SECTION 6: STEEL STRUCTURES

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STEEL STRUCTURES

Commentary is opposite the text it annotates.

6.1—SCOPE

This Section addresses the design of steel components, splices, and connections for straight or horizontally-curved beam and girder structures, frames, trusses and arches, cable-stayed and suspension systems, and metal deck systems, as applicable.

When applied to curved steel girders, these provisions shall be taken to apply to the design and construction of highway superstructures with horizontally-curved steel I-shaped or single-cell box-shaped longitudinal girders with radii greater than 100 ft. Exceptions to this limit shall be based on a thorough evaluation of the application of the bridge under consideration consistent with structural fundamentals.

An outline of the steps for the design of steel girder bridges is presented in Appendix C6.

C6.1

The LRFD provisions have no span limit. There has been a history of construction problems associated with curved bridges with spans greater than about 350 ft. Large girder self-weight may cause critical stresses and deflections during erection when the steel work is incomplete. Large lateral deflections and girder rotations associated with longer spans tend to make it difficult to fit up cross-frames. Large curved steel bridges have been built successfully; however, these bridges deserve special considerations such as the possible need for more than one temporary support in large spans.

Most of the provisions for proportioning main elements are grouped by structural action:

- Tension and combined axial tension, flexure, and flexural and/or torsional shear (Article 6.8)
- Compression and combined axial compression, flexure, and flexural and/or torsional shear (Article 6.9)
- Flexure, flexural shear, and torsion:
    - I-sections (Article 6.10)
    - Composite box sections (Article 6.11)
    - Noncomposite box sections and other miscellaneous sections (Article 6.12)

Provisions for connections and splices are contained in Article 6.13.

Article 6.14 contains provisions specific to particular assemblages or structural types, e.g., through-girder spans, trusses, orthotropic deck systems, and arches.

For certain types of steel structures, benefits may be gained by applying advanced analysis methods for the design of the structure and/or its components. Using these methods, the member and structure stability are assessed using a second-order analysis directly considering initial geometric imperfections and residual stress effects. These methods provide greater rigor for consideration of innovative structural systems and member geometries. In addition, they provide capabilities for recognizing reserve capacities not addressed by the Section 6 provisions. Using these procedures, the members may be checked for their local "cross-section level" resistance given refined estimates of the internal strength demands as influenced by the member and overall system stability effects. These types of capabilities typically would be applied by focusing on a limited set of potentially critical factored design load combinations. Hendy and Murphy (2007) discuss the application of these types of methods in the context of steel bridge design according to the Eurocodes. Advanced analysis methods are an area of continued evolution as computer hardware and software continue to grow in their power and capabilities. Generally, advanced analysis procedures must be calibrated to established physical test results considering appropriate nominal initial geometric imperfections and residual stresses.

6.2—DEFINITIONS

Abutment—An end support for a bridge superstructure.

Aspect Ratio—In any rectangular configuration, the ratio of the lengths of the sides.

Beam—A structural member whose primary function is to transmit loads to the support primarily through flexure and shear. Generally, this term is used when the component is made of rolled shapes.

Beam-Column—A structural member whose primary function is to resist both axial loads and bending moments.

Bend-Buckling Resistance—The maximum load that can be carried by a web plate without experiencing theoretical elastic local buckling due to bending.

Biaxial Bending—Simultaneous bending of a member or component about two perpendicular axes.

Bifurcation—The phenomenon whereby an ideally straight or flat member or component under compression may either assume a deflected position or may remain undeflected, or an ideally straight member under flexure may either deflect and twist out-of-plane or remain in its in-plane deflected position.

Bifurcation Analysis—An analysis used to determine the buckling or bifurcation load.

Block Shear Rupture—Failure of a bolted web connection of coped beams or any tension connection by the tearing out of a portion of a plate along the perimeter of the connecting bolts.

Bolt Assembly—The bolt, nut(s), and washer(s).

Box Flange—A flange that is connected to two webs. The flange may be a flat unstiffened plate, a stiffened plate or a flat plate with reinforced concrete attached to the plate with shear connectors.

Bracing Member—A member intended to brace a main member or part thereof against lateral movement.

Buckling Load—The load at which an ideally straight member or component under compression assumes a deflected position.

Built-Up Member—A member made of structural steel elements that are welded, bolted, or riveted together.

Charpy V-Notch Impact Requirement—The minimum energy required to be absorbed in a Charpy V-notch test conducted at a specified temperature.

Charpy V-Notch Test—An impact test complying with AASHTO T 243M/T 243 (ASTM A673/A673M).

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

Clear Distance of Bolts—The distance between edges of adjacent bolt holes.

Clear End Distance of Bolts—The distance between the edge of a bolt hole and the end of a member.

Closed-Box Section—A flexural member having a cross-section composed of two vertical or inclined webs which has at least one completely enclosed cell. A closed-section member is effective in resisting applied torsion by developing shear flow in the webs and flanges.

Collapse Load—That load that can be borne by a structural member or structure just before failure becomes apparent.

Compact Flange—For a composite section in negative flexure or a noncomposite section, a discretely braced compression flange with a slenderness at or below which the flange can sustain sufficient strains such that the maximum potential flexural resistance is achieved prior to flange local buckling having a statistically significant influence on the response, provided that sufficient lateral bracing requirements are satisfied to develop the maximum potential flexural resistance.

Compact Section—A composite section in positive flexure satisfying specific steel grade, web slenderness and ductility requirements that is capable of developing a nominal resistance exceeding the moment at first yield, but not to exceed the plastic moment.

Compact Unbraced Length—For a composite section in negative flexure or a noncomposite section, the limiting unbraced length of a discretely braced compression flange at or below which the maximum potential flexural resistance can be achieved prior to lateral–torsional buckling having a statistically significant influence on the response, provided that sufficient flange slenderness requirements are satisfied to develop the maximum potential flexural resistance.

Compact Web—For a composite section in negative flexure or a noncomposite section, a web with a slenderness at or below which the section can achieve a maximum flexural resistance equal to the plastic moment prior to web bend-buckling having a statistically significant influence on the response, provided that sufficient steel grade, ductility, flange slenderness and/or lateral bracing requirements are satisfied.

Component—A constituent part of a structure or member.

Composite Beam—A steel beam connected to a deck so that they respond to force effects as a unit.

Composite Column—A structural compression member consisting of either structural shapes embedded in concrete, or a steel tube filled with concrete designed to respond to force effects as a unit.

Composite Concrete-Filled Steel Tube (CFST)—A composite member consisting of a circular steel tube and concrete fill, which may be used for piers, columns, piles, or drilled shafts.

Composite Girder—A steel flexural member connected to a concrete slab so that the steel element and the concrete slab, or the longitudinal reinforcement within the slab, respond to force effects as a unit.

Connection—A weld or arrangement of bolts that transfers normal and/or shear stresses from one element to another.

Constant Amplitude Fatigue Threshold—The nominal stress range below which a particular detail can withstand an infinite number of repetitions without fatigue failure.

Contiguous Cross-Frames/Diaphragms—Intermediate cross-frames or diaphragms arranged in a continuous line across an entire I-girder bridge cross-section.

Continuously Braced Flange—A flange encased in concrete or anchored by shear connectors for which flange lateral bending effects need not be considered. A continuously braced flange in compression is also assumed not to be subject to local or lateral–torsional buckling.

Cracked Section—A composite section in which the concrete is assumed to carry no tensile stress.

Critical Load—The load at which bifurcation occurs as determined by a theoretical stability analysis.

Cross-Frame—A transverse truss framework connecting adjacent longitudinal flexural components or inside a tub section or closed box used to transfer and distribute vertical and lateral loads and to provide stability to the compression flanges. Sometimes synonymous with the term diaphragm.

Cross-Section Distortion—Change in shape of the cross-section profile due to torsional loading.

Curved Girder—An I-, closed-box, or tub girder that is curved in a horizontal plane.

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Deck—A component, with or without wearing surface, that supports wheel loads directly and is supported by other components.

Deck System—A superstructure, in which the deck is integral with its supporting components, or in which the effects of deformation of supporting components on the behavior of the deck is significant.

Deck Truss—A truss system in which the roadway is at or above the level of the top chord of the truss.

Detail Category—A grouping of components and details having essentially the same fatigue resistance.

Diaphragm—A vertically oriented solid transverse member connecting adjacent longitudinal flexural components or inside a closed-box or tub section to transfer and distribute vertical and lateral loads and to provide stability to the compression flanges.

Discontinuous Cross-Frames/Diaphragms—Intermediate cross-frames or diaphragms arranged in a discontinuous line across an I-girder bridge cross-section.

Discretely Braced Flange—A flange supported at discrete intervals by bracing sufficient to restrain lateral deflection of the flange and twisting of the entire cross-section at the brace points.

Distortion-Induced Fatigue—Fatigue effects due to secondary stresses not normally quantified in the typical analysis and design of a bridge.

Edge Distance of Bolts—The distance perpendicular to the line of force between the center of a hole and the edge of the component.

Effective Area—The reduced cross-sectional area of a member containing slender cross-section elements and/or any longitudinally stiffened plates to account for the influence of local buckling and postbuckling on the overall member axial compressive resistance, or the reduced cross-sectional area of a longitudinally stiffened compression flange in a flexural member to account for the post-buckling resistance of the flange element.

Effective Length—The equivalent length KL used in compression formulas and determined by a bifurcation analysis.

Effective Length Factor—The ratio between the effective length and the unbraced length of the member measured between the centers of gravity of the bracing members.

Effective Net Area—Net area modified to account for the effect of shear lag.

Effective Span—A defined span length used in the calculation of shear-lag effects.

Effective Width—The reduced width of a plate or concrete slab which, with an assumed uniform stress distribution, produces the same effect on the behavior of a structural member as the actual plate width with its nonuniform stress distribution.

Elastic—A structural response in which stress is directly proportional to strain and no deformation remains upon removal of loading.

Elastic Analysis—Determination of load effects on members and connections based on the assumption that the material stress–strain response is linear and the material deformation disappears on removal of the force that produced it.

Elastic-Perfectly Plastic (Elastic-Plastic)—An idealized material stress–strain curve that varies linearly from the point of zero strain and zero stress up to the yield point of the material, and then increases in strain at the value of the yield stress without any further increases in stress.

End Distance of Bolts—The distance along the line of force between the center of a hole and the end of the component.

End Panel—The end section of a truss or a web panel adjacent to the discontinuous end of a girder.

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Engineer—A licensed structural engineer responsible for the design of the bridge or review of the bridge construction.

Eyebar—A tension member with a rectangular section and enlarged ends for a pin connection.

Factored Load—The product of the nominal load and a load factor.

Fastener—Generic term for welds, bolts, rivets, or other connecting devices.

Fatigue—The initiation and/or propagation of cracks due to a repeated variation of normal stress with a tensile component.

Fatigue Design Life—The number of years that a detail is expected to resist the assumed traffic loads without fatigue cracking. In the development of these Specifications, it has been taken as 75 years.

Fatigue Life—The number of repeated stress cycles that results in fatigue failure of a detail.

Fatigue Resistance—The maximum stress range that can be sustained without failure of the detail for a specified number of cycles.

Finite Fatigue Life—The number of cycles to failure of a detail when the maximum probable stress range exceeds the constant amplitude fatigue threshold.

First-Order Analysis—Analysis in which equilibrium conditions are formulated on the undeformed structure; that is, the effect of deflections is not considered in writing equations of equilibrium.

Fit Condition—The deflected girder geometry in which the cross-frames or diaphragms are detailed to connect to the girders.

Flange Extensions—Extensions of the flange(s) of a box-section member, measured from the outside surface of the webs, to allow for welding of the flange(s) to the webs.

Flange Lateral Bending—Bending of a flange about an axis perpendicular to the flange plate due to lateral loads applied to the flange and/or nonuniform torsion in the member.

Flange Local Buckling (FLB)— Limit state of buckling of a compression flange within a cross-section.

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 without a change in cross-sectional shape.

Force—Resultant of distribution of stress over a prescribed area. Generic term signifying axial loads, bending moment, torques, and shears.

Fracture-Critical Member (FCM)—A steel primary member or portion thereof subject to tension whose failure would probably cause a portion of or the entire bridge to collapse.

Fracture Toughness—A measure of the ability of a structural material or element to absorb energy without fracture. It is generally determined by the Charpy V-notch test.

Gauge of Bolts—The distance between adjacent lines of bolts; the distance from the back of an angle or other shape to the first line of bolts.

Girder—A structural component whose primary function is to resist loads in flexure and shear. Generally, this term is used for fabricated sections.

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Global Lateral–Torsional Buckling—Buckling mode in which a system of girders buckle as a unit with an unbraced length equal to the clear span of the girders.

Grip—Distance between the nut and the bolt head.

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

Half Through-Truss Spans—A truss system with the roadway located somewhere between the top and bottom chords. It precludes the use of a top lateral system.

HSS—A square, rectangular, or hollow structural steel section produced in accordance with a pipe or tubing product specification.

Hybrid Section—A fabricated steel section with a web that has a specified minimum yield strength lower than one or both flanges.

Inelastic Action—A condition in which deformation is not fully recovered upon removal of the load that produced it.

Inelastic Redistribution—The redistribution of internal force effects in a component or structure caused by inelastic deformations at one or more sections.

Instability—A condition reached in the loading of a component or structure in which continued deformation results in a decrease of load-resisting capacity.

Interior Panel—The interior section of a truss or a web panel not adjacent to the discontinuous end of a girder.

Joint—Area where two or more ends, surfaces, or edges are attached. Categorized by type of fastener used and method of force transfer.

Lacing—Plates or bars to connect components of a member.

Lateral Bending Stress—The normal stress caused by flange lateral bending.

Lateral Bracing—A truss placed in a horizontal plane between two I-girders or two flanges of a tub girder to maintain cross-sectional geometry, and provide additional stiffness and stability to the bridge system.

Lateral Bracing Component—A component utilized individually or as part of a lateral bracing system to prevent buckling of components and/or to resist lateral loads.

Lateral Connection Plate—A plate used to interconnect lateral bracing members for attachment to a flexural member.

Lateral–Torsional Buckling (LTB)—Buckling of a component involving lateral deflection and twist.

Level—That portion of a rigid frame that includes one horizontal member and all columns between that member and the base of the frame or the next lower horizontal member.

Limit State—A condition in which a component or structure becomes unfit for service and is judged either to be no longer useful for its intended function or to be unsafe. Limits of structural usefulness include brittle fracture, plastic collapse, excessive deformation, durability, fatigue, instability, and serviceability.

Load Effect—Moment, shear, axial force or torque induced in a member by loads applied to the structure.

Load Path—A succession of components and joints through which a load is transmitted from its origin to its destination.

Load-Induced Fatigue—Fatigue effects due to the in-plane stresses for which components and details are explicitly designed.

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Local Buckling—The buckling of a plate element in compression.

Locked-In Forces—The internal forces induced into a I-girder structural system when Steel Dead Load Fit (SDLF) or Total Dead Load Fit (TDLF) detailing is employed. These internal forces are caused by the lack-of-fit detailed between the cross-frames and the girders in the base fully-cambered No-Load (NL) geometry. These internal forces would remain if the structure's dead loads were theoretically removed.

Longitudinally Loaded Weld—Weld with applied stress parallel to the longitudinal axis of the weld.

Longitudinally Stiffened Plate Panel—The portion of a longitudinally stiffened plate bounded in the width direction by the centerlines of individual longitudinal stiffeners or the centerline of a longitudinal stiffener and the inside of the laterally-restrained longitudinal edge of a longitudinally stiffened plate, and bounded in the length direction by diaphragms and/or transverse stiffeners.

Longitudinal Stiffener—A stiffener attached along the length of a component plate of a member to provide additional local and overall compressive resistance to that component.

Major Axis—The centroidal axis about which the moment of inertia is a maximum; also referred to as the "major principal axis."

Net Tensile Stress—The algebraic sum of two or more stresses in which the total is tension.

No-Load Fit (NLF) Detailing—A method of detailing in which the cross-frames or diaphragms are detailed such that their connection work points fit with the corresponding work points on the girders without any force-fitting, with the girders assumed erected in their fully-cambered (plumb) geometry under zero load. NLF detailing is also synonymously referred to as "fully-cambered fit detailing."

Noncompact Flange—For a composite section in negative flexure or a noncomposite section, a discretely braced compression flange with a slenderness at or below the limit at which localized yielding within the member cross-section associated with a hybrid web, residual stresses and/or cross-section monosymmetry has a statistically significant effect on the nominal flexural resistance.

Noncompact Section—A composite section in positive flexure for which the nominal resistance is not permitted to exceed the moment at first yield.

Noncompact Unbraced Length—For a composite section in negative flexure or a noncomposite section, the limiting unbraced length of a discretely braced compression flange at or below the limit at which the onset of yielding in either flange of the cross-section with consideration of compression–flange residual stress effects has a statistically significant effect on the nominal flexural resistance.

Noncompact Web—For a composite section in negative flexure or a noncomposite section, a web satisfying steel grade requirements and with a slenderness at or below the limit at which theoretical elastic web bend-buckling does not occur for elastic stress values, computed according to beam theory, smaller than the limit of the nominal flexural resistance.

Noncomposite Section—A steel beam where the deck is not connected to the steel section by shear connectors.

Nonslender Cross-Section Element—A longitudinally unstiffened plate element within the cross-section of a member having a slenderness small enough such that the element is able to develop its full nominal yield strength in uniform axial compression without any reduction due to local buckling.

Nonuniform Torsion—An internal resisting torsion in thin-walled sections, also known as warping torsion, producing shear stress and normal stresses, and under which cross-sections do not remain plane. Members developing nonuniform torsion resist the externally applied torsion by warping torsion and St. Venant torsion. Each of these components of internal resisting torsion varies along the member length, although the externally applied concentrated torsion may be uniform along the member between two adjacent points of torsional restraint. Warping torsion is dominant over St. Venant torsion in members having open cross sections, whereas St. Venant torsion is dominant over warping torsion in members having closed cross sections.

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Open Section—A flexural member having a cross-section which has no enclosed cell. An open-section member resists torsion primarily by nonuniform torsion, which causes normal stresses at the flange tips.

Orthotropic Deck—A deck made of a steel plate stiffened with open or closed steel ribs welded to the underside of a steel plate.

Permanent Deflection—A type of inelastic action in which a deflection remains in a component or system after the load is removed.

Phased Construction—Construction in which a bridge is built in separate units with a longitudinal construction joint between them.

Pier—A column or connected group of columns or other configuration designed to be an interior support for a bridge superstructure.

Pitch—The distance between the centers of adjacent bolt holes or shear connectors along the line of force.

Plastic Analysis—Determination of load effects on members and connections based on the assumption of rigid-plastic behavior; i.e., that equilibrium is satisfied throughout the structure and yield is not exceeded anywhere. Second-order effects may need to be considered.

Plastic Hinge—A yielded zone which forms in a structural member when the plastic moment is attained. The beam is assumed to rotate as if hinged, except that the plastic moment capacity is maintained within the hinge.

Plastic Moment—The resisting moment of a fully-yielded cross-section.

Plastic Strain—The difference between total strain and elastic strain.

Plastic Stress Distribution Method (PSDM)—An equilibrium method using full yield strength of the steel in tension and compression, and a uniform concrete stress distribution with a magnitude of stress equal to $0.95f'_c$ over the entire compressive region, to determine a nominal material-based P-M interaction curve for CFST members without consideration of buckling.

Plastification—The process of successive yielding of fibers in the cross-section of a member as bending moment is increased.

Plate—A flat rolled product whose thickness exceeds 0.25 in.

Plateau Strength—The maximum potential flexural resistance of a compression flange based on either flange local buckling (FLB) or lateral–torsional buckling (LTB) defined by a plateau at flange slenderness values less than or equal to λ_{pf} or at unbraced lengths less than or equal to L_p, respectively.

Portal Frames—End transverse truss bracing or Vierendeel bracing to provide for stability and to resist wind or seismic loads.

Post-Buckling Resistance—The load that can be carried by a member or component after buckling.

Primary Member—A steel member or component that transmits gravity loads through a necessary as-designed load path. These members are therefore subjected to more stringent fabrication and testing requirements; considered synonymous with the term "main member."

Prismatic Member—A member having a constant cross section along its length.

Prying Action—Lever action that exists in connections in which the line of application of the applied load is eccentric to the axis of the bolt, causing deformation of the fitting and an amplification of the axial force in the bolt.

Redistribution Moment—An internal moment caused by yielding in a continuous span bending component and held in equilibrium by external reactions.

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Redistribution of Moments—A process that results from formation of inelastic deformations in continuous structures.

Redistribution Stress—The bending stress resulting from the redistribution moment.

Redundancy—The quality of a bridge that enables it to perform its design function in a damaged state.

Redundant Member—A member whose failure does not cause failure of the bridge.

Required Fatigue Life—A product of the single-lane average daily truck traffic, the number of cycles per truck passage, and the design life in days.

Residual Stress—The stresses that remain in an unloaded member or component after it has been formed into a finished product by cold bending, and/or cooling after rolling or welding.

Reverse Curvature Bending—A bending condition in which end moments on a member cause the member to assume an S shape.

Rigid Frame—A structure in which connections maintain the angular relationship between beam and column members under load.

St. Venant Torsion—That portion of the internal resisting torsion in a member producing only pure shear stresses on a cross section, also referred to as "pure torsion" or "uniform torsion."

Second-Order Analysis—Analysis in which equilibrium conditions are formulated on the deformed structure; that is, in which the deflected position of the structure is used in writing the equations of equilibrium.

Secondary Member—A steel member or component that does not transmit gravity loads through a necessary as-designed load path.

Service Loads—Loads expected to be supported by the structure under normal usage.

Shape Factor—The ratio of the plastic moment to the yield moment, or the ratio of the plastic section modulus to the elastic section modulus.

Shear–Buckling Resistance—The maximum load that can be supported by a web plate without experiencing theoretical buckling due to shear.

Shear Connector—A mechanical device that prevents relative movements both normal and parallel to an interface.

Shear Flow—Shear force per unit width acting parallel to the edge of a plate element.

Shear Lag—Nonlinear distribution of normal stress across a component due to shear distortions.

Sheet—A flat rolled product whose thickness is between 0.006 and 0.25 in.

Single Curvature Bending—A deformed shape of a member in which the center of curvature is on the same side of the member throughout the unbraced length.

Skew Angle—The angle between the axis of support relative to a line normal to the longitudinal axis of the bridge, i.e. a zero-degree skew denotes a rectangular bridge.

Slab—A deck composed of concrete and reinforcement.

Slender Cross-Section Element—A longitudinally unstiffened plate element within the cross-section of a member having a slenderness large enough such that nominal local buckling of the element may occur and may have an impact on its resistance in uniform axial compression prior to developing its full nominal yield strength.

Slender Flange—For a composite section in negative flexure or a noncomposite section, a discretely braced compression flange with a slenderness at or above which the nominal flexural resistance is governed by elastic flange local buckling, provided that sufficient lateral bracing requirements are satisfied.

Slender Unbraced Length—For a composite section in negative flexure or a noncomposite section, the limiting unbraced length of a discretely braced compression flange at or above which the nominal flexural resistance is governed by elastic lateral–torsional buckling.

Slender Web—For a composite section in negative flexure or a noncomposite section, a web with a slenderness at or above which the theoretical elastic bend-buckling stress in flexure is reached in the web prior to reaching the yield strength of the compression flange.

Slenderness Ratio—The ratio of the effective length of a member to the radius of gyration of the member cross-section, both with respect to the same axis of bending, or the full or partial width or depth of a component divided by its thickness.

Splice—A group of bolted connections, or a welded connection, sufficient to transfer the moment, shear, axial force, or torque between two structural elements joined at their ends to form a single, longer element.

Stay-in-Place Formwork—Permanent metal or precast concrete forms that remain in place after construction is finished.

Staged Deck Placement—Placement of a concrete bridge deck in successive stages with a longitudinal and/or transverse construction joint between them.

Steel Dead Load Fit (SDLF) Detailing—A method of detailing in which the cross-frames or diaphragms are detailed such that their connection work points fit with the corresponding work points on the girders with the steel dead load vertical deflections and the associated girder major-axis rotations at the connection plates subtracted from the fully-cambered geometry of the girders, and with the girder webs assumed in an ideal plumb position under the Steel Dead Load (SDL) at the completion of the steel erection. SDLF detailing is also synonymously referred to as “erected-fit detailing.”

Stiffener—A member, usually an angle or plate, attached to a plate or web of a beam or girder to distribute load, to transfer shear, or to prevent buckling of the member to which it is attached.

Stiffness—The resistance to deformation of a member or structure measured by the ratio of the applied force to the corresponding displacement.

Strain Compatibility Method (SCM)—An equilibrium method using strain compatibility-based stress distributions in concrete and steel to determine a nominal material-based P-M interaction curve of CFST members without consideration of buckling.

Strain Hardening—Phenomenon wherein ductile steel, after undergoing considerable deformation at or just above the yield point, exhibits the capacity to resist substantially higher loading than that which caused initial yielding.

Strain-Hardening Strain—For structural steels that have a flat or nearly flat plastic region in the stress–strain relationship, the value of the strain at the onset of strain hardening.

Stress Range—The algebraic difference between extreme stresses resulting from the passage of a load.

Strong-Axis—The centroidal axis about which the moment of inertia is a maximum.

Subpanel—A stiffened web panel divided by one or more longitudinal stiffeners.

Sway Bracing—Transverse vertical bracing between truss members.

System Redundant Member (SRM)—A steel primary member or portion thereof subject to tension for which the redundancy is not known by engineering judgment, but which is demonstrated to have redundancy through a refined analysis. SRMs must be identified and designated as such by the Engineer on the contract plans, and designated in the contract documents to be fabricated according to Clause 12 of the AASHTO/AWS D1.5M/D1.5 *Bridge Welding Code*. An SRM need not be subject to the hands-on in-service inspection protocol for an FCM as described in 23 CFR 650.

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Tensile Strength—The maximum tensile stress that a material is capable of sustaining.

Tension-Field Action—The behavior of a girder panel under shear in which diagonal tensile stresses develop in the web and compressive forces develop in the transverse stiffeners in a manner analogous to a Pratt truss.

Through-Girder Spans—A girder system where the roadway is below the top flange.

Through-Thickness Stress—Bending stress in a web or box flange induced by distortion of the cross section.

Through-Truss Spans—A truss system where the roadway is located near the bottom chord and where a top chord lateral system is provided.

Tie Plates—Plates used to connect components of a member.

Tied Arch—An arch in which the horizontal thrust of the arch rib is resisted by a horizontal tie.

Toe of the Fillet—Termination point of a fillet weld or a rolled section fillet.

Torsional Buckling—A buckling mode in which a compression member twists about its shear center.

Torsional Shear Stress—Shear stress induced by St. Venant torsion.

Total Dead Load Fit (TDLF) Detailing—A method of detailing in which the cross-frames or diaphragms are detailed such that their connection work points fit with the corresponding work points on the girders with the total dead load vertical deflections and the associated girder major-axis rotations at the connection plates subtracted from the fully-cambered geometry of the girders, and with the girder webs assumed in an ideal plumb position under the Total Dead Load (TDL). TDLF detailing is also synonymously referred to as "final-fit detailing."

Transverse Connection Plate—A vertical stiffener attached to a beam or girder to which a cross-frame, diaphragm, floorbeam, or stringer is connected.

Transverse Stiffener—A stiffener attached to a component plate normal to the longitudinal axis of the member to provide additional shear or axial compressive resistance.

Transversely Loaded Weld—Weld with applied stress perpendicular to the longitudinal axis of the weld.

Trough-Type Box Section—A U-shaped section without a common top flange.

True Arch—An arch in which the horizontal component of the force in the arch rib is resisted by an external force supplied by its foundation.

Tub Section—An open-topped steel girder which is composed of a bottom flange plate, two inclined or vertical web plates, and an independent top flange attached to the top of each web. The top flanges are connected with lateral bracing members.

Unbraced Length—Distance between brace points resisting the mode of buckling or distortion under consideration; generally, the distance between panel points or brace locations.

Von Mises Yield Criterion—A theory which states that the inelastic action at a point under a combination of stresses begins when the strain energy of distortion per unit volume is equal to the strain energy of distortion per unit volume in a simple tensile bar stressed to the elastic limit under a state of uniaxial stress. This theory is also called the "maximum strain-energy-of-distortion theory." Accordingly, shear yield occurs at 0.58 times the yield strength.

Warping Stress—Normal stress induced in the cross section by warping torsion and/or by distortion of the cross section.

Warping Torsion—That portion of the total resistance to torsion in a member producing shear and normal stresses that is provided by resistance to out-of-plane warping of the cross section.

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Web Crippling—The local failure of a web plate in the immediate vicinity of a concentrated load or bearing reaction due to the transverse compression introduced by this load.

Web Panel—A length of girder web in-between adjacent transverse intermediate web stiffeners or a transverse intermediate web stiffener and a bearing stiffener. Web panels are classified as either end panels or interior panels.

Web Slenderness Ratio—The depth of a web between flanges divided by the web thickness.

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

Yield Moment—In a member subjected to flexure, the moment at which an outer fiber first attains the yield stress.

Yield Strength—The stress at which a material exhibits a specified limiting deviation from the proportionality of stress to strain.

Yield–Stress Level—The stress determined in a tension test when the strain reaches 0.005 in. per in.

6.3—NOTATION

$2a$ = length of the non-welded root face in the direction of the thickness of the loaded plate (in.). For fillet welded connections, the quantity ($2a/t_p$) shall be taken equal to 1.0 (C6.6.1.2.5)
A = detail category constant $(\text{ksi})^3$; gross cross-sectional area of the box-section, including any longitudinal stiffeners (in.^{2}); moment arm for calculating the moment resistance of the flanges at a point of splice (in.) (6.6.1.2.5) (6.12.2.2.2e) (C6.13.6.1.3b)
A_b = projected bearing area on a pin plate (in.^{2}); area of a single reinforcing bar in a composite concrete-filled steel tube (in.^{2}); area of the bolt corresponding to the nominal diameter (in.^{2}) (6.8.7.2) (C6.12.2.3.3) (6.13.2.7)
A_{bot} = area of the bottom flange (in.^{2}) (6.10.10.1.2)
A_c = gross cross-sectional area of the corner pieces of a noncomposite box-section member (in.^{2}); cross-sectional area of concrete (in.^{2}); net cross-sectional area of the concrete in a composite concrete-filled steel tube (in.^{2}); area of the concrete deck (in.^{2}) (6.9.4.2.2a) (6.9.5.1) (6.9.6.3.2) (D6.3.2)
A_d = minimum required cross-sectional area of a diagonal member of top lateral bracing for tub sections (in.^{2}) (C6.7.5.3)
$ADTT$ = average daily truck traffic over the design life (6.6.1.2.5)
$ADTT_{SL}$ = single-lane $ADTT$ (6.6.1.2.5)
A_e = effective net area (in.^{2}); effective flange area (in.^{2}) (6.6.1.2.3) (6.13.6.1.3b)
A_{eff} = effective area of the cross section taken as the summation of the effective areas of the cross-section elements determined as specified in Article 6.9.4.2.2a or E6.1.1, or determined as specified in Article 6.9.4.2.2c for circular tubes and round HSS (in.^{2}); effective area of a longitudinally stiffened compression flange (in.^{2}) (6.9.4.2.2a) (6.12.2.2.2d)
$\left(A_{eff}\right)_{sp}$ = effective area of a longitudinally stiffened plate determined as specified in Article E6.1.3 (in.^{2}) (E6.1.1)
A_{eR} = effective tributary area of a laterally-restrained longitudinal edge of the longitudinally stiffened plate under consideration (in.^{2}) (E6.1.3)
A_{es} = effective area of an individual stiffener strut (in.^{2}) (E6.1.3)
A_f = area of the inclined bottom flange (in.^{2}); area of a box flange including longitudinal flange stiffeners (in.^{2}); sum of the area of fillers on both sides of a connecting plate (in.^{2}); area of flange transmitting a concentrated load (in.^{2}) (C6.10.1.4) (C6.11.11.2) (6.13.6.1.4) (6.13.7.2)
A_{fce} = effective compression flange area, including corners and flange extensions (in.^{2}) (6.12.2.2.2)
A_{fn} = sum of the flange area and the area of any cover plates on the side of the neutral axis corresponding to D_n in a hybrid section (in.^{2}) (6.10.1.10.1)
A_{ft} = gross tension-flange area, including corners and flange extensions (in.^{2}) (6.12.2.2.2)

A_g = gross area of a member (in.^{2}); gross cross-sectional area of the member (in.^{2}); total gross cross-sectional area of the member (in.^{2}); gross cross-sectional area of a tube (in.^{2}); gross area of the tension flange (in.^{2}); gross cross-sectional area of the effective Whitmore section determined based on 30 degree dispersion angles (in.^{2}); gross area of the section based on the design wall thickness (in.); gross area of the flange under consideration (in.^{2}); gross area of all plates in the cross section intersecting the spliced plane (in.^{2}) (6.6.1.2.3) (6.8.2.1) (6.9.4.2.2a) (6.9.4.2.2c) (6.10.1.8) (6.12.1.2.3b) (6.13.6.1.3b) (6.14.2.8.4) (6.14.2.8.6)

A_{gf} = gross area of the tension flange (in.^{2}) (6.8.2.3.3)

A_{gR} = gross area of a laterally-restrained longitudinal edge of a longitudinally stiffened plate (in.^{2}) (E6.1.3)

A_{gs} = gross area of an individual stiffener strut (in.^{2}) (E6.1.3)

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

A_{nc} = projected area of concrete for a single stud shear connector or group of connectors approximated from the base of a rectilinear geometric figure that results from projecting the failure surface outward $1.5h_h$ from the centerline of the single connector or, in the case of a group of connectors, from a line through a row of adjacent connectors (in.^{2}) (6.16.4.3)

A_{nco} = projected area of concrete failure for a single stud shear connector based on the concrete breakout resistance in tension (in.^{2}) (6.16.4.3)

A_{nf} = net area of the tension flange determined as specified in Article 6.8.3 (in.^{2}) (6.8.2.3.3)

A_o = enclosed area within a box section (in.^{2}); cross-sectional area enclosed by the mid-thickness of the walls of a box-section member (in.^{2}) (C6.7.4.3) (6.12.2.2.2e)

A_p = smaller of either the connected plate area on the side of the connection with the filler or the sum of the splice plate area on both sides of the connected plate (in.^{2}) (6.13.6.1.4)

A_{pn} = area of the projecting elements of a stiffener outside of the web-to-flange welds but not beyond the edge of the flange (in.^{2}) (6.10.11.2.3)

A_r = total cross-sectional area of the longitudinal reinforcement (in.^{2}) (6.9.5.1)

A_{rb} = area of the bottom layer of longitudinal reinforcement within the effective concrete deck width (in.^{2}) (D6.1)

A_{rs} = total area of the longitudinal reinforcement within the effective concrete deck width (in.^{2}) (D6.3.2)

A_{rt} = area of the top layer of longitudinal reinforcement within the effective concrete deck width (in.^{2}) (D6.1)

A_s = cross-sectional area of the steel section (in.^{2}); total area of longitudinal reinforcement over the interior support within the effective concrete deck width (in.^{2}); area of the concrete deck (in.^{2}); gross area of an individual longitudinal stiffener, excluding the tributary width of the longitudinally stiffened plate under consideration (in.^{2}) (6.9.5.1) (6.10.10.3) (D6.3.2) (E6.1.3)

A_{sb} = total cross-sectional area of the internal reinforcement bars in a composite concrete-filled steel tube (in.^{2}) (6.9.6.3.2)

A_{sc} = cross-sectional area of a stud shear connector (in.^{2}) (6.10.10.4.3)

A_{st} = cross-sectional area of the steel tube in a composite concrete-filled steel tube (in.^{2}) (6.9.6.3.2)

A_t = area of the tension flange (in.^{2}) (D6.3.2)

A_{tn} = net area along the plane resisting tension stress (in.^{2}) (6.13.4)

A_v = cross-sectional area of transverse reinforcement that intercepts a diagonal shear crack in a concrete-encased shape (in.^{2}) (6.12.3.1)

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

A_{vn} = net area along the plane resisting shear stress (in.^{2}); net area of the connection element subject to shear (in.^{2}) (6.13.4) (6.13.5.3)

A_w = gross area of the webs (in.^{2}); area of the web of a steel section (in.^{2}); moment arm for calculating the horizontal force in the web for composite sections in positive flexure at a point of splice (in.) (6.12.2.2.2) (6.12.2.3.1) (C6.13.6.1.3c)

a = distance between connectors (in.); center-to-center distance between flanges of adjacent boxes in a multiple box section (in.); longitudinal spacing of transverse flange stiffeners (in.); distance from the center of a bolt to the edge of a plate subject to a tensile force due to prying action (in.); longitudinal spacing between locations of transverse stiffeners or diaphragms that provide transverse lateral restraint to a longitudinally stiffened plate (in.) (6.9.4.3.1) (6.11.2.3) (C6.11.11.2) (6.13.2.10.4) (E6.1.3)

a_{max} = largest of the longitudinal spacings to the adjacent transverse stiffeners or diaphragms providing lateral restraint to the plate (in.) (CE6.1.5.1)

a_{min} = smallest of the longitudinal spacings to the adjacent transverse stiffeners or diaphragms providing lateral restraint to the plate (in.) (E6.1.5.2)

a_{wc} = ratio of two times the web area in compression to the area of the compression flange (6.10.1.10.2) (A6.2.1)

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$2a$ = length of the nonwelded root face in the direction of the thickness of the loaded discontinuous plate element (in.) (6.6.1.2.5)

B = overall width of rectangular HSS member, measured 90 degrees to the plane of the connection (in.); factor defining the sloping straight line representing the finite-life portion of the fatigue resistance of a channel shear connector (6.8.2.2) (6.10.10.2)

b = width of a rectangular plate element (in.); width of the body of an eyebar (in.); element width as specified in Table 6.9.4.2.1-1 (in.); the smaller of d_o and D (in.); clear projecting width of the flange under consideration measured from the outside surface of the web (in.); distance between the toe of the flange and the centerline of the web (in.); width of a concrete-encased shape perpendicular to the plane of flexure (in.); distance from the center of a bolt to the toe of the fillet of a connected part (in.); unsupported width of a cross-section plate component in an arch rib (in.); width of a longitudinally unstiffened plate (in.); longitudinal stiffener plate element width (in.) (C6.7.4.3) (6.7.6.3) (6.9.4.2.1) (6.10.11.1.3) (6.12.2.2.2b) (6.12.2.2.5) (6.12.2.3.1) (6.13.2.10.4) (6.14.4.1) (E6.1.1) (E6.1.4)

b_1, b_2 = individual flange widths (in.) (C6.9.4.1.3)

b_c = full width of the compression flange (in.) (D6.1)

b_e = effective width of the element under consideration determined as specified in Article 6.9.4.2.2b for slender elements, and taken equal to b for nonslender elements (in.); effective width of a longitudinally unstiffened compression flange (in.); effective width of a longitudinally unstiffened plate (in.) (6.9.4.2.2a) (6.12.2.2.2g) (E6.1.1)

b_f = full width of the flange (in.); flange width (in.); total inside width between the plate elements providing lateral restraint to the longitudinal edges of the flange plate element under consideration (in.); for I-sections, full width of the widest flange within the field section under consideration; for tub sections, full width of the widest top flange within the field section under consideration (in.); flange width (in.). For double angles, b_f shall be taken as the sum of the widths of the outstanding legs, not including any gap in-between the angles (C6.7.4.2) (6.8.2.2) (6.9.4.5) (6.10.11.1.2) (6.12.2.2.4d)

b_{fc} = full width of the compression flange; width of the compression flange (in.); compression flange width between webs (6.10.1.10.2) (6.10.9.3.2) (6.11.8.2.2)

b_{fce} = effective width of the compression flange, including corners and flange extensions (in.) (C6.12.2.2.2c)

b_{fi} = inside width of the box section flange; for welded box sections, the clear width of the flange under consideration between the webs. For HSS, the provisions of Article 6.12.1.2.4 apply (in.) (6.12.2.2.2b)

b_{fo} = outside width of the box-section taken as the distance from the outside to the outside of the box-section webs (in.) (6.12.2.2.2b)

b_{ft} = width of the tension flange (in.); width of a box flange in tension between webs (in.); width of the tension flange, including corners and flange extensions (in.) (6.10.9.3.2) (6.11.9) (C6.12.2.2.2c)

b_ℓ = length of the longer leg of an unequal-leg angle (in.); projecting width of a longitudinal stiffener (in.) (6.9.4.4) (6.10.11.1.3)

b_m = gross width of each plate of a box-section member taken between the mid-thickness of the adjacent plates (in.) (6.12.2.2.2e)

b_s = length of the shorter leg of an unequal-leg angle (in.); effective width of the concrete deck (in.) (6.9.4.4) (6.10.1.10.2)

b_{sp} = total inside width between the plate elements providing lateral restraint to the longitudinal edges of a longitudinally stiffened plate (in.) (E6.1.3)

b_t = projecting width of a transverse or bearing stiffener (in.); width of the projecting stiffener element (in.); full width of the tension flange (in.) (6.10.11.1.2) (6.10.11.1.3) (D6.1)

b_{tfs} = smallest top-flange width within the unspliced individual girder field section under consideration (in.) (C6.10.2.2)

