D}{t} \right) \sqrt{\mu}} \leq \phi_c F_{au} \tag{8.6.3.5-1}$$

- For polygonal sections (hexadecagonal, dodecagonal, octagonal, and square tubular sections):

$$F_{cr} = \phi_c \frac{3.27 E_1 K_1}{\left(\frac{b}{t} \right)^2 \mu} \leq \phi_c F_{au} \tag{8.6.3.5-2}$$

where K_1 and μ are determined according to Article 8.6.3.2.

C8.6.3.5

Compression design of FRP hollow tubes is usually controlled by local buckling of the wall, except for unusual combinations of very long members with large axial loads. Equations provided for the compressive stress for short-column action are developed for orthotropic materials, but they may be used for planar isotropic materials by setting $E_1 = E_2$ in the equations for K_1 and μ.

In practice, the critical buckling stress of round tubular sections is about 50 percent of the theoretical critical buckling stress as a result of geometrical and material imperfections. To account for this difference between the theoretical and the actual buckling stress, the theoretical buckling stress in Eq. 8.6.3.5-1 has been multiplied by a factor of 0.50

8.6.3.6—Tension Resistance

The tension resistance on the net cross-sectional area may be calculated as follows:

$$F_{tr} = \phi_t F_{tu} \tag{8.6.3.6-1}$$

The net cross-sectional area shall be calculated as the remaining area after discounting holes or other discontinuities in the member.

C8.6.3.6

Bolt holes and handholes produce abrupt reductions in the cross-sectional area of the member that generate stress concentrations. The Designer should account for the stress concentrations that may reduce the capacity of the member under tension. Information for computing the reduction in capacity due to stress concentrations at discontinuities in tension members is given in the *Structural Plastics Design Manual*.

8.6.3.7—Shear Resistance

The shear resistance for tubular members under transverse loads or torsion may be calculated as follows:

$$F_{vr} = \phi_v \left[\frac{0.533F_{tu}\left(1+\mu_{12}\right)}{\mu}\right]\left[\frac{G}{F_{vr}}\right]\left[\frac{t}{R_1}\right] \leq \phi_v F_{vu} \tag{8.6.3.7-1}$$

where μ is determined according to Article 8.6.3.2.

C8.6.3.7

The shear resistance equation is provided for orthotropic materials, but may be used for planar isotropic materials by setting $E_1 = E_2$ in the equation for μ. To determine the computed shear stress, f_v, equations to compute maximum shear stresses due to transverse loads and torsion are provided in Appendix B.

8.6.4—Combined Stresses

Members subjected to combined bending, axial compression, or tension may be proportioned to meet the limitations of Article 8.6.4.1 or Article 8.6.4.2, as applicable.

8.6.4.1—Bending and Compression

Members subjected to bending and compression may satisfy the following equations:

$$\frac{f_a}{\phi_c F_{cr}} + \frac{B_2 f_b}{\phi_b F_{br}} \leq 1.0 \tag{8.6.4.1-1}$$

where:

$$B_2 = 1 - \frac{f_a}{F'_e} \tag{8.6.4.1-2}$$

$$F'_e = \phi_c \frac{\pi^2 E_c}{\left(\dfrac{k_b L}{r_b}\right)^2} \tag{8.6.4.1-3}$$

and:

$$\frac{f_a}{\phi_c F_{cr}} + \frac{f_b}{\phi_b F_{br}} \leq 1.0 \tag{8.6.4.1-4}$$

where F_{cr} in Eq. 8.6.4.1-2 is the compression resistance due to local buckling.

C8.6.4.1

For practical design, simplified combined resistance ratio equations similar to those used for metal structures have been adopted for FRP (ASCE 1985). Although composite materials do not fail according to maximum principal stress criterion, the simplified equations should give conservative approximations. Research is needed in this area to develop equations based on the failure criteria of FRP.

Two equations are presented to check combined bending and compression stresses. Eq. 8.6.4.1-1 includes the term:

$$B_2 = 1 - \frac{f_a}{F'_e} \tag{C8.6.4.1-1}$$

which accounts for the second-order moments that occur as a result of the P-Δ effect. The equation is intended for intermediate unbraced locations where the member is susceptible to lateral displacement. Eq. 8.6.4.1-4 is intended for locations at the end of the member where lateral displacement is restrained. The combined stresses at such locations may, in some cases, exceed those at the intermediate points. B_2 is 1.0 where the second-order moments are determined from analysis per Section 4.

For biaxial bending, except for round and polygonal tubular sections, the second term of Eq. 8.6.4.1-1 can be substituted by:

$$\frac{f_{bx}}{\phi_b F_{brx}\left(1 - \dfrac{f_a}{F'_{ex}}\right)} + \frac{f_{by}}{\phi_b F_{bry}\left(1 - \dfrac{f_a}{F'_{ey}}\right)} \leq 1.0 \tag{C8.6.4.1-2}$$

and the second term $f_b/\phi F_{br}$ of Eq. 8.6.4.1-2 can be substituted by $f_{bx}/\phi F_{brx} + f_{by}/\phi F_{bry}$.

8.6.4.2—Bending and Tension

Members subjected to bending and tension may satisfy the following equation:

$$\frac{f_t}{\phi_t F_{tr}} + \frac{f_b}{\phi_b F_{br}} \leq 1.0 \qquad (8.6.4.2\text{-}1)$$

C8.6.4.2

With the exception of round tubular sections and polygonal sections, the term $f_b/\phi F_{br}$ can be substituted by $f_{bx}/\phi F_{brx} + f_{by}/\phi F_{bry}$ in Eq. 8.6.4.2-1.

8.7—REFERENCES

ASTM. 2001. "Standard Specification for Reinforced Thermosetting Plastic Poles," ASTM D4923-01. In *Annual Book of ASTM Standards*. American Society for Testing and Materials, West Conshohocken, PA. (withdrawn 2010, no replacement)

ASTM. 2010. "Standard Test Method for Compressive Properties of Rigid Plastics," ASTM D695-10. In Annual Book of ASTM Standards. American Society for Testing and Materials, West Conshohocken, PA.

ASTM. 2010. "Standard Test Method for Flexural Properties of Unreinforced and Reinforced Plastics and Electrical Insulating Materials," ASTM D790-10. In *Annual Book of ASTM Standards*. American Society for Testing and Materials, West Conshohocken, PA.

ASTM. 2010. "Standard Test Method for Shear Strength of Plastics by Punch Tool," ASTM D732-10. In *Annual Book of ASTM Standards*. American Society for Testing and Materials, West Conshohocken, PA.

ASTM. 2010. "Standard Test Method for Tensile Properties of Plastics," ASTM D638-10. In *Annual Book of ASTM Standards*. American Society for Testing and Materials, West Conshohocken, PA.

ASTM. 2012. "Standard Practice for Operating Fluorescent Ultraviolet (UV) Lamp Apparatus for Exposure of Nonmetallic Materials," ASTM G154-12a. In *Annual Book of ASTM Standards*. American Society for Testing and Materials, West Conshohocken, PA.

ASTM. 2014. "Standard Test Method for Tensile Properties of Polymer Matrix Composite Materials," ASTM D3039/ D3039m-14. In *Annual Book of ASTM Standards*. American Society for Testing and Materials, West Conshohocken, PA.

ASCE. 1985. *Structural Plastics Design Manual*, ASCE Manuals and Reports on Engineering Practice No. 63. American Society of Civil Engineers, New York, NY.

ASM International. 1995. *Engineered Materials Handbook*. ASM International, Materials Park, OH.

Barbero, E. J., and R. A. Ritchey. 1993. "A Universal Design Equation for FRP Columns," Paper presented at the 48th Annual Conference, Composites Institute, Cincinnati, OH, February 8–11, 1993. The Society of the Plastics Industry, Inc., Washington, DC.

Barbero, E. J., and I. G. Raftoyiannis. 1993. "Local Buckling of FRP Beams and Columns," *Journal of Materials in Civil Engineering*. American Society of Civil Engineers, New York, NY, Vol. 5, No. 3 (August 1993).

Johnson, A. F. 1985. "Design of RP Cylinders under Buckling Loads." Paper presented at the 40th Annual Conference, Reinforced Plastics/Composites Institute, Atlanta, GA, January 28–February 1, 1985, The Society of the Plastics Industry, Inc., Washington, DC.

Masmoudi, Slaughter, W. S., N. A. Fleck, and B. Budiansky. 1993. "Compressive Failure of Fiber Composites: The Roles of Multiaxial Loading and Creep," *Journal of Engineering Materials and Technology*. American Society of Mechanical Engineers, New York, NY, Vol. 115.

Masmoudi, R., Mohamed, H., and S. Metiche. 2008. "Finite Element Modeling For Deflection and Bending Responses of GFRP Poles," *Journal of Reinforced Plastics and Composites*, Vol. 27, No. 6.

SECTION 9: WOOD DESIGN

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SECTION 9:

WOOD DESIGN

9.1—SCOPE

This Section specifies design provisions for wood structural supports for highway signs, luminaires, and traffic signals. The provisions of this Section apply only to cantilevered wood posts and poles. The design provisions for wood posts and poles are generally based on the *National Design Specification for Wood Construction* (NDS, 2010), including the *Design Values for Wood Construction* (NDS Supplement, 2012), except as modified herein.

C9.1

The design provisions of this Section are applicable to common post and pole usages. Additional design provisions given in the NDS may be required for other member types or usages.

9.2—DEFINITIONS

Adjusted Design Value—Reference design value multiplied by applicable adjustment factors.

Class—Group of poles that have approximately the same load-carrying capacity regardless of species.

Dressed Lumber—Lumber that has been surfaced by a planing machine on one or more sides or edges.

Dry Condition—Condition of having relatively low moisture content, i.e., not more than 19 percent for sawn lumber.

Effective Modulus of Elasticity—Modulus of elasticity multiplied by applicable adjustment factors listed in Table 9.4.3.3-2.

Grade—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—Direction, size, arrangement, appearance, or quality of the fibers in wood or lumber.

Green Condition—Condition of having relatively high moisture content, i.e., more than 19 percent for sawn lumber.

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—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 Specification for Wood Construction by the American Wood Council.

NELMA—Northeastern Lumber Manufacturers Association, a grading agency.

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.

Nominal Size—As applied to timber or lumber, the size by which it is specified and sold; often differs from the actual size.

Normal Load Duration—Condition of fully stressing a member to its adjusted design stress by the application of the full design load for a cumulative period of approximately 10 years.

NSLB—Northern Softwood Lumber Bureau, a grading agency.

Oil-Borne Preservative—A preservative that is introduced into wood in the form of an oil-based solution.

Pole—Solid wood member, round in cross-section, of any size or length, usually used with the larger end in the ground.

Post—Solid wood member with a square or nearly square cross-section, with the width not more than 2 in. greater than the thickness.

Preservative—Any substance that is effective in preventing the development and action of wood-decaying fungi, borers of various kinds, and harmful insects.

Size Adjustment Factors—Factors that adjust reference design values listed in Table 9.4.2.3-1 for effects of member size.

Reference Design Value—The reference stress value or modulus of elasticity specified in the *NDS*.

SPIB—Southern Pine Inspection Bureau, a grading agency.

Strain Pole—Poles that support span-wires for traffic signals

Structural Lumber—Lumber that has been graded and assigned design values based on standardized procedures to ensure acceptable reliability.

Visually 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—West Coast Lumber Inspection Bureau, a grading agency.

Wet-Use—Use conditions where the moisture content of the wood in service exceeds the dry condition.

WWPA—Western Wood Products Association, a grading agency.

9.3—NOTATION

a = support condition parameter (0.7 for cantilever members) (9.4.3.2) (C9.4.3.2)
b = width of rectangular bending member (in.) (C9.7)
C_M = wet service factor (9.4.3.2) (9.4.3.3) (9.4.4.2.1)
C_u = untreated factor for timber poles (9.4.2.3) (9.4.3.3) (9.4.4.4)
D = diameter of round section (in.) (9.4.3.2) (C9.7)
D_b = diameter at groundline section of the member (in.) (9.4.3.2) (C9.4.3.2)
D_t = diameter at top of the member (in.) (9.4.3.2)
d = depth of rectangular bending member (in.) (9.4.3.2) (C9.7) (9.9)
d_b = dimension of one side of the rectangular section at groundline section of the member (in.) (9.4.3.2) (C9.4.3.2)
d_t = dimension of one side of the rectangular section at top of the member (in.) (9.4.3.2)
E = reference modulus of elasticity (ksi) (9.4.2.3) (9.4.3.3) (9.4.4.2.1)
E' = adjusted modulus of elasticity (ksi) (9.4.2.3) (9.4.3.3) (9.9)
F_b = reference bending design value (ksi) (9.4.2.3) (9.4.3.3) (9.4.4.2.1) (9.6) (C9.6) (9.9)
F'_b = adjusted bending stress (ksi) (9.4.2.3) (9.4.3.3)
F'_{bx} = adjusted bending stress about x axis (strong axis) (ksi) (C9.9)
F'_{by} = adjusted bending stress about y axis (weak axis) (ksi) (C9.9)
F_c = reference compression design value parallel to grain (ksi) (9.4.2.3) (C9.4.2.3) (9.4.3.2) (9.4.3.3) (9.4.2.2.1)
F'_c = adjusted compression stress parallel to grain (ksi) (9.4.2.3) (9.8) (9.9)
F_{cE} = critical buckling stress for compression members (ksi) (9.9) (C9.9)
F_{cp} = reference compression design value perpendicular to grain (ksi) (9.4.2.3) (9.4.2.1) (9.4.3.3) (9.4.4.2.1)
F'_{cp} = adjusted compression stress perpendicular to grain (ksi) (9.4.2.3) (9.4.3.3)
F_t = reference tension design value parallel to grain (ksi) (9.4.2.3) (9.4.3.3) (9.4.4.2.1)
F'_t = adjusted tension stress parallel to grain (ksi) (9.4.2.3)
F_v = reference shear design value parallel to grain (horizontal shear) (ksi) (9.4.2.3) (9.4.3.3) (9.4.4.2.1)
F'_v = adjusted shear stress parallel to grain (horizontal shear) (ksi) (9.4.2.3) (9.4.3.3) (9.7)

f_b = computed bending stress (ksi) (9.6) (9.9)
f_{bx} = computed bending stress about x axis (strong axis) (ksi) (C9.9)
f_{by} = computed bending stress about y axis (weak axis) (ksi) (C9.9)
f_c = computed compression stress parallel to grain (ksi) (C9.4.2.3) (C9.7) (9.8) (9.9) (C9.9)
f_v = computed shear stress parallel to grain (ksi) (9.7)
I = moment of inertia of cross-section about centroidal axis (in.^{4}) (C9.6) (9.7)
K_{cE} = Euler buckling coefficient for columns (9.9)
K_F = LRFD conversion factor (9.4.2.3) (9.4.3.3) (9.5.2.3)
L_e = effective length of a bending or compression member (in.) (9.9)
L_u = unsupported length of bending member (in.) (9.9)
M = bending moment (kip-in.) (C9.6)
Q = static moment of area about the neutral axis (in.^{3}) (C9.7)
V = shear force (kips) (C9.7)
y = distance to the outer fiber (in.) (C9.6)
λ = time effect factor (9.4.2.3) (9.4.3.3) (9.4.4.3)
ϕ = resistance factor (9.4.2.3) (9.4.3.3)
ϕ_b = resistance factor for bending (9.5.2.2)
ϕ_c = resistance factor for compression (9.5.2.2)
ϕ_{cperp} = resistance factor for compression perpendicular (9.5.2.2)
ϕ_{Emin} = resistance factor for elastic modulus minimum (9.5.2.2)
ϕ_t = resistance factor for tension (9.5.2.2)
ϕ_v = resistance factor for shear (9.5.2.2)

9.4—MATERIAL

9.4.1—General

This Article addresses the following wood products for:

- Posts and
- Round poles.

C9.4.1

Posts and poles are the most commonly used wood products for structural supports for highway signs, luminaires, and traffic signals. This Article is limited to the coverage of wood posts from visually graded lumber and round timber poles.

Visually graded lumber is a type of structural lumber graded by visual examination based on certain rules established by the grading agency.

In general, posts are used to support small structures such as roadside signs. Round timber poles are used as vertical supports for street lighting or strain poles for temporary span-wire configurations.

Engineered wood products such as laminated veneer lumber may be used for structural supports such as posts. Design of these products, however, should be based on technical information provided by the manufacturer and approved by the Owner, because the reference design values could vary for products from different manufacturers.

Reference design values apply to normal load duration. Normal load duration is defined as the condition of fully stressing a member to its adjusted stress by the application of the full design load for a cumulative period of approximately 10 yr.

9.4.2—Posts

9.4.2.1—General

In calculating stresses for posts, the net section of the member shall be used. The net section shall be determined by deducting from the gross section the projected area of all material removed by boring, grooving, dapping, notching, or other means.

9.4.2.2—Dimensions

Stresses in posts shall be computed on the basis of the net dimension of the cross-section. For 4-in. (nominal) wide posts, the net dry dressed dimensions shall be used in stress checks regardless of the moisture content at the time of manufacture or use. For 5-in. (nominal) and wider posts, the net green dressed dimensions shall be used in stress checks regardless of the moisture content at the time of manufacture or use.

C9.4.2.2

For all standard dressed posts 4 in. (nominal) and wider, the dressed dimensions used in stress checks are equal to 0.5 in. less than the nominal dimensions.

9.4.2.3—Reference Design Values

Reference design values for posts are given in Tables 9.4.2.3-1 and 9.4.2.3-2.

Reference design values shall be multiplied by all applicable factors as shown in Table 9.4.2.3-3 to determine the adjusted stresses and the effective modulus of elasticity of wood members.

C9.4.2.3

Reference design values listed in Tables 9.4.2.3-1 and 9.4.2.3-2 are provided for some common species and grades of lumber for posts. For other species or grades not listed in these tables, the NDS values should be used.

Round or square cross-sections are not susceptible to lateral torsional buckling. However, lateral torsional buckling should be considered in the case of rectangular sections bent about their major axis. A beam stability factor is provided by the NDS to modify the adjusted stresses in cases of bending of rectangular members. The modification of the adjusted bending stress for rectangular sign posts may be neglected when the post depth does not exceed the post thickness by more than 2 in.

Cantilever members such as those covered by this Section (i.e., posts and poles) are usually subjected to small axial compressive loads. Therefore, a reduction in the compressive stress to account for the slenderness of the member is not considered herein. Slenderness should be considered for members subjected to appreciable axial compressive loads. As conservative criteria, slenderness effects should be considered when f_c/F_c is greater than 0.1. A column stability factor is provided by the NDS to modify the adjusted compressive stresses for the effects of member slenderness.

Table 9.4.2.3-1—Reference Design Values for Visually Graded Lumber Posts for 4 × 4 in. through 4 × 6 in. (See note d for adjustments of reference design values.)

Species and Commercial Grade	Bending F_b ksi	Tension Parallel to Grain, F_t ksi	Shear Parallel to Grain, F_v ksi	Compression Perpendicular to Grain, F_{cp} Ksi	Compression Parallel to Grain, F_c ksi	Modulus of Elasticity, E ksi ×10^3	Grading Rules Agency
Douglas Fir-Larch							WCLIB WWPA
Select Structural	1.500	1.000	0.180	0.625	1.700	1.900	
No. 1	1.000	0.675	0.180	0.625	1.500	1.700	
No. 2	0.900	0.575	0.180	0.625	1.350	1.600	
Hem Fir							WCLIB
Select Structural	1.400	0.925	0.150	0.405	1.500	1.600	WWPA
No. 1	0.975	0.625	0.150	0.405	1.350	1.500	
No. 2	0.850	0.525	0.150	0.405	1.300	1.300	
Southern Pine 4 × 4 in.							SPIB
Select Structural	2.850	1.600	0.175	0.565	2.100	1.800	
No. 1	1.850	1.050	0.175	0.565	1.850	1.700	
No. 2	1.500	0.825	0.175	0.565	1.650	1.600	
Southern Pine 4 × 5 or 6 in.							SPIB
Select Structural	2.550	1.400	0.175	0.565	2.000	1.800	
No. 1	1.650	0.900	0.175	0.565	1.750	1.700	
No. 2	1.250	0.725	0.175	0.565	1.600	1.600	
Spruce-Pine-Fir (South)							NELMA NSLB
Select Structural	1.300	0.575	0.135	0.335	1.200	1.300	WCLIB
No. 1	0.875	0.400	0.135	0.335	1.050	1.200	WWPA
No. 2	0.775	0.350	0.135	0.335	1.000	1.100	
Western Cedar							WCLIB
Select Structural	1.000	0.600	0.155	0.425	1.000	1.100	WWPA
No. 1	0.725	0.425	0.155	0.425	0.825	1.000	
No. 2	0.700	0.425	0.155	0.425	0.650	1.000	

Notes:

a. Reference design values for other grades or species of posts are given in the NDS.

b. Reference design values are based on dry service conditions (i.e., moisture content less than or equal to 19 percent). For wet service conditions (i.e., moisture content greater than 19 percent), provisions of Article 9.4.4.2.1 shall be considered.

c. Values for modulus of elasticity are average values that conform to ASTM D245 and ASTM D1990. Adjustments in modulus of elasticity have been taken to reflect appropriate increases for seasoning; increases for density where applicable; and, where required, reductions have been made to account for the effect of grade on stiffness.

d. For all species other than southern pine, the tabulated bending, tension, and compression parallel to grain reference design values shall be multiplied by the following size-adjustment factors to determine the actual reference design values (size adjustment factors have already been incorporated in the tabulated values for southern pine):

Size-Adjustment Factors

Post Size	F_b	F_t	F_c
4 × 4 in.	1.5	1.5	1.15
4 × 5 in.	1.4	1.4	1.1
4 × 6 in.	1.3	1.3	1.1

Table 9.4.2.3-2—Reference Design Values for Visually Graded Lumber Posts 5 × 5 in. and Larger

Species and Commercial Grade	Bending F_b ksi	Tension Parallel to Grain, F_t ksi	Shear Parallel to Grain, F_v ksi	Compression Perpendicular to Grain, F_{cp} ksi	Compression Parallel to Grain, F_c ksi	Modulus of Elasticity, E ksi × 10^3	Grading Rules Agency
Douglas Fir-Larch							WCLIB
Select Structural	1.500	1.000	0.170	0.625	1.150	1.600	
No. 1	1.200	0.825	0.170	0.625	1.000	1.600	
No. 2	0.750	0.475	0.170	0.625	0.700	1.300	
Douglas Fir-Larch							WWPA
Select Structural	1.500	1.000	0.170	0.625	1.150	1.600	
No. 1	1.200	0.825	0.170	0.625	1.000	1.600	
No. 2	0.700	0.475	0.170	0.625	0.700	1.300	
Hem Fir							WCLIB
Select Structural	1.200	0.800	0.140	0.405	0.950	1.300	
No. 1	0.975	0.650	0.140	0.405	0.850	1.300	
No. 2	0.575	0.375	0.140	0.405	0.575	1.100	
Hem Fir							WWPA
Select Structural	1.200	0.800	0.140	0.405	0.950	1.300	
No. 1	0.950	0.650	0.140	0.405	0.800	1.300	
No. 2	0.525	0.350	0.140	0.405	0.575	1.100	
Southern Pine							SPIB
Select Structural	1.500	1.000	0.165	0.425	0.950	1.500	
No. 1	1.350	0.900	0.165	0.425	0.825	1.500	
No. 2	0.850	0.550	0.165	0.425	0.525	1.200	
Spruce-Pine-Fir (South)							NELMA
Select Structural	1.000	0.475	0.125	0.335	0.700	1.200	NSLB
No. 1	0.800	0.550	0.125	0.335	0.625	1.200	WWPA
No. 2	0.475	0.325	0.125	0.335	0.425	1.000	
Western Cedar							WCLIB
Select Structural	1.100	0.725	0.140	0.425	0.925	1.000	
No. 1	0.875	0.600	0.140	0.425	0.800	1.000	
No. 2	0.550	0.350	0.140	0.425	0.550	0.800	
Western Cedar							WWPA
Select Structural	1.100	0.725	0.140	0.425	0.925	1.000	
No. 1	0.875	0.600	0.140	0.425	0.800	1.000	
No. 2	0.500	0.350	0.140	0.425	0.550	0.800	

Notes:

a. Reference design values for other grades or species of posts are given in the NDS.

b. Reference design values are based on dry service conditions (i.e., moisture content less than or equal to 19 percent), except for Southern Pine. For wet service conditions (i.e., moisture content greater than 19 percent), provisions of Article 9.4.4.2.1 shall be considered. Reference design values for Southern Pine are based on wet service conditions, and they may be used for dry service conditions.

c. Values for modulus of elasticity are average values that conform to ASTM D245 and ASTM D1990. Adjustments in modulus of elasticity have been taken to reflect appropriate increases for seasoning; increases for density where applicable; and, where required, reductions have been made to account for the effect of grade on stiffness.

Table 9.4.2.3-3—Adjusted Stresses and Effective Modulus of Elasticity for Posts

Adjusted bending stress	$F'_b = \phi\,\lambda\,K_F\,C_M F_b$
Adjusted tension stress (parallel to grain)	$F'_t = \phi\,\lambda\,K_F\,C_M F_t$
Adjusted compression stress (perpendicular to grain)	$F'_{cp} = \phi\,\lambda\,K_F\,C_M F_{cp}$
Adjusted compression stress (parallel to grain)	$F'_c = \phi\,\lambda\,K_F\,C_M F_c$
Adjusted shear stress (parallel to grain)	$F'_v = \phi\,\lambda\,K_F\,C_M F_v$
Effective modulus of elasticity	$E' = C_M E$

Notes:

a. C_M is the wet service factor defined in Article 9.4.4.2.1 for posts.

c. The F_b, F_t, and F_c values for posts shall include applicable adjustments from the size-adjustment factors given in Table 9.4.2.3-1.

d. λ is the time effect factor.

e. ϕ is the resistance factor.

9.4.3—Poles

9.4.3.1—General

In calculating stresses for posts, the net section of the member shall be used. The net section shall be determined by deducting from the gross section the projected area of all material removed by boring, grooving, dapping, notching, or other means.

9.4.3.2—Dimensions

Design dimensions of poles shall conform to applicable provisions of ASTM D3200.

For the calculations of compression stresses and buckling loads of tapered compression members with rectangular cross-section, the representative dimension, d, for each face of the member shall be determined as:

$$d = d_t + \left(d_b - d_t\right)\left[a - 0.15\left(1 - \frac{d_t}{d_b}\right)\right] \qquad (9.4.3.2\text{-}1)$$

For the design of tapered members with round cross-section, the representative diameter, D, shall be derived using Eq. 9.4.3.2-1 by replacing d_b by D_b, and d_t by D_t.

Calculations of the computed compression stress parallel to grain, f_c, shall be based on the representative dimensions of d, for rectangular members, or the representative diameter, D, for round members.

C9.4.3.2

Poles are grouped by class according to their required minimum circumference at 6 ft. from the butt of the pole. Tables C9.4.3.2-1 and C9.4.3.2-2 provide typical dimensions for wood poles. For other species not presented in the tables, reference should be made to ASTM D3200.

Poles of a given class and length have approximately the same load-carrying capacity regardless of species. Therefore, poles can be specified by class number and length without reference to species.

Eq. 9.4.3.2-1 provides the cross-section dimensions of wood members at the critical section for compression. The support condition parameter a in Eq. 9.4.3.2-1 accounts for the particular support conditions at the ends of the tapered member. The value a = 0.70 is valid only for tapered cantilever posts and poles.

The values of d_b and D_b are at the groundline section of the member. The taper of a pole may be approximated from the tabulated values for minimum circumference at top of pole and at 6 ft from the butt.

Table C9.4.3.2-1—Dimensions of Red Pine Poles

Class	**1**	**2**	**3**	**4**	**5**
Minimum circumference at top in.	27	25	23	21	19
Length of pole ft	Minimum circumference at 6 ft from butt, in.				
20	32.5	30.5	28.5	26.5	24.5
25	36.0	33.5	31.0	29.0	27.0
30	39.0	**36.5**	**34.0**	**31.5**	**29.0**
35	41.5	**38.5**	**36.0**	**33.5**	**31.0**
40	**44.0**	**41.0**	**38.0**	**35.5**	**33.0**
45	**46.0**	**43.0**	**40.0**	**37.0**	**34.5**
50	**48.0**	**45.0**	**42.0**	39.0	36.0
55	**49.5**	**46.5**	**43.5**	40.5	—
60	**51.5**	**48.0**	**45.0**	42.0	—

Notes:

a. Dimensions for other species are given in ASTM D3200.

b. Classes and lengths for which circumferences at 6 ft from the butt, listed in bold-face type, are the preferred standard sizes. Those shown in light type are included for engineering purposes only.

Table C9.4.3.2-2—Dimensions of Douglas Fir and Southern Pine Poles

Class	**1**	**2**	**3**	**4**	**5**
Minimum circumference at top in.	27	25	23	21	19
Length of pole (ft)	Minimum circumference at 6 ft from butt in.				
20	31.0	29.0	27.0	25.0	23.0
25	33.5	31.5	29.5	27.5	25.5
30	36.5	**34.0**	**32.0**	**29.5**	**27.5**
35	39.0	**36.5**	**34.0**	**31.5**	**29.0**
40	**41.0**	**38.5**	**36.0**	**33.5**	**31.0**
45	**43.0**	**40.5**	**37.5**	**35.0**	**32.5**
50	**45.0**	**42.0**	**39.0**	36.5	34.0
55	**46.5**	**43.5**	**40.5**	38.0	—
60	**48.0**	**45.0**	**42.0**	39.0	—

Notes:

a. Dimensions for other species are given in ASTM D3200.

b. Classes and lengths for which circumferences at 6 ft. from the butt, listed in bold-face type, are the preferred standard sizes. Those shown in light type are included for engineering purposes only.

9.4.3.3—Reference Design Values

Reference design values for round timber poles are given in Table 9.4.3.3-1 to be adjusted by factors in Table 9.4.3.3-2.

C9.4.3.3

Table 9.4.3.3-1 lists reference design values using this recommendation for three species of poles. Designs preformed for each of these species require the same class or approximately the same class of pole for a given loading and length of pole. For poles specified by class and length only, designs based on the southern pine species will result in the average class requirement for these species.

Table 9.4.3.3-1—Reference Design Values for Treated Round Poles

Species	Bending, F_b ksi	Tension Parallel to Grain, F_t ksi	Shear Parallel to Grain, F_v ksi	Compression Perpendicular to Grain, F_{cp} ksi	Compression Parallel to Grain, F_c ksi	Modulus of Elasticity, E ksi × 10^3
Douglas Fir	1.900	—	0.015	0.230	1.000	1.500
Red Pine	1.450	—	0.085	0.155	0.725	1.280
Southern Pine	1.850	—	0.110	0.250	0.950	1.500

Note:
a. Reference design values are based on wet service conditions (i.e., moisture content greater than 19 percent), and may be used for dry service conditions (i.e., moisture content less than or equal to 19 percent).

Table 9.4.3.3-2—Adjusted Stresses and Effective Modulus of Elasticity for Poles

Adjusted bending stress	$F'_b = \phi\, \lambda\, K_F\, C_u\, F_b$
Adjusted tension stress (parallel to grain)	—
Adjusted compression stress (perpendicular to grain)	$F'_{cp} = \phi\, \lambda\, K_F C_u F_{cp}$
Adjusted compression stress (parallel to grain)	$F'_c = \phi\, \lambda\, K_F\, C_u\, F_c$
Adjusted shear stress (parallel to grain)	$F'_v = \phi\, \lambda\, K_F\, C_u\, F_v$
Effective modulus of elasticity	$E' = 1.0\, E$

Notes:

b. C_u is the untreated factor defined in Article 9.4.4.4.

c. The F_b, F_t, and F_c values for posts shall include applicable adjustments from the size-adjustment factors given in Tables 9.4.2.3-1 and 9.4.2.3-2.

9.4.4—Adjustment Factors for Reference Design Values

9.4.4.1—General

Adjustment factors shall be applied consistent with the in-service condition. The adjustments are different for posts and poles.

9.4.4.2—Wet Service Factors

9.4.4.2.1—Wet Service Factor Posts

Reference design values from Tables 9.4.2.3-1 and 9.4.2.3-2 shall be used for the design of posts under dry service conditions, where the moisture content in use will be a maximum of 19 percent. If the moisture content of the post is expected to exceed 19 percent, the reference design values shall be multiplied by the wet service factor, C_M, specified in Table 9.4.4.2.1-1.

Table 9.4.4.2.1-1—Wet Service Factor (Posts), C_M

Post Size	F_b	F_t	F_v	F_{cp}	F_c	E
Posts 4 × 4 in. through 4 × 6 in.	0.85	1.00	0.97	0.67	0.80	0.90
Posts 5 × 5 in. and larger	1.00	1.00	1.00	0.67	0.91	1.00

Notes:

a. When $F_b \leq 1.150$ ksi, $C_M = 1.0$ for posts 4 × 4 in. through 4 × 6 in.

b. When $F_c \leq 0.750$ ksi, $C_M = 1.0$ for F_c of posts 4 × 4 in. through 4 × 6 in.

c. For southern pine posts 5 × 5 in. and larger, $C_M = 1.0$ for all reference design values.

9.4.4.2.2—Wet Service Factor Poles

Reference design values in Table 9.4.3.3-1 shall be used for the design of poles under wet service conditions, and their use is conservative for dry service conditions.

9.4.4.3—Time Effect Factor

The time effect factor, λ, shall be:

λ = 1.0 for the extreme limit state, and

λ = 0.6 for the Strength I limit state.

9.4.4.4—Untreated Factor for Poles

When poles are air dried or kiln dried only prior to pressure treatment, C_u shall be taken as follows:

- C_u shall be taken as 1.18 for Southern Pine and
- C_u shall be taken as 1.11 for other species.

C9.4.4.4

Reference design values listed in Table 9.4.3.3-1 are provided for treated poles, and they include an adjustment to compensate for strength reductions due to steam conditioning or boultonizing prior to treatment. When poles are air dried or kiln dried only prior to treatment, the untreated factor can be used.

Poles are often ordered by class number only; therefore, the exact seasoning or conditioning process of the supplied poles may not be known. ASTM D3200 provides general information on seasoning and conditioning processes for various species of poles.

9.5—LIMIT STATES

9.5.1—General

Support components shall be proportioned to satisfy the requirements of the strength and extreme event limit states.

Axial, shear, and bending stresses shall be computed based on the linear elastic theory.

9.5.2—Strength Limit States

9.5.2.1—General

Strength and stability shall be considered using the applicable strength load combinations specified in Table 3.4.1-1. Factored resistance shall be the product of nominal resistance as determined in accordance with the applicable provisions of Articles 9.5.2.2 and 9.5.2.3.

9.5.2.2—Resistance Factors

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

- For bending, $\phi_b = 0.85$
- For tension, $\phi_t = 0.80$
- For compression, $\phi_c = 0.90$
- For compression perpendicular, $\phi_{cperp} = 0.90$
- For elastic modulus minimum, $\phi_{E\,min} = 0.85$
- For shear, $\phi_v = 0.75$

9.5.2.3—LRFD Conversion Factors

LRFD conversion factors, K_F, for the strength limit states shall be taken as follows:

- For bending, $K_F = 2.54$
- For tension, $K_F = 2.70$
- For compression, $K_F = 2.40$
- For compression perpendicular, $K_F = 1.67$
- For elastic modulus minimum, $K_F = 1.76$
- For shear, $K_F = 2.88$

9.5.3—Extreme Event Limit State

All applicable load combinations in Table 3.4-1 for the extreme event limit state shall be investigated. The resistance and LRFD conversion factors for the extreme event shall be as defined in the strength limit state in Articles 9.5.2.2 and 9.5.2.3.

9.6—COMPONENTS IN FLEXURE

Members subjected to flexural stress shall be proportioned for the Strength I and extreme event limit states so that:

$$\frac{f_b}{F_b'} \leq 1.0 \tag{9.6-1}$$

where:

f_b = the stress due to the loads (ksi), and

F_b' = the resistance defined in Articles 9.4.2.3 and 9.4.3.3 (ksi).

C9.6

The general expression to compute the maximum flexural stress is:

$$f_b = \frac{My}{I} \text{ (ksi)} \tag{C9.6-1}$$

where:

M = the bending moment (in.-kips),

I = the moment of inertia (in.^{4}), and

y = the distance to the outer fiber where the stress is computed (in.).

9.7—COMPONENTS IN SHEAR

Members subjected to shear stress shall be proportioned for the Strength I and extreme event limit states so that:

$$\frac{f_v}{F_v'} \leq 1.0 \tag{9.7-1}$$

where:

f_v = the stress due to the loads (ksi), and

F_v' = the resistance defined in Articles 9.4.2.3 and 9.4.3.3.

C9.7

The general expression to compute the maximum shear stress parallel to grain is:

$$f_v = \frac{VQ}{Ib} \text{ (ksi)} \tag{C9.7-1}$$

where:

V = shear force (kip),

I = moment of inertia (in^4),

Q = first moment of area of the section above the neutral axis (in^3), and

b = section thickness at the location of interest (in.).

For a rectangular bending member of width b, and depth d, Eq. C9.7-1 may be expressed as:

$$f_v = \frac{3V}{2bd} \text{ (ksi)} \tag{C9.7-2}$$

and, for a solid round bending member of diameter D, Eq. C9.7-1 may be expressed as:

$$f_v = \frac{16V}{3\pi D^2} \text{ (ksi)} \tag{C9.7-3}$$

9.8—COMPONENTS IN COMPRESSION

Members subjected to compressive stress shall be proportioned for the Strength I and extreme event limit states so that:

$$\frac{f_c}{F_c'} \leq 1.0 \tag{9.8-1}$$

where:

f_c = the stress due to the loads (ksi), and

F_c' = the resistance defined in Articles 9.4.2.3 and 9.4.3.3, (ksi).

9.9—COMPONENTS WITH AXIAL AND FLEXURE ACTIONS

Members subjected to a combination of bending and axial compression shall be proportioned so that:

$$\left(\frac{f_c}{F_c'}\right)^2 + \frac{B_2 f_b}{F_b'} \leq 1.0 \tag{9.9-1}$$

where:

$$F_{cE} = \frac{0.75 K_{cE} E'}{\left(\frac{L_e}{D}\right)^2} \tag{9.9-2}$$

for round members, and:

$$F_{cE} = \frac{K_{cE} E'}{\left(\frac{L_e}{d}\right)^2} \tag{9.9-3}$$

calculated in the plane of bending for rectangular members. K_{cE} shall be 0.3 for posts and timber poles.

The effective column length L_e may be taken as 2.1 L_u for cantilever members where L_u is the unsupported length

Members subjected to a combination of bending and axial tension shall be proportioned so that:

$$\left(\frac{f_c}{F_c'}\right) + \frac{f_b}{F_b'} \leq 1.0 \tag{9.9-4}$$

C9.9

B_2 accounts for second-order effect as outlined in Section 4. Alternatively, B_2 may be computed as:

$$B_2 = \frac{1}{1 - \frac{f_c}{F_{cE}}}$$

is a factor that accounts for secondary bending moments that occur as a result of the P-delta effects. Secondary bending effects can normally be neglected for posts of roadside signs. Secondary effects can be neglected when the axial stress ratio $\frac{f_c}{F_{cE}}$ is less than 0.1.