C = ratio of the shear-buckling resistance to the shear yield strength determined by Eqs. 6.10.9.3.2-4, 6.10.9.3.2-5 or 6.10.9.3.2-6, as applicable, using the appropriate shear-buckling coefficient; torsional constant for noncomposite circular tubes and round HSS (in.^{3}) (6.10.9.2) (6.12.1.2.3b)

C_a = smallest distance from center of stud to the edge of the concrete (in.) (6.16.4.3)

C_b = moment gradient modifier (6.10.1.6)

C_{bs} = system moment gradient modifier for the elastic global lateral–torsional buckling resistance of a span acting as a system (6.10.3.4.2)

$CFST$ = concrete-filled steel tube (6.4.1)

C_w = warping torsional constant (in.^{6}) (6.9.4.1.3)

C_1, C_2, C_3 = composite column constants specified in Table 6.9.5.1-1 (6.9.5.1)

c = distance from the centroid of the noncomposite steel section under consideration to the centroid of the compression flange (in.); distance from the neutral axis of the effective cross section to the extreme fiber

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of the compression flange (in.); distance from the center of the longitudinal reinforcement to the nearest face of a concrete-encased shape in the plane of bending (in.); one half the chord length for a given stress state in a composite concrete-filled steel tube (in.); largest distance from the neutral axis to the extreme fiber of the transverse stiffener considered in the calculation of I_t (in.) (6.10.3.4.2) (C6.12.2.2.2c) (6.12.2.3.1) (C6.12.2.3.3) (E6.1.5.2)

c_b = one half the chord length for a given stress state of a fictional tube modeling the internal reinforcement in a composite concrete-filled steel tube (in.) (C6.12.2.3.3)

c_c = distance from the neutral axis of the effective cross section to the mid-thickness of the rectangular portion of the compression flange area, considering the effective width of the slender flange and using the gross cross section for the web (in.) (C6.12.2.2.2c)

c_1 = effective width imperfection adjustment factor determined from Table 6.9.4.2.2b-1 (6.9.4.2.2b)

c_2 = effective width imperfection adjustment factor determined from Table 6.9.4.2.2b-1 (6.9.4.2.2b)

c_3 = effective width imperfection adjustment factor determined from Table 6.9.4.2.2b-1 (6.9.4.2.2b)

c_{rb} = distance from the top of the concrete deck to the centerline of the bottom layer of longitudinal concrete deck reinforcement (in.) (D6.1)

c_{rt} = distance from the top of the concrete deck to the centerline of the top layer of longitudinal concrete deck reinforcement (in.) (D6.1)

D = diameter of a pin (in.); outside diameter of a circular tube or round Hollow Structural Section (HSS) (in.); total inside width between the plate elements providing lateral restraint to the longitudinal edges of the web plate element under consideration (in.); outside diameter of the steel tube in a composite concrete-filled steel tube (in.); web depth (in.); depth of the web plate measured along the slope (in.); outside diameter of the steel tube (in.); for welded box sections, the clear distance between the flanges. For HSS, the provisions of Article 6.12.1.2.4 apply (in.); outside diameter of the tube (in.); web depth of an arch rib (in.) (6.7.6.2.1) (6.9.4.2.2c) (6.9.4.5) (6.9.6.2) (6.10.1.9.1) (6.11.9) (6.12.1.2.3b) (6.12.2.2.2b) (6.12.2.2.3) (6.14.4.2)

D' = depth at which a composite section reaches its theoretical plastic moment capacity when the maximum strain in the concrete deck is at its theoretical crushing strain (in.) (C6.10.7.3)

D_c = depth of the web in compression in the elastic range (in.) (6.10.1.9.1) (A6.1) (D6.3.1)

$DC1$ = permanent load acting on the noncomposite section (C6.10.11.3.1)

$DC2$ = permanent load acting on the long-term composite section (C6.10.11.3.1)

D_{ce} = depth of the web in compression in the elastic range, considering early nominal yielding in tension when $S_{xte} < S_{xce}$, and using the effective noncomposite box cross section based on the effective width of the compression flange, b_e, calculated as specified in Article 6.9.4.2.2 with F_{cr} taken equal to F_{yc}. For welded sections, depth from the inside of the compression flange. For HSS, the provisions of Article 6.12.1.2.4 apply (in.) (6.12.2.2.2c)

D_{cp} = depth of the web in compression at the plastic moment (in.) (6.10.6.2.2) (D6.3.2)

D_{cpe} = depth of the web in compression at the plastic moment determined using the effective box cross section based on the effective width of the compression flange. For welded sections, depth from the inside of the compression flange. For HSS, the provisions of Article 6.12.1.2.4 apply (in.) (6.12.2.2.2c)

D_n = larger of the distances from the elastic neutral axis of the cross section to the inside face of either flange in a hybrid section, or the distance from the neutral axis to the inside face of the flange on the side of the neutral axis where yielding occurs first when the neutral axis is at the mid-depth of the web (in.) (6.10.1.10.1)

D_p = distance from the top of the concrete deck to the neutral axis of the composite section at the plastic moment (in.) (6.10.7.1.2)

D_t = total depth of the composite section (in.) (6.10.7.1.2)

DW = wearing surface load (C6.10.11.3.1)

d = full nominal depth of the section (in.); section depth (in.); diameter of a stud shear connector (in.); depth of the tee or width of the double angle web leg subject to tension or compression, as applicable (in.); total depth of the tee or double angle web legs, as applicable (in.); depth of the rectangular bar (in.); depth of the member in the plane of flexure (in.); depth of the member in the plane of shear (in.); nominal diameter of a bolt (in.) (6.8.2.2) (C6.10.8.2.3) (6.10.10.2) (6.12.2.2.4c) (6.12.2.2.4e) (6.12.2.2.7) (6.12.2.3.1) (6.12.3.1) (6.13.2.4.2)

d_b = depth of a beam in a rigid frame (in.) (6.13.7.2)

d_c = depth of a column in a rigid frame (in.); distance from the plastic neutral axis to the midthickness of the compression flange used to compute the plastic moment (in.) (6.13.7.2) (D6.1)

d_{ce} = distance from the extreme fiber of the compression flange to the neutral axis of the effective section of a noncomposite box section (in.) (C6.12.2.2.2c)

d_{fs} = web depth for ribs with webs that are longitudinally unstiffened; maximum distance between the compression or the tension flange and the adjacent longitudinal stiffener for ribs with webs that are longitudinally stiffened (in.) (6.14.4.2)
d_h = depth of haunch (in.) (6.16.4.3)
d_o = transverse stiffener spacing (in.); the smaller of the adjacent web panel widths (in.) (6.10.9.3.2) (6.10.11.1.3)
d_{rb} = distance from the plastic neutral axis to the centerline of the bottom layer of longitudinal concrete deck reinforcement used to compute the plastic moment (in.) (D6.1)
d_{rt} = distance from the plastic neutral axis to the centerline of the top layer of longitudinal concrete deck reinforcement used to compute the plastic moment (in.) (D6.1)
d_s = distance from the centerline of the closest plate longitudinal stiffener or from the gauge line of the closest angle longitudinal stiffener to the inner surface or leg of the compression flange element (in.); distance from the plastic neutral axis to the midthickness of the concrete deck used to compute the plastic moment (in.) (6.10.1.9.2) (D6.1)
d_t = distance from the plastic neutral axis to the midthickness of the tension flange used to compute the plastic moment (in.) (D6.1)
d_w = distance from the plastic neutral axis to the middepth of the web used to compute the plastic moment (in.) (D6.1)
E = modulus of elasticity of steel (ksi); elastic modulus of the steel tube in a composite concrete-filled steel tube (ksi); elastic modulus of the internal steel reinforcement in a composite concrete-filled steel tube (ksi) (6.9.4.1.2) (6.9.6.2) (6.9.6.3.2)
E_c = elastic modulus of the concrete in a composite concrete-filled steel tube (ksi); modulus of elasticity of concrete (ksi) (6.9.6.3.2) (6.10.1.1.1b)
E_e = modified modulus of elasticity of steel for a composite column (ksi) (6.9.5.1)
EI_{eff} = effective composite flexural cross-sectional stiffness of a composite concrete-filled steel tube (kip-in^2) (6.9.6.3.2)
EXX = classification number for weld metal (C6.13.3.2.1)
e = the clear spacing between adjacent ribs, centerline rib plate to centerline rib plate (in.) (6.14.3.2.3)
e_p = distance between the centroid of the cross section and the resultant force perpendicular to the spliced plane in gusset plates (in.) (6.14.2.8.6)
F = force (kip) (6.16.4.2)
FCM = fracture-critical member (6.6.2.2)
F_{cb} = nominal axial compression buckling resistance of the flange under compression alone (ksi) (6.11.8.2.2)
F_{cr} = nominal compressive resistance of the member calculated from Eq. 6.9.4.2.2a-2 (ksi); elastic lateral–torsional buckling stress (ksi); flexural shear buckling resistance for noncomposite circular tubes and round HSS (ksi); elastic local buckling stress (ksi); critical stress (ksi); stress in the spliced section at the limit of usable resistance (ksi); elastic lateral–torsional buckling stress (ksi) (6.9.4.2.2b) (6.10.1.6) (6.12.1.2.3b) (6.12.2.2.3) (6.12.2.2.4e) (6.14.2.8.6) (A6.3.3)
F_{crs} = local buckling stress for the stiffener (ksi) (6.10.11.1.3)
F_{crw} = nominal web bend-buckling resistance (ksi) (6.10.1.9.1)
F_{cv} = nominal shear resistance of the cross-section element under consideration, under shear alone, calculated as specified in Article 6.9.2.2.2 (ksi); for noncomposite rectangular box-section members, including square and rectangular HSS, and for webs of I- and H-section members, the nominal shear resistance of the cross-section element under consideration, under shear alone, and for noncomposite circular tubes and round HSS, the flexural shear or the torsional shear resistance specified in Article 6.12.1.2.3b, as applicable, or where consideration of additive torsional and flexural shear stresses is required, the torsional shear resistance specified in Article 6.12.1.2.3b (ksi); torsional shear buckling resistance for circular tubes and round HSS (ksi) (6.7.4.4.3) (6.9.2.2.2) (6.12.1.2.3b)
F_{cvx}, F_{cvy} = nominal shear resistance of a cross-section element parallel to the x-axis taken as F_{cv} for that element determined as specified in Article 6.9.2.2.2; nominal shear resistance of a cross-section element parallel to the y-axis taken as F_{cv} for that element determined as specified in Article 6.9.2.2.2, respectively (ksi) (6.9.2.2.2)
F_e = modified yield stress for a composite column (ksi) (6.9.5.1)
F_{el} = elastic local buckling stress (ksi) (6.9.4.2.2b)
F_{exx} = classification strength of weld metal (ksi) (C6.13.3.2.1)
F_{fat} = radial fatigue shear range per unit length, taken as the larger of either F_{fat1} or F_{fat2} (kip/in.) (6.10.10.1.2)
F_{fat1} = radial fatigue shear range per unit length due to the effect of any curvature between brace points (kip/in.) (6.10.10.1.2)

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F_{fat2} = radial fatigue shear range per unit length due to torsion caused by effects other than curvature, such as skew (kip/in.) (6.10.10.1.2)

F_ℓ = statically equivalent uniformly distributed lateral force due to the factored loads from concrete deck overhang brackets (kip/in.) (C6.10.3.4.1)

FLB = flange local buckling (C6.10.8.2.1) (CA6.3.1) (CD6.4.1) (CD6.4.2)

F_{max} = maximum potential compression flange flexural resistance (ksi) (C6.10.8.2.1)

F_n = average stress on the gross area of a welded plate supported along two longitudinal edges (ksi); nominal flexural resistance of a flange (ksi) (C6.9.4.2.2b) (C6.10.8.2.3)

F_{nc} = nominal flexural resistance of a compression flange (ksi) (6.9.2.2.1)

$F_{nc(FLB)}$ = nominal compression flange local buckling flexural resistance (ksi) (CD6.4.1)

F_{nt} = nominal flexural resistance of a tension flange (ksi) (6.9.2.2.1)

F_p = total radial force in the concrete deck at the point of maximum positive live load plus impact moment for the design of the shear connectors at the strength limit state, taken equal to zero for straight spans or segments (kip) (6.10.10.4.2)

F_{px} = transverse seismic shear force on the deck within the span under consideration (6.16.4.2)

F_{rc} = net range of cross-frame force at the top flange (kip) (6.10.10.1.2)

F_s = vertical force on the connection between a longitudinal and a transverse flange stiffener (kip) (C6.11.11.2)

F_T = total radial force in the concrete deck between the point of maximum positive live load plus impact moment and the centerline of an adjacent interior support for the design of shear connectors at the strength limit state, taken equal to zero for straight spans or segments (kip) (6.10.10.4.2)

F_u = minimum tensile strength of steel (ksi); specified minimum tensile strength of the tension flange determined as specified in Table 6.4.1-1 (ksi); specified minimum tensile strength of a stud shear connector (ksi); specified minimum tensile strength of a connected part (ksi); tensile strength of a connected element (ksi); specified minimum tensile strength of a gusset plate (ksi) (6.4.1) (6.8.2.3.3) (6.10.10.4.3) (6.13.2.9) (6.13.5.3) (6.14.2.8.6)

F_{ub} = specified minimum tensile strength of a bolt (ksi) (6.13.2.7)

F_{vr} = factored torsional shear resistance of a box flange (ksi) (6.11.1.1)

F_w = vertical force on the connection between a transverse flange stiffener and a box section web (kip) (C6.11.11.2)

F_y = specified minimum yield strength of a pin (ksi); specified minimum yield strength of a pin plate (ksi); specified minimum yield strength of the eyebar (ksi); specified minimum yield strength of a plate element or longitudinally stiffened plate panel (ksi); specified minimum yield strength of a connected material (ksi); specified minimum yield strength of a gusset plate (ksi); specified minimum yield strength of steel (ksi); specified minimum yield strength of a longitudinally stiffened plate element (ksi); smallest specified minimum yield strength of a stiffened plate and transverse stiffener (ksi) (6.7.6.2.1) (6.7.6.3) (6.8.7.2) (6.9.4.5) (6.9.5.1) (6.13.4) (6.14.2.8.3) (E6.1.4) (E6.1.5.2)

F_{yb} = specified minimum yield strength of the internal steel reinforcement in a composite concrete-filled steel tube (ksi) (6.9.6.3.2)

F_{yc} = specified minimum yield strength of a compression flange (ksi) (6.10.1.6)

F_{yf} = specified minimum yield strength of a flange (ksi); specified minimum yield strength of the flange under consideration (ksi) (C6.10.1.3) (6.14.4.2)

F_{yr} = specified minimum yield strength of the longitudinal reinforcement (ksi); compression–flange stress at the onset of nominal yielding within the cross section, including residual stress effects but not including compression–flange lateral bending, taken as the smaller of $0.7F_{yc}$ and F_{yw}, but not less than $0.5F_{yc}$ (ksi) smaller of the compression–flange stress at the onset of nominal yielding, with consideration of residual stress effects, or the specified minimum yield strength of the web (ksi) (6.9.5.1) (6.10.8.2.2) (6.11.8.2.2)

F_{yrb} = specified minimum yield strength of the bottom layer of longitudinal concrete deck reinforcement (ksi) (D6.1)

F_{yrs} = specified minimum yield strength of the longitudinal concrete deck reinforcement (ksi) (D6.3.2)

F_{yrt} = specified minimum yield strength of the top layer of longitudinal concrete deck reinforcement (ksi) (D6.1)

F_{ys} = specified minimum yield strength of a stiffener (ksi); specified minimum yield strength of the stiffener (ksi) (6.10.11.1.3) (6.10.11.1.3)

F_{ysp} = specified minimum yield strength of a longitudinally stiffened plate (ksi) (E6.1.2)

F_{yst} = specified minimum yield strength of the steel tube in a composite concrete-filled steel tube (ksi) (6.9.6.2) (6.9.6.3.2)

F_{yt} = specified minimum yield strength of a tension flange (ksi) (6.9.2.2.1)

F_{yw} = specified minimum yield strength of the web (ksi) (6.10.11.1.3)

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f = shear flow in a box section (kip/in.) (C6.11.1.1)

f_0 = stress due to the factored loads without consideration of flange lateral bending at a brace point opposite to the one corresponding to f_2, calculated from the moment envelope value that produces the largest compression at this point in the flange under consideration, or the smallest tension if this point is never in compression; positive for compression and negative for tension (ksi) (6.10.8.2.3)

f_1 = smaller longitudinal stress at the longitudinal edges of the plate element or longitudinally stiffened plate panel at the cross section under consideration, taken as positive in compression and negative in tension (ksi); stress at the opposite end of an unbraced length from f_2 representing the intercept of the most critical assumed linear stress distribution through either f_2 and f_{mid}, or through f_2 and f_0, taken as $2f_{mid} - f_2 \geq f_0$ (ksi) (6.9.4.5) (6.10.8.2.3)

f_2 = larger longitudinal compressive stress at the longitudinal edges of the plate element or longitudinally stiffened plate panel at the cross section under consideration, taken as positive (ksi); largest compressive stress due to the factored loads without consideration of lateral bending at either end of an unbraced length calculated from the critical moment envelope value; always taken as positive unless stress is zero or tensile at both ends of the unbraced length in which case f_2 is taken as zero (ksi) (6.9.4.5) (6.10.8.2.3)

f_{bu} = largest value of the compressive stress throughout the unbraced length in the flange under consideration, calculated without consideration of flange lateral bending (ksi) (6.10.1.6)

f_{by} = stress in a box flange at an interior pier due to the factored loads caused by major-axis bending of the internal diaphragm over the bearing sole plate (ksi) (C6.11.8.1.1)

f_c = maximum longitudinal compressive stress acting on the gross cross section in the plate element or longitudinally stiffened plate panel under consideration at the service limit state and for constructibility (ksi); compression flange stress due to the Service II loads calculated without consideration of flange lateral bending (ksi); sum of the various compression flange flexural stresses caused by the different loads, i.e., $DC1$, $DC2$, DW and $LL+IM$, acting on their respective sections (ksi) (6.9.4.5) (6.10.4.2.2) (D6.3.1)

f'_c = specified minimum 28-day compressive strength of concrete (ksi); specified minimum 28-day compressive strength of the concrete in a composite concrete-filled steel tube (ksi) (6.9.5.1) (6.9.6.3.2)

f_d = shear stress in a box flange at an interior pier caused by the internal diaphragm vertical shear due to the factored loads (ksi) (C6.11.8.1.1)

f_{DC1} = compression flange stress caused by the factored permanent load applied before the concrete deck has hardened or is made composite, calculated without consideration of flange lateral bending (ksi) (6.10.1.10.2)

f_{DC2} = compression flange stress caused by the factored permanent load acting on the long-term composite section, calculated without consideration of flange lateral bending (ksi) (C6.10.11.3.1)

f_f = flange stress due to the Service II loads calculated without consideration of flange lateral bending (ksi) (6.10.4.2.2)

f_ℓ = flange lateral bending stress (ksi); second-order compression flange lateral bending stress (ksi); flange lateral bending stress due to the Service II loads (ksi); lateral bending stress in the flange under consideration at an interior-pier section (ksi) (6.10.1.6) (6.10.4.2.2) (A6.1.1) (B6.4.2.1)

$f_{\ell 1}$ = first-order compression flange lateral bending stress at a section, or the maximum first-order lateral bending stress in the compression flange throughout the unbraced length, as applicable (ksi) (6.10.1.6)

f_{LL+IM} = compression flange stress caused by the factored vehicular live load plus impact acting on the short-term composite section, calculated without consideration of flange lateral bending (ksi) (C6.10.11.3.1)

f_{mid} = stress due to the factored loads without consideration of flange lateral bending at the middle of the unbraced length of the flange under consideration, calculated from the moment envelope value that produces the largest compression at this point, or the smallest tension if this point is never in compression; positive for compression and negative for tension (ksi) (6.10.8.2.3)

f_n = normal stress in the inclined bottom flange of a variable web depth member (ksi); largest of the specified minimum yield strengths of each component included in the calculation of A_{fn} for a hybrid section when yielding occurs first in one of the components, or the largest of the elastic stresses in each component on the side of the neutral axis corresponding to D_n at first yield on the opposite side of the neutral axis (ksi) (C6.10.1.4) (6.10.1.10.1)

f_r = modulus of rupture of concrete (ksi) (6.10.1.7)

f_s = flexural stress due to the factored loads in a longitudinal web stiffener (ksi); largest of the longitudinal stresses due to the factored loads in the panels of a box flange on either side of a transverse flange stiffener (ksi) (6.10.11.3.1) (C6.11.11.2)

f_{sr} = bending stress range in the longitudinal reinforcement over an interior pier (ksi) (6.10.10.3)

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f_t = stress due to the factored loads on the gross area of a tension flange calculated without consideration of flange lateral bending (ksi); sum of the various tension-flange flexural stresses caused by the different loads, i.e., *DC*1, *DC*2, *DW*, and *LL+IM*, acting on their respective sections (ksi) (6.10.1.8) (D6.3.1)

f_v = St. Venant torsional shear stress in a box flange due to the factored loads (ksi); St. Venant torsional shear stress in the flange due to the factored loads at the section under consideration (ksi) (6.11.3.2) (6.11.7.2.2)

f_{ve} = factored shear stress due to torsion in the cross-section element under consideration (ksi); total factored shear stress due to torsion and/or flexure, as applicable, calculated in a cross-section element oriented parallel to the *x*- or *y*- axis of the cross section used in calculating the reduction factor, Δ, applied to the factored axial resistance to account for the effect of torsional and/or flexural shear in a noncomposite box-section member, circular tube, or an I- or H-section member (ksi); total factored resultant shear stress due to flexure and/or shear stress due to torsion within the wall of a noncomposite circular tube (ksi) (6.7.4.4.3) (6.9.2.2.2) (C6.9.2.2.2)

f_{vex} = total factored shear stress due to torsion and/or flexure, as applicable, calculated in a cross-section element oriented parallel to the *x*-axis of the cross section used in calculating the reduction factor, Δ_x, applied to the factored flexural resistance about the *x*-axis to account for the effect of torsional and/or flexural shear in a noncomposite box-section member, circular tube, or an I- or H-section member (ksi) (6.9.2.2.2)

f_{vey} = total factored shear stress due to torsion and/or flexure, as applicable, calculated in a cross-section element oriented parallel to the *y*-axis of the cross section used in calculating the reduction factor, Δ_y, applied to the factored flexural resistance about the *y*-axis to account for the effect of torsional and/or flexural shear in a noncomposite box-section member, circular tube, or an I- or H-section member (ksi); factored flexural shear stress in the web of an I- or H-section member (ksi) (6.9.2.2.2) (C6.9.2.2.2)

f_{vex1}, f_{vex2} = factored flexural shear stresses parallel to the *x*-axis within the component plates of a noncomposite box-section member, or in the flanges of an I- or H-section member (ksi) (C6.9.2.2.2)

f_{vey1}, f_{vey2} = factored flexural shear stresses parallel to the *y*-axis within the component plates of a noncomposite box-section member (ksi) (C6.9.2.2.2)

f_{xx} = various compression flange flexural stresses caused by the different factored loads, i.e., *DC*1, *DC*2, *DW*, and *LL+IM,* acting on their respective sections (ksi) (C6.10.11.3.1)

G = shear modulus of elasticity for steel = 0.385*E* (ksi) (6.9.4.1.3) (E6.1.3)

g = distance between lines of bolts (in.); gauge between bolts (in.) (6.8.3) (6.13.2.6.2)

H = overall height of rectangular HSS member, measured in the plane of the connection (in.) (6.8.2.2)

H_w = horizontal force in the web at a point of splice (kip) (C6.13.6.1.3c)

h = distance between flange centroids (in.); distance between centroids of individual component shapes perpendicular to the member axis of buckling (in.); depth between the centerline of the flanges (in.) (C6.9.4.1.3) (6.9.4.3.1) (C6.10.8.2.3)

h_{eff} = effective embedment depth of a stud shear connector (in.) (6.16.4.3)

h_h = effective height of the stud aabove the top of the haunch to the underside of the head (in.) (6.16.4.3)

h_o = distance between flange centroids (in.) (6.12.2.2.5)

I = moment of inertia of the short-term composite section, or optionally in regions of negative flexure of straight girders only, the moment of inertia of the steel section plus the longitudinal reinforcement if the concrete is not considered to be effective in tension in computing the range of longitudinal stress (in.^{4}); moment of inertia of the effective internal interior-pier diaphragm within a box section (in.^{4}) (6.10.10.1.2) (C6.11.8.1.1)

I_c = uncracked moment of inertia of the concrete in a composite concrete-filled steel tube about the centroidal axis (in.^{4}) (6.9.6.3.2)

I_{eff} = effective noncomposite moment of inertia about the vertical centroidal axis of a single girder within the span under consideration used in calculating the elastic global lateral–torsional buckling resistance of the span (in.^{4}) (6.10.3.4.2)

I_ℓ = moment of inertia of a longitudinal web stiffener including an effective width of web taken about the neutral axis of the combined section (in.^{4}); required moment of inertia of a longitudinal flange stiffener taken about an axis parallel to a box flange and taken at the base of the stiffener (in.^{4}) (6.10.11.1.3) (6.11.11.2)

I_p = lateral moment of inertia of a unit width of a longitudinally stiffened plate (in.^{3}) (E6.1.3)

I_{ps} = polar moment of inertia of a longitudinal stiffener alone about the attached edge (in.^{4}) (E6.1.4)

I_s = actual moment of inertia of a flange longitudinal stiffener taken about an axis parallel to a box flange and taken at the base of the stiffener (in.^{4}); moment of inertia of an individual stiffener strut composed of the stiffener plus the tributary width of the longitudinally stiffened plate under consideration, taken about an axis parallel to the face of the longitudinally stiffened plate and passing through the centroid of the gross combined area of the longitudinal stiffener and its tributary plate width (in.^{4}) (6.11.8.2.3) (E6.1.3)

I_{si} = moment of inertia of the internal steel reinforcement in a composite concrete-filled steel tube about the centroidal axis (in.^{4}) (6.9.6.3.2)
I_{st} = moment of inertia of the steel tube in a composite concrete-filled steel tube about the centroidal axis (in.^{4}) (6.9.6.3.2)
I_t = moment of inertia of a web transverse stiffener taken about the edge in contact with the web for single stiffeners and about the mid-thickness of the web for stiffener pairs (in.^{4}); moment of inertia of a flange transverse stiffener taken about an axis through its centroid and parallel to its bottom edge (in.^{4}); moment of inertia of a transverse stiffener, including a width of the stiffened plate equal to $9t_{sp}$, but not more than the actual dimension available, on each side of the stiffener avoiding any overlap with contributing parts to adjacent stiffeners or diaphragms, taken about the centroidal axis of the combined section (in.^{4}) (6.10.11.1.3) (C6.11.11.2) (E6.1.5.2)
I_{t1} = minimum moment of inertia of the transverse stiffener required for the development of the web shear buckling resistance (in.^{4}) (6.10.11.1.3)
I_{t2} = minimum moment of inertia of the transverse stiffener required for the development of the full web shear buckling plus postbuckling tension-field action resistance (in.^{4}) (6.10.11.1.3)
I_x = moments of inertia about the major principal axis of the cross section (in.^{4}); noncomposite moment of inertia about the horizontal centroidal axis of a single girder within the span under consideration (in.^{4}) (6.9.4.1.3) (6.10.3.4.2)
I_{xe} = effective moment inertia of the cross section about the axis of bending (in.^{4}) (6.12.2.2.2c)
I_y = moments of inertia about the minor principal axis of the cross section (in.^{4}); noncomposite moment of inertia about the vertical centroidal axis of a single girder within the span under consideration (in.^{4}); moment of inertia about the y-axis (in.^{4}) (6.9.4.1.3) (6.10.3.4.2) (6.12.2.2.4c)
I_{yc} = moment of inertia of the compression flange of a steel section about the vertical axis in the plane of the web (in.^{4}) ; moment of inertia of the compression flange about the vertical centroidal axis of a single girder within the span under consideration (in.^{4}) (6.10.2.2) (6.10.3.4.2) (A6.1)
I_{yt} = moment of inertia of the tension flange of a steel section about the vertical axis in the plane of the web (in.^{4}); moment of inertia of the tension flange about the vertical centroidal axis of a single girder within the span under consideration (in.^{4}) (6.10.2.2) (6.10.3.4.2) (A6.1)
IM = dynamic load allowance from Article 3.6.2
J = St. Venant torsional inertia (in.^{4}); transverse stiffener bending rigidity parameter; St. Venant torsional constant of the gross cross section (in.^{4}) (C6.7.4.3) (6.10.11.1.3) (6.12.2.2.2e)
J_s = St. Venant torsional constant of a longitudinal stiffener alone, not including the contribution from the stiffened plate (in.^{4}) (E6.1.4)
K = effective length factor; effective length factor in the plane of buckling determined as specified in Article 4.6.2.5; effective length factor as specified in Article 4.6.2.5; effective column length factor taken as 0.50 for chord splices (6.9.3) (6.9.4.1.2) (6.9.6.3.2) (6.14.2.8.6)
$(KL/r)_{eff}$ = effective slenderness ratio for a single-angle compression member (6.9.4.1.1)
K_h = hole size factor for bolted connections (6.13.2.8)
$K\ell$ = effective length in the plane of buckling (in.) (E6.1.1)
K_s = surface condition factor for bolted connections (6.13.2.8)
$K_x\ell_x$ = effective length for flexural buckling about the x-axis (in.) (6.9.4.1.3)
$K_y\ell_y$ = effective length for flexural buckling about the y-axis (in.) (6.9.4.1.3)
$K_z\ell_z$ = effective length for torsional buckling (in.) (6.9.4.1.3)
$K\ell/r$ = slenderness ratio (6.9.3)
k = plate-buckling coefficient considering any gradient in the longitudinal stress; elastic web bend-buckling coefficient; shear-buckling coefficient for webs; plate-buckling coefficient for uniform normal stress in box flanges; distance from the outer face of the flange to the toe of a web fillet of a rigid frame member to be stiffened (in.); distance from the outer face of a flange resisting a concentrated load or a bearing reaction to the web toe of the fillet (in.) (6.9.4.5) (6.10.1.9.1) (6.10.9.3.2) (6.11.8.2.2) (6.13.7.2) (D6.5.2)
k_c = flange local buckling coefficient (6.9.4.2.1)
k_s = shear buckling coefficient; plate-buckling coefficient for shear stress (C6.9.2.2.2) (6.11.8.2.2)
k_{sf} = elastic web bend-buckling coefficient for fully restrained longitudinal edge conditions (C6.10.1.9.1)
k_{ss} = elastic web bend-buckling coefficient for simply-supported longitudinal edge conditions (C6.10.1.9.1)

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L = actual span length bearing to bearing along the centerline of the bridge (ft); maximum length of the connection longitudinal welds or the out-to-out distance between the bolts in the connection parallel to the line of force (in.); length of the span under consideration (in.); distance from a single bolt to the free edge of the member measured parallel to the line of applied force (in.) (6.7.2) (6.8.2.2) (6.10.3.4.2) (C6.13.2.9)
L_b = spacing of intermediate diaphragms (ft); diaphragm or cross-frame spacing (ft); unbraced length (in.); unbraced length for lateral displacement or twist, as applicable (in.) (6.7.4.2) (C6.7.4.2) (6.12.2.2.2e) (6.12.2.2.7)
L_c = length of a channel shear connector (in.); clear distance between bolt holes or between the bolt hole and the end of the member in the direction of the applied bearing force (in.) (6.10.10.4.3) (6.13.2.9)
L_{cp} = length of a cover plate (ft) (6.10.12.1)
L_{db} = developed unbraced length along the vertical curve of an arch rib between the brace points (in.) (6.14.4.5)
L_{eff} = effective span for the calculation of shear lag (in.) (6.12.2.2.2g)
LFD = load factor design
L_{fs} = length of the unspliced individual girder field section under consideration (in.) (C6.10.2.2)
LL = vehicular live load
L_{mid} = in a gusset plate connection, the distance from the last row of fasteners in the compression member under consideration to the first row of fasteners in the closest adjacent connected member, measured along the line of action of the compressive axial force (in.) (6.14.2.8.4)
L_n = arc length between the point of maximum positive live load plus impact moment and the centerline of an adjacent interior support (ft) (6.10.10.4.2)
L_p = limiting unbraced length to achieve the nominal flexural resistance of $R_bR_hF_{yc}$ under uniform bending (in.); arc length between an end of the girder and an adjacent point of maximum positive live load plus impact moment (ft); limiting unbraced length to achieve the nominal flexural resistance $R_fR_bR_{pc}M_{yce}$ under uniform moment (in.); limiting unbraced length to achieve the nominal flexural resistance M_p under uniform bending (in.) (6.10.1.6) (6.10.10.4.2) (6.12.2.2.2e) (6.12.2.2.5)
L_r = limiting unbraced length to achieve the onset of nominal yielding in either flange under uniform bending with consideration of compression flange residual stress effects (in.); limiting unbraced length for calculation of the lateral–torsional buckling resistance (in.) (6.7.4.2) (6.12.2.2.2e)
$LRFD$ = load and resistance factor design
L_{splice} = in a gusset plate connection, the center-to-center distance between the first lines of fasteners in the adjoining chords at a chord splice (6.14.2.8.6)
LTB = lateral–torsional buckling (C6.10.8.2.1) (C6.10.8.2.3) (CA6.3.1) (CA6.3.3) (CD6.4.1) (CD6.4.2)
L_v = distance between points of maximum and zero shear (in.) (6.12.1.2.3b)
ℓ = unbraced member length (in.); distance between the work points of the joints measured along the length of the angle (in.); unbraced length of a composite concrete-filled steel tube column (in.); unbraced length in the plane of buckling (in.); buckling length of individual stiffener struts, taken equal to the smaller of a and ℓ_c (in.) (6.8.4) (6.9.4.1.2) (6.9.4.4) (6.9.6.3.2) (E6.1.3)
ℓ_c = characteristic buckling length of the stiffener struts of a longitudinally stiffened plate (in.) (E6.1.3)
M = bending moment about the major-axis of the cross section (kip-in.) (C6.10.1.4) (B6.6.2)
M_0 = bending moment due to the factored loads at a brace point opposite to the one corresponding to M_2, calculated from the moment envelope value that produces the largest compression at this point in the flange under consideration, or the smallest tension if this point is never in compression; positive when it causes compression and negative when it causes tension in the flange under consideration (kip-in.) (A6.3.3)
M_1 = bending moment at the opposite end of an unbraced length from M_2 representing the intercept of the most critical assumed linear stress distribution through either M_2 and M_{mid}, or through M_2 and M_0, taken as $2M_{mid} - M_2 \geq M_0$ (kip-in.); bending moment about the major-axis of the cross-section at the brace point with the lower moment due to the factored loads adjacent to an interior-pier section from which moments are redistributed taken as either the maximum or minimum moment envelope value, whichever produces the smallest permissible unbraced length (kip-in.) (A6.3.3) (B6.2.4)
M_2 = largest major-axis bending moment due to the factored loads at either end of an unbraced length causing compression in the flange under consideration, calculated from the critical moment envelope value; always taken as positive unless the moment is zero or causes tension in the flange under consideration at both ends of the unbraced length in which case M_2 is taken as zero (kip-in.); bending moment about the major-axis of the cross section at the brace point with the higher moment due to the factored loads adjacent to an interior-pier section from which moments are redistributed taken as the critical moment envelope value (kip-in.) (A6.3.3) (B6.2.4)
M_{AD} = additional bending moment that must be applied to the short-term composite section to cause nominal yielding in either steel flange (kip-in.) (D6.2.2)

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M_c = column moment due to the factored loading in a rigid frame (kip-in.) (6.13.7.2)
M_{cr} = elastic lateral–torsional buckling moment (kip-in.) (6.12.2.2.4c)
M_{D1} = bending moment caused by the factored permanent load applied before the concrete deck has hardened or is made composite (kip-in.) (D6.2.2)
M_{D2} = bending moment caused by the factored permanent load applied to the long-term composite section (kip-in.) (D6.2.2)
M_e = critical elastic moment envelope value due to the factored loads at an interior-pier section from which moments are redistributed (kip-in.) (B6.3.3.1)
M_{gs} = elastic global lateral–torsional buckling resistance of a span (kip-in.) (6.10.3.4.2)
M_ℓ = lateral bending moment in the flanges due to the eccentric loadings from concrete deck overhang brackets (kip-in.) (C6.10.3.4.1)
M_{max} = maximum potential flexural resistance based on the compression flange (kip-in.) (C6.10.8.2.1)
M_{mid} = major-axis bending moment due to the factored loads at the middle of an unbraced length, calculated from the moment envelope value that produces the largest compression at this point in the flange under consideration, or the smallest tension if this point is never in compression; positive when it causes compression and negative when it causes tension in the flange under consideration (kip-in.) (A6.3.3)
M_n = nominal flexural resistance of a section (kip-in.); nominal flexural resistance of a composite concrete-filled steel tube as a function of the nominal axial resistance, P_n (kip-in.) (6.10.7.1.1) (C6.12.2.3.3)
M_{nc} = nominal flexural resistance based on the compression flange (kip-in.) (6.9.2.2.1) (A6.3)
$M_{nc(FLB)}$ = nominal flexural resistance based on compression flange local buckling (kip-in.) (CD6.4.2)
M_{nt} = nominal flexural resistance based on the tension flange (kip-in.) (6.9.2.2.1) (A6.1.2)
M_p = plastic moment (kip-in.) (6.10.7.1.2) (D6.1)
M_{pe} = plastic moment using the effective box cross section based on the effective width of the compression flange (kip-in.); negative-flexure effective plastic moment at interior-pier sections from which moments are redistributed (kip-in.) (6.12.2.2.2c) (B6.3.3.1)
M_{ps} = plastic moment resistance of the steel section of a concrete-encased member (kip-in.) (6.12.2.3.1)
M_r = factored tension rupture flexural resistance about the axis of bending under consideration (kip-in.); factored flexural resistance (kip-in.) (6.8.2.3.3) (6.12.1.2.1)
M_{rd} = redistribution moment (kip-in.) (B6.3.3.1)
M_{rx}, M_{ry} = factored flexural resistance about the x- and y-axes, respectively, excluding tension flange rupture (kip-in.) (6.8.2.3.1)
M_{rxc} = factored flexural resistance about the x-axis taken as ϕ_f times the nominal flexural resistance about the x-axis considering compression buckling, determined as specified in Article 6.10 or 6.12, as applicable (kip-in.) (6.8.2.3.1)
M_{rxpe} = for I-section members, ϕ_f times the plastic moment about the x-axis neglecting any web longitudinal stiffeners; for noncomposite box-section members, ϕ_f times the effective plastic moment about the x-axis, based on the effective compression flange area as defined in Article 6.12.2.2.2c or 6.12.2.2.2d, as applicable, and neglecting any web longitudinal stiffeners (kip-in.) (6.8.2.3.1)
M_{ryc} = factored flexural resistance about the y-axis taken as ϕ_f times the nominal flexural resistance about the y-axis considering compression buckling, determined as specified in Article 6.12, as applicable; $M_{ryc} = M_{ryt} = M_{ry}$ for I-section members (kip-in.) (6.8.2.3.1)
M_{rype} = for I-section members, ϕ_f times the plastic moment about the y-axis neglecting any web longitudinal stiffeners; for noncomposite box-section members, ϕ_f times the effective plastic moment about the y-axis, based on the effective compression flange area as defined in Article 6.12.2.2.2c or 6.12.2.2.2d, as applicable, and neglecting any web longitudinal stiffeners (kip-in.) (6.8.2.3.1)
M_u = moment due to the factored loads (kip-in.); factored moment about the axis of bending at the cross section containing the flange bolt holes taken as positive for tension in the flange under consideration and negative for compression (kip-in.); largest value of the major-axis bending moment throughout the unbraced length causing compression in the flange under consideration (kip-in.) (6.7.6.2.1) (6.8.2.3.3) (6.10.1.6)
M_{ux}, M_{uy} = factored moments about the x- and y-axes, respectively (kip-in.); factored biaxial bending moments about the x- and y-axes, respectively, applied to a noncomposite box-section member, circular tube, or an I- or H-section member (kip-in.) (6.8.2.3.1) (C6.9.2.2.2)
M_y = yield moment (kip-in.); yield moment = $F_y S_x$ (kip-in.) (6.10.7.1.2) (6.12.2.2.4b)
M_{yc} = yield moment with respect to the compression flange (kip-in.); yield moment of the composite section of a concrete-encased shape (kip-in.) (6.9.2.2.1) (6.12.2.3.1)
M_{yce} = yield moment of the effective noncomposite box section with respect to the compression flange taken as $F_{yc}S_{xce}$ for sections in which $S_{xce} \leq S_{xte}$, and calculated as the moment at nominal first yielding of the compression flange, considering early nominal yielding in tension, for sections in which $S_{xte} < S_{xce}$ (kip-in.) (6.12.2.2.2c)