For biaxial bending in rectangular members, the second term of Eq. 9.9-1 can be modified to:

$$\frac{B_{2x} f_{bx}}{F_{bx}'} + \frac{B_{2y} f_{by}}{F_{by}'}$$

where B_{2x} and B_{2y} are the second-order factors in the orthogonal directions, x and y, respectively.

9.10—REFERENCES

ASTM. 2011. “Standard Practice for Establishing Clear Wood Strength Values,” ASTM D2555-06(2011). *Annual Book of ASTM Standards*. American Society for Testing and Materials, West Conshohocken, PA.

ASTM. 2011. “Standard Practice for Establishing Structural Grades and Related Allowable Properties for Visually Graded Lumber,” ASTM D245-06(2011). *Annual Book of ASTM Standards*. American Society for Testing and Materials, West Conshohocken, PA.

ASTM. 2012. “Standard Practice for Establishing Allowable Stresses for Round Timber Piles,” ASTM D2899-12. In *Annual Book of ASTM Standards*. American Society for Testing and Materials, West Conshohocken, PA.

ASTM. 2012. “Standard Specification for Round Timber Piles,” ASTM D25-12. *Annual Book of ASTM Standards*. American Society for Testing and Materials, West Conshohocken, PA.

ASTM. 2012. “Standard Specification and Test Method for Establishing Recommended Design Stresses for Round Timber Construction Poles,” ASTM D3200-74(2012). *Annual Book of ASTM Standards*. American Society for Testing and Materials, West Conshohocken, PA.

AWC. 2012. *National Design Specification for Wood Construction.* American Wood Council, Leesburg, VA.

AWPA. 2012. *Book of Standards 2013*. American Wood Protection Association (formerly the American Wood Preservers’ Association), Birmingham, AL.

ASTM. 2014. “Standard Practice for Establishing Allowable Properties for Visually Graded Dimension Lumber from In-Grade Tests of Full-Size Specimens,” ASTM D1990-14. *Annual Book of ASTM Standards*. American Society for Testing and Materials, West Conshohocken, PA.

SECTION 10: SERVICEABILITY REQUIREMENTS

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SECTION 10:

SERVICEABILITY REQUIREMENTS

10.1—SCOPE

This Section provides serviceability requirements for support structures.

10.2—DEFINITIONS

Camber—The condition of the horizontal support being arched.

Quadri-Chord Truss—A horizontal member composed of four longitudinal chords connected by bracing.

Rake—To slant or incline from the vertical.

Tri-Chord Truss—A horizontal member composed of three longitudinal chords connected by bracing.

10.3—NOTATION

E	=	modulus of elasticity (ksi) (C10.5)
H	=	height of vertical support (in.) (C10.5)
I	=	moment of inertia of vertical support (in.^{4}) (C10.5)
L	=	distance between supports for an overhead bridge structure; distance from vertical support to free end for horizontal cantilevered support (in.) (10.4.1) (C10.4.1) (10.4.3.1) (10.5) (C10.5)
M	=	moment caused by dead loads applied to the vertical support at the connection of the horizontal support (lb-in.) (C10.5)
r	=	radius of gyration (in.) (10.4.3.1)
u	=	prefabricated camber (slope) in the horizontal cantilevered arm (in./in.) (C10.5)
δ_{DL}	=	deflection at free end of horizontal support under dead load (in.) (C10.5)
δ_P	=	deflection at tip of vertical support under dead load from horizontal cantilevered support (in.) (C10.5)
δ_{PDL}	=	deflection at free end of horizontal support caused by slope at the tip of the vertical support (in.) (C10.5)
δ_{TOTAL}	=	total dead load deflection at free end of horizontal support (in.) (C10.5)
θ	=	rotation at the top of the pole (radians) (C10.5)

10.4—DEFLECTION

Highway support structures of all materials should be designed to have adequate structural stiffness that will result in acceptable serviceability performance. Deflections for specific structure types shall be limited as provided in Articles 10.4.1 and 10.4.2. Permanent camber for specific structure types shall be provided per Article 10.5.

C10.4

The deflection limits that are set by these Specifications are to serve two purposes. The first purpose is to provide an aesthetically pleasing structure under dead load conditions. The second purpose is to provide adequate structural stiffness that will result in acceptable performance under applied loads.

10.4.1—Overhead Bridge Supports for Signs and Traffic Signals

For overhead bridge monotube and truss structures supporting signs and traffic signals, the maximum vertical deflection of the horizontal support resulting from Service I load combination shall be limited to $L/150$, where L is the span length.

C10.4.1

Research was sponsored by the Arizona Department of Transportation (Ehsani et al., 1984; Martin et al., 1985) to determine an appropriate deflection limitation for steel monotube bridge support structures. This research included field tests and analytical studies using computer modeling. The studies investigated the static and dynamic behavior of monotube bridge sign support structures and determined a dead load deflection limit that should be specified for monotube bridge structures. The 1989 Interim Specifications were revised to limit deflection to the span divided by 150 for dead and ice load applications based on this research.

A later study (Lundgren, 1989) indicated that because the deflection criterion was an aesthetic limitation, it could be increased to the span divided by 100; however, no additional work has been found to justify changing the deflection limit to a more liberal value. Although this study considered only steel members, the deflection limit has been generalized for other materials because aesthetics was the governing consideration.

Other types of overhead bridge sign supports (i.e., two-chord, tri-chord, and quadri-chord trusses) generally have higher stiffness than the monotube type. A dead load deflection limit of the span divided by 150 (i.e., $L/150$) may be adopted as a conservative limit for those types of overhead bridge sign and traffic signal support structures made with two-chord, tri-chord, or quadri-chord trusses.

10.4.2—Cantilevered Supports for Signs, Luminaires, and Traffic Signals

10.4.2.1—Vertical Supports

The horizontal deflection limits for vertical supports, such as street lighting poles, traffic signal structures, and sign structures, shall be as follows:

- Under Service I load combination, the deflection at the top of vertical supports with transverse load applications shall be limited to 2.5 percent of the structure height; and
- Under Service I load combination, the slope at the top of vertical supports with moment load applications shall be limited to 0.35 in./ft.

For luminaire support structures under Service I load combination (i.e., dead load and wind), deflection shall be limited to 15 percent of the structure height.

Deflections shall be computed by usual methods or equations for elastic deflections. For prestressed concrete members, the effects of cracking and reinforcement on member stiffness shall be considered.

C10.4.2.1

The dead load deflection and slope limitations were developed based on aesthetic considerations. The 2.5 percent deflection limit was developed for transverse load applications, such as strain pole applications, where a dead load caused by span-wire tension could cause unsightly deflection. The horizontal linear displacement at the top of the structure is measured in relation to a tangent to the centerline at the structure's base. The slope limitation of 0.35 in./ft, which is equivalent to an angular rotation of 1 degree-40 minutes, was initially developed for street lighting poles with a single mast arm, where the mast arm applied a concentrated dead load moment that could also cause unsightly deflections. It is measured by the angular rotation of the centerline at the top of the structure in relation to the centerline at its base. The concentrated moment loads result from the effect of eccentric loads of single or unbalanced multiple horizontally mounted arm members and their appurtenances.

The 15 percent deflection limitation for the Service I load combination constitutes a safeguard against the design of highly flexible structures. It is intended mainly for high-level lighting poles. The deflections are calculated with the unit load factors defined in Article 3.4, and second-order effects are normally considered in the analysis.

10.4.2.2—Horizontal Supports

Adequate stiffness shall be provided for the horizontal supports of cantilevered sign and traffic signal structures that will result in acceptable serviceability performance.

Galloping and truck gust-induced vibration deflections of cantilevered single-arm sign supports and traffic signal

C10.4.2.2

No dead load deflection limit is prescribed for horizontal supports of cantilevered sign and traffic signal structures. Stiffness requirements are determined by the Designer. Structures are typically raked or the horizontal supports are cambered such that the dead load deflection at the end of the arm is above a horizontal reference. Camber requirements for arms should not be excessive so as to result in a serviceability problem, as specified in Article 11.7.

cantilevered sign and traffic signal structures are provided in Article 10.5.

10.4.3—Vibration

Structural supports that are susceptible to damaging vibrations and not designed for fatigue in accordance with Section 11 (with exception of common lighting poles and roadside signs) should be equipped with appropriate damping or energy-absorbing devices.

All aluminum overhead sign and traffic signal support structures should be equipped with appropriate damping or energy-absorbing devices to prevent significant wind-induced vibration in the structure, both before and after installation of sign panels or traffic signals.

C10.4.3

A mitigation device is not always mandatory if the structure is designed for fatigue in accordance with Section 11. Should the structure exhibit vibrations in the field, a mitigation device may be considered.

Section 11 contains provisions for designing various structural supports for fatigue using design loads that are a result of wind-induced vibrations and truck gust-induced vibrations.

Vibrations may be caused by wind-induced loads, such as galloping or vortex shedding. Moving traffic, such as a large truck passing under overhead sign structures, may induce gusts on nearby structures. Vibrations may also be a result of support movement, such as those found on bridges and elevated roadways.

For street-lighting poles, reducing vibration that is caused by wind or traffic-induced vibration of elevated roadways is important to reduce the potential for fatigue damage and to increase lamp life (Van Dusen, 1965 and 1968). Mitigation by using a Stockbridge-type damper is suggested by Burt and LeBlanc (1974) and by Dusel and Bon (1986). Vibrations caused by wind have been controlled in street lighting poles with the impact damper (Minor, 1973).

The Stockbridge-type vibration damper has been used to control vibration of aluminum overhead bridge sign structures (Lengel and Sharp 1969). For steel traffic signal structures with mast arms, research (McDonald et al., 1995) has suggested avoiding configurations that are susceptible to galloping, such as rigidly mounted traffic signals. Before these configurations (e.g., signals with an articulated connection to the arm) are used on a given structure, their acceptability from a traffic control perspective should be investigated. Permanent horizontal plane sign panels have been shown to reduce or eliminate galloping vibrations for some installations with rigidly mounted traffic signals, as discussed in Article 11.7.1.

Steel and aluminum overhead bridge sign and/or traffic signal support structures and cantilevered sign supports may be subject to damaging vibrations and oscillations when sign panels and/or traffic signals are not in place during erection or maintenance of the structure. To avoid these vibrations and oscillations, considerations should be given to providing temporary damping devices attached to the structure, such as blank sign panels.

10.4.3.1—Requirements for Individual Truss Members

The Specifications' limitations for L/r ratios should be adequate to prevent excessive vibration.

C10.4.3.1

Vibration in truss structures can occur in individual members. Slender tension members and redundant diagonals are particularly susceptible. Resistance to local vibrations can be provided by increasing member stiffness, thereby reducing flexural deflection and increasing natural frequencies.

10.5—CAMBER

Permanent camber equal to $L/1000$, where L is the unsupported length of the horizontal support, shall be provided in addition to dead load camber for overhead sign and traffic signal structures.

C10.5

The camber requirement applies to overhead bridge sign and traffic signal supports and to sign and traffic signal supports with a horizontal cantilevered support. The permanent camber can aid in compensating for deflections resulting from foundation rotation. The permanent camber is in addition to the dead load camber, which compensates for dead load deflection.

Camber is the condition of the horizontal support being arched. Permanent camber is the condition of the horizontal support being arched upward after application of the dead loads. The horizontal support should be arched upward such that the vertical distance from the attachment point(s) to location of maximum deflection for the horizontal support is equal to $L/1000$. The permanent camber provides the visual effect of a low-pitched arch, which is more appealing than a horizontal support that is deflected downward.

Permanent camber can be provided by raking the vertical support and/or cambering the horizontal support. Raking the vertical support involves installing the vertical support with an initial rotation from vertical. The vertical support is raked during construction by adjusting the leveling nuts at the base of the structure. Raking the vertical support may result in the anchor bolts not being perpendicular to the support's base plate, and it can result in anchor bolt nuts not being properly tightened against the base plate. Cambering the horizontal support involves fabricating the support with an initial slope or curvature.

The following procedure may be used to calculate the camber required to compensate for dead load deflection in a cantilevered sign support structure with a monotube vertical support.

The cantilevered horizontal support should be cambered during fabrication, such that the permanent camber after application of dead load is a minimum of $L/1000$ above the horizontal plane, where L is the span of the horizontal support.

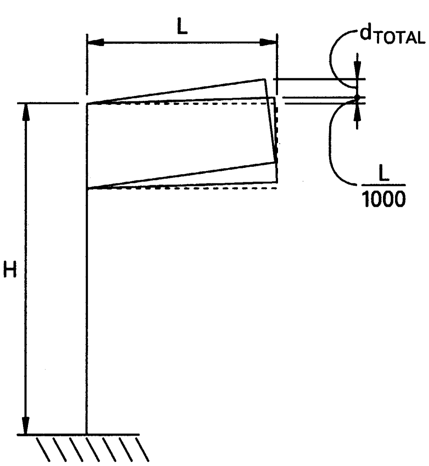

Figure 10.5-1—Camber of Cantilevered Sign Structure

In lieu of a more rigorous structural analysis, the following procedure that references Figure 10.5-1 may be used for a nontapered vertical support with a constant stiffness:

1. Determine horizontal deflection at top of vertical support due to dead load (deflection of vertical support not shown in figure), δ_P:

$$\delta_P = \frac{MH^2}{2EI} \tag{C10.5-1}$$

where:

M is the bending moment at the pole-arm connection (in-kips),
H is the height of the arm (in.),
E is the modulus of elasticity (ksi),
I is the pole moment of inertia (in.^{4}).

2. Determine the rotation at the top of the pole:

$$\theta_P = \frac{MH}{EI} \tag{C10.5-2}$$

3. Determine deflection of horizontal support due to slope at tip of vertical support, δ_{PDL}:

$$\delta_{PDL} = \theta_P L \tag{C10.5-3}$$

where:

θ_p = the rotation at the top of the pole (radian), and

L = the length of the arm (in.).

4. Determine deflection of horizontal support due to dead load acting on the horizontal support, δ_{DL}. Assuming a fixed end at the connection.

5. Calculate total dead load deflection at the tip of the cantilevered arm:

$$\delta_{\mathrm{TOTAL}} = \delta_{DL} + \delta_{PDL} \tag{C10.5-4}$$

6. Determine the slope u of the prefabricated camber in the horizontal support, such that:

$$u = \frac{1}{1000} + \frac{\delta_{DL}}{L} + \frac{\delta_{PDL}}{L} \tag{C10.5-5}$$

This slope will result in a final deflection at the end of the horizontal arm equal to $L/1000$ above the horizontal plane.

7. Provide fabrication details indicating the prefabricated camber (slope) u in horizontal support.

The above procedure does not consider the raking of the vertical support.

A slope for some cantilevered horizontal supports has been provided by tilting the arm by a small angle at its base connection.

When the total dead load deflection is small, some vertical supports have been raked to compensate for the full deflection of the cantilevered horizontal support.

10.6—REFERENCES

Brockenbrough, R. L. 1970. *Suggested Structural Design Criteria for Steel Lighting Standards*, US Steel Report 6*,* Project No. 57-019-450(6), January 1970.

Burt, J. O., and E. J. LeBlanc. 1974. *Luminaire Vibration Suppression Study,* Report No. 75. Louisiana Department of Highways, Baton Rouge, LA; Federal Highway Administration, Washington, DC.

Dusel, J. P., and V. D. Bon. 1986. *Effectiveness of Stockbridge-Type Vibration Dampers on Reducing Fatigue Failures of Luminaires and Mast Arms on Lighting Standards Installed on Bridges*. California Department of Transportation, Sacramento, CA.

Ehsani, M. R., S. K. Chakrabarti, and R. Bjorhovde. 1984. *Static and Dynamic Behavior of Monotube Span-Type Sign Structures*, Report No. FHWA/AZ-84/194-I. Arizona Department of Transportation, Phoenix, AZ.

Lengel, J. S., and M. L. Sharp. 1969. "Vibration and Damping of Aluminum Overhead Sign Structures," *Highway Research Record*, Transportation Research Board, National Research Council, Washington, DC, No. 259.

Lundgren, H. R. 1989. *Evaluation of Monotube Sign Support Structure*, Report No. FHWA-AZ89-829. Arizona Department of Transportation, Phoenix, AZ.

Martin, K. A., M. R. Ehsani, and R. Bjorhovde. 1985. *Field Testing of Monotube Sign Support Structures,* Report No. FHWA/AZ-86/237. Arizona Department of Transportation, Phoenix, AZ.

McDonald, J. R., K. C. Mehta, W. W. Oler, and N. Pulipaka. 1995. *Wind Load Effects on Signs, Luminaires, and Traffic Signals Structures*, Report No. 1303-F. Wind Engineering Research Center, Texas Department of Transportation, Lubbock, TX.

Minor, R. C. 1973. "Damp Destructive Pole Vibration," *Electric Light and Power*. December 1973.

Van Dusen, H. A. 1968. "Street Lighting Luminaire Vibration," *Illuminating Engineering*. February 1968.

Van Dusen, H. A. and D. Wandler. 1965. "Street Lighting Pole Vibration Research," *Illuminating Engineering.* November 1965.

11: FATIGUE DESIGN

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SECTION 11:

FATIGUE DESIGN

11.1—SCOPE

This Section contains provisions for the fatigue design of cantilevered and noncantilevered steel and aluminum structural supports for highway signs, luminaires, and traffic signals.

C11.1

This Section focuses on fatigue, which is defined herein as the damage that may result in fracture after a sufficient number of stress fluctuations. It is based on NCHRP Report 412, *Fatigue Resistant Design of Cantilevered Signal, Sign and Light Supports* (Kaczinski et al, 1998), NCHRP Report 469, *Fatigue-Resistant Design of Cantilever Signal, Sign, and Light Supports* (Dexter and Ricker, 2002), NCHRP Report 494, *Structural Supports for Highway Signs, Luminaires, and Traffic Signals* (Fouad et al, 2003), NCHRP Web Only Document 176: *Cost Effective Connection Details for Highway Sign, Luminaire and Traffic Signal Structures, Final Report of NCHRP Project 10-70* (Roy et al, 2011), and NCHRP Report 718, *Development of Fatigue Loading and Design Methodology for High Mast Lighting Towers* (Connor et al., 2012).

11.2—DEFINITIONS

Constant-Amplitude Fatigue Threshold (or Limit) (CAFT or CAFL)—Nominal stress range below which a particular fatigue detail can withstand an infinite number of repetitions without fatigue failure.

Fatigue—Damage resulting in fracture caused by stress fluctuations.

In-Plane Bending—Bending in-plane for the main member (column). At the connection of an arm or arm's built-up box to a vertical column, the in-plane bending stress range in the column is a result of galloping or truck-induced gust loads on the arm and/or arm's attachments.

Limit State Wind Load Effect—A specifically defined loading criteria.

Load-Bearing Attachment—Attachment to main member where there is a load range in the attachment itself in addition to any primary stress range in the main member.

High-Level Luminaire Support—Truss-type or pole-type tower that provides lighting at heights greater than 55 ft, typically using four to twelve luminaires.

High-Mast Lighting Tower (HMLT)—Another description for a pole-type high-level luminaire support.

Nonload-Bearing Attachment—Attachment to main member where the only significant stress range is the primary stress in the main member.

Out-of-Plane Bending—Bending out-of-plane for the main member (column). At the connection of an arm or arm's built-up box to a vertical column, the out-of-plane bending stress range in the column is a result of natural wind-gust loads on the arm and the arm's attachments.

Pressure Range—Pressure due to a limit state wind load effect that produces a stress range.

Stress Range—The algebraic difference between extreme stresses used in fatigue design.

Typical Light Support—vertical light support less than 55 ft tall.

Yearly Mean Wind Velocity—Long-term average of the wind speeds for a given area.

11.3—NOTATION

A = finite life constant (ksi^3) (11.9.3)

C_{BC} = bolt circle ratio, $\dfrac{D_{BC}}{D_T}$ (11.9.3.1)

C_d = appropriate drag coefficient from Section 3 for given attachment or member (11.7.1.2) (C11.7.1.2) (11.7.1.3) (C11.7.1.3) (11.7.2) (C11.7.2)

C_{OP} = opening ratio, $\dfrac{D_{OP}}{D_T}$ (11.9.3.1) (C11.9.3.1)

D_{BC} = diameter of the fastener (bolt) circle through in the transverse plate (in.) (11.9.3.1) (C11.9.3.1)

D_{OP} = diameter of concentric opening in the transverse plate (in.) (11.9.3.1) (C11.9.3.1)

D_T = external diameter of a round tube or outer flat-to-flat distance of a multi-sided tube at top of transverse plate (in.) (11.9.3.1) (C11.9.3.1)

d = diameter of a circular section (ft) (11.7.1.2) (11.7.1.3)

H = effective weld throat (in.) (11.9.3.1)

h_{ST} = height of longitudinal attachment (stiffener) (in.) (11.9.3.1)

I_F = fatigue importance factors applied to limit state wind load effects to adjust for the desired level of structural reliability (11.6) (11.7) (11.7.1.1) (C11.7.1.1) (11.7.1.2) (C11.7.1.2) (11.7.1.3)

K_F = finite life fatigue stress concentration factor (11.9.3.1) (C11.9.3.1)

K_I = infinite life fatigue stress concentration factor (11.9.3.1) (C11.9.3.1)

L = slip-splice overlap length (in.) (11.9.3.1)

N = number of wind load induced stress cycles expected during the life time of the structure (11.9.3)

$\mathrm{N_B}$ = number of fasteners in the transverse plate (11.9.3.1)

N_S = number of sides (11.9.3.1) (C11.9.3.1)

N_{ST} = number of longitudinal attachments (stiffeners) (11.9.3.1) (C11.9.3.1)

P_{CW} = combined wind pressure range for fatigue design of HMLTs (psf) (11.7.2) (C11.7.2)

P_{FLS} = fatigue limit state wind pressure range (psf) (11.7.2) (C11.7.2) (11.9.3.1)

P_G = galloping-induced vertical shear pressure range (psf) (11.7.1.1)

P_{NW} = natural wind gust pressure range (psf) (11.7.1.2) (C11.7.1.2)

P_{TG} = truck-induced gust pressure range (psf) (11.7.1.3)

r = radius of chord or column (in.) (11.9.3.1)

r_b = inside bend radius of a plate (in.) (11.9.3.1) (C11.9.3.1)

S_R = nominal stress range of the main member or branching member (ksi) (11.9.3.1)

t = thickness (in.) (11.9.3.1)

t_b = wall thickness of branching member (in.) (11.9.3.1)

t_c = wall thickness of main member (column) (in.) (11.9.3.1)

t_p = plate thickness of attachment (in.) (11.9.3.1)

t_{ST} = thickness of longitudinal attachment (stiffener) plate (in.) (11.9.3.1)

t_T = thickness of tube (in.) (11.9.3.1) (C11.9.3.1)

t_{TP} = thickness of transverse plate (in.) (11.9.3.1) (C11.9.3.1)

V_{mean} = yearly mean wind velocity for a given area (mph) (C11.7.1.2)

V_T = truck speed for truck-induced wind gusts (mph) (11.7.1.3) (11.7.1.3)

α = ovalizing parameter for bending in the main member (11.9.3.1)

Δf = wind load induced stress range (ksi) (11.5)

$(\Delta F)_n$ = nominal fatigue resistance (ksi) (11.5.1) (11.9.3) (C11.9.3)

$(\Delta F)_{TH}$ = constant amplitude fatigue threshold (ksi) (11.9.3) (C11.9.3)

ΔF = fatigue resistance stress range (ksi) (11.5) (11.5.1)

$\Delta\sigma$ = indication of stress range in member (11.9.3.1)

γ = load factor (11.5) (11.5.1) (11.9.3)

ϕ = resistance factor (11.5) (11.5.1) (11.9.3)

11.4—APPLICABLE STRUCTURE TYPES

Design for fatigue shall be required for the following type structures:

a. overhead cantilevered sign structures,

b. overhead cantilevered traffic signal structures,

c. high-mast lighting towers (HMLT),

d. overhead noncantilevered sign structures, and

e. overhead noncantilevered traffic signal structures.

C11.4

NCHRP Report 412 and NCHRP Web Only Document 176 are the basis for the fatigue design provisions for cantilevered structures. NCHRP Report 494 is the basis for the fatigue design provisions for non-cantilevered support structures. The fatigue design procedures outlined in this Section may be applicable to steel and aluminum structures in general. However, only specific types of structures are identified for fatigue design in this Article. Common lighting poles and roadside signs are not included because they are smaller structures and normally have not exhibited fatigue problems. An exception would be square lighting poles, as they are much more prone to fatigue than round or multi-sided cross-sections having eight or more sides. Caution should be exercised regarding the use of square lighting poles even when a fatigue design is performed. The provisions of this Section are not applicable for the design of span-wire (strain) poles.

11.5—DESIGN CRITERIA

Cantilevered and noncantilevered support structures shall be designed for fatigue to resist wind-induced stresses. Stress ranges on all components, openings, mechanical fasteners, and weld details shall be limited to satisfy:

$$\gamma(\Delta f) \le \phi(\Delta F) \quad (11.5\text{-}1)$$

where Δf is the wind load induced stress range; ΔF is the fatigue resistance, γ is the load factor per the Fatigue I limit state defined in Table 3.4-1, and ϕ is the resistance factor equal to 1.0.

Fatigue design of the support structures may be conducted using the nominal stress-based classifications of typical connection details as provided in Article 11.9.1 and Table 11.9.3.1-1, or using the alternate local stress-based and/or experiment-based methodologies presented in Appendix C. Support structures shall be proportioned such that the wind-induced stress is below the constant amplitude fatigue threshold (CAFT) providing infinite life.

The remaining fatigue life of existing steel structures may be assessed based on a finite life. The finite life methodology shall only be used to evaluate the fatigue life of existing structures and shall not be used in the design of new structural elements.

C11.5

Fatigue design of connection details in support structures may be as per nominal stress- or local stress-based and/or experiment-based methodologies. The nominal stress-based design approach using classification of typical connection details and their fatigue resistances as provided in Article 11.9.1 and Table 11.9.3.1-1 should suffice in most cases. However, if a connection detail is employed that has not been addressed in Table 11.9.3.1-1, an alternate local stress-based and/or experiment-based methodology as provided in Appendix C may be used for fatigue design. It is important that the stresses are calculated in agreement with the definition of stress used for a particular design methodology.

Accurate load spectra for defining fatigue loadings are generally not available. Assessment of stress fluctuations and the corresponding number of cycles for all wind-induced events (lifetime loading histogram) is difficult. With this uncertainty, the design of sign, high-level luminaire, and traffic signal supports for a finite fatigue life is unreliable. Therefore, an infinite life fatigue design approach is recommended.

The infinite life fatigue design approach should ensure that a structure performs satisfactorily for its design life to an acceptable level of reliability without significant fatigue damage. While some fatigue cracks may initiate at local stress concentrations, there should not be any time dependent propagation of these cracks. This is typically the case for structural supports where the wind-load cycles in 25 years or more are expected to exceed 100 million cycles, whereas typical weld details exhibit Constant Amplitude Fatigue

Threshold (CAFT) at 10 to 20 million cycles. It may be noted that in the predecessor to these specifications, the CAFT was termed as Constant Amplitude Fatigue Limit (CAFL).

An accurate assessment of the lifetime wind induced stress range histogram is required for assessing finite life fatigue performance. Thus, designing new structures for finite fatigue life is impractical. Where an accurate assessment of the lifetime wind induced stress range histogram is available, a finite fatigue life may be considered for estimating remaining life of existing structures at the discretion of the Owner.

The equivalent static wind load effects as specified in Article 11.7 are to be considered for infinite life fatigue design. The wind effects for evaluating finite fatigue life should be obtained from analysis based on historical wind records, or directly from field measurements on the subject or similar structures situated in the same or similar wind environments, as approved by the Owner.

Due to significantly lower fatigue resistance compared to steel, remaining life assessment of aluminum structures is not advised.

11.5.1—Nominal Stress-Based Design

For nominal stress-based design, Equation 11.5.1-1 is rewritten as:

$$\gamma(\Delta f)_n \leq \phi(\Delta F)_n \qquad (11.5.1\text{-}1)$$

where:

$(\Delta f)_n$ = the wind-induced nominal stress range defined in Article 11.9.2,

$(\Delta F)_n$ = the nominal fatigue resistance as specified in Article 11.9.3 for the various detail classes identified in Article 11.9.1,

γ = the load factor per the Fatigue I limit state defined in Table 3.4-1, and

ϕ = the resistance factor equal to 1.0.

C11.5.1

Fatigue-critical details may be designed such that the nominal stress ranges experienced by the details are less than the nominal fatigue resistance of respective detail classes. For fatigue design classification of typical support structure details, the applicable nominal stress ranges and their fatigue resistances are provided in Articles 11.9.1, 11.9.2, and 11.9.3.

11.6—FATIGUE IMPORTANCE FACTORS

A fatigue importance factor, I_F, that accounts for the risk of hazard to traffic and damage to property shall be applied to the limit state wind-load effects specified in Article 11.7. Fatigue importance factors for traffic signal and sign support structures exposed to the three wind load effects are presented in Table 11.6-1. Fatigue importance categories for HMLTs are provided in Table 11.6-2.

C11.6

Fatigue importance factors are introduced into the Specifications to adjust the level of structural reliability of cantilevered and noncantilevered support structures. Fatigue importance factors should be determined by the Owner.

The importance categories and fatigue importance factors (rounded to the nearest 0.05) are results from NCHRP Reports 469 and 494. Two types of support structures are presented in Table 11.6-1. Structures classified as Category I present a high hazard in the event of failure and should be designed to resist rarely-occurring wind loading and vibration phenomena. It is recommended that all structures without effective mitigation devices on roadways with a speed limit in excess of 35 mph and average daily traffic (ADT) exceeding 10,000 or average daily truck traffic (ADTT) exceeding 1,000 should be classified as Category I structures. ADT and ADTT are for one direction regardless of the number of lanes.

NCHRP Report 718 provides fatigue loads and the associated importance factors for HMLTs (Connor et al., 2010).

Structures without mitigation devices may be classified as Category I if any of the following apply:

- Cantilevered sign structures with a span in excess of 50 ft or high-mast towers in excess of 100 ft,
- Large sign structures, both cantilevered and noncantilevered, including changeable message signs, and
- Structures located in an area that is known to have wind conditions that are conducive to vibration.

Structures should be classified as Category III if they are located on roads with speed limits of 35 mph or less. Structures that are located such that a failure will not affect traffic may be classified as Category III. High-mast lighting towers are addressed in Article 11.7.2 and these supports do not carry an importance factor per se.

All structures not explicitly meeting the Category I or Category III criteria should be classified as Category II.

Maintenance and inspection programs should be considered integral to the selection of the fatigue importance category.

There are many factors that affect the selection of the fatigue category and engineering judgment is required.

The fatigue importance categories for HMLTs found in Table 11.6-2 are based on the research conducted as part of NCHRP Report 718 (2012). The fatigue importance categories for HMLTs have been separated and simplified from those in Table 11.6-1. Since HMLTs are generally only used on high ADT roadways, whether a pole can or cannot fall in the path of traffic is selected as the critical parameter.

Table 11.6–1—Fatigue Importance Factors, I_F

Fatigue Category			Fatigue Importance Factor, I_F		
			Galloping	Natural Wind Gusts	Truck-Induced Gusts
Cantilevered	I	Sign Traffic Signal	1.0 1.0	1.0 1.0	1.0 1.0
	II	Sign Traffic Signal	0.70 0.65	0.85 0.80	0.90 0.85
	III	Sign Traffic Signal	0.40 0.30	0.70 0.55	0.80 0.70
Noncantilevered	I	Sign Traffic Signal	– –	1.0 1.0	1.0 1.0
	II	Sign Traffic Signal	– –	0.85 0.80	0.90 0.85
	III	Sign Traffic Signal	– –	0.70 0.55	0.80 0.70

Notes:

a. Structure is not susceptible to this type of loading.

Table 11.6–2—Fatigue Importance Categories for HMLTs

Hazard Level	Importance Category
High (distance to roadway ≤ height of HMLT)	I
Low (distance to roadway > height of HMLT)	II

11.7—FATIGUE DESIGN LOADS

To avoid large-amplitude vibrations and to preclude the development of fatigue cracks in various connection details and at other critical locations, cantilevered and noncantilevered support structures as defined in Article 11.4 shall be designed to resist each of the following applicable limit state equivalent static wind loads acting separately. These loads shall be used to calculate nominal stress ranges near fatigue-sensitive connection details described in Article 11.5 and deflections for service limits described in Article 11.8.

In lieu of using the equivalent static pressures provided in this Specification, a dynamic analysis of the structure may be performed using appropriate dynamic load functions derived from reliable data.

Fatigue loading provisions for HMLTs are differentiated from those associated with other structures. HMLTs shall be designed for the loading given in Article 11.7.2.

C11.7

Cantilevered and noncantilevered support structures are exposed to several wind phenomena that can produce cyclic loads. Vibrations associated with these cyclic forces can become significant. NCHRP Report 412 identified galloping, vortex shedding, natural wind gusts, and truck-induced gusts as wind-loading mechanisms that can induce large-amplitude vibrations and/or fatigue damage in cantilevered traffic signal, sign, and light support structures. NCHRP Report 494 identified natural wind gusts and truck-induced gusts as wind-loading mechanisms that can induce large-amplitude vibrations and/or fatigue damage in noncantilevered traffic signal and sign support structures.

The amplitude of vibration and resulting stress ranges are increased by the low levels of stiffness and damping possessed by many of these structures. In some cases, the vibration is only a serviceability problem because motorists cannot clearly see the mast arm attachments or are concerned about passing under the structures. In other cases, where deflections may or may not be considered excessive, the magnitudes of stress ranges induced have resulted in the development of fatigue cracks at various connection details including the anchor bolts.

The provisions for fatigue loading of HMLTs are based on the research conducted as part of NCHRP Report 718 which developed a loading spectrum inclusive of all applicable load effects due to natural wind.

The wind-loading phenomena specified in this section possess the greatest potential for creating large-amplitude vibrations in cantilevered support structures. In particular, galloping and vortex shedding are aeroelastic instabilities that typically induce vibrations at the natural frequency of the structure (i.e., resonance). These conditions can lead to fatigue failures in a relatively short period of time.

Design pressures for four fatigue wind-loading mechanisms are presented as an equivalent static wind pressure range, or a shear stress range in the case of galloping. These pressure (or shear stress) ranges should be applied as prescribed by static analysis to determine stress ranges near fatigue-sensitive details. In lieu of designing for galloping or vortex-shedding limit state fatigue wind load effects, mitigation devices may be used as approved by the Owner. Mitigation devices are discussed in NCHRP Reports 412, 469, and 718.

11.7.1—Sign and Traffic Signal Structures

Equivalent static wind loads for the fatigue design of sign and traffic signals structures shall be determined from Articles 11.7.1.1 through 11.7.1.3 as applicable. The structures included in this section are defined in Article 11.4 and the associated commentary.

11.7.1.1—Galloping

Overhead cantilevered sign and traffic signal support structures shall be designed for galloping-induced cyclic loads by applying an equivalent static shear pressure vertically to the surface area, as viewed in normal elevation, of all sign panels and/or traffic signal heads and back plates rigidly mounted to the cantilevered horizontal support. The vertical shear pressure range shall be equal to the following:

$$P_G = 21 I_F \text{ (psf)} \qquad (11.7.1.1\text{-}1)$$

where

I_F = fatigue importance factor

21 = pressure (psf)

In lieu of designing to resist periodic galloping forces, cantilevered sign and traffic signal structures may be erected with effective vibration mitigation devices. Vibration mitigation devices should be approved by the Owner, and approval should be based on historical or research verification of device vibration damping characteristics.

Alternatively, for traffic signal structures, the Owner may choose to install approved vibration mitigation devices if structures exhibit a galloping problem. The mitigation devices should be installed as quickly as possible after the galloping problem appears.

The Owner may choose to exclude galloping loads for the fatigue design of overhead cantilevered sign support structures with quadri-chord (i.e., four-chord) horizontal trusses.

C11.7.1.1

Galloping, or Den Hartog instability, results in large-amplitude, resonant oscillations in a plane normal to the direction of wind flow. It is usually limited to structures with nonsymmetrical cross-sections, such as sign and traffic signal structures with attachments to the horizontal cantilevered arm. Structures without attachments to the cantilevered horizontal arm support are not susceptible to galloping-induced wind load effects.

The results of wind tunnel (Kaczinski et al., 1998) and water tank (McDonald et al., 1995) testing, as well as the oscillations observed on cantilevered support structures in the field, are consistent with the characteristics of the galloping phenomena. These characteristics include the sudden onset of large-amplitude, across-wind vibrations that increase with increases in wind velocity. Galloping is typically not caused by wind applied to the support structure, but rather applied to the attachments to the horizontal cantilevered arm, such as signs and traffic signals.

The geometry and orientation of these attachments, as well as the wind direction, directly influence the susceptibility of cantilevered support structures to galloping. Traffic signals are more susceptible to galloping when configured with a backplate. In particular, traffic signal attachments configured with or without a backplate are more susceptible to galloping when subject to flow from the rear. Galloping of sign attachments is independent of aspect ratio and is more prevalent with wind flows from the front of the structure.

By conducting wind tunnel tests and analytical calibrations to field data and wind tunnel test results, an equivalent static vertical shear of 21 psf was determined for the galloping phenomenon. This vertical shear range should be applied to the entire frontal area of each of the sign and traffic signal attachments in a static analysis to determine stress ranges at critical connection details. For example, if a 8 × 10 ft sign panel is mounted to a horizontal mast arm, a static force of 1.680 × I_F, kips should be applied vertically at the area centroid of the sign panel. A study (Florea et al., 2007) has shown that the equivalent static force that an attachment experiences depends on the location along the arm where it is attached. Equivalent static pressures or vertical shear ranges applied to the frontal area of each sign or traffic signal attachment are greater towards the tip of the mast arm. The specification does not consider the effect of the attachment location when calculating the galloping force. Further testing is necessary to verify this and to suggest location-specific ranges.

A pole with multiple horizontal cantilevered arms may be designed for galloping loads applied separately to each individual arm, and need not consider galloping simultaneously occurring on multiple arms.

Overhead cantilevered sign support structures with quadri-chord horizontal trusses do not appear to be susceptible to galloping because of their inherent stiffness.

Two possible means exist to mitigate galloping-induced oscillations in cantilevered support structures. The dynamic properties of the structure or the aerodynamic properties of the attachments can be adequately altered to mitigate galloping or the installation of a device providing positive aerodynamic damping can be used to alter the structure's response from the aerodynamic effects on the attachments.

A method of providing positive aerodynamic damping to a traffic signal structure involves installing a sign blank mounted horizontally and directly above the traffic signal attachment closest to the tip of the mast arm. This method has been shown to be effective in mitigating galloping-induced vibrations on traffic signal support structures with horizontally mounted traffic signal attachments (McDonald et al., 1995). For vertically mounted traffic signal attachments, a sign blank mounted horizontally near the tip of the mast arm has mitigated large-amplitude galloping vibrations occurring in traffic signal support structures. This sign blank is placed adjacent to a traffic signal attachment, and a separation exists between the sign blank and the top of the mast arm. In both cases, the sign blanks are required to provide a sufficient surface area for mitigation to occur. However, the installation of sign blanks may influence the design of structures for truck-induced wind gusts by increasing the projected area on a horizontal plane. NCHRP Reports 412 and 469 provide additional discussion on this possible mitigation device and on galloping susceptibility and mitigation.

11.7.1.2—Natural Wind Gust

Cantilevered and noncantilevered overhead sign and overhead traffic signal supports shall be designed to resist an equivalent static natural wind gust pressure range of:

$$P_{NW} = 5.2 C_d I_F \text{ (psf)} \quad (11.7.1.2\text{-}1)$$

where:

I_F = fatigue importance factor

5.2 = pressure (psf)

C_d = the appropriate drag coefficient based on the yearly mean wind velocity of 11.2 mph specified in Section 3, "Loads," for the considered element to which the pressure range is to be applied.

If Eq. C11.7.1.2-1 is used in place of Eq. 11.7.1.2-1, C_d may be based on the location-specific yearly mean wind velocity V_{mean}. The natural wind gust pressure range shall be applied in the horizontal direction to the exposed area of all support structure members, signs, traffic signals, and/or miscellaneous attachments. Designs for natural wind gusts shall consider the application of wind gusts for any direction of wind.

The design natural wind gust pressure range is based on a yearly mean wind speed of 11.2 mph. For locations with more detailed wind records, particularly sites with higher wind speeds, the natural wind gust pressure may be modified at the discretion of the Owner.

C11.7.1.2

Because of the inherent variability in the velocity and direction, natural wind gusts are the most basic wind phenomena that may induce vibrations in wind-loaded structures. The equivalent static natural wind gust pressure range specified for design was developed with data obtained from an analytical study of the response of cantilevered support structures subject to random gust loads (Kaczinski et al., 1998).

Because V_{mean} is relatively low, the largest values of C_d for the support may be used.

This parametric study was based on the 0.01 percent exceedance for a yearly mean wind velocity of 11.2 mph, which is a reasonable upper bound of yearly mean wind velocities for most locations in the country. There are locations, however, where the yearly mean wind velocity is larger than 11.2 mph. For installation sites with more detailed information regarding yearly mean wind speeds (particularly sites with higher wind speeds), the following equivalent static natural wind gust pressure range shall be used for design:

$$P_{NW} = 5.2 C_d \left(\frac{V_{\text{mean}}}{11.2\text{mph}} \right)^2 I_F \text{ (psf)} \quad (C11.7.1.2\text{-}1)$$

The largest natural wind gust loading for an arm or pole with a single arm is from a wind gust direction perpendicular to the arm. For a pole with multiple arms, such as two perpendicular arms, the critical direction for the natural wind gust is usually not normal to either arm. The design natural wind gust pressure range should be applied to the exposed surface areas seen in an elevation view orientated perpendicular to the assumed wind gust direction.

11.7.1.3—Truck-Induced Gust

Cantilevered and noncantilevered overhead sign support structures shall be designed to resist an equivalent static truck gust pressure range of

$$P_{TG} = 18.8 C_d I_F \text{ (psf)} \quad (11.7.1.3\text{-}1)$$

where

I_F = fatigue importance factor

18.8 = pressure (psf)

C_d = the drag coefficient based on the truck speed of 65 mph from Section 3 for the considered element to which the pressure range is to be applied.

If Eq. C11.7.1.3-1 is used in place of Eq. 11.7.1.3-1, C_d should be based on the considered truck speed V_T. The pressure range shall be applied in the vertical direction to the horizontal support as well as the area of all signs, attachments, walkways, and/or lighting fixtures projected on a horizontal plane. This pressure range shall be applied along any 12-ft length to create the maximum stress range,

C11.7.1.3

The passage of trucks beneath support structures may induce gust loads on the attachments mounted to the horizontal support of these structures. Although loads are applied in both horizontal and vertical directions, horizontal support vibrations caused by forces in the vertical direction are most critical. Therefore, truck gust pressures are applied only to the exposed horizontal surface of the attachment and horizontal support.

A pole with multiple horizontal cantilever arms may be designed for truck gust loads applied separately to each individual arm and need not consider truck gust loads applied simultaneously to multiple arms.

Recent vibration problems on sign structures with large projected areas in the horizontal plane, such as variable message sign (VMS) enclosures, have focused attention on vertical gust pressures created by the passage of trucks beneath the sign.

The design pressure calculated from Eq. 11.7.1.3-1 is based on a truck speed of 65 mph. For structures installed at locations where the posted speed limit is much less than 65 excluding any portion of the structure not located directly above a traffic lane. The equivalent static truck pressure range may be reduced for locations where vehicle speeds are less than 65 mph.

The magnitude of applied pressure range may be varied depending on the height of the horizontal support and the attachments above the traffic lane. Full pressure shall be applied for heights up to and including 20 ft, and then the pressure may be linearly reduced for heights above 20 ft to a value of zero at 33 ft.

The truck-induced gust loading shall be excluded unless required by the Owner for the fatigue design of overhead traffic signal support structures.

mph, the design pressure may be recalculated based on this lower truck speed. The following equation may be used:

$$P_{TG} = 18.8 C_d \left(\frac{V_T}{65\text{mph}} \right)^2 I_F \text{ (psf)} \quad \text{(C11.7.1.3-1)}$$

where

V_T = truck speed (mph).

The given truck-induced gust loading should be excluded unless required by the Owner for the fatigue design of overhead traffic signal structures. Many traffic signal structures are installed on roadways with negligible truck traffic. In addition, the typical response of traffic signal structures from truck-induced gusts is significantly overestimated by the design pressures prescribed in this article. This has been confirmed in a study (Albert et al., 2007) involving full-scale field tests where strains were monitored on cantilevered traffic signal structures. Over 400 truck events were recorded covering a variety of truck types and vehicle speeds; only 18 trucks produced even a detectable effect on the cantilevered traffic signal structures and the strains were very small relative to those associated with the design pressures in this Article.

11.7.2—High-Mast Lighting Towers Fatigue

High-mast lighting towers shall be designed for fatigue to resist the combined wind effect using the equivalent static pressure range of

$$P_{CW} = P_{FLS} C_d \quad \text{(11.7.2-1)}$$

where

P_{FLS} = the fatigue-limit-state static pressure range presented in Table 11.7.2-1.

HMLTs are defined as being 55 ft or taller structures. Luminaires less than 55 ft tall do not need to be designed for fatigue.

For the structural element considered, C_d is the appropriate drag coefficient specified in Section 3 and shall be based on the yearly mean wind velocity, V_{mean}. The combined wind effect pressure range shall be applied in the horizontal direction to the exposed area of all high-mast lighting tower components. Designs for combined wind shall consider the application of wind from any direction.

The yearly mean wind velocity used in determining P_{FLS} shall be as given in Figure 11.7.2-1. For all islands adjacent to the Alaska mainland and west coast Alaska mainland, use Ranges G and H (>11 mph). For Alaska inlands, use Ranges E and F (9–11 mph). For all Hawaii islands use Range Ranges E and F (9-11 mph).

Designers are cautioned of the effects of topography when considering location-specific mean wind velocity in their design. These effects can cause considerable variation

C11.7.2

NCHRP Report 718 is the basis for fatigue loads identified in this section. Prior to 2012, these Specifications made no distinction between high-mast lighting towers and other signal or sign support structures. Failures resulting from wind-induced fatigue led to field testing, laboratory wind tunnel testing, and analytical studies to determine appropriate load models for the fatigue design of HMLTs.

The combined wind load specified for HMLTs was derived from the effects of the entire wind-load spectrum and therefore includes all ranges of wind speed. It is recocognized that the drag coefficient varies with wind speed.

The value of P_{FLS} is intended to produce the same fatigue damage generated by the variable amplitude spectrum using a single equivalent constant amplitude load (P_{CW}). P_{FLS} was derived using constant values of C_d (using Section 3) and the values of P_{CW} measured at each pole (NCHRP 718) to simplify the approach. Hence use of values other than those in Section 3 will result in erroneous estimates of P_{CW}.

The in-service performance of HMLTs shorter than 55 ft appears to suggest that fatigue is not a critical limit state. Cracking has been primarily observed in HMLTs greater than 100 ft tall. The limit of 55 ft was selected somewhat arbitrarily to be well below the 100 ft height. However, although these specifications do not require HMLTs shorter than 55 ft to be designed for fatigue, fatigue resistance details should be selected and careful installation practices followed.

If the Engineer suspects that the HMLT will be subjected to high yearly mean wind speeds, the HMLT is placed in a location where local wind effects may be great (e.g., on a bluff), or previous performance of similar HMLTs in wind speed. For locations with more detailed wind records, the yearly mean wind velocity may be modified at the discretion of the Owner.

has been poor, consideration should be given to designing the structure for fatigue using the provisions contained herein.

For normal installations, the height shall be defined as the distance from the bottom of the base plate to the tip of the pole, not including the distance the lighting fixture may extend beyond the top.

The fatigue-limit-state static pressure range values listed in Table 11.7.2-1 account for fatigue importance factors and variation in mean wind speed. The combined wind pressure range includes the cumulative fatigue damage effects of vortex shedding.

Figure 11.7.2-1 serves as a broad guide for determining regional mean wind speed. Local conditions are known to vary and may not necessarily be represented by the map. NCHRP Reports 412 and 718 found the design method to be conservative in most cases; however, designers are encouraged to check local wind records and/or consider topographical effects in choosing a yearly mean wind speed for design if the local wind conditions are suspected to be more severe than suggested by Figure 11.7.2-1. It is not recommended to use design pressure ranges less than suggested by Figure 11.7.2-1.

Table 11.7.2-1—Fatigue-Limit-State Pressure Range for HMLT Design, P_{FLS}

Fatigue Design Case	Importance Category I	Importance Category II
$V_{mean} \leq 9$ mph	6.5 psf	5.8 psf
9 mph $< V_{mean} \leq 11$ mph	6.5 psf	6.5 psf
$V_{mean} > 11$ mph	7.2 psf	7.2 psf

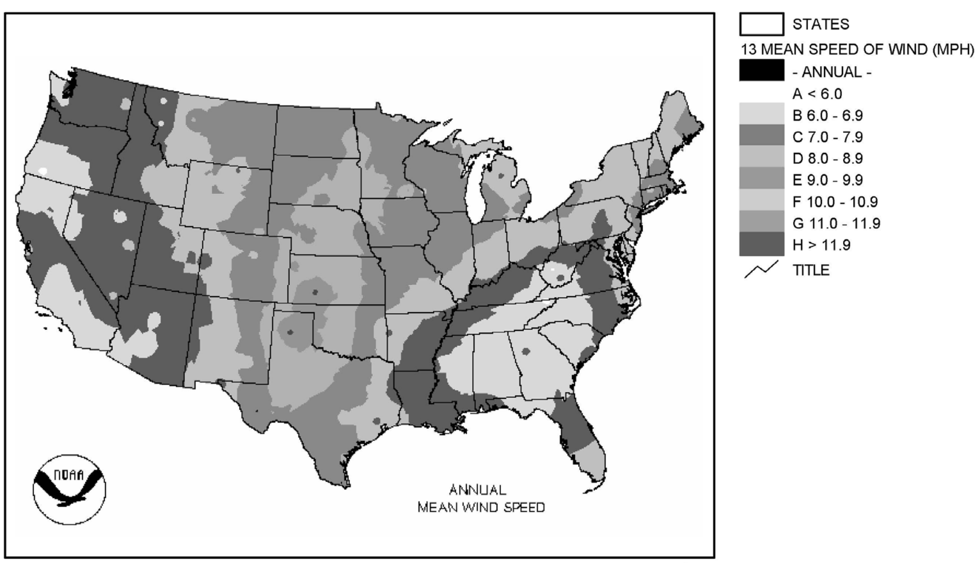


Figure 11.7.2-1 Yearly Mean Wind Speed, mph

No separate load is specified to account for vortex shedding since it is incorporated in the equivalent static combined wind pressure range, P_{CW} used for fatigue design in Article 11.7.2.

High-mast lighting towers are highly susceptible to vibrations induced by vortex shedding, leading to the rapid accumulation of potentially damaging stress cycles that lead to fatigue failure. NCHRP Report 718 studied the response

Where serviceability and maintenance requirements due to vortex shedding induced vibrations are an issue, devices such as strakes, shrouds, mechanical dampers, etc. may be used to mitigate the effect.

of these structures in the field and determined that the previous edition did not properly quantify vortex shedding. Rather than separate the effect of vortex shedding from all other wind phenomena, a loading spectrum was developed to encompass all possible wind load effects. The fatigue-limit-state static wind pressures listed in Table 11.7.2-1 represent this combined wind load effect.

Maintenance and serviceability issues resulting from vortex shedding may have a detrimental effect on the performance of HMLTs. Issues with anchor bolts loosening and rattling of the luminaire have been known to occur. Where fatigue-prone details exist which may shorten the life of HMLTs due to a lower fatigue resistance than initially considered, or in cases where the service life of an HMLT initially designed for a finite lifetime may wish to be extended, mitigation devices have proved reliable in reducing the number of damaging stress cycles. Information pertaining to the performance and sizing of strakes and shrouds on HMLTs is presented in NCHRP Report 718 and FHWA-WY-10/02F Report *Reduction of Wind-Induced Vibrations in High-mast Light Poles* (Ahearn and Puckett, 2010). Durability of the mitigation technique and the impacts on luminary lowering mechanisms should be considered.

11.8—DEFLECTION

Galloping and truck gust-induced vertical deflections of cantilevered single-arm sign supports and traffic signal arms and non-cantilevered supports should not be excessive. Excessive deflections can prevent motorists from clearly seeing the attachments, and may cause concern about passing under the structures.

C11.8

Because of the low levels of stiffness and damping inherent in cantilevered single mast arm sign and traffic signal support structures, even structures that are adequately designed to resist fatigue damage may experience excessive vertical deflections at the free end of the horizontal mast arm. The primary objective of this provision is to minimize the number of motorist complaints.

NCHRP Report 412 recommends that the total deflection at the free end of single-arm sign supports and all traffic signal arms be limited to 8 in. vertically, when the equivalent static design wind effect from galloping and truck-induced gusts are applied to the structure. NCHRP Report 494 recommends applying the 8 in. vertical limit to noncantilevered support structures. Double-member or truss-type cantilevered horizontal sign supports were not required to have vertical deflections checked because of their inherent stiffness. There are no provisions for a displacement limitation in the horizontal direction.

11.9—FATIGUE RESISTANCE

11.9.1—Detail Classification

All fatigue sensitive details in the connections and components in support structures shall be designed in accordance with their respective detail classifications. Detail classifications for typical components, mechanical fasteners, and welded details in support structures are tabulated in Table 11.9.3.1-1.

All connections shall be detailed as required in Article 5.6.

C11.9.1

Classification of components, mechanical fasteners, and welded details in typical support structures that are susceptible to fatigue cracking is provided in Table 11.9.3.1-1. The detail classes are consistent with the detail categories in the fatigue design provisions of the AASHTO LRFD Bridge Design Specifications (LRFD Design).

The details shown in Table 11.9.3.1-1 are developed based on a review of state departments of transportation standard drawings and manufacturers' literature, and are grouped into six sections based on application. The list is not a complete set of all possible connection details; rather it is intended to include the most commonly used connection details in support structures. Any detail that is not listed in Table 11.9.3.1-1 may be classified based on alternate methodologies provided in Appendix C.

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

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

11.9.2—Stress Range

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

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

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

C11.9.2

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

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

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

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

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

11.9.3—Fatigue Resistance

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

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

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

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

where

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

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

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

Fatigue resistance of typical fatigue-sensitive connection details in support structures for finite and infinite life designs shall be determined from Table 11.9.3.1-1. The fatigue stress concentration factors as functions of connection geometry in tubular structures shall be determined as given in Article 11.9.3.1. The potential location of cracking in each detail is identified in the table. "Longitudinal" implies that the

C11.9.3

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

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

The constant A and the constant amplitude fatigue threshold $(\Delta F)_{TH}$ for the detail classes specified in Table 11.9.3.1-1 are consistent with steel detail categories in LRFD Design. Figure C11.9.3.1-1 is a graphical representation of the nominal fatigue resistance for detail categories as per LRFD Design.

direction of applied stress is parallel to the longitudinal axis of the detail, and "transverse" implies that the direction of applied stress is perpendicular to the longitudinal axis of the detail.

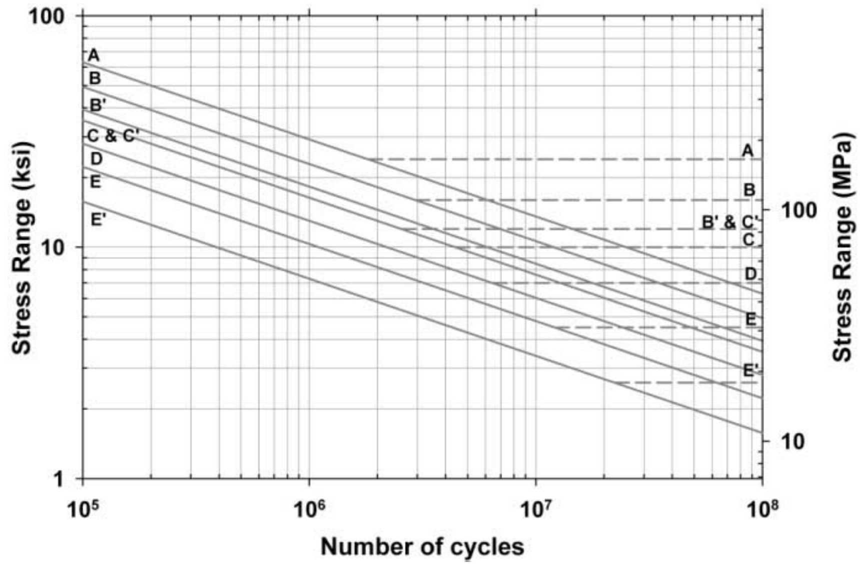


Figure C11.9.3-1—Stress Range vs. Number of Cycles

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

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

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

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

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

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

11.9.3.1—Stress Concentration Factors

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

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

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

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

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

D_{BC} = diameter of circle through the fasteners in the transverse plate (for connections with two or more fastener circles, use the outer most circle diameter) (in.)

D_{OP} = diameter of concentric opening in the transverse plate (in.)

D_T = external diameter of a round tube or outer flat-to-flat distance of a multisided tube at top of transverse plate (in.)

h_{ST} = height of longitudinal attachment (stiffener) (in.)

N_B = number of fasteners in the transverse plate

C11.9.3.1

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

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

N_S = number of sides
N_{ST} = number of longitudinal attachment (stiffener)
t_{ST} = thickness of longitudinal attachment (stiffener) plate (in.)
t_T = thickness of tube (in.)
t_{TP} = thickness of transverse plate (in.)

$$C_{BC} = \frac{D_{BC}}{D_T}$$

$$C_{OP} = \frac{D_{OP}}{D_T}$$

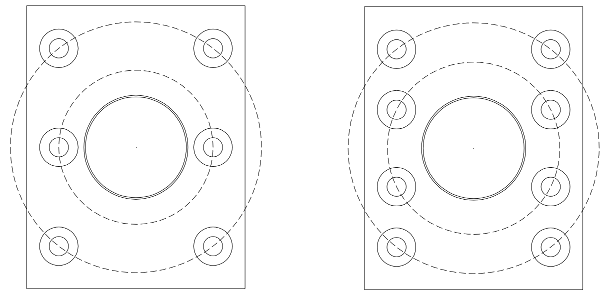


Figure C11.9.3.1–1—Bolt Circle Example

Equations for fatigue stress concentration factors were determined based on parametric finite element analyses and were verified by test results. The ranges of the parameters describing the connection geometry in the studies covered the ranges determined from state departments of transportation's drawings and manufacturer's literature. Fatigue resistance was determined based on the local stress-based methodology presented in Appendix C. Based on these results, the fatigue resistance of the tube-to-transverse plate connection details were classified in terms of separate fatigue stress concentration factors K_F and K_I for finite and infinite life regimes respectively. While the fatigue stress concentration factor for finite life design incorporates the effect of connection geometry, the fatigue stress concentration factor for infinite life design also includes the geometric effect of the weld toe notch.

Experimental and analytical studies demonstrated that the fatigue resistance of tube-to-transverse plate connections is a function of the relative flexibility of the tube and the transverse plate. Reducing the relative flexibility of the transverse plate can significantly increase the fatigue resistance of the connection. The relative flexibility of the transverse plate depends on:

1. the thickness of the transverse plate;
2. the opening in the transverse plate (in groove-welded connections);
3. the number of fasteners;
4. the bolt circle ratio, defined as the ratio of the bolt circle diameter to the tube diameter.

In addition, the diameter and thickness of the tube affects the relative stiffness. Reducing the opening size and/or increasing the plate thickness are the most cost-effective means of reducing the flexibility of the transverse plate and increasing the connection fatigue resistance.

Fatigue performance of a stiffened tube-to-transverse plate, fillet-welded connection is a function of:

1. the thickness of the transverse plate;
2. the thickness of the tube;
3. the stiffener shape and size (thickness, height, and angle); and
4. the number of stiffeners (or stiffener spacing).

Optimized, stiffened, tube-to-transverse plate, fillet-welded connections can provide a cost-effective solution in support structures employing larger diameter and thicker tubes.

For stiffened, fillet-welded, tube-to-transverse plate connections, the finite life fatigue stress concentration factor at the fillet-weld toe on the tube (Table 11.9.3.1-2) is obtained by modifying the finite life stress concentration factor for the fillet-welded tube-to-transverse plate connection detail.

Compared to a round tube of similar size, welded tube-to-transverse plate connections in multisided sections exhibit less fatigue resistance with decreasing roundness. The deviation in fatigue performance of multisided sections from round shapes depends on:

1. the outer flat-to-flat dimension of a multisided tube;
2. the thickness of the tube;
3. the number of sides in the multisided section; and
4. the internal bend radius.

The fatigue stress concentration factors for tube-to-transverse plate connections in multisided cross sections should be obtained by multiplying Eq. 11.9.3.1-1 by the fatigue stress concentration factors of the respective details for round sections, except for the stiffener termination on the tube of a stiffened fillet-welded tube-to-transverse-plate connection. Parametric studies show that the fatigue stress concentration factor for finite life at the stiffener termination on the tube remains same for round and multisided sections irrespective of the number of sides and bend radius.

Table 11.9.3.1-1—Fatigue Details of Cantilevered and Noncantilevered Support Structures

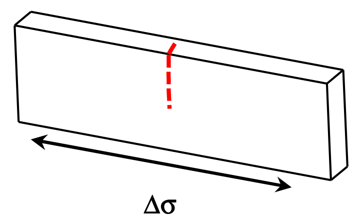
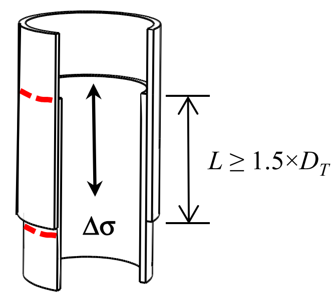
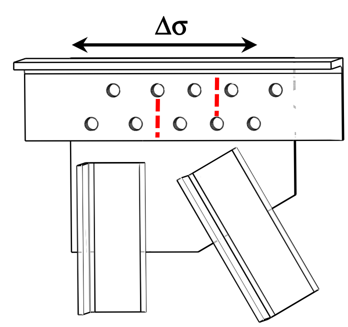
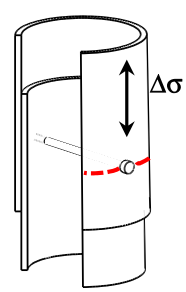
Description	Finite Life Constant, $A \times 10^8$ ksi^3	Threshold[g], $(\Delta F)_{TH}$ ksi	Potential Crack Location	Example
SECTION 1—PLAIN MATERIAL				
1.1 With rolled or cleaned surfaces. Flame-cut edges with *ANSI/AASHTO/AWS D1.5* (Article 3.2.2) smoothness of 1000 µ-in. or less.	250.0	24.0	Away from all welds or structural connections.	
1.2 Slip-joint splice where L is greater than or equal to 1.5 diameters.	120.0	16.0	In a section at the edge of tube splice.	High-level lighting poles. 
SECTION 2—MECHANICALLY FASTENED CONNECTIONS				
2.1 Net section of fully tightened, high-strength (ASTM A325, A490) bolted connections.	120.0	16.0	In the net section originating at the side of the hole.	Bolted joints. 
2.2 Net section of other mechanically fastened connections.	22.0	7.0	In the net section originating at the side of the hole.	

Table 11.9.3.1-1—Fatigue Details of Cantilevered and Noncantilevered Support Structures (continued)

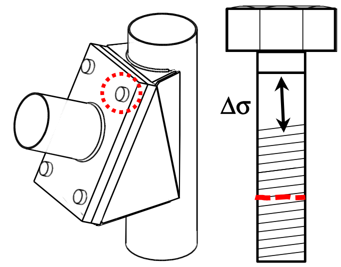
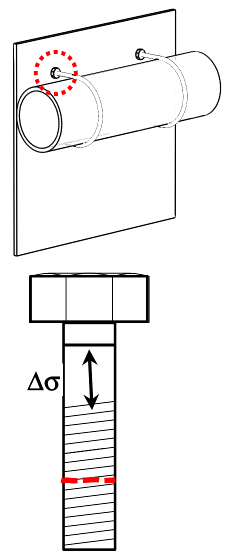
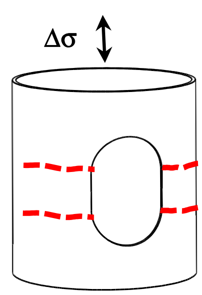
2.3 Anchor bolts or other fasteners in tension; stress range based on the tensile stress area. Misalignments of less than 1:40 with firm contact existing between anchor bolt nuts, washers, and base plate.	22.0	7.0	At the root of the threads extending into the tensile stress area.	Anchor bolts. Bolted mast-arm-to-column connections. 
2.4 Connection of members or attachment of miscellaneous signs, traffic signals, etc. with clamps or U-bolts.	22.0	7.0	At the root of the threads extending into the tensile stress area.	
SECTION 3—HOLES AND CUTOUTS				
3.1 Net section of unreinforced holes and cutouts.	250.0	24.0 (See note e)	In tube wall at edge of unreinforced handhole.	Wire outlet holes. Drainage holes. Unreinforced handholes. 

Table 11.9.3.1-1—Fatigue Details of Cantilevered and Noncantilevered Support Structures (continued)

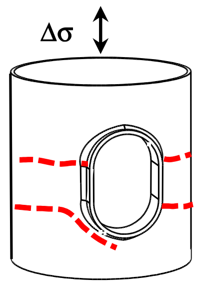
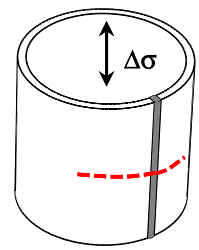
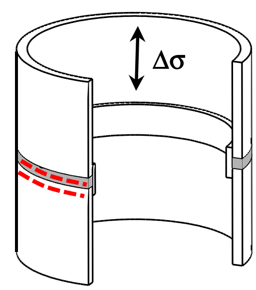
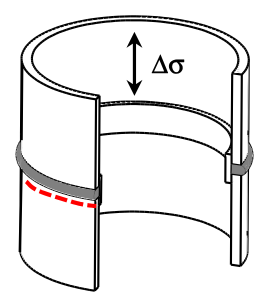
3.2 Reinforced holes and cutouts. At root of reinforcement-to-tube weld	 120.0	 16.0	In tube wall and hole reinforcement from root of reinforcement-to-tube weld.	Reinforced handholes. 
At toe of reinforcement-to-tube weld	22.0	7.0 (See note e)	In tube wall and hole reinforcement from the toe of reinforcement-to-tube weld.	
SECTION 4—GROOVE-WELDED CONNECTIONS				
4.1 Tubes with continuous full- or partial penetration groove-welds parallel to the direction of the applied stress.	61.0	12.0	In the weld away from the weld termination.	Longitudinal seam welds. 
4.2 Full-penetration groove-welded splices with welds ground to provide a smooth transition between members (with or without backing ring removed).	22.0	7.0	In weld through the throat or along the fusion boundary.	Column or mast-arm butt-splices. 
4.3 Full-penetration groove-welded splices with weld reinforcement not removed (with or without backing ring removed).	11.0	4.5	In tube wall along weld toe.	Column or mast-arm butt-splices. 

Table 11.9.3.1-1—Fatigue Details of Cantilevered and Noncantilevered Support Structures (continued)

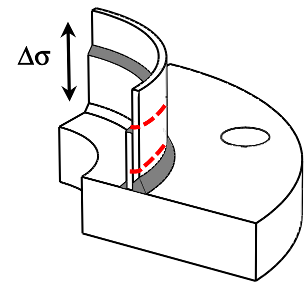
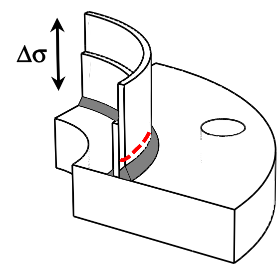
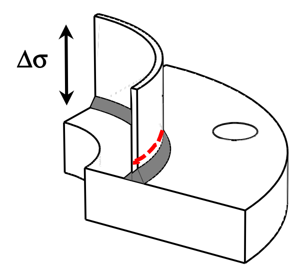
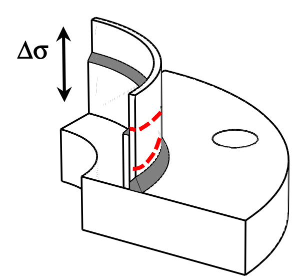
4.4 Full-penetration groove-welded tube-to-transverse plate connections with backing ring attached to the plate with a full penetration weld, or with a continuous fillet-weld around interior face of backing ring, and the backing ring welded to the tube with a continuous fillet-weld at top face of backing ring.	$K_F \le 1.6$: 11.0 $1.6 < K_F \le 2.3$: 3.9	$K_I \le 3.0$: 10.0 $3.0 < K_I \le 4.0$: 7.0 $4.0 < K_I \le 6.5$: 4.5	In tube wall along groove-weld toe or backing ring top weld toe.	Column-to-base plate connections. Mast-arm-to-flange-plate connections. 
4.5 Full-penetration groove-welded tube-to-transverse plate connections with backing ring attached to the plate with a full penetration weld, or with a continuous fillet-weld around interior face of backing ring, and the backing ring not welded to the tube.	$KF \le 1.6$: 11.0 $1.6 < KF \le 2.3$: 3.9	$KI \le 3.0$: 10.0 $3.0 < KI \le 4.0$: 7.0 $4.0 < KI \le 6.5$: 4.5	In tube wall along groove-weld toe.	Column-to-base-plate connections. Mast-arm-to-flange-plate connections. 
4.6 Full penetration groove-welded tube-to-transverse plate connections welded from both sides with back-gouging (without backing ring).	$KF \le 1.6$: 11.0 $1.6 < KF \le 2.3$: 3.9	$KI \le 3.0$: 10.0 $3.0 < KI \le 4.0$: 7.0 $4.0 < KI \le 6.5$: 4.5	In tube wall along groove-weld toe.	Column-to-base-plate connections. Mast-arm-to-flange-plate connections. 
4.7 Full-penetration groove-welded tube-to-transverse plate connections with the backing ring not attached to the plate, and the backing ring welded to the tube with a continuous fillet-weld at top face of backing ring.	$KF \le 1.6$: 11.0 $1.6 < KF \le 2.3$: 3.9	$KI \le 3.0$: 10.0 $3.2 < KI \le 4.0$: 7.0 $4.0 < KI \le 6.5$: 4.5	In tube wall along groove-weld toe or backing ring top weld toe.	Column-to-base-plate connections. Mast-arm-to-flange-plate connections. 

Table 11.9.3.1-1—Fatigue Details of Cantilevered and Noncantilevered Support Structures (continued)

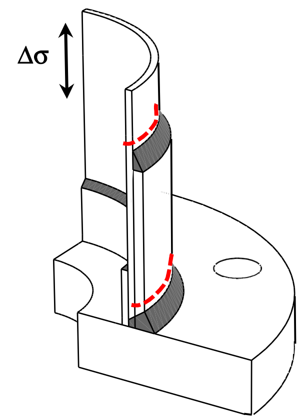
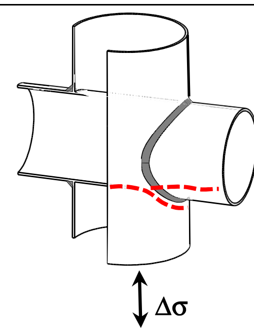
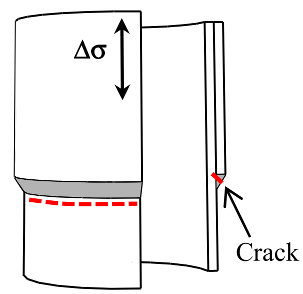
4.8 Full-penetration groove-welded tube-to-transverse-plate connections with the backing ring not attached to the plate, and the backing ring welded to the tube with a continuous fillet weld at top face of backing ring. The connection externally reinforced with a $^{3}/_{8}$ in thick sleeve extending at least half of the tube diameter above the transverse plate and welded to the tube at top and bottom. The transverse plate minimum 3 in thick, having an opening not exceeding 12 in or half of the tube diameter, whichever is less, and fastened by 12 anchor bolts distributed uniformly around the transverse plate	44	10	In tube wall along groove-weld toe, or along sleeve top-weld toe, or backing ring top-weld toe.	Column-to-base-plate connections. 
4.9 Partial penetration groove-welded mast-arm-to-column pass-through connections.	11.0	4.5	In column wall at the mast-arm-to-column weld toe, or in column and mast-arm walls from the mast-arm-to-column weld root.	
SECTION 5—FILLET-WELDED CONNECTIONS				
5.1 Fillet-welded lap splices.	11.0	4.5	In tube wall along weld toe or through weld throat.	Column or mast-arm lap splices. 

Table 11.9.3.1-1—Fatigue Details of Cantilevered and Noncantilevered Support Structures (continued)

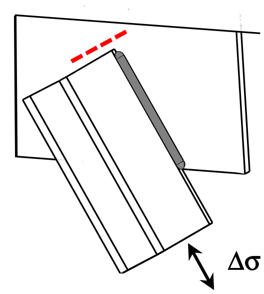
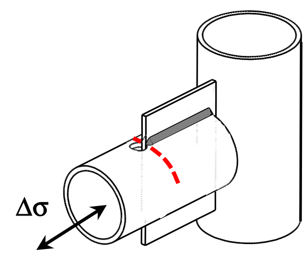
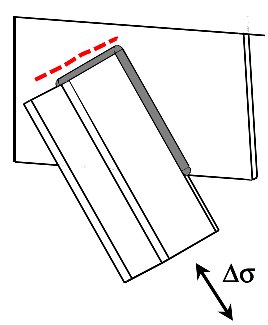
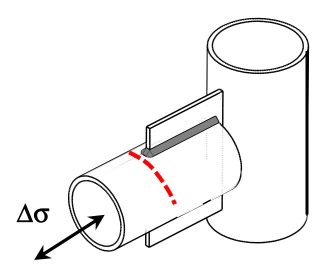
5.2 Members with axial and bending loads with fillet-welded end connections without notches perpendicular to the applied stress. Welds distributed around the axis of the member so as to balance welds stresses.	11.0	4.5	In base metal at the toe of the longitudinal weld termination.	Angle-to-gusset connections with welds terminated short of plate edge.  Slotted tube-to-gusset connections with coped holes (see note d). 
5.3 Members with axial and bending loads with fillet-welded end connections with notches perpendicular to the applied stress. Welds distributed around the axis of the member so as to balance weld stresses.	3.9	2.6	In base metal at the toe of transverse weld.	Angle-to-gusset connections.  Slotted tube-to-gusset connections without coped holes. 

Table 11.9.3.1-1—Fatigue Details of Cantilevered and Noncantilevered Support Structures (continued)

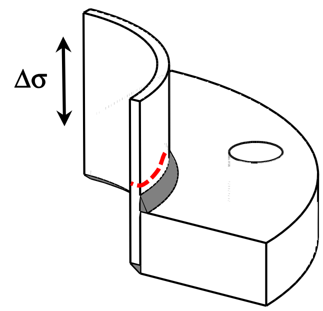
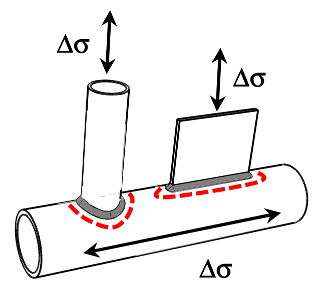
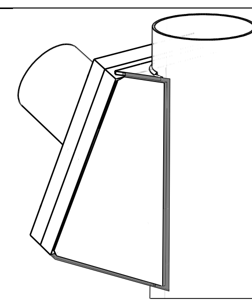
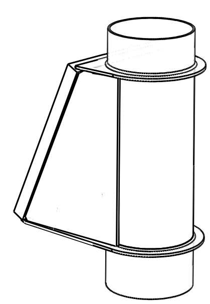
5.4 Fillet-welded tube-to-transverse plate connections	$K_F \leq 3.2$: 3.9	$K_I \leq 4.0$: 7.0 $4.0 < K_I \leq 6.5$: 4.5 $6.5 < K_I \leq 7.7$: 2.6	In tube wall along fillet-weld toe.	Column-to-base-plate or mast-arm-to-flange-plate socket connections. 
5.5 Fillet-welded T-, Y-, and K-tube-to-tube, angle-to-tube, or plate-to-tube connections.		(See notes a and b)	In tube wall along fillet-weld toe.	Chord-to-vertical or chord-to-diagonal truss connections (see note a). Mast-arm directly welded to column (see note b). 
5.6 Fillet-welded gusseted box connections.		(See note f)		
5.7 Fillet-welded ring-stiffened box connections.		(See note f)		

Table 11.9.3.1-1—Fatigue Details of Cantilevered and Noncantilevered Support Structures (continued)

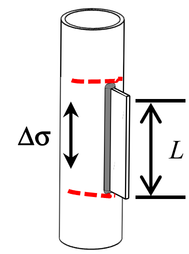
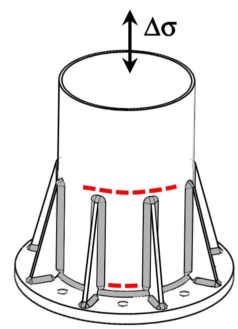
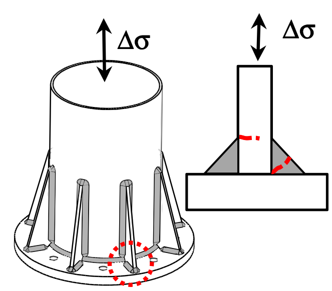
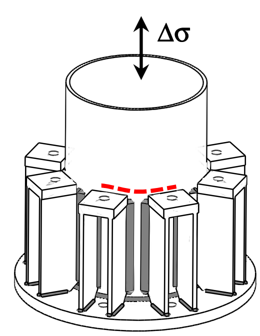
SECTION 6—ATTACHMENTS				
6.1 Longitudinal attachments with partial- or full-penetration groove-welds, or fillet-welds, in which the main member is subjected to longitudinal loading:			In the main member at the toe of the weld at the termination of attachment.	
L < 2 in.:	44.0	10.0		
2 in. ≤ *L* ≤ 12t and 4 in.:	22.0	7.0		
L > 12t or 4 in. when t ≤ 1 in.:	11.0	4.5		
6.2 Tube-to-transverse plate connections stiffened by longitudinal attachments with partial- or full penetration groove-welds, or fillet-welds in which the tube is subjected to longitudinal loading and the welds are wrapped around the attachment termination.	$K_F \leq 2.5$: 11.0 (See detail 5.4)	$K_I \leq 5.5$: 7.0 (See detail 5.4)	In tube wall at the toe of the attachment to tube weld at the termination of attachment. In tube wall at the toe of tube-to-transverse plate weld.	
6.3 Transverse load-bearing partial joint penetration groove-welded or fillet-welded attachments where t ≤ 0.5 in. and the main member is subjected to minimal axial and/or flexural loads (When t > 0.5 in., see note c).	*44.0*	*10.0*	In base metal at the weld toe or through weld throat.	Longitudinal stiffeners welded to base plates. 
6.4 Tube-to-transverse plate connections stiffened by longitudinal stool type attachments with partial- or full penetration groove-welds, or fillet-welds in which the tube is subjected to longitudinal loading and the welds are wrapped around the attachment termination.	*11.0*	*2.6*	In tube wall at the toe of the attachment to tube weld at termination of attachment.	