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M_{yt} = yield moment with respect to the tension flange determined as specified in Article D6.2 (kip-in.) (6.9.2.2.1)
N = tital number of stress cycles over the fatigue design life; length of bearing, taken greater than or equal to k at end bearing locations (in.) (6.6.1.2.5) (D6.5.2)
NDT = nondestructive testing (6.6.1.2.3)
N_b = concrete breakout resistance in tension of a single stud shear connector in cracked concrete (kip) (6.16.4.3)
N_n = nominal tensile resistance of a single stud shear connector (kip) (6.16.4.3)
N_r = factored tensile resistance of a single stud shear connector (kip) (6.16.4.3)
N_s = number of shear planes per bolt; number of slip planes per bolt (6.13.2.7) (6.13.2.8)
N_u = seismic axial force demand per stud at the support cross-frame or diaphragm location under consideration (kip) (6.14.4.3)
n = number of stress range cycles per truck passage; modular ratio; number of shear connectors in a cross section; minimum number of shear connectors over the region under consideration; short-term modular ratio; number of equally spaced longitudinal flange stiffeners; number of uniformly spaced internal reinforcing bars in a composite concrete-filled steel tube; number of longitudinal stiffeners (6.6.1.2.5) (6.9.5.1) (6.10.1.1.1b) (6.10.10.1.2) (6.10.10.4.1) (6.11.8.2.3) (C6.12.2.3.3) (E6.1.3)
n_{ac} = number of additional shear connectors required in the regions of points of permanent load contraflexure for sections that are noncomposite in negative-flexure regions (6.10.10.3)
P = unfactored axial dead load for a composite concrete-filled steel tube (kip); total nominal shear force in the concrete deck for the design of the shear connectors at the strength limit state (kip) (6.9.6.3.2) (6.10.10.4.1)
P_{1n} = longitudinal force in the girder over an interior support for the design of the shear connectors at the strength limit state (kip) (6.10.10.4.2)
P_{1p} = longitudinal force in the concrete deck at the point of maximum positive live load plus impact moment for the design of the shear connectors at the strength limit state (kip) (6.10.10.4.2)
P_{2n} = longitudinal force in the concrete deck over an interior support for the design of the shear connectors at the strength limit state (kip) (6.10.10.4.2)
P_{2p} = longitudinal force in the girder at the point of maximum positive live load plus impact moment for the design of the shear connectors at the strength limit state (kip) (6.10.10.4.2)
P_c = plastic force in the compression flange used to compute the plastic moment (kip) (D6.1)
P_{cr} = nominal compressive resistance of the member calculated from Eq. 6.9.4.1.1-1 or 6.9.4.1.1-2, as applicable, using A_g (kip) (6.9.4.2.2a)
P_e = elastic critical buckling resistance determined as specified in Article 6.9.4.1.2 for flexural buckling, as specified in Article 6.9.4.1.3 for torsional buckling or flexural–torsional buckling, as applicable (kip); Euler buckling load (kip); elastic critical buckling resistance for flexural buckling of a composite concrete-filled steel tube (kip); elastic critical buckling resistance determined as specified in Article 6.14.2.8.4 for gusset plate buckling (kip) (6.9.4.1.1) (6.9.5.1) (6.9.6.3.2) (6.14.2.8.4)
P_{esF} = elastic flexural buckling resistance of an individual stiffener strut (kip) (E6.1.3)
P_{esT} = plate torsional stiffness contribution to the elastic buckling resistance of an individual stiffener strut (kip) (E6.1.3)
P_{fy} = design yield resistance of the flange at a point of splice (kip) (6.13.6.1.3b)
P_h = horizontal component of the flange force in the inclined bottom flange of a variable web depth member (kip) (C6.10.1.4)
P_ℓ = statically equivalent concentrated lateral concrete deck overhang bracket force placed at the middle of the unbraced length (kip) (C6.10.3.4.1)
P_n = nominal bearing resistance on pin plates (kip); nominal axial compressive resistance (kip); nominal compressive resistance of a composite concrete-filled steel tube (kip); total longitudinal force in the concrete deck over an interior support for the design of the shear connectors at the strength limit state, taken as the lesser of either P_{1n} or P_{2n} (kip); nominal compressive resistance of an idealized Whitmore section (kip) (6.8.7.2) (6.9.2.1) (6.9.6.3.2) (6.10.10.4.2) (6.14.2.8.4)
P_{nR} = nominal compressive resistance provided by an individual laterally-restrained longitudinal edge of a longitudinally stiffened plate (kip) (E6.1.3)
P_{ns} = nominal compressive resistance of an individual stiffener strut composed of the stiffener plus the tributary width of the longitudinally stiffened plate under consideration (kip) (E6.1.3)
P_{nsF} = nominal flexural buckling resistance of an individual stiffener strut (kip) (E6.1.3)
P_{nsp} = nominal compressive resistance of the compression flange calculated as specified in Article E6.1.3 (kip); nominal compressive resistance of a longitudinally stiffened component plate (kip) (6.12.2.2.2d) (E6.1.3)
P_{ny} = nominal axial tensile resistance for yielding in the gross section (kip) (6.8.2.1)
P_o = nominal yield resistance = F_yA_g (kip); compressive resistance of a composite concrete-filled steel tube column without consideration of buckling (kip) (6.9.4.1.1) (6.9.6.3.2)

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P_{os} = nominal yield resistance given by Eq. E6.1.1-9 (kip) (E6.1.1)

P_p = total longitudinal force in the concrete deck at the point of maximum positive live load plus impact moment for the design of the shear connectors at the strength limit state, taken as the lesser of either P_{1p} or P_{2p} (kip) (6.10.10.4.2)

P_r = factored axial tensile resistance (kip); factored bearing resistance on pin plates (kip); for cross sections subjected to axial tension, factored tensile rupture resistance of the net section based on Eq. 6.8.2.1-2; for cross sections subjected to axial compression, factored tensile yield resistance of the cross section based on Eq. 6.8.2.1-1 (kip); factored axial compressive resistance of a steel pile (kip); factored compressive resistance as specified in Article 6.9.2.1 (kip); factored compressive resistance of a composite concrete-filled steel tube (kip); factored axial resistance of bearing stiffeners (kip); factored compressive resistance of gusset plates (kip); factored resistance of piles in compression (ksi) (6.8.2.1) (6.8.7.2) (6.8.2.3.3) (6.9.2.2.1) (6.9.6.3.2) (6.10.11.2.4a) (6.14.2.8.4) (6.15.3.1)

P_{rb} = plastic force in the bottom layer of longitudinal deck reinforcement used to compute the plastic moment (kip) (D6.1)

P_{rt} = plastic force in the top layer of longitudinal deck reinforcement used to compute the plastic moment (kip) (D6.1)

P_{ry} = factored tensile resistance based on tension yielding, obtained from Eq. 6.8.2.1-1 (kip) (6.8.2.3.1)

P_s = plastic compressive force in the concrete deck used to compute the plastic moment (kip) (D6.1)

P_T = total longitudinal force in the concrete deck between the point of maximum positive live load plus impact moment and the centerline of an adjacent interior support for the design of the shear connectors at the strength limit state, taken as the sum of P_p and P_n (kip) (6.10.10.4.2)

P_t = minimum required bolt tension (kip); plastic force in the tension flange used to compute the plastic moment (kip) (6.13.2.8) (D6.1)

P_u = factored tensile axial force (kip); maximum factored axial force at the cross section containing the flange bolt holes taken as positive in tension and negative in compression (kip); factored compressive axial force throughout the member length in Eqs. 6.9.2.2.1-1 through 6.9.2.2.1-3 (kip); direct tension or shear force on a bolt due to the factored loads (kip); axial compressive force effect resulting from factored loads (kip) (6.8.2.3.1) (6.8.2.3.3) (6.9.2.2.1) (6.13.2.10.4) (6.13.2.11)

P_{up} = total factored longitudinal compression force in the plate under consideration, determined from a structural analysis considering the gross cross section, including the longitudinal stiffeners, and including all sources of factored longitudinal normal compressive stresses from axial loading and from flexure (kip) (E6.1.5.2)

P_{ups} = factored axial force within a longitudinal stiffener strut, composed of the longitudinal stiffener and the tributary plate width, determined from a structural analysis considering the gross cross section (kip) (CE6.1.5.1)

P_v = vertical component of the flange force in the inclined bottom flange of a variable web depth member (kip) (C6.10.1.4)

P_w = plastic force in the web used to compute the plastic moment (kip) (D6.1)

P_{yeR} = effective yield load of an individual laterally-restrained longitudinal edge of a longitudinally stiffened plate (kip) (E6.1.3)

P_{yes} = effective yield load of an individual stiffener strut (kip) (E6.1.3)

P_{ys} = yield load of an individual stiffener strut (kip) (E6.1.3)

p = pitch of shear connectors along the longitudinal axis (in.); staggered pitch between two adjacent lines of staggered bolt holes (in.) (6.10.10.1.2) (6.13.2.6.3)

Q = first moment of the transformed short-term area of the concrete deck about the neutral axis of the short-term composite section, or optionally in regions of negative flexure of straight girders only, the first moment of the longitudinal reinforcement about the neutral axis of the composite section if the concrete is not considered to be effective in tension in computing the range of longitudinal stress (in.^{3}); first moment of one-half the effective box-flange area at an interior pier about the neutral axis of the effective internal diaphragm section (in.^{3}) (6.10.10.1.2) (C6.11.8.1.1)

Q_n = nominal shear resistance of a single shear connector (kip) (6.10.10.4.1)

Q_r = factored shear resistance of a single shear connector (kip); factored shear resistance of a single stud shear connector (kip) (6.10.10.4.1) (6.16.4.3)

Q_u = prying tension per bolt due to the factored loads (kip); seismic shear demand per stud at the support cross-fram or diaphragm location under consideration due to the governing orthogonal combination of seismic shears (kip) (6.13.2.10.4) (6.16.4.3)

R = transition radius of welded attachments as shown in Table 6.6.1.2.3-1 (in.); girder radius at the centerline of the bridge (ft); minimum girder radius within a panel (ft); minimum girder radius over the length, L_n

(ft); reduction factor applied to the factored shear resistance of bolts passing through fillers; radius of curvature of an arch rib at the mid-depth of the web for the section under consideration (in.) (6.6.1.2.3) (6.7.2) (6.7.4.2) (6.10.10.4.2) (6.13.6.1.4) (6.14.4.1)

R_b = web load-shedding factor (6.10.1.6)
R_f = compression flange slenderness factor (6.12.2.2.2c)
R_h = hybrid factor (6.10.1.10.1)
R_n = nominal resistance of a bolt, connection or connected material (kip) or (ksi); nominal resistance to a concentrated loading (kip) (6.13.2.2) (D6.5.2)
R_p = reduction factor for holes taken equal to 0.90 for bolt holes punched full size and 1.0 for bolt holes drilled full size or subpunched and reamed to size (6.8.2.1)
$(R_{pB})_n$ = nominal bearing resistance on pins (kip) (6.7.6.2.2)
$(R_{pB})_r$ = factored bearing resistance on pins (kip) (6.7.6.2.2)
R_{pc} = web plastification factor for the compression flange (6.12.2.2.2c) (A6.2.1)
R_{pt} = web plastification factor for the tension flange (A6.1.4) (A6.2.1)
R_r = factored resistance of a bolt, connection or connected material (kip) or (ksi); factored resistance in tension of connection elements (kip); factored tensile resistance of gusset plates (kip) (6.13.2.2) (6.13.5.2) (6.14.2.8.5)
$(R_{sb})_n$ = nominal bearing resistance for the fitted end of bearing stiffeners (kip) (6.10.11.2.3)
$(R_{sb})_r$ = factored bearing resistance for the fitted end of bearing stiffeners (kip) (6.10.11.2.3)
R_u = factored concentrated load or bearing reaction (kip) (D6.5.2)
r = minimum radius of gyration of a tension or compression member (in.); radius to the mid-thickness of the tube (in.); radius of gyration of a built-up member about an axis perpendicular to a perforated plate (in.); radius of gyration of a longitudinal web stiffener including an effective width of web taken about the neutral axis of the combined section (in.); radius to the outside of the steel tube in a composite concrete-filled steel tube (in.) (6.8.4) (C6.9.2.2.2) (6.9.4.3.2) (6.10.11.3.3) (C6.12.2.3.3)
r_1, r_2 = reduction factors applied in the calculation of the nominal compressive resistance of noncomposite rectangular box cross-sections containing one or more flange elements in the direction associated with column flexural buckling in which the flange element or elements contain longitudinal stiffeners, and where $\lambda_{max} > \lambda_r$ (E6.1.1)
r_b = radius to the center of the internal reinforcing bars in a composite concrete-filled steel tube (in.) (C6.12.2.3.3)
r_i = minimum radius of gyration of an individual component shape (in.); radius to the inside of the steel tube in a composite concrete-filled steel tube (in.) (C6.9.4.3.1) (C6.12.2.3.3)
r_{ib} = radius of gyration of an individual component shape relative to its centroidal axis parallel to the member axis of buckling (in.) (6.9.4.3.1)
r_m = radius to the center of the steel tube in a composite concrete-filled steel tube (in.) (C6.12.2.3.3)
r_n = nominal bearing pressure at bolt holes (ksi) (C6.13.2.9)
r_s = radius of gyration about the axis normal to the plane of buckling (in.); radius of gyration of a stiffener strut about an axis parallel to the plane of the stiffened plate (in.) (6.9.4.1.2) (E6.1.4)
r_t = effective radius of gyration for lateral–torsional buckling (in.) (6.10.8.2.3) (A6.3.3)
r_{ts} = radius of gyration used in the determination of L_r (in.) (6.12.2.2.5)
r_x = radius of gyration about the x-axis (in.); radius of gyration about the geometric axis of the angle parallel to the connected leg (in.) (6.9.4.1.3) (6.9.4.4)
r_y = radius of gyration about the y-axis (in.); radius of gyration of the gross noncomposite box-section about its minor principal axis, including any longitudinal stiffeners (in.^{4}); (6.9.4.1.3) (6.12.2.2.2e)
r_{yc} = radius of gyration of the compression flange with respect to a vertical axis in the plane of the web (in.) (C6.10.8.2.3)
r_z = radius of gyration about the minor principal axis of the angle (in.) (6.9.4.4)
r_σ = desired bending stress ratio in a horizontally-curved I-girder, taken equal to $|f_\ell/f_{bu}|$ (C6.7.4.2)
$\bar{r}_o$ = polar radius of gyration about the shear center (in.) (6.9.4.1.3)
S = elastic section modulus about the axis of bending (in.^{3}); elastic section modulus (in.^{3}) (C6.12.2.2.1) (6.12.2.2.3)
S_g = elastic gross section modulus of gusset plates and splice plates (in.^{3}) (6.14.2.8.6)
S_n = elastic net section modulus of gusset plates and splice plates (in.^{3}) (6.14.2.8.6)
S_{LT} = long-term composite elastic section modulus (in.^{3}) (D6.2.2)
S_{NC} = noncomposite elastic section modulus (in.^{3}) (D6.2.2)
SRM = system redundant member (6.6.2.2)
S_s = elastic section modulus of a transverse flange stiffener (in.^{3}) (C6.11.11.2)

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S_{ST} = short-term composite elastic section modulus (in.^{3}) (D6.2.2)
S_t = minimum elastic section modulus about the axis of bending under consideration (in.^{3}) (6.8.2.3.3)
S_x = elastic section modulus to an inclined bottom flange of a variable web depth member (in.^{3}); elastic section modulus about the x-axis with respect to the tip of the tee stem or double angle web legs, as applicable (in.^{3}); elastic section modulus about the x-axis (in.^{3}); section modulus about the major geometric axis (in.^{3}); (C6.10.1.4) (6.12.2.2.4b) (6.12.2.2.5) (6.12.2.2.7)
S_{xc} = elastic section modulus about the major axis of the section to the compression flange taken as M_{yc}/F_{yc} (in.^{3}); elastic section modulus about the x-axis with respect to the compression flange (in.^{3}) (6.9.2.2.1) (6.12.2.2.4d) (A6.1.1)
S_{xce} = effective elastic section modulus about the axis of bending to the compression flange determined using the effective width for the compression flange, b_e, calculated as specified in Article 6.9.4.2.2b with F_{cr} taken equal to F_{yc}, and the gross area for the tension flange (in.^{3}) (6.12.2.2.2c)
S_{xt} = elastic section modulus about the major axis of the section to the tension flange taken as M_{yt}/F_{yt} (in.^{3}) (6.9.2.2.1) (A6.1.2)
S_{xte} = effective elastic section modulus about the axis of bending to the tension flange determined using the effective width for the compression flange, b_e, calculated as specified in Article 6.9.4.2.2 with F_{cr} taken equal to F_{yc}, and the gross area for the tension flange (in.^{3}) (6.12.2.2.2c)
S_y = elastic section modulus about the axis parallel with the web (in.^{3}) (6.12.2.2.1)
s = pitch of any two consecutive bolts in a staggered chain (in.); longitudinal spacing of transverse reinforcement in a concrete-encased shape (in.); spacing of bolts on a single line or in a staggered pattern adjacent to a free edge of an outside plate or shape (in.) (6.8.3) (6.12.3.1) (6.13.2.6.2)
s_t = maximum transverse spacing between shear connectors on a composite box flange (in.) (6.11.10)
T = internal torque due to the factored loads (kip-in.); base metal thickness of the thicker part joined in a fillet-welded connection given in Table 6.13.3.4-1 (in.) (C6.11.1.1) (6.13.3.4)
T_n = nominal torsional resistance for noncomposite circular tubes and round HSS (kip-in.); nominal resistance of a bolt in axial tension or in combined axial tension and shear (kip) (6.12.1.2.3a) (6.13.2.2)
T_r = factored torsional resistance for noncomposite circular tubes and round HSS (kip-in.); factored resistance of a bolt in axial tension or in combined axial tension and shear (kip) (6.12.1.2.3a) (6.13.2.2)
T_u = factored torque (kip-in.); factored torque at the section under consideration (kip-in.); tensile force per bolt due to Load Combination Service II (kip) (C6.9.2.2.2) (6.12.1.2.3a) (6.13.2.11)
t = thickness of plate or plates (in.); element thickness (in.); thickness of the element under consideration (in.); wall thickness of a circular tube or round HSS (in.); wall thickness of the steel tube in a composite concrete-filled steel tube (in.); distance from the centroid of the noncomposite steel section under consideration to the centroid of the tension flange (in.); wall thickness of the tube (in.); thickness of each plate of the box-section member (in.); thickness of an individual double angle web leg (in.); width of the rectangular bar parallel to the axis of bending (in.); thickness of the thinner outside plate or shape (in.); thickness of the connected material (in.); thickness of the thinnest connected part (in.); thickness of the cross-section plate component under consideration in an arch rib (in.); thickness of a longitudinally unstiffened plate (in.); thickness of a longitudinally stiffened plate (in.); longitudinal stiffener plate element thickness (in.) (C6.7.4.3) (6.9.4.2.1) (6.9.4.2.2a) (6.9.4.2.2c) (6.9.6.2) (6.10.3.4.2) (6.12.1.2.3b) (6.12.2.2.2e) (6.12.2.2.4e) (6.12.2.2.7) (6.13.2.6.2) (6.13.2.9) (6.13.2.10.4) (6.14.4.1) (E6.1.1) (E6.1.2) (E6.1.4)
t_b = thickness of a fictional tube modeling the internal reinforcement in a composite concrete-filled steel tube (in.); thickness of the flange transmitting the concentrated force in a rigid-frame connection (in.) (C6.12.2.3.3) (6.13.7.2)
t_c = thickness of the flange of the member to be stiffened in a rigid-frame connection (in.) (6.13.7.2)
t_f = flange thickness (in.); thickness of the flange plate element or longitudinally stiffened flange plate panel under consideration (in.); flange thickness of a channel shear connector (in.); thickness of the flange under consideration. For HSS, the provisions of Article 6.12.1.2.4 apply (in.); thickness of the flange resisting a concentrated load or bearing reaction (in.) (C6.9.4.1.3) (6.9.4.5) (6.10.10.4.3) (6.12.2.2.2b) (D6.5.3)
t_{fc} = thickness of the compression flange (in.) (6.10.1.10.2)
t_{ft} = thickness of the tension flange (in.) (6.10.9.3.2)
t_g = gusset plate thickness (6.14.2.8.4)
t_{haunch} = thickness of the concrete haunch measured from the top of the web to the bottom of the concrete deck (in.) (C6.13.6.1.3b)
t_p = thickness of a loaded discontinuous plate element (in.); thickness of a projecting stiffener element (in.) (6.6.1.2.5) (6.10.11.1.2)

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t_s = thickness of a concrete deck (in.); thickness of the projecting longitudinal stiffener element (in,) (6.10.1.10.2) (6.11.11.2)
t_{sp} = thickness of a longitudinally stiffened plate (in.); thickness of a stiffened plate (in.) (E6.1.3) (E6.1.5.2)
t_w = web thickness (in.); thickness of the web plate element under consideration (in.); web thickness of a channel shear connector (in.); thickness of the webs. For box sections with different thickness webs, the smaller web thickness. For HSS, the provisions of Article 6.12.1.2.4 apply (in.); thickness of the web (in.); thickness of the tee stem (in.); thickness of the web to be stiffened in a rigid-frame connection (in.); web thickness of an arch rib (in.) (6.9.4.2.1) (6.9.4.5) (6.10.10.4.3) (6.12.2.2.2b) (6.12.2.2.4e) (6.12.2.2.5) (6.13.7.2) (6.14.4.2)
U = reduction factor to account for shear lag in connections subjected to a tension load; reduction factor to account for shear lag; 1.0 for components in which force effects are transmitted to all elements, and as specified in Article 6.8.2.2 for other cases (6.6.1.2.3) (6.8.2.1)
U_{bs} = reduction factor for block shear rupture resistance taken equal to 0.50 when the tension stress is non-uniform and1.0 when the tension stress is uniform (6.13.4)
V = additional shear force for built-up members with perforated plates (kip); factored vertical shear force in the internal interior-pier diaphragm of a box section due to flexure plus St. Venant torsion (kip) (6.9.4.3.2) (C6.11.8.1.1)
V_c = nominal shear resistance provided by the concrete infill (kips) (6.12.3.2.2)
V_{cr} = shear-yielding or shear-buckling resistance (kip); web shear-yielding or shear-buckling resistance of the web panel under consideration (kip) (6.10.3.3) (6.10.11.1.3)
V_f = vertical shear force range under the Fatigue Load Combination (kip) (6.10.10.1.2)
V_{fat} = longitudinal fatigue shear range per unit length (kip/in.) (6.10.10.1.2)
V_n = nominal shear resistance (kip); nominal shear-yielding or shear-buckling plus postbuckling tension-field action resistance of the web panel under consideration (kip) (6.10.9.1) (6.10.11.1.3)
V_{ni} = the nominal interface shear resistance determined as specified in Article 5.7.4 (C6.16.4.2)
V_p = plastic shear force (kip) (6.10.9.2)
V_r = factored flexural shear resistance (kip); factored shear resistance of gusset plates (kip) (6.12.1.2.3) (6.14.2.8.3)
V_s = nominal shear resistance provided by the steel tube (kips) (6.12.3.2.2)
V_{sr} = horizontal fatigue shear range per unit length (kip/in.); vector sum of the horizontal fatigue shear range and the torsional fatigue shear range in the concrete deck for a composite box flange (kip/in.) (6.10.10.1.2) (6.11.10)
V_u = shear due to the factored loads (kip); shear in the web acting on the noncomposite section under consideration due to the factored load for constructibility specified in Article 3.4.2.1 (kip); maximum shear due to the factored loads in the web panel under consideration (kip); vertical shear due to the factored loads on one inclined web of a box section (kip); for noncomposite circular tubes, including round HSS, factored flexural shear at the section under consideration. For noncomposite rectangular box-section members subject to torsion, including square and rectangular HSS, the factored shear in each web element shall be taken as the sum of the flexural and St. Venant torsional shears (kip) (6.7.6.2.1) (6.10.3.3) (6.10.11.1.3) (6.11.9) (6.12.1.2.3a)
V_{ui} = shear due to the factored loads along one inclined web of a box section (kip) (6.11.9)
$V_{u\ell 1}$ = first-order force transferred by the attachment of a longitudinal stiffener due to any directly applied factored loads on the stiffener perpendicular to the plane of the stiffened plate (kip) (CE6.1.5.1)
V_{ut1} = first-order force in the direction perpendicular to the plane of the stiffened plate at the attachment of a longitudinal stiffener due to any directly applied factored loads (kip) (CE6.1.5.1)
V_{ux}, V_{uy} = factored biaxial flexural shear forces (kip) (C6.9.2.2.2)
W = total weight of the deck, steel girders, and, where applicable, cap beam, plus one-half the column weight within the span under consideration (kip) (6.16.4.2)
W_{px} = weight of the deck plus one-half of the weight of the steel girders in the span under consideration (kip) (6.16.4.2)
w = leg size of the reinforcement or contour fillet, if any, in the direction of the thickness of the loaded discontinuous plate element (in.); center-to-center distance between the top flanges of a box section (in.); plate width (in.); width of the plate between the centerlines of the individual longitudinal stiffeners and/or between the centerline of a longitudinal stiffener and the inside of the laterally-restrained longitudinal edge of a longitudinally stiffened plate, as applicable (in.); effective length of deck assumed acting radial to the girder (in.); length of the channel measured transverse to the direction of the flange (in.); larger of the width of a box flange between longitudinal flange stiffeners or the distance from a web to the nearest longitudinal flange stiffener (in.); widths of the plate between the centerlines of the individual longitudinal

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stiffeners and/or between the centerline of a longitudinal stiffener and the inside of the laterally-restrained longitudinal edge of a longitudinally stiffened plate, as applicable, equal to the tributary width in the case of equally spaced longitudinal stiffeners (in.) (6.6.1.2.5) (C6.7.5.3) (6.8.2.2) (6.9.4.5) (6.10.10.1.2) (6.10.10.2) (6.11.8.2.3) (6.12.2.2.2b)

w_e = effective width of the plate between the longitudinal stiffeners or between a longitudinal stiffener and the laterally-restrained longitudinal edge of the longitudinally stiffened plate under consideration, as applicable (in.) (E6.1.3)

w_g = girder spacing for a two-girder system or the distance between the two exterior girders of the unit for a three-girder system (in.) (6.10.3.4.2)

w_h = width of haunch perpendicular to bridge span axis (in,) (6.16.4.3)

x_o = distance along the x-axis between the shear center and centroid of the cross section (in.) (6.9.4.1.3)

$\bar{x}$ = distance from the centroid of the member to the surface of the gusset or connection plate (in.); connection eccentricity used in calculating the shear lag reduction factor, U (in.) (6.6.1.2.3) (6.8.2.2)

y = distance from the center of the steel tube to the neutral axis for a given stress state in a composite concrete-filled steel tube (in.) (C6.12.2.3.3)

y_o = distance along the y-axis between the shear center and centroid of the cross section (in.) (6.9.4.1.3)

$\bar{y}$ = distance from the plastic neutral axis to the top of the element where the plastic neutral axis is located (in.) (D6.1)

Z = curvature parameter for determining required longitudinal web stiffener rigidity; plastic section modulus (in.^{3}); plastic section modulus of the steel section of a composite concrete-encased shape about the axis of bending (in.^{3}) (6.10.11.3.3) (6.12.2.2.3) (6.12.2.3.1)

Z_r = shear fatigue resistance of an individual shear connector (kip) (6.10.10.1.2)

Z_x = plastic section modulus about the x-axis (in.^{3}) (6.12.2.2.4b)

Z_y = plastic section modulus about the axis parallel with the web (in.^{3}) (6.12.2.2.1)

α = separation ratio = $h/2r_{ib}$; factor defining the sloping straight line representing the finite-life portion of the fatigue shear resistance of an individual stud shear connector (6.9.4.3.1) (6.10.10.2)

β = factor equal to two times the area of the web based on D_n divided by A_{fn} used in computing the hybrid factor; factor defining the approximate ratio of D_p to $D_t/7.5$ at which a composite section in positive flexure reaches M_p; curvature correction factor for longitudinal web stiffener rigidity (6.10.1.10.1) (C6.10.7.1.2) (6.10.11.3.3)

η = load modifier related to ductility, redundancy and operational importance (C6.6.1.2.2)

γ = load factor specified in Table 3.4.1-1; the ratio of A_f to A_p for filler plate design (6.6.1.2.2) (6.13.6.1.4)

Δ = reduction factor applied to the factored axial resistance to account for the effect of torsional and/or flexural shear in a noncomposite box-section member or circular tube; reduction factor for the maximum stress in a box flange (6.8.2.3.2) (6.11.3.2)

(Δf) = live load stress range due to the passage of the fatigue load (ksi) (6.6.1.2.2)

$(\Delta F)^c{}_n$ = nominal fatigue resistance for Detail Category C (ksi) (6.6.1.2.5)

$(\Delta F)_n$ = nominal fatigue resistance (ksi) (6.6.1.2.2) (6.6.1.2.5)

(ΔF_{TH}) = constant amplitude fatigue threshold (ksi) (6.6.1.2.5)

Δ_x, Δ_y = reduction factors applied to the factored flexural resistance about the x- and y-axes, respectively, to account for the effect of torsional and/or flexural shear in a noncomposite box-section member or circular tube (6.8.2.3.2)

λ = slenderness, b_f/t_f, w/t_f, or D/t_w, of the slender flange or web plate element or longitudinally stiffened plate panel under consideration; normalized column slenderness factor (6.9.4.5) (6.9.5.1)

λ_f = slenderness ratio for the compression flange; slenderness ratio for the flange; compression flange slenderness = b_{fi}/t_{fc}; flange slenderness = $b_f/2t_f$; flange slenderness of the channel = b_f/t_f (6.10.8.2.2) (6.12.2.2.1) (6.12.2.2.2c) (6.12.2.2.4d) (6.12.2.2.5) (A6.3.2)

γ_p = the maximum load factor for DC specified in Table 3.4.1-2, or the maximum load factor specified in Article 3.4.2.1 for DC and any construction loads that are applied to the fully erected steelwork (C6.7.2)

λ_{max} = maximum w/t of the panels within a longitudinally stiffened flange plate (E6.1.1)

λ_p = limiting slenderness ratio for the box flange (6.11.10)

λ_{pf} = limiting slenderness ratio for a compact flange (6.10.8.2.2)

λ_{pw} = limiting slenderness for a compact web (6.12.2.2.2c)

$\lambda_{pw(Dc)}$ = limiting slenderness ratio for a compact web corresponding to $2D_c/t_w$ (A6.2.2)

$\lambda_{pw(Dcp)}$ = limiting slenderness ratio for a compact web corresponding to $2D_{cp}/t_w$ (A6.2.1)

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λ_r = width-to-thickness or slenderness ratio limit as specified in Table 6.9.4.2.1-1; nonslender limit for longitudinally stiffened plate panels defined in Article E6.1.2; corresponding width-to-thickness ratio limit for the longitudinal stiffener plate element under consideration (6.9.4.2.1) (E6.1.1) (E6.1.4)
λ_{rf} = limiting slenderness ratio for a noncompact flange (6.10.8.2.2)
λ_{rw} = limiting slenderness ratio for a noncompact web, expressed in terms of $2D_c/t_w$; limiting slenderness ratio for a noncompact web (6.10.1.10.2) (6.10.6.2.3) (A6.2.1)
λ_{rwD} = limiting slenderness ratio for a noncompact web, expressed in terms of D/t_w (6.10.1.10.2)
λ_w = web slenderness; slenderness ratio for the web based on the elastic moment (6.12.2.2.2c) (A6.2.2)
ψ = constant used in determining the required moment of inertia of longitudinal stiffeners for box flanges (6.11.11.2)
ψ_{ed} = edge modification factor for stud shear connectors (6.16.4.3)
ψ_g = group effect modification factor for transverse or longitudinal spacing (6.16.4.3)
ρ = factor equal to the smaller of F_{yw}/f_n and 1.0 used in computing the hybrid factor (6.10.1.10.1)
ρ_t = the larger of F_{yw}/F_{crs} and 1.0 (6.10.11.1.3)
ρ_w = shear ratio used in the design of transverse stiffeners adjacent to web panels subject to postbuckling tension-field action (6.10.11.1.3)
θ = angle of inclination of the bottom flange of a variable web depth member (degrees); angle of inclination of the web plate of a box section to the vertical (degrees); framing angle of compression member relative to an adjoining member in a gusset-plate connection (C6.10.1.4) (6.11.9) (6.14.2.8.3)
θ_b = angle defining the length c_b for a given stress state in a composite concrete-filled steel tube (radians) (C6.12.2.3.3)
θ_p = plastic rotation at an interior-pier section (radians) (B6.6.2)
θ_{RL} = plastic rotation at which the moment at an interior-pier section nominally begins to decrease with increasing θ_p (radians) (B6.6.2)
θ_s = angle defining the length c for a given stress state in a composite concrete-filled steel tube (radians) (C6.12.2.3.3)
ν = Poisson's ratio = 0.3 (E6.1.3)
σ_{flg} = range of longitudinal fatigue stress in the bottom flange without consideration of flange lateral bending (ksi) (6.10.10.1.2)
ϕ = resistance factor; resistance factor applied to the modulus of rupture of the concrete (6.5.4.2) (6.10.1.7)

ϕ_b = resistance factor for bearing (6.5.4.2)
ϕ_{bb} = resistance factor for bolts bearing on material (6.5.4.2)
ϕ_{bs} = resistance factor for block shear (6.5.4.2)
ϕ_c = resistance factor for axial compression and combined axial compression and flexure (6.5.4.2)
ϕ_{cg} = resistance factor for truss gusset plate compression (6.5.4.2)
ϕ_{cs} = resistance factor for truss gusset chord splice (6.5.4.2)
ϕ_{da} = resistance factor during pile driving (6.5.4.2)
ϕ_{e1} = resistance factor for shear on the effective area of the weld metal in complete penetration welds; resistance factor for tension normal to the effective area of the weld metal in partial penetration welds (6.5.4.2)
ϕ_{e2} = resistance factor for shear parallel to the axis of the weld metal in partial penetration welds; resistance factor for shear in the throat of the weld metal in fillet welds (6.5.4.2)
ϕ_f = resistance factor for flexure (6.5.4.2) (A6.1.1)
ϕ_s = resistance factor for shear in bolts (6.5.4.2)
ϕ_{sc} = resistance factor for shear connectors (6.5.4.2)
ϕ_{sd} = resistance factor for shakedown (CB6.4.2.1)
ϕ_{st} = resistance factor for shear connectors in tension (6.5.4.2)
ϕ_T = resistance factor for torsion (6.5.4.2)
ϕ_t = resistance factor for tension in bolts (6.5.4.2)
ϕ_u = resistance factor for fracture on the net section of tension members (6.5.4.2)
ϕ_v = resistance factor for shear (6.5.4.2)
ϕ_{vu} = resistance factor for shear rupture of connection elements (6.5.4.2)
ϕ_{vy} = resistance factor for truss gusset plate shear yielding (6.5.4.2)
ϕ_w = resistance factor for web crippling (6.5.4.2)
ϕ_y = resistance factor for yielding on the gross section of tension members (6.5.4.2)

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

χ = reduction factor applied to the nominal compressive resistance of noncomposite rectangular box cross sections containing one or more longitudinally stiffened flange plates in the direction associated with column flexural buckling, and where $\lambda_{max} > \lambda_r$ (E6.1.1)

$\sum_{lusp}$ = summation over all longitudinally unstiffened cross-section plates (6.9.4.2.2a)

$\sum_{c}$ = summation over all the corner areas of a noncomposite box-section member (6.9.4.2.2a)

$\sum_{lsp}$ = summation over all the longitudinally stiffened cross-section plates (E6.1.1)

6.4—MATERIALS

6.4.1—Structural Steels

Structural steels shall conform to the requirements specified in Table 6.4.1-1, and the design shall be based on the minimum properties indicated.

The modulus of elasticity and the thermal coefficient of expansion of all grades of structural steel shall be assumed as 29,000 ksi and 6.5×10^{-6} in./in./°F, respectively.

AASHTO M 270M/M 270, Grade 36 (ASTM A709/A709M, Grade 36), may be used in thicknesses over 4.0 in. for nonstructural applications or bearing assembly components.

Quenched and tempered alloy steel structural shapes and seamless mechanical tubing with a specified maximum tensile strength not exceeding 140 ksi for structural shapes

C6.4.1

The term "yield strength" is used in these Specifications as a generic term to denote either the minimum specified yield point or the minimum specified yield strength.

The yield strength in the direction parallel to the direction of rolling is of primary interest in the design of most steel structures. In welded bridges, notch toughness is of equal importance. Other mechanical and physical properties of rolled steel, such as anisotropy, ductility, formability, and corrosion resistance, may also be important to ensure the satisfactory performance of the structure.

No specification can anticipate all of the unique or especially demanding applications that may arise. The literature on specific properties of concern and appropriate supplementary material production or quality requirements, provided in the AASHTO and ASTM Material Specifications and the AASHTO/AWS D1.5M/D1.5 *Bridge Welding Code*, should be considered, if appropriate.

AASHTO M 270M/M 270 (ASTM A709/A709M), Grade HPS 70W, has replaced AASHTO M 270M/M 270 (ASTM A709/A709M), Grade 70W, and AASHTO M 270M/M 270 (ASTM A709/A709M), Grade HPS 100W, has replaced AASHTO M 270M/M 270 (ASTM A709/A709M), Grade 100 and 100W in Table 6.4.1-1. The intent of these replacements is to encourage the use of HPS steel over the older bridge steels of the same strength level due to its enhanced properties. The older steels are still available, but are not recommended for use and should be used only with the approval of the Owner. The maximum available plate lengths of AASHTO M 270M/M 270 (ASTM A709/A709M), Grade HPS 70W and HPS 100W, are a function of the processing of the plate, with longer lengths of Grade HPS 70W produced as as-rolled plate. The maximum available plate lengths of these steels should be determined in consultation with the material producers.

or 145 ksi for seamless mechanical tubing may be used, provided that:

- the material meets all other mechanical and chemical requirements of AASHTO M 270M/M 270 (ASTM A709/A709M), Grade HPS 100W, and
- the design is based upon the minimum properties specified for AASHTO M 270M/M 270 (ASTM A709/A709M), Grade HPS 100W.

Except as specified herein, structural tubing shall be either cold-formed welded or seamless tubing conforming to ASTM A1085, ASTM A500/A500M, Grade B or Grade C, or ASTM A847/A847M; or hot-formed welded or seamless tubing conforming to ASTM A501/501M or ASTM A618/A618M.

Thickness limitations relative to rolled shapes and groups shall comply with ASTM A6/A6M.

Steel tubing for composite Concrete-Filled Steel Tubes (CFSTs) designed according to the provisions of Article 6.9.6 shall conform to the requirements of:

- American Petroleum Institute (API) Standard 5L, minimum Grade X42, PSL1 or PSL2, or
- ASTM A252/A252M, Grade 3, with all welds satisfying the requirements of the current version of the AWS D1.1/D1.1M *Structural Welding Code—Steel*.

ASTM A500/A500M cautions that structural tubing manufactured to that specification may not be suitable for applications involving dynamically loaded elements in welded structures where low-temperature notch-toughness properties may be important. As such, the use of this material should be carefully examined with respect to its specific application in consultation with the Owner. Where this material is contemplated for use in applications where low-temperature notch-toughness properties are deemed important, consideration should be given to requiring that the material satisfy the Charpy V-notch toughness requirements specified in Article 6.6.2. ASTM A1085 is an improved specification for cold-formed welded carbon steel hollow structural sections (HSS) that is more suitable for dynamically loaded structures.

Table 6.4.1-1—Minimum Mechanical Properties of Structural Steel by Shape, Strength, and Thickness

AASHTO Designation	M 270M/ M 270 Grade 36	M 270M/ M 270 Grade 50	M 270M/ M 270 Grade 50S	M 270M/ M 270 Grade 50W	M 270M/ M 270 Grade HPS 50W	M 270M/ M 270 Grade HPS 70W	M 270M/ M 270 Grade HPS 100W	
Equivalent ASTM Designation	A709/ A709M Grade 36	A709/ A709M Grade 50	A709/ A709M Grade 50S	A709/ A709M Grade 50W	A709/ A709M Grade HPS 50W	A709/ A709M Grade HPS 70W	A709/ A709M Grade HPS 100W	
Thickness of Plates, in.	Up to 4.0 incl.	Up to 4.0 incl.	Not Applicable	Up to 4.0 incl.	Up to 4.0 incl.	Up to 4.0 incl.	Up to 2.5 incl.	Over 2.5 to 4.0 incl.
Shapes	All Groups	All Groups	All Groups	All Groups	N/A	N/A	N/A	N/A
Minimum Tensile Strength, F_u, ksi	58	65	65	70	70	85	110	100
Specified Minimum Yield Point or Specified Minimum Yield Strength, F_y, ksi	36	50	50	50	50	70	100	90

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6.4.2—Pins, Rollers, and Rockers

Steel for pins, rollers, and expansion rockers shall conform to the requirements in Table 6.4.2-1, Table 6.4.1-1, or Article 6.4.7.

Expansion rollers shall be not less than 4.0 in. in diameter.

Table 6.4.2-1—Minimum Mechanical Properties of Pins, Rollers, and Rockers by Size and Strength

AASHTO Designation with Size Limitations	M 169 4.0 in. in dia. or less	M 102M/ M 102 to 20.0 in. in dia.	M 102M/ M 102 to 20.0 in. in dia.	M 102M/ M 102 to 10.0 in. in dia.	M 102M/ M 102 to 20.0 in. in dia.
ASTM Designation Grade or Class	A108 Grades 1016 to 1030 incl.	A668/ A668M Class C	A668/ A668M Class D	A668/ A668M Class F	A668/ A668M Class G
Specified Minimum Yield Point, F_y, ksi	36	33	37.5	50	50

6.4.3—Bolts, Nuts, and Washers

6.4.3.1—High-Strength Structural Fasteners

6.4.3.1.1—High-Strength Bolts

High-strength bolts used as structural fasteners shall conform to ASTM F3125/F3125M. The specified minimum tensile strengths of ASTM F3125 bolts shall be taken as specified in Table 6.4.3.1.1-1.