Table 11.9.3.1-1—Fatigue Details of Cantilevered and Noncantilevered Support Structures (continued)

Notes:

a. In a branching member with respect to the stress in the branching member:

$$(\Delta F)_{TH} = 1.2 \text{ ksi} \text{ ; when } r/t \leq 24 \text{ for the chord member}$$

$$(\Delta F)_{TH} = 1.2 \times \left(\frac{24}{\frac{r}{t}} \right)^{0.7} \text{ ksi ; when } r/t > 24 \text{ for the chord member}$$

In a chord member with respect to the stress in the chord member: $(\Delta F)_{TH} = 4.5$ ksi.

b. In a branching member with respect to the stress in the branching member: $(\Delta F)_{TH} = 1.2$ ksi
In main member with respect to the stress in the main member (column):

$$(\Delta F)_{TH} = 1.0 \text{ ksi} \text{ ; when } r/t \leq 24 \text{ for the chord member}$$

$$(\Delta F)_{TH} = 1.0 \times \left(\frac{24}{\frac{r}{t}} \right)^{0.7} \text{ ksi ; when } r/t > 24 \text{ for the chord member}$$

where:
The nominal stress range in the main member equals $(S_R)_{\text{main member}} = (S_R)_{\text{branching member}} (t_b/t_c)\ \alpha$
where t_b is the wall thickness of the branching member, t_c is the wall thickness of the main member (column), and α is the ovalizing parameter for the main member equal to 0.67 for in-plane bending and equal to 1.5 for out-of-plane bending in the main member.
$(S_R)_{\text{branching member}}$ is the calculated nominal stress range in the branching member induced by fatigue design loads. (See Article C11.9.3.)
The main member shall also be designed for $(\Delta F)_{TH} = 4.5$ ksi using the elastic section of the main member and moment just below the connection of the branching member.

c. When $t > 0.5$ in., $(\Delta F)_{TH}$ shall be the lesser of 10.0 ksi or the following:

$$(\Delta F)_{TH} = 10.0 \times \left(\frac{0.0055 + 0.72 \dfrac{H}{t_p}}{t_p^{\frac{1}{6}}} \right) \text{ksi}$$

where H is the effective weld throat in in., and t_p is the attachment plate thickness, in.

d. The diameter of coped holes shall be the greater of 1 in., twice the gusset plate thickness, or twice the tube thickness.

e. Reinforced and unreinforced holes and cutouts shall be detailed as shown in Figures 5.6.6.1-1, 5.6.6.1-2, and 5.5.6.1-3.

f. The standard fillet-welded gusseted box or ring-stiffened box connections in Article 5.6.7 shall be used for infinite life.

g. Threshold values are tabulated for steel details. Threshold values of aluminum details shall be obtained by dividing the respective threshold values with 2.6.

Table C11.9.3.1-1—Fatigue Details of Support Structures Tested in the Laboratory

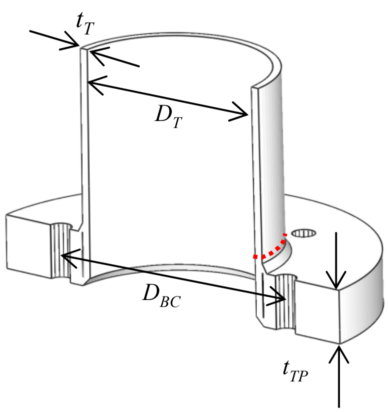
Description	Identification of Parameters	Tube Configuration	Detail Parameters	Finite Life Constant, $A \times 10^8$ ksi^3	Threshold, ΔF_{TH} ksi
Fillet-welded tube-to-transverse plate connections		Round	$t_T = 0.179$ in. $D_T = 10$ in. $t_{TP} = 2$ in. $D_{BC} = 23.3$ in. $N_B = 4$	3.9 ($K_F = 2.8$)	4.5 ($K_I = 5.6$)
		Round	$t_T = 0.239$ in. $D_T = 13$ in. $t_{TP} = 2$ in. $D_{BC} = 20$ in. $N_B = 4$	3.9 $K_F = 2.9$	4.5 ($K_I = 6.2$)
		Multisided	$t_T = {}^3/_{16}$ in. $D_T = 10$ in. $t_{TP} = 2$ in. $D_{BC} = 23.3$ in. $N_B = 4$ $N_S = 8$ $r_b = 0.5$ in.	3.9 ($K_F = 3.2$)	2.6 ($K_I = 6.6$)
		Multisided	$t_T = {}^1/_4$ in. $D_T = 13$ in. $t_{TP} = 2$ in. $D_{BC} = 20$ in. $N_B = 4$ $N_S = 8$ $r_b = 0.5$ in	($K_F = 3.5$)	2.6 ($K_I = 7.6$)
		Multisided	$t_T = {}^5/_{16}$ in. $D_T = 24$ in. $t_{TP} = 3$ in. $D_{BC} = 30$ in. $N_B = 16$ $N_S = 16$ $r_b = 4$ in.	3.9 ($K_F = 2.9$)	4.5 ($K_I = 6.5$)

Table C11.9.3.1-1—Fatigue Details of Support Structures Tested in the Laboratory (continued)

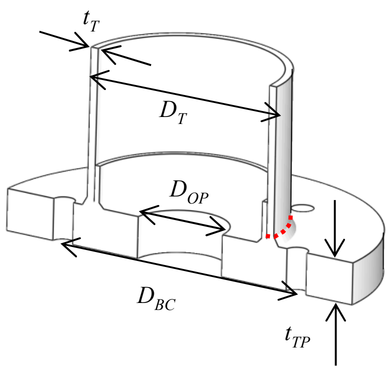
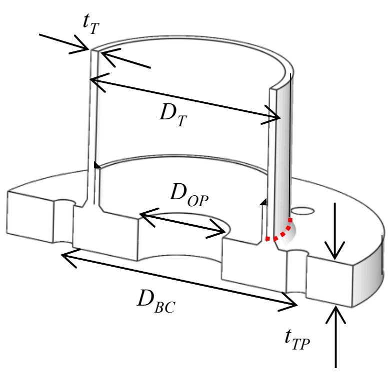
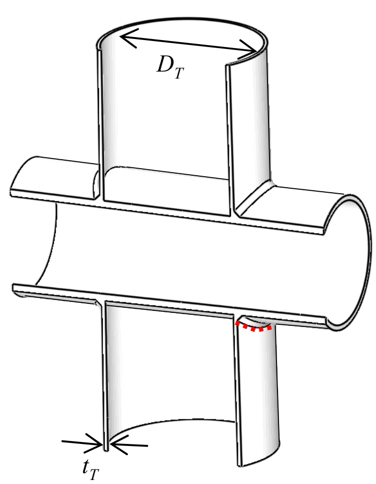
Full-penetration groove-welded tube-to-transverse plate connections with backing ring attached to the plate with a full penetration weld, or with a continuous fillet-weld around interior face of backing ring, and the backing ring not welded to the tube.		Round	t_T = 0.179 in. D_T = 10 in. t_{TP} = 2 in. D_{BC} = 23.3 in. N_B = 4 D_{OP} = 7.6 in.	3.9 (K_F = 2.0)	7.0 (K_I = 3.8)
		Round	t_T = 0.239 in. D_T = 13 in. t_{TP} = 2 in. D_{BC} = 20 in. N_B = 4 D_{OP} = 4 in.	11.0 (K_F = 1.6)	10.0 (K_I = 2.7)
		Round	t_T = 0.239 in. D_T = 13 in. t_{TP} = 2.5 in. D_{BC} = 20 in. N_B = 4 D_{OP} = 7 in	11.0 (K_F = 1.6)	10.0 (K_I = 2.9)
Full-penetration groove-welded tube-to-transverse plate connections with the backing ring not attached to the plate, and the backing ring welded to the tube with a continuous fillet-weld at top face of backing ring.		Round	t_T = 0.179 in. D_T = 10 in. t_{TP} = 2 in. D_{BC} = 23.3 in. N_B = 4 D_{OP} = 8.6 in	3.9 (K_F = 2.2)	4.5 (K_I = 4.3)
		Round	t_T = 0.239 in. D_T = 13 in. t_{TP} = 2 in. D_{BC} = 20 in. N_B = 4 D_{OP} =11.5 in.	3.9 (K_F = 2.3)	4.5 (K_I = 4.7)
		Multisided	t_T = $^5/_{16}$ in. D_T = 24 in. t_{TP} = 2.5 in. D_{BC} = 30 in. N_B = 8 D_{OP} = 14 in. N_S = 16 r_b = 4 in.	3.9 (K_F = 1.9)	7.0 (K_I = 4.0)
Partial penetration groove-welded mast-arm-to-column pass-through connections.		Round	t_T = $^1/_4$ in. D_T = 13 in.	11.0 (K_F = 2.1)	4.5 (K_I = 2.4)

Table C11.9.3.1-1—Fatigue Details of Support Structures Tested in the Laboratory (continued)

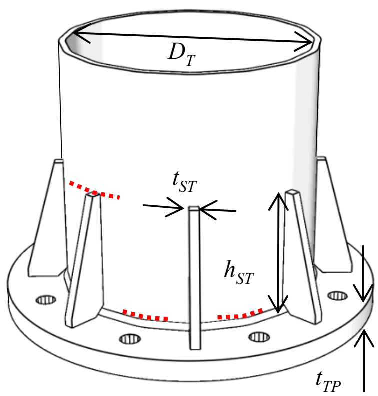
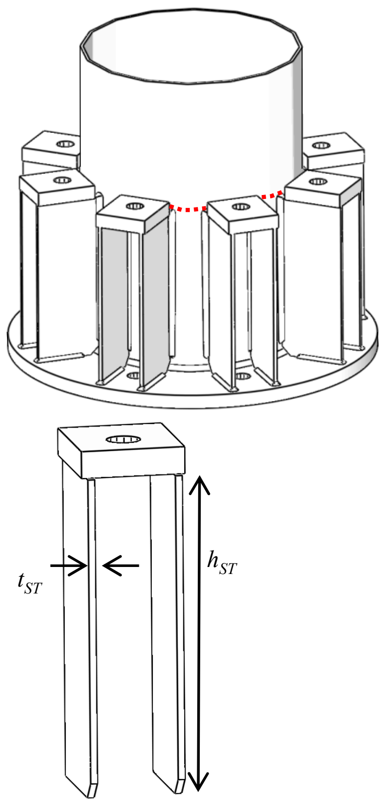
Tube-to-transverse plate connections stiffened by longitudinal attachments with partial- or full penetration groove-welds, or fillet-welds in which the tube is subjected to longitudinal loading and the welds are wrapped around the attachment termination.		Multisided	$t_T = {}^5/_{16}$ in. $D_T = 24$ in. $t_{TP} = 2$ in. $D_{BC} = 30$ in. $N_B = 8$ $N_S = 16$ $r_b = 4$ in $N_{ST} = 8$ $h_{ST} = 18$ in. $t_{ST} = {}^3/_8$ in.	Cracking at top of stiffener 11.0 ($K_F = 2.4$)	7.0 ($K_I = 5.3$)
				Cracking at end plate fillet-weld toe on tube wall 3.9 ($K_F = 1.9$)	7.0 ($K_I = 4.0$)
Tube-to-transverse plate connections stiffened by longitudinal stool type attachments with partial- or full penetration groove-welds, or fillet-welds in which the tube is subjected to longitudinal loading and the welds are wrapped around the attachment termination.		Multisided	$t_T = {}^5/_{16}$ in. $D_T = 24$ in. $t_{TP} = 2$ in. $D_{BC} = 30$ in. $N_B = 8$ $N_S = 16$ $r_b = 1$ in. $N_{ST} = 8$ $h_{ST} = 18$ in. $t_{ST} = {}^3/_8$ in.	11.0 ($K_F = 2.3$)	2.6 ($K_I = 3.2$)

Table 11.9.3.1-2—Fatigue Stress Concentration Factors, K_F

Section Type	Detail	Location	Fatigue Stress Concentration Factor for Finite Life, K_F	Section Type
Round	Fillet-welded tube-to-transverse plate connections	Fillet-weld toe on tube wall	$K_F = 2.2 + 4.6 \times (15 \times t_T + 2) \times \left(D_T^{1.2} - 10\right) \times \left(C_{BC}^{0.03} - 1\right) \times t_{TP}^{-2.5}$ Valid for: 0.179 in. $\leq t_T \leq$ 0.5 in.; 8 in. $\leq D_T \leq$ 50 in.; 1.5 in. $\leq t_{TP} \leq$ 4 in.; 1.25 $\leq C_{BC} \leq$ 2.5	(11.9.3.1–2)
	Groove-welded tube-to-transverse plate connections	Groove-weld toe on tube wall	$K_F = 1.35 + 16 \times (15 \times t_T + 1) \times (D_T - 5) \times \left(\dfrac{C_{BC}^{0.02} - 1}{4 \times C_{OP}^{-0.7} - 3}\right) \times t_{TP}^{-2}$ Valid for: 0.179 in. $\leq t_T \leq$ 0.625 in.; 8 in. $\leq D_T \leq$ 50 in.; 1.5 in. $\leq t_{TP} \leq$ 4 in.; 1.25 $\leq C_{BC} \leq$ 2.5; 0.3 $\leq C_{OP} \leq$ 0.9	(11.9.3.1–3)
	Fillet-welded tube-to-transverse plate connections stiffened by longitudinal attachments	Weld toe on tube wall at the end of attachment	$K_F = \left(\dfrac{t_{ST}^{0.4}}{t_T^{0.7}} + 0.3\right) \times \left(0.4 \times \dfrac{D_T^{0.8}}{N_{ST}^{1.2}} + 0.9\right)$ Valid for: 0.25 in. $\leq t_{ST} \leq$ 0.75 in.; $8 \leq N_{ST}$; 0.25 in. $\leq t_T \leq$ 0.625 in.; 24 in. $\leq D_T \leq$ 50 in.	(11.9.3.1–4)
	Fillet-welded tube-to-transverse plate connections stiffened by longitudinal attachments	Fillet-weld toe on tube wall	$K_F = \left[\left(130 \times \dfrac{D_T^{0.15}}{N_{ST}^{1.5}} + 1\right) \times \left(\dfrac{0.13}{h_{ST} + 7}\right) \times \left(\dfrac{6.5}{t_{ST}^{0.5}} - 1\right) \times K_F \text{ as per Eq. 11.9.3.1-1}\right]$ Valid for: 12 in. $\leq h_{ST} \leq$ 42 in.; 0.25 in. $\leq t_{ST} \leq$ 0.75 in.; $8 \leq N_{ST}$; 24 in. $\leq D_T \leq$ 50 in.	(11.9.3.1–5)
Multisided	As above	As above	Multiply respective K_F above by: $\left[1 + (D_T - r_b) \times N_S^{-2}\right]$ Valid for: 8 in. $\leq D_T \leq$ 50 in.; 1 in. $\leq r_b \leq$ 4 in.; $8 \leq N_S \leq 16$	(11.9.3.1–6)

11.10—REFERENCES

AASHTO. 2002. *Standard Specifications for Highway Bridges*, 17th Edition. American Association of State Highway and Transportation Officials, Washington, DC.

AASHTO. 2013. *Standard Specifications for Structural Supports for Highway Signs, Luminaires, and Traffic Signals*, Sixth Edition. American Association of State Highway and Transportation Officials, Washington, DC.

Ahearn, E. B., and J. A. Puckett. 2010. *Reduction of Wind-Induced Vibrations in High-Mast Light Poles*, Report No. FHWA-WY-10/02F, University of Wyoming, Laramie, WY.

Albert M. N., L. Manuel, K. H. Frank, and S. L. Wood. 2007. *Field Testing of Cantilevered Traffic Signal Structures under Truck-Induced Gust Loads*, Report No. FHWA/TX-07/4586-2. Center for Transportation Research, Texas Department of Transportation, Austin, TX.

Amir, G., and A. Whittaker. 2000. "Fatigue-Life Evaluation of Steel Post Structures II: Experimentation," *Journal of Structural Engineering*. American Society of Civil Engineers, New York, NY. Vol. 126, No. 3, Vol. 2 (March 2000), pp. 331–340.

ASTM. 2012. "Standard Specification for Structural Bolts, Steel, Heat Treated, 120/105 ksi Minimum Tensile Strength," A325-12, *Annual Book of ASTM Standards*. American Society for Testing Materials, West Conshohocken, PA.

ASTM. 2014. "Standard Specification for Structural Bolts, Alloy Steel, Heat Treated, 150 ksi Minimum Tensile Strength," A490-14a, *Annual Book of ASTM Standards*. American Society for Testing Materials, West Conshohocken, PA.

AWS. 2010, *Structural Welding Code- Steel, D1.1/D1.1M*. American Welding Society, Miami, FL

Connor, R. J., S. H. Collicott, A. M. DeSchepper, R. J. Sherman, and J. A. Ocampo. 2012. *Development of Fatigue Loading and Design Methodology for High-Mast Lighting Towers*, NCHRP Report 718. Transportation Research Board, National Research Council, Washington, DC.

Cook, R. A., D. Bloomquist, A. M. Agosta, and K. F. Taylor. 1996. *Wind Load Data for Variable Message Signs*, Report No. FL/DOT/RMC/0728-9488. University of Florida, Gainesville, FL. Report prepared for Florida Department of Transportation.

Creamer, B. M., K. G. Frank, and R. E. Klingner. 1979. *Fatigue Loading of Cantilever Sign Structures from Truck Wind Gusts*, Report No. FHWA/TX-79/10+209-1F. Center for Transportation Research, Texas State Department of Highways and Public Transportation, Austin, TX.

Dexter, R. J., and K. W. Johns. 1998. *Fatigue-Related Wind Loads on Highway Support Structures: Advanced Technology for Large Structural Systems*, Report No. 98-03. Lehigh University, Bethlehem, PA.

Dexter, R., and M. Ricker. 2002. *Fatigue-Resistant Design of Cantilevered Signal, Sign, and Light Supports*, NCHRP Report 469. Transportation Research Board, National Research Council, Washington, DC.

Fisher, J. W., A. Nussbaumer, P. B. Keating, and B. T. Yen. 1993. *Resistance of Welded Details Under Variable Amplitude Long-Life Fatigue Loading*, NCHRP Report 354. Transportation Research Board, National Research Council, Washington, DC.

Florea M. J., L. Manuel, K. H. Frank, and S. L. Wood. 2007. *Field Tests and Analytical Studies of the Dynamic Behavior and the Onset of Galloping in Traffic Signal Structures*, Report No. FHWA/TX-07/4586-1. Center for Transportation Research, Texas Department of Transportation, Austin, TX.

Fouad, F. H., J. S Davison, N. Delatte , E. A. Calvert , S. E. Chen, E. Nunez, and R. Abdalla, 2003. "Structural Supports for Highway Signs, Luminaires, and Traffic Signals," NCHRP Report 494. Transportation Research Board, National Research Council, Washington, DC.

Kaczinski, M. R., R. J. Dexter, and J. P. Van Dien. 1998. *Fatigue Resistant Design of Cantilevered Signal, Sign and Light Supports*, NCHRP Report 412. Transportation Research Board, National Research Council, Washington, DC.

Koenigs, M. T., T. A. Botros, D. Freytag, and K. H. Frank. 2003. *Fatigue Strength of Signal Mast Arm Connections*, Report No. FHWA/TX-04/4178-2. Center for Transportation Research, Texas Department of Transportation, Austin, TX.

McDonald, J. R., K. C. Mehta, W. Oler, and N. Pulipaka. 1995. *Wind Load Effects on Signals, Luminaires and Traffic Signal Structures*, Report No. 1303-1F. Wind Engineering Research Center, Texas Tech University, Lubbock, TX.

NOAA. 2010. *Comparative Climatic Data for the United States through 2010*, National Climatic Data Center, Asheville, NC.

Roy, S., Y. C. Park, R. Sause, J. W. Fisher, and E.K. Kaufmann. 2011. *Cost Effective Connection Details for Highway Sign, Luminaire and Traffic Signal Structures*. NCHRP Web Only Document 176 (Final Report for NCHRP Project 10-70). Transportation Research Board, National Research Council, Washington, DC.

Stam, A., N. Richman, C. Pool, C. Rios, T. Anderson, and K. Frank. 2011. *Fatigue Life of Steel Base Plate-to-Pole Connections for Traffic Structures*. Report No. FHWA/TX-11/9-1526-1. Texas Department of Transportation, Austin, TX.

SECTION 12: BREAKAWAY SUPPORTS

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SECTION 12:

BREAKAWAY SUPPORTS

12.1—SCOPE

Breakaway supports shall be provided based on the guidelines for use and location, as specified in Section 2. Breakaway supports shall be designed to yield, fracture, or separate when struck, thereby minimizing injury to the occupants and damage to the vehicle.

This Section addresses the structural, breakaway, and durability requirements for structures required to yield, fracture, or separate when struck by an errant vehicle. Structure types addressed include roadside sign, luminaire, call box, and pole top mounted traffic signal supports.

Breakaway devices shall meet the requirements herein and of the *Manual for Assessing Safety Hardware* (MASH) (2009). Additional guidelines for breakaway devices may be found in the *Roadside Design Guide* (2011).

C12.1

The term "breakaway support" refers to all types of sign, luminaire, call box, and pole top mounted traffic signal supports that are safely displaced under vehicle impact. Breakaway requirements of mailboxes and utility poles may be found in the *Roadside Design Guide*.

MASH is an update to and supersedes NCHRP Report 350*, Recommended Procedures for the Safety Performance Evaluation of Highway Features,* for the purposes of evaluating new safety hardware devices. MASH does not supersede any guidelines for the design of roadside safety hardware, which are contained within the AASHTO *Roadside Design Guide*. An implementation plan for MASH that was adopted jointly by AASHTO and FHWA states that all highway safety hardware accepted prior to the adoption of MASH—using criteria contained in NCHRP Report 350—may remain in place and may continue to be manufactured and installed. In addition, highway safety hardware accepted using NCHRP Report 350 criteria is not required to be retested using MASH criteria. However, new highway safety hardware not previously evaluated must utilize MASH for testing and evaluation.

12.2—DEFINITIONS

Breakaway—A design feature that allows a sign, luminaire, call box, or pole top mounted traffic signal support to yield, fracture, or separate near ground level on impact.

Call Box—Telephone device placed on a short post to allow emergency calls by stranded motorists.

Manufacturer—Company that makes a finished component.

Hinge—The weakened section of a support post designed to allow the post to rotate when impacted by a vehicle.

12.3—DESIGN OF BREAKAWAY SUPPORTS

Breakaway supports shall be designed to meet both the structural and the dynamic performance requirements of Articles 12.4 and 12.5, respectively.

As requested by the Owner, certification of both breakaway and structural adequacy shall be provided by the Manufacturer. Design calculations or test data of production samples to support certification shall be provided, if requested by the Owner. The data shall indicate a constant ability to produce a device that will meet both breakaway and structural requirements.

C12.3

The Manufacturer is responsible for breakaway testing and for submitting test reports to FHWA for review and approval. Manufacturers of luminaire poles with associated breakaway base components will commonly provide copies of the FHWA testing approval to the Owners. This approval, however, does not include any consideration of the structural adequacy of the component. Structural adequacy should be demonstrated to the Owner by the Manufacturer. In general, any breakaway support component should provide the same or greater structural strength than the support post or pole using the breakaway device.

12.4—STRUCTURAL PERFORMANCE

Breakaway supports shall be designed to carry the loads, as provided in Section 3, using the appropriate resistances for the material used, as stipulated in these Specifications.

Where the structural adequacy of the breakaway support or components associated with the breakaway feature is in question, load tests shall be performed. The load tests shall be performed and evaluated based on the following criteria:

- Breakaway supports shall be tested to determine their ultimate strengths. The loading arrangement and structure configuration shall be selected to maximize the deflection and stresses in the critical regions of the structure or breakaway component. More than one test load arrangement shall be used should a single arrangement not demonstrate the ultimate strength of the breakaway support. The breakaway support shall be tested in a manner that closely models field support conditions.
- The test load shall not be less than the Extreme I limit state with load factors provided in Table 3.4-1.
- Three samples for each test load arrangement shall be tested to determine the ultimate load that the breakaway support assembly is capable of supporting in the weakest direction.
- If one of the ultimate loads differs from the mean by more than ten percent, three additional samples shall be tested. Average of the lowest three ultimate loads out of the six test to determine the ultimate load.

C12.4

Typically, static load tests are conducted to verify the structural capacity of the breakaway support. The structure is required to withstand the design loads with appropriate safety. Further, because of the nature of the breakaway devices, additional tests such as fatigue and corrosion may be required by the Owner. In such cases, the Owner and Manufacturer should agree on the specific test requirements.

In general, breakaway devices should be tested to determine if they provide bending strengths compatible with the posts or poles they support. The structural support, including the breakaway device, may be tested for bending, shear, torsion, tension, or compression, to demonstrate the load-carrying capacity. For testing, a length of pole or post suitable for application of the test load may be attached to the breakaway device. The pole or post test length should model the actual structure as to thicknesses, attachment bolts, and so forth. The distance from the breakaway device to the application point of the test load should be at least five times the maximum major bending dimension of the pole or post. Additonally, the upper hinge mechanisms on certain large breakaway sign supports should be subjected to the structural performance considerations.

12.5—BREAKAWAY DYNAMIC PERFORMANCE

Breakaway supports shall meet the impact test evaluation criteria of Article 12.5.1, Article 12.5.2, or both. Additional provisions of Article 12.5.3 shall be considered.

12.5.1—Impact Test Evaluation Criteria

Criteria for testing, documentation, and evaluation of breakaway supports shall be performed in accordance with the guidelines of MASH (2009).

12.5.2—Analytical Evaluation of Impact Tests

As permitted by the Owner, numerical evaluation of impact tests may be allowed in lieu of physical testing provided that an analytical model has been proven to accurately and conservatively predict the dynamic performance of the structural breakaway support, and deformation of, or intrusion into the passenger compartment is not likely. Verification of the analytical model shall be supported by an adequate number of full-scale impact tests.

12.5.3—Additional Requirement

The following provision ensures predictable and safe displacement of the breakaway support.

Substantial remains of breakaway supports shall not project more than 4 in. above a line between the straddling wheels of a vehicle on 60-in. centers. The line connects any point on the ground surface on one side of the support to a point on the ground surface on the other side, and it is aligned radially or perpendicular to the centerline of the roadway.

C12.5.3

Breakaway support mechanisms are designed to function properly when loaded primarily in shear. Most mechanisms are designed to be impacted at bumper height, typically 18–20 in. above the ground. If impacted at a significantly higher point, the bending moment in the breakaway base may be sufficient to bind the mechanism, resulting in nonactivation of the breakaway device. For this reason, it is critical that breakaway supports not be located near ditches or on steep slopes or at similar locations where a vehicle is likely to be partially airborne at the time of impact. The type of soil may also affect the activation mechanisms of some breakaway supports. Additional guidance on typical breakaway supports may be found in the *Roadside Design Guide*.

Breakaway supports, including those placed on roadside slopes, must not allow impacting vehicles to snag on either the foundation or any substantial remains of the support. Surrounding terrain may be required to be graded to permit vehicles to pass over any nonbreakaway portion of the installation that remains in the ground or rigidly attached to the foundation. The specified limit on the maximum stub height lessens the possibility of snagging the undercarriage of a vehicle after a support has broken away from its base, and minimizes vehicle instability if a wheel hits the stub. The necessity of this requirement is based on field observations. Application of the clearance requirement is illustrated in Figure C12.5.3-1.

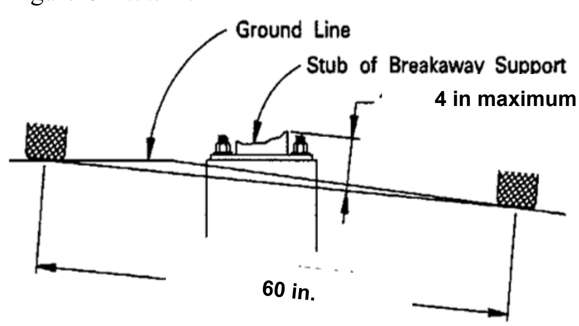

Figure C12.5.3-1—Stub Height Requirements

12.6—REFERENCES

AASHTO. 2009. *Manual for Assessing Safety Hardware* (MASH), First Edition. American Association of State Highway and Transportation Officials, Washington, DC.

AASHTO. 2011. *Roadside Design Guide*, Fourth Edition, RSDG-4-M. American Association of State Highway and Transportation Officials, Washington, DC.

Ross, H. E. 1995. "Evolution of Roadside Safety." In *Transportation Research Circular 435: Roadside Safety Issues.* Transportation Research Board, National Research Council, Washington, DC.

Ross, H. E. 1995. "Update to NCHRP Report 230." In *Transportation Research Circular 451: International Crash Test Standards for Roadside Safety Features.* Transportation Research Board, National Research Council Washington, DC.

Ross, H. E., D. L. Sicking, R. A. Zimmer, and J. D. Michie. 1993. *Recommended Procedures for the Safety Performance Evaluation of Highway Features*, NCHRP Report 350. Transportation Research Board, National Research Council, Washington, DC.

SECTION 13: FOUNDATION DESIGN

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SECTION 13:

FOUNDATION DESIGN

13.1—SCOPE

Provisions of this Section specify design requirements for drilled shafts, spread footings, piles, and screw-in helixes for the foundations of structural supports of signs, luminaires, and traffic signals.

Design of foundations shall be based on the *AASHTO LRFD Bridge Design Specifications* (*LRFD Design*, 2014) for design requirements not addressed in this Section. Foundations shall be designed to resist the factored loads given in Section 3 and induced reactions, in accordance with the general principles of this Section. Foundation settlement, rotation, and overall stability (i.e., sliding and overturning) should be controlled to alleviate the possibility of failure of the structure or its having an unsightly appearance. Selection of foundation type shall be based on considerations such as the magnitude and direction of loading, depth to suitable bearing materials, frost depth, and ease and cost of construction.

Section 10 of *LRFD Design* may be use to determine the resistance.

C13.1

The material contained in this Section is general in nature and no attempt has been made to specify definite design criteria for foundations.

Section 10 of *LRFD Design* may be use to determine the resistance to resist the loads as provide in Section 3 of these Specifications. This is the preferred method.

Simplified methods are provided herein. These methods were included in the *Standard Specifications for Structural Supports for Highway Signs, Luminaires, and Traffic Signals* and have been adapted for use in this Section. In some cases load effects are converted back to an allowable stress level to be used with allowable resistances.

13.2—DEFINITIONS

Drilled Shaft—Also referred to as drilled pier, cast-in-drilled-hole pile, drilled caisson, or large bored pile; a foundation, constructed by placing concrete in a drilled hole with or without steel reinforcement.

Pile—A long, relatively slender foundation, installed by driving, drilling, auguring, or jetting.

Screw-in Helix—Galvanized steel foundation installed by rotary equipment.

Spread Footing—A generally rectangular or square prism of concrete that distributes the load of the vertical support to the subgrade.

13.3—NOTATION

c = ultimate shear strength of cohesive soil (cohesion) (ksf) (C13.6.1.1)
D = width or diameter of foundation (ft) (C13.6.1.1)
F = lateral soil reaction at toe of drilled shaft in cohesionless soil (kip) (C13.6.1.1)
H = M_F/V_F (ft) (C13.6.1.1)
K_p = passive earth pressure coefficient (C13.6.1.1)
L = embedded length of foundation (ft) (C13.6.1.1)
M_F = factored moment at groundline from loads computed according to Section 3, “Loads” (kip-ft) (C13.6.1.1)
M_u = factored maximum moment in the shaft (kip-ft) (C13.6.1.1)
q = coefficient (ft) (C13.6.1.1)
V_F = factored shear load at groundline computed according to Section 3, “Loads” (kip) (C13.6.1.1)
ϕ = angle of internal friction (degrees) (C13.6.1.1)
γ = effective unit weight of soil (kip/ft^3) (C13.6.1.1)

13.4—DETERMINATION OF SOIL PROPERTIES

A geotechnical study that may include subsurface explorations shall be performed for each substructure element to provide the necessary information for the design and construction of foundations. The extent of exploration shall be based on subsurface conditions, structure type, and project requirements.

Laboratory tests and in-situ tests shall be conducted conforming to AASHTO, ASTM, or Owner-supplied standards, to obtain soil parameters that are necessary for the analysis or design of foundations.

As a minimum, the geotechnical study or subsurface exploration and testing should provide information on the design bearing pressure, lateral resistance, groundwater elevation, unit weight of soils, angle of internal friction, cohesion strength, and other geotechnical features that could affect the design of the foundation for a particular site.

Subsurface explorations may be waived if the following conditions are met:

- The structure type will pose an insignificant hazard if the foundation fails, and
- A reasonable estimate is made for the subsurface condition.

C13.4

Regardless of the type of foundation used, comprehensive soil information is valuable information for foundation designs. In-place strength tests, particularly standard penetration tests, are very beneficial and usually satisfactory for determining the soil strength data required for design.

13.5—FOUNDATION BEARING CAPACITY

The bearing capacity of the foundation may be estimated using analytical procedures given in *LRFD Design* or other generally accepted theories, based on soil and rock parameters measured by in-situ and/or laboratory tests.

13.5.1—Bearing Resistance

The soil-bearing resistance that may be used in designing the foundation will depend on the type of foundation used and the supporting soil.

Section 10 of *LRFD Design* may be used to determine the resistance.

13.6—DRILLED SHAFTS

Drilled shafts shall be cast-in-place concrete and may include deformed steel reinforcement, structural steel sections, permanent steel casing, or a combination of these elements as required by the design.

Drilled shafts shall be designed to support the design loads with adequate bearing, lateral resistance, and structural capacity with tolerable settlements and lateral displacements. Drilled shafts shall provide adequate resistance for applied torsional loads.

C13.6

Drilled shafts may be considered to resist high lateral or uplift loads when suitable soil conditions are present.

Common construction methods entail drilling a hole to the required foundation depth, and then filling it with reinforced or unreinforced concrete. The vertical support structure (e.g., pole) is usually anchored to the concrete shaft through anchor bolts.

It is also possible to directly embed a portion of a pole by making a cylindrical hole in the ground approximately 12 in. larger than the pole diameter and backfilling with well-graded crushed stone or unreinforced concrete. This is a commonly used foundation for prestressed concrete, wood, and fiber-reinforced composite poles.

13.6.1—Geotechnical Design

13.6.1.1—Embedment

Shaft embedment shall be sufficient so as to provide suitable vertical and lateral load capacities and acceptable displacements. Section 10 of *LRFD Design* may be used to determine the resistance.

In lieu of more detailed procedures, Broms' approximate procedures for the estimation of embedment as outlined in the commentary may be used.

C13.6.1.1

Methods of analysis that use manual computation were developed by Broms (1964a and 1964b). They are discussed in detail by Hannigan et al. (2006). Analysis methods that model the horizontal soil resistance using *P-y* curves were developed by Reese (1984). This analysis has been well developed and software is available for analyzing single piles and pile groups (Reese 1986, Williams et al. 2003, and Hannigan et al. 2006).

For simplicity, Broms' procedures for embedment length in cohesive and cohesionless soils are summarized below regarding the ultimate lateral soil resistance. Certain structures may warrant additional considerations regarding limitations to the lateral translation at the top of the shaft. Some structures or soil conditions may require a more detailed final design procedure than the Broms' procedures.

Broms studied laterally loaded piles in cohesive and cohesionless soils. Simplifying assumptions concerning the distribution of the soil reactions along the pile and statics were used to estimate the lateral resistance. Broms' assumptions for the distribution of a cohesive soil's reactions at ultimate load are shown in Figure C13.6.1.1-1. Broms' solution for cohesive soils may be presented by the following equation from which the required embedment length, L, can be determined:

$$L = 1.5D + q\left[1 + \sqrt{2 + \frac{(4H + 6D)}{q}}\right] \qquad \text{(C13.6.1.1-1)}$$

in which:

$$H = \frac{M_F}{V_F} \qquad \text{(C13.6.1.1-2)}$$

and:

$$q = \frac{V_F}{9cD} \qquad \text{(C13.6.1.1-3)}$$

where:

D = shaft diameter (ft)
c = the ultimate shear strength of cohesive soil (ksf)
M_F = the factored moment at groundline (kip-ft)
V_F = the factored shear at groundline (kip)

For the required embedment length L, the maximum moment in the shaft can be calculated as

$$M_u = V_F\left(H + 1.5D + 0.5q\right) \qquad \text{(C13.6.1.1-4)}$$

and is located at $(1.5D + q)$ below groundline.

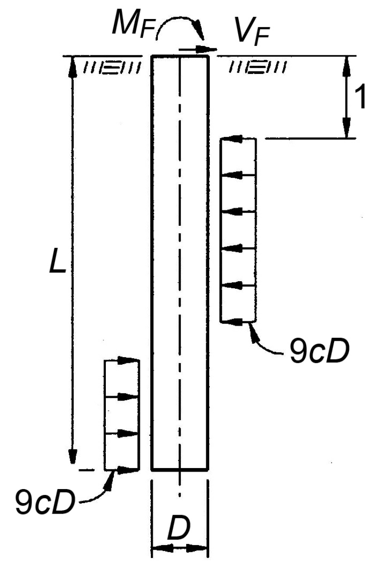

Figure C13.6.1.1-1—Foundation in Cohesive Soil

Broms' assumptions for the distribution of a cohesionless soil's reactions at ultimate load are shown in Figure C13.6.1.1-2. For cohesionless soils, Broms' procedure may be given by the following equations, from which the required embedment length, L, can be determined by using trial and error:

$$L^3 - \frac{2V_F L}{K_p \gamma D} - \frac{2M_F}{K_p \gamma D} = 0 \qquad \text{(C13.6.1.1-5)}$$

where:

$$K_p = \tan^2\left(45 + \frac{\phi}{2}\right) \qquad \text{(C13.6.1.1-6)}$$

ϕ = angle of internal friction (degrees), and
γ = effective unit weight of the soil (k/ft^3) including load factors. See *LRFD Design,* Section 3.4.1 for guidance on load factors for soil. Conservatively, a unit load factor may be used.

For the required embedment length, L, the maximum moment in the shaft can be calculated as:

$$M_u = V_F\left(H + 0.54\sqrt{\frac{V_F}{\gamma DK_p}}\right) \qquad \text{(C13.6.1.1-7)}$$

and is located at:

$$\left(0.82\sqrt{\frac{V_F}{\gamma DK_p}}\right)$$

below groundline.

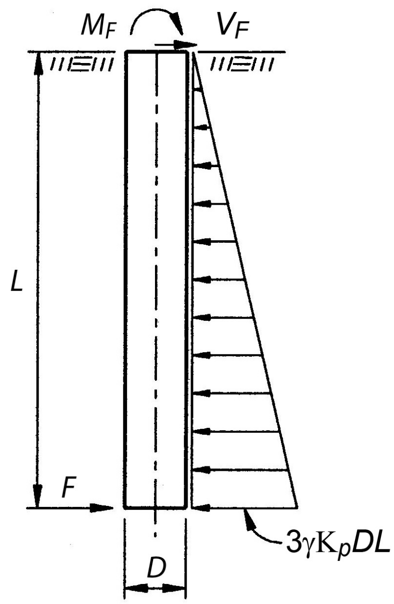

Figure C13.6.1.1-2—Foundation in Cohesionless Soil

13.6.1.2—Capacity

The axial capacity, lateral capacity, and movements of the drilled shaft in various types of soils may be estimated according to methods prescribed in *LRFD Design.*

13.6.2—Structural Design

The structural design of drilled shafts shall be in accordance with the provisions for the design of reinforced concrete given in *LRFD Design*.

13.6.2.1—Details

Drilled shaft diameters should be sized in 6-in. increments. A minimum concrete cover of 3 in. over steel reinforcement shall be required. The reinforcing cage shall be adequately supported and secured against displacement before concrete is placed.

C13.6.2.1

The minimum concrete cover is required for protection of reinforcement against corrosion. It is measured from the concrete surface to the outermost surface of the ties or spirals of the reinforcing cage. The cage must be adequately supported by bar chairs or other means to prevent its displacement by workers or concrete placement. Larger covers may need to be specified by the Designer in certain cases to ensure that the minimum cover required for protection is provided.

13.7—SPREAD FOOTINGS

Spread footing-type foundations may be used to distribute the design loads to the supporting soil strata. A vertical shaft or stem may be constructed with the footing of such a size to accommodate the anchor bolts and base plates required for the pole support.

Spread footings shall be designed to support the design loads with adequate bearing and structural capacity, and tolerable settlements. The footing shall provide resistance to sliding and overturning.

13.7.1—Geotechnical Design

The bearing capacity and settlement of the spread footing in various types of soils may be estimated according to methods prescribed in *LRFD Design*. Uplift due to the eccentricity of the loading shall be restricted to one corner of the footing. The tension area shall not exceed 25 percent of the total bearing area of the footing.

C13.7.1

A portion of a spread footing may be subjected to uplift due to the eccentricity of applied loads. For a footing loaded eccentrically about only one axis, uplift occurs when the resultant pressure on the base of the footing is located at a distance from the footing's centroid that exceeds $^1/_6$ of the footing-plan dimension.

In general, it is good practice to avoid such cases of large eccentricity where uplift occurs. In some limited applications under high wind loads, however, it may be reasonable to allow some uplift for more economical designs. The provisions of this Section provide limitations on the area of uplift. Restricting uplift to one corner implies biaxial bending. For uniaxial bending, uplift is not permitted.

13.7.2—Structural Design

The structural design of spread footings shall be in accordance with the provisions for the design of reinforced concrete given in *LRFD Design*.

For spread footings on piles, computations for moments and shears may be based on the assumption that the reaction from any pile is concentrated at each pile's center.

13.8—PILES

Piling should be considered when adequate soil conditions cannot be found within a reasonable depth. Piling may also be used where the potential for large, unacceptable settlements exists.

13.8.1—Geotechnical Design

The axial capacity, lateral capacity, and settlement of piles in various types of soils may be estimated according to the methods prescribed in *LRFD Design*.

13.8.2—Structural Design

The structural design of piling of different materials shall be in accordance with the provisions given in *LRFD Design*.

13.9—SCREW-IN HELIX

Screw-in helix foundations shall consist of a galvanized round pipe or tube with a formed helix plate at the embedded end and a connection plate at the top end. The foundation's lateral load capacity is a function of its length, diameter, and the properties of the soil.

C13.9

Screw-in helix foundations are typically used for street lighting poles, pole top mounted traffic signal supports, and other small structures. These foundations can be installed by conventional rotary equipment in a short amount of time, and they have the capability of being retrieved from the soil and reused at other locations.

13.10—REFERENCES

AASHTO. 2002. Standard Specifications for Highway Bridges, 17th Edition. American Association of State Highway and Transportation Officials, Washington, DC.

AASHTO. 2010. AASHTO LRFD. Bridge Construction Specifications, Third Edition. American Association of State Highway and Transportation Officials, Washington, DC.

AASHTO. 2014. AASHTO LRFD Bridge Design Specifications, Seventh Edition. American Association of State Highway and Transportation Officials, Washington, DC.

AASHTO. 2014. Standard Specifications for Transportation Materials and Methods of Sampling and Testing, 34rd Edition. American Association of State Highway and Transportation Officials, Washington, DC.

ACI. 1993. "Design and Construction of Drilled Piers," ACI 336.3R-93. Manual of Concrete Practice. American Concrete Institute, Farmington Hills, MI.

ACI. 1995. "Suggested Analysis and Design Procedures for Combined Footings and Mats," ACI 336.2R-88 (Reapproved 1993). In Manual of Concrete Practice. American Concrete Institute, Farmington Hills, MI.

ACI. 2011. Building Code Requirements for Structural Concrete, ACI 318-2011. American Concrete Institute, Farmington Hills, MI.

Broms, B. B. 1964a. "Lateral Resistance of Piles in Cohesionless Soils," Journal of the Soil Mechanics and Foundations Division. American Society of Civil Engineers, Washington, DC, May 1964, Vol. 90, No. SM3.

Broms, B. B. 1964b. "Lateral Resistance of Piles in Cohesive Soils," Journal of the Soil Mechanics and Foundations Division. American Society of Civil Engineers, Washington, DC, March 1964, Vol. 90, No. SM2.

Broms, B. B. 1965. "Design of Laterally Loaded Piles," Journal of the Soil Mechanics and Foundations Division. American Society of Civil Engineers, Washington, DC, Vol. 91, No. SM3 (May 1965).

Brown, D.A, J. P. Turner, and R. J. Castelli. 2010. Drilled Shafts: Construction Procedures and LRFD Design Methods, NHI Course No. 132014, Geotechnical Engineering Circular No. 10, FHWA NHI-10-016. Federal Highway Administration, U.S. Department of Transportation, Washington, DC.

GAI Consultants, Inc. 1982. Laterally Loaded Drilled Pier Research, Volume 1: Design Methodology. Electric Power Research Institute, Palo Alto, CA.

Hannigan, P. J., G. G. Goble, G. Thendean, G. E. Likins, and F. Rausche. 2006. Design and Construction of Driven Pile Foundations, FHWA-NHI-05-042 and NHI-05-043. Federal Highway Administration, U.S. Department of Transportation, Washington, D.C., Vols. I and II.

Reese, L. C. 1984. Handbook on Design of Piles and Drilled Shafts under Lateral Load, Report No. FHWA-IP-84-11. Federal Highway Administration, U.S. Department of Transportation, Washington, DC.

Reese, L. C. 1986. Behavior of Piles and Pile Groups Under Lateral Load, FHWA/RD-85/106. Federal Highway Administration, Office of Engineering and Highway Operations Research and Development, U.S. Department of Transportation, Washington, DC, p. 311.

Reese, L. C., and M. W. O'Neill. 1988. Drilled Shafts: Construction Procedures and Design Methods, Report No. FHWA-HI-88-042. Federal Highway Administration, U.S. Department of Transportation, Washington, DC.

Williams, M. E., M. McVay, and M. I. Hoit. 2003. "LRFD Substructure and Foundation Design Programs." In Proc., International Bridge Conference, June 9–11, 2003, Pittsburgh, PA.

SECTION 14: FABRICATION, MATERIALS, AND DETAILING

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SECTION 14:

FABRICATION, MATERIALS, AND DETAILING

14.1—SCOPE

This Section specifies fabrication provisions, specific information regarding materials, and detailing for structural supports for highway signs, luminaires, and traffic signals.

C14.1

The fabrication provisions of this Section are applicable to common structural supports for highway signs, luminaires, and traffic signals. Additional provisions may be required for unusual member types or usages.

14.2—NOTATION

D = nominal bolt diameter (14.5.2.4) (14.5.2.5)

H = height of backing ring at a groove-welded tube-to-transverse-plate connection (in.) (14.5.2.4) (C14.4.4.2)

R = root gap at a groove-welded tube-to-transverse-plate connection (in.) (14.4.4.2)

t = thickness of tube (in.) (14.4.4.2) (C14.4.4.2)

θ = angle of the sound beam for ultrasonic inspection of groove welds (degrees) (14.4.4.2)

14.3—WORKING DRAWINGS

The Contractor shall expressly understand that the Engineer's approval of the working drawings submitted by the Contractor addresses the requirements for "strength and detail," and that the Engineer assumes no responsibility for errors in dimensions.

Working drawings must be approved by the Engineer prior to performance of the work involved and such approval shall not relieve the Contractor of any responsibility under the contract for the successful completion of the work.

14.3.1—Shop Drawings

As required by the Owner, the Contractor shall submit copies of the detailed shop drawings to the Engineer for approval. Shop drawings shall be submitted sufficiently in advance of the start of the work to allow time for review by the Engineer and corrections by the Contractor, if any, without delaying the work.

Shop drawings shall give full, detailed dimensions and sizes of component parts of the structure and details of all miscellaneous parts, such as pins, nuts, bolts, and drains.

Where specific orientation of plates is required, the direction of rolling of plates shall be shown.

Unless otherwise specified in the contract documents, shop drawings shall identify all materials that shall be used to produce each piece.

14.3.2—Camber Diagram

A camber diagram shall be furnished to the Engineer by the Fabricator showing the camber at each panel point for trusses or arch ribs, and at the location of field splices and fractions of span length not greater than quarter points for continuous beams and girders or rigid frames. The camber diagram shall show calculated cambers to be used in preassembly of the structure.

14.3.3—Marking and Shipping

Each member shall be painted or marked with an erection mark for identification, and an erection diagram showing these marks shall be furnished to the Engineer. Metal stamping shall not be used to mark aluminum parts.

The Contractor shall furnish to the Engineer as many copies of material orders, shipping statements, and erection diagrams as the Engineer may direct. The mass (weight) of the individual members shall be shown on the statements. Members having a mass (weight) of more than 6.5 kips shall have the weight marked on them. Structural members shall be loaded on trucks or cars in such a manner that they may be transported and unloaded at their destination without being damaged.

Bolts, nuts, and washers from each rotational-capacity lot shall be shipped in the same container. If there is only one production lot number for each size of nut and washer, the nuts and washers may be shipped in separated containers. The gross weight of any container shall not exceed 0.3 kips. A list showing the quantity and description of materials shall be plainly marked on the outside of each container.

14.3.4—Full-Size Tests

When full-size tests of fabricated structural members are required in the contract documents, the Contractor shall provide suitable facilities, material, supervision, and labor necessary for making and recording the required tests.

14.4—STEEL STRUCTURES

14.4.1—Materials

This Article addresses the required material properties for steel structural supports for highway signs, luminaires, and traffic signals.

14.4.1.1—Minimum Thickness of Material

Unless otherwise required by the Owner's specifications, the minimum thickness of material for main supporting members of steel truss-type supports shall be 0.1793 in. For secondary members, such as bracing and truss webs, the minimum thickness shall be

C14.4.1.1

Main members are those that are strictly necessary to ensure integrity of a structural system. Secondary members are those that are provided for redundancy and stability of a structural system. Minimum thickness requirements are based on considerations such as

0.125 in. The minimum thickness of material for all members of pole-type supports and truss-type luminaire arms shall be 0.125 in. These limits may be reduced no more than ten percent for material designated by gage numbers.

Steel supports for small roadside signs may be less than 0.125 in. in thickness.

corrosion resistance and importance of the member for the structural system. The thickness 0.1793 in. is associated with 7 gage sheet steel material.

Supports without an external breakaway mechanism that have thicknesses less than 0.125 in. have shown good safety characteristics in that they readily fail under vehicle impact, with little damage to the vehicle or injury to the occupants. These thinner supports should be used on those installations considered to have a relatively short life expectancy, such as small roadside signs.

14.4.1.2—Anchor Bolt Types

Cast-in-place anchor bolts shall be used in new construction.

The following requirements shall apply:

- Anchor bolts may be headed through the use of a preformed bolt head or by other means, such as a nut, flat washer, or plate.
- Hooked anchor bolts with a yield strength not exceeding 55 ksi may be used.
- Deformed reinforcing bars may be used as anchor bolts.
- Anchor bolt materials shall be in accordance with Article 5.16.2.

C14.4.1.2

Research (Jirsa et al., 1984) has shown that headed cast-in-place anchor bolts perform significantly better than hooked anchor bolts, regarding possible pull-out prior to development of full tensile strength. Caution should be exercised when using deformed reinforcing bars as anchor bolts, because no fatigue test data are available on threaded reinforcing bar. The ductility of deformed reinforcing bars, as measured by elongation, can be significantly less than most other anchor bolts.

14.4.2—Bolted Connections

14.4.2.1—Bolts

Bolts for primary structural connections shall be high-strength galvanized bolts conforming to ASTM A325 with matching nuts and washers.

Bolts and other fasteners for secondary connections shall conform to ASTM A307 or A325 and be galvanized.

14.4.2.2—Holes

Holes for bolted connections shall be in accordance with the *AASHTO LRFD Bridge Construction Specifications* (AASHTO, 2010).

14.4.3—Slip Type Field Splice

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

C14.4.3

Applies only to tapered poles.

14.4.4—Welded Connections

Welding design and fabrication shall be in accordance with the latest edition of the *AWS Structural*

C14.4.4

Recommendations for proper detailing of fatigue critical welded connections are included in Table 11.9.3.1-1. All welds should be considered for fatigue

Welding Code D1.1—Steel. All welds shall be considered structural welds.

14.4.4.1—Stiffened Tube-to-Transverse-Plate Connections

In stiffened fillet-welded tube-to-transverse-plate connections (socket connections), only tapered stiffeners having a termination angle of 15 degrees on the tube at the tip of the stiffener shall be used. Alternative termination angles may be approved by the Owner based on sound engineering practices. The termination angle of the weld shall comply with Article 5.6.4 except that alternative angles may be approved by the Owner based on sound engineering practices.

The minimum height of stiffeners shall be 12 in. At least eight stiffeners shall be used, equally spaced around the tube wall. The stiffener spacing shall not exceed 16 in. Alternative spacing requirements may be approved by the Owner based on sound engineering practices.

When stiffened fillet-welded tube-to-transverse-plate connections are used, the minimum thickness of the tube wall shall be 0.25 in.

The ratio of the stiffener thickness to the tube wall thicknesses shall not exceed 1.25:1.00.

Stiffeners having a transition radius shall not be used unless approved by the Owner based upon sound engineering practices.

14.4.4.2—Backing Rings

In full-penetration groove-welded tube-to-transverse-plate connections, the thickness of the backing ring shall not exceed 0.25 in. The height of the backing ring, when

performance, e.g., poor quality seal welds have led to fatigue cracking.

C14.4.4.1

As used herein, the term "sound engineering practices" is intended to mean an appropriate evaluation of analysis, testing, or acceptable field performance, singly or in combination as approved by the Owner.

In support structures employing larger-diameter and thicker tubes, optimized stiffened tube-to-transverse-plate fillet-welded connections can provide a cost-effective design compared to an increased transverse plate thickness. Parametric studies (Roy et al., 2011) demonstrated that the fatigue performance of a stiffened connection is a function of the geometric parameters of the connection: the tube thickness, the transverse plate thickness, the stiffener shape and size (thickness, height, and angle), and the number of stiffeners (or stiffener spacing). A large stiffener thickness relative to the tube can attract more stress into the stiffeners and can increase distortion of the tube. On the other hand, relatively thin stiffeners can reduce distortion of the tube but fail to sufficiently reduce the stress at the fillet weld and can cause fatigue cracking through the throat of the stiffener-to-transverse-plate weld. A ratio of stiffener thickness to tube thickness of 1.25 provides an optimum solution with equal likelihood of fatigue cracking at the stiffener termination and at the tube-to-transverse-plate weld.

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

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

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

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

C14.4.4.2

In full-penetration groove-welded tube-to-transverse-plate connections with the backing ring welded to the plate and the tube wall, fatigue cracking can occur both at welded to the tube at the top prior to performing ultrasonic inspection of the groove weld, shall be given by the following equation rounded to the nearest 1 in.

$$H = 2t\left(\tan\theta\right) + R \qquad (14.4.4.2\text{-}1)$$

where:

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

t = thickness of tube (in.),

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

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

For tube-to-transverse-plate connection employing an external collar, the tube thickness for the above equation shall include the thickness of the collar and the tube.

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

the groove-weld toe and the backing ring top-weld toe on the tube wall. Depending on the diameter and thickness of the tube and the height and thickness of the backing ring, the backing ring can participate in transferring forces from the tube to the transverse plate and can introduce variability in the fatigue performance of the connection. Providing a 2 in. × 0.25 in. backing ring limits this participation to a reasonable level in typical support structures. However, when the backing ring is welded to the tube at the top, this weld interferes with the ultrasonic inspection of the groove weld by allowing the sound wave to travel from the outside of the shaft through the weld into the backing ring. The sound wave then gets trapped in the backing ring and does not reach the groove weld. For a successful inspection, the weld at the top of the backing ring should be above the centerline of the probe. According to AWS D1.1 (AWS, 1996), the ultrasonic beam should bounce at least once to the area of inspection, which creates a full "V" signal. From experience, a shallow beam angle such as 70 degrees produces the best results. Thus, with thicker tubes, a 45-degree bevel, and a root gap, the probe placement must be higher and therefore the backing ring needs to be taller. The backing ring heights for different tube thicknesses are given in Table C14.4.4.2-1 for a root gap of 0.25 in. and the ultrasonic beam angle of 70 degrees.

Table C14.4.4.2-1—Required Backing Ring Height

t	H
Up to $^5/_{16}$ in.	2 in.
Greater than $^5/_{16}$ in. and up to $^1/_2$ in.	3 in.
Greater than $^1/_2$ in. and up to $^{11}/_{16}$ in.	4 in.

This requirement for backing ring height is not applicable if the top weld of the backing ring is made after the ultrasonic inspection of the groove weld, or if the backing ring is not welded at the top. In such cases, a maximum 2-in. high backing ring would be sufficient. When welded to the tube, the backing ring provides a redundant load path when the tube-to-transverse-plate groove weld develops fatigue cracking.

14.4.4.3—Mast-Arm-to-Pole Connections

Mast-arm-to-pole connections employing a fillet-welded gusseted box or a ring-stiffened box shall be detailed as shown in Figures C5.6.7-1 and C5.6.7-2. Other details may be used if approved by the Owner.

C14.4.4.3

Fillet-welded gusseted boxes or ring-stiffened boxes at the mast-arm-to-pole connections tested in the laboratory in full-size specimens (Roy et al., 2011) did not develop any fatigue cracking under both in-plane and out-of-plane loading. These connections were tested at various load levels and in some specimens were subjected to more than 40 million cycles. In all specimens, fatigue cracking occurred in other critical details in the structure, such as the tube-to-transverse-plate welds in the mast arm, the pole, and the hand holes. In-service fatigue cracking of these connections has been reported. Fatigue testing has shown the advantage of ring stiffeners that completely encircle a pole relative to a built-up box connection. For built-up box connections, it is recommended that the width of the box be at least the same as the diameter of the column (i.e., the sides of the box are tangent to the sides of the column). Ring-stiffened box connections are more fabrication-intensive and should be employed in geographic regions where support structures are expected to experience significant wind-induced oscillations. In other regions, gusseted-box connections are expected to provide satisfactory performance.

14.4.4.4—Circumferential Welded Splices

Full-penetration groove welds shall be used for pole and arm sections joined by circumferential welds.

C14.4.4.4

These circumferential welds are critical welds and should have proper inspection and controlled repair work. Inspection may be performed by nondestructive methods of radiography or ultrasonics or by destructive tests acceptable to the Owner. It is not intended that the inspection requirements be mandatory for small arms, such as luminaire arms, unless specified by the Owner.

14.4.4.5—Longitudinal Seam Welds

Longitudinal seam welds for pole and arm sections shall have 60 percent minimum penetration or fusion, except as follows:

- longitudinal seam welds within 6 in. of circumferential tube-to-tube-splice full penetration groove welds, tube-to-plate full penetration grove welds, or (pole-type) luminaire, traffic signal, and sign supports shall be full-penetration groove welds, and
- longitudinal seam welds, on the female section of telescopic (i.e., slip type) field splices of lighting (pole-type) luminaire, traffic signal, and sign supports, shall be full-penetration groove welds for a length equal to the minimum splice length plus 6 in.

C14.4.4.5

A 60 percent weld penetration implies that the depth that the fused cross section (not counting reinforcement) extends into the joint from the weld face is 60 percent of the material thickness. Where electric resistance welding (ERW) is used, welding takes place simultaneously throughout the thickness of the tube wall. In this case, 60 percent fusion is needed to comply with the requirement.

Full-penetration seam groove welds on the female section of telescopic field splices are difficult to achieve in small tube diameters (13 in. or less) because of inadequate access from the inside of the tube for back gouging and welding. In lieu of full-penetration groove welds in this area, external longitudinal reinforcement bars that are at least as thick as the tube to which they are being welded have been used as an acceptable method to ensure seam integrity. This alternate weld connection may be used if approved by the Owner. See Figure C14.4.4.5-1.

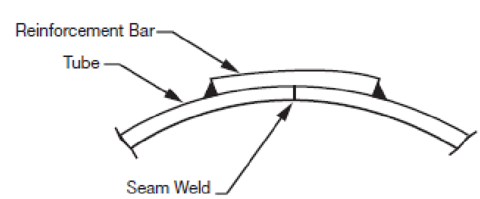

Figure C14.4.4.5-1 Seamweld Reinforcement Bar

If the seam weld is placed on the upper side of the arm (tension side under dead load), it can be a crack initiation point when the arm is subjected to fatigue loading. The best orientation of the seam weld is in the lower quadrant of the arm on the compression side of the member under dead load. The performance of tubes with spiral seam welds has not been evaluated by research. Their use is not currently recommended for fatigue sensitive components.

Seam welds for cantilever arm sections shall be located in the lower quadrant of the arm (compression side under dead load).

14.4.4.6—Tube-to-Transverse Plate Connection Welds

Welded tube-to-transverse-plate connections for high-level pole-type luminaire supports, overhead cantilever sign supports, overhead bridge sign supports with single-column end supports, common luminaire supports, and traffic signal supports shall use full-penetration groove welds or socket-type joint with two fillet welds.

Full-penetration groove welds shall be used on laminated sections (tube within tube).

All fillet welds and fillet reinforcements approximately transverse to the fatigue stress direction for tube-to-transverse plate connections shall be unequal leg welds, with the long leg of the fillet weld along the tube. The weld should be sized so that the termination of the longer weld leg contacts the tube surface at an angle of approximately 30 degrees.

For stiffened tube-to-transverse plate connections, the fillet weld connecting the stiffeners to the tube or the reinforcing fillets of a full penetration weld connecting the stiffeners to the tube shall be wrapped around the stiffener termination on the tube wall. For fillet welded connection of the stiffener to the tube, the wrapped around weld at the stiffener termination shall not be ground.

For full-penetration groove-welded tube-to-transverse plate connections externally reinforced with a sleeve, the fillet weld between the reinforcing sleeve and the tube shall be an unequal leg weld, with the long leg of the fillet weld along the tube. The weld should be sized so that the termination of the longer weld leg contacts the tube surface at an angle of approximately 30 degrees.

For full-penetration groove-welded tube-to-transverse-plate connections with backing rings, the backing ring should be welded to the tube wall in larger diameter tubes with adequate access through the base plate center hole so the weld quality can be adequately controlled. If this cannot be done, the backing ring should not be welded to the tube. Where the backing ring is welded to the tube, the weld shall be considered as a structural weld. In galvanized structures, if the backing ring is not welded to the plate or to the tube wall, all resulting gaps should be sealed by caulking after galvanizing to prevent ingress of moisture.

C14.4.4.6

This Article applies to poles and arms. Laminated structures have been used, but fatigue testing has not yet been performed on laminated-tube-to-base-plate connections.

Laboratory test results demonstrated that the fatigue strength of a fillet-welded tube-to-transverse plate connection can be improved by using an unequal leg fillet weld, compared to equal leg welds. Significant scatter was observed in the test results, however, where unequal leg fillet welds were used. This scatter in test results could be attributed to the variation in the fabricated weld geometry and particularly the weld toe angle from the specified nominal value. The overall profile of the weld should be specified to support the 30-degree requirement, such as specifying an uneven leg weld, rather than just specifying the termination angle. Deposition of single-pass welds satisfying the above geometry is limited to small welds (materials thickness of 11 gage or less). The Owner should consider the difficulties in specifying, fabricating, and inspecting the weld termination angle if setting any additional requirements on the termination angle itself and the tolerances allowed. In thin-walled tubular support structures, the welds act as tiny stiffeners, affecting the geometric stresses and the fatigue resistance of welded connections.

Full-penetration groove-welded tube-to-transverse-plate connections are usually fabricated with a backing ring. In galvanized structures, the backing ring is often welded to the plate and the tube wall to avoid ingress of acid in the gaps between the backing ring, the tube wall, and the plate during pickling in the galvanizing process. Any trapped acid in the gaps may cause crevice corrosion or hydrogen-related cracking when exposed to moisture in service.

In tubes having a diameter smaller than 16 in., it is difficult to ensure a quality weld between the tube and the backing ring at the top, where premature fatigue cracking from the toe of this weld on the tube wall may limit the fatigue resistance of the connection.

In stiffened tube-to-transverse-plate connections, tapered stiffeners with a wrapped-around weld at the terminus are cost-effective. The wrap-around weld serves as a seal weld for galvanizing.

These connections should be considered as transverse load-bearing attachments when determining the size of stiffener-to-transverse-plate fillet weld or partial-penetration groove weld required to prevent cracking through the weld throat (Detail 6.3 in Table 11.9.3.1-1).

14.4.4.7—Hand-Hole Welds and Other Structural Welds

Hand-hole welds and other welds attaching appurtenances to poles or arms shall be continuous in areas of high stress concentrations. Starting and stopping of the welding process shall be limited to areas of lowest stress. For reinforced hand holes and appurtenances in poles and arms, the best locations for starting and stopping the welding process are at points located on a longitudinal axis of symmetry of the tube coinciding with the axis of symmetry of the hand hole or appurtenance (e.g., the top and bottom of the hand-hole rim or appurtenance in vertical poles). Owners may approve other weld start and stop locations based on sound engineering practices.

Welds shall be either partial-penetration groove welds or fillet welds.

Where reinforcing bars are used around hand holes, the bars shall be made continuous using a full penetration butt weld, ground flush.

14.4.4.8—Weld Inspection

All welds shall be visually inspected (VT).

In addition to visual inspection, full-penetration welds for all structures that are designed according to the requirements of Section 11 shall be inspected by magnetic particle testing (MT) or ultrasonic testing (UT), based on the thinnest mating material:

Thickness < 0.25 in.	MT
Thickness ≥ 0.25 in.	UT

Full-penetration laminated tube-to-transverse-plate welds shall be inspected by MT, after the welding of each individual ply.

As an alternative, the Owner may require that full-penetration groove welds be inspected by radiographic testing (RT) or by destructive methods acceptable to the Owner. The full length of all full-penetration groove welds on all members of all structures shall be inspected, except for welds to arms less than or equal to 6 in. in diameter over their entire length.

Full-penetration groove welds associated with tube-to-base-plate and tube-to-arm-plate connections details having a constant amplitude fatigue threshold (CAFT) of 10 ksi or less shall be ultrasonically inspected for toe cracks after galvanizing. This inspection is in addition to the volumetric inspection required after fabrication.

C14.4.4.8

Ultrasonic inspection is normally prequalified for material thicknesses of 0.3125 in. or greater, but has been shown to be effective for thicknesses down to and including 0.25 in. Additional prequalification of inspection is required within this thinner thickness range per AWS Structural Welding Code—Steel Annex S “UT Examination of Welds by Alternative Techniques.” Note that AWS D1.1 Annex S (AWS, 2010) (Annex S) is not part of AWS D1.1, but is included by AWS for informational purposes. The UT techniques described in Annex S are proven methods that have been used in the shipbuilding and offshore oil/gas industries for many years. Reliable ultrasonic inspections of laminated tube-to-transverse-plate welds are not obtainable due to the transition of the wall thickness layers.

Radiography is generally an expensive method of nondestructive testing requiring special safety precautions, influencing manufacturing productivity, and extending lead times. Inspection requirements are not mandatory for welds to arms less than or equal to 6 in. in diameter over their entire length unless specified by the Owner.

Cracking after galvanizing at the toe of the weld connecting the tube to the transverse plate has been observed. These initial cracks reduce the fatigue performance of the connection. Ultrasonic testing of the connections using a small angle beam transducer can be used to detect the shallow toe cracks. Research has shown these toe cracks can be successfully repaired in the shop.

In addition to visual inspection, partial-penetration groove welds and fillet welds for all structures that are designed according to the requirements of Section 11 shall be inspected by MT or by destructive methods acceptable to the Owner. A required length of all partial-penetration groove welds and fillet welds shall be inspected on a random 25 percent of all structures as shown in Table 14.4.4.8-1, except for welds to arms less than or equal to 6 in. in diameter over their entire length. The structures to be inspected shall be selected by the Owner, if requested. If there are fewer than four structures, at least one structure shall be randomly selected.

With each welding repair, the micro-structure of the heated steel may change, potentially decreasing its strength and fatigue resistance.

Table 14.4.4.8-1—Required Inspection Length for Partial-Penetration Groove Welds and Fillet Welds

Location of Partial-Penetration Groove Welds and Fillet Welds	Required Inspection Length
Seams	4 inches in every 4 feet of length, starting from connection end
Tube-to-transverse plates	Full length
Hand holes and other appurtenances	Full length
Only one-time repair by welding of deficient welds is allowed without written permission of the Owner.	

14.4.5—Castings

14.4.5.1—Mild Steel Castings

Steel castings for use in highway bridge components shall conform to ASTM A781, Class 70 (Class 485), or ASTM A27, Class 70 or Grade 70-36, unless otherwise specified.

14.4.5.2—Chromium Alloy-Steel Castings

Chromium alloy-steel castings shall conform to ASTM A743. Grade CA 15 shall be furnished unless otherwise specified.

14.4.5.3—Iron Castings

Materials:

- Gray Iron Castings—Gray iron castings shall conform to ASTM A48, Class 30, unless otherwise specified in the contract documents.

- Ductile Iron Castings—Ductile iron castings shall conform to ASTM A536, Grade 60-40-18, unless otherwise specified in the contract documents. In addition to the specified test coupons, test specimens from parts integral with the castings, such as risers, shall be tested for castings with a weight (mass) more than 1.0 kip to determine that the required quality is obtained in the castings in the finished condition.

- Malleable Castings—Malleable castings shall conform to ASTM A47. Grade 35018 shall be furnished unless otherwise specified in the contract documents.

14.4.6—Fabrication Tolerances

Welded and seamless steel pipe members shall comply with the dimensional tolerances specified in ASTM A53.

Welded and seamless steel structural tubing members shall comply with the dimensional tolerances specified in ASTM A500, A501, or A595.

Plates and other shapes shall comply with the dimensional tolerances specified in ASTM A6.

The diameter of round tapered steel tubing members or the dimension across the flat of square, rectangular, octagonal, dodecagonal, and hexadecagonal straight or tapered steel tubing members shall not vary more than two percent from specified dimension.

C14.4.6

ASTM A53, A500, A501, and A595, which are currently listed in the Specifications, establish dimensional tolerances for steel pipe and round, tapered steel tubing members. ASTM A6 establishes only rolling tolerances for steel plates and shapes prior to fabrication.

This Article provides dimensional tolerances for straight or tapered steel tube members fabricated from plates. Some sign, signal, and luminaire manufacturers utilize tubes produced in their facilities. These tubular members should be consistent with industry standards for commercially produced pipe and tubing.

14.4.7—Protection

14.4.7.1—General

Steel structures shall be protected from the effects of corrosion by means such as galvanizing, metalizing, painting, or other methods approved by the Designer or Owner. Corrosion likely to occur as a result of entrapped moisture or other factors shall be eliminated or minimized by appropriate design and detailing. Positive means to drain moisture and condensation shall be provided unless the member is completely sealed.

14.4.7.2—Painted Structures

For painted structures, the materials and methods shall conform to the current Owner's requirements.

14.4.7.3—Galvanized Structures

Hot-dip galvanizing after fabrication shall conform to the requirements of ASTM A123.

It is preferable that tubular steel pole shafts to be galvanized have a silicon content equal to or less than 0.06 percent. Other components, such as base plates, should have silicon content controlled as required to prevent detrimental galvanizing effects. The placement of drainage and vent holes shall not adversely affect the strength requirements of a galvanized member. Damage to the coating shall be repaired after erection by a method approved by the Owner.

For structural bolts and other steel hardware, hot-dip galvanizing shall conform to the requirements of ASTM A153. Exposed parts of anchor bolts shall be zinc coated or otherwise suitably protected. The zinc coating shall

C14.4.7.3

Drainage and vent holes result in a reduction of a member's net section and cause stress risers, thereby reducing fatigue resistance. Holes shall be placed at noncritical locations where these reductions will not result in the member's strength being less than the required strength for maximum design loadings or fatigue.

Guidance on design and detailing to facilitate galvanizing may be found in "Design of Products to be Hot-Dip Galvanized after Fabrication" available from the American Galvanizers Association.

extend a minimum of 4 in. into the concrete. Steel anchorages located below grade and not encased in concrete shall require further corrosion protection in addition to galvanizing.

14.5—ALUMINUM STRUCTURES

14.5.1—Materials

This Article addresses the required material properties for aluminum structural supports for highway signs, luminaires, and traffic signals.

14.5.1.1—General

For principal materials used for structural members, minimum mechanical properties for non-welded aluminum alloys shall be as given in Table 6.4.1-1, and for welded aluminum alloys in Table 6.4.2-1. Applicable ASTM specifications are Designations B209, B210, B211, B221, B241, B247, B308, and B429.

C.14.5.1.1

For aluminum alloys not found in Tables 6.4.1-1 and 6.4.2-2, reference should be made to the *Aluminum Design Manual* (ADM)*, "*Specifications for Aluminum Structures*"* (Aluminum Association, 2000).

14.5.1.2—Storage of Materials

Material shall be stored out of contact with the ground, free from dirt, grease, and foreign matter and out of contact with dissimilar materials such as uncoated steel.

14.5.1.3—Minimum Thickness of Material

The minimum thickness of the material for primary structural members shall be 0.125 in. Aluminum supports for small roadside signs may be less than 0.125 in. in thickness. Abrasion blasting shall not be used on aluminum less than or equal to 0.125 in. thick.

C14.5.1.3

The minimum recommended thickness for welded aluminum is 0.125 in.

14.5.1.4—Dimensional Tolerances

The diameter of round extruded aluminum tubing members or the dimension across the flat of square, rectangular, octagonal, dodecagonal, and hexadecagonal straight or tapered aluminum tubing members shall comply with the applicable dimensional tolerances specified in the *Aluminum Standards and Data* (ASD) (Aluminum Association, 2009). For tapered round aluminum members, the diameter tolerance variation shall not vary more than two percent of the specified dimension.

14.5.1.5—Plates

14.5.1.5.1—Direction of Rolling

Unless otherwise specified in the contract documents, plates for main members and splice plates for flanges and main tension members only shall be cut and fabricated so that the primary direction of rolling is parallel to the direction of the main tensile and/or compressive stresses.

14.5.1.5.2—Plate Edges

Plates more than 0.5 in. thick carrying calculated stress shall not be sheared. All edges that have been cut by the arc process shall be planed to remove edge cracks. Oxygen cutting shall not be used. Re-entrant corners shall be filleted to a radius of 0.75 in. or more.

14.5.1.6—Bent Plates

14.5.1.6.1—General

Bend lines in unwelded, load-carrying, rolled aluminum plates shall be perpendicular to the direction of rolling.

Before bending, the corners of the plates shall be rounded to a radius of 0.0625 in. throughout the portion of the plate over which the bending is to occur.

14.5.1.6.2—Cold Bending

Cold bending shall not produce cracking. For 90-degree bends, bend radii measured to the concave face of the metal shall not be less than those listed in Table 14.5.1.6.2-1.

C14.5.1.6.2

Recommended bend radii for 90-degree cold bends for other alloys may be found in Table 7.6 of ASD.

Table 14.5.1.6.2-1—Minimum Bend Radii (in.) for 90-Degree Bends

	Plate Thickness (in.)			
	0.1875	0.25	0.375	0.50
Alloy	**Minimum Bend Radius, in.**			
5083-H321	0.28	0.35	0.79	1.25
5086-H116	0.28	0.47	0.98	1.42
5456-H116	0.38	0.59	1.18	1.65
6061-T6	0.55	0.83	1.77	2.36

14.5.1.7—Identification of Aluminum Alloys during Fabrication

The Contractor shall issue cutting instructions and mark individual pieces so as to be able to identify the material used for each piece. Metal stamping marks, scribe lines, and center punch marks shall not be used where they will remain on fabricated material.

Material furnished from stock shall be identified by lot and mill test reports.

Upon request by the Engineer, the Contractor shall furnish an affidavit certifying that the identification of pieces has been maintained in accordance with this specification.

C14.5.1.7

ASD gives color codes for additional alloys and other information on identification markings used by aluminum producers.

During fabrication prior to assembly, each piece shall clearly show its material specification. Writing the material specification number on the piece or by using the identification color codes shown in Table C14.5.1.7-1 shall be taken as compliance with this provision.

Aluminum alloys not listed in Table C14.5.1.7-1 shall be marked with colors listed in ASD.

Any piece which will be subject to fabrication that might obscure its identification prior to assembly shall have a substantial tag affixed showing the material specification number.

Table C14.5.1.7-1—Identification Color Codes

Alloy	Color
5083	Red and Gray
5086	Red and Orange
6061	Blue
6060	Yellow and Green

14.5.2—Bolted Connections

14.5.2.1—Bolted Connections and Anchor Bolts

Design of bolted connections shall conform to the *Aluminum Design Manual* (ADM) Chapters B and J (Aluminum Association, 2010).

Fasteners shall not be considered to share load in combinations with welds.

When the line of action of the resultant force does not coincide with the center of gravity of the fastener or weld group, the effect of the eccentricity shall be considered.

Design and installation of steel anchor bolts for aluminum structures shall be in accordance with Article 5.16.

14.5.2.2—Anchor Bolts

Anchor bolts for aluminum structures shall follow guidelines in Article 14.4.1.2 for steel structures.