Type 3 bolts shall be used with weathering steels.

Corrosion-resistant coatings may be applied to Grade A325, F1852, and A490 bolts, as specified in ASTM F3125/F3125M.

Table 6.4.3.1.1-1—Specified Minimum Tensile Strengths

Grade	Specified Minimum Tensile Strength, ksi
A325	120
F1852	120
A490	150
F2280	150

C6.4.3.1.1

The ASTM F3125/F3125M standard bundles the previous ASTM A325, A325M, A490, A490M, F1852, and F2280 standards together and only the ASTM F3125/F3125M standard will be maintained moving forward. ASTM F3125/F3125M Grades F1852 and F2280 are the "twist-off" equivalents of Grades A325 and A490, respectively.

Galvanizing is not an acceptable option within ASTM F3125/F3125M for Grade A490 or F2280 bolts, but Grade A490 bolts may be coated with a zinc/aluminum coating in accordance with ASTM F1136/F1136M or F2833.

Rotational capacity testing of the fastener assemblies by the Manufacturer in accordance with Annex A2 of F3125/F3125M is required for Grade A325 and A490 fastener assemblies as specified in Article 11.5.5.4.2 of the *AASHTO LRFD Bridge Construction Specifications*.

Alternative fasteners satisfying the requirements of ASTM F3125/F3125M may be used subject to the approval of the Engineer.

6.4.3.1.2—Nuts Used with ASTM F3125 Bolts

Nuts used with ASTM F3125 bolts shall be as listed in ASTM F3125/F3125M as recommended or suitable for the bolt.

6.4.3.1.3—Washers Used with ASTM F3125 Bolts

Hardened washers used with ASTM F3125 bolts shall be as listed in ASTM F3125/F3125M as recommended or suitable for the bolt.

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6.4.3.1.4—Direct Tension Indicators

Direct tension indicators (DTIs) conforming to the requirements of ASTM F959/F959M may be used in conjunction with bolts, nuts, and washers. Captive DTI/nuts shall be considered permissible for use, provided both the DTI and hardened heavy hex nut meet the mechanical property requirements of their respective ASTM standards. DTIs that incorporate a self-indicating feature shall also be considered permissible for use. The use of the self-indicating feature to replace the use of feeler gauges shall be subject to the approval of the Engineer. DTIs installed over oversize or slotted holes in an outer ply shall satisfy the applicable provisions of Article 6.13.2.3.2.

C6.4.3.1.4

DTIs are washers with protrusions on one face that measure load by compressing the protrusions on the DTI with a proportional reduction in the gap in the spaces between the protrusions. Attaining the required tension is verified by the number of "gauge refusals," which are gaps that are too tight to permit the insertion of a feeler gauge of the prescribed thickness.

A DTI affixed to a hardened heavy hex nut by the fastener manufacturer is referred to as a captive DTI/nut. An assembly which incorporates a self-indicating feature to signal sufficient compression of the protrusions is referred to as a self-indicating DTI.

Installation provisions and verification procedures for DTIs are covered in the *AASHTO LRFD Bridge Construction Specifications* (AASHTO, 2017).

6.4.3.2—Low-Strength Steel Bolts

Low-strength steel bolts used as structural fasteners shall conform to ASTM A307 Grade A or B or an equivalent alternative subject to the approval of the Engineer. The specified minimum tensile strength of ASTM A307 bolts shall be taken as 60 ksi.

C6.4.3.2

Low-strength steel bolts, also referred to as unfinished, common, machine, ordinary, or rough bolts, are typically designated as ASTM A307 bolts. The ASTM standard for A307 bolts covers two grades of fasteners, Grades A and B. An equivalent alternative may be used for these bolts subject to the approval of the Engineer.

6.4.3.3—Fasteners for Structural Anchorage

6.4.3.3.1—Anchor Rods

Anchor rods shall conform to ASTM F1554.

C6.4.3.3.1

Fasteners for structural anchorage are covered in a separate article so that other requirements for high-strength bolts are not applied to anchor rods. The term "anchor rods," which is used in these Specifications, is considered synonymous with the term "anchor bolts," which has also been used.

6.4.3.3.2—Nuts Used with Anchor Rods

Nuts used with ASTM F1554 anchor rods shall conform to ASTM A563 or ASTM A194/A194M Grade 2H.

Nuts to be galvanized shall be heat treated Grade DH or DH3. All galvanized nuts should be lubricated with a lubricant containing a visible dye in accordance with ASTM A563, Supplementary Requirements S1 and S2.

6.4.4—Stud Shear Connectors

Welded stud shear connectors shall satisfy all applicable requirements of the AASHTO/AWS D1.5M/D1.5 *Bridge Welding Code*, and shall have a specified minimum yield and tensile strength of 50.0 ksi and 60.0 ksi, respectively.

C6.4.4

Specifications for material, manufacturing, physical properties, certification, and welding of stud shear connectors are provided in the AASHTO/AWS D1.5M/D1.5 *Bridge Welding Code*.

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6.4.5—Weld Metal

Weld metal shall conform to the requirements of the AASHTO/AWS D1.5M/D1.5 *Bridge Welding Code*.

C6.4.5

The AWS designation systems are not consistent. For example, there are differences between the system used for designating electrodes for shielded metal arc welding and the system used for designating submerged arc welding. Therefore, when specifying weld metal and/or flux by AWS designation, the applicable specification should be reviewed to ensure a complete understanding of the designation reference.

6.4.6—Cast Metal

6.4.6.1—Cast Steel and Ductile Iron

Cast steel shall conform to one of the following:

- AASHTO M 103M/M 103 (ASTM A27/A27M), Grade 70-36, unless otherwise specified;
- AASHTO M 163M/M 163 (ASTM A743/A743M) Grade CA15, unless otherwise specified.

Ductile iron castings shall conform to ASTM A536, Grade 60-40-18, unless otherwise specified.

6.4.6.2—Malleable Castings

Malleable castings shall conform to ASTM A47/A47M, Grade 35018. The specified minimum yield strength shall not be less than 35.0 ksi.

6.4.6.3—Cast Iron

Cast iron castings shall conform to AASHTO M 105 (ASTM A48/A48M), Class 30.

6.4.7—Stainless Steel

Stainless steel may conform to one of the following:

- ASTM A240/A240M,
- ASTM A276/A276M, or
- ASTM A666.

Stainless steel not conforming to the above-listed specifications may be used, provided that it conforms to the chemical and mechanical requirements of one of the above-listed specifications or other published specifications that establish its properties and suitability and that it is subjected to analyses, tests, and other controls to the extent and in the manner prescribed by one of the listed specifications.

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6.4.8—Cables

6.4.8.1—Bright Wire

Bright wire shall conform to ASTM A510/A510M.

6.4.8.2—Galvanized Wire

Galvanized wire shall conform to ASTM A641/A641M.

6.4.8.3—Epoxy-Coated Wire

Epoxy-coated wire shall conform to ASTM A99.

6.4.8.4—Bridge Strand

Bridge strand shall conform to ASTM A586 or ASTM A603.

6.4.9—Dissimilar Metals

Where steel components, including those made of stainless steel, are in contact with aluminum alloy components in the presence of an electrolyte, the aluminum shall be kept from direct contact with the steel. The steel components may include structural members, structural fasteners, washers and/or nuts.

C6.4.9

Galvanic corrosion can occur when steel components, including those made of stainless steel, are coupled with aluminum in the presence of an electrolyte. The aluminum part acts as an anode and will be sacrificed in time. This galvanic corrosion can be prevented by isolating the two materials from each other. Materials such as dielectric elastomeric spacers have also been used to keep aluminum alloy parts from direct contact with steel or other dissimilar metals. Additional information can be found in AASHTO (2013, 2015) under the Aluminum Design Section.

6.5—LIMIT STATES

6.5.1—General

The structural behavior of components made of steel or steel in combination with other materials shall be investigated for each stage that may be critical during construction, handling, transportation, and erection as well as during the service life of the structure of which they are part.

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

6.5.2—Service Limit State

The provisions of Article 2.5.2.6 shall apply as applicable.

Flexural members shall be investigated at the service limit state as specified in Articles 6.10 and 6.11.

C6.5.2

The intent of the service limit state provisions specified for flexural members in Articles 6.10 and 6.11 is primarily to prevent objectionable permanent deformations due to localized yielding that would impair rideability under expected severe traffic loadings.

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6.5.3—Fatigue and Fracture Limit State

Components and details shall be investigated for fatigue as specified in Article 6.6.

The fatigue load combinations specified in Table 3.4.1-1 and the fatigue live load specified in Article 3.6.1.4 shall apply.

Flexural members shall be investigated at the fatigue and fracture limit state as specified in Articles 6.10 and 6.11.

Bolts subject to tensile fatigue shall satisfy the provisions of Article 6.13.2.10.3.

Fracture toughness requirements shall be in conformance with Article 6.6.2.1.

6.5.4—Strength Limit State

6.5.4.1—General

Strength and stability shall be considered using the applicable strength load combinations specified in Table 3.4.1-1.

A special load combination for investigating the constructibility of steel superstructure components for which the force effects due to loads applied to the fully erected steelwork are quantified shall be considered as specified in Article 3.4.2.1.

6.5.4.2—Resistance Factors

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

- For flexure $\phi_f = 1.00$
- For shear $\phi_v = 1.00$
- For axial compression, steel only $\phi_c = 0.95$
- For axial compression and combined axial compression and flexure in composite CFSTs $\phi_c = 0.90$
- For axial compression, composite columns $\phi_c = 0.90$
- For tension, fracture in net section $\phi_u = 0.80$
- For tension, yielding in gross section $\phi_y = 0.95$
- For torsion $\phi_T = 1.00$
- For bearing on pins in reamed, drilled or bored holes and on milled surfaces $\phi_b = 1.00$
- For bolts bearing on material $\phi_{bb} = 0.80$
- For shear connectors $\phi_{sc} = 0.85$
- For ASTM F3125 bolts in tension $\phi_t = 0.80$
- For ASTM A307 bolts in tension $\phi_t = 0.80$
- For ASTM F1554 anchor rods in tension $\phi_t = 0.80$
- For ASTM A307 bolts in shear $\phi_s = 0.75$
- For ASTM F1554 anchor rods in shear $\phi_s = 0.75$
- For ASTM F3125 bolts in shear $\phi_s = 0.80$
- For block shear $\phi_{bs} = 0.80$
- For shear, rupture in connection element $\phi_{vu} = 0.80$

C6.5.4.2

Base metal ϕ as appropriate for resistance under consideration. The resistance factors used for compression loading for composite CFST members are larger than the resistance factors used for reinforced concrete members because composite CFST members have more predictable strength values under compression than reinforced concrete members (Marson and Bruneau, 2004; Roeder, Lehman, and Bishop, 2010).

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

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- For truss gusset plate compression $\phi_{cg} = 0.75$
- For truss gusset plate chord splices $\phi_{cs} = 0.65$
- For truss gusset plate shear yielding $\phi_{vy} = 0.80$
- For web crippling $\phi_w = 0.80$

- For weld metal in complete penetration welds:
  - shear on effective area $\phi_{e1} = 0.85$
  - tension or compression normal to effective area—same as base metal
  - tension or compression parallel to axis of the weld—same as base metal

- For weld metal in partial penetration welds:
  - shear parallel to axis of weld $\phi_{e2} = 0.80$
  - tension or compression parallel to axis of weld—same as base metal
  - compression normal to the effective area—same as base metal
  - tension normal to the effective area $\phi_{e1} = 0.80$

- For weld metal in fillet welds:
  - shear in throat of weld metal $\phi_{e2} = 0.80$

- For resistance during pile driving $\phi_{da} = 1.00$
- For axial resistance of piles in compression and subject to damage due to severe driving conditions where use of a pile tip is necessary:
  - H-piles $\phi_c = 0.50$
  - pipe piles $\phi_c = 0.60$

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

- For axial resistance of piles in compression under good driving conditions where use of a pile tip is not necessary:
  - H-piles $\phi_c = 0.60$
  - pipe piles $\phi_c = 0.70$

- For combined axial and flexural resistance of undamaged piles:
  - axial resistance for H-piles $\phi_c = 0.70$
  - axial resistance for pipe piles $\phi_c = 0.80$
  - flexural resistance $\phi_f = 1.00$

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

- For shear connectors in tension $\phi_{st} = 0.75$

6.5.5—Extreme Event Limit State

All applicable extreme event load combinations in Table 3.4.1-1 shall be investigated.

All resistance factors for the extreme event limit state, except those specified for bolts and shear connectors, shall be taken to be 1.0.

All resistance factors for ASTM F1554 bolts used as anchor rods for the extreme event limit state shall be taken to be 1.0.

Bolted slip-critical connections within a seismic load path shall be proportioned according to the requirements

C6.5.5

During earthquake motion, there is the potential for full reversal of design load and inelastic deformations of members or connections, or both. Therefore, slip of bolted joints located within a seismic load path cannot and need not be prevented during a seismic event. A special inspection of joints and connections, particularly in fracture-critical members, should be performed as described in the AASHTO *Manual for Bridge Evaluation* after a seismic event.

of Article 6.13.2.1.1. The connections shall also be proportioned to provide shear, bearing, and tensile resistance in accordance with Articles 6.13.2.7, 6.13.2.9, and 6.13.2.10, as applicable, at the extreme event limit state. Standard holes or short-slotted holes normal to the line of force shall be used in such connections.

To prevent excessive deformations of bolted joints due to slip between the connected plies under earthquake motions, only standard holes or short-slotted holes normal to the line of force are permitted in bolted joints located within a seismic load path. For such holes, the upper limit of $2.4dtF_u$ on the bearing resistance is intended to prevent elongations due to bearing deformations from exceeding approximately 0.25 in. It should be recognized, however, that the actual bearing load in a seismic event may be much larger than that anticipated in design and the actual deformation of the holes may be larger than this theoretical value. Nonetheless, the specified upper limit on the nominal bearing resistance should effectively minimize damage in moderate seismic events.

6.6—FATIGUE AND FRACTURE CONSIDERATIONS

6.6.1—Fatigue

6.6.1.1—General

Fatigue shall be categorized as load- or distortion-induced fatigue.

C6.6.1.1

In the AASHTO *Standard Specifications for Highway Bridges* (2002), the provisions explicitly relating to fatigue deal only with load-induced fatigue.

6.6.1.2—Load-Induced Fatigue

6.6.1.2.1—Application

The force effect considered for the fatigue design of a steel bridge detail shall be the live load stress range. For flexural members with shear connectors provided throughout their entire length, and with concrete deck reinforcement satisfying the provisions of Article 6.10.1.7, dead load and live load stresses and live load stress ranges for fatigue design at all sections in the member due to loads applied to the composite section may be computed assuming the concrete deck to be effective for both positive and negative flexure. The long-term composite section shall be used for the dead loads and the short-term composite section shall be used for the live loads.

Residual stresses shall not be considered in investigating fatigue.

C6.6.1.2.1

Concrete can provide significant resistance to tensile stress at service load levels. Recognizing this behavior will have a significantly beneficial effect on the computation of fatigue stress ranges in top flanges in regions of stress reversal and in regions of negative flexure. By utilizing shear connectors in these regions to ensure composite action in combination with the required one percent longitudinal reinforcement wherever the longitudinal tensile stress in the concrete deck exceeds the factored modulus of rupture of the concrete, crack length and width can be controlled so that full-depth cracks should not occur. When a crack does occur, the stress in the longitudinal reinforcement increases until the crack is arrested. Ultimately, the cracked concrete and the reinforcement reach equilibrium. Thus, the concrete deck may contain a small number of staggered cracks at any given section. Properly placed longitudinal reinforcement prevents coalescence of these cracks.

It has been shown that the level of total applied stress is insignificant for a welded steel detail. Residual stresses due to welding are implicitly included through the specification of stress range as the sole dominant stress parameter for fatigue design. This same concept of considering only stress range has been applied to rolled, bolted, and riveted details where far different residual stress fields exist. The application to nonwelded details is conservative. A complete stress range cycle may include both a tensile and compressive component. Only the live

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load plus dynamic load allowance effects need be considered when computing a stress range cycle; permanent loads do not contribute to the stress range. Tensile stresses propagate fatigue cracks. Material subjected to a cyclical loading at or near an initial flaw will be subject to a fully effective stress cycle in tension, even in cases of stress reversal, because the superposition of the tensile residual stress elevates the entire cycle into the tensile stress region.

These provisions shall be applied only to details subjected to a net applied tensile stress. In regions where the unfactored permanent loads produce compression, fatigue shall be considered only if the compressive stress is less than the maximum live load tensile stress caused by the Fatigue I load combination specified in Table 3.4.1-1.

Fatigue design criteria need only be considered for components or details subject to effective stress cycles in tension and/or stress reversal. If a component or detail is subject to stress reversal, fatigue is to be considered no matter how small the tension component of the stress cycle is since a flaw in the tensile residual stress zone could still be propagated by the small tensile component of stress. The decision on whether or not a tensile stress could exist is based on the Fatigue I Load Combination because this is the largest stress range a detail is expected to experience often enough to propogate a crack. When the tensile component of the stress range cycle resulting from this load combination exceeds the compressive stress due to the unfactored permanent loads, there is a net tensile stress in the component or at the detail under consideration, and therefore, fatigue must be considered. If the tensile component of the stress range does not exceed the compressive stress due to the unfactored permanent loads there is no net tensile stress. In this case, the stress cycle is compression—compression and a fatigue crack will not propogate beyond a heat-affected zone.

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

6.6.1.2.2—Design Criteria

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

C6.6.1.2.2

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

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

where:

γ = load factor specified in Table 3.4.1-1 for the fatigue load combination
(Δf) = force effect, live load stress range due to the passage of the fatigue load as specified in Article 3.6.1.4 (ksi)
$(\Delta F)_n$ = nominal fatigue resistance as specified in Article 6.6.1.2.5 (ksi)

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

but for the fatigue limit state,

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

6.6.1.2.3—Detail Categories

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

Except as specified herein for components and details on fracture-critical members, where the projected 75-year single lane Average Daily Truck Traffic $(ADTT)_{SL}$ is less than or equal to the applicable value specified in Table 6.6.1.2.3-2 for the Detail Category under consideration, the Fatigue II load combination specified in Table 3.4.1-1 may be used in combination with the nominal fatigue resistance for finite life specified in Article 6.6.1.2.5. Otherwise, the Fatigue I load combination shall be used in combination with the nominal fatigue resistance for infinite life specified in Article 6.6.1.2.5. The single-lane Average Daily Truck Traffic $(ADTT)_{SL}$ shall be computed as specified in Article 3.6.1.4.2.

For details on members in tension, or with a tension element, that are classified as Fracture-Critical Members, the Fatigue I load combination specified in Table 3.4.1-1 shall be used in combination with the nominal fatigue resistance for infinite life specified in Article 6.6.1.2.5.

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

C6.6.1.2.3

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

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

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

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

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

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

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

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

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

The use of fatigue details classified as Detail Category C or better is encouraged on longitudinal members in tension, or with a tension element, that are classified as Fracture-Critical Members. This does not apply to certain transverse and/or secondary members in these structures.

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

Considering the increased Fatigue I live load factor, γ_{LL}, for orthotropic deck details specified in Article 3.4.4 and the cycles per truck passage, n, for orthotropic deck plate connections subjected to wheel-load cycling, e.g., rib-to-deck welds, specified in Table 6.6.1.2.5-2, the 75-year $ADTT_{SL}$ equivalent to infinite life calculated from Eq. C6.6.1.2.3-1 is 740 trucks per day for fatigue Category C orthotropic deck-plate connection details and 3,700 trucks per day for all other fatigue Category C orthotropic deck details. Thus, finite life design may produce more economical designs for the detail under consideration on lower-volume roadways with 75-year $ADTT_{SL}$ values equal to or less than these values.

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

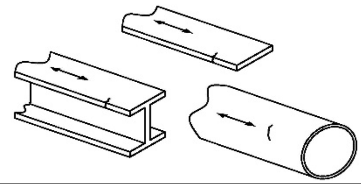
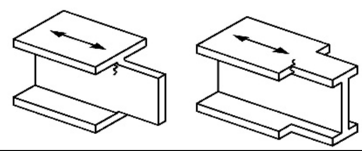
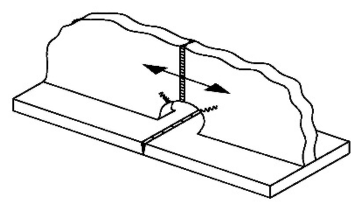
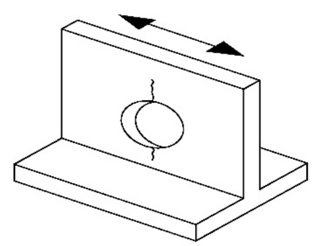
Description	Category	Constant A (ksi)^3	Threshold $(\Delta F)_{TH}$ ksi	Potential Crack Initiation Point	Illustrative Examples
Section 1—Plain Material away from Any Welding					
1.1 Base metal, except noncoated weathering steel, with rolled or cleaned surfaces, or base metal with thermal-cut edges with a surface roughness value of 1,000 μ-in. or less, but without re-entrant corners.	A	250×10^8	24	Away from all welds or structural connections	
1.2 Noncoated weathering steel base metal with rolled or cleaned surfaces designed and detailed in accordance with FHWA (1989), or noncoated weathering steel base metal with thermal-cut edges with a surface roughness value of 1,000 μ-in. or less, but without re-entrant corners.	B	120×10^8	16	Away from all welds or structural connections	
1.3 Base metal of members with re-entrant corners at copes, cuts, block-outs or other geometrical discontinuities made to the requirements of AASHTO/AWS D1.5, except weld access holes.	C	44×10^8	10	At any external edge	
1.4 Base metal of rolled cross sections with weld access holes made to the requirements of AASHTO/AWS D1.5.	C	44×10^8	10	In the base metal at the re-entrant corner of the weld access hole	
1.5 Base metal at the net section of open holes in members made to the requirements of AASHTO/AWS D1.5 (Brown et al. 2007), except as specified in Condition 1.6. All stresses shall be computed on the net section. (Note: See Condition 2.1 for holes with pretensioned high-strength bolts installed in standard-size holes.)	D	22×10^8	7	In the net section originating at the side of the hole	

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

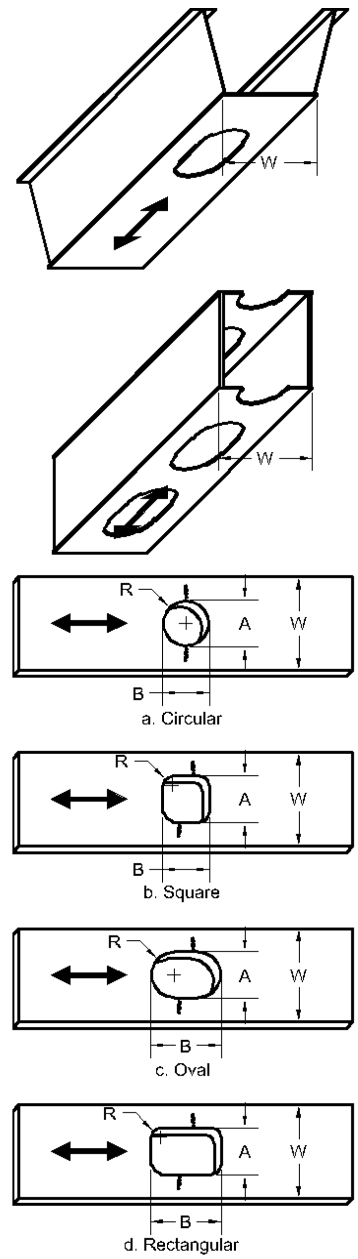
Description	Category	Constant A (ksi)^3	Threshold $(\Delta F)_{TH}$ ksi	Potential Crack Initiation Point	Illustrative Examples
1.6 Base metal at the net section of manholes or hand holes made to the requirements of AASHTO/AWS D1.5, in which the width of the hole is at least 0.30 times the width of the plate ($A \geq 0.30W$) (Bonachera Martin and Connor, 2017). The geometry of the hole shall be: a. circular; or b. square with corners filleted at a radius at least 0.10 the width of the plate ($R \geq 0.10W$); or c. oval ($B > A$), elongated parallel to the primary stress range; or d. rectangular ($B > A$), elongated parallel to the primary stress range, with corners filleted at a radius at least 0.10 times the width of the plate ($R \geq 0.10W$). All holes shall be centered on the plate under consideration, and all stresses shall be computed on the net section. (Note: Condition 1.5 shall apply for all holes in cross sections in which other smaller open holes or holes with nonpretensioned fasteners are located anywhere within the net section of the larger hole, and minimum edge distance requirements specified in Article 6.13.2.6.6 are satisfied for the smaller holes.)	C	44×10^8	10	In the net section originating at the side of the hole	 a. Circular b. Square c. Oval d. Rectangular

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

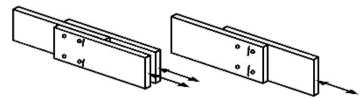
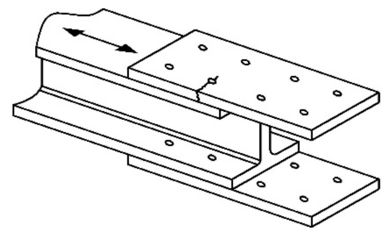
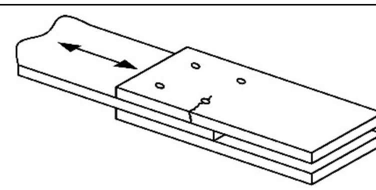
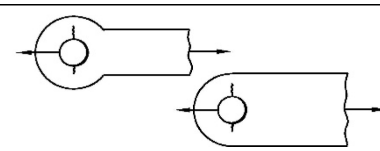
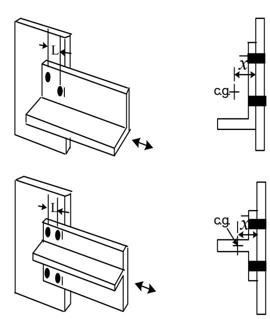
Description	Category	Constant A (ksi)^3	Threshold $(\Delta F)_{TH}$ ksi	Potential Crack Initiation Point	Illustrative Examples
Section 2—Connected Material in Mechanically Fastened Joints					
2.1 Base metal at the gross section of high-strength bolted joints designed as slip-critical connections with pretensioned high-strength bolts installed in holes drilled full size or subpunched and reamed to size—e.g., bolted flange and web splices and bolted stiffeners. (Note: see Condition 2.3 for bolt holes punched full size; see Condition 2.5 for bolted angle or T-section member connections to gusset or connection plates.)	B	120×10^8	16	Through the gross section near the hole	
2.2 Base metal at the net section of high-strength bolted joints designed as bearing-type connections but fabricated and installed to all requirements for slip-critical connections with pretensioned high-strength bolts installed in holes drilled full size or subpunched and reamed to size. (Note: see Condition 2.3 for bolt holes punched full size; see Condition 2.5 for bolted angle or T-section member connections to gusset or connection plates.)	B	120×10^8	16	In the net section originating at the side of the hole	
2.3 Base metal at the net or gross section of high-strength bolted joints with pretensioned bolts installed in holes punched full size (Brown et al., 2007); and base metal at the net section of other mechanically fastened joints, except for eyebars and pin plates, e.g., joints using ASTM A307 bolts or non-pretensioned high-strength bolts. (Note: see Condition 2.5 for bolted angle or T-section member connections to gusset or connection plates).	D	22×10^8	7	In the net section originating at the side of the hole or through the gross section near the hole, as applicable	
2.4 Base metal at the net section of eyebar heads or pin plates (Note: for base metal in the shank of eyebars or through the gross section of pin plates, see Condition 1.1 or 1.2, as applicable.)	E	11×10^8	4.5	In the net section originating at the side of the hole	
2.5 Base metal in angle or T-section members connected to a gusset or connection plate with high-strength bolted slip-critical connections. The fatigue stress range shall be calculated on the effective net area of the member, $A_e = UA_g$, in which $U = (1-\bar{x}/L)$ and where A_g is the gross area of the member. $\bar{x}$ is the distance from the centroid of the member to the surface of the gusset or connection plate and L is the out-to-out distance between the bolts in the connection parallel to the line of force. The effect of the moment due to the eccentricities in the connection shall be ignored in computing the stress range (McDonald and Frank, 2009). The fatigue category shall be taken as that specified for Condition 2.1. For all other types of bolted connections, replace A_g with the net area of the member, A_n, in computing the effective net area according to the preceding equation and use the appropriate fatigue category for that connection type specified for Condition 2.2 or 2.3, as applicable.	See applicable Category above	See applicable Constant above	See applicable Threshold above	Through the gross section near the hole, or in the net section originating at the side of the hole, as applicable	

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

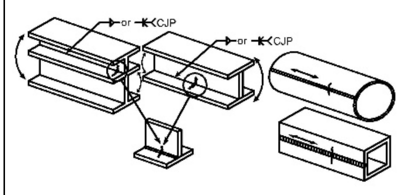
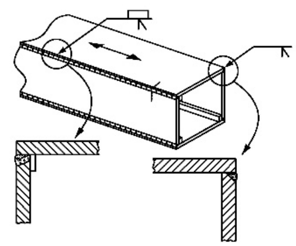
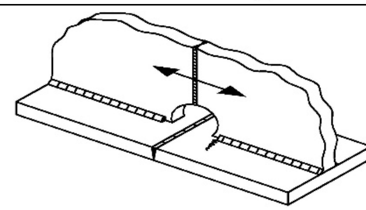
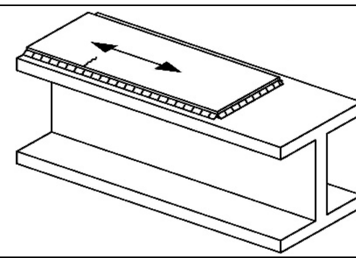
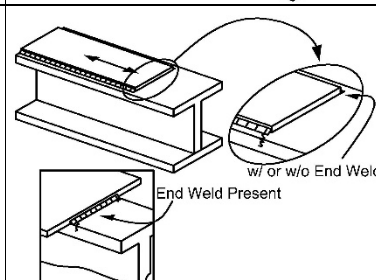
Description	Category	Constant A (ksi)^3	Threshold $(\Delta F)_{TH}$ ksi	Potential Crack Initiation Point	Illustrative Examples
Section 3—Welded Joints Joining Components of Built-up Members					
3.1 Base metal and weld metal in members without attachments built up of plates or shapes connected by continuous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

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

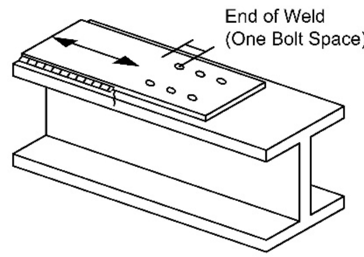
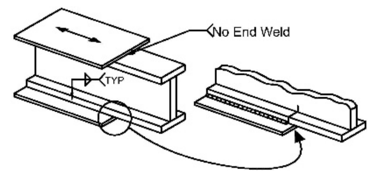
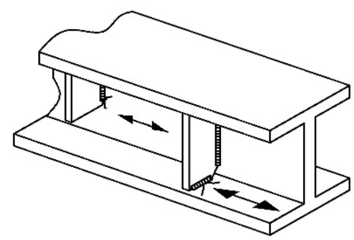
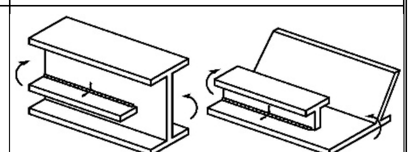
Description	Category	Constant A (ksi)^3	Threshold $(\Delta F)_{TH}$ ksi	Potential Crack Initiation Point	Illustrative Examples
Section 3—Welded Joints Joining Components of Built-Up Members (continued)					
3.6 Base metal at the termination of partial length welded cover plates with slip-critical bolted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

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

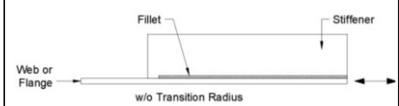
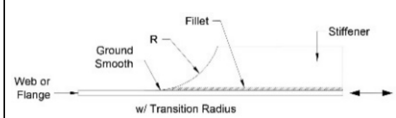
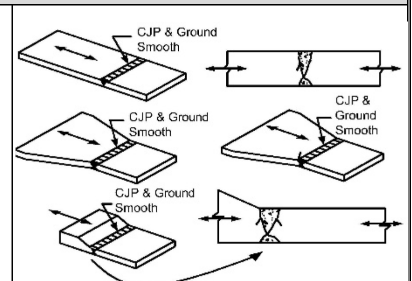
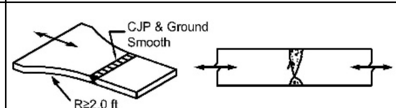
Description	Category	Constant A (ksi)^3	Threshold $(\Delta F)_{TH}$ ksi	Potential Crack Initiation Point	Illustrative Examples
Section 4—Welded Stiffener Connections (continued)					
4.3 Base metal at the termination of longitudinal stiffener-to-web or longitudinal stiffener-to-box flange welds:				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

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

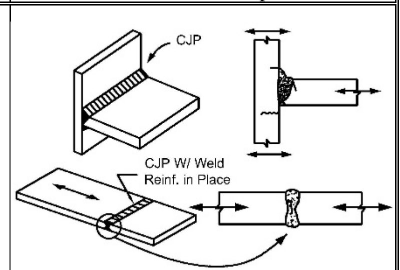
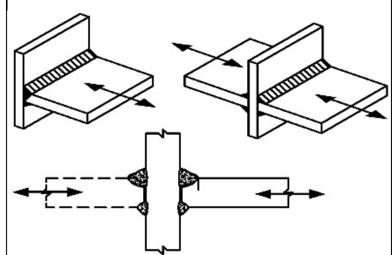
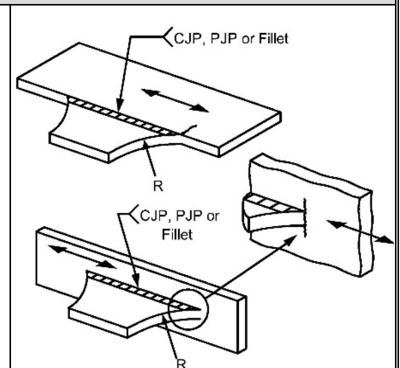
Description	Category	Constant A (ksi)^3	Threshold $(\Delta F)_{TH}$ ksi	Potential Crack Initiation Point	Illustrative Examples
5.3 Base metal and weld metal in or adjacent to the toe of complete joint penetration groove welded T or corner joints, or in complete joint penetration groove welded butt splices, with or without transitions in thickness having slopes no greater than 1:2.5 when weld reinforcement is not removed. (Note: cracking in the flange of the "T" may occur due to out-of-plane bending stresses induced by the stem).	C	44×10^8	10	From the surface discontinuity at the toe of the weld extending into the base metal or along the fusion boundary	
5.4 Base metal and weld metal at details where loaded discontinuous plate elements are connected with a pair of fillet welds or partial joint penetration groove welds on opposite sides of the plate normal to the direction of primary stress.	C as adjusted in Eq. 6.6.1.2.5-4	44×10^8	10	Initiating from the geometrical discontinuity at the toe of the weld extending into the base metal or initiating at the weld root subject to tension extending up and then out through the weld	
Section 6—Transversely Loaded Welded Attachments					
6.1 Base metal in a longitudinally loaded component at a transversely loaded detail (e.g., a lateral connection plate) attached by a weld parallel to the direction of primary stress and incorporating a transition radius R: With the weld termination ground smooth:				Near point of tangency of the radius at the edge of the longitudinally loaded component or at the toe of the weld at the weld termination if not ground smooth	
$R \geq 24$ in.	B	120×10^8	16		
24 in. $> R \geq 6$ in.	C	44×10^8	10		
6 in. $> R \geq 2$ in.	D	22×10^8	7		
2 in. $> R$	E	11×10^8	4.5		
For any transition radius with the weld termination not ground smooth.	E	11×10^8	4.5		
(Note: Condition 6.2, 6.3 or 6.4, as applicable, shall also be checked.)					