14.5.2.3—Bolt Material

Aluminum bolt material shall meet ASTM F486 and be 6061-T6 or 7075-T3. Nuts shall meet ASTM F467. Nuts for bolts larger than $^1/_4$ inch shall be 6061-T6 or 6262-T9. Flat washers shall be Alclad 2024-T4. Spring lock washers shall be 7075-T6.

Carbon steel bolts, nuts, and washers shall be galvanized by hot dip meeting ASTM A153 or by mechanical means meeting ASTM B695. Galvanized fasteners and nuts shall be lubricated in accordance with ASTM A563. A490 bolts shall not be used.

Stainless steel bolts, nuts, and washers shall be 300 series. Bolts shall meet ASTM F593, A193, or A320. Nuts shall meet ASTM F594 or A194.

C14.5.2.3

ASTM A325 hot-dip galvanized high-strength bolts are normally used for structural connections in aluminum structures.

ASTM A490 high-strength steel bolts are not used in aluminum structures because they may become embrittled by galvanizing. Galvanizing is typically required to prevent galvanic corrosion of aluminum in contact with steel.

14.5.2.4—Holes and Slots

The nominal diameter of a bolt hole shall not be more than 0.0625 in. greater than the nominal diameter of the fastener unless slip critical joints are used.

C14.5.2.4

The width of slots for bolted connections shall not be more than 0.0625 in. wider than the nominal diameter of the bolt. If the nominal length of the slot is more than 2.5*D* (*D* = nominal bolt diameter) or the edge distance is less than 2*D*, the edge distance perpendicular to the length of the slot and slot length shall be sized to avoid overstressing the material along the slot. Bearing load connections should be made so that the action of the load is perpendicular to the slot. Slip critical connections can be made with the load action at any orientation to the slot.

To avoid overstressing the material along the slot the designer, as a minimum, should check bearing, rupture, and beam action deformation on the edge side of the slot when the force action is perpendicular to the slot.

14.5.2.5—Minimum Spacing and Edge Distance of Bolts

The distance between bolt centers shall not be less than 2.5*D* (*D* = nominal bolt diameter).

The distance from the center of a bolt to the edge of a part shall not be less than 1.5*D*.

See Article 6.9 for bearing design strength.

14.5.3—Welded Connections and Fit Up

14.5.3.1—Weld Inspection

All welds shall be visually inspected (VT). In addition to visual inspection, full-penetration welds for all structures that are designed according to the requirements of Section 11 shall be inspected by ultrasonic testing (UT) or dye penetrant testing (PT), based on the thinnest mating material:

Thickness < 0.25 in. PT

Thickness ≥ 0.25 in. UT

As an alternative, the Owner may require that full-penetration groove welds be inspected by destructive methods acceptable to the Owner. The full length of all full-penetration groove welds on all members of all structures shall be inspected, except for welds to arms less than or equal to 6 in. in diameter over their entire length.

In addition to visual inspection, partial-penetration groove welds and fillet welds for all structures that are designed according to the requirements of Section 11 shall be inspected by dye penetrant testing or by destructive methods acceptable to the Owner. A required length of all partial-penetration groove welds and fillet welds shall be inspected on a random 25 percent of all structures, except for welds to arms less than or equal to 6 in. in diameter over their entire length. The structures to be inspected shall be selected by the Owner, if requested. If there are fewer than four structures, at least one structure shall be randomly selected.

C14.5.3.1

There are no prequalified weld inspections in AWS D1.2 (AWS, 2008). When UT, RT, or other weld inspection is required by the contract documents, the extent of testing, the procedure, and the acceptance criteria shall be specified therein.

14.5.3.2—Welded Connections

Surfaces and edges to be welded shall be smooth, uniform, clean, and free of defects which would adversely affect the quality of the weld.

Brackets, clips, shipping devices, or other material not required by the contract documents shall not be welded or tacked to any member unless specified in the contract documents and approved by the Engineer.

14.5.4—Fit of Stiffeners

End bearing stiffeners and stiffeners intended as supports for concentrated loads shall bear fully on the component to which they transmit load or from which they receive load. Stiffeners not intended to support concentrated loads shall have a tight fit unless specified otherwise.

C14.5.4

Full bearing may be obtained by milling, grinding, or in the case of compression regions, welding.

14.5.5—Abutting Joints

Abutting ends of compression members of trusses and posts or columns shall be milled or saw-cut to give a square joint and uniform bearing. At other joints, the distance between adjacent members shall not exceed 0.375 in.

14.5.6—Facing of Bearing Surfaces

The surface finish of bearing, base plates, and other bearing surfaces that come in contact with each other or concrete shall meet ANSI B46.1, Surface Roughness, Waviness, and Lay, Part 1 (see Table 14.5.6-1).

Table 14.5.6-1—ANSI Surface Roughness Requirements

Bearing Surfaces	Surface Finish
Milled ends of compression members, milled or ground ends of stiffeners and fillers	ANSI 12.5 μm (500 μin.) (RMS)
Bridge rollers and rockers fillers	ANSI 6.3 μm (250 μin.) (RMS)
Pins and pin-hole fillers	ANSI 3.2 μm (125 μin.) (RMS)
Sliding bearings fillers	ANSI 3.2 μm (125 μin.) (RMS)

14.5.7—Straightening Material

When permitted by the Engineer, straightening of plates, angles, other shapes, and built-up members shall be done by methods that will not produce fracture or other damage to the metal. Distorted members shall be straightened by mechanical means or by heat straightening. Heat straightening of non-heat-treatable alloys and of heat-treatable alloys after heat treatment shall be done only under controlled procedures and with the approval of the Engineer. Heat straightening shall conform to ANSI/AWSD1.2 .

C14.5.7

Aluminum may be heated for short periods of time to temperatures up to 400 degrees without significant loss of strength. Temperature and duration limits are given in ANSI/AWS D1.2 Table 3.2 (AWS, 2008). Heating aluminum alloys with magnesium contents greater than three percent, which includes 5083, 5086, and 5456, to temperatures between 150°F and 450°F will also result in decreased resistance to exfoliation corrosion.

14.5.8—Holes

14.5.8.1—General

Holes shall be:

- drilled to the nominal hole diameter, or
- subpunched to a diameter smaller than the nominal hole diameter and then reamed to the nominal hole diameter, or
- subdrilled to a diameter smaller than the nominal hole diameter and then reamed to the nominal hole diameter.

The difference between a subpunched hole diameter and the nominal hole diameter shall be at least one-fourth the thickness of the part and in no case less than 0.03125 in.

C14.5.8.1

Punching holes to the nominal diameter is not preferred for aluminum parts subject to fatigue.

14.5.8.2—Reamed or Drilled Holes

Reamed or drilled holes shall be cylindrical, perpendicular to the members. Burrs shall be removed. The diameter of holes produced by drilling or reaming shall not be more than 0.03125 in. greater than the nominal diameter of the hole.

14.5.8.3—Accuracy of Hole Groups

14.5.8.3.1—Accuracy before Reaming

After assembling, but before any reaming, the holes in any contiguous group shall allow a cylindrical pin that is:

- 0.125 in. smaller in diameter than the nominal diameter of the hole to enter at least 75 percent of the holes perpendicular to the face of the member without drifting
- 0.1875 in. smaller in diameter than the nominal diameter of the hole to enter every hole perpendicular to the face of the member without drifting

14.5.8.3.2—Accuracy After Reaming

After reaming or drilling, 85 percent of the holes in any contiguous group shall show no offset greater than 0.03125 in. between parts.

14.5.8.4—Locating Holes

Holes shall be:

- subpunched or subdrilled in unassembled parts and reamed when the parts are assembled,

- drilled to the nominal diameter using a template or numerically controlled drilling, or
- drilled to the nominal diameter while the parts are assembled.

Parts may be assembled in the shop or in the field for the fabrication of holes.

14.5.9—Annealing and Stress Relieving

Holes shall be fabricated after all heat treatment has been completed. Aluminum structural members shall not receive heat treatment after welding.

14.5.10—Castings

Aluminum-alloy sand castings shall conform to ASTM B/B26. Aluminum-alloy permanent mold castings shall conform to ASTM B108.

14.5.11—Protection

Structures of the aluminum alloys covered by these Specifications are not ordinarily painted. Surfaces shall be painted where the aluminum alloy parts are in contact with or fastened to steel members or other dissimilar materials, the structures are to be exposed to extremely corrosive conditions, or the Owner has requested it be done for reason of appearance.

Preparation, cleaning, and painting are covered in the following Articles. Treatment and painting of the structure in accordance with United States Military Specification MIL-T-704 is also acceptable.

C14.5.11

The reason that most aluminum structures are not painted is that aluminum surfaces develop a thin, tough oxide film that protects the surface against further oxidation. If the surface is scraped so that the oxide film is removed, a new film is formed immediately unless oxygen is kept from the surface. The alloying ingredients that give aluminum particular properties, such as extra-high strength, affect resistance to corrosion. Painting is not needed for the medium-strength alloys in general structural use in atmospheric exposure.

14.5.11.1—Galvanic Corrosion (Contact with Dissimilar Materials)

Where the aluminum alloy parts are in contact with or fastened to steel members or other dissimilar materials, the aluminum shall be kept from direct contact with the steel or other material by painting as follows.

Steel surfaces to be placed in contact with aluminum shall be painted with good-quality non-lead-containing priming paint, such as zinc molybdate alkyd-type primer in accordance with Federal Specification TT-P-645B. This is to be followed by two coats of paint consisting of 2 lb of aluminum paste pigment (ASTM D 962-88, Type 2, Class B) per gallon of varnish meeting Federal Specification TT-V-81, Type II, or equivalent. Where severe corrosion conditions are expected, additional protection can be obtained by applying a sealant capable of excluding moisture during prolonged service to the faying surfaces in addition to the zinc molybdate alkyd-type primer. Aluminized, hot-dip galvanized, or electro-galvanized steel placed in contact with aluminum need not be painted.

C14.5.11.1

Galvanic corrosion can occur when another metal, such as steel, is coupled to aluminum in the presence of an electrolyte. An aluminum part bolted to a steel structure with moisture allowed in the faying surface or an aluminum part in concrete and coupled to the steel reinforcement are examples. The aluminum parts may act as an anode and be sacrificed in time. The attack can be prevented by isolating the two materials from each other.

Materials such as elastomeric spacers have also been used to keep aluminum alloy parts from direct contact with steel or other dissimilar materials.

Stainless steel (300 series) placed in contact with aluminum need not be painted except in high chloride-containing environments.

Aluminum shall not be placed in direct contact with porous materials that may absorb water and cause corrosion. When such contacts cannot be avoided, an insulating barrier between the aluminum and the porous material shall be installed. Before installation, the aluminum surfaces shall be given a heavy coat of alkali-resistant bituminous primer or other coating having equivalent protection to provide this insulating barrier. Aluminum in contact with concrete or masonry shall be similarly protected in cases where moisture is present and corrodents will be trapped between the surfaces.

Aluminum surfaces to be embedded in concrete ordinarily need not be painted, unless corrosive components are added to the concrete or unless the assembly is subjected for extended periods to extremely corrosive conditions. In such cases, aluminum surfaces shall be protected by one of the following methods:

(a) given one coat of suitable quality paint, such as zinc molybdate primer conforming to Federal Specification No. TT-P-645B or its equivalent;

(b) given a heavy coating of alkali-resistant bituminous paint; or

(c) wrapped with a suitable plastic tape applied in such a manner as to provide adequate protection at the overlaps.

Aluminum shall not be embedded in concrete to which corrosive components such as chlorides have added if the aluminum will be electrically connected to steel.

Prepainted aluminum generally does not need additional painting, even in contact with other materials such as wood, concrete, or steel. Under extremely corrosive conditions, additional protection shall be provided as described in the preceding paragraphs.

14.5.11.2—Overall Painting

Paintings or other coatings shall be in accordance with the Owner's requirements.

C14.5.11.2

Structures of the alloys covered by these Specifications either are not ordinarily painted for surface protection or are made of prepainted aluminum components.

14.5.11.3—Anodizing

An anodized finish may be provided, if specified by the Owner.

C14.5.11.3

Anodizing is an electro-chemical process that results in a colored aluminum oxide layer on the pole surface. The Owner should be aware that anodized finishes may result in color variations between extrusions, coatings, and weldments.

14.6—PRESTRESSED CONCRETE STRUCTURES

This Article addresses the required fabrication and material properties for prestressed concrete structural supports for highway signs, luminaires, and traffic signals.

14.6.1—General

Concrete structures shall be designed and fabricated to provide protection of the prestressing tendons and reinforcing steel against corrosion throughout the life of the structure. Concrete materials and pole fabrication shall be in accordance with Section 8 of the *AASHTO LRFD Bridge Construction Specifications* (2010) and Owner requirements. Aggregates from sources known to have experienced alkali-silica reactions shall be prohibited. Portland cement with low alkali content, i.e. less than 0.6 percent, should be specified to ensure long-term durability. Additional requirements may be specified for structures in highly corrosive environments including the use of special concrete additives, coatings, epoxy-coated strands, or an increase in concrete cover.

14.6.2—Concrete Cover

The minimum clear concrete cover for prestressed and non-prestressed reinforcement shall be as follows:

- $^3/_4$ in. for centrifugally cast poles, and
- 1 in. for static cast poles.

Cover may be reduced to $^1/_2$ in. for street lighting poles, subject to Owner approval. For prestressed concrete structures exposed to severe corrosive environments, the minimum cover shall be increased by 50 percent.

C14.6.2

Severe corrosive environments include exposure to deicing salt, water or airborne sea salt, and airborne chemicals in heavy industrial areas. The centrifugal casting process results in a highly consolidated concrete that is denser than normal concrete, and hence the reduction in cover requirements for centrifugally cast (spun) poles.

Coulomb testing of concrete mixes may be required by the Owner. Coulomb testing should conform to AASHTO T 277.

14.6.3—Fabrication Tolerances

The following manufacturing tolerances shall apply:

- Length shall vary by no more than 2 in., or 1 in. plus $^1/_4$ in. per 10 ft in length, whichever is greater.
- Outside diameter shall vary by no more than $^1/_4$ in. for spun poles. For static cast poles, tolerance shall not vary in cross-section dimensions for poles less than 24 in., $\pm^3/_8$ in.; 24 in. to 36 in., $\pm^1/_2$ in.; over 36 in., $\pm^5/_8$ in.,
- Wall thickness shall be not less than minus 12 percent of the design thickness or $^1/_4$ in., whichever is greater.
- Deviation from longitudinal axis shall vary no more than $^1/_4$ in. per 10 ft of length, applicable for the entire length or any segment thereof.

- Mass shall vary no more than 10 percent of the design mass.
- End squareness shall vary no more than $^1/_2$ in. per 1 ft of diameter. For poles with base plates, more stringent requirements shall be specified.
- Longitudinal reinforcement shall vary no more than $^1/_4$ in. for individual elements, and no more than $^1/_8$ in. for the centroid of a group.

Circumferential wire spacing shall be a maximum of 4 in., except at the ends (measured from either the top or bottom to a distance of 1 ft), where the maximum spacing shall be 1 in. Circumferential wire shall be within $1^1/_2$ in. of its specified location, except at the ends (measured from either top or bottom to a distance of 1 ft) where the spacing location shall be within $\pm^1/_4$ in. The number of spirals of cold-drawn circumferential wire along any 5 ft of length shall not be less than required by design.

14.6.4—Inspection

The quality of materials, the process of manufacture, and the finished poles shall be subjected to inspection and approval by the Owner, the Designer, or both.

Inspection records shall include the:

- quality and proportions of concrete materials,
- strength of the concrete,
- placement of reinforcements,
- mixing, placing, and curing of concrete and tensioning prestressing tendons.

14.6.5—Protective Systems

This Article addresses the protective systems for prestressed concrete structural supports for highway signs, luminaires, and traffic signals.

14.6.5.1—Concrete Finish

The manufacturer of prestressed concrete members should notify the Engineer of the following surface conditions:

- all honeycombs deep enough to expose prestressing steel,
- defects that may affect bond length or transfer length, and
- any area that the manufacturer believes to be detrimental.

The Engineer responsible for inspection at the manufacturing plant will consult with the manufacturer and Owner to determine the corrective action required to repair the member. Prestressed members that cannot be effectively repaired will be rejected. The Engineer is responsible for final acceptance.

14.6.5.2—Surface Treatment

When plans call for surface paint, stain, or other coatings, Owner requirements and the coating manufacturer's recommendations for surface preparation shall be followed.

C14.6.5.2

A variety of surface coatings are available and can aid in preservation of the concrete member by preventing degradation of the concrete material and also by preventing corrosion of the internal steel reinforcement. A common cause of coating failures is lack of surface preparation of the concrete surface prior to the coating application. The manufacturer's recommendations should be followed closely and may include requirements on cure time, relative humidity of the concrete, surface texture, removal of membrane-forming compounds used for curing, and other requirements.

14.7—COMPOSITE (FIBER-REINFORCED POLYMER) STRUCTURES

The provisions of this Article apply only to fiberglass-reinforced polymer composites (FRP). Other structural composites may be used if approved by the Owner.

C14.7

Applications of fiber-reinforced polymers are expanding as the use of these materials increases. Although reinforced plastic composites have been successfully employed in major structural applications, the use of the material is relatively new and there is lack of information on its behavior and design. This Article provides guidelines for the design of structural supports manufactured from fiberglass-reinforced composites.

14.7.1—Materials

This Article addresses the required material properties for composite (fiberglass reinforced polymer) structural supports for highway signs, luminaires, and traffic signals.

FRP shall be composed of two principal constituents: resin and glass fiber reinforcement.

C14.7.1

The Specifications cover primarily FRP composites, which are the most widely used composites for civil engineering applications requiring structural reliability. FRP is composed of two principal constituents, namely polymer resin and glass fiber reinforcement. The FRP composite possesses superior properties not available to each constituent alone. The glass fiber reinforcement, which is significantly higher in strength than the polymer resin, constitutes the main load-carrying element of the composite. The polymer resin undergoes large deformations while the load is being transmitted to the glass fibers. From a practical standpoint, FRP may be considered an elastic material, which exhibits a stress-strain behavior that is fairly linear up to failure. The material does not yield or exhibit a permanent set due to transient overloads.

14.7.1.1—Polymer Resins

Unsaturated polymer shall be used as the principal matrix resin.

C14.7.1.1

The unsaturated polymer resin is a cross-linked thermosetting plastic that is hard and brittle and fractures on impact. However, when reinforced with high-strength fibers, it develops reliable structural qualities.

14.7.1.2—Glass Fiber Reinforcement

Reinforcement for FRP composites shall be E-, C-, or S-glass. Glass fiber may take any of the following forms:

C14.7.1.2

The high-strength glass fibers reinforce the polymer resin and provide the strength and stiffness required for structural purposes. Three types of glass are commonly

- continuous strands (i.e., rovings and yams),
- mats,
- chopped strands,
- milled fibers, or
- fabrics.

used as fiber reinforcement: E-, C-, and S-glass. E-glass, or electrical grade, is for general-purpose structural uses as well as for good heat resistance and high electrical properties. C-glass, or chemical grade, is best used for resistance to chemical corrosion. S-glass, or high silica, is a special glass for high heat resistance that also has enhanced structural properties. Of the three types, E-glass is the most common in engineering applications.

14.7.2—Connections

Connections shall utilize either bolted connections or threaded connections that are integral to the member. If bolted connections are used, all fasteners and associated hardware shall be galvanized or stainless steel.

Other connection types may be used if approved by the Owner. Computations, laboratory testing, or both may be required at the discretion of the Engineer to demonstrate that factored loads will not exceed the member's nominal resistance. The manufacturer's recommendations on fabrication and construction should be followed.

C14.7.2

Many proprietary systems are available from various manufacturers and may or may not have undergone rigorous laboratory testing as part of the manufacturers' own product development program. Experience with the short- and long-term service of FRP members is somewhat limited. Engineering judgment should be exercised when considering FRP connection elements.

14.7.3—Fabrication Tolerances

FRP members shall be fabricated to conform to the dimensional tolerances given in ASTM D3917.

C14.7.3

ASTM D3917 defines dimensional tolerances for standard pultruded shapes for thermosetting fiberglass reinforced polymers. The tolerances define dimensional criteria for cross sections, width and diameter, surface flatness, angularity, and camber.

Custom shapes may have dimensional requirements that differ from those defined in ASTM D3917.

14.7.4—Manufacturing Methods

FRP members shall be manufactured by generally accepted methods that ensure high quality, good performance, and reliable mechanical properties of the members produced. (See Table 8.5-1.)

C14.7.4

Various processes are employed for the manufacturing of FRP. Manufacturing processes that are commonly used for structural supports include filament winding, pultrusion, and centrifugal casting.

From a structural standpoint, the manufacturing process can markedly influence the structural properties of the material. Other factors that affect the properties of the FRP laminate are the orientation of the glass fibers and the fiber content. Aligning fibers in a single direction provides high stiffness and strength parallel to the fibers, but properties in the perpendicular direction approach those of the plastic matrix. Typically, in pole structures, the glass reinforcement is primarily oriented in the longitudinal direction with minimum reinforcement in the transverse direction. The glass-to-resin ratio (by weight) is usually used as a measure of the fiber content. The strength and stiffness properties of FRP generally increase with increasing the glass-to-resin ratio. Typical mechanical properties of FRP laminates are shown in Table 8.5-1.

Three manufacturing processes that are commonly used for structural supports are filament winding, pultrusion, and centrifugal casting. Filament winding is the oldest and most common method of producing FRP poles. A filament-winding machine is used where glass fiber strands are impregnated with polymer resin and continuously wound onto a tapered mandrel. The number of windings and the angle at which the glass fiber strands are wound on the mandrel are controlled and set according to design. The laminate is then cured, sometimes with the assistance of an external heat source. On completion, the mandrel is removed.

Pultrusion is a continuous molding process where selected reinforcements are fed continuously in pre-determined amounts and preplanned layering from multiple creels through a resin bath. The resin-impregnated reinforcements are pulled through a die that determines the sectional geometry of the product and controls reinforcement and resin content. Resin cure is initiated in the heated section of the die. The product is drawn through the die by a puller mechanism and is cut to the desired length. Pultrusion is appropriate for any shape that may be extruded, but it is limited to prismatic members.

Centrifugal casting is a method most suited for cylindrical shapes such as pipes, poles, and tubing. In this process, glass fabric in a predetermined amount and configuration is placed in a hollow steel mold. The mold is rotated (spun) at high speeds during which resin is injected and distributed uniformly throughout the reinforcement. Centrifugal forces further distribute and compact the resin and reinforcement against the wall of the rotating mold. Curing of the product is accelerated through the application of an external heat source.

14.7.5—Testing

Full-scale structural testing shall be used to verify the strength and deflection of FRP members. The bending test criteria for FRP poles are summarized in Article 8.6.2.1. Other tests that may be required are given in Article 8.6.2.2.

C14.7.5

Because FRP poles are usually round tubular tapered members whose performance is dependent upon the composition of the material and the manufacturing procedure, testing is required to determine the bending and torsional strength, and the weathering resistance of FRP poles. Cracking and early failure can occur at handholes during bending of poles, and early pole attachment failures can occur at cast shoe bases (i.e., pole-to-tube junction). These items can also be checked through testing.

14.7.6—Other Testing

The following additional tests shall be performed if required by the Owner:

- torsional strength per ASTM D4923
- fatigue strength per ASTM D4923
- weathering resistance per ASTM G154
- adhesion of coatings

- color change from UV exposure
- fatigue strength of connections

14.7.7—Determination of Mechanical Properties of FRP

The strength of FRP laminates shall be determined by testing using flat sheet samples in accordance with the ASTM standards listed in Table 8.6.3.1-1. Samples shall be manufactured in the same manner as that proposed for the structural member. For structural members where the fiber orientation changes along the member, sheet samples shall be taken at locations of critical stresses.

C14.7.7

Because the mechanical properties of the FRP material could vary significantly depending on the particular composition and the manufacturing process, the test samples must be representative of the actual conditions in the final product. The proposed resistance factors shown in Table 8.6.3.1-1 are maximum values based on common industry practice. Other values may be used when agreed upon by the Owner and the manufacturer.

14.7.8—Minimum Protection for FRP Members

FRP members shall be protected from UV radiation to minimize degradation of the structural properties of the member. FRP members as fabricated shall not show evidence of exposed fibers, cracks, crazes, or checks on the member surface. UV protection of FRP members shall be provided by using one or more of the following methods:

- surface veil,
- urethane coating, and
- UV stabilizers.

Other UV protection methods may be used if proven effective and agreed upon by the Owner and manufacturer.

C14.7.8

Ultraviolet rays and heat from solar radiation degrade the natural molecular structure of FRP. Other weathering elements such as industrial pollutants or salt spray can accelerate the degradation due to UV radiation. The results of exposing FRP to any of these conditions can be discoloration, embrittlement, and loss of mechanical strength, electrical insulation, and resistance properties. Chemical stabilizers, fillers, and weather-resistant paints or coatings should be used to provide the necessary protection. The service life of coatings is dependent on the coating's quality, adhesion, and thickness.

14.8—WOOD STRUCTURES

14.8.1—Materials

This Article addresses the required material properties for wood structural supports for highway signs, luminaires, and traffic signals.

The provisions of this Article apply only to cantilevered wood posts and poles. The provisions for wood posts and poles are generally based on the National Design Specification for Wood Construction (NDS, 2012), including the Design Values for Wood Construction (NDS Supplement, 2012), except as modified herein.

C14.8.1

The design provisions of this Article are applicable to common post and pole usages given in Article 9.4. Additional design provisions given in the NDS may be required for other member types or usages.

14.8.1.1—Wood Products

This Article covers the following wood products:

- wood posts, and
- round timber poles.

C14.8.1.1

Posts and poles are the most commonly used wood products for structural supports for highway signs, luminaires, and traffic signals. This Article is limited to the coverage of wood posts from visually graded lumber and round timber poles. Visually graded lumber is a type of structural lumber graded by visual examination based on certain rules established by the grading agency. In general, posts are used to support small structures such as roadside signs. Round timber poles are used as vertical supports for street lighting or strain poles for temporary span-wire configurations. Engineered wood products such as laminated veneer lumber may be used for structural supports such as posts. Design of engineered wood products, however, should be based on technical information provided by the manufacturer and approved by the Owner because the basic design values could vary for products from different manufacturers.

14.8.2—Connections

Mechanical connections and their installation shall conform to the requirements of the NDS.

Connection hardware shall be hot-dip galvanized or stainless steel.

C14.8.2

Components at mechanical connections, including the wood members, connecting elements, and fasteners, should be proportioned so that the design strength equals or exceeds the required strength for the loads acting on the structure. The strength of the connected wood components should be evaluated considering the net section, eccentricity, shear, tension perpendicular to grain, and other factors that may reduce component strength.

14.8.3—Minimum Protection for Wood Products

Wood products shall be protected from biological attack of wood-destroying organisms, such as decay, fungi, insects, and marine borers. Minimum accepted preservative treatments for wood posts and poles shall be in accordance with Articles 14.8.3.1 and 14.8.3.2, respectively.

All Preservatives shall be registered with the U.S. Environmental Protection Agency.

C14.8.3

Preservative treatments are those that guard wood against decay, insects, and marine borers. The three basic types of pressure preservatives are:

- creosote and creosote solutions,
- oil-borne treatments (e.g., preservatives dissolved in hydrocarbon solvents), and
- waterborne preservatives.

There are a number of variations on each of these categories. The choice of the preservative treatment and the required retention are specified by the standards from the American Wood Protection Association (AWPA) or by the evaluation reports issued by the International Code Council Evaluation Service (ICC-ES).

The use and disposal of some wood preservatives may be controlled or restricted by various local, state, or governmental agencies.

14.8.3.1—Preservative Treatment for Posts

Posts shall be pressure treated in accordance with the American Wood Protection Association (AWPA) U1-13, Commodity Specification: B. Posts or AASHTO M 133.

14.8.3.2—Preservative Treatment for Poles

Round poles shall be pressure treated in accordance with APWA U1-11, Commodity Specification: D. Poles or AASHTO M 133.

14.9—REFERENCES

AASHTO. 2010. *AASHTO LRFD Bridge Construction Specifications.* American Association of State Highway and Transportation Officials, Washington, DC.

AASHTO. 2012. *Standard Specification for Preservatives and Pressure Treatment Processes for Timber*, AASHTO M 133. American Association of State Highway and Transportation Officials, Washington, DC.

AASHTO. 2014. *LRFD Bridge Design Specifications*, 7th Edition. American Association of State Highway and Transportation Officials, Washington, DC.

ACI. 1995. *Code Requirements for Nuclear Safety-Related Concrete Structures and Commentary*, Appendix B, "Steel Embedments," ACI 349-06. American Concrete Institute, Farmington Hills, MI.

AGA. 2012. *Design of Products to Be Hot-Dip Galvanized after Fabrication*. American Galvanizer's Association, Centennial, CO.

Aluminum Association. 2010. *Aluminum Design Manual*, Specifications for Aluminum Structures. Aluminum Association, Washington, DC.

Aluminum Association. 2013. *Aluminum Standards and Data*. Aluminum Association, Washington, DC.

ASME. 2009. S*urface Texture, Surface Roughness, Waviness and Lay*, ANSI/ASME B46.1-2009. American Society of Mechanical Engineers, New York, NY.

ASTM. 1999. "Standard Specification for Ferritic Malleable Iron Castings," ASTM A47/A47M-99, *Annual Book of ASTM Standards*. American Society for Testing Materials, West Conshohocken, PA.

ASTM. 2003. "Standard Specification for Gray Iron Castings," A48/A48M-03, *Annual Book of ASTM Standards*. American Society for Testing Materials, West Conshohocken, PA.

ASTM. 2006. "Standard Practice for Operating Light and Water Exposure Apparatus (Fluorescent UV Condensation Type) for Exposure of Non-Metallic Materials," G154-06, *Annual Book of ASTM Standards*. American Society for Testing Materials, West Conshohocken, PA.

ASTM. 2008. "Standard Specification for Aluminum Powder and Paste Pigments for Paints," D962-81(2008), *Annual Book of ASTM Standards*. American Society for Testing Materials, West Conshohocken, PA.

ASTM. 2009. "Standard Specification for Zinc Coating (Hot-Dip) on Iron and Steel Hardware," A153/A153M-09, *Annual Book of ASTM Standards*. American Society for Testing Materials, West Conshohocken, PA.

ASTM. 2009. "Standard Specification for Aluminum and Aluminum-Alloy Die Forgings, Hand Forgings, and Rolled Ring Forgings," B247-09, *Annual Book of ASTM Standards*. American Society for Testing Materials, West Conshohocken, PA.

ASTM. 2009. "Standard Specification for Coatings of Zinc Mechanically Deposited on Iron and Steel," B695-04(2009), *Annual Book of ASTM Standards*. American Society for Testing Materials, West Conshohocken, PA.

ASTM. 2009. "Standard Specification for Reinforced Thermosetting Plastic Poles," D4923-92 (Withdrawn 2010), *Annual Book of ASTM Standards*. American Society for Testing Materials, West Conshohocken, PA.

ASTM. 2009. "Standard Specification for Stainless Steel Nuts," F594-09, *Annual Book of ASTM Standards*. American Society for Testing Materials, West Conshohocken, PA.

ASTM. 2010. "Standard Specification for Aluminum-Alloy 6061-T6 Standard Structural Profiles," B308/B308M-10, *Annual Book of ASTM Standards*. American Society for Testing Materials, West Conshohocken, PA.

ASTM. 2010. "Standard Specification for Aluminum-Alloy Extruded Structural Pipe and Tube," B429/B429M-10, *Annual Book of ASTM Standards*. American Society for Testing Materials, West Conshohocken, PA.

ASTM. 2011. "Standard Practice for Preparation of Use and Care Booklets for Vacuum Cleaners," F486-01(2011), *Annual Book of ASTM Standards*. American Society for Testing Materials, West Conshohocken, PA.

ASTM. 2012. "Standard Specification for Pipe, Steel, Black and Hot-Dipped, Zinc-Coated, Welded and Seamless," A53/A53M-12, *Annual Book of ASTM Standards*. American Society for Testing Materials, West Conshohocken, PA.

ASTM. 2012. "Standard Specification for Structural Bolts, Steel, Heat Treated, 120/105 ksi Minimum Tensile Strength," A325-12, *Annual Book of ASTM Standards*. American Society for Testing Materials, West Conshohocken, PA.

ASTM. 2012. "Standard Specification for Castings, Iron-Chromium, Iron-Chromium-Nickel, Corrosion Resistant, for General Industrial Use," ASTM A743/A743M-12, *Annual Book of ASTM Standards*. American Society for Testing Materials, West Conshohocken, PA.

ASTM. 2012. "Standard Specification for Castings, Steel and Alloy, Common Requirements, for General Industrial Use," A781/A781M-12b, *Annual Book of ASTM Standards*. American Society for Testing Materials, West Conshohocken, PA.

ASTM. 2012. "Standard Specification for Aluminum and Aluminum-Alloy Drawn Seamless Tubes," B210-12, *Annual Book of ASTM Standards*. American Society for Testing Materials, West Conshohocken, PA.

ASTM. 2012. "Standard Specification for Aluminum and Aluminum-Alloy Rolled or Cold Finished Bar, Rod, and Wire," B211-12, *Annual Book of ASTM Standards*. American Society for Testing Materials, West Conshohocken, PA.

ASTM. 2012. "Standard Specification for Aluminum and Aluminum-Alloy Seamless Pipe and Seamless Extruded Tube," B241/B241M-12, *Annual Book of ASTM Standards*. American Society for Testing Materials, West Conshohocken, PA.

ASTM. 2012. "Standard Specification for Dimensional Tolerance of Thermosetting Glass-Reinforced Plastic Pultruded Shapes," D3917-12, *Annual Book of ASTM Standards*. American Society for Testing Materials, West Conshohocken, PA.

ASTM. 2013. "Standard Specification for Zinc (Hot-Dip Galvanized) Coatings on Iron and Steel Products," A123/A123M-13, *Annual Book of ASTM Standards*. American Society for Testing Materials, West Conshohocken, PA.

ASTM. 2013. "Standard Specification for Ferritic Malleable Iron Castings," ASTM A27/A27M-13, *Annual Book of ASTM Standards*. American Society for Testing Materials, West Conshohocken, PA.

ASTM. 2013. "Standard Specification for Cold-Formed Welded and Seamless Carbon Steel Structural Tubing in Rounds and Shapes," A500/A500M-13, *Annual Book of ASTM Standards*. American Society for Testing Materials, West Conshohocken, PA.

ASTM. 2013. "Standard Specification for Nonferrous Nuts for General Use," F467-13, *Annual Book of ASTM Standards*. American Society for Testing Materials, West Conshohocken, PA.

ASTM. 2013. "Standard Specification for Stainless Steel Bolts, Hex Cap Screws, and Studs," F593-13a, *Annual Book of ASTM Standards*. American Society for Testing Materials, West Conshohocken, PA.

ASTM. 2014. "Standard Specification for General Requirements for Rolled Structural Steel Bars, Plates, Shapes, and Sheet Piling," ASTM A6/A6M-14, *Annual Book of ASTM Standards*. American Society for Testing Materials, West Conshohocken, PA.

ASTM. 2014. "Standard Specification for Alloy-Steel and Stainless Steel Bolting for High Temperature or High Pressure Service and Other Special Purpose Applications," A193/A193M-14a, *Annual Book of ASTM Standards*. American Society for Testing Materials, West Conshohocken, PA.

ASTM. 2014. "Standard Specification for Carbon and Alloy Steel Nuts for Bolts for High Pressure or High Temperature Service, or Both," A194/A194M-14a, *Annual Book of ASTM Standards*. American Society for Testing Materials, West Conshohocken, PA.

ASTM. 2014. "Standard Specification for Carbon Steel Bolts, Studs, and Threaded Rod 60000 PSI Tensile Strength," A307-14, *Annual Book of ASTM Standards*. American Society for Testing Materials, West Conshohocken, PA.

ASTM. 2014. "Standard Specification for Alloy-Steel and Stainless Steel Bolting for Low-Temperature Service," A320/A320M-14, *Annual Book of ASTM Standards*. American Society for Testing Materials, West Conshohocken, PA.

ASTM. 2014. “Standard Specification for Structural Bolts, Alloy Steel, Heat Treated, 150 ksi Minimum Tensile Strength,” A490-14a, *Annual Book of ASTM Standards*. American Society for Testing Materials, West Conshohocken, PA.

ASTM. 2014. “Standard Specification for Hot-Formed Welded and Seamless Carbon Steel Structural Tubing,” A501/A501M-14, *Annual Book of ASTM Standards*. American Society for Testing Materials, West Conshohocken, PA.

ASTM. 2014. “Standard Specification for Ductile Iron Castings,” ASTM A536-84(2014), *Annual Book of ASTM Standards*. American Society for Testing Materials, West Conshohocken, PA.

ASTM. 2014. “Standard Specification for Carbon and Alloy Steel Nuts,” A563-07a(2014), *Annual Book of ASTM Standards*. American Society for Testing Materials, West Conshohocken, PA.

ASTM. 2014. “Standard Specification for Steel Tubes, Low-Carbon or High-Strength Low-Alloy, Tapered for Structural Use,” A595/A595M-14, *Annual Book of ASTM Standards*. American Society for Testing Materials, West Conshohocken, PA.

ASTM. 2014. “Standard Specification for Aluminum-Alloy Sand Castings,” B26/B26M-14, *Annual Book of ASTM Standards*. American Society for Testing Materials, West Conshohocken, PA.

ASTM. 2014. “Standard Specification for Aluminum-Alloy Permanent Mold Castings,” B108/B108M-14, *Annual Book of ASTM Standards*. American Society for Testing Materials, West Conshohocken, PA.

ASTM. 2014. “Standard Specification for Aluminum and Aluminum-Alloy Sheet and Plate,” B209-14, *Annual Book of ASTM Standards*. American Society for Testing Materials, West Conshohocken, PA.

ASTM. 2014. “Standard Specification for Aluminum and Aluminum-Alloy Extruded Bars, Rods, Wire, Profiles, and Tubes,” B221-14, *Annual Book of ASTM Standards*. American Society for Testing Materials, West Conshohocken, PA.

AWC. 2012. *National Design Specifications (NDS) for Wood Construction with Supplement: Design Values for Wood Construction,*” American Wood Council, Leesburg, VA.

AWPA. 2011. “User Specification for Treated Wood,” AWPA U1-11. American Wood Protection Association (formerly the American Wood Preservers’ Association), Birmingham, AL.

AWPA. 2013. *Book of Standards 2013.* American Wood Protection Association (formerly the American Wood Preservers’ Association), Birmingham, AL.

AWS. 2010. *Structural Welding Code—Steel*, ANSI/AWS D1.1-10. American Welding Society, Miami, FL.

AWS. 2008. *Structural Welding Code—Aluminum*, ANSI/AWS Dl.2/D1.2M:2008. American Welding Society, Miami, FL.

Jirsa, J. O., N. T. Cichy, M. R. Calzadilla, W. H. Smart, M. P. Pavluvcik, and J. E. Breen. 1984. *Strength and Behavior of Bolt Installations Anchored in Concrete Piers*, Report No. FHWA/TX-85/51+305-1F. Texas State Department of Highways and Public Transportation, Austin, TX.

RCSC. 2004. *Specification for Structural Joints Using ASTM A325 or A490 Bolts*. Research Council on Structural Connections, Chicago, IL.