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

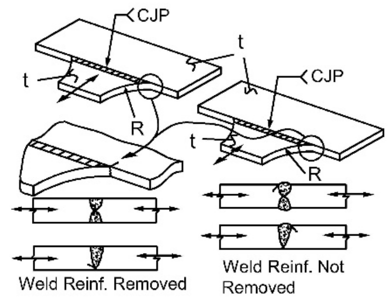
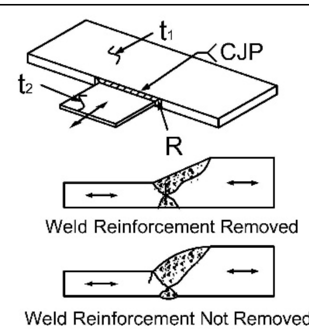
Description	Category	Constant A (ksi)^3	Threshold $(\Delta F)_{TH}$ ksi	Potential Crack Initiation Point	Illustrative Examples
Section 6—Transversely Loaded Welded Attachments (continued)					
6.2 Base metal in a transversely loaded detail (e.g., a lateral connection plate) attached to a longitudinally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

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

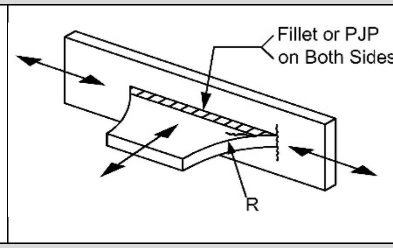
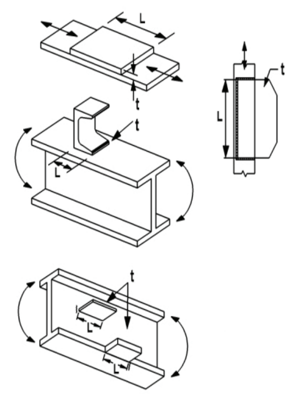
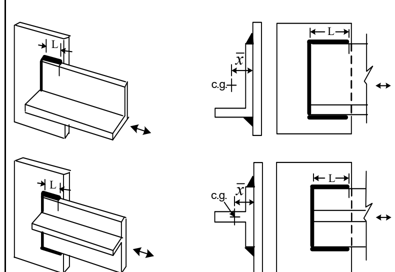
Description	Category	Constant A (ksi)^3	Threshold $(\Delta F)_{TH}$ ksi	Potential Crack Initiation Point	Illustrative Examples
Section 6—Transversely Loaded Welded Attachments (continued)					
6.4 Base metal in a transversely loaded detail (e.g., a lateral connection plate) attached to a longitudinally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

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

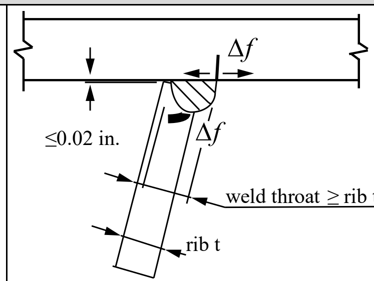
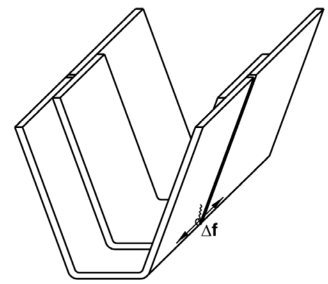
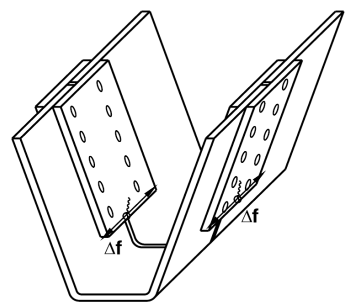
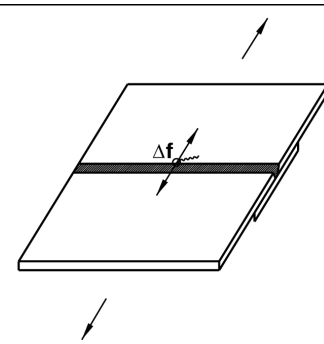
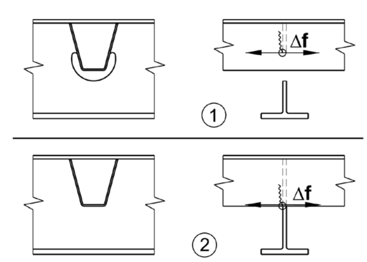
Description	Category	Constant A (ksi)^3	Threshold $(\Delta F)_{TH}$ ksi	Potential Crack Initiation Point	Illustrative Examples
Section 8—Orthotropic Deck Details					
8.1 Rib to Deck Weld—One-sided (60% min) penetration weld with root gap ≤ 0.02 in. prior to welding. Weld throat ≥ rib wall thickness.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

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

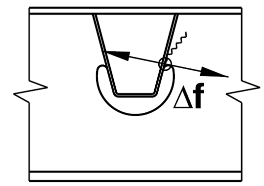
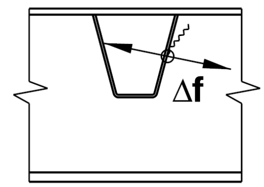
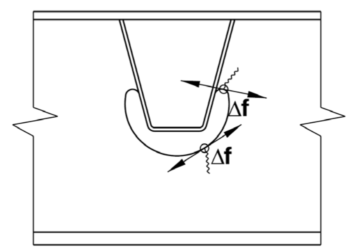
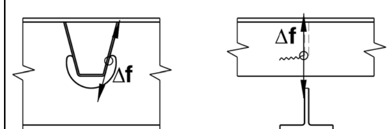
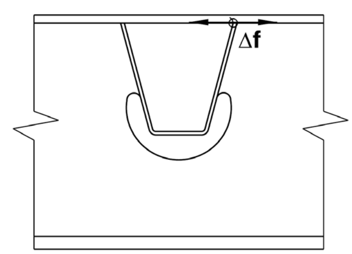
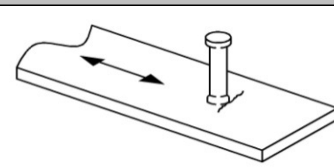
Description	Category	Constant A (ksi)^3	Threshold $(\Delta F)_{TH}$ ksi	Potential Crack Initiation Point	Illustrative Examples
Section 8—Orthotropic Deck Details (cont.)					
8.6 Rib to FB Weld (FB Web)—FB web at rib to FB weld (fillet, PJP, or CJP) Allowable Design Level 1 or 3	C (see Note 1)	44×10^8	10	See Figure	 
8.7 FB Cutout—Base metal at edge with "smooth" thermal-cut finish as per AWS D1.5 Allowable Design Level 1 or 3	A	250×10^8	24	See Figure	
8.8 Rib Wall at Cutout—Rib wall at rib to FB weld (fillet, PJP, or CJP) Allowable Design Level 1 or 3	C	44×10^8	10	See Figure	
8.9 Rib to Deck Plate at FB Allowable Design Level 1 or 3	C	44×10^8	10	See Figure	
Note 1: Where stresses are dominated by in-plane component at fillet or PJP welds, Eq. 6.6.1.2.5-4 shall be considered. In this case, Δf should be calculated at the mid-thickness and the extrapolation procedure as per Article 9.8.3.4.4 need not be applied.					
Section 9—Miscellaneous					
9.1 Base metal at stud-type shear connectors attached by fillet or automatic stud welding	C	44×10^8	10	At the toe of the weld in the base metal	

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

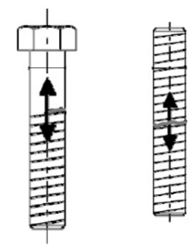
Description	Category	Constant A (ksi^3)	Threshold $(\Delta F)_{TH}$ ksi	Potential Crack Initiation Point	Illustrative Examples
Section 9—Miscellaneous (continued)					
9.2 Nonpretensioned high-strength bolts, common bolts, threaded anchor rods, and hanger rods with cut, ground, or rolled threads. Use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

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

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

6.6.1.2.4—Detailing to Reduce Constraint

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

C6.6.1.2.4

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

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

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

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

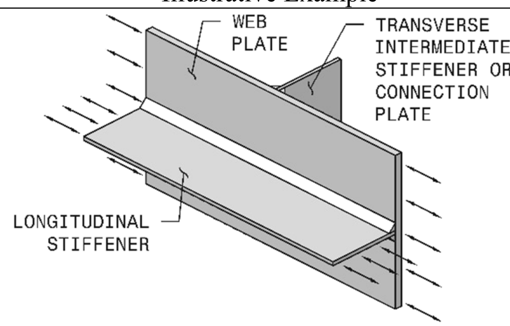
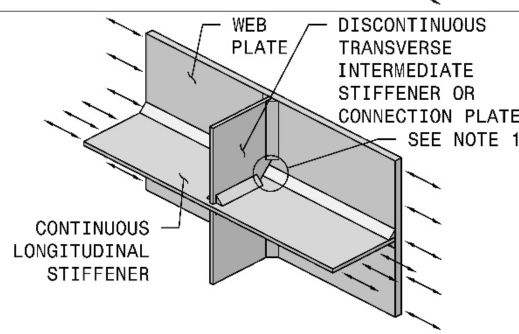
Description	Net Stress State at Location of Intersection (under Strength Load Combination I)	Illustrative Example
At locations where a longitudinal stiffener is on the opposite side of the web from a transverse intermediate stiffener or connection plate	Tension, Compression, or Reversal	
At locations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

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

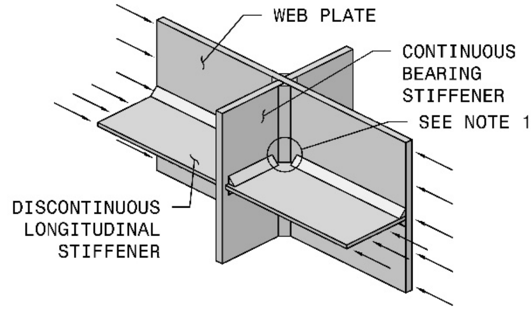
Description	Net Stress State at Location of Intersection (under Strength Load Combination I)	Illustrative Example
At locations where a longitudinal stiffener must intersect a bearing stiffener, or must intersect the outermost bearing stiffener if more than one pair of stiffeners is used Note the bearing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

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

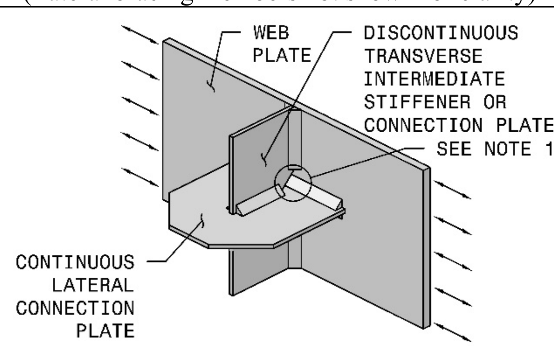
Description	Net Stress State at Location of Intersection (under Strength Load Combination I)	Illustrative Example (Lateral bracing members not shown for clarity)
When it is not practical to attach a lateral connection plate to a flange, and the lateral connection plate must be placed on the same side of the web as a transverse intermediate stiffener or connection plate. See also the	Tension, Compression, or Reversal	

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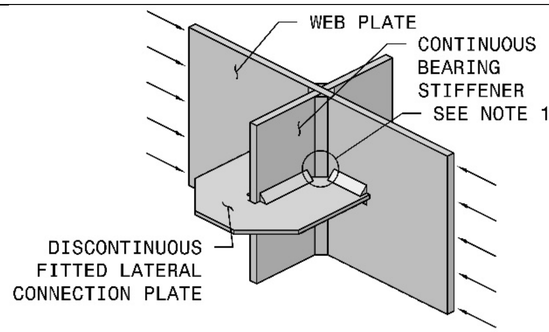
Table 6.6.1.2.4-2 (continued)—Details to Avoid Conditions Susceptible to Constraint-Induced Fracture at the Intersection of Lateral Connection Plates and Vertical Stiffeners Welded to the Web

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

6.6.1.2.5—Fatigue Resistance

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

- For the Fatigue I load combination and infinite life:

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

- For the Fatigue II load combination and finite life:

C6.6.1.2.5

The requirement on higher-traffic-volume bridges that the maximum stress range experienced by a detail be less than the constant-amplitude fatigue threshold provides a theoretically infinite fatigue life. This requirement is reflected in Eq. 6.6.1.2.5-1. Values of n for longitudinal members have been revised based on the calibration reported in Kulicki et al. (2015).

The fatigue resistance above the constant amplitude fatigue threshold, in terms of cycles, is inversely proportional to the cube of the stress range, e.g., if the stress range is reduced by a factor of 2, the fatigue life increases by a factor of 2^3. This is reflected in

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

in which:

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

where:

A = constant taken from Table 6.6.1.2.5-1 (ksi^3)

n = number of stress range cycles per truck passage taken from Table 6.6.1.2.5-2

$(ADTT)_{SL}$ = single-lane *ADTT* as specified in Article 3.6.1.4

$(\Delta F)_{TH}$ = constant-amplitude fatigue threshold taken from Table 6.6.1.2.5-3 (ksi)

Eq. 6.6.1.2.5-2. Orthotropic deck details that are connected to the deck plate (e.g., the rib-to-deck weld) are subjected to cycling from direct individual wheel loads. Thus, the passage of one design truck results in five fatigue load cycles as each axle produces one load cycle. The force effect (Δf) can be conservatively taken as the worst case from the five wheels or by application of Miner's Rule to determine the effective stress range from the group of wheels.

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

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

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

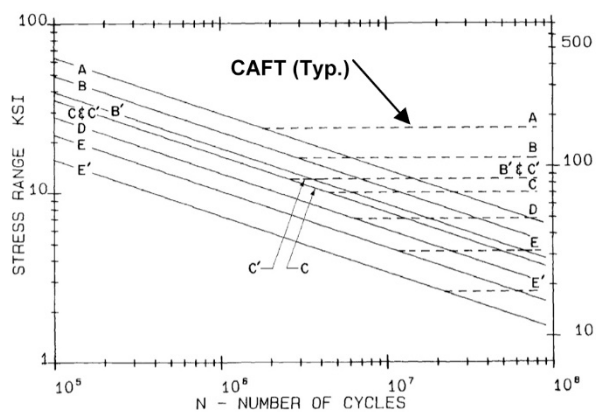


Figure C6.6.1.2.5-1—Stress Range Versus Number of Cycles

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

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

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

where:

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

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

t_p = thickness of loaded plate (in.)

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

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

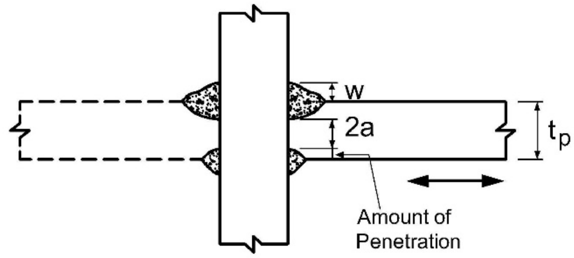


Figure C6.6.1.2.5-2—Loaded Discontinuous Plate Element Connected by a Pair of Partial Joint Penetration Groove Welds

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

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

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

Detail Category	Constant, A (ksi^3)
A	250.0×10^8
B	120.0×10^8
B′	61.0×10^8
C	44.0×10^8
C′	44.0×10^8
D	22.0×10^8
E	11.0×10^8
E′	3.9×10^8
ASTM F3125/F3125M, Grades A325 and F1852 Bolts in Axial Tension	17.1×10^8
ASTM F3125/F3125M, Grades A490 and F2280 Bolts in Axial Tension	31.5×10^8

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

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

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

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

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

Table 6.6.1.2.5-3—Constant-Amplitude Fatigue Thresholds

Detail Category	Threshold (ksi)
A	24.0
B	16.0
B′	12.0
C	10.0
C′	12.0
D	7.0
E	4.5
E′	2.6
ASTM F3125/F3125M, Grades A325 and F1852 Bolts in Axial Tension	31.0
ASTM F3125/F3125M, Grades A490 and F2280 Bolts in Axial Tension	38.0

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6.6.1.3—Distortion-Induced Fatigue

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

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

C6.6.1.3

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

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

6.6.1.3.1—Transverse Connection Plates

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

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

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

Where intermediate connecting diaphragms are used:

- on rolled beams in straight bridges with composite reinforced decks whose supports are normal or skewed not more than 10 degrees from normal, and
- with the intermediate diaphragms placed in contiguous lines parallel to the supports,

less than full-depth end angles or connection plates may be bolted or welded to the beam web to connect the diaphragms. The end angles or plates shall be at least two-thirds the depth of the web. For bolted angles, a minimum gap of 3.0 in. shall be provided between the top and bottom bolt holes and each flange. Bolt spacing requirements specified in Article 6.13.2.6 shall be satisfied. For welded angles or plates, a minimum gap of 3.0 in. shall be provided between the top and bottom of the end-angle or plate welds and each flange; the heel and toe of the end angles or both sides of the connection plate, as applicable, shall be welded to the beam web. Welds

C6.6.1.3.1

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

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

For box sections, webs are often joined to top flanges and cross-frame connection plates and transverse stiffeners are installed, and then these assemblies are attached to the common box flange. In order to weld the webs continuously to the box flange inside the box section, the details in this case should allow the welding head to clear the bottom of the connection plates and stiffeners. A similar detail may also be required for any intermediate transverse stiffeners that are to be attached to the box flange. Suggested details are shown in AASHTO/NSBA (2016). The Engineer is advised to consult with fabricators regarding the preferred approach for fabricating the box section and provide alternate details on the plans, if necessary.

shall not be placed along the top and bottom of the end angles or connection plates.

6.6.1.3.2—Lateral Connection Plates

If it is not practical to attach lateral connection plates to flanges, lateral connection plates on stiffened webs should be located a vertical distance not less than one-half the width of the flange above or below the flange. Lateral connection plates attached to unstiffened webs should be located at least 6.0 in. above or below the flange but not less than one-half of the width of the flange, as specified above.

The ends of lateral bracing members on the lateral connection plate shall be kept a minimum of 4.0 in. from the web and any transverse stiffener.

Lateral connection plates shall be centered on transverse stiffeners, whether or not the plate is on the same side of the web as the stiffener. The detailing of welded lateral connection plates shall also satisfy the provisions of Article 6.6.1.2.4.

C6.6.1.3.2

The specified minimum distance from the flange is intended to reduce the concentration of out-of-plane distortion in the web between the lateral connection plate and the flange to a tolerable magnitude. It also provides adequate electrode access and moves the connection plate closer to the neutral axis of the girder to reduce the impact of the weld termination on fatigue strength.

This requirement reduces potential distortion-induced stresses in the gap between the web or stiffener and the lateral members on the lateral plate. These stresses may result from vibration of the lateral system. It also facilitates painting and field inspection.

The typical detail where the lateral connection plate is on the same side of the web as the transverse stiffener is illustrated in Table 6.6.1.2.4-2. With regard to the second detail shown in Table 6.6.1.2.4-2, when there is interference from a cross-frame or diaphragm attached to the bearing stiffener, or from a transverse floorbeam or internal plate diaphragm, two separate lateral connection plates may be used on each side of the bearing stiffener, floorbeam, or internal plate diaphragm. Each individual connection plate is to be welded to the bearing stiffener, floorbeam, or internal plate diaphragm as shown, with the welds either stopped short of any free edges as shown in Table 6.6.1.2.4-2 or wrapped for sealing as specified in Article 6.13.3.7. When more than one pair of bearing stiffeners is used, two separate lateral connection plates may again be used with each individual connection plate welded to the outer face of the outermost stiffeners. Should a single lateral connection plate be used in this case, it need not be fitted around each individual stiffener, and need only be welded to the outer face of the outermost stiffeners.

6.6.1.3.3—Orthotropic Decks

Detailing shall satisfy all requirements of Article 9.8.3.6.

C6.6.1.3.3

The purpose of this provision is to control distortion-induced fatigue of deck details subject to local secondary stresses due to out-of-plane bending.

6.6.2—Fracture

6.6.2.1—Member or Component Designations and Charpy V-Notch Testing Requirements

Member or component designations as primary or secondary shall be determined according to Table 6.6.2.1-1, unless otherwise designated on the contract plans.

Primary members or components, or portions thereof, subject to a net tensile stress under Strength Load Combination I shall be designated on the contract

C6.6.2.1

Designation of members or components as shown in Table 6.6.2.1-1 is essential because this information affects various aspects of fabrication such as material purchasing, welding, and inspection. Fillet and partial joint penetration groove welds on primary members and their components have special requirements in the AASHTO/AWS D1.5M/D1.5 *Bridge Welding Code* for plans. Unless otherwise indicated on the contract plans, Charpy V-notch testing shall be required for primary members or components that are subject to a net tensile stress, or for portions thereof located in designated tension zones, under Strength Load Combination I, except for diaphragm and cross-frame members and mechanically fastened or welded cross-frame gusset plates in horizontally-curved bridges.

magnetic particle testing. In addition, for complete joint penetration groove welds in primary members, the type and frequency of radiographic or ultrasonic testing depends on whether the component is subject to tension or compression, and the acceptance criteria are different. Certain fabrication methods, such as punching holes full size, are limited to certain secondary and primary members according to Article 11.4.8.1.1 of the *AASHTO LRFD Bridge Construction Specifications*.

Therefore, Article 6.6.2.1 requires that primary members or components, or portions thereof, subject to a net tensile stress under Strength Load Combination I be designated on the contract plans. Designation of secondary members or components, as listed in Table 6.6.2.1-1, as primary members or components should be carefully scrutinized as this will invoke more costly and complex fabrication and testing requirements that do not add significant value and are not necessary. Secondary members or components that are specifically designated as primary members or components by the Owner should be indicated as such on the contract plans, and also whether or not those members or components, or portions thereof, are subject to a net tensile stress under Strength Load Combination I.

Table 6.6.2.1-1 is not necessarily all-inclusive and the designation of any members or components that are not listed in the table is subject to the judgment of the Engineer.

The basis and philosophy for the supplemental impact requirements specified in the *AASHTO Standard Specifications for Transportation Materials and Methods of Sampling and Testing* is given in AISI (1975).

Specifying that Charpy V-notch testing be performed for members, components, or portions thereof for which such testing is not required according to the provisions specified herein, e.g., secondary members, adds additional complexity and cost without providing any significant additional value. Designation of entire members or components as subject to a net tensile stress, rather than merely their tension zones where applicable, also adds unnecessary cost and is inconsistent with the objectives of the fracture control plan specified in the *AASHTO/AWS D1.5M/D1.5 Bridge Welding Code*. Although classified as primary members, some cross-frames and diaphragms and their components in curved bridges have become subject to Charpy V-notch testing requirements, yet in many years of practice, no specific concerns related to the fatigue and fracture performance of these members have been demonstrated.

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Table 6.6.2.1-1—Member or Component Designations

Member or Component Description	Member or Component Designation
Girders, beams, stringers, floorbeams, bent caps, bulkheads, and straddle beams	Primary
Truss chords, diagonals, verticals, and portal and sway bracing members	Primary
Arch ribs and built-up or welded tie girders	Primary
Rigid frames	Primary
Gusset plates and splice plates in trusses, arch ribs, tie girders, and rigid frames	Primary
Splice plates and cover plates in girders, beams, stringers, floorbeams, bent caps, and straddle beams	Primary
Bracing members supporting arch ribs	Primary
Permanent bottom-flange lateral bracing members and mechanically fastened or welded bottom-flange lateral connection plates in straight and horizontally-curved bridges	Primary
Top flange lateral bracing members or struts and top flange lateral connection plates in straight and horizontally-curved bridges	Secondary
Diaphragm and cross-frame members and mechanically fastened or welded cross-frame gusset plates in straight bridges	Secondary
Diaphragm and cross-frame members and mechanically fastened or welded cross-frame gusset plates in horizontally-curved bridges	Primary
Diaphragm and cross-frame members, and mechanically fastened or welded cross-frame gusset plates and bearing stiffeners at supports in bridges located in Seismic Zones 3 or 4	Primary
Bearings, filler plates, sole plates, and masonry plates	Secondary
Mechanically fastened or welded longitudinal web and flange stiffeners	Primary
Mechanically fastened or welded transverse intermediate web stiffeners, transverse flange stiffeners, bearing stiffeners, and transverse connection plates	Secondary
Mechanically fastened or welded batten plates and stay plates, lacing, and continuous nonperforated or perforated plates in built-up members	Primary
Eyebars and hanger plates	Primary
Miscellaneous structural components or attachments not mentioned above joining two primary members	Primary
Miscellaneous nonstructural components or attachments (e.g., expansion dams, drainage material, brackets, other miscellaneous attachments)	Secondary

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Charpy V-notch impact energy requirements shall be in accordance with AASHTO M 270M/M 270 (ASTM A709/A709M) for the specified temperature zone.

The appropriate temperature zone for the Charpy V-notch requirements shall be determined from the applicable minimum service temperature specified in Table 6.6.2.1-2, and shall be designated in the contract documents.

Table 6.6.2.1-2—Temperature Zone Designations for Charpy V-notch Requirements

Minimum Service Temperature	Temperature Zone
0°F and above	1
−1°F to −30°F	2
−31°F to −60°F	3

The Charpy V-notch impact energy requirements, specified in AASHTO M 270M/M 270 (ASTM A709/A709M) and shown in Table C6.6.2.1-1, vary depending on the grade of steel, and the applicable minimum service temperature. Fracture-Critical Members (FCMs) are subject to more stringent Charpy V-notch impact energy requirements than nonfracture-critical components. Steel grades for nonfracture-critical tension members or components subject to Charpy V-notch testing are designated with the suffix T in AASHTO M 270M/M 270 (ASTM A709/A709M), and steel grades for fracture-critical tension members or components subject to Charpy V-notch testing are designated with the suffix F.

Table C6.6.2.1-1—CVN Impact Energy Requirements

Grade (Y.P./Y.S.)	Thickness (in.)	Fracture-Critical				Nonfracture-Critical		
		Min. Test Value Energy (ft-lbf.)	Zone 1 (ft-lbf. @ °F)	Zone 2 (ft-lbf. @ °F)	Zone 3 (ft-lbf. @ °F)	Zone 1 (ft-lbf. @ °F)	Zone 2 (ft-lbf. @ °F)	Zone 3 (ft-lbf. @ °F)
36	$t \leq 4$	20	25 @ 70	25 @ 40	25 @ 10	15 @ 70	15 @ 40	15 @ 10
50/50S/ 50W	$t \leq 2$	20	25 @ 70	25 @ 40	25 @ 10	15 @ 70	15 @ 40	15 @ 10
	$2 < t \leq 4$	24	30 @ 70	30 @ 40	30 @ 10	20 @ 70	20 @ 40	20 @ 10
HPS 50W	$t \leq 4$	24	30 @ 10	30 @ 10	30 @ 10	20 @ 10	20 @ 10	20 @ 10
HPS 70W	$t \leq 4$	28	35 @ −10	35 @ −10	35 @ −10	25 @ −10	25 @ −10	25 @ −10
HPS 100W	$t \leq 2½$	28	35 @ –30	35 @ –30	35 @ −30	25 @ –30	25 @ –30	25 @ −30
	$2½ < t \leq 4$	36	not permitted	not permitted	not permitted	35 @ –30	35 @ –30	35 @ −30

6.6.2.2—Fracture-Critical Members

The Engineer shall have the responsibility for identifying and designating on the contract plans which primary members or portions thereof are fracture-critical members (FCMs). The contract documents shall require that all members meeting the definition of an FCM be fabricated according to the provisions of Clause 12 specified in the AASHTO/AWS D1.5M/D1.5 *Bridge Welding Code*. Members or portions thereof that are not subject to a net tensile stress under Strength Load Combination I shall not be designated as FCMs.

A primary member or portion thereof subject to tension, for which the redundancy is not known by engineering judgment but which is demonstrated to have redundancy in the presence of a simulated fracture in that member through the use of a refined analysis, shall be designated as a System Redundant Member (SRM) in the contract documents. The contract documents shall further indicate that SRMs are to be fabricated according to the provisions of Clause 12 specified in the AASHTO/AWS

C6.6.2.2

Secondary members and diaphragm or cross-frame members in horizontally-curved bridges should not be designated as FCMs.

When designating some rolled shapes as FCMs, it may not be possible to secure shapes that are produced using fine-grained practices. In such cases, the fine-grained practices should be waived.

Where refined analysis has demonstrated that collapse would not occur following simulated failure of a member for which the redundancy is not known by engineering judgment, the members or portions thereof should not be subjected to the hands-on in-service inspection requirements described in 23 CFR 650. FHWA (June 20, 2012) Memorandum recommends identifying such members or portions thereof as System Redundant Members (SRMs), and noting in the contract documents that SRMs are to be fabricated in accordance with Clause 12 of the AASHTO/AWS D1.5M/D1.5 *Bridge Welding Code*.

D1.5M/D1.5 *Bridge Welding Code*. The criteria, assumptions, and other pertinent information related to the refined analysis used to demonstrate the redundancy shall be retained and included in the inspection records or permanent bridge file.

Any attachment, except for bearing sole plates, having a length in the direction of the tension stress greater than 4.0 in. that is welded to a tension area of a component of an FCM shall be considered part of the tension component and shall be considered fracture critical.

The criteria for a refined analysis used to demonstrate the presence of redundancy in the structure have not yet been codified. Therefore, the loading cases to be studied, location of potential cracks, degree to which the dynamic effects associated with a fracture are included in the analysis, and fineness of models and choice of element type should all be agreed upon by the Owner and the Engineer. The ability of a particular software product to adequately capture the complexity of the problem should also be considered and the choice of software should be mutually agreed upon by the Owner and the Engineer. Relief from the full factored loads associated with the Strength I Load Combination of Table 3.4.1-1 should be considered, as should the number of loaded design lanes versus the number of striped traffic lanes.

The exclusion of bearing sole plates from FCM designation is because they are located in regions of low or zero tensile stress at the ends of spans or in regions of compression at interior supports of continuous spans. Preheating to fracture critical temperatures may also be harmful to some bearing materials. Bearings and sole plates are also designated as secondary members in Table 6.6.2.1-1, exempt from FCM designation.

6.7—GENERAL DIMENSION AND DETAIL REQUIREMENTS

6.7.1—Effective Length of Span

Span lengths shall be taken as the distance between centers of bearings or other points of support.

6.7.2—Dead Load Camber and Detailing of Structural Components

Steel structures should be cambered during fabrication to compensate for dead load deflection and vertical alignment.

Deflection due to steel weight and concrete weight shall be reported separately. Deflections due to future wearing surfaces or other loads not applied at the time of construction shall be reported separately.

Vertical camber shall be specified to account for the computed dead load deflection.

When staged deck placement or phased construction is specified, the sequence of load application and the change in the composite stiffness during the different stages of the deck placement shall be considered when establishing girder cambers.

Selective changes to component length, as appropriate, may be used for truss, arch, and cable-stayed systems to:

- adjust the dead load deflection to comply with the final geometric position,
- reduce or eliminate rib shortening, and

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Article 6.7.2 requires that, generally, the effects of staged deck placement as well as the impact of phased construction are to be considered when establishing girder cambers. In addition, these effects are an important consideration subsequently in setting screed requirements for construction. AASHTO/NSBA (2019a) and NHI (2015b) provide further guidance on these considerations.

Skewed and curved I-girder bridges exhibit torsional displacements, or twisting, of the individual girders and of the overall bridge cross section under load, including the loads during construction. As a result, the girder webs can only be plumb in one load condition. The fit condition of an I-girder bridge refers to the targeted girder dead load geometry for which the cross-frames or diaphragms are detailed to connect to the girders. The girder geometry used by the Fabricator/Detailer to detail the cross-frames or diaphragms is based on the vertical deflections provided in the contract documents that are associated with the selected fit condition.

The fit condition influences the vertical orientation of the girders under various stages of dead and live load, the associated rotation demands on the bearings, the

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- adjust the dead load moment diagram in indeterminate structures.

The contract documents should state the fit condition for which the cross-frames or diaphragms are to be detailed for the following I-girder bridges:

- straight bridges where one or more support lines are skewed more than 20 degrees from normal;
- horizontally-curved bridges where one or more support lines are skewed more than 20 degrees from normal and with an L/R in all spans less than or equal to 0.03; and
- horizontally-curved bridges with or without skewed supports and with a maximum L/R greater than 0.03.

where:

L = actual span length bearing to bearing along the centerline of the bridge (ft)
R = girder radius at the centerline of the bridge (ft)

A Total Dead Load Fit should not be specified for curved I-girder bridges with or without skew and with a maximum L/R greater than 0.03. Such bridges may be detailed for Steel Dead Load Fit or No-Load Fit, unless the maximum L/R is greater than or equal to 0.2. In this case, either the bridge should be detailed for No-Load Fit or the additive locked-in force effects associated with the Steel Dead Load Fit detailing should be considered.

The provisions of Article 2.5.2.6.1 related to bearing rotations also shall be considered.

forces required for assembly of the steel during construction, and the internal forces for which the cross-frames or diaphragms and girders must be designed (NSBA, 2019b). The fit condition must always be indicated for the bridge types specified in Article 6.7.2 so that the Fabricator/Detailer can complete the shop drawings and fabricate the bridge components. Since the fit condition directly influences the cross-frame fabricated geometry, as well as the above items, the fit condition should ideally be selected by the Engineer, who best knows the loads and capacities of the structural members, in consultation with a Fabricator and Erector. The fit condition generally should be selected to accomplish the following objectives, in order of priority: 1) facilitate the construction of the bridge; 2) offset large girder dead load twist rotations and corresponding lateral movements at the deck joints and barrier rails, which occur predominantly at sharply skewed abutment lines; 3) reduce the dead load forces in the cross-frames or diaphragms and the flange lateral bending stresses in the girders in straight skewed bridges; and 4) select the load condition in which the girders will be approximately plumb. The plumbness condition should not be specified by the Engineer; girder plumbness is dictated by the fit condition.

The three most common fit conditions are:

- No-Load Fit (NLF),
- Steel Dead Load Fit (SDLF), and
- Total Dead Load Fit (TDLF).

For the bridge types specified in Article 6.7.2, the contract documents should indicate the fit condition for which the cross-frames or diaphragms are to be detailed.

NLF refers to the condition where the cross-frames or diaphragms are detailed to fit to the girders in their fabricated, plumb, fully cambered position under zero load. In this case, the girder webs will be out-of-plumb after any dead load is applied, except at non-skewed bearing lines. At skewed bearing lines, this out-of-plumbness should be considered in the detailing of the deck, deck joint, barrier joint, and bearings, as applicable. Girder dead load layovers at highly skewed abutments can be substantial when NLF detailing is employed.

SDLF refers to the condition where the cross-frames or diaphragms are detailed to fit to the girders in their ideally plumb as-deflected positions under the self-weight of the steel at the completion of the erection. SDLF is common and effective for straight I-girder bridges and for most horizontally-curved I-girder bridges (NSBA, 2016a; NSBA, 2016b). SDLF is favored for ease of construction of straight skewed I-girder bridges since the steel dead load corresponds to the condition where all the girders are erected and all the cross-frames or diaphragms are connected. Straight bridges detailed for this condition typically require less forced fit-up of the cross-frames or diaphragms, particularly if the girders are allowed to deflect under their self-weight before

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installing the cross-frames or diaphragms. Where an SDLF is employed, dead loads applied after the completion of the steel erection will introduce a final and permanent twist into the girders.

TDLF refers to the condition where the cross-frames or diaphragms are detailed to fit to the girders in their ideally plumb as-deflected positions under the total dead load. The total dead load typically includes the weight of the concrete deck, but not the weight of any superimposed dead loads. In phased construction or where superimposed dead loads cause significant girder deflection, it may also be desirable to consider the effect of the superimposed dead loads. In the case of TDLF detailing, the Erector will leave the site with the girders out-of-plumb since the total dead load will not yet have been applied. However, TDLF gives approximately plumb girder webs once the bridge is subjected to its total dead load. For straight skewed I-girder bridges, or for horizontally-curved skewed I-girder bridges with L/R in all spans less than or equal to 0.03, TDLF can be effective for span lengths up to approximately 200 ft (NSBA, 2016a; NSBA, 2016b). Practice has demonstrated that the use of TDLF for longer-span straight skewed bridges, and for horizontally-curved bridges with or without skew and with a maximum L/R greater than 0.03, can potentially render the bridge unconstructable because the girders cannot be twisted as readily in these bridges to facilitate erection (NSBA, 2016a; NSBA, 2016b). Curved I-girders, in particular, resist the twisting required to fit the steel together via their coupled resistance to major-axis bending and twisting. This behavior tends to increase the difficulty of fitting the steel together during the steel erection.

When either SDLF or TDLF is specified, the girders are fabricated plumb and are twisted out-of-plumb during steel erection in the opposite direction that they tend to twist under the application of the corresponding targeted dead load to connect them with the cross-frames or diaphragms. This compensating twist in the girders is achieved by introducing locked-in internal forces in the system during the erection.

Although the use of refined analysis methods is not required for these bridges, these methods, when utilized, do allow for consideration of dead load cross-frame or diaphragm forces and flange lateral bending stresses. In straight skewed I-girder bridges, these dead load force effects are partially offset by the corresponding locked-in force effects at the completion of the steel erection (White et al., 2012; White et al., 2015). It is important to recognize that the dead load force effects, when determined from a refined analysis model, typically do not include the locked-in force effects due to SDLF or TDLF detailing of the cross-frames or diaphragms. That is, the analysis model corresponds to the assumption of NLF.

White et al. (2015) describe a procedure for directly determining the locked-in force effects from SDLF or TDLF detailing as part of a refined analysis. Otherwise,

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in lieu of considering the locked-in force effects directly within the structural analysis, the approximations discussed below may be employed. For straight skewed I-girder bridges detailed for TDLF where the skew index, I_s, defined in Article 4.6.3.3.2 is greater than 1.0, the recommendations provided in Article C6.7.4.2 to lessen transverse stiffness effects should be applied and the direct calculation of the influence of the dead-load fit detailing on the girder vertical reactions and girder major-axis bending stresses via a refined analysis should be considered. In addition, for curved and skewed I-girder bridges with a maximum L/R greater than 0.03, calculation of the influence of dead load fit detailing on the girder vertical reactions via a refined analysis should be considered (White et al., 2015). The following procedures do not address the effects due to the bracket loads supporting the eccentric deck overhangs during deck construction. These effects may be estimated separately as described in Article C6.10.3.4.1 and combined as appropriate with the other dead load effects discussed below.

For straight skewed I-girder bridges that are detailed for TDLF, the total dead load cross-frame or diaphragm forces and flange lateral bending stresses, when determined from a refined analysis not including the influence of dead load fit detailing, may be reduced to account for the corresponding locked-in force effects introduced into the structural system during the steel erection. In this case, a net reduced load factor, $(\gamma_p)_{red}$, to be applied to the unfactored total dead load cross-frame or diaphragm forces and flange lateral bending stresses may be conservatively taken as:

$$\left(\gamma_p\right)_{\text{red}} = \left(\gamma_p - 0.4\right) \tag{C6.7.2-1}$$

where:

γ_p = the maximum load factor for DC specified in Table 3.4.1-2, or the maximum load factor specified in Article 3.4.2.1 for DC and any construction loads that are applied to the fully erected steelwork, as applicable.

The above net reduced load factor, applied to the unfactored total dead load cross-frame or diaphragm forces and flange lateral bending stresses, gives a lower-bound estimate of the corresponding beneficial locked-in force effects from TDLF detailing (White et al., 2015). Smaller net reduced load factors may be applied to the unfactored cross-frame or diaphragm forces and flange lateral bending stresses at the discretion of the Owner. In straight skewed bridges detailed for TDLF, the Engineer should also check the cross-frame or diaphragm forces and the flange lateral bending stresses for the fit-up force effects during the steel erection. These effects may be estimated as the negative of the corresponding unfactored

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concrete dead load force effects, which should then be multiplied by γ_p (White et al., 2015).

White et al. (2015) recommend that the specified load factor, γ_p, should be applied directly to the *DC* cross-frame or diaphragm forces and flange lateral bending stresses for straight skewed bridges detailed for SDLF. Significant cross-frame or diaphragm force and flange lateral bending stress reductions are achievable in straight skewed bridges detailed for SDLF; however, in the most extreme cases studied by White et al. (2015), incidental and elastic deformation effects in the structural system lead to negligible corresponding locked-in force effects for SLDF. It should be emphasized that the best estimate of the internal force reductions, when either SDLF or TDLF is employed, is by calculation of the locked-in force effects directly within the structural analysis.

In curved I-girder bridges, the locked-in force effects tend to be additive with the corresponding dead load effects. These additive locked-in force effects tend to be particularly significant for bridges with a maximum L/R greater than or equal to 0.2 that are detailed for SDLF (White et al., 2015). Detailing curved I-girder bridges with or without skew and with a maximum L/R greater than or equal to 0.2 for NLF avoids the introduction of these additional locked-in force effects. Should it be desired to detail such bridges for SDLF, NSBA (2016b) provides an approximate approach for estimating the additional locked-in force effects when these effects are not determined as part of a refined analysis. For bridges with smaller L/R that are detailed for an SDLF, the horizontal curvature effects are smaller, and hence the additive locked-in force effects are smaller and may be neglected.

The additional locked-in force effects are more significant for TDLF detailing; however, TDLF detailing is strongly discouraged for horizontally-curved bridges with or without skew and with a maximum L/R greater than 0.03 (NSBA, 2016a; NSBA, 2016b). Since the locked-in force effects from SDLF and TDLF detailing associated with horizontal curvature tend to be additive with the corresponding dead load force effects, the optional reduction of the total dead load cross-frame or diaphragm forces and flange lateral bending stresses discussed previously is not applicable to horizontally-curved or curved and skewed bridges.

Various factors can cause the girders to deviate from the ideal plumb geometry under the targeted dead load condition, particularly when TDLF detailing is specified. These factors include connection tolerances, fabrication tolerances, accuracy of the structural analysis models, accuracy of the dead load deflection estimates, stiffness of the deck forming (which is typically neglected in analysis calculations), and sequential casting and early stiffness gains of the deck concrete. Except in unusual cases involving substantial global displacement amplification of a slender I-girder bridge unit in its noncomposite condition during the deck placement due to stability effects, such as discussed in Article 6.10.3.4.2, deviation from the ideal plumb condition due to the deflection of the structure generally has a negligible influence on the structural resistance (White et al., 2012). However, substantial deviation from the targeted geometry is an indication that the dead load internal forces and internal stresses in the structure may differ significantly from their calculated values. AASHTO/NSBA (2014b) suggests a tolerance on the deviation from the theoretical erected web position.

Shop assembly of the entire bridge or any significant portion of the bridge is not customary and is typically not needed, except possibly for highly complex framing detailed for NLF. Such a requirement adds unnecessary cost to projects that utilize less complex and more conventional framing. Full shop assembly cannot be done if the bridge has been detailed for TDLF.

Tub girders with properly designed top flange lateral bracing effectively behave as closed sections, and as such, they are torsionally stiff. Straight or slightly curved tub girders with top flange lateral bracing, but without external intermediate cross-frames or diaphragms, generally exhibit little twist under non-composite loading. Tub girders with longer spans and more significant curvature are potentially subject to more significant twisting, but this is often controlled and minimized by providing external intermediate cross-frames or diaphragms. Tub girders are typically designed and detailed to be oriented normal to the cross-slope of the roadway and their webs are detailed to be of equal depth (AASHTO/NSBA, 2006). Thus, the concepts of NLF, SDLF, and TDLF do not directly apply. Also, since tub girders are inherently torsionally stiff, it is difficult to twist them in the field to achieve fit-up of external cross-frames or diaphragms. As a result, tub girder external cross-frames or diaphragms are typically detailed and fabricated to fit under no-load or a specific intermediate steel dead load condition depending on the intended erection sequence. In addition, depending on the magnitude of their twist deformations, tub girders may need to be detailed and fabricated with a built-in reverse twist so that when they twist under dead load, they deflect to a position normal to the roadway cross-slope. The camber of the two webs in skewed and/or curved tub-girder bridges can be significantly different.

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6.7.3—Minimum Thickness of Steel

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

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

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

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For orthotropic decks, research and development and general design improvements domestically and abroad have demonstrated that a minimum deck plate thickness of 0.625 in. has addressed the causes of many problems resulting from overly flexible decks. Although analysis may indicate that deck plates less than 0.625 in. thick could be satisfactory, experience shows that a minimum thickness of 0.625 in. is advisable both from construction and long-term performance points of view.

6.7.4—Diaphragms and Cross-Frames

6.7.4.1—General

Diaphragms or cross-frames may be placed at the end of the structure, across interior supports, and intermittently along the span. The need for diaphragms or cross frames shall be investigated for all stages of assumed construction procedures and the final condition.

This investigation should include, but not be limited to, the following:

- transfer of lateral wind loads from the bottom of the girder to the deck and from the deck to the bearings;
- provision of lateral support to the fascia girders between cross-frame or diaphragm locations to control torsional stresses and rotations due to loads applied to the overhangs, particularly during concrete deck placement;
- stability of the bottom flange for all loads when it is in compression;
- stability of the top flange in compression prior to curing of the deck;
- consideration of any flange lateral bending effects; and
- distribution of vertical dead and live loads applied to the structure.

Metal stay-in-place deck forms should not be assumed to provide adequate stability to the top flange in compression prior to curing of the deck.

If permanent cross-frames or diaphragms are included in the structural model used to determine force effects, they shall be designed for all applicable limit states for the calculated force effects. At a minimum, diaphragms and cross-frames shall be designed to transfer wind loads according to the provisions of Article 4.6.2.7

C6.7.4.1

The arbitrary requirement for diaphragms spaced at not more than 25.0 ft in the AASHTO *Standard Specifications* (2002) has been replaced by a requirement for rational analysis that will often result in the elimination of fatigue-prone attachment details.

Bracing of horizontally-curved members is more critical than for straight members. Diaphragm and cross-frame members resist forces that are critical to the proper functioning of curved-girder bridges. Since they transmit the forces necessary to provide equilibrium, they are considered primary members. Therefore, force effects in the bracing members must be computed and considered in the design of these members and their connections when one or more of the conditions specified in Article and shall meet all applicable slenderness requirements in Article 6.8.4 or Article 6.9.3. Force effects in diaphragm and cross-frame members in horizontally-curved bridges, exceeding one or more of the conditions specified in Article 4.6.1.2.4 for neglecting the effects of curvature, shall be computed and considered in the design of the members and their connections for all applicable limit states.

Connection plates for diaphragms and cross-frames shall satisfy the requirements specified in Article 6.6.1.3.1. Where the diaphragm flanges or cross-frame chords are not attached directly to the girder flanges, provisions shall be made to transfer the calculated horizontal force in diaphragms or cross-frames to the flanges through connection plates, except in cases where less than full-depth end angles or connection plates are used for connecting intermediate diaphragms as permitted in Article 6.6.1.3.1.

At the end of the bridge and intermediate points where the continuity of the slab is broken, the edges of the slab shall be supported by diaphragms or other suitable means as specified in Article 9.4.4.

4.6.1.2.4 for neglecting the effects of curvature in the analysis is exceeded.

If the diaphragm flanges or cross-frame chords are not attached directly to the girder flanges, forces from these elements are transferred through the connection plates. The eccentricity between the diaphragm flanges or cross-frame chords and the girder flanges should be recognized in the design of the connection plates and their connection to the web and flange.

The term connection plate as used herein refers to a transverse stiffener attached to the girder to which a cross-frame or diaphragm is connected.

6.7.4.2—I-Section Members

Diaphragms or cross-frames for rolled beams and plate girders should be as deep as practicable, but at a minimum should be at least 0.5 of the beam depth for rolled beams and 0.75 of the girder depth for plate girders. Cross-frames in horizontally-curved bridges should contain diagonals and top and bottom chords.

End diaphragms shall be designed for forces and distortion transmitted by the deck and deck joint. End moments in diaphragms shall be considered in the design of the connection between the longitudinal component and the diaphragm. Diaphragms with span-to-depth ratios greater than or equal to 4.0 may be designed as beams.

Where supports are not skewed, intermediate diaphragms or cross-frames should be placed in contiguous lines normal to the girders.

Where support lines are not skewed more than 20 degrees from normal, intermediate diaphragms or cross-frames may be placed in contiguous skewed lines parallel to the skewed support lines.

Where support lines are skewed more than 20 degrees from normal, intermediate diaphragms or cross-frames shall be normal to the girders and may be placed in contiguous or discontinuous lines.

Where a support line at an interior pier is skewed more than 20 degrees from normal, elimination of the diaphragms or cross-frames along the skewed interior support line may be considered at the discretion of the Owner. Where discontinuous intermediate diaphragm or cross-frame lines are employed normal to the girders in the vicinity of that support line, a skewed or normal

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For the purpose of this Article, as it applies to horizontally-curved girders, the term "normal" shall be taken to mean normal to a local tangent.

Intermediate diaphragms or cross-frames should be provided at nearly uniform spacing in most cases, for efficiency of the structural design, for constructibility, and/or to allow the use of simplified methods of analysis for calculation of flange lateral bending stresses, such as those discussed in Articles C4.6.1.2.4b, C4.6.2.7.1, and C6.10.3.4.1. Closer spacings may be necessary adjacent to interior piers, in the vicinity of skewed supports, and in some cases, near midspan.

Diaphragms with span-to-depth ratios less than 4.0 act as deep beams and should be evaluated by considering principal stresses rather than by beam theory.

Allowance of skewed intermediate diaphragms or cross-frames where support lines, are not skewed more than 20 degrees from normal is consistent with past practice. Where support lines are skewed more than 20 degrees from normal, it may be advantageous to place the intermediate diaphragms or cross-frames oriented normal to the girders in discontinuous lines, to selectively omit certain diaphragms or cross-frames, and/or to stagger the diaphragms or cross-frames in adjacent bays between the girders, in such a manner that the transverse stiffness of the bridge is reduced, particularly in the vicinity of the supports. Omission of highly-stressed diaphragms or cross-frames—particularly in the vicinity of the obtuse corners of a span—interrupts and reduces the stiffness of the corresponding transverse load path by forcing load transfer via girder flange lateral bending. This is often

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diaphragm or cross-frame should be matched with each bearing that resists lateral force.

If the end diaphragm or cross-frame is skewed, the effect of the tangential component of force transmitted by the skewed unit on the girder shall be considered.

Diaphragms or cross-frames at supports shall be proportioned to transmit all lateral components of force from the superstructure to the bearings that provide lateral restraint.

beneficial as long as the unbraced lengths between the diaphragm or cross-frame locations satisfy the flange resistance requirements given in these specifications. The above practices tend to decrease the diaphragm or cross-frame forces and increase the girder flange lateral bending. In certain cases involving excessively stiff transverse load paths, the diaphragm or cross-frame forces may be decreased to the extent that the associated flange lateral bending stresses are also reduced. The resulting flange lateral bending moments may differ from those estimated using Eq. C4.6.1.2.4b-1, or equivalent; therefore, a special investigation of flange lateral bending moments and diaphragm or cross-frame forces is recommended. Where the flange sizes are increased due to the additional flange lateral bending, this increase often is not significant. In fact, the increased cost resulting from the larger flange sizes is often offset by the reduced cost of providing fewer and smaller diaphragms or cross-frames and smaller diaphragm or cross-frame connections.

Where support lines are skewed more than 20 degrees from normal and cross frames or diaphragms are provided along the skewed support line, the first intermediate cross-frames or diaphragms placed normal to the girders adjacent to the skewed support ideally should be offset, where practicable, by a minimum of the larger of $4b_f$ or $0.4L_b$ from the support, where b_f is the largest girder flange width within the unbraced lengths on either side of the first cross-frame or diaphragm, and L_b is the unbraced length between the first and the second intermediate cross-frame or diaphragm from the support along the girder under consideration (White et al., 2015). This practice helps to alleviate the introduction of a stiff load path that will attract and transfer large transverse forces to the skewed support, particularly at the obtuse corners of a skewed span. In some cases, the limit of $0.4L_b$ may be difficult to achieve, in which case the offset should be made as large as practicable but not less than $4b_f$. At the acute corners of severely skewed bridge spans, the above requirements may result in an excessive unbraced length on the fascia girder. In this case, a cross-frame with top and bottom chords but without diagonal members can be framed from the first interior girder to the fascia girder at a small offset from the support, perpendicular to the girders, to avoid inducing a large transverse stiffness while also providing adequate lateral support to the fascia girder.

Where practicable, the smallest unbraced lengths between intermediate diaphragm or cross-frame locations within the bridge spans should not be less than $4b_f$ or $0.4L_b$, where b_f is defined in the above paragraph and L_b is the smallest unbraced length adjacent to the unbraced length under consideration. The use of unbraced lengths smaller than $4b_f$ or $0.4L_b$ tends to result in the associated cross-frames working more like a contiguous cross-frame line rather than a discontinuous one. Similar to the selection of the offsets from the skewed supports, the limit of $0.4L_b$

may be difficult to achieve in certain cases. In these situations, the value 4bf is the recommended lower limit for the smallest unbraced lengths, offsets, or stagger distances.

White et al. (2015) recommend framing of the diaphragms or cross-frames within straight skewed spans using arrangements such as that shown in Figure C6.7.4.2-1 to both reduce the number of diaphragms or cross-frames required within the bridge as well as to reduce the overall transverse stiffness effects. In Figure C6.7.4.2-1, the diaphragms or cross-frames adjacent to the bearing lines are all placed at the same offset distance relative to the skewed bearing lines, satisfying the above offset recommendations. The other intermediate diaphragms or cross-frames are placed at a constant spacing along the span length to satisfy the flange resistance requirements given in these specifications. In addition, every other diaphragm or cross-frame is intentionally omitted within the bays between the interior girders of the bridge plan. This relaxes the large transverse stiffness that would otherwise be developed in the short diagonal direction between the obtuse corners of the span. This concept and other beneficial framing concepts are discussed further in NSBA (2016b).

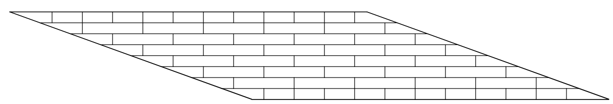

Figure C6.7.4.2-1—Beneficial Staggered Diaphragm or Cross-frame Framing Arrangement for a Straight Bridge with Parallel Skew

At skewed interior piers in continuous-span bridges, NHI (2011) and White et al. (2015) found that transverse stiffness effects are alleviated most effectively by placing diaphragms or cross-frames along the skewed bearing line and locating normal intermediate diaphragms or cross-frames at greater than or equal to the minimum offset from the bearing lines discussed above. Framing of a normal intermediate cross-frame into or near a bearing location along a skewed support line is strongly discouraged unless the cross-frame diagonals are omitted (NSBA, 2016b).

For curved and skewed spans, omitting diaphragms or cross-frames in the vicinity of skewed bearing lines can help to alleviate uplift at critical bearing locations; however, this is typically at the expense of larger diaphragm or cross-frame forces and larger bridge deflections compared to the use of contiguous intermediate diaphragm or cross-frame lines with the recommended offset provided at the skewed bearing lines. Contiguous diaphragm or cross-frame lines are necessary within the span of curved I-girder bridges to develop the width of the bridge structural system for resistance of the overall torsional effects. As such, the use of discontinuous diaphragm or cross-frame lines near a skewed bearing line in these bridge types involves competing considerations. Diaphragms or cross-frames can be omitted to alleviate uplift considerations at certain

The spacing, L_b, of intermediate diaphragms or cross-frames in horizontally-curved I-girder bridges shall not exceed the following in the erected condition:

$$L_b \leq L_r \leq R/10 \tag{6.7.4.2-1}$$

where:

L_r = limiting unbraced length determined from Eq. 6.10.8.2.3-5 (ft)
R = minimum girder radius within the panel (ft)

In no case shall L_b exceed 30.0 ft.

bearings, and potentially to relieve excessive diaphragm or cross-frame forces due to transverse stiffness effects in certain cases—for instance, if the horizontal curvature is relatively small and the skew is significant. However, omission of too many diaphragms or cross-frames may result in a larger than desired increase in the diaphragm or cross-frame forces and bridge system deflections due to the horizontal curvature effects when the bridge is significantly curved.

For skewed diaphragms or cross-frames, connection plates should be oriented in the plane of the transverse bracing. The connection plate must be able to transfer force between the girder and the bracing without undue distortion. Welding of skewed connection plates to the girder may be problematic where the plate forms an acute angle with the girder.

The spacing of intermediate diaphragms and cross-frames in horizontally-curved I-girder bridges in the erected condition is limited to $R/10$, which is consistent with past practice. The spacing is also limited to L_r from Eq. 6.10.8.2.3-5, where L_r is a limiting unbraced length to achieve the onset of nominal yielding in either flange under uniform bending with consideration of compression flange residual stress effects prior to lateral–torsional buckling of the compression flange. Limiting the unbraced length to L_r theoretically precludes elastic lateral–torsional buckling of the compression flange. At unbraced lengths beyond L_r, significant flange lateral bending is likely to occur and the amplification factor for flange lateral bending specified in Article 6.10.1.6 will tend to become large even when an effective length factor for lateral–torsional buckling and/or a moment gradient factor, C_b, is considered.

Eq. C6.7.4.2-1 may be used as a guide for preliminary framing in horizontally-curved I-girder bridges:

$$L_b = \sqrt{\frac{5}{3} r_\sigma R b_f} \tag{C6.7.4.2-1}$$

where:

b_f = flange width (ft)
L_b = diaphragm or cross-frame spacing (ft)
r_σ = desired bending stress ratio equal to $\left| f_\ell / f_{bu} \right|$
R = girder radius (ft)

A maximum value of 0.3 may be used for the bending stress ratio, r_σ. Eq. C6.7.4.2-1 was derived from the V-load concept (Richardson, Gordon and Associates, 1976) and has been shown to yield a good correlation with three-dimensional finite-element analysis results if the cross-frame spacing is relatively uniform (Davidson et al., 1996).

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6.7.4.3—Composite Box-Section Members

Diaphragms shall be provided within composite box-section members at each support to resist cross-section distortion of the box, and shall be designed to resist torsional moments in the box and transmit vertical and lateral forces from the box to the bearings.

For cross sections consisting of two or more boxes, external cross-frames or diaphragms shall be used between the boxes at end supports. External cross-frames or diaphragms shall be provided between girder lines at interior supports, unless analysis indicates that the boxes are torsionally stable without these members, particularly during erection. Internal cross-frames or diaphragms shall be provided at locations of external cross-frames or diaphragms.

Internal diaphragms provided for continuity or to resist torsional forces generated by structural members shall be connected to the webs and flanges of the box section. An access hole at least 18.0 in. wide and 24.0 in. high should be provided within each internal intermediate diaphragm. Design of the diaphragm shall consider the effect of the access hole on the stresses. Reinforcement around the hole may be required.

Intermediate internal diaphragms or cross-frames shall be provided. For all single box sections, horizontally-curved sections, and multiple box sections in cross sections of bridges not satisfying the requirements of Article 6.11.2.3 or with box flanges that are not fully effective according to the provisions of Article 6.11.1.1, the spacing of the internal diaphragms or cross-frames shall not exceed 40.0 ft.

Webs of internal and external diaphragms shall satisfy Eq. 6.10.1.10.2-1. The nominal shear resistance of internal and external diaphragm webs shall be determined from Eq. 6.10.9.3.3-1.

C6.7.4.3

External diaphragms with aspect ratios, or ratios of length to depth, less than 4.0 and internal diaphragms act as deep beams and should be evaluated by considering principal stresses rather than by simple beam theory. Fatigue-sensitive details on these diaphragms and at the connection of the diaphragms to the flanges should be investigated considering the principal tensile stresses.

Boxes may undergo excessive rotation in some cases when the concrete deck is placed if intermediate diaphragms or cross-frames are not provided between boxes. If analysis shows that such rotations are anticipated, temporary cross-frames may be employed. Removal of such temporary members may lead to failure of remaining bolts, creating a safety concern. The effect of the release of bracing forces on the bridge can be investigated by considering the effect of reversal of member loads. Removal of temporary cross-frames having large forces may cause increased deck stresses.

Until the deck on a tub section hardens, internal cross-frames or diaphragms and lateral top flange bracing are required to stabilize the tub section. For straight boxes without skew satisfying the requirements of Article 6.11.2.3 and with fully effective box flanges, transverse bending stresses and longitudinal warping stresses due to cross-section distortion have often been shown to be small (Johnston and Mattock, 1967) and may be neglected. Torsion may be significant, however, if the deck weight acting on the box is unsymmetrical. A reduction in the number of permanent internal cross-frames or diaphragms and/or top lateral bracing members in such boxes is permitted when checked by proper analysis. Internal cross-frames or diaphragms should be placed at or near points of maximum moment and near both sides of field splices. The Engineer should also consider the need for additional temporary or permanent internal cross-frames or diaphragms, which may be required for transportation, construction, and at the lifting points of each shipping piece.

Cross-sectional distortion stresses are typically controlled by the internal cross-frames or diaphragms, with the spacing of these members not to exceed 40.0 ft for the cases specified herein. For the specific cases listed in Article 6.11.1.1, transverse bending stresses due to cross-section distortion are explicitly limited to 20.0 ksi at the strength limit state. Adequate internal cross-frames or diaphragms must be introduced to meet this limit, and should also be designed to control the longitudinal warping stresses due to the critical factored torsional loads. Such stresses should not exceed approximately ten percent of the longitudinal stresses due to major-axis bending at the strength limit state.

In cases with widely spaced internal cross-frames or diaphragms, additional struts between the top flanges of tub sections may be necessary in order to satisfy the constructibility provisions of Article 6.11.3.2. As indicated in Article C6.11.3.2, struts that are part of top lateral

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bracing systems attached to the flanges at points where internal cross-frames or diaphragms do not exist may be considered to act as brace points at the discretion of the Engineer.

Where distortion of the section is adequately controlled by the internal cross-frames or diaphragms, acting in conjunction with a top lateral bracing system in the case of tub sections, the St. Venant torsional inertia, J, for a box section may be determined as:

$$J = 4\frac{A_o^2}{\sum \frac{b}{t}} \tag{C6.7.4.3-1}$$

where:

A_o = area enclosed by the box section (in.^{2})
b = width of rectangular plate element (in.)
t = thickness of plate (in.)

In tub sections with inclined webs with a slope exceeding 1 to 4 and/or where the unbraced length of the top flanges exceeds 30.0 ft, additional intermediate internal cross frames, diaphragms, or struts may be required to increase the resistance of discretely braced top flanges of tub sections to lateral bending resulting from a uniformly distributed transverse load acting on the flanges. This lateral load results from the change in the horizontal component of the web dead load shear plus the change in the St. Venant torsional dead load shear per unit length along the member, and is discussed further in Article C6.11.3.2.

Because of the critical nature of internal and external diaphragms, particularly at supports, any reliance on post-buckling resistance is not advisable. Satisfaction of Eq. 6.10.1.10.2-1 ensures that theoretical bend buckling of internal and external diaphragm webs will not occur for elastic stress levels at or below the yield stress.

Limiting the nominal shear resistance of diaphragm webs to the shear buckling or shear yield resistance according to Eq. 6.10.9.3.3-1 prevents any reliance on post-buckling shear resistance. Bearing stiffeners on internal diaphragms act as transverse stiffeners in computing the nominal shear resistance.

A portion of the box flange width equal to six times its thickness may be considered effective with an internal diaphragm.

The attachment of internal cross-frame connection plates to box flanges is discussed further in Article C6.6.1.3.1.

6.7.4.4—Noncomposite Box-Section Members

6.7.4.4.1—General

Diaphragms, where provided, should be connected to the webs and flanges of all noncomposite box-section

C6.7.4.4.1

For welded and nonwelded built-up noncomposite box-section members subject to torsion, cross-sectional

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members, where practical. The diaphragms shall be designed to resist cross-section distortion of the box and shall be designed to resist torsional moments in the box applied to or resisted at the diaphragm location, and to transmit vertical and lateral forces from the box to the bearings, as applicable.

distortion stresses are controlled by internal diaphragms or cross-frames. As specified in Article 6.12.2.2.2a, factored transverse plate bending stresses due to cross-section distortion should be limited to 20.0 ksi at the strength limit state in noncomposite box-section members subject to large torques.

6.7.4.4.2—Square and Rectangular HSS Members

For all square and rectangular HSS members, placement of diaphragms at the member ends should be considered.

C6.7.4.4.2

Holes may be provided in diaphragms of HSS members to accommodate functions such as drainage, hot dip zinc coating, utilities, and access to connections.

6.7.4.4.3—Welded and Nonwelded Built-Up Noncomposite Box-Section Members

Diaphragms shall be provided within welded and nonwelded built-up noncomposite box-section members at each support and at the ends of the member, unless the ends of the member are connected to other members that serve to retain the shape of the box cross section. The placement of additional diaphragms at locations of any externally applied concentrated loads shall be considered.

An access hole at least 18.0 in. wide and 24.0 in. high, where practical, should be provided within each internal intermediate diaphragm. Design of the diaphragm shall consider the effect of the access hole on the stresses. Reinforcement around the hole may be required.

Where practical, cross-frames may be used in lieu of diaphragms at locations other than at supports. Connection plates for internal cross-frames shall satisfy the provisions of Article 6.6.1.3.1, as applicable.

In members subject to torsion in which:

$$f_{ve} > 0.2\phi_T F_{cv} \quad (6.7.4.4.3\text{-}1)$$

where:

ϕ_T = resistance factor for torsion specified in Article 6.5.4.2

f_{ve} = factored shear stress due to torsion in the cross-section element under consideration (ksi)

F_{cv} = nominal shear resistance of the cross-section element under consideration, under shear alone, calculated as specified in Article 6.9.2.2.2 (ksi)

the spacing of internal intermediate diaphragms or cross-frames within the member should not exceed 40.0 ft. Internal intermediate diaphragms should be spaced a minimum of 2.0 ft apart.

C6.7.4.4.3

Diaphragms acting as flexural members over supports in larger noncomposite box-section members, or when connecting multiple noncomposite box-section members, should also satisfy the applicable requirements of Article 6.7.4.3.

Where practical, access holes in internal intermediate diaphragms should be wide and high enough to allow for convenient maintenance and inspection access. The holes should be placed in the diaphragms in a position that will allow convenient access when passing through the box. Any stiffeners on the diaphragms should be placed back from the edges of the openings.

White et al. (2019b) discuss special moment of inertia requirements for top or bottom struts of internal intermediate cross-frames that serve as transverse stiffeners to enhance the compressive resistance of a longitudinally stiffened plate element.

The factored torsional shear stresses, f_{ve}, may be computed using the appropriate equation given in Article C6.9.2.2.2 in lieu of a refined analysis.

The recommended minimum spacing of internal intermediate diaphragms is to prevent an inspector from having to straddle two diaphragms at one time.

6.7.4.5—Trusses and Arches

Diaphragms shall be provided at the connections to floorbeams and at other connections or points of application of concentrated loads in truss and arch members. Internal diaphragms may also be provided to maintain member alignment.

Gusset plates engaging a pedestal pin at the end of a truss shall be connected by a diaphragm. The webs of the pedestal should be connected by a diaphragm wherever practical.

If the end of the web plate or cover plate is 4.0 ft or more from the point of intersection of the members, a diaphragm shall be provided between gusset plates engaging main members.

6.7.5—Lateral Bracing

6.7.5.1—General

The need for lateral bracing shall be investigated for all stages of assumed construction procedures and the final condition.

Where required, lateral bracing should be placed either in or near the plane of a flange or chord being braced. Investigation of the requirement for lateral bracing shall include, but not be limited to:

- transfer of lateral wind loads to the bearings as specified in Article 4.6.2.7,
- transfer of lateral loads as specified in Article 4.6.2.8, and
- control of deformations and cross-section geometry during fabrication, erection, and placement of the deck.

Permanent bottom flange lateral bracing members shall be considered to be primary members. Lateral bracing members not required for the final condition should not be considered to be primary members, and may be removed at the Owner's discretion.

If permanent lateral bracing members are included in the structural model used to determine live load force effects, the force effects in the members shall be computed and considered in the design of the members and their connections for all applicable limit states. The provisions of Articles 6.8.4 and 6.9.3 shall apply.

Connection plates for lateral bracing shall satisfy the requirements specified in Article 6.6.1.3.2.

When lateral bracing is designed for seismic loading, the provisions of Article 4.6.2.8 shall apply.

C6.7.5.1

In I-girder bridges, bottom flange lateral bracing creates a pseudo-closed section formed by the I-girders connected with the bracing and the hardened deck, and therefore becomes load carrying. Cross-frame forces increase with the addition of bottom flange bracing because the cross-frames act to retain the shape of the pseudo-box section. In addition, moments in the braced girders become more equalized and the bracing members are also subject to significant live load forces.

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6.7.5.2—I-Section Members

Continuously braced flanges should not require lateral bracing.

The need for lateral bracing adjacent to supports of I-girder bridges to provide rigidity during construction should be considered.

C6.7.5.2

Wind-load stresses in I-sections may be reduced by:

- changing the flange size,
- reducing the diaphragm or cross-frame spacing, or
- adding lateral bracing.

The relative economy of these methods should be investigated.

To help prevent significant relative horizontal movement of the girders in spans greater than 200 ft during construction, it may be desirable to consider providing either temporary or permanent lateral bracing in one or more panels adjacent to the supports of I-girder bridges. For continuous-span bridges, such bracing would only be necessary adjacent to interior supports and should be considered at the free ends of continuous units. Such a system of lateral bracing can also provide a stiffer load path for wind loads acting on the noncomposite structure during construction to help reduce the lateral deflections and flange lateral bending stresses. Top lateral bracing is preferred. Bottom lateral bracing can provide a similar function, but unlike top bracing, would be subject to significant live load forces in the finished structure that would have to be considered.

For horizontally-curved bridges, when the curvature is sharp and temporary supports are not practical, it may be desirable to consider providing both top and bottom lateral bracing to ensure pseudo-box action while the bridge is under construction. Top and bottom lateral bracing provides stability to a pair of I-girders.

If temporary lateral bracing is used, the analysis method used must be able to recognize influence of the lateral bracing.

6.7.5.3—Tub Section Members

Top lateral bracing shall be provided between common flanges of individual tub sections. For straight girders, the need for a full-length lateral bracing system shall be investigated to ensure that deformations of the tub section are adequately controlled and that stability of the tub section members is provided during erection and placement of the concrete deck. During deck casting, the stability of the compression flanges between panel points of the lateral bracing system shall be investigated. If a full-length lateral bracing system is not provided, the local stability of the top flanges and global stability of the individual tub sections shall be investigated for the Engineer's assumed construction sequence. For horizontally-curved girders, a full-length lateral bracing system shall be provided and the stability of compression flanges between panel points of the lateral bracing system shall be investigated during deck casting.

Top lateral bracing shall be designed to resist shear flow in the pseudo-box section due to the factored loads before the concrete deck has hardened or is made

C6.7.5.3

Investigation will generally show that a lateral bracing system is not required between multiple tub sections.

The shear center of an open tub section is located below the bottom flange (Heins, 1975). The addition of top lateral bracing raises the shear center closer to the center of the resulting pseudo-box section, significantly improving the torsional stiffness.

In addition to resisting the shear flow before the concrete deck has hardened or is made composite, top lateral bracing members are also subject to significant forces due to flexure of the noncomposite tub. In the absence of a more refined analysis, Fan and Helwig (1999) provide an approach for estimating these forces.

Top lateral bracing members are also subject to forces due to wind loads acting on the noncomposite pseudo-box section during construction.

For straight tub sections with spans less than about 150 ft, as a minimum, at least one panel of horizontal composite. Forces in the bracing due to flexure of the tub shall also be considered during construction based on the Engineer's assumed construction sequence.

If the bracing is attached to the webs, the cross-sectional area of the tub for shear flow shall be reduced to reflect the actual location of the bracing, and a means of transferring the forces from the bracing to the top flange shall be provided.

lateral bracing should be provided within the tub on each side of a lifting point. The need for additional lateral bracing to resist the shear flow resulting from any net torque on the steel section due to unequal factored deck weight loads acting on each side of the top flanges, or any other known eccentric loads acting on the steel section during construction, should be considered. Cross-section distortion and top-flange lateral bending stresses may need to be considered when a tub with a partial-length bracing system is subjected to a net torque. A full-length lateral bracing system should be considered for cases where the torques acting on the steel section are deemed particularly significant, e.g., tub-section members resting on skewed supports and/or tub-section members on which the deck is unsymmetrically placed. If a full-length system is not provided in a straight tub-section member, the Engineer must ensure the local and global stability of the top flanges and the tub-section member, respectively, during the assumed construction sequence. For straight tub sections with spans greater than about 150 ft, a full-length lateral bracing system should be provided within the tub.

For both straight and horizontally-curved tub sections, a full-length lateral bracing system forms a pseudo-box to help limit distortions brought about by temperature changes occurring prior to concrete deck placement, and to resist the torsion and twist caused by any eccentric loads acting on the steel section during construction. AASHTO (1993) specified that diagonal members of the top lateral bracing for tub sections satisfy the following criterion:

$$A_d \geq 0.03w \qquad \text{(C6.7.5.3-1)}$$

where:

A_d = minimum required cross-sectional area of one diagonal (in.^{2})
w = center-to-center distance between the top flanges (in.)

Satisfaction of this criterion was intended to ensure that the top lateral bracing would be sized so that the tub would act as a pseudo-box section with minimal warping torsional displacement and normal stresses due to warping torsion less than or equal to ten percent of the major-axis bending stresses. This criterion was developed assuming tub sections with vertical webs and ratios of section width-to-depth between 0.5 and 2.0, and an X-type top lateral bracing system with the diagonals placed at an angle of 45 degrees relative to the longitudinal centerline of the tub-girder flanges (Heins, 1978). Although this criterion may not necessarily be directly applicable to other bracing configurations and cross-section geometries, it is recommended that Eq. C6.7.5.3-1 still be used as a guideline to ensure that a reasonable minimum area is provided for the diagonal bracing members.

Single-diagonal top lateral bracing systems are preferred over X-type systems because there are fewer pieces to fabricate and erect and fewer connections. However, forces in alternating Warren-type single-diagonal top lateral bracing members, as shown in Figure C6.7.5.3-1, due to flexure of the tub section can sometimes result in the development of significant lateral bending stresses in the top flanges. In lieu of a refined analysis, Fan and Helwig (1999) provide an approach for estimating the top-flange lateral bending stresses due to these forces. If necessary, the flange lateral bending stresses and forces in the bracing members in this case can often be effectively mitigated by the judicious placement of parallel single-diagonal members, or a Pratt-type configuration, in each bay in lieu of a Warren-type configuration as shown in Figure C6.7.5.3-2. In this configuration, the members should be oriented based on the sign of the torque so that the forces induced in these members due to torsion offset the compressive or tensile forces induced in the same members due to flexure of the tub section. The forces in the lateral bracing system are very sensitive to the casting sequence. If the member sizes have been optimized based upon an assumed casting sequence, it is imperative that the assumed casting sequence be shown in the contract documents. Field tests have shown that forces in the top lateral system after the deck has been cast are negligible.

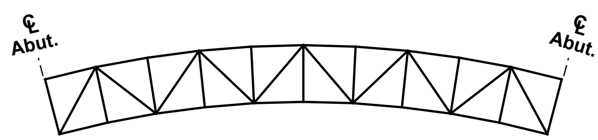

Figure C6.7.5.3-1—Warren-type Single-diagonal Top Lateral Bracing System for Tub Section Member: Plan View

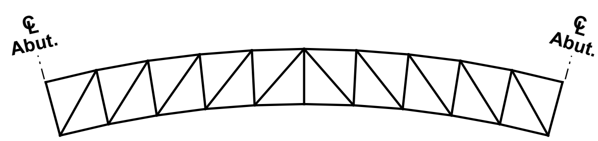

Figure C6.7.5.3-2—Pratt-type Single-diagonal Top Lateral Bracing System for Tub Section Member: Plan View

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Where the forces in the bracing members are not available from a refined analysis, the shear flow across the top of the pseudo-box section can be computed from Eq. C6.11.1.1-1 assuming the top lateral bracing acts as an equivalent plate. The resulting shear can then be computed by multiplying the resulting shear flow by the width *w*, and the shear can then be resolved into the diagonal bracing member(s). Should it become necessary for any reason to compute the St. Venant torsional stiffness of the pseudo-box section according to Eq. C6.7.4.3-1, formulas are available (Kollbrunner and Basler, 1966; Dabrowski, 1968) to calculate the thickness of the equivalent plate for different possible configurations of top lateral bracing.

Top lateral bracing should be continuous across field splice locations.

6.7.5.4—Trusses

Through-truss spans and deck truss spans shall have top and bottom lateral bracing. If an x-system of bracing is used, each member may be considered effective simultaneously if the members meet the slenderness requirements for both tension and compression members. The members should be connected at their intersections.

The member providing lateral bracing to compression chords should be as deep as practical and connected to both flanges.

Floorbeam connections should be located so that the lateral bracing system will engage both the floorbeam and the main supporting members. Where the lateral bracing system intersects a joint formed by a floorbeam and a main longitudinal member, the lateral member shall be connected to both members.

6.7.6—Pins

6.7.6.1—Location

Pins should be located so as to minimize the force effects due to eccentricity.

6.7.6.2—Resistance

6.7.6.2.1—Combined Flexure and Shear

Pins subjected to combined flexure and shear shall be proportioned to satisfy:

$$\frac{6.0\,M_u}{\phi_f\,D^3F_y} + \left(\frac{2.2\,V_u}{\phi_v\,D^2F_y}\right)^3 \leq 0.95 \qquad (6.7.6.2.1\text{-}1)$$

where:

D = diameter of pin (in.)
M_u = moment due to the factored loads (kip-in.)

C6.7.6.2.1

The development of Eq. 6.7.6.2.1-1 is discussed in Kulicki (1983).

V_u = shear due to the factored loads (kip)
F_y = specified minimum yield strength of the pin (ksi)
ϕ_f = resistance factor for flexure as specified in Article 6.5.4.2
ϕ_v = resistance factor for shear as specified in Article 6.5.4.2

The moment, M_u, and shear, V_u, should be taken at the same design section along the pin.

6.7.6.2.2—Bearing

The factored bearing resistance on pins shall be taken as:

$$\left(R_{pB}\right)_r = \phi_b \left(R_{pB}\right)_n \qquad (6.7.6.2.2\text{-}1)$$

in which:

$$\left(R_{pB}\right)_n = 1.5tDF_y \qquad (6.7.6.2.2\text{-}2)$$

where:

t = thickness of plate (in.)
D = diameter of pin (in.)
ϕ_b = resistance factor for bearing as specified in Article 6.5.4.2

C6.7.6.2.2

For the design of new pins subjected to significant rotations, such as for rocker bearings or hinges, the coefficient 1.5 in Eq. 6.7.6.2.2-2 may be halved to 0.75 at the discretion of the Engineer. This accounts for increased wear over the life of pins used for applications with significant rotations. An equivalent approach to that suggested above was used for allowable stress design in the AASHTO *Standard Specifications* (2002). For the evaluation of existing pins subjected to significant rotations, the 1.5 coefficient in Eq. 6.7.6.2.2-2 should not be halved.

6.7.6.3—Minimum Size Pin for Eyebars

The diameter of the pin, D, shall satisfy:

$$D \geq \left(\frac{3}{4} + \frac{F_y}{400}\right)b \qquad (6.7.6.3\text{-}1)$$

where:

F_y = specified minimum yield strength of the eyebar (ksi)
b = width of the body of the eyebar (in.)

6.7.6.4—Pins and Pin Nuts

Pins shall be of sufficient length to secure a full bearing of all parts connected upon the turned body of the pin. The pin shall be secured in position by:

- hexagonal recessed nuts,
- hexagonal solid nuts with washers, or
- if the pins are bored through, a pin cap restrained by pin rod assemblies.

Pin or rod nuts shall be malleable castings or steel and shall be secured in position by cotter pins through the threads or by burring the threads. Commercially available

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lock nuts may be used as an alternate to burring the threads or use of cotter pins.

6.7.7—Heat-Curved Rolled Beams and Welded Plate Girders

6.7.7.1—Scope

This Article pertains to rolled beams and constant depth welded I-section plate girders heat-curved to obtain a horizontal curvature. Structural steels conforming to AASHTO M 270M/M 270 (ASTM A709/A709M), Grades 36, 50, 50S, 50W, HPS 50W, HPS 70W or HPS 100W (Grades 250, 345, 345S, 345W, HPS 345W, HPS 485W or HPS 690W) may be heat-curved.

6.7.7.2—Geometric Limitations

The provisions of Article 11.4.12.2.2 of the *AASHTO LRFD Bridge Construction Specifications* regarding cross-sectional limitations and radius limitations on heat curving shall apply. For each field section of a horizontally-curved rolled beam or welded section I-girder, the Engineer shall indicate on the contract documents whether or not heat curving is permitted according to the specified limitations.

6.7.8—Bent Plates

Structural steel plates and bars that are to be cold or hot bent shall satisfy the requirements for bent plates specified in Article 11.4.3.3 of the *AASHTO LRFD Bridge Construction Specifications*.

C6.7.8

Article 11.4.3.3 of the *AASHTO LRFD Bridge Construction Specifications* limits the minimum bend radius for all grades and thicknesses of steel conforming to Structural Steel for Bridges, AASHTO M 270M/M 270 (ASTM A709/A709M) used in fracture-critical or nonfracture-critical applications, and where the bend lines are oriented perpendicular to the direction of final rolling of the plate, to 5.0t where t is the thickness of the plate. The radius is measured to the concave face of the plate. Where the bend lines are oriented parallel to the final rolling direction, the minimum bend radius is increased to 7.5t. Web splice plates, fillers, gusset plates not serving as chord splices, connection plates, and web stiffeners are not included in the rolling direction requirements. For cross-frame or diaphragm connection plates up to 0.75 in., the minimum bending radii may be taken as 1.5t. These limits are to ensure that the bending of the plate has not significantly lowered the toughness and ductility of the plate. Smaller radius bends may be used with the approval of the Engineer.

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6.8—TENSION MEMBERS

6.8.1—General

Members and splices subjected to axial tension shall be investigated for:

- yield on the gross section using Eq. 6.8.2.1-1 and
- fracture on the net section using Eq. 6.8.2.1-2.

Holes larger than those typically considered for connectors such as bolts shall be deducted in determining the gross section area.

The determination of the net section shall require consideration of:

- the gross area from which deductions will be made or reduction factors applied, as appropriate;
- deductions for all holes in the design cross section; correction of the bolt hole deductions for the stagger rule specified in Article 6.8.3;
- application of the reduction factor U specified in Article 6.8.2.2 for members and Article 6.13.5.2 for splice plates and other splicing elements to account for shear lag; and
- application of the 85-percent maximum area efficiency factor for splice plates and other splicing elements specified in Article 6.13.5.2.

Tension members shall satisfy the slenderness requirements specified in Article 6.8.4 and the fatigue requirements of Article 6.6.1. Block shear strength shall be investigated at end connections as specified in Article 6.13.4.

C6.8.1

Holes typically deducted where determining the gross section include pin holes, access holes, and perforations.

6.8.2—Tensile Resistance

6.8.2.1—General

The factored tensile resistance, P_r, shall be taken as the lesser of the values given by Eqs. 6.8.2.1-1 and 6.8.2.1-2.

$$P_r = \phi_y P_{ny} = \phi_y F_y A_g \tag{6.8.2.1-1}$$

$$P_r = \phi_u P_{nu} = \phi_u F_u A_n R_p U \tag{6.8.2.1-2}$$

where:

- P_{ny} = nominal tensile resistance for yielding in gross section (kip)
- F_y = specified minimum yield strength (ksi)
- A_g = gross cross-sectional area of the member (in.^{2})
- F_u = tensile strength (ksi)
- A_n = net area of the member as specified in Article 6.8.3 (in.^{2})
- R_p = reduction factor for holes taken equal to 0.90 for bolt holes punched full size and 1.0 for bolt

C6.8.2.1

The reduction factor, U, does not apply when checking yielding on the gross section because yielding tends to equalize the nonuniform tensile stresses caused over the cross section by shear lag. The reduction factor, R_p, conservatively accounts for the reduced fracture resistance in the vicinity of bolt holes that are punched full size (Brown et al., 2007). No reduction in the net section fracture resistance is required for holes that are drilled full size or subpunched and reamed to size. The reduction in the factored resistance for punched holes was previously accounted for by increasing the hole size for design by 0.625 in., which penalized drilled and subpunched and reamed holes and did not provide a uniform reduction for punched holes since the reduction varied with the hole size.

Due to strain hardening, a ductile steel loaded in axial tension can resist a force greater than the product of its gross area and its yield strength prior to fracture. However, excessive elongation due to uncontrolled holes drilled full size or subpunched and reamed to size

U = reduction factor to account for shear lag; 1.0 for components in which force effects are transmitted to all elements, and as specified in Article 6.8.2.2 for other cases

ϕ_y = resistance factor for yielding of tension members as specified in Article 6.5.4.2

ϕ_u = resistance factor for fracture of tension members as specified in Article 6.5.4.2

yielding of gross area not only marks the limit of usefulness but it can precipitate failure of the structural system of which it is a part. Depending on the ratio of net area to gross area and the mechanical properties of the steel,the component can fracture by failure of the net area at a load smaller than that required to yield the gross area. General yielding of the gross area and fracture of the net area both constitute measures of component strength. The relative values of the resistance factors for yielding and fracture reflect the different reliability indices deemed proper for the two modes.

The part of the component occupied by the net area at fastener holes generally has a negligible length relative to the total length of the member. As a result, the strain hardening is quickly reached and, therefore, yielding of the net area at fastener holes does not constitute a strength limit of practical significance, except perhaps for some builtup members of unusual proportions.

For welded connections, A_n is the gross section less any access holes in the connection region.

6.8.2.2—Reduction Factor, *U*

The shear lag reduction factor, *U,* shall be used when investigating the tension fracture check specified in Article 6.8.1 at the strength limit state.

In the absence of more refined analysis or tests, the reduction factors specified herein may be used to account for shear lag in connections.

The shear lag reduction factor, *U,* may be calculated as specified in Table 6.8.2.2-1. For open cross-section members, the calculated value of *U* should not be taken to be less than the ratio of the gross area of the connected element or elements to the member gross area.

C6.8.2.2

The provisions of Article 6.8.2.2 are adapted from the 2016 AISC Specification Section D3, Effective Net Area for design of tension members. These provisions specify that for open cross-section members, such as W, M, S, C or HP shapes, tees, and single and double angles, the calculated value of *U* should not be taken to be less than the ratio of the gross area of the connected element or elements to the member gross area. The preceding provision does not apply to closed sections, such as HSS, nor to plates.

The effect of the moment due to the eccentricities in the connection in angle members and light structural tee members loaded eccentrically in axial tension may be ignored in the design of the member and the connections (AISC, 2016b); the effect of the connection eccentricity is addressed through the use of the shear lag reduction factor, *U*.

Examples of the distances $\bar{x}$ and L used in the calculation of the reduction factor *U* for all types of tension members, except plates and Hollow Structural Section (HSS) members, are illustrated in Figure C6.8.2.2-1.