Roy, S., Y. C. Park, R. Sause, J. W. Fisher, and E.J. Kaufmann. 2011. *Cost Effective Connection Details for Highway Sign, Luminaire and Traffic Signal Structures*. NCHRP Web Only Document 176 (Final Report for NCHRP Project 10-70). Transportation Research Board, National Research Council, Washington, DC.

SECTION 15: CONSTRUCTION

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SECTION 15:

CONSTRUCTION

15.1—SCOPE

This Section specifies construction provisions for structural supports for highway signs, luminaires, and traffic signals.

C15.1

The construction provisions of this Section are applicable to common structural supports for highway signs, luminaires, and traffic signals. Additional provisions may be required for unusual member types or usages.

15.2—DEFINITIONS

CAFL—Constant Amplitude Fatigue Limit
CAFT—Constant Amplitude Fatigue Threshold
DTI—direct tension indicating
GFRP—glass fiber reinforced polymer
Snug-tight—the maximum nut rotation resulting from the full effort of one person on a 12-in. long wrench or equivalent

15.3—NOTATION

d_b = nominal bolt diameter (in.) (C15.6.3)
F_t = installation pretension (kips) (C15.6.3)
T_v = verification torque (C15.6.3)

15.4—GENERAL

The contractor shall provide all tools, machinery, and equipment necessary to erect the structure.

All construction of structural supports shall be completed in accordance with all applicable provisions of the *AASHTO LRFD Bridge Construction Specifications* (AASHTO, 2010), and Owner requirements.

15.4.1—Notice of Beginning of Work

The Contractor shall give the Engineer ample notice of the beginning of work at the job site, so that inspection may be provided. No construction shall begin before the Engineer has been notified.

15.4.2—Inspector Authority

The Inspector shall have the authority to reject materials or work which does not fulfill the requirements of these Specifications.

It is expressly understood that inspection at the mill and shop shall not relieve the Contractor of any responsibility in regard to defective material or work and the necessity for replacing the same at the Contractor's cost.

The acceptance of any material or finished members by the Inspector shall not be a bar to their subsequent rejection if found defective. Rejected materials and work shall be replaced or corrected as soon as practical by the Contractor.

All work shall be performed in accordance with the contract documents.

C15.4.2

Inspection at the mill or shop is intended as a means of facilitating the work and avoiding errors.

15.4.3—Storage and Handling of Materials

Material shall be stored out of contact with the ground and dissimilar materials such as uncoated steel. Materials shall be free from dirt, grease, and foreign matter.

The materials to be stored on the job site shall be placed on skids above ground and kept clean and well drained. Girders and beams shall be placed upright and shored. If the Contractor's scope of work is for erection only, the Contractor shall check the material received against the shipping lists and report promptly in writing any shortage or damage. After material is received by the Contractor, the Contractor shall be responsible for any damage to or loss of material.

15.5—ERECTION

This Article addresses the erection of structural supports for highway signs, luminaires, and traffic signals.

15.5.1—Working Drawings

As required by the Owner, working drawings shall be submitted to the Engineer for review prior to performance of the work involved. Such review shall not relieve the Contractor of any responsibility under the contract for the successful completion of the work.

15.5.1.1—Shop Drawings

As required by the Owner, the Contractor shall submit copies of the detailed shop drawings to the Engineer for review. Shop drawings shall be submitted sufficiently in advance of the start of the affected work to allow time for review by the Engineer and corrections by the Contractor, if any, without delaying the work.

Shop drawings for structures shall give full detailed dimensions and sizes of component parts of the structure and details of all miscellaneous parts, such as pins, nuts, bolts, drains, etc.

C15.5.1.1

Shop drawings may include reinforcing or anchor bolt details for foundations as well as structural drawings.

15.5.1.2—Erection Drawings

As required by the Owner, the Contractor shall submit drawings fully illustrating the proposed method of erection. The drawings shall show details of all falsework bents, lifting devices, and attachments to the members. They will also show the sequence of erection, location of cranes, crane capacities, and location of lifting points on the members and the weights of the members. The drawings shall be complete in detail for all anticipated phases and conditions during erection. Calculations may be required to demonstrate that during erection factored resistances are not exceeded and that member capacities and final geometry will be correct.

Calculations shall be prepared under supervision of a licensed Professional Engineer (or Structural Engineer if so required by state licensure). When required, sealed drawings shall be provided to the owner.

C15.5.1.2

Support structures may involve long spans, heavy poles on frame assemblies, long and heavy mast arms, or difficult site access which may necessitate detailed erection planning.

Erection drawings should also indicate bolt tightening patterns, sequencing, and methods to assure proper bolt tension.

15.5.2—Erection Stresses

Any erection stresses induced in the structure as a result of erection which differs from the contract documents shall be accounted for by the Contractor. Erection design calculations for such changed methods shall be prepared at the Contractor's expense and submitted to the Engineer. The calculations shall indicate any change in stresses or change in behavior for the temporary and final structures. Additional material required to keep both the temporary and final force effects within the limits used in design shall be provided at the Contractor's expense.

The Contractor shall be responsible for providing temporary bracing or stiffening devices to limit stresses in individual members or segments of the structure during erection.

15.5.3—Maintaining Alignment and Camber

During erection, the Contractor shall be responsible for supporting segments of the structure in a manner that will produce the proper alignment and camber in the completed structure. Bracing shall be provided and installed by the Contractor as necessary during erection to provide stability and assure correct geometry.

C15.5.3

The Contractor must assure that components of some structures, such as most arms and cantilever planar trusses, are not installed upside down.

15.6—ANCHOR BOLTS

This Article addresses installation of cast-in-place, grouted, adhesive-bonded, expansion, and undercut steel anchors for highway signs, luminaires, and traffic signals.

15.6.1—New Construction

Anchor bolts for new construction shall be cast-in-place and conform to the requirements of Article 5.16. Anchor bolts shall be set with templates and adequately braced to assure proper anchor rod location and projection.

15.6.2—Post-Installed Anchors

Post-installed concrete anchors, including all bonded anchor systems (including grout, chemical compound, and adhesives), and undercut steel anchors shall be prequalified by universal test standards designed to allow approved anchor systems to be employed for any construction attachment use.

Tests for adhesive-bonded and other bonding compounds shall be conducted in accordance with ASTM E1512."

Embedment anchor details shall comply with ACI 318-95 (ACI, 1995), *Building Code Requirements for Structural Concrete*, "Appendix D, Anchoring to Concrete."

For anchor systems other than mechanical expansion anchors, the Contractor shall provide the Engineer with

C15.6.2

Post-installed concrete anchors may be required as part of structure retrofit or modification. Load tests of post-installed anchors may be used to verify capacity.

Design and acceptance testing must consider creep behavior under sustained loads.

certified test reports prepared by an independent laboratory documenting that the system is capable of achieving the minimum tensile strength of the embedment steel.

Installation of post-installed anchors shall be in accordance with the manufacturer's requirements.

15.6.3—Anchor Bolt Tightening

All anchor bolts shall be adequately tightened to prevent loosening of nuts and to reduce the susceptibility to fatigue damage. Anchor bolts in double-nut connections shall be pretensioned. Anchor bolts in single-nut connections shall be tightened to at least one half of the pretensioned condition. Anchor preload shall not be considered in design.

C15.6.3

The fatigue strength of anchor bolt connections is directly influenced by several installation conditions. Most important, all anchor bolt nuts shall be adequately tightened to eliminate the possibility of nuts becoming loose under service load conditions. When nuts become loose, the anchor bolts are more susceptible to fatigue damage. The most common method of pretensioning anchor bolts is the turn-of-nut method. Top nut rotation requirements to achieve proper anchor bolt pretensioning are given in Table C15.6.3-1. For single-nut connections, one half of the pretensioned condition may be estimated as 50 percent of the values for the turn-of-nut method and can be estimated by knowing the length of anchor bolt between the top of the foundation and the bottom of the top nut. The elongation that produces one-half of the yield load on the anchor bolt over this length is calculated. The required number of nut turns is then determined using the calculated elongation and the anchor bolt thread pitch.

Table C15.6.3-1—Top-Nut Rotation for Turn-of-Nut Pretensioning of Double-Nut Moment Connections

Anchor Bolt	Nut Rotation beyond Snug-Tight [a,b,c]	
Diameter, in.	**F1554 Grade 36**	**F1554 Grades 55 and 105, A449, A615, and A706 Grade 60**
$\leq 1^1/_2$	$^1/_6$ turn	$^1/_3$ turn
$>1^1/_2$	$^1/_{12}$ turn	$^1/_6$ turn

[a] Nut rotation is relative to anchor bolt. The tolerance is plus 20 degrees ($^1/_{18}$ turn).
[b] Applicable only to double-nut moment connections.
[c] Use a beveled washer if the nut is not in firm contact with the base plate or if the outer face of the base plate is sloped more than 1:40.

Anchor bolt preload does not affect the ultimate strength of a connection, but it does improve connection performance at working load levels. Fuchs et al. (1995) state that anchor bolt preload will affect the behavior of the anchor bolt at service loads and has practically no influence at failure load levels.

The testing described in NCHRP Report No. 412 (Kaczinski et al., 1998) shows that the Constant Amplitude Fatigue Threshold (CAFT) for anchor bolts is nearly the same for both snug and pretensioned installations. (In previous editions of this specification, the CAFT was termed Constant Amplitude Fatigue Limit, CAFL). Therefore, snug-tightened and pretensioned anchor bolts are designed for strength and fatigue in the same manner. Whenever practical, however, anchor bolts should be pretensioned. Although no benefit is considered when designing pretensioned anchor bolts for infinite life, it should be noted that the pretensioned condition reduces the possibility of anchor bolt nuts becoming loose under service-load conditions. As a result, the pretensioned condition is inherently better with respect to the performance of anchor bolts.

The following procedure adapted from Garlich and Thorkildsen (2005) should be considered when pretensioning double-nut moment connections. It has been derived from numerous references, including Till and Lefke (1994), James et al. (1997), Johns and Dexter (1998), and Dexter and Ricker (2002).

1. Verify that the nuts can be turned onto the bolts past the elevation corresponding to the bottom of each in-place leveling nut and be backed off by the effort of one person using a 12-in. long wrench or equivalent (i.e., without employing a pipe extension on the wrench handle).

2. Clean and lubricate the exposed threads of all anchor bolts and leveling nuts. Re-lubricate the exposed threads of the anchor bolts and the threads of the leveling nuts if more than 24 hours has elapsed since earlier lubrication, or if the anchor bolts and leveling nuts have become wet since they were first lubricated.

3. Turn the leveling nuts onto the anchor bolts and align the nuts to the same elevation. Place structural washers on top of the leveling nuts (one washer corresponding to each anchor bolt).

4. Install the base plate atop the structural washers that are atop the leveling nuts, place structural washers on top of the base plate (one washer corresponding to each anchor bolt), and turn the top nuts onto the anchor bolts.

5. Tighten top nuts to a snug-tight condition in a star pattern. Snug-tight is defined as the maximum nut rotation resulting from the full effort of one person using a 12-in. long wrench or equivalent. A star tightening pattern is one in which the nuts on opposite or near-opposite sides of the anchor bolt circle are successively tightened in a pattern resembling a star. (e.g., For an 8-bolt circle with anchor bolts sequentially numbered 1 to 8, tighten nuts in the following bolt order: 1, 5, 7, 3, 8, 4, 6, 2.)

6. Tighten leveling nuts to a snug-tight condition in a star pattern.

7. Before final tightening of the top nuts, mark the reference position of each top nut in a snug-tight condition with a suitable marking on one flat with a corresponding reference mark on the base plate at each bolt. Then incrementally turn the top nuts using a star pattern until achieving the required nut rotation specified in Table C15.6.3-1. Turn the nuts in at least two full tightening cycles (passes). After tightening, verify the nut rotation. Using a torque wrench, the verification torque, computed as shown below, should be applied to the top nuts. Inability to

achieve the verification torque may indicate thread stripping.

$$T_v = 0.12 d_b F_t$$

where:

T_v = verification torque
d_b = nominal bolt diameter (in.)
F_t = installation pretension (kips)
($F_t = 0.5F_y$ for Grade 36 bolts and 0.6 F_y for other bolts)

8. Retightening of installation by use of torque is recommended 48 hours after bolt tightening to account for any creep in the galvanizing within the threads.The retightening torque is 110 percent of the verification torque.

Direct-tension-indicating (DTI) washers provide a means of verifying that the anchor bolt preload is achieved. Direct-tension indicators for anchor bolts in diameters up to $2^1/_2$ in. and other applications are covered by ASTM F2437. Specifications include DTIs for Grade 55 and Grade 105 anchor bolts with preload of 60 percent of the yield strength. Use of DTIs with oversize base plate holes may require plate washers in addition to the hardened washers. It is recommended that DTIs be placed between the leveling nut and base plate to assure that the top nuts are fully tensioned to the base plate.

15.6.4—Placement of Anchor Bolts

Anchor bolts shall be installed with the following horizontal tolerances for each bolt location:

- Variation between the centers of any two bolts within an anchor bolt group: $\pm^1/_8$ in.
- Variation in dimension between the actual and established centerline of the anchor bolt group: $\pm^1/_4$ in.

Anchor bolts shall be placed vertically within ±1 in. of the proposed elevation.

C15.6.4

Proper location and alignment of anchor bolts is important to ensure that base plates that may be shop-fabricated have good fit up. The base plate fabricator should consider producing templates which can be used to by the Contractor in the field to promote precise placement of anchor bolts.

The anchor bolt tolerances given are consistent with the AISC *Code of Standard Practice for Steel Buildings and Bridges* (AISC, 2010). Vertical anchor bolt placement must assure sufficient thread projection so that anchor rod nuts are fully engaged. Thread length should be detailed with recognition of placement tolerances.

15.6.5—Plumbness of Anchor Bolts

Anchor bolts shall be installed with misalignments of less than 1:40 from vertical. After installation, firm contact shall exist between the anchor bolt nuts, washers, and base plate on any anchor bolt installed in a misaligned position.

C15.6.5

Vertical misalignment is a common installation condition that can influence the fatigue strength of anchor bolts. However, research (Kaczinski et al., 1998) has determined that bending stresses resulting from misalignments up to 1:40 do not need to be considered in stress calculations when designing anchor bolts provided that firm contact exists between the anchor bolt nuts, washers, and base plate. Where appropriate, a beveled washer may be utilized. Kaczinski et al. verified the alignment limit.

15.7—BOLTED CONNECTIONS

15.7.1—General

Design of bolted connections shall be in accordance with this book except as provided for anchor bolts in Article 5.16.

15.7.2—High-Strength Bolts

Installation of high-strength bolts shall be in conformance with the *AASHTO LRFD Bridge Construction Specifications* (AASHTO, 2010) or Owner's specifications. Beveled washers (hardened) shall be used where faying surfaces have a slope of more than 1:20 with respect to a plane normal to the bolt axis.

C15.7.2

High-strength bolts (ASTM A325) are commonly used in splice connections and pole-to-arm connections. Proper installation pretension is required to provide fatigue resistance.

15.7.3—End Plate Connections

One galvanized hardened steel washer conforming to AASHTO M 293 (ASTM F436) at each connection bolt shall be used between the end plate of a mast arm and the mounting plate of the pole to ensure plate-to-plate contact is provided at each bolt. The Owner may choose to exclude this requirement.

C15.7.3

Cutting of the end plate, welding of the mast arm to the end plate, and hot-dip galvanizing of the connection may cause distortion of the end plate. Similar distortions may occur in the mounting plate on the pole. The distortion of the plates makes it impossible to ensure uniform contact between the plates which may prevent tightening procedures from providing reasonable assurance of the actual final tensions. The washers ensure firm contact under the bolt heads and nuts in the connection, which improves ability to ensure the designed tension is met. Research test models constructed in this manner at the University of Texas have shown that acceptable fatigue performance is possible with the washers in place, and the washers do not reduce the fatigue strength of the connection. Filling gaps with hardened steel U-shaped shim plates provides an alternative means that may provide acceptable connection performance.

15.7.4—Miscellaneous Bolts and Fasteners

Miscellaneous bolts and fasteners shall be installed as provided by the fabrication/erection drawings. Fasteners shall be hot-dip galvanized or stainless steel. Lock washers, lock nuts, or double nuts shall be used. Miscellaneous bolts and fasteners shall be tightened in accordance with supplier requirements.

C15.7.4

Mild steel fasteners conforming to ASTM A307 or SAE Grades 1 and 2 are typically used for miscellaneous fasteners, U-bolts, etc. These fasteners may be installed using torque values shown in Table C15.7.4-1.

Table C15.7.4-1—Installation Torque for Mild Steel Fasteners

Bolt Size, Dia. (in.)	Minimum Torque (ft-lb)
$^3/_8$	15
$^1/_2$	37
$^5/_8$	74
$^3/_4$	120
$^7/_8$	190

Some fasteners may be intended not to be tightened in order to allow some movement. Double nuts, cotter pins, or other means should be used to prevent these fasteners from fully loosening.

Stainless steel bolts should be Type 304 or 316 stainless steel in accordance with ASTM F593. Matching nuts and washers should be provided. Bolts can be supplied either hot-or cold-finished. Installation may utilize the torque values shown in Table C15.7.4-2.

Table C15.7.4-2—Installation Torque for Stainless Steel Bolts

	Installation Torque	
Bolt Size Dia. (in.)	**Type 304 (ft-lb)**	**Type 316 (ft-lb)**
$^1/_2$	43	45
$^5/_8$	93	97
$^3/_4$	128	132
1	287	300

15.8—STEEL STRUCTURES

15.8.1—Inspection

Structural steel shall be inspected at the job site by the Contractor upon delivery and after erection.

15.8.2—Protective Systems

This Article addresses the protective systems for steel structural supports for highway signs, luminaires, and traffic signals.

15.8.2.1—General

Steel structures shall be protected from the effects of corrosion by means such as galvanizing, metalizing, painting, or other methods approved by the Designer or Owner. Corrosion likely to occur as a result of entrapped moisture, or other factors shall be eliminated or minimized by appropriate design and detailing. Positive means to drain moisture and condensation shall be provided unless the member is completely sealed.

15.8.2.2—Painted Structures

For painted structures, the materials and methods for any field painting shall conform to the Owner's requirements. Application of coatings, including any field touch-up, shall conform to the coating manufacturer's requirements.

15.8.2.3—Galvanized Structures

Field touchup and repair to galvanized coatings shall be made using a cold galvanizing compound in conformance with ASTM A780/A780M, or other Owner specified procedure.

15.8.3—Field Assembly

The parts shall be accurately assembled as specified in the contract documents or erection drawings, and any match-marks shall be followed. The material shall be carefully handled so that no parts will be bent, broken, or otherwise damaged. Hammering that will damage or distort the members is prohibited. Bearing surfaces, faying surfaces, and other surfaces to be in permanent contact shall be cleaned before the members are assembled. Splices and field connections shall have one quarter of the holes filled with bolts and one quarter filled with cylindrical erection pins before installing and tensioning bolts in the unfilled holes.

Any field welding shall be performed in accordance with the requirements of AWS D1.1-10 (AWS, 2010).

15.8.4—Misfits

The correction of minor misfits involving small amounts of reaming, cutting, grinding, and chipping shall be included in the Contractor's scope of work and shall be at the Contractor's expense. Any errors in the shop fabrication or deformation resulting from handling and transporting may be cause for rejection, however.

The Contractor shall be responsible for all misfits, errors, and damage, and shall make the necessary corrections and replacements.

15.8.5—Bolted Connections

Parts shall be assembled, well pinned, and firmly drawn together before drilling, reaming, or bolting. All joint surfaces, including surfaces to be under bolt heads or nuts, shall be free of dirt or other foreign material. Assemblies shall be taken apart, if necessary, for the removal of burrs and shavings.

Drifting done during assembling shall be limited to that which brings the parts into position and shall not enlarge holes or distort the metal.

Bolts used as fit-up bolts may be reused for the final installation. If other fit-up bolts are used, they shall be of the same nominal diameter as the final bolts, and cylindrical erection pins shall be 0.03 in. larger.

15.9—ALUMINUM STRUCTURES

15.9.1—General

This Article describes the erection of aluminum structures and structural aluminum portions of other structures in accordance with these Specifications and the contract documents.

Details of design which are permitted to be selected by the Contractor shall conform to the current *LRFD Specifications for Structural Supports for Highway Signs, Luminaires, and Traffic Signals* and subsequent interim specifications, and Owner requirements.

15.9.2—Inspection

Structural aluminum components shall be inspected by the Contractor at the job site upon delivery and after erection.

15.9.3—Bolted Connections

Parts shall be firmly drawn together before bolting. All joint surfaces, including surfaces to be under bolt heads or nuts, shall be free of dirt or other foreign material. Assemblies shall be taken apart, if necessary, for the removal of burrs and shavings.

Drifting done during assembling shall be limited to that which brings the parts into position and shall not enlarge holes or distort the metal.

Bolt installation shall be in accordance with Article 15.7.

Stainless steel bolts used in primary connections shall have single or double nuts unless otherwise specified in the contract documents. Lock washers may be included. Beveled washers shall be used where faying surfaces have a slope of more than 1:20 with respect to a plane normal to the bolt axis. A washer shall be provided under the turned part. Bolts shall be installed to the torque as specified by the bolt manufacturer.

Stainless steel splice bolts are sometimes used in aluminum structures for corrosion resistance. Since these are not high-strength bolts, lock washers may be used. Bolts should be installed to torque at values provided by the bolt supplier.

15.9.4—Field Assembly

The parts shall be accurately assembled as specified in the contract documents or erection drawings, and any match-marks shall be followed. The material shall be carefully handled so that no parts will be bent, broken, or otherwise damaged. Hammering that will injure or distort the members is prohibited. Bearing surfaces, faying surfaces, and other surfaces to be in permanent contact shall be cleaned before the members are assembled. Splices and field connections shall have one quarter of the holes filled with bolts and one quarter filled with cylindrical erection pins before installing and tensioning bolts in the unfilled holes.

Bolts used as fit-up bolts may be reused for the final installation. If other fit-up bolts are used, they shall be of the same nominal diameter as the final bolts, and cylindrical erection pins shall be 0.03 in. larger.

15.9.5—Misfits

The correction of minor misfits involving small amounts of reaming, cutting, grinding, and chipping shall be included in the Contractor’s scope of work and shall be at the Contractor’s expense. Any errors in the shop fabrication or deformation resulting from handling and transporting may be cause for rejection, however.

The Contractor shall be responsible for all misfits, errors, and damage, and shall make the necessary corrections and replacements.

15.9.6—Cleaning and Treatment of Metal Surfaces

Prior to field painting of structures, all surfaces to be painted shall be cleaned immediately before painting by a method that will remove all dirt, oil, grease, chips, and other foreign substances.

Exposed metal surfaces shall be cleaned with a suitable chemical cleaner, such as a solution of phosphoric acid and organic solvents meeting United States Military Specification MIL-M-10578. Abrasion-blasting shall not be used on aluminum less than or equal to 0.125 in. thick.

15.10—PRESTRESSED CONCRETE STRUCTURES

15.10.1—Inspection

Prestressed concrete components shall be inspected by the Contractor upon delivery to the job site and after erection.

15.10.2—Protective Systems

This Article addresses the protective systems for prestressed concrete structural supports for highway signs, luminaires, and traffic signals.

15.10.2.1—Concrete Finish

The erector of prestressed concrete members shall notify the Engineer of the following surface conditions:

- all honeycombs deep enough to expose prestressing steel,
- damage caused by shipping or erection,
- any area that the erector believes to be detrimental.

The Engineer responsible for inspection at the project site will consult with the Owner and manufacturer to determine the corrective action required to repair the member. Prestressed members that cannot be effectively repaired will be rejected. The Engineer is responsible for final acceptance.

15.10.2.2—Surface Treatment

When plans call for surface paint, stain, or other coatings, the coating manufacturer's recommendations for surface preparation should be followed.

Damage to coatings due to shipping or erection shall be repaired in accordance with the manufacturer's requirements.

C15.10.2.2

A variety of surface coatings are available and can aid in preservation of the concrete member by preventing degradation of the concrete material and also by preventing corrosion of the internal steel reinforcement. A common cause of coating failures is lack of preparation of the concrete surface prior to the coating application. The manufacturer's recommendations should be followed. They may include requirements regarding cure time, relative humidity of the concrete, surface texture, removal of membrane-forming compounds used for curing, and other requirements.

15.11—COMPOSITE (GLASS FIBER REINFORCED POLYMER) STRUCTURES

15.11.1—Erection

This Article addresses the erection of glass fiber reinforced polymer (FRP) composite structural supports for highway signs, luminaires, and traffic signals.

15.11.2—Rigging

Rigging used for erection shall consist of straps or padded slings that will not cause damage to the components.

The Contractor shall be responsible for providing temporary bracing or stiffening devices to limit stresses in individual members or segments of the structure during erection.

15.11.3—Protective Systems

Any damage to member protective systems due to shipping or erection shall be repaired in accordance with the manufacturer's requirements. FRP members that cannot be effectively repaired shall be rejected.

15.12—WOOD STRUCTURES

15.12.1—Erection

Wood poles shall be erected plumb, with temporary bracing used as required.

15.12.2—Fasteners

Bolts and other fasteners shall be hot-dipped galvanized steel or stainless steel and installed per manufacturer's recommendations. Holes shall be accurately drilled.

15.12.3—Preservative Treatment for Posts

Surfaces of field-drilled holes or cuts, and all damaged areas, shall be field treated per AWPA Standard M4, Standard for the Care of Treated Wood Products (AWPA, 2010).

C15.12.3

Many preservative products are only available to licensed applicators, therefore a system different from what was originally used may be needed for field treatment.

15.13—FOUNDATIONS

15.13.1—Construction Requirements

Construction of foundations shall be in accordance with applicable sections of the *AASHTO LRFD Bridge Construction Specifications* (AASHTO, 2010) and Owner requirements.

15.13.2—Concrete

Concrete for portions of a foundation extending from the top of foundation (or pier, pedestal, etc.) to a minimum of two feet below finished grade shall be of a structural concrete possessing suitable durability for exposed structures.

C 15.13.2

Foundations are subject to frequent wet and dry cycles, salt exposure, runoff, etc. which can lead to concrete deterioration.

15.13.3—Reinforcing

15.13.3.1—Corrosion Protection

Reinforcing steel for exposed portions of foundations shall be suitably protected from corrosion.

C15.13.3.1

Coated reinforcing should be used, subject to Owner requirements. Additional concrete cover may also be used.

15.13.3.2—Placement

Vertical reinforcing in piers, pedestals, and shafts shall extend to within 2 in. of the top of foundations.

Placement of ties shall extend the full height of piers, pedestals, and shafts. At least two additional ties at six-in. spacing shall be located below the upper most tie.

C15.13.3.2

Careful placement of ties along with added ties near the top of the pier, shaft, or pedestal is recommended to improve concrete confinement at the anchor bolts. This is particularly important in areas subject to hurricane winds or seismic events.

15.13.4—Prefabricated Foundations

Prefabricated foundations of concrete on galvanized steel shall be installed in accordance with manufacturer's requirements. Drawing and installation details and procedures for prefabricated foundations shall be submitted to the Owner prior to installation. The Contractor shall verify that field conditions and soils types and strengths match those used to develop the installation drawings.

Helical pier foundations shall be installed to develop the installation torque as required in the design drawing. A record of the installation torque shall be provided for each foundation.

Precast concrete foundations shall be installed to elevations shown in the design drawing with pier tops level and piers plumb. The Contractor shall verify that supporting soils have the bearing capacity specified on the design drawings. Unsuitable soils shall be removed and replaced with compacted structural fill in accordance with Owner requirements. Backfill shall be compacted fill confirming to Owner requirements.

C15.13.4

Prefabricated concrete bases are sometimes used for sign, signal, and lighting foundations. Provisions should be preapproved by the Owner and should provide complete engineering data to support use of the products in any specific applications.

Helical fabricated steel piers are often used for smaller signal and lighting poles. They are installed by turning or screwing the helical pier into the ground. Foundation capacity is a function of the installation torque and thus close control of this torque is a basic inspection parameter.

15.13.5—Use of Grout

Grout, when specified under base plates in a load-carrying application, shall be non-shrink. Grout shall not contain any chlorides or other harmful additives that could cause corrosion of the anchor bolts. Grout shall not be considered as a load-carrying element in double-nut connections.

Grout, when specified under base plates in a load carrying application, shall be a proprietary prepackaged non-shrink grout conforming to ASTM C1107/C1107M. Minimum grout compressive strength shall be 5000 psi at 28 days, and shall be placed in accordance with Manufacturer's specifications.

Prior to placing grout, all anchor bolt nuts shall be tightened as outlined in Article 15.6.3. Grout shall

C15.13.5

Compressive load from the base plate in double-nut connections should be supported directly by the anchor bolt leveling nuts. In installations without leveling nuts, the base plate is placed directly on a grout pad. Fuchs et. Al. (1995) note that in this type of installation, a grout failure may occur before any other type of failure.

The use of grout under double-nut base plates is discouraged as it can lead to anchor bolt and leveling-nut corrosion (Garlich and Thorkildsen, 2005). It also inhibits inspection of leveling nuts and anchor bolts, and, in many cases, the grout is of poor quality. Cook et al. (2000) report that properly installed nonshrink grout pads may improve performance of double-nut base plates, particularly for flexible plates.

completely fill the area beneath the base plate. Grout shall not be considered as a load-carrying element in double-nut connections.

If grout is to be placed, a form may be needed to contain the grout below the plate while providing an annulus for the electrical connections. The grout shoulders should not extend above the bottom of the base plate, and a means of drainage from below the base plate should be provided. See Figure C15.13-1.

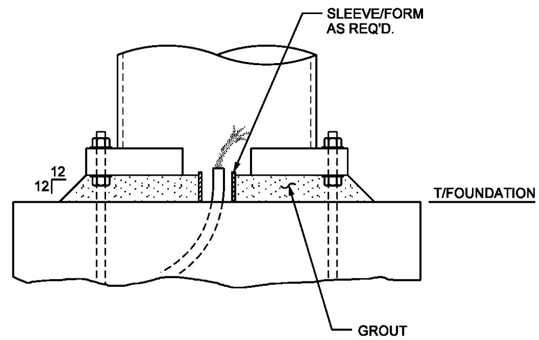

Figure C15.13-1—Grouted Base Plate

15.14—REFERENCES

AASHTO. 1990. “Standard Specification for Steel Anchor Bolts,” M 314. In *Standard Specifications for Transportation Materials and Methods of Sampling and Testing*. American Association of State Highway and Transportation Officials, Washington, DC.

AASHTO. 2002. *Standard Specifications for Highway Bridges*, 17th Edition, HB-17. American Association of State Highway and Transportation Officials, Washington, DC.

AASHTO. 2014. “Standard Specification for Zinc (Hot-Dip Galvanized) Coatings on Iron and Steel Products,” M 111M/M 111. In *Standard Specifications for Transportation Materials and Methods of Sampling and Testing*. American Association of State Highway and Transportation Officials, Washington, DC.

AASHTO. 2010. Zinc Coating (Hot-Dip) on Iron and Steel Hardware, M 232M/M 232. In *Standard Specifications for Transportation Materials and Methods of Sampling and Testing*. American Association of State Highway and Transportation Officials, Washington, DC.

AASHTO. 2010. *AASHTO LRFD Bridge Construction Specifications*, 3rd Edition. American Association of State Highway and Transportation Officials, Washington, DC.

ACI. 1991. *State-of-the-Art Report on Anchorage to Concrete*, ACI 355.1R-91. American Concrete Institute, Farmington Hills, MI.

ACI. 1995. *Building Code Requirements for Structural Concrete*, ACI 318-95. American Concrete Institute, Farmington Hills, MI.

ACI. 1995. *Code Requirements for Nuclear Safety Related Concrete Structures,* Appendix B, “Steel Embedments,” ACI 349-90. American Concrete Institute, Farmington Hills, MI.

AISC. 2010. *Code of Standard Practice for Steel Buildings and Bridges*. American Institute of Steel Construction, Chicago, IL.

Aluminum Association. 1997. *Aluminum Standards and Data.* Aluminum Association, Washington, DC.

Aluminum Association. 2000. *Aluminum Design Manual*, “Specifications for Aluminum Structures.” Aluminum Association, Washington, DC.

Aluminum Association. 2002. *Welding Aluminum: Theory and Practice*. Aluminum Association, Washington, DC.

ASTM. 2007. “Standard Test Methods for Bond Performance of Bonded Anchors,” ASTM E1512-01(2007), *Annual Book of ASTM Standards.* American Society for Testing Materials, West Conshohocken, PA.

ASTM. 1998. “Standard Specification for Anchor Bolts, Steel, 36, 55, and 105-ksi Yield Strength,” ASTM F1554-94, *Annual Book of ASTM Standards.* American Society for Testing Materials, West Conshohocken, PA.

ASTM. 2009. “Standard Practice for Repair of Damaged and Uncoated Areas of Hot-Dip Galvanized Coatings,” ASTM A780/A780M-09, *Annual Book of ASTM Standards.* American Society for Testing Materials, West Conshohocken, PA.

ASTM. 2011. “Standard Specification for Hardened Steel Washers,” ASTM F436-11, *Annual Book of ASTM Standards.* American Society for Testing Materials, West Conshohocken, PA.

ASTM. 2014. “Standard Specification for Structural Bolts, Steel, Heat Treated, 120/105 ksi Minimum Tensile Strength,” A325-14, *Annual Book of ASTM Standards*. American Society for Testing Materials, West Conshohocken, PA.

ASTM. 2013. “Standard Specification for Stainless Steel Bolts, Hex Cap Screws, and Studs,” F593-13a, *Annual Book of ASTM Standards*. American Society for Testing Materials, West Conshohocken, PA.

ASTM. 2014. “Standard Specification for Carbon and Alloy Steel Compressible-Washer-Type Direct Tension Indicators for Use with Cap Screws, Bolts, Anchors, and Studs,” ASTM F2437-14 *Annual Book of ASTM Standards.* American Society for Testing Materials, West Conshohocken, PA.

ASTM. 2014. “Standard Specification for Carbon Steel Bolts, Studs, and Threaded Rod 60000 PSI Tensile Strength,” A307-14, *Annual Book of ASTM Standards*. American Society for Testing Materials, West Conshohocken, PA.

ASTM. 2014. “Standard Specification for Packaged Dry, Hydraulic-Cement Grout (Nonshrink),” ASTM C1107/C1107M-14, *Annual Book of ASTM Standards.* American Society for Testing Materials, West Conshohocken, PA.

AWC. 2012. *National Design Specification (NDS) for Wood Construction.* American Wood Council, Leesburg, VA.

AWPA. 2011. *Standard for the Care of Preservative-Treated Wood Products*, AWPA M4-11. American Wood Protection Association (formerly the American Wood Preservers’ Association), Birmingham, AL.

AWPA. 2014. *Book of Standards 2014*. American Wood Protection Association (formerly the American Wood Preservers’ Association), Birmingham, AL.

AWS. 2008. Structural Welding Code—Aluminum, ANSI/AWS Dl.2/D1.2M:2008. American Welding Society, Miami, FL.

AWS. 2010. *Structural Welding Code-Steel*, AWS D1.1/D1.1M:2010. American Welding Society, Miami, FL.

Cook, R. A., G. T. Doerr, and R. E. Klingner. 1989. *Design Guide for Steel-to-Concrete Connections*, Report No. FHWA lTX-89+ 1126-4F. Center for Transportation Research, Texas State Department of Highways and Public Transportation, Austin, TX.

Cook, R. A., D. S. Ellifritt, S. E. Schmid, A. Adediran, and W. Beese. 1995. *Design Procedure for Annular Base Plates*, Final Project Report No. FL/DOT/RMC/0697-8804, Structures and Materials Research Report No. 95-4. Florida Department of Transportation, Tallahassee, FL.

Cook, R. A., M. Tia, K. B. Fisher, D. D. Darku, and M. H. Ansley. 2000. *Use of Grout Pads for Sign and Lighting Structures Part 1—Structural Evaluation*, Structures Research Report No. BB-512. University of Florida, Gainesville, FL.

Davis, J. R. 1993. *Aluminum and Aluminum Alloys*. ASM International, Materials Park, OH.

Dexter, R., and M. Ricker. 2002. *Fatigue-Resistant Design of Cantilever Signal, Sign, and Light Supports*, NCHRP Report 469. Transportation Research Board, National Research Council, Washington, DC.

Fuchs, W., R. Eligenhausen, and J. E. Breen. 1995. Concrete Capacity Design Approach for Fastening to Concrete, *ACI Structural Journal 92,* No.1 (January-February 1995). American Concrete Institute, Farmington Hills, MI.

Garlich, M. J. and E. T. Thorkildsen. 2005. *Guidelines for the Installation, Inspection, Maintenance and Repair of Structural Supports for Highway Signs, Luminaires, and Traffic Signals*, Publication No. FHWA NHI 05-036. Federal Highway Administration, U.S. Department of Transportation, Washington, D.C.

James, et al. 1997. *Tightening Procedures for Large-Diameter Anchor Bolts*, Texas Transportation Institute, Texas A&M University, College Station, TX.

Johns, K. W., and R. J. Dexter. 1998. *Fatigue Related Wind Loads on Highway Support Structures*; Final ATLSS Report No. 98-03. Lehigh University, Bethlehem, PA. Prepared for New Jersey Department of Transportation.

Kaczinski, M. R., R. J. Dexter, and J. P. Van Dien. 1998. *Fatigue-Resistant Design of Cantilevered Signal, Sign and Light Supports*, NCHRP Report No. 412. Transportation Research Board, National Research Council, Washington, DC.

Kissell, J. R., and R. L. Ferry. 1995. *Aluminum Structures: A Guide to Their Specifications and Design*. John Wiley and Sons, New York, NY.

Till, R.D and N. A. Lefke. 1994. *The Relationship Between Torque, Tension, and Nut Rotation of Large Diameter Anchor Bolts*, Materials and Technology Division, Michigan Dept. of Transportation, Lansing, MI

U.S. DOD. 1998. *Metal Conditioning and Rust Remover*, Military Specification MIL-M-10578. U.S. Dept. of Defense, Washington, DC.

SECTION 16: INSPECTION AND REPORTING (ADVISORY)

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SECTION 16:

INSPECTION AND REPORTING (ADVISORY)

16.1—SCOPE

This Section specifies recommended inspection and reporting practices for ancillary structures such as highway signs, luminaires, and traffic signals.

C16.1

Much of the information presented in this Section is based on The Federal Highway Administration (FHWA) *Guidelines for the Installation, Inspection, Maintenance and Repair of Structural Supports for Highway Signs, Luminaires, and Traffic Signals* (FHWA Guidelines, 2005).

16.2—TYPES OF INSPECTION

The types of inspection may vary over the service life of an ancillary structure in order to reflect the intensity required. Listed below are five inspection types that define levels consistent with the site-visit frequency, type of structure, and associated details. These inspection types are consistent with those performed on bridges.

16.2.1—Initial Inspections

An initial inspection is the first that meets the purposes outlined below and becomes part of the permanent record. The initial inspection should be a fully documented investigation performed by persons meeting the qualifications for inspection personnel.