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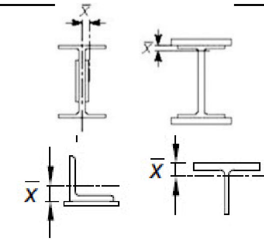
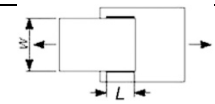
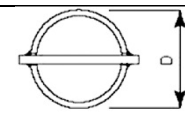
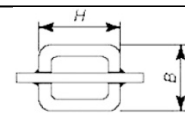
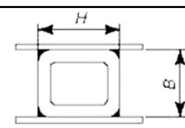
Table 6.8.2.2-1—Shear Lag Factors for Connections to Tension Members

Case	Description of Element		Shear Lag Factor, U	Example
1	All tension members where the tension load is transmitted directly to each of cross-sectional elements by fasteners or welds (except as in Cases 4, 5, and 6).		$U = 1.0$	—
2	All tension members, except plates and HSS, where the tension load is transmitted to some but not all of the cross-sectional elements by fasteners or longitudinal welds. (Alternatively, for W, M, S, and HP shapes, Case 7 may be used. For angles, Case 8 may be used.)		$U = 1 - \frac{\bar{x}}{L}$	
3	All tension members where the tension load is transmitted by transverse welds to some but not all of the cross-sectional elements.		$U = 1.0$ and A_n = area of the directly connected elements	—
4	Plates, angles, channels with welds at heels, tees, and W-shapes with connected elements, where the tension load is transmitted by longitudinal welds only. See Case 2 for definition of $\bar{x}$. L shall not be less than 4 times the weld size.		$U = \frac{3L^2}{3L^2 + w^2}\left(1 - \frac{\bar{x}}{L}\right)$	
5	Round HSS with a single concentric gusset plate through slots in the HSS.		$L \geq 1.3D \ldots U = 1.0$ $D \leq L < 1.3D \ldots U = 1 - \frac{\bar{x}}{L}$ $\bar{x} = \frac{D}{\pi}$	
6	Rectangular HSS	with a single concentric gusset plate	$L \geq H \ldots U = 1 - \frac{\bar{x}}{L}$ $\bar{x} = \frac{B^2 + 2BH}{4(B+H)}$	
		with 2 side gusset plates	$L \geq H \ldots U = 1 - \frac{\bar{x}}{L}$ $\bar{x} = \frac{B^2}{4(B+H)}$	
7	W, M, S, or HP shapes or tees cut from these shapes (If U is calculated per Case 2, the larger value is permitted to be used.)	with flange connected with 3 or more fasteners per line in direction of loading	$b_f \geq \frac{2}{3}d \ldots U = 0.90$ $b_f < \frac{2}{3}d \ldots U = 0.85$	—
		with web connected with 4 or more fasteners per line in direction of loading	$U = 0.70$	—
8	Single and double angles (If U is calculated per Case 2, the larger value is permitted to be used.)	with 4 or more fasteners per line in direction of loading	$U = 0.80$	—
		with 3 fasteners per line in direction of loading (with fewer than 3 fasteners per line in direction of loading, use Case 2)	$U = 0.60$	—

where:

L = length of connection (in.)
w = plate width (in.)
$\bar{x}$ = connection eccentricity (in.)

B = overall width of rectangular HSS member, measured 90 degrees to the plane of the connection (in.)
D = outside diameter of round HSS (in.)
H = overall height of rectangular HSS member, measured in the plane of the connection (in.)
d = full nominal depth of the section; for tees, depth of the section from which the tee was cut (in.)
b_f = flange width (in.)

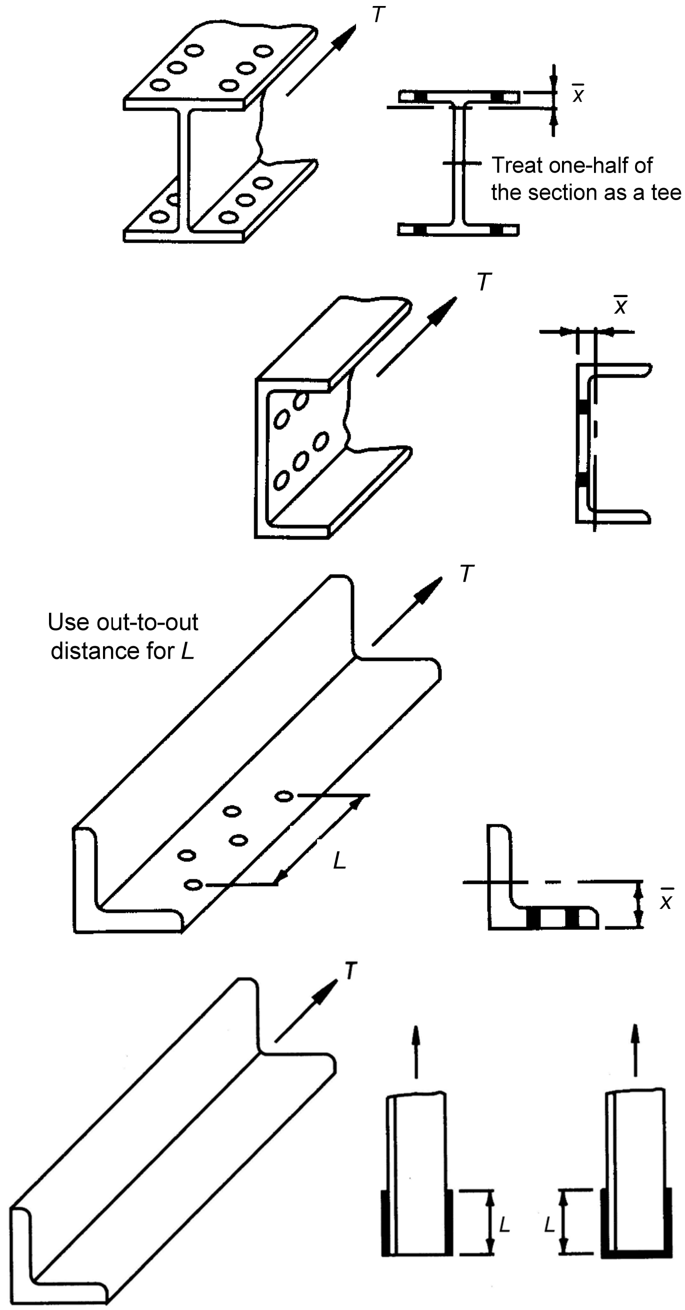


Figure C6.8.2.2-1—Determination of $\bar{x}$ or L in the Calculation of the Shear Lag Reduction Factor, U

For members with combinations of longitudinal and transverse welds, L is the maximum length of the longitudinal welds. The transverse weld does not significantly affect the fracture resistance based on shear lag. The presence of the transverse weld does little to influence the transfer of the load into the unattached elements of the member cross section. The connection length L is defined for general cases as the maximum length of the longitudinal welds or the out-to-out distance between the bolts in the connection parallel to the line of force (in.).

6.8.2.3—Combined Axial Tension, Flexure, and Flexural and/or Torsional Shear

C6.8.2.3

These provisions address the strength interaction for any combination of axial tension, uniaxial or biaxial flexure, and flexural and/or torsional shear, including combinations where one or more of the individual actions may be zero.

6.8.2.3.1—General

The factored moments, M_{ux} and M_{uy}, and factored axial tensile load, P_u, calculated by elastic analysis shall satisfy the relationships specified herein, as applicable, with all ratios taken as positive. In addition, the member tension rupture provisions of Article 6.8.2.3.3 shall be satisfied at welded and/or bolted connections and at cross sections having a net area reduction due to bolt holes. Interaction with torsional and/or flexural shear, as applicable, shall be considered as specified in Article 6.8.2.3.2.

- Except as permitted herein, the following relationships shall be employed for all types of members:
  - If $\frac{P_u}{P_{ry}} < 0.2$, then

$$\frac{P_u}{2P_{ry}} + \left(\frac{M_{ux}}{M_{rx}} + \frac{M_{uy}}{M_{ry}}\right) \leq 1.0 \qquad (6.8.2.3.1\text{-}1)$$

  - If $\frac{P_u}{P_{ry}} \geq 0.2$, then

$$\frac{P_u}{P_{ry}} + \frac{8}{9}\left(\frac{M_{ux}}{M_{rx}} + \frac{M_{uy}}{M_{ry}}\right) \leq 1.0 \qquad (6.8.2.3.1\text{-}2)$$

- For noncomposite I- and box-section members, the following alternative relationships may be employed in combination:

$$\frac{P_u}{P_{ry}} + \left(\frac{M_{ux}}{M_{rxpe}} + \frac{M_{uy}}{M_{rype}}\right) \leq 1.0 \qquad (6.8.2.3.1\text{-}3)$$

C6.8.2.3.1

Eqs. 6.8.2.3.1-1 and 6.8.2.3.1-2 represent the stability and overall strength interaction effects for uniaxial or biaxial bending combined with axial tension and general yielding under axial tension and flexure. Figure C6.8.2.3.1-1 shows the shape of this strength envelope.

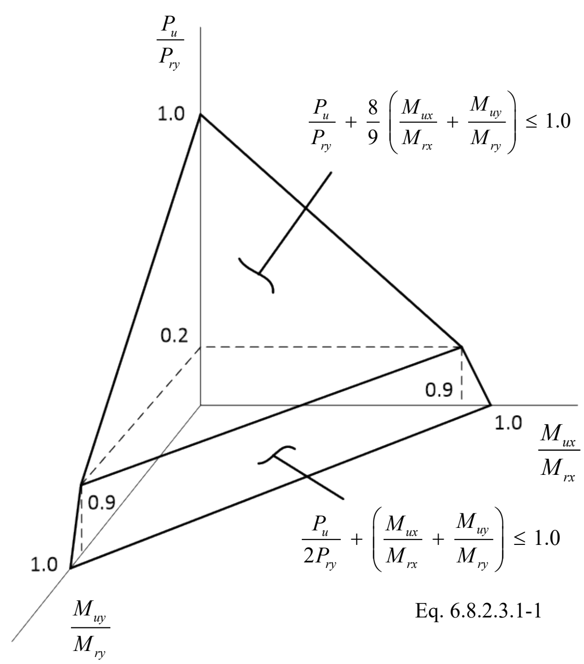


Figure C6.8.2.3.1-1—Interaction Between Axial Tension and Biaxial Bending Corresponding to Eqs. 6.8.2.3.1-1 and 6.8.2.3.1-2

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$$\left(\frac{M_{ux}}{M_{rxc}} + \frac{M_{uy}}{M_{ryc}}\right) \leq 1.0 \qquad (6.8.2.3.1\text{-}4)$$

where:

P_{ry} = factored tensile resistance based on tension yielding, obtained from Eq. 6.8.2.1-1 (kip)
P_u = factored tensile axial force (kip)
M_{rx} = factored flexural resistance about the x-axis taken as ϕ_f times the nominal flexural resistance about the x-axis determined as specified in Article 6.10, 6.11, or 6.12, as applicable, excluding tension flange rupture (kip-in.)
M_{rxc} = factored flexural resistance about the x-axis taken as ϕ_f times the nominal flexural resistance about the x-axis considering compression buckling, determined as specified in Article 6.10 or 6.12, as applicable (kip-in.)
M_{rxpe} = for I-section members, ϕ_f times the plastic moment about the x-axis neglecting any web longitudinal stiffeners; for noncomposite box-section members, ϕ_f times the effective plastic moment about the x-axis, based on the effective compression flange area as defined in Article 6.12.2.2.2c or 6.12.2.2.2d, as applicable, and neglecting any web longitudinal stiffeners (kip-in.)
M_{ry} = factored flexural resistance about the y-axis taken as ϕ_f times the nominal flexural resistance about the y-axis determined as specified in Article 6.12, as applicable, excluding tension flange rupture (kip-in.)
M_{ryc} = factored flexural resistance about the y-axis taken as ϕ_f times the nominal flexural resistance about the y-axis considering compression buckling, determined as specified in Article 6.12, as applicable; $M_{ryc} = M_{ryt} = M_{ry}$ for I-section members (kip-in.)
M_{rype} = for I-section members, ϕ_f times the plastic moment about the weak axis; for noncomposite box-section members, ϕ_f times the effective plastic moment about the y-axis, based on the effective compression flange area as defined in Article 6.12.2.2.2c or 6.12.2.2.2d, as applicable, and neglecting any web longitudinal stiffeners (kip-in.)
M_{ux}, M_{uy} = factored moments about the x- and y-axes, respectively (kip-in.)
ϕ_f = resistance factor for flexure specified in Article 6.5.4.2

For all cross-section plate elements that are supported along two longitudinal edges and are slender as defined in Article 6.9.4.2.2a, and for slender panels of

When M_{rx} is influenced by lateral–torsional buckling, M_{ux}/M_{rx} depends on the overall length effects associated with the lateral–torsional buckling strength limit state. Therefore, in this case, Eqs. 6.8.2.3.1-1 and 6.8.2.3.1-2 are not cross-section resistance checks. Hence, the largest value of M_{ux}/M_{rx} associated with the unbraced length relevant to the lateral–torsional buckling resistance should be considered along with the other cross-section based values of M_{uy}/M_{ry} and P_u/P_{ry}. That is, when M_{rx} is governed by lateral–torsional buckling, there is one M_{ux}/M_{rx} value for a given unbraced length that may be combined with the individual cross-section based M_{uy}/M_{ry} and P_u/P_{ry} values. Furthermore, one should recognize that the largest value of M_{ux}/M_{rx} does not necessarily occur for the load combination that gives the largest value of M_{ux}, as discussed in Article C6.9.2.2.1.

Considering the above attributes, the largest M_{ux}/M_{rx} associated with the unbraced length relevant to the lateral–torsional buckling resistance may be combined conservatively with the largest M_{uy}/M_{ry} and P_u/P_{ry} values along this length in evaluating Eqs. 6.8.2.3.1-1 and 6.8.2.3.1-2.

When M_{rx} is governed by a limit state other than lateral–torsional buckling, all the resistance terms in Eqs. 6.8.2.3.1-1 and 6.8.2.3.1-2 are cross-section based, and therefore, these equations may be evaluated on a cross-section by cross-section basis along the member length. The Engineer is referred to Article C6.9.2.2.1 for a more detailed discussion of when cross-section by cross-section based evaluation of strength interaction equations is and is not appropriate.

Eqs. 6.8.2.3.1-3 and 6.8.2.3.1-4 define the strength interaction curve shown in Figure C6.8.2.3.1-2, which conservatively recognizes that axial tension tends to have a negligible to beneficial impact on the flexural resistances associated with compression buckling; therefore, the unity check value from the compression-buckling based strength interaction equation, Eq. 6.8.2.3.1-4, need not consider the influence of the axial tension. Eq. 6.8.2.3.1-4 limits the flexural resistances to a linear interaction between M_{rxc} and M_{ryc}. The flexural resistance is not reduced below that associated with Eq. 6.8.2.3.1-4 until a linear interaction between the yield load in tension, P_{ry}, and the corresponding plastic moments, M_{rxpe} or M_{rype}, is reached according to Eq. 6.8.2.3.1-3. For doubly symmetric I-section members, Eqs. 6.8.2.3.1-3 and 6.8.2.3.1-4 provide an accurate to conservative representation of the C_b modifier effect defined in Section H1.2 of AISC (2016b).

When Eq. 6.8.2.3.1-3 is employed, all of the resistance terms are cross-section based, and therefore, this equation may be checked on a cross-section by cross-section basis along the length. However, when Eq. 6.8.2.3.1-4 is employed and M_{rxc} is influenced by lateral–

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longitudinally stiffened plates as defined in Article E6.1.2, the provisions of Article 6.9.4.5 also shall be satisfied.

torsional buckling, the above discussion pertaining to M_{rx} applies; hence, the largest value of M_{ux}/M_{rxc} associated with the length relevant to the lateral–torsional buckling resistance should be considered along with the other cross-section based values of M_{uy}/M_{ryc} along this length.

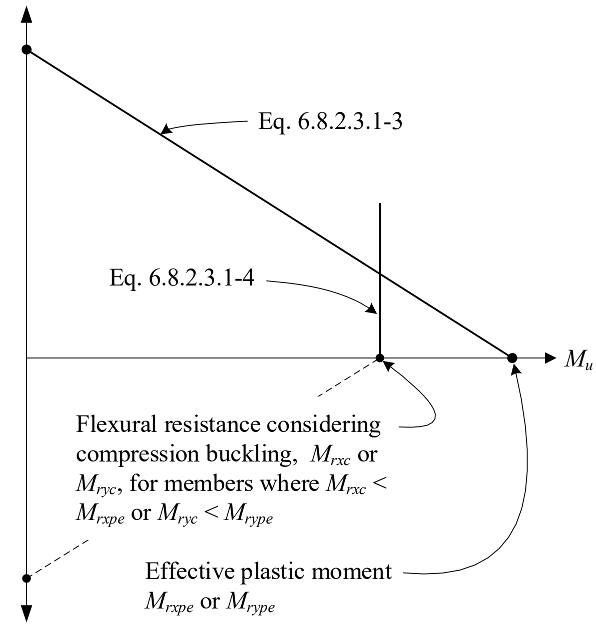

Figure C6.8.2.3.1-2—Interaction between Axial Tension and Compression, Flexural Yielding and Buckling in Flexural Compression Corresponding to Eqs. 6.8.2.3.1-3 and 6.8.2.3.1-4

For information on computing the factored flexural resistances in terms of stress about the x- and y-axes, and further discussion of the proper application of these equations, refer to Articles 6.9.2.2.1 and C6.9.2.2.1.

6.8.2.3.2—Interaction with Torsional and/or Flexural Shear

For the following member types:

- noncomposite rectangular box-section members, including square and rectangular HSS, and
- noncomposite circular tubes, including round HSS,

when the member is subjected to torsion resulting in a maximum ratio of the factored torsional shear stress to the corresponding cross-section element factored shear resistance, $f_{ve}/\phi_T F_{cv}$, greater than 0.2, the factored torsional shear stresses shall be considered within the applicable strength interaction equations as specified herein.

For general members, including those specified above, where P_u/P_{ry} is greater than 0.05, the factored flexural shear stresses shall be considered within the applicable strength interaction equations as specified

C6.8.2.3.2

The factored torsional shear stresses in the cross-section element under consideration for use in the computation of Δ, Δ_x, and Δ_y may be computed using the equations given in Article C6.9.2.2.2 in lieu of a refined analysis.

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herein; otherwise, the factored flexural shear stresses need not be considered.

Where the consideration of both the torsional and flexural shear stresses is required, the torsional and flexural shear stresses shall be summed based on their corresponding directions in each of the plate elements of the cross section, given the internal loadings. Where either of the above exclusion conditions are met, the corresponding contributions to the shear stresses shall be taken as zero. When the corresponding flexural shear, torsional shear and/or combined shear stress, as applicable, is nonzero:

- P_{ry} shall be multiplied by Δ,
- M_{rx}, M_{rxc} and M_{rxpe} shall be multiplied by Δ_x, and
- M_{ry}, M_{ryc} and M_{rype} shall be multiplied by Δ_y,

in Eqs. 6.8.2.3.1-1 through 6.8.2.3.1-4. ϕ_T is the resistance factor for torsion specified in Article 6.5.4.2. Δ, Δ_x, Δ_y, and F_{cv} shall be computed as specified in Article 6.9.2.2.2.

For elements of noncomposite rectangular box-section members, including square and rectangular HSS, the smallest value of Δ determined for each of the cross-section elements shall be used for Δ, the smallest Δ_x from the two flange elements of the cross section parallel to the x-axis and contributing to M_{rx} shall be used for Δ_x, and the smallest Δ_y from the two flange elements of the cross section parallel to the y-axis and contributing to M_{ry} shall be used for Δ_y.

For noncomposite circular tubes, including round HSS, only one calculation of Δ is required, based on the cross-section torque and/or the vector combination of the cross-section shears V_{ux} and V_{uy}, and this Δ shall be applied to each of the terms in the applicable member strength interaction equation.

For I-section members, the cross-section shear stresses due to torque shall be neglected. For these member types, Δ_x and Δ_y shall be taken equal to 1.0 in all cases, and the only the flexural shear stresses in the web shall be considered in the calculation of Δ.

6.8.2.3.3—Tension Rupture under Axial Tension or Compression Combined with Flexure

The following locations:

- Cross sections containing bolt holes in one or more flanges that are subjected to tension under combined axial tension or compression and flexure at connection or nonconnection locations,
- Cross sections at connection or nonconnection locations subjected to axial tension and flexure and containing bolt holes in other cross-section elements, or

C6.8.2.3.3

Eq. 6.8.2.3.3-1 addresses the strength interaction between flexure and axial tension or compression pertaining to tension rupture at the locations listed. This equation focuses on the specific axial force, tension or compression, combined with the specific moment at the cross section under consideration. The axial strength ratio term is negative in Eq. 6.8.2.3.3-1, causing a beneficial subtractive effect, when the cross section having the bolt holes is subjected to axial compression. The axial strength ratio term is positive, causing an additive effect, when the cross section is subjected to axial tension. This Article is a generalization of Section H4 of AISC (2016b),

- Cross sections at welded connections subjected to axial tension and flexure,

shall satisfy:

$$\frac{P_u}{P_r} + \frac{M_u}{M_r} \leq 1.0 \qquad (6.8.2.3.3\text{-}1)$$

in which:

M_r = factored tension rupture flexural resistance about the axis of bending under consideration (kip-in.)

$$= 0.84\left(\frac{A_{nf}}{A_{gf}}\right)F_u S_t \leq F_{yt} S_t \qquad (6.8.2.3.3\text{-}2)$$

where:

A_{nf} = net area of the tension flange determined as specified in Article 6.8.3; for sections not containing a flange loaded in flexural tension, and for sections at welded connections, A_{nf}/A_{gf} shall be taken equal to 1.0 (in.^{2})

A_{gf} = gross area of the tension flange; for sections not containing a flange loaded in flexural tension, and for sections at welded connections, A_{nf}/A_{gf} shall be taken equal to 1.0 (in.^{2})

F_u = specified minimum tensile strength determined as specified in Table 6.4.1-1 of the cross-section element under consideration (ksi)

F_{yt} = specified minimum yield strength of the cross-section element under consideration (ksi)

S_t = minimum elastic section modulus of the gross cross section about the axis of bending under consideration (in.^{3})

M_u = factored moment about the principal axis of bending under consideration at the cross section under consideration; positive for tension and negative for compression in the cross-section element under consideration (kip-in.)

P_r = for cross sections subjected to axial tension, factored tensile rupture resistance of the net section based on Eq. 6.8.2.1-2; for cross sections subjected to axial compression, factored tensile yield resistance of the cross section based on Eq. 6.8.2.1-1 (kip)

P_u = maximum factored axial force at the cross section under consideration, positive in tension and negative in compression (kip)

Each flange subjected to tension due to combined axial force and flexure shall be checked separately; otherwise, only the point on the cross section subjected to maximum tension due to combined axial force and flexure shall be checked. The moment, M_u, shall be checked separately

including the handling of tension flanges with holes in flexural members as specified in Article 6.10.1.8. The variable S_t is taken conservatively as the minimum elastic section modulus as in AISC (2016b).

Angle members and light structural tee members loaded eccentrically in axial tension are to be designed only for axial tension; the moment effects due to connection eccentricities are addressed in the calculation of the shear lag reduction factor, U, in Article 6.8.2.2.

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and independently about each principal axis of bending of the cross section.

For noncomposite box-section members, the flange areas of the gross cross section shall be reduced to account for shear lag, as applicable, in the calculation of S_t, as specified in Article 6.12.2.2.2g.

6.8.3—Net Area

The net area, A_n, of an element is the product of the thickness of the element and its smallest net width. The width of each standard bolt hole shall be taken as the nominal diameter of the hole. The width of oversize and slotted holes, where permitted for use in Article 6.13.2.4.1, shall be taken as the nominal diameter or width of the hole, as applicable, specified in Article 6.13.2.4.2. The net width shall be determined for each chain of holes extending across the member or element along any transverse, diagonal, or zigzag line.

The net width for each chain shall be determined by subtracting from the width of the element the sum of the widths of all holes in the chain and adding the quantity $s^2/4g$ for each space between consecutive holes in the chain, where:

s = pitch of any two consecutive holes (in.)
g = gauge of the same two holes (in.)

For angles, the gauge for holes in opposite adjacent legs shall be the sum of the gauges from the back of the angles less the thickness.

C6.8.3

The development of the "$s^2/4g$" rule for estimating the effect of a chain of holes on the tensile resistance of a section is described in McGuire (1968). Although it has theoretical shortcomings, it has been used for a long time and has been found to be adequate for ordinary connections.

In designing a tension member, it is conservative and convenient to use the least net width for any chain together with the full tensile force in the member. It is sometimes possible to achieve an acceptable, slightly less conservative design by checking each possible chain with a tensile force obtained by subtracting the force removed by each bolt ahead of that chain, i.e., closer to midlength of the member from the full tensile force in the member. This approach assumes that the full force is transferred equally by all bolts at one end.

6.8.4—Limiting Slenderness Ratio for Tension Members

For members subject to tension only, other than rods, eyebars, cables, and plates, or for evaluating the tension slenderness of compression members subject to stress reversal, the following slenderness requirements shall apply:

- For primary members $\frac{\ell}{r} \leq 200$
- For secondary members $\frac{\ell}{r} \leq 240$

where:

ℓ = unbraced length (in.)
r = radius of gyration (in.)

For evaluating the compression slenderness of tension members subject to stress reversal, the provisions of Article 6.9.3 shall apply.

6.8.5—Built up Members

6.8.5.1—General

The main elements of tension members built up from rolled or welded shapes shall be connected by continuous plates with or without perforations or by tie plates with or without lacing. Welded connections between shapes and plates shall be continuous. Bolted connections between shapes and plates shall conform to the provisions of Article 6.13.2.

C6.8.5.1

Perforated plates, rather than tie plates and/or lacing, are now used almost exclusively in builtup members. However, tie plates with or without lacing may be used where special circumstances warrant. Limiting design proportions are given in AASHTO *Standard Specifications* (2002) and AISC (2016b).

6.8.5.2—Perforated Plates

The ratio of length in the direction of stress to width of holes shall not exceed 2.0.

The clear distance between holes in the direction of stress shall not be less than the transverse distance between the nearest line of connection bolts or welds. The clear distance between the end of the plate and the first hole shall not be less than 1.25 times the transverse distance between bolts or welds.

The periphery of the holes shall have a minimum radius of 1.5 in.

The unsupported widths at the edges of the holes may be assumed to contribute to the net area of the member. Where holes are staggered in opposite perforated plates the net area of the member shall be considered the same as for a section having holes in the same transverse plane.

6.8.6—Eyebars

6.8.6.1—Factored Resistance

The factored resistance of the body of the eyebar shall be taken as specified in Eq. 6.8.2.1-1.

C6.8.6.1

Eq. 6.8.2.1-2 does not control because the net section in the head is at least 1.35 greater than the section in the body.

6.8.6.2—Proportions

Eyebars shall have a uniform thickness not less than 0.5 in. or more than 2.0 in.

The transition radius between the head and the body of an eyebar shall not be less than the width of the head at the centerline of the pin hole.

The net width of the head at the centerline of the pin hole shall not be less than 135 percent the required width of the body.

C6.8.6.2

The net dimension of the head beyond the pin hole taken in the longitudinal direction shall not be less than 75 percent of the width of the body.

The width of the body shall not exceed eight times its thickness.

The center of the pin hole shall be located on the longitudinal axis of the body of the eyebar. The pin-hole diameter shall not be more than 0.03125 in. greater than the pin diameter.

For steels having a specified minimum yield strength greater than 70 ksi, the hole diameter shall not exceed five times the eyebar thickness.

The limitation on the hole diameter for steel with specified minimum yield strengths above 70 ksi, which is not included in the AASHTO *Standard Specifications* (2002), is intended to prevent dishing beyond the pin hole (AISC, 2016b).

6.8.6.3—Packing

The eyebars of a set shall be symmetrical about the central plane of the member and as parallel as practicable. They shall be restrained against lateral movement on the pins and against lateral distortion due to the skew of the bridge.

The eyebars shall be so arranged that adjacent bars in the same panel will be separated by at least 0.5 in. Ring-shaped spacers shall be provided to fill any gaps between adjacent eyebars on a pin. Intersecting diagonal bars that are not sufficiently spaced to clear each other at all times shall be clamped together at the intersection.

C6.8.6.3

The eyebar assembly should be detailed to prevent corrosion-causing elements from entering the joints.

Eyebars sometimes vibrate perpendicular to their plane. The intent of this provision is to prevent repeated eyebar contact by providing adequate spacing or by clamping.

6.8.7—Pin-Connected Plates

6.8.7.1—General

Pin-connected plates should be avoided wherever possible.

The provisions of Article 6.8.2.1 shall be satisfied.

6.8.7.2—Pin Plates

The factored bearing resistance on pin plates, P_r, shall be taken as:

$$P_r = \phi_b P_n = \phi_b A_b F_y \qquad (6.8.7.2\text{-}1)$$

where:

P_n = nominal bearing resistance (kip)
A_b = projected bearing area on the plate (in.^{2})
F_y = specified minimum yield strength of the plate (ksi)
ϕ_b = resistance factor for bearing specified in Article 6.5.4.2

The main plate may be strengthened in the region of the hole by attaching pin plates to increase the thickness of the main plate.

If pin plates are used, they shall be arranged to minimize load eccentricity and shall be attached to the main plate by sufficient welds or bolts to transmit the bearing forces from the pin plates into the main plate.

6.8.7.3—Proportions

The combined net area of the main plate and pin plates on a transverse cross section through the centerline of the pin hole shall not be less than 1.4 times the required net area of the main plate away from the hole.

The combined net area of the main plate and pin plates beyond the pin hole taken in a longitudinal direction shall not be less than the required net area of the main plate away from the pin hole.

The center of the pin hole shall be located on the longitudinal axis of the main plate. The pin hole diameter shall not be more than 0.03125 in. greater than the pin diameter.

For steels having a specified minimum yield strength greater than 70.0 ksi, the hole diameter shall not exceed five times the combined thickness of the main plate and pin plates.

The combined thickness of the main plate and pin plates shall not be less than 12 percent of the net width from the edge of the hole to the edge of the plate or plates. The thickness of the main plate shall not be less than 12 percent of the required width away from the hole.

C6.8.7.3

The proportions specified in this Article assure that the member will not fail in the region of the hole if the strength limit state is satisfied in the main plate away from the hole.

6.8.7.4—Packing

Pin-connected members shall be restrained against lateral movement on the pin and against lateral distortion due to the skew of the bridge.

C6.8.7.4

The pin-connected assembly should be detailed to prevent corrosion-causing elements from entering the joints.

6.9—COMPRESSION MEMBERS

6.9.1—General

The provisions of this Article shall apply to prismatic noncomposite and composite steel members subjected to either axial compression or combined axial compression and flexure.

Arches shall also satisfy the requirements of Article 6.14.4.

Compression chords of half-through trusses shall also satisfy the requirements of Article 6.14.2.9.

C6.9.1

Conventional column design formulas contain allowances for imperfections and eccentricities permissible in normal fabrication and erection. The effect of any significant additional eccentricity should be accounted for in bridge design.

6.9.2—Compressive Resistance

6.9.2.1—Axial Compression

The factored resistance of components in compression, P_r, shall be taken as:

$$P_r = \phi_c P_n \quad (6.9.2.1\text{-}1)$$

where:

P_n = nominal compressive resistance as specified in Articles 6.9.4 or 6.9.5, as applicable (kip)

ϕ_c = resistance factor for compression as specified in Article 6.5.4.2

6.9.2.2—Combined Axial Compression, Flexure, and Flexural and/or Torsional Shear

C6.9.2.2

These provisions address the strength interaction for any combination of axial compression, uniaxial or biaxial flexure, and flexural and/or torsional shear, including combinations where one or more of the individual actions may be zero.

6.9.2.2.1—General

Except as permitted otherwise in Articles 6.9.4.4 and 6.9.6.3, the factored moments, M_{ux} and M_{uy}, and factored axial compressive load, P_u, calculated by elastic analysis shall satisfy the following relationships, as applicable, with all ratios taken as positive:

- For members in which all of the cross-section elements are defined as compact for flexure according to the provisions of Articles A6.2.1, A6.3.2, 6.11.6.2.2, and 6.12.2.2.2c, as applicable:
  - If $\frac{P_u}{P_r} < 0.2$, then

$$\frac{P_u}{2P_r} + \left(\frac{M_{ux}}{M_{rx}} + \frac{M_{uy}}{M_{ry}}\right) \leq 1.0 \quad (6.9.2.2.1\text{-}1)$$

  - If $\frac{P_u}{P_r} \geq 0.2$, then

$$\frac{P_u}{P_r} + \frac{8}{9}\left(\frac{M_{ux}}{M_{rx}} + \frac{M_{uy}}{M_{ry}}\right) \leq 1.0 \quad (6.9.2.2.1\text{-}2)$$

- The following may be employed for all types of members:

$$\frac{P_u}{P_r} + \frac{M_{ux}}{M_{rx}} + \frac{M_{uy}}{M_{ry}} \leq 1.0 \quad (6.9.2.2.1\text{-}3)$$

C6.9.2.2.1

Interaction equations in members subjected to axial tension or compression in combination with other loading effects generally involve significant design simplification. Such equations involving exponents of 1.0 on the moment ratios are often conservative. More exact, nonlinear interaction curves are available and are discussed in Ziemian (2010). If these interaction equations are used, additional investigation of service limit state stresses may be necessary to avoid premature yielding.

Eqs. 6.9.2.2.1-1 and 6.9.2.2.1-2 are identical to Eqs. (H1-1a) and (H1-1b) of AISC (2016b). They were selected for use in that Specification after being compared with a number of alternative formulations considering the results from refined inelastic analyses of 82 frame sidesway cases (Kanchanalai, 1977) involving rolled wide-flange section members. The strength envelope represented by these equations is similar to that shown for the axial tension and biaxial bending case in Figure C6.8.2.3.1-1. Eqs. 6.9.2.2.1-1 and 6.9.2.2.1-2 provide an accurate to conservative approximation of the resistances under combined loading for members in which all the cross-section elements are compact. Such members are potentially able to develop significant distributed yielding within their cross sections for small axial load and dominant flexural loading. As such, these types of members are able to develop a "knee" in the interaction curve between their flexural and axial compressive resistances. Members with other cross-section types generally have limited capability to develop such a "knee".

where:

P_r = factored compressive resistance as specified in Article 6.9.2.1 (kip)
P_u = factored compressive axial force (kip)
M_{rx} = factored flexural resistance about the x-axis taken equal to ϕ_f times the nominal flexural resistance about the x-axis determined as specified in Article 6.10, 6.11, or 6.12, as applicable, excluding tension flange rupture (kip-in.)
M_{ry} = factored flexural resistance about the y-axis taken equal to ϕ_f times the nominal flexural resistance about the y-axis determined as specified in Article 6.12, as applicable, excluding tension flange rupture (kip-in.)
M_{ux} = factored moment about the x-axis calculated as specified below (kip-in.)
M_{uy} = factored moment about the y-axis calculated as specified below (kip-in.)
ϕ_f = resistance factor for flexure specified in Article 6.5.4.2

The moments, M_{ux} and M_{uy}, shall be determined by:

- a second-order elastic analysis that accounts for the magnification of moment caused by the factored axial load, or
- the approximate single-step adjustment method specified in Article 4.5.3.2.2b, or a comparable amplification factor-based procedure.

For sections where the nominal flexural resistance about the major axis of the section is expressed in terms of stress, the factored flexural resistance about that axis in Eqs. 6.9.2.2.1-1 through 6.9.2.2.1-3 should be taken as:

$$M_{rx} = \text{the smaller of } \phi_f F_{nc} S_{xc} \text{ and } \phi_f F_{nt} S_{xt} \quad (6.9.2.2.1\text{-}4)$$

where:

F_{yc} = specified minimum yield strength of the compression flange (ksi)
F_{yt} = specified minimum yield strength of the tension flange (ksi)
F_{nc} = nominal flexural resistance of the compression flange (ksi)
F_{nt} = nominal flexural resistance of the tension flange (ksi)
M_{yc} = yield moment with respect to the compression flange determined as specified in Article D6.2 (kip-in.)
M_{yt} = yield moment with respect to the tension flange determined as specified in Article D6.2 (kip-in.)
S_{xc} = elastic section modulus about the major axis of the section to the compression flange taken as M_{yc}/F_{yc} (in.^{3})

For members where Eqs. 6.9.2.2.1-1 and 6.9.2.2.1-2 apply, the member ultimate resistances tend to involve extensive yielding in both tension and compression. For other member types, the member strength interaction is usually governed by additive compression buckling effects from axial compression and flexure, which are captured accurately to conservatively by the linear interaction Eq. 6.9.2.2.1-3.

The strength interaction between flexure and axial tension or compression pertaining to tension flange rupture at a cross section containing holes in the tension flange is addressed in Article 6.8.2.3.3. Article 6.8.2.3.3 provides a separate linear interaction equation focusing on the specific axial force combined with the specific moment causing the flexural tension in the flange at a given cross section.

P_u, M_{ux}, and M_{uy} are concurrent factored axial and flexural forces determined from a structural analysis. The cross sections exhibiting the maximum strength ratios generally are located at different positions along the member unbraced lengths. The beam-column strength interaction equations generally are not intended to be applied on a cross-section by cross-section basis along the member length, where typically only one of the strength ratios is maximum at a given cross section, and the other ratios are at their nonmaximum values. The cross-section by cross-section approach to evaluating the strength interaction is appropriate only when: 1) all of the flexural strength ratios are cross-section based, e.g., when all M_{uy}/M_{ry} values are combined with M_{ux}/M_{rx} values that are based on either flange local buckling, tension flange yielding, or the plateau strength in lateral–torsional buckling; and 2) the member is prismatic and the axial force P_u is constant along the length such that P_u/P_r is a constant value along the member length. The axial compressive strength ratio, P_u/P_r, and the lateral–torsional buckling strength ratio, M_{ux}/M_{rx}, when M_{rx} is less than the plateau strength, depend on the overall length effects associated with the corresponding stability behavior. These strength ratios are not cross-section limit states checks. Hence, performing the member strength interaction checks on a cross-section by cross-section basis with the equations provided in this Article is generally unconservative unless the above conditions are satisfied.

When P_u and/or M_{ux} vary along the member length, the appropriate resistance terms in the strength ratios P_u/P_r and/or M_{ux}/M_{rx} at a given cross-section should be the value of the axial load and/or the major-axis bending moment at that cross section when the overall member resistance is reached within the corresponding buckling mode. These separate maximum strength ratios along the member unbraced length are then combined together in the member strength interaction checks. If P_u varies along the length relevant to the governing buckling mode, it is common to determine P_r based on the assumption of constant axial compression and to use the largest value of P_u along this length in determining P_u/P_r. More

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S_{xt} = elastic section modulus about the major axis of the section to the tension flange taken as M_{yt}/F_{yt} (in.^{3})

For sections where the nominal flexural resistance about the major axis of the section is determined according to the provisions of Appendix A6, the factored flexural resistance about that axis in Eqs. 6.9.2.2.1-1 through 6.9.2.2.1-3 should be taken as:

$$M_{rx} = \text{the smaller of } \phi_f M_{nc} \text{ and } \phi_f M_{nt} \tag{6.9.2.2.1-5}$$

where:

M_{nc} = nominal flexural resistance based on the compression flange (kip-in.)

M_{nt} = nominal flexural resistance based on the tension flange (kip-in.)

Interaction with torsional and/or flexural shear, as applicable, shall be considered as specified in Article 6.9.2.2.2.

For all cross-section plate elements that are supported along two longitudinal edges and are slender as defined in Article 6.9.4.2.2a, and for slender panels of longitudinally stiffened plates as defined in Appendix E6.1.2, the provisions of Article 6.9.4.5 also shall be satisfied.

rigorously, the buckling resistance P_r can be determined as the internal axial force at a given cross section at overall buckling of the member, considering the influence of the variation of the axial force along the member length, and the corresponding P_u at this cross section can be divided by this P_r to obtain the governing strength ratio (White and Jeong, 2019). This more rigorous approach can also capture the influence of nonprismatic member geometry. This approach, along with the potentially beneficial influence of continuity effects and interaction buckling effects between the adjacent unbraced lengths, is discussed further in White and Jeong (2019).

In lieu of a more refined analysis, when considering the strength interaction at a given cross section, the maximum P_u/P_r and the maximum M_{ux}/M_{rx} values throughout the length relevant to the governing buckling modes should be used along with any cross-section based M_{ux}/M_{rx} and M_{uy}/M_{ry} values. As a simplification, the Engineer can combine the maximum P_u/P_r, M_{ux}/M_{rx} and M_{uy}/M_{ry} values throughout each of the smaller of the unbraced lengths ℓ_x, ℓ_y, ℓ_z, i.e., the respective column buckling unbraced lengths, and/or L_b, i.e., the lateral–torsional buckling unbraced lengths, for a given location along the overall member length when evaluating the strength interaction equations. This accounts for the stability interactions between responses that are maximum at different cross sections along the member length, while recognizing that, if say the largest P_u/P_r and/or M_{ux}/M_{rx} values are located at positions far removed from each other, i.e., at positions farther apart than the smaller of ℓ_x, ℓ_y, ℓ_z, and/or L_b, or from other cross-section based M_{ux}/M_{rx} or M_{uy}/M_{ry} values being applied within the strength interaction checks, the combination of these maximum values in the strength interaction is conservative. The physical beam column experiences all of these effects together, and the only way of determining the true interaction between these effects is to employ some type of advanced analysis method that considers them together within a consistent mechanics-based context, as discussed briefly in Article C6.1.