The initial inspection purpose is two-fold:

- Gather all structure inventory data and relevant information normally collected by the Owner.
- Determine baseline conditions and identify existing problems or locations of potential problems.

This inspection should take place shortly after the structure is completed.

C16.2.1

Although the structure may have been inspected previously, the records may not be available or may be incomplete.

16.2.2—Routine Inspections

Routine inspections are periodically scheduled and include observations, measurements, or both, needed to determine the physical and functional condition of the ancillary structure. Changes from "initial" or previously recorded conditions shall be identified to ensure that the structure continues to satisfy present service requirements. Routine inspections may be used to obtain or verify inventory data and to scan for significant deficiencies.

These inspections can consist of at-grade inspections possibly without traffic lane closure.

Routine inspections should be carefully documented with photographs and a written report that includes recommendations for maintenance or repair, and for scheduling the follow-up routine, in-depth, or special inspections, if necessary.

Other media such as video, spoken narrative, etc. may support the report.

16.2.3—Damage Inspections

A damage inspection is unscheduled and used to assess structural damage resulting from environmental factors or human actions (e.g., major storm events or collisions). The scope should be sufficient to determine the need for structure repair or removal. The intensity may vary significantly depending upon the extent of the damage.

C16.2.3

It is often necessary to inspect the entire structure to adequately assess the damage brought on by environmental factors or human actions.

16.2.4—In-Depth Inspections

An in-depth inspection is a close-up, hands-on examination of one or more members to identify any deficiencies not readily detectable by a routine inspection. When appropriate or necessary to fully ascertain the existence of or the extent of any deficiencies, nondestructive testing, other material tests, or both may be appropriate. This inspection type is typical for ancillary structures that span over traffic lanes.

This type of inspection may be scheduled independently of a routine inspection, although generally at a longer time interval, or it may be a follow-up for damage or initial inspections.

C16.2.4

Traffic control may be necessary to obtain access.

16.2.5—Interim Inspections

An interim inspection should typically be recommended by the team leader if a structure is determined to have deficiencies or other problems that require more frequent examinations than the typical inspection frequency. An interim inspection might also be required after temporary repairs.

16.3—INSPECTION FREQUENCY

It is up to the program manager to determine the appropriate inspection frequency for the structure inventory. To make a risk-based assessment, the inspection frequency for ancillary structures should consider factors including material type, structure type, condition, importance, accessibility, age, and allocated resources.

C16.3

Many Owners are initiating ancillary structure programs and determining inspection frequency after the first inspection cycle is complete and deficiencies are categorized. At that time, Owners understand what type of issues are present in the inventory and possible consequences of inaction. Access challenges and traffic control can also be considered with these initial inspections.

The FHWA Guidelines (2005) recommend the following considerations for inspection frequencies:

- **Material Issues—**It is recommended that in-depth inspections be conducted at a 24-month frequency for aluminum structures based on past performance issues. If structures are performing well, then the 24-month frequency could be revised.
- **Redundancy Issues—**For cantilever, butterfly, and other non-redundant structures, a 48-month inspection frequency is recommended.
- **Typical Sign Bridges—**A typical two-support sign bridge with steel supports and superstructure need only be inspected in-depth every 72-months. Routine or ground inspections can be conducted

more frequently to check for corrosion at the base, connection problems, impact damage, or significant truss problems.

- **Traffic Issues**—If 'severe' restrictions exist, frequencies stated above can be extended but supplemented by routine ground inspections. Refer to the FHWA *Guidelines for the Installation, Inspection, Maintenance, and Repair of Structure Supports for Highway Signs, Luminaires, and Traffic Signals* (FHWA, 2005) for additional guidance on inspection frequencies.

16.4—QUALIFICATIONS AND RESPONSIBILITIES OF INSPECTION PERSONNEL

16.4.1—General

Inspections are not required by federal regulations. The inspection of ancillary structures has similarities to the inspection of highway bridges plus some special considerations. Due to the large number of welded members used in most ancillary structures, it is desirable that at least one team member have experience in visual weld inspection as well as training in locating and recognizing fatigue cracks.

C16.4.1

All inspection personnel should be able to physically perform the work. Although bucket trucks are typically used to access sign bridges, an adequate in-depth inspection cannot always be performed from a bucket and climbing may be necessary. In addition, traffic control is typically necessary and could involve lifting and transporting items such as traffic cones, advanced warning signs, and sign stands.

16.4.2—Ancillary Structure Inspection Program Manager

The inspection program manager is in charge of the program and is assigned or delegated the duties and responsibilities for ancillary structure inspection, reporting, and inventory. The program manager provides overall leadership and is available to inspection team leaders to provide guidance.

The inspection program manager should be a licensed professional engineer or have a minimum 10 years of experience in ancillary structure inspections in a responsible charge and have completed comprehensive bridge or ancillary structure inspection training.

C16.4.2

The program manager provides overall supervision and is available for consultation with team leaders. Ideally, the position requires a general understanding of design, load rating, new construction, rehabilitation, inspection or condition evaluation, and maintenance.

Judgment is important to determine the urgency of problems and to implement the necessary short-term remedial actions for public safety. When appropriate, the specialized knowledge and skills of associate engineers in such fields as structural design, construction, materials, maintenance, electrical equipment, machinery, soils, or emergency repairs should be sought.

16.4.3—Inspection Team Leader

A team leader is in charge of an inspection team and is responsible for planning, preparing, and performing field inspections of the ancillary structures. A qualified team leader should be at the site at all times during each initial, routine, in-depth, interim, and damage inspection.

The team leader should have a minimum of five years of experience in structure assignments in a responsible capacity or National Institute for Certification in Engineering Technologies (NICET) Level III or IV certification in structure inspection and have completed comprehensive bridge or ancillary structure inspection training.

16.4.4—Inspection Team Member

An inspection team will usually include a team leader and a team member whose qualifications can be project specific. A team member should be able to appropriately assist the team leader throughout the inspection process.

16.5—SAFETY

16.5.1—General

Safety of the inspection team and the public is paramount. A safety program should be developed to provide inspection personnel with information concerning their safety and health, including information that pertains to the proper operation of inspection tools and equipment. This program should embody applicable state and federal legislation governing safety and health in the bridge inspection work environment.

16.5.2—Personnel Safety

Personal protective clothing shall be worn at all times, including a hard hat, reflective vest, safety glasses, and appropriate footwear. Proper hearing, sight, and face protection methods should be practiced whenever using manual or power tools. All equipment, safety devices, and machinery should be kept in good operating condition.

Inspection vehicles and equipment should be operated in accordance with the operating manuals provided by the manufacturer. Personnel should be trained in the safe use of the vehicles and equipment as well as emergency procedures in the event of equipment failure.

Belts, lanyards, harnesses, and other personal safety equipment should be used in accordance with applicable standards when working from heights. All lifelines, belts, lanyards, and other equipment should be maintained in good repair. Worn or damaged equipment shall be discarded. In addition, inspection personnel should be cautioned to keep safety equipment clean and away from potentially harmful chemicals such as gasoline, dye penetrant, oil, or combinations thereof.

C16.5.2

Safety programs provide a guide to inspection personnel but are not a substitute for good judgment. Each ancillary structure site is unique. In situations where unusual working conditions may exist, specialized safety precautions may be required. Inspection personnel should have first aid training.

16.5.3—Public Safety

In the interest of public safety, proper procedures for traffic control and work zone protection should be employed during inspections.

C16.5.3

The *Manual on Uniform Traffic Control Devices* (MUTCD) (FHWA, 2009) as supplemented by state and local authorities should be used as a guide for such procedures.

16.6—PLANNING, SCHEDULING, EQUIPMENT

16.6.1—Planning

The inspection plan should be developed based on a review of the structure record and may require a pre-inspection site visit. The following steps should be considered:

- Determine the type of inspection required.
- Determine the type of equipment and tools necessary to perform the inspection.
- Determine which members and locations are noted in previous inspections or maintenance records to have existing defects or areas of concern.
- Estimate the duration of the inspection and the scheduled work hours.
- Establish coordination with, or notification of, other agencies or the public, as needed.
- Assemble field recording forms.
- Decide whether nondestructive testing is appropriate.
- Determine whether the structure contains members or details requiring special attention such as fatigue-prone details and nonredundant members.
- Determine whether there are structures nearby that are also scheduled for inspection and that require a similar crew with similar tools and equipment.

The key to the effective, safe performance of any ancillary structure inspection is proper advance planning and preparation.

16.6.2—Scheduling

So far as is practicable, inspections should be scheduled at times which offer the most desirable conditions for thorough inspections considering weather, traffic, and other events.

16.6.3—Equipment

Ancillary structure inspection equipment consists of items used for accessing and performing inspection tasks. The equipment requirements should become part of the structure record.

16.6.3.1—Access Methods and Equipment

The variation in types of structures requires that a broad range of techniques and equipment be used to gain access to the structural elements and to perform the inspection. Equipment often includes ladders and power-lift vehicles. In selecting the use of such equipment, the need for traffic control, lane closures, or both, should be considered.

16.6.3.2—Inspection Methods and Equipment

The inspection methods and equipment will depend on the type of inspection. Planning may require a pre-inspection site visit. If structural plans are available, the pre-inspection should be conducted with plans-in-hand to allow preliminary verification of structure configuration and details.

The pre-inspection plan should address:

- means of access,
- areas of potential concern that will require close attention during subsequent inspections, and
- the basis for making decisions that may be a result of timing, weather conditions, traffic control, and possible utility power loss.

16.7—INSPECTION FORMS AND REPORTS

A clear and detailed report should be made for each inspection visit. The sources and dates of all information contained in a report should be clearly evident. Nomenclature used to describe the ancillary structure components should be consistent.

Photographs should be taken in the field to illustrate defects and be cross referenced in the body of the report. They should be used to supplement written notes concerning the location, physical characteristics, and extent of deficiencies.

All indications of distress and deterioration should be noted with sufficient accuracy so that future inspectors can readily make a comparison of condition. If warranted, recommendations for repair and maintenance should be included.

16.8—ELEMENT LEVEL INSPECTIONS

The element set presented below includes two element types identified as National Ancillary Structure Elements (NASE) and Ancillary Structure Management Elements (ASME). The combination of these two element types comprise the AASHTO element set.

All elements, whether they are NASE or ASME, have the same general requirements:

- a standard number of condition states, and
- a standard number of comprised condition states such as good, fair, poor, and severe general descriptions.

A detailed description of each element is located in Appendix D.

16.8.1—National Ancillary Structure Elements

The NASE represent the primary structural elements necessary to determine the overall condition and safety of structures. It follows a similar description and philosophy as the National Bridge Elements and is designed to remain consistent from agency to agency across the country in order to facilitate the capture of element conditions at the national level.

16.8.2—Ancillary Structure Management Elements

ASME include components such as sign panels, signal heads, and protective coating systems that are important to an Owner but may not affect the load-carrying capacity of a structure.

16.9—PROCEDURES

16.9.1—General

Defects found require a thorough investigation to determine and evaluate their cause.

C16.9.1

The cause of most defects is readily evident; however, it may take considerable time and effort to determine the cause of some defects and to fully assess their seriousness.

16.9.2—Inspection Procedures

Inspection procedures are available elsewhere. Check with the Owner for the specific procedures to be used.

C16.9.2

Refer to the FHWA Guidelines (FHWA, 2005) for guidance on inspection procedures. In addition, the AASHTO *Manual for Bridge Evaluation* (2011) and the FHWA *Bridge Inspector's Reference Manual* (2012) are good resources regarding inspection.

16.10—REFERENCES

AASHTO. 2011. *Manual for Bridge Evaluation*. American Association of State Highway and Transportation Officials, Washington, DC.

FHWA. 2005. *Guidelines for the Installation, Inspection, Maintenance and Repair of Structural Supports for Highway Signs, Luminaires, and Traffic Signals*, FHWA-NHI-05-036. Federal Highway Administration, U.S. Department of Transportation, Washington DC.

FHWA. 2009. *Manual on Uniform Traffic Control Devices*. Federal Highway Administration, U.S. Department of Transportation, Washington, DC.

FHWA. 2012. *Bridge Inspector's Reference Manual*. Federal Highway Administration, U.S. Department of Transportation, Washington, DC.

SECTION 17: ASSET MANAGEMENT (ADVISORY)

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SECTION 17:

ASSET MANAGEMENT (ADVISORY)

17.1—SCOPE

Transportation agencies must balance limited resources against increasing needs of an aging highway system. The best action for each ancillary structure, considered alone, is not necessarily the best action for the highway system considering funding constraints. The optimal actions cannot be determined without first establishing the implications from a system-wide perspective. Engineers, administrators, and public officials have acknowledged the need for analytical methods and procedures to assess the current and predict future conditions of structures. Such methods and procedures are used to determine an optimal allocation of funds within a system of structures among various types of maintenance, repair, rehabilitation, and replacement choices.

The advent of asset management systems is a response to this need. Asset management systems require the data and results from condition evaluation (i.e., inspection) as well as analytical (e.g., fatigue) evaluation and deterioration rate algorithms.

This Section provides an overview of a successful asset management system for ancillary structures, including

- personnel organization,
- data and records required,
- replacement options, and
- repair and maintenance tasks recommended.

17.2—NOTATION

A	=	fatigue detail category constant (ksi^3) (17.5.2.3)
a	=	width, measured across member, of a square or rectangular hole (in.) (17.5.4.1) (17.5.4.2) (17.5.4.3)
b	=	length, measured along member, of a square or rectangular hole (in.) (17.5.4.1) (17.5.4.2) (17.5.4.3)
C	=	circumference of tube (in.) (17.5.3.1) (C17.5.3.1) (17.5.3.2) (17.5.3.3) (C17.5.3.3)
D	=	outside diameter of tube (in.) (17.5.4.1) (17.5.4.2) (17.5.4.3)
d	=	diameter of a round hole (in.) (17.5.4.1) (17.5.4.2) (17.5.4.3)
L	=	maximum dimension of dent, measured straight across the largest dimension of the dent (in.) (17.5.3.1) (17.5.3.2) (17.5.3.3)
N	=	total number of fatigue cycles induced by wind during the life of a structure (17.5.2.2) (17.5.2.3) (17.5.2.4)
N_{day}	=	number of fatigue cycles induced by wind per day for a given mean wind speed (17.5.2.5)
P_{eff}	=	constant amplitude effective static wind pressure range for fatigue evaluation with finite life (psf) (17.5.2.2) (17.5.2.3) (17.5.2.4)
P_{fls}	=	fatigue limit state static wind pressure range, used to check for infinite fatigue life (psf) (17.5.2.2)
t	=	thickness of tube wall (in.) (17.5.3.1) (17.5.3.2) (17.5.3.3)
V_{mean}	=	mean wind speed for location under study for fatigue evaluation (mph) (17.5.2.5)
W	=	width of dent, measured along original circumference of tube (in.) (17.5.3.1) (C17.5.3.1) (17.5.3.2) (17.5.3.3)
δ	=	depth of dent, measured relative to the original shape (in.) (17.5.3.1)

Δf = stress range at fatigue-prone detail resulting from constant-amplitude effective static wind pressure range (ksi) (17.5.2.2) (17.5.2.3) (17.5.2.4) (17.5.3.2)

$(\Delta F)_{TH}$ = constant amplitude fatigue threshold at fatigue-prone detail, for which detail has infinite life at lesser stress (ksi) (17.5.2.2)

17.3—MANAGEMENT ORGANIZATION

At a minimum, the management organization responsible for ancillary structures should include the positions outlined in Article 16.4. These positions should be staffed with personnel possessing the minimum qualifications as outlined in that section.

17.4—COMPONENTS OF AN ANCILLARY STRUCTURE FILE

17.4.1—General

Collection of inventory information and the ability to organize and categorize the inventory is of paramount importance for an ancillary structure inspection program.

C17.4.1

Typically, this inventory information is stored with the condition information in an ancillary structure file. This is similar to data pertaining to a bridge's construction and inspection history stored in a bridge file.

The ancillary structure file can be contained within physical records, but more frequently the information is stored electronically in a database. The database can house not only current and historical data on condition and inventory items, but can store images of as-built drawings, digital photos from inspections, scans of old inspection reports, video, and other media.

17.4.2—Inventory Data

For Owners who do not perform structural inspections of ancillary structures, inventory data should still be collected and recorded to document the number and type of structures. As is applicable, the following information should be recorded for each ancillary structure at a minimum:

1. Structure name
2. Owner name
3. Location description
4. Latitude/longitude (GPS coordinates)
5. County
6. Route
7. Milepost
8. Structure type
9. Material type for span
10. Material type for supports
11. Structure configuration
12. Span length
13. Structure height
14. Minimum vertical clearance
15. Minimum horizontal clearances
16. Installation year

C17.4.2

Although not essential, it is useful to have an electronic database that can show the user both the latest inventory data as well as temporally track changes in inventory, whether due to physical modifications to the ancillary structure or due to corrections of incorrectly recorded data.

17. Installation contract number
18. Number of signs/lights
19. Dimensions of signs
20. Descriptions of signs
21. Presence of plans
22. Presence of walkway
23. Presence of lighting
24. Date of rehabilitation

17.4.3—Condition Data

For Owners who perform structural inspections, all condition data enumerated in Section 16, Appendix D, and the general information below should be collected.

- Date inspected and inspection type,
- Recommended inspection frequency,
- Access equipment required for inspection,
- Traffic control requirements for inspection,
- Entity performing inspection (name of agency, consultant, etc.), and
- Inspection team leader (name).

C17.4.3

Although not essential, it is useful to have an electronic database that can show the user both the latest condition data as well as temporally track changes in condition data, whether due to time-dependent deterioration of the ancillary structure or due to corrections of incorrectly recorded data.

17.5—REPLACEMENT CONSIDERATIONS

17.5.1—General

Ancillary structures may be replaced for many of reasons including:

- Structural condition,
- Functionality,
- Roadway improvements, and
- Aesthetics.

C17.5.1

Consideration of replacement for ancillary structures is typically based on either structure condition, planned roadway improvements, or both. Due to the relatively low cost of ancillary structures compared to bridges or highway widening/resurfacing, the replacement of these structures is often coordinated with large-scale highway improvement projects.

For example, with an interchange reconstruction, all of the sign structures and high-mast lights within the project limits may be replaced even if some of the structures are in relatively good condition. The interchange will then have all new hardware and will age uniformly. If other highway improvements are not planned in the near future, then often structures will be replaced on a case-by-case basis, usually when they are in extremely poor condition and pose a danger to the motoring public. Refer to Appendix D for some of the condition states where this would be applicable.

17.5.2—Estimated Remaining Fatigue Life

17.5.2.1—General

Fatigue considerations often govern the structure's remaining life. Remaining fatigue life is based on its fatigue-prone (i.e., welded) details and may be computed as outlined below.

C17.5.2.1

The estimated remaining fatigue life presented here is based on the research from Roy et al (2011). The process is specific for high-mast lighting towers. For information and guidance on estimating remaining fatigue life for other ancillary structures, refer to *Cost-Effective Connection Details for Highway Sign, Luminaire, and Traffic Signal Structures* (NCHRP, 2011).

If complete replacement of structures at an interchange, intersection, or section of roadway is not planned in the near future, it may be desirable to estimate the remaining life to aid budget planning for their replacement at a later time.

Remaining life based on corrosion rates may be estimated if consistent thickness readings are taken during structural inspections conducted at regular intervals. This would be an estimate of remaining life based on the current and projected structural condition combined with an analysis of the cross section.

17.5.2.2—Infinite Life vs. Finite Life

The fatigue limit state static wind pressure range, used to check for infinite fatigue life is:

$P_{fls} = 5.8$ psf

This fatigue wind load should be applied to an analysis model of the ancillary structure to determine the nominal stress (i.e., based on the nominal cross-section rather than localized effects) at locations near fatigue-prone details. The resulting nominal stress computed at the location of the detail, Δf, is a stress range that can be compared with the constant amplitude fatigue threshold for each fatigue-prone detail, $(\Delta F)_{TH}$. See Table 11.9.3.1-1.

If Δf is less than or equal to $(\Delta F)_{TH}$, then the detail is considered to have an infinite fatigue life and no further analysis is required. Otherwise, the fatigue life of the detail, and thus of the structure, is finite and the total number of fatigue cycles, N, induced by wind during the lifetime of the structure may be estimated.

The constant amplitude effective static wind pressure range, used for analysis when fatigue life is finite, should be:

$P_{eff} = 1.3$ psf

Two methods may be used for computing the stress range, Δf, corresponding to the finite fatigue life: nominal stress-based design and local stress-based design. Either one is acceptable; it is not necessary to use both.

C17.5.2.2

The first step in checking for fatigue life is to determine if a structure has an infinite fatigue life based on the various fatigue details. If this is the case, then no further consideration of fatigue will be necessary and the useful life of the structure may only be affected by its condition (e.g., deterioration due to corrosion or impact damage).

17.5.2.3—Finite Life with Nominal Stress-Based Design

The constant amplitude static wind load P_{eff} should be applied to an analysis model to determine the nominal stress (i.e., based on the nominal cross-section rather than localized effects) at locations near fatigue-prone details. The resulting nominal stress computed at the location of the detail, Δf, is a stress range that can be used in conjunction with the fatigue detail category constant, A, for each fatigue-prone detail, found in Table 11.9.3.1-1. The total number of fatigue cycles induced by wind during the lifetime of the structure may be computed as:

$$N = A/(\Delta f)^3 \tag{17.5.2.3-1}$$

17.5.2.4—Finite Life with Local Stress-Based Design

As an alternative to the nominal stress-based design of the preceding section, welded connections in support structures may be evaluated using local stresses obtained from finite element (FE) analysis. The FE analyses may be performed using any available program that has been validated by the user.

The constant amplitude static wind load P_{eff} should be applied to the analysis model to determine the local stress (i.e., based on a three-dimensional detailed mesh to account for the localized effects) at locations near fatigue-prone details. To achieve proper local stiffness and improved stress prediction, the nominal weld geometry should be modeled. The FE model should assume linear material properties. The resulting maximum local stress at the location of the detail, Δf, is a stress range that can be used in conjunction with the following equation to compute the total number of fatigue cycles induced by wind during the lifetime of the structure:

$$N = 44\times10^{8}\ \text{ksi}^{3}/(\Delta f)^{3} \qquad (17.5.2.4\text{-}1)$$

17.5.2.5—Remaining Life

The number of fatigue cycles induced by wind per day, N_{day}, may be taken from the following table according to the mean wind speed, V_{mean}, for the structure's location.

Table 17.5.2.5-1 Stress-Range Cycles for Evaluation

Mean Wind Speed	N_{day}
$V_{mean} \leq 9$ mph	9,500
9 mph < $V_{mean} \leq 11$ mph	15,000
$V_{mean} > 11$ mph	23,000
Vortex Shedding Mitigated [a]	7,000

Note:

a. Dampers are one example of retrofit methods in which vortex shedding and vibratory effects may be mitigated.

Based on the total number of fatigue cycles, N, and the number of cycles per day, N_{day}, the total number of days of the structure's life may be estimated. Knowing the current age of the structure, the approximate number of remaining days of fatigue life may be computed.

17.5.3—Dents

Dents in steel structures categorized as small and negligible on resistance may be assessed per Article 17.5.3.1. Larger dents may be assessed for decreased resistance per Article 17.5.3.2.

C17.5.3

These criteria are based on the California Department of Transportation (Caltrans) memorandum "Dents and Un-reinforced Holes in Circular Steel Tube Posts" (2010).

Dents are sometimes present in various steel members of structures. These may be due to impact damage from errant highway vehicles, or they may have been present from the erection of the ancillary structure itself (e.g., crane pick without proper rigging). Various guidelines regarding the aspect ratio of the dent and other considerations can be used to determine whether further analysis is needed. In many cases, the effect of a small dent on the strength of the structural steel member is negligible. In other cases, the effect of a larger dent on the strength of the structural steel member can be satisfactorily approximated with reduction factors provided that the other considerations (e.g., plumbness and location of dent relative to other discontinuities) are still satisfied.

17.5.3.1—Negligible Dents in Compact/Non-Compact Circular Structural Steel Tubes for Posts or Mast Arms

Small dents in compact circular steel tubes are not likely to have a large impact on the resistance. Impact of dents on fatigue performance may be minor in many cases. Small dents may be ignored if all of the following are met:

- No cracking exists in the steel or sharply folded kinks in the dent.
- The dent is located away from other considerations such as holes and welded-on plates.
- For posts with dents: Under typical dead loads, neither out-of-plumb nor out-of-straightness of a post exceeds 2.5 percent of post length (often Owners may want to impose a tighter limit for aesthetic reasons or specialized structural or clearance issues).
- For mast arms with dents: Under typical dead loads, extra bend in the mast arm does not create unacceptable aesthetic, structural, or clearance issues.
- $\delta \leq 2t$ where δ (in.) is depth of dent measured relative to the original shape and t is thickness of tube wall (in.).
- $W \leq 0.1C$ where W is width of dent measured along original circumference of tube (in.) which has a total circumference of C (in.).
- $L < 0.2C$ where L is maximum dimension of dent measured straight across the largest dimension of the dent and C is the total circumference of the tube (in.).

C17.5.3.1

For deeper dents, the width of dent may have greater influence than the depth.

Width of dent, W, may be estimated by measuring the arc length of the non-dented side and then subtracting this from the original circumference, C.

17.5.3.2—Negligible Dents in Slender Circular Structural Steel Tubes for Posts or Mast Arms

Small dents can be ignored if all of the following are met:

- No cracking exists in the steel or sharply folded kinks in the dent.
- The dent is located away from other considerations such as holes and welded-on plates.
- For posts with dents: Under typical dead loads, neither out-of-plumb nor out-of-straightness of a post exceeds 2.5 percent of post length (often Owners may want to

impose a tighter limit for aesthetic reasons, specialized structural issues, or clearance issues).

- For mast arms with dents: Under typical dead loads, extra bend in the mast arm does not create unacceptable aesthetic, structural, or clearance issues.
- $\delta \leq t$ where δ (in.) is depth of dent measured relative to the original shape and t is thickness of tube wall (in.)
- $W \leq 0.05C$ where W is width of dent measured along original circumference of tube (in.) which has a total circumference of C (in.).

$L < 0.1C$ where L is maximum dimension of dent measured straight across the largest dimension of the dent (in.) and C is the total circumference of the tube (in.).

17.5.3.3—Large Dents

For dents that meet all of the "negligible dent" criteria except for one or more of the limitations on dent dimensions, the resistances for axial, bending, and shear specified in Section 5 may be decreased according to Tables 17.5.3.3-1 to 17.5.3.3-3, in lieu of a more detailed analysis. Linear interpolation is allowed. Resistances are based on the section properties of the cross-section prior to denting. For members not meeting the non-compact section criteria, the high-mast column in the tables may be used; however, eventual replacement is recommended for these situations.

C 17.5.3.3

Research on compact tubes suggests that if the dented portion affects more than one quarter of the circumference it may fundamentally change local buckling behavior under axial load. Use of these tables where W/C is greater than 0.20 is not recommended.

Refer to the ASCE *Journal of Structural Engineering* article "Compression Behavior of Nonslender Cylindrical Steel Members with Small and Large-Scale Geometric Imperfections" (2006). This research performed a small number of tests and included FE models on sections of steel pipe with D/t of approximately 48 and a yield strength of approximately 48 ksi. A dent with a depth of 1.8 times the wall thickness decreased axial strength by 4 percent in testing. A dent with two times the wall thickness decreased axial strength by 8 percent in FE modeling. The failure modes associated with these small dents involved a mechanism where the buckle at the dent tended to stop growing in width at approximately one quarter of the circumference (which may have been due to local arching action). In FE modeling, a dent approximately one half the circumference resulted in a different buckled shape that did not exhibit similar self-limiting behavior through local arching.

Table 17.5.3.3-1—Dent Adjustment Factors for Axial Resistance

Maximum *W/C*	Adjustment Factor for Axial Capacity[a] (Typical Ancillary Structures)	Adjustment Factor for Axial Capacity[a] (High Mast [b])
0.05	0.90	0.80
0.10	0.80	0.60
0.15	0.70	0.50
0.20	0.60	Not addressed
More than 0.20	Not addressed	Not addressed

Notes:
a. These values are conservative for tension.
b. Includes other sections not meeting non-compact criteria.

Table 17.5.3.3-2—Dent Adjustment Factors for Bending Resistance

Maximum *W/C*	Adjustment Factor for Bending Capacity[a] (Typical Ancillary Structures)	Adjustment Factor for Bending Capacity[a] (High Mast[b])
0.05	0.90	0.80
0.10	0.80	0.60
0.15	0.70	0.50
0.20	0.60	Not addressed
More than 0.20	Not addressed	Not addressed

Notes:
a. These values are conservative for tension.
b. Includes other sections not meeting non-compact criteria.

Table 17.5.3.3-3—Dent Adjustment Factors for Shear Resistance

Maximum *L/C*	Adjustment Factor for Shear Capacity (Typical Ancillary Structures)	Adjustment Factor for Shear Capacity (High Mast[a])
0.20	0.90	0.80
0.30	0.80	0.60
0.40	0.70	0.50
0.50	0.60	Not addressed
More than 0.5	Not addressed	Not addressed

Notes:
a. These values are conservative for tension.

For situations falling outside the range of the tables or where the considerations corresponding to the "negligible dent" assumption are no longer met (e.g., dents located near other discontinuities), a more detailed FE analysis should be performed to model the stress concentrations at the dent.

17.5.4—Unreinforced Holes

Unreinforced holes may be assessed per Articles 17.5.4.1 to 17.5.4.3.

C17.5.4

These criteria are based on the CalTrans memorandum "Dents and Un-reinforced Holes in Circular Steel Tube Posts" (2012).

Unreinforced holes are sometimes present in steel members. These may be access or utility holes that were added at some point during the service life, or they may have been present from before the erection. Various guidelines regarding the aspect ratio of the hole and other considerations may be used to determine whether further analysis is required. In many cases, the effect of a small hole on the strength and fatigue life is negligible. In other cases, the effect of a larger hole on the strength can be satisfactorily approximated with reduction factors provided that the other considerations (e.g., roundness of corners and location of hole relative to other discontinuities) are still satisfied.

Refer to the experimental results in “A Tube with a Rectangular Cut-Out, Part 1: Subject to Pure Bending” and “A Tube with a Rectangular Cut-Out, Part 2: Subject to Axial Compression” in the IME *Journal of Mechanical Engineering Science* (2006).

17.5.4.1—Negligible Holes in Compact/Non-Compact Circular Structural Steel Tubes for Posts or Mast Arms

Impacts of round holes on fatigue are likely to be negligible for most common structures because welded details usually have worse fatigue categories. However, for some situations, it may be advisable to investigate fatigue implications of holes more thoroughly, especially in cases of square or rectangular holes with the small corner radii or in rectangular holes where the width, *a*, measured across member, is significantly greater than the length, *b*, measured along member. Small holes usually can be ignored if all of the following are met:

- Hole does not have irregular edges, gouges, or sharp corners (corners should be rounded for straight-sided holes). In general, holes should be drilled rather than flame-cut.
- Hole is located away from other considerations like dents, other holes, welded-on plates, etc.
- $d \leq 0.1D$ for round holes, where d is diameter of a round hole (in.) and D is outside diameter of tube (in.).
- $a \leq 0.05D$ for square or rectangular holes, where a is width measured across member (in.) and D is outside diameter of tube (in.).
- $b \leq 0.05D$ for square or rectangular holes, where b is length measured along member (in.) and D is outside diameter of tube (in.).

17.5.4.2—Negligible Holes in Slender Circular Structural Steel Tubes for Posts or Mast Arms

Small holes usually can be ignored if all of the following are met:

- The hole does not have irregular edges, gouges, or sharp corners (corners should be rounded for straight-sided holes). In general, holes should be drilled rather than flame-cut.

- The hole is located away from other considerations like dents, other holes and welded-on plates.
- $d \leq 0.05D$ for round holes, where d is diameter of a round hole (in.) and D is outside diameter of tube (in.).
- $a \leq 0.05D$ for square or rectangular holes, where a is width measured across member (in.) and D is outside diameter of tube (in.).
- $b \leq 0.05D$ for square or rectangular holes, where b is length measured along member (in.) and D is outside diameter of tube (in.).

17.5.4.3—Large Holes

With larger holes, additional consideration of fatigue implications may be warranted. For holes that meet all of the "negligible unreinforced hole" criteria except for one or more of the limitations on hole dimensions, the resistances for axial, bending, and shear specified in Section 5 may be decreased according to Tables 17.5.4.3-1 to 17.5.4.3-3, in lieu of a more detailed analysis. Linear interpolation is allowed. Resistances are based on the section properties of the cross-section prior to introduction of the hole. These tables assume the rectangular hole's width, a (in.), measured across member, is not significantly greater than its length, b (in.), measured along member. For members not meeting the non-compact section criteria, the high-mast column in the tables may be used.

C17.5.4.3

Holes in tubes may have a larger impact on the ability of the tube to resist torsion or shear than a similarly sized dent.

Table 17.5.4.3-1—Hole Adjustment Factors for Axial Resistance

Maximum d/D (round) Maximum a/D and Maximum b/D (square or rectangular)	Adjustment Factor for Axial Capacity[a] (Typical Ancillary Structures)	Adjustment Factor for Axial Capacity[a] (High Mast[b])
0.15	0.85	0.70
0.30	0.70	0.60
0.45	0.60	0.50
0.60	0.50	Not addressed
More than 0.60	Not addressed	Not addressed

Notes:
a. These values are conservative for tension.
b. Includes other sections not meeting non-compact criteria.

Table 17.5.4.3-2—Hole Adjustment Factors for Bending Resistance

Maximum *d*/*D* (round) Maximum *a*/*D* and Maximum *b*/*D* (square or rectangular)	Adjustment Factor for Bending Capacity[a] (Typical Ancillary Structures)	Adjustment Factor for Bending Capacity[a] (High Mast[b])
0.15	0.90	0.80
0.30	0.80	0.60
0.45	0.70	0.50
0.60	0.60	Not addressed
More than 0.60	Not addressed	Not addressed

Notes:
a. These values are conservative for compact sections.
b. Includes other sections not meeting non-compact criteria.

Table 17.5.4.3-3—Hole Adjustment Factors for Shear Resistance

Maximum *d*/*D* (round) Maximum *a*/*D* and Maximum *b*/*D* (square or rectangular)	Adjustment Factor for Shear Capacity (Typical Ancillary Structures)	Adjustment Factor for Shear Capacity (High Mast[a])
0.20	0.90	0.80
0.30	0.80	0.60
0.40	0.70	0.50
0.50	0.60	Not addressed
More than 0.50	Not addressed	Not addressed

Note:
a. Includes other sections not meeting non-compact criteria.

For situations falling outside the range applicability or where the considerations corresponding to the “negligible hole” assumption are no longer met (e.g., holes located near other discontinuities), a more detailed FE analysis should be performed to estimate the stress concentrations and the evaluate fatigue potential at the hole.

17.6—MAINTENANCE PROGRAM

17.6.1—General

Owners should have a maintenance program that operates in conjunction with the inspection program.

C17.6.1

Although rehabilitation or major repairs are usually not viable since it will be more economical to simply replace the ancillary structure, minor repairs (e.g., patching of holes and dents) and retrofits (e.g., the addition of dampers) should be performed when necessary. In addition, regular maintenance activities (e.g., removal of overgrown vegetation at foundations, replacement of luminaire bulbs, cleaning of signs) should be programmed to mitigate future difficulties.

17.6.2—Prioritization of Work

Maintenance activities should be prioritized.

C17.6.2

Due to budgetary constraints, funding for maintenance and repairs is often minimal.

The incorporation of general, routine maintenance tasks increases the service life of the structure, but other maintenance, retrofit, and repair tasks should be programmed using a "triage" methodology in order to address the most urgent needs first.

Life-safety is the top priority, but conditions (e.g., detrimental fatigue details) can be present that, while not immediately posing a threat of structural failure, may significantly reduce the useful life of the structure.

Situations which could lead to imminent failure (e.g., cracked welds or broken anchor bolts) should receive a higher priority than serious but non-immediate items (e.g., large dents or gouges, excessive vibrations in mast arms, loose anchor bolts) which in turn should receive a higher priority than more routine items (e.g., missing hand hole covers, loose fasteners, spot loss of galvanization or paint).

17.6.3—Repairs

Many types of repairs may be performed that increase the service life and be economically viable when compared to a total structure replacement.

C17.6.3

A few of these potential repairs are listed:

Foundations—seal cracks in concrete with epoxy and repair spalls and delaminations with mortar,

Anchor rods—drill and epoxy grout new rods to supplement insufficiently embedded or fractured rods,

Gouges in tubes—grind with a transition slope and regularly inspect due to fatigue potential of the repaired area,

Corrosion in tubes—eliminate sources of moisture or acidity by regrading to eliminate excess soil; removing vegetation; cleaning up bird or animal excrement, nests, and burrows; augment areas of significant section loss with steel collars or concrete encasement,

Small cracks in tube base metal or weldment—drill holes to arrest cracks or perform hammer peening on welds.

Longitudinal splits or large cracks in tubes—wrap affected areas with fiber reinforced polymer (FRP) sheets or strips to strengthen them and transfer load, especially at welded connections, and

Vibrations—install dampers (e.g., Stockbridge damper, Wyoming strand damper).

For additional guidance on repairs, refer to the FHWA Guidelines.

17.7—REFERENCES

CalTrans. 2012. Draft Memorandum: Dents and Un-reinforced Holes in Circular Steel Tube Posts, California Department of Transportation, Sacramento, CA.

Connor, R. J., S. H. Collicott, A. M. DeSchepper, R. J. Sherman, and J. A. Ocampo. 2012. *Development of Fatigue Loading and Design Methodology for High-Mast Lighting Towers*, NCHRP Project 718. National Cooperative Highway Research Program, Washington, DC.

FHWA. 2005. *Guidelines for the Installation, Inspection, Maintenance and Repair of Structural Supports for Highway Signs, Luminaires, and Traffic Signals*, FHWA-NHI-05-036. Federal Highway Administration, U.S. Department of Transportation, Washington DC.

Roy, S., Y. C. Park, R. Sause, J. W. Fisher, and E. K. Kaufmann. 2011. *Cost Effective Connection Details for Highway Sign, Luminaire, and Traffic Signal Structures*, NCHRP Web Only Document 176 (Final Report for NCHRP Project 10-70). Transportation Research Board, National Research Council, Washington, DC.

Tutuncu, I., and T. D. O'Rourke. 2006. "Compression Behavior of Nonslender Cylindrical Steel Members with Small and Large-Scale Geometric Imperfections," *Journal of Structural Engineering*. American Society of Civil Engineers, Reston, VA. August 2006, pp. 1234–1241.

Vartdal, B. J., S. T. S. Al-Hassani, and S. J. Burley. 2006. "A Tube with a Rectangular Cut-Out, Part 1: Subject to Pure Bending, Proceedings of the Institution of Mechanical Engineers, Part C" *Journal of Mechanical Engineering Science*. Institution of Mechanical Engineers, London, U.K., May 2006, pp. 625–643.

Vartdal, B. J., S. T. S. Al-Hassani, and S. J. Burley. 2006. "A Tube with a Rectangular Cut-Out, Part 2: Subject to Axial Compression, Proceedings of the Institution of Mechanical Engineers, Part C" *Journal of Mechanical Engineering Science*, May 2006, pp. 645–652.

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