In addition to properly considering the length effects associated with P_u/P_r and M_{ux}/M_{rx} as discussed above, it is important to note that, due to the influence of the moment gradient factor, C_b, on the lateral–torsional buckling resistance, it is possible that a load combination with a major-axis bending moment smaller than the maximum M_{ux} value for the relevant load combinations under consideration can have a larger strength ratio M_{ux}/M_{rx} than the load combination corresponding to the largest M_{ux}. When evaluating the various load combinations for a given design, the concurrent loadings for the load combination having the largest M_{ux}/M_{rx} must be checked. The maximums of each of the strength ratios P_u/P_r, M_{ux}/M_{rx}, and M_{uy}/M_{ry} from all relevant load combinations may be summed conservatively to evaluate the resistance under the combined loading.

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S_{xc} and S_{xt} are defined in Eq. 6.9.2.2.1-4 as equivalent values that account for the combined effects of the loads acting on different sections in composite members.

For I- and H-shaped sections, the nominal flexural resistance about an axis parallel to the web is determined according to the provisions of Article 6.12.2.2.1.

For tees and double angles subject to combined axial compression and flexure in which the axial and flexural stresses in the flange of the tee or the flange legs of the angles are additive in compression, e.g., when a tee is used as a bracing member and the connection of this member is made to the flange, a bulge in the interaction curve occurs. As a result, Eqs. 6.9.2.2.1-1 and 6.9.2.2.1-2 may significantly underestimate the resistance in such cases. Alternative approaches attempting to capture this bulge have proven to be generally inconclusive or incomplete. Therefore, it is recommended that Eqs. 6.9.2.2.1-1 and 6.9.2.2.1-2 be conservatively applied to these cases. Should significant additional resistance be required, the use of one or more of these alternative approaches, as described in White (2012), may be considered.

Tee stems and double-angle web legs in which the toe of the stem or leg is in flexural compression are not considered as compact elements; therefore, Eq. 6.9.2.2.1-3 should be applied in cases where the toe of the tee stem or double-angle web legs are subject to flexural compression. Tee stems and double angle web legs in which the toe of the stem or leg is in flexural tension are considered as compact elements; thus, Eqs. 6.9.2.2.1-1 and 6.9.2.2.1-2 may be applied in this case.

6.9.2.2.2—Interaction with Torsional and/or Flexural Shear

For the following member types:

- noncomposite rectangular box-section members, including square and rectangular HSS, and
- noncomposite circular tubes, including round HSS,

when the member is subjected to torsion resulting in a maximum ratio of the factored torsional shear stress to the corresponding cross-section element factored shear resistance, $f_{ve}/\phi_T F_{cv}$, greater than 0.2, the factored torsional shear stresses shall be considered within the applicable strength interaction equations as specified herein.

For general members, including those specified above, where P_u/P_r is greater than 0.05, the factored flexural shear stresses shall be considered within the applicable strength interaction equations as specified herein; otherwise, the factored flexural shear stresses need not be considered.

Where the consideration of both the torsional and flexural shear stresses is required, the torsional and flexural shear stresses shall be summed based on their corresponding directions in each of the plate elements of

C6.9.2.2.2

Eqs. 6.9.2.2.2-1 through 6.9.2.2.2-3 address the influence of torsional and/or flexural shear, as applicable, on the resistance of noncomposite rectangular box-section members and noncomposite circular tubes, including round HSS. In addition, they address the interaction between the flexural shear resistance and the axial resistance of I-section members. I-section members with thin webs, subjected to significant axial force, potentially have a measurable interaction between their flexural shear and axial load resistances. The interaction between torsional and/or flexural shear stresses in I-section member flanges with other member resistances is neglected in these provisions.

Eqs. 6.9.2.2.2-1 through 6.9.2.2.2-3 are based on an interaction between the shear resistance and the combined axial and flexural resistance of the member and its component plates in which the axial and flexural strength ratios are taken as linear terms, with an exponent of 1, and the torsional and/or flexural shear strength ratio is taken as a quadratic term with an exponent of 2. The interaction with the torsional and/or flexural shear is applied to the axial compressive and flexural resistance terms, rather than writing a separate term involving the

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the cross section, given the internal loadings. Where either of the above exclusion conditions are met, the corresponding contributions to the shear stresses shall be taken as zero. When the corresponding flexural shear, torsional shear and/or combined shear stress, as applicable is nonzero:

- P_r shall be multiplied by Δ,
- M_{rx} shall be multiplied by Δ_x, and
- M_{ry} shall be multiplied by Δ_y

in Eqs. 6.9.2.2.1-1 through 6.9.2.2.1-3.

Δ, Δ_x, and Δ_y shall be computed as follows:

$$\Delta = 1 - \left(\frac{f_{ve}}{\phi_T F_{cv}}\right)^2 > 0 \qquad (6.9.2.2.2\text{-}1)$$

$$\Delta_x = 1 - \left(\frac{f_{vex}}{\phi_T F_{cvx}}\right)^2 > 0 \qquad (6.9.2.2.2\text{-}2)$$

$$\Delta_y = 1 - \left(\frac{f_{vey}}{\phi_T F_{cvy}}\right)^2 > 0 \qquad (6.9.2.2.2\text{-}3)$$

where:

ϕ_T = resistance factor for torsion specified in Article 6.5.4.2

f_{ve} = total factored shear stress due to torsion and/or flexure, as applicable, calculated in a cross-section element oriented parallel to the *x*- or *y*-axis of the cross section (ksi)

f_{vex} = total factored shear stress due to torsion and/or flexure, as applicable, calculated in a cross-section element oriented parallel to the *x*-axis of the cross section (ksi)

f_{vey} = total factored shear stress due to torsion and/or flexure, as applicable, calculated in a cross-section element oriented parallel to the *y*-axis of the cross section (ksi)

F_{cvx},
F_{cvy} = nominal shear resistance of a cross-section element under consideration taken as F_{cv} for that element as specified below (ksi)

torsional and/or flexural shear in the strength interaction equations.

The interaction assumed by Eqs. 6.9.2.2.2-1 through 6.9.2.2.2-3 gives an accurate to moderately conservative representation of the plastic strength interaction between normal and shear stresses obtained from the von Mises yield criterion, the inelastic buckling interaction in plates subjected to combined uniform axial compression and shear, and the elastic buckling interaction in plates subjected to combined bending within the plane of the plate and shear (Ziemian, 2010). The theoretical interaction curve between the normalized strength ratios is circular for each of these cases, which would result in the expressions within Eqs. 6.9.2.2.2-1 through 6.9.2.2.2-3 being taken to the ½, or square root, power. However, the plate elastic buckling interaction between uniform axial compression and shear is approximated more closely by an interaction equation involving a linear term for the axial compressive strength ratio and a quadratic term for the shear strength ratio, which results in the form given by Eqs. 6.9.2.2.2-1 through 6.9.2.2.2-3 (Ziemian, 2010). The largest difference between the overall strengths predicted by the circular interaction and the interaction using a linear term for the axial compressive strength ratio and a quadratic term for the shear strength ratio is 15 percent, corresponding to a shear strength ratio of 0.707. Also, the ultimate shear resistance in the theoretical elastic shear buckling range is larger than the theoretical elastic shear buckling resistance, due to postbuckling action. In these provisions, the interaction based on using a linear term for the axial compressive and flexural strength ratios is adopted to characterize the member for all of the types of loading considered. This is consistent with the form of the interaction equations in AISC (2016b) for torsion combined with axial force and flexure.

Although the interaction between the shear resistance and the axial and flexural resistances for box-section members is based largely on the theoretical strength interactions for the individual component plates, these interaction relationships are conflated into an overall member interaction relationship. This is comparable to the handling of the strength interactions for these types of members in AISC (2016b). Schilling (1965) shows that an interaction equation with the axial and moment strength ratios combined linearly and the shear strength ratio combined as a quadratic term gives a conservative estimate of the overall member resistance for noncomposite circular tube members governed by overall member elastic or inelastic buckling (Ziemian, 2010). Schilling's results provide additional justification for the use of Eqs. 6.9.2.2.2-1 through 6.9.2.2.2-3 for these types of members.

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F_{cv} = for noncomposite rectangular box-section members, including square and rectangular HSS, and for webs of I- and H-section members, the nominal shear buckling resistance of the cross-section element under consideration, under shear alone, calculated from Eq. 6.11.8.2.2-5, 6.11.8.2.2-6, or Eq. 6.11.8.2.2-7, as applicable, with b_{fc} taken as the total inside width of the element under consideration between the other cross-section elements it is connected to. For an element with longitudinal stiffeners, the panel width, w, shall be substituted for b_{fc} in the above equations and the shear buckling coefficient, k_s, shall be calculated as specified in Article 6.11.8.2.3. For noncomposite circular tubes, including round HSS, the flexural or torsional shear buckling resistance specified in Article 6.12.1.2.3b, as applicable, or where consideration of additive torsional and flexural shear stresses is required, the torsional shear buckling resistance specified in Article 6.12.1.2.3b (ksi).

For elements of noncomposite rectangular box-section members, including square and rectangular HSS, the smallest value of Δ determined for each of the cross-section elements shall be used for Δ, the smallest Δ_x from the two flange elements of the cross section parallel to the x-axis and contributing to M_{rx} shall be used for Δ_x, and the smallest Δ_y from the two flange elements of the cross section parallel to the y-axis and contributing to M_{ry} shall be used for Δ_y.

For noncomposite circular tubes, including round HSS, only one calculation of Δ is required, based on the cross-section torque and/or the vector combination of the cross-section shears V_{ux} and V_{uy}, and this Δ shall be applied to each of the terms in the applicable member strength interaction equation.

For I-section members, the cross-section shear stresses due to torque shall be neglected. For these member types, Δ_x and Δ_y shall be taken equal to 1.0 in all cases, and only the flexural shear stresses in the web shall be considered in the calculation of Δ.

Figure C6.9.2.2.2-1 shows a representative rectangular box-section profile subjected to biaxial bending moments, M_{ux} and M_{uy}, biaxial flexural shears, V_{ux} and V_{uy}, and torque, T_u, illustrating the terms employed in Eqs. 6.9.2.2.2-1 through 6.9.2.2.2-3. The box-section component plates may all have different thicknesses; however, in the unusual case where all the plates have a different thickness, it is recommended that the predominant web plates, i.e., the pair of plates subjected to larger shear stresses due to V_{ux} and V_{uy} and smaller uniform flexural stresses due to M_{ux} and M_{uy}, should be assumed to have the smaller thickness of these two plates. As such, the principal axes of the cross section are aligned with the walls of the box.

In Figure C6.9.2.2.2-1, the shear, V_{ux}, contributes to the shear stresses f_{vex1} and f_{vex2} within the plates that are parallel to the x-axis. The shear, V_{uy}, contributes to the shear stresses f_{vey1} and f_{vey2} within the plates that are parallel to the y-axis. In addition, the torque, T_u, contributes to the shear stresses in all of the plates. Each of the cross-section plates has a shear resistance, F_{cv}. The shear resistances of the plates parallel to the x-axis are referred to generally as F_{cvx}, and the corresponding shear resistances of the plates parallel to the y-axis are referred to as F_{cvy}. The shear resistance of any of the component plates is referred to generically as F_{cv}.

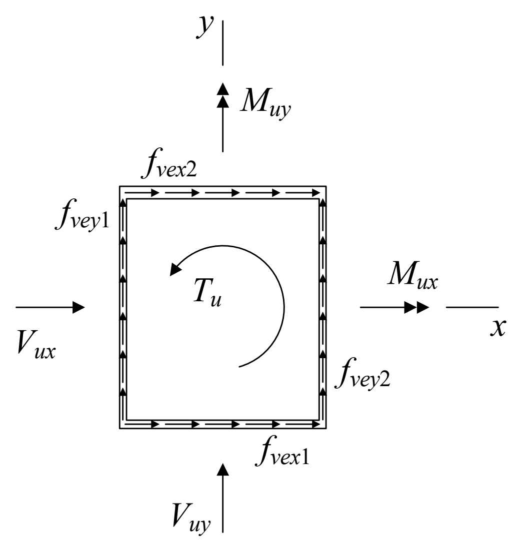

Figure C6.9.2.2.2-1—Representative Box-Section Profile Showing Internal Forces and Corresponding Plate-Element Stresses

Eq. 6.9.2.2.2-1 recognizes that the interaction between axial load and shear at the strength limit state can be based conservatively on the maximum f_{ve}/F_{cv} from all of the component plates. Eqs. 6.9.2.2.2-2 and 6.9.2.2.2-3 recognize that the interaction between the flexure about a given principal axis and the shear at the strength limit state is predominantly due to the associated f_{vex}/F_{cvx} or f_{vey}/F_{cvy} values. The ratio f_{vex}/F_{cvx} has a predomininant impact on the flexural resistance M_{rx}, and the ratio f_{vey}/F_{cvy} has a predominant impact on M_{ry}. The impact of the web shear ratios in either bending direction on the associated flexural resistance, i.e., f_{vex}/F_{cvx} on M_{ry} and f_{vey}/F_{cvy} on M_{rx}, is taken to be negligible.

Figure C6.9.2.2.2-2 shows a comparable illustration to Figure C6.9.2.2.2-1 for a representative circular tube section. In this case, the shears can be added vectorially and the resultant shear can be applied to the cross section to calculate the maximum contribution to the shear strength ratio from the flexural shears. The shear stresses from torsion are constant throughout the circumference of the tube, assuming constant thickness of the tube, and can be added to the maximum flexural shear stress to obtain the maximum total stress due to combined flexure and torsion. The corresponding f_{ve}/F_{cv} ratio is applied conservatively to P_r, M_{rx} and M_{ry}. For circular tubes subjected to combined flexural shear and torsion, the shear resistance is taken conservatively as the cross-section torsional shear buckling resistance, written in terms of stress, since this is smaller than the corresponding flexural shear buckling resistance.

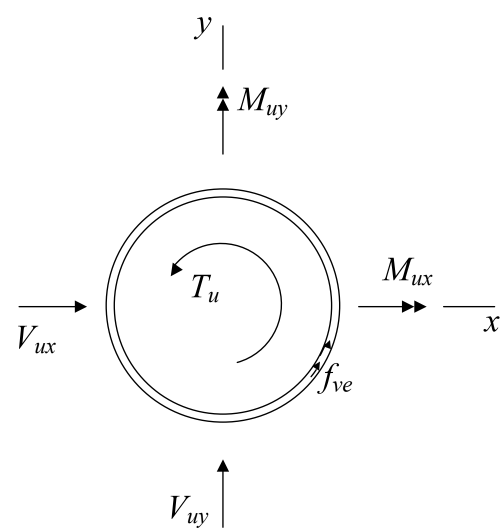

Figure C6.9.2.2.2-2—Representative Circular Tube Cross-section Profile Showing Internal Forces and Corresponding Element Stresses

As specified in Article 6.12.1.2.2, circular tube members subject to flexural shear and torsion must be checked using Eq. 6.12.1.2.3a-5 in addition to the interaction of the flexural or torsional shear resistances with the member axial and flexural resistances using the equations within this Article.

Figure C6.9.2.2.2-3 shows an illustration of a representative I- or H-section member, subjected to biaxial bending, M_{ux} and M_{uy}, biaxial shear, V_{ux} and V_{uy}, and torque, T_u. For these member types, the shear stresses due to torsion, and the flange shear stresses due to flexure are generally small and are assumed negligible. However, thin-web I-section members can be subject to significant web shear stresses. These stresses may have a significant influence on the member axial load resistance. For instance, the interaction between axial compression and web shear may be measurable in edge girders of cable-stayed bridges. Eq. 6.9.2.2.2-1 captures this strength limit state response.

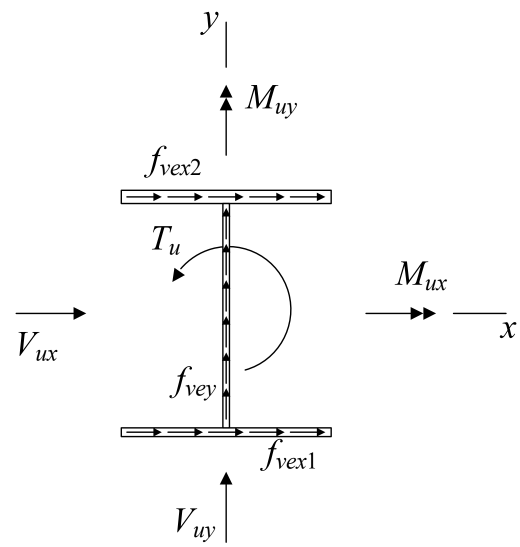

Figure C6.9.2.2.2-3—Representative I- or H-section Profile Showing Internal Forces and Corresponding Plate-Element Stresses

For I-section and H-section member webs and box-section member webs and flanges, the shear resistance is based on the theoretical shear buckling resistances, as represented by Eqs. 6.11.8.2.2-5, 6.11.8.2.2-6 and 6.11.8.2.2-7, using k_s = 5.34 for longitudinally unstiffened plates by reference to Article 6.11.8.2.2, and using the shear buckling coefficient from Article 6.11.8.2.3 for longitudinally stiffened plates. White et al. (2019b) provide alternative equations, derived from Eurocode 3 Part 1-5, that quantify the shear buckling resistance for longitudinally stiffened plates in noncomposite box-section members having any number of longitudinal stiffeners, not necessarily equally spaced, which account for the additional lateral restraint from transverse stiffeners when the transverse stiffeners are spaced at less than or equal to three times the plate width. These equations may be employed to realize a larger shear resistance for close transverse stiffener spacing.

In I- and H-section members subjected to torsion, the flanges may be subjected to significant additional lateral bending due to the restraint of warping. This additional flange lateral bending may be considered by calculating M_{uy}/M_{ry} considering each of the individual flanges as a separate component, and then combining the larger of these M_{uy}/M_{ry} values with the other strength ratios in the appropriate strength interaction equations. Alternatively, for I-section members subjected to major- and minor-axis bending plus torsion, the one-third rule provisions of

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Article 6.10 may be employed to assess these combined effects.

Also, for circular tubes, the shear resistance is taken as the theoretical flexural or torsional shear buckling resistance when considering the force interaction effects in these provisions. The interaction between shear postbuckling and the other member resistances is considered to not be sufficiently established, for general cases, to permit the consideration of interaction of shear postbuckling resistance with the other resistances.

The interaction of flange flexural shear stresses with the axial and flexural resistances of the member is assumed to be negligible for I-section and H-section members designed by these Specifications. The weak-axis shear resistance of these types of members is checked using Article 6.12.1.2.3a.

When the maximum ratio of the factored torsional shear stress to the corresponding cross-section element factored shear resistance, $f_{ve} / \phi_T F_{cv}$ is less than 0.2, the reduction in the member and plate axial compressive and flexural resistances due to the torsional shear stress is less than 4 percent, and is therefore neglected. Furthermore, when P_u / P_r is less than or equal to 0.05, the influence of this term on the unity check in Eqs. 6.9.2.2.1-1 through 6.9.2.2.1-3 is always less than or equal to 0.05. As such, the effect of the axial force on the design of the member may be neglected, as indicated in Articles C6.10.6.1 and C6.11.6.2.1.

In addition, when P_u / P_r is less than or equal to 0.05, the influence of flexural shear stresses may be neglected when applying Eqs. 6.9.2.2.2-1 through 6.9.2.2.2-3. This recognizes the well-established observation that moment-shear strength interaction is small and may be neglected in I- and box-girder flexural members (White et al., 2008; Johansson et al., 2007; AISC, 2016b). When evaluating the factored shear resistance of a member subject to torsion, additive shear stresses from flexure and from torsion are to be considered. However, these additive shear stresses need not be considered in evaluating the strength interaction in Article 6.9.2.2.2 when the exclusion clauses permitting the torsional shear stress and/or the flexural shear stress to be neglected are satisfied.

Flexural shear stresses in flanges need not be considered in Eqs. 6.9.2.2.2-1 through 6.9.2.2.2-3. The flange shear stresses due to flexure, which are tangent to the wall of the flange plate, are maximum at the connection to the webs, zero at a location within the middle of the plate, and the net shear force in the flanges is zero. The flexural shear stresses in flange elements of I-section members subjected to major-axis bending also need not be considered in Eqs. 6.9.2.2.2-1 through 6.9.2.2.2-3 for similar reasons. That is, flexural shear stresses only need to be considered in the member web or webs.

It should be noted that in box-section members, the flanges associated with one direction of bending are the webs associated with the other direction of bending, and

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vice versa. Therefore, for members subjected to biaxial bending, the same elements must be designed as a web element for bending in one direction, and as a flange element for bending in the other direction.

When addressing the additional interaction with torsion and/or flexural shear using the equations specified in Article 6.9.2.2.1, the maximum torsional shear and/or flexural shear ratios, $f_{ve}/\phi_T F_{cv}$, $f_{vex}/\phi_T F_{cvx}$, and $f_{vey}/\phi_T F_{cvy}$, which produce the corresponding minimum Δ, Δ_x and Δ_y values, are to be combined with the other strength ratios determined according to Article 6.9.2.2.1 within each of the smallest unbraced lengths along the overall member length. Based on the above considerations, the f_{ve}, f_{vex} and/or f_{vey} values in Article 6.9.2.2.1 may be taken as the combined torsional and flexural shear stresses, the torsional shear alone, the flexural shear alone, or zero.

In lieu of a refined analysis, the factored torsional shear stresses in the cross-section element under consideration for use in Eqs. 6.9.2.2.2-1 through 6.9.2.2.2-3 may be calculated as follows:

- For noncomposite circular tubes, including round HSS:

$$= \frac{T_u}{2\pi r^2 t} \qquad \text{(C6.9.2.2.2-1)}$$

- For web and flange elements of noncomposite rectangular box-section members, including square and rectangular HSS:

$$= \frac{T_u}{2A_o t} \qquad \text{(C6.9.2.2.2-2)}$$

where:

A_o = enclosed area within the box section (in.^{2})
T_u = factored torque (kip-in.)
r = radius to the mid-thickness of the tube (in.)
t = element thickness (in.); for HSS, the provisions of Article 6.12.1.2.4 shall apply.

6.9.3—Limiting Slenderness Ratio for Compression Members

For members subject to compression only or for evaluating the compression slenderness of tension members subject to stress reversal, the following slenderness requirements shall apply:

- For primary members: $\dfrac{K\ell}{r} \leq 120$
- For secondary members: $\dfrac{K\ell}{r} \leq 140$

where:

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K = effective length factor specified in Article 4.6.2.5
ℓ = unbraced length (in.)
r = radius of gyration (in.)

For evaluating the tension slenderness of compression members subject to stress reversal, the provisions of Article 6.8.4 shall apply.

For the purpose of this Article only, the radius of gyration may be computed on a notional section that neglects part of the area of a component, provided that:

- the capacity of the component based on the actual area and radius of gyration exceeds the factored loads, and
- the capacity of the notional component based on a reduced area and corresponding radius of gyration also exceeds the factored loads.

6.9.4—Noncomposite Members

6.9.4.1—Nominal Compressive Resistance

6.9.4.1.1—General

The nominal compressive resistance, P_n, shall be taken as the smallest value based on the applicable modes of flexural buckling, torsional buckling, and flexural–torsional buckling as follows:

- Applicable buckling modes for doubly symmetric members:
  - Flexural buckling
  - Torsional buckling for open-section members in which the effective torsional unbraced length is larger than the effective lateral unbraced length
- Applicable buckling modes for singly symmetric members:
  - Flexural buckling
  - Flexural–torsional buckling for open-section members
- Applicable buckling modes for unsymmetric members:
  - Flexural–torsional buckling for open-section members, except that single-angle members shall be designed according to the provisions of Article 6.9.4.4 using the flexural buckling resistance equations with an effective slenderness ratio $(KL/r)_{eff}$

C6.9.4.1.1

Eqs. 6.9.4.1.1-1 and 6.9.4.1.1-2 are equivalent to the corresponding axial compressive resistance equations given in AISC (2016b). These baseline equations are applicable for cross sections without any longitudinal stiffeners and in which all the component elements satisfy the corresponding width-to-thickness or slenderness ratio limits specified in Article 6.9.4.2.1. The equations are written in terms of the critical elastic buckling resistance, P_e, and the equivalent nominal yield resistance, P_o, to facilitate the calculation of the nominal resistance for members subject to buckling modes in addition to, or other than, flexural buckling. Also, this form of the resistance equations may be used to conveniently calculate P_n when a refined buckling analysis is employed to assess the stability of trusses, frames or arches in lieu of utilizing an effective length factor approach (White, 2012). In such cases, P_e in Eqs. 6.9.4.1.1-1 and 6.9.4.1.1-2 would be taken as the axial load in a given member taken from the analysis at incipient elastic buckling of the structure or subassemblage.

Eqs. 6.9.4.1.1-1 and 6.9.4.1.1-2 are approximately the same as column strength curve 2P of Ziemian (2010). These equations are based on a mean out-of-straightness of L/1500. The development of the mathematical form of these equations is described in Tide (1985), and the structural reliability they are intended to provide in the context of building design applications is discussed in Galambos (2006). Due to the large torsional stiffness of closed-section members, the reduction in the resistance due to the influence of torsional buckling deformations is small. Therefore, only flexural buckling is considered for closed-section members.

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- Applicable buckling modes for closed-section members:
  - Flexural buckling

For compression members with cross sections composed only of nonslender longitudinally unstiffened elements satisfying the width-to-thickness or slenderness ratio limits specified in Article 6.9.4.2.1, P_n shall be determined as follows:

- If $\dfrac{P_o}{P_e} \leq 2.25$, then:

$$P_n = \left[0.658^{\left(\frac{P_o}{P_e}\right)}\right] P_o \qquad (6.9.4.1.1\text{-}1)$$

- Otherwise:

$$P_n = 0.877 P_e \qquad (6.9.4.1.1\text{-}2)$$

where:

A_g = gross cross-sectional area of the member (in.^{2})
F_y = specified minimum yield strength; for nonhomogeneous cross-section members, F_y may be taken as the smallest specified minimum yield strength of all the cross-section elements in lieu of a more refined calculation (ksi)
P_e = elastic critical buckling resistance determined as specified in Article 6.9.4.1.2 for flexural buckling, and as specified in Article 6.9.4.1.3 for torsional bucking or flexural–torsional buckling, as applicable (kip)
P_o = nominal yield resistance = $F_y A_g$ (kip)

Table 6.9.4.1.1-1 may be used for guidance in selecting the appropriate potential column-buckling mode(s) to be considered in the determination of P_n, and the equations to use for the calculation of P_e.

For compression members with cross sections containing any slender elements, P_n shall be determined as specified in Article 6.9.4.2.2. For compression members with cross sections containing any longitudinally stiffened plates, P_n shall be determined as specified in Article E6.1.

For compression members with cross sections containing any slender elements, i.e., cross sections containing one or more longitudinal unstiffened elements not satisfying the corresponding width-to-thickness or slenderness ratio limits specified in Article 6.9.4.2.1, P_n is instead to be determined according to the provisions of Article 6.9.4.2.2 to account for the effect of potential local buckling of those elements on the overall buckling resistance of the member. For longitudinally stiffened plates, the effects of the longitudinal stiffeners and their tributary plate width acting as stiffener struts, as well as the potential local buckling of the individual stiffened plate panels, are addressed directly in Article E6.1.3.

For nonhomogeneous cross-section members, this Article specifies conservatively that F_y may be taken as the smallest specified minimum yield strength of all the cross-section elements for the calculation of P_o. White et al. (2019b) discuss a more rigorous approach for the calculation of P_o for nonhomogeneous members with doubly-symmetric I- and box-section profiles.

For bearing stiffeners, only the limit state of flexural buckling is applicable. In addition, given the width-to-thickness ratio limits for bearing stiffener cross-section elements specified by Article 6.10.11.2.2 and 6.10.11.2.4b, bearing stiffeners are effectively composed only of nonslender elements. The contribution of a thin web to the bearing stiffener axial compressive resistance is accounted for within the effective section provisions of Article 6.10.11.2.4b. As such, Article 6.9.4.1.1 is applicable for calculating the axial compressive resistance of bearing stiffeners.

The need for consideration of local buckling of slender plate elements via Article 6.9.4.2.2, and the need for consideration of longitudinal stiffeners and their tributary plate width acting as stiffener struts as well as the potential local buckling of the individual stiffened plate panels via Article E6.1, is avoided by using longitudinally unstiffened plates satisfying the requirement of Eq. 6.9.4.2.1-1. This equation references the nonslender longitudinally unstiffened plate limits specified in Table 6.9.4.2.1-1.

In some cases, it may be more economical to use members having one or more slender plates in which the strength is reduced due to local buckling effects. For instance, rolled wide-flange sections with ratios of $d/b_f \geq 1.7$, where d is the section depth and b_f is the flange width, typically have slender webs for uniform axial compression. Webs of welded I- and box sections also typically are classified as slender elements for axial compression according to Eq. 6.9.4.2.1-1. Flanges of welded box-section members subject to bending and axial compression may be slender with respect to the axial compressive resistance in regions of low bending moment, since less plate thickness is needed to resist the axial force plus bending in these regions for a given plate width. The stems of a significant number of rolled T-sections and one or both legs of many rolled angle sections are also classified as slender elements. Furthermore, a large number of square or rectangular HSS profiles have slender wall elements corresponding to a member subjected to axial compression.

Article CE6.1.3 discusses cases where it may or may not be beneficial to consider longitudinal stiffening of cross-section plate elements.

Flowcharts illustrating the application of the provisions of Article 6.9.4 for determining the compressive resistance of noncomposite I- or box-section members, with or without longitudinally stiffened plates, are provided in Article C6.5.1 in Appendix C6.

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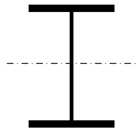
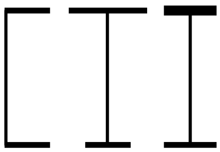
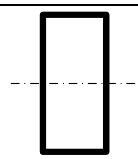
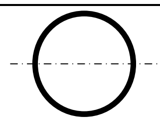
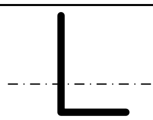
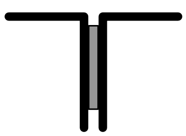
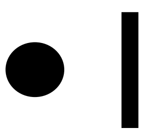
Table 6.9.4.1.1-1—Selection Table for Determination of Nominal Compressive Resistance, P_n

Cross Section	Potential Column-Buckling Mode	Applicable Equation for P_e
	FB	(6.9.4.1.2-1)
	and if $K_z\ell_z > K_y\ell_y$: TB	(6.9.4.1.3-1) Note: see also Article C6.9.4.1.3
	FB	(6.9.4.1.2-1)
	and: FTB	(6.9.4.1.3-2) Note: see also Article C6.9.4.1.3
	FB	(6.9.4.1.2-1) Note: for built-up sections, see also Article 6.9.4.3
	FB	(6.9.4.1.2-1)
	FB	(6.9.4.1.2-1)
	and: FTB	(6.9.4.1.3-2) Note: see also Article C6.9.4.1.3
	FB	(6.9.4.1.2-1) Note: see also Articles 6.9.4.4 and C6.9.4.4
	FB	(6.9.4.1.2-1) Note: see also Article 6.9.4.3
	and: FTB	(6.9.4.1.3-2) Note: see also Article C6.9.4.1.3
	FB	(6.9.4.1.2-1)
Unsymmetric Open-Sections	FTB	(6.9.4.1.3-7) Note: see also Article C6.9.4.1.3
Unsymmetric Closed-Sections	FB	(6.9.4.1.2-1)
Bearing Stiffeners	FB	(6.9.4.1.2-1) Note: See also Article 6.10.11.2.4

where:

FB = flexural buckling
TB = torsional buckling
FTB = flexural–torsional buckling

Note: For compression members with cross sections containing any slender elements, P_n is to be determined according to the provisions of Article 6.9.4.2.2 to account for the effect of potential local buckling of those cross-section elements on the overall compressive resistance of the member. For compression members with cross sections containing any longitudinally stiffened plates, P_n is to be determined according to the provisions of Article E6.1.1 to account for the action of the longitudinal stiffeners and their tributary plate widths acting as stiffener struts including any local buckling effects in slender longitudinally stiffened plate panels. In such cases, the same potential column-buckling modes and corresponding values of P_e given in this table shall be considered in the computation of P_n.

6.9.4.1.2—Elastic Flexural Buckling Resistance

In lieu of an alternative buckling analysis, the elastic critical buckling resistance, P_e, based on flexural buckling shall be taken as:

$$P_e = \frac{\pi^2 E}{\left(\frac{K\ell}{r_s}\right)^2} A_g \quad (6.9.4.1.2\text{-}1)$$

where:

E = modulus of elasticity of steel (ksi)
A_g = gross cross-sectional area of the member (in.^{2})
K = effective length factor in the plane of buckling determined as specified in Article 4.6.2.5
ℓ = unbraced length in the plane of buckling (in.)
r_s = radius of gyration about the axis normal to the plane of buckling (in.)

C6.9.4.1.2

Flexural buckling of concentrically loaded compression members refers to a buckling mode in which the member deflects laterally without twist or a change in the cross-sectional shape. Flexural buckling involves lateral displacements of the member cross sections in the direction perpendicular to the *x*- or *y*-axes that are resisted by the flexural rigidities, EI_x or EI_y, of the member, respectively.

Eq. 6.9.4.1.2-1 should be used to calculate the critical flexural buckling resistances about the *x*- and *y*-axes, with the smaller value taken as P_e for use in Eq. 6.9.4.1.1-1 or 6.9.4.1.1-2, as applicable.

6.9.4.1.3—Elastic Torsional Buckling and Flexural–Torsional Buckling Resistance

In lieu of an alternative buckling analysis, for open-section doubly symmetric members, the elastic critical buckling resistance, P_e, based on torsional buckling shall be taken as:

$$P_e = \left[\frac{\pi^2 EC_w}{\left(K_z \ell_z\right)^2} + GJ\right]\frac{A_g}{I_x + I_y} \quad (6.9.4.1.3\text{-}1)$$

where:

A_g = gross cross-sectional area of the member (in.^{2})
C_w = warping torsional constant (in.^{6})
G = shear modulus of elasticity for steel = 0.385E (ksi)
I_x, I_y = moments of inertia about the major and minor principal axes of the cross section, respectively (in.^{4})
J = St. Venant torsional constant (in.^{4})
$K_z\ell_z$ = effective length for torsional buckling (in.)

In lieu of an alternative buckling analysis, for open-section singly symmetric members where *y* is the axis of symmetry of the cross section, the elastic critical buckling resistance, P_e, based on flexural–torsional buckling shall be taken as:

$$P_e = \left(\frac{P_{ey} + P_{ez}}{2H}\right)\left[1 - \sqrt{1 - \frac{4P_{ey}P_{ez}H}{\left(P_{ey} + P_{ez}\right)^2}}\right] \quad (6.9.4.1.3\text{-}2)$$

in which:

C6.9.4.1.3

Torsional buckling of concentrically loaded compression members refers to a buckling mode in which the member twists about its shear center. Torsional buckling applies only for open-section doubly symmetric compression members for which the locations of the centroid and shear center coincide. Torsional buckling will rarely control and need not be considered for doubly symmetric I-section members satisfying the cross-section proportion limits specified in Article 6.10.2, unless the effective length for torsional buckling is significantly larger than the effective length for *y*-axis flexural buckling. The effective length for torsional buckling, $K_z\ell_z$, is typically taken as the length between locations where the member is prevented from twisting. That is, in many cases, $K_z\ell_z$ can be taken conservatively as $1.0\ell_z$. For a cantilever member fully restrained against twisting and warping at one end with the other end free, $K_z\ell_z$ should be taken as 2ℓ where ℓ is the length of the member (White, 2012). For a member with twisting and warping restrained at both ends, $K_z\ell_z$ may be taken as 0.5ℓ. For a doubly symmetric I-section, C_w may be taken as $I_y h^2/4$, where *h* is the distance between flange centroids, in lieu of a more precise analysis. For closed sections, C_w may be taken equal to zero and *GJ* is relatively large. Because of the large *GJ*, torsional buckling and flexural–torsional buckling need not be considered for closed sections, including built-up members connected by lacing bars, batten plates, perforated plates, or any combination thereof.

Flexural–torsional buckling of concentrically loaded compression members refers to a buckling

$$H = 1 - \frac{y_o^2}{\bar{r}_o^2} \quad (6.9.4.1.3\text{-}3)$$

$$P_{ey} = \frac{\pi^2 E}{\left(\frac{K_y \ell_y}{r_y}\right)^2} A_g \quad (6.9.4.1.3\text{-}4)$$

$$P_{ez} = \left(\frac{\pi^2 E C_w}{(K_z \ell_z)^2} + GJ\right)\frac{1}{\bar{r}_o^2} \quad (6.9.4.1.3\text{-}5)$$

$$\bar{r}_o^2 = y_o^2 + \frac{I_x + I_y}{A_g} \quad (6.9.4.1.3\text{-}6)$$

where:

$K_y\ell_y$ = effective length for flexural buckling about the y-axis (in.)
$\bar{r}_o$ = polar radius of gyration about the shear center (in.)
r_y = radius of gyration about the y-axis (in.)
y_o = distance along the y-axis between the shear center and centroid of the cross section (in.)

In lieu of an alternative buckling analysis, for open-section unsymmetric members, the elastic critical buckling resistance, P_e, based on flexural–torsional buckling shall be taken as the lowest root of the following cubic equation:

$$\left(P_e - P_{ex}\right)\left(P_e - P_{ey}\right)\left(P_e - P_{ez}\right) - P_e^2\left(P_e - P_{ey}\right)\left(\frac{x_o}{\bar{r}_o}\right)^2 - P_e^2\left(P_e - P_{ex}\right)\left(\frac{y_o}{\bar{r}_o}\right)^2 = 0 \quad (6.9.4.1.3\text{-}7)$$

in which:

$$P_{ex} = \frac{\pi^2 E}{\left(\frac{K_x \ell_x}{r_x}\right)^2} A_g \quad (6.9.4.1.3\text{-}8)$$

$$\bar{r}_o^2 = x_o^2 + y_o^2 + \frac{I_x + I_y}{A_g} \quad (6.9.4.1.3\text{-}9)$$

where:

$K_x\ell_x$ = effective length for flexural buckling about the x-axis (in.)
r_x = radius of gyration about the x-axis (in.)

mode in which the member twists and bends simultaneously without a change in the cross-sectional shape. Compression members composed of open singly symmetric cross sections, where the y-axis is defined as the axis of symmetry of the cross-section, can fail either by flexural buckling about the x-axis or by torsion combined with flexure about the y-axis. Compression members composed of open unsymmetric cross sections, or members with no cross-section axis of symmetry, fail by torsion combined with flexure about the x- and y-axes. In both of the preceding cases, the centroid and shear center of the cross section do not coincide. As buckling occurs, the axial stresses have a lateral component resulting from the lateral deflection of the member. This lateral component, acting about the shear center of the cross section, causes simultaneous twisting of the member. The degree of interaction between the torsional and flexural deformations determines the reduction of this buckling load in comparison to the flexural buckling load (Ziemian, 2010). As the distance between the centroid and shear center increases, the twisting tendency increases and the flexural–torsional buckling load decreases. Flexural–torsional buckling may be a critical mode of failure for thin-walled open-section singly symmetric compression members, e.g., tees, double angles, and channels, and for open-section unsymmetric compression members due to their relatively low torsional rigidity. For open-section singly symmetric members, the critical flexural–torsional buckling resistance is always smaller than the critical flexural buckling resistance about the y-axis, P_{ey}. Therefore, in such cases, only flexural buckling about the x-axis need be considered along with flexural–torsional buckling. For open-section unsymmetric members, only flexural–torsional buckling is applicable; flexural buckling about the x- and y-axes need not be checked except that single-angle members are to be designed according to the provisions of Article 6.9.4.4 using only the flexural buckling resistance equations with an effective slenderness ratio $(KL/r)_{eff}$ (AISC, 2016b).

Eqs. 6.9.4.1.3-2 through 6.9.4.1.3-6 assume that the y-axis is defined as the axis of symmetry of the cross section. Therefore, for a channel, the y-axis should actually be taken as the x-axis of the cross section, or the axis of symmetry for the channel section, when applying these equations. C_w should conservatively be taken equal to zero for tees and double angles in the application of these equations. Refer to Article C6.12.2.2.4 for additional information on the calculation of the St. Venant torsional constant J for tees and double angles. For channels, refer to Article C6.12.2.2.5 for additional information on the calculation of C_w and J.

For singly symmetric I-section compression members with equal flange widths and differing flange thicknesses, flexural–torsional buckling need not be considered as long as $0.67 \leq t_{f1}/t_{f2} \leq 1.5$ and

x_o = distance along the x-axis between the shear center and centroid of the cross section (in.)

$K_z\ell_z \le K_y\ell_y$, where t_{f1} and t_{f2} are the flange thicknesses and K_z and K_y are the effective length factors for torsional buckling and for flexural buckling about the y-axis, respectively (White, 2012). However, flexural–torsional buckling should always be checked for singly symmetric I-sections that are loaded in axial compression when the flange widths are different. C_w may be computed as follows for such sections in lieu of a more precise analysis (Salmon and Johnson, 1996):

$$C_w = \frac{t_f h^2}{12}\left(\frac{b_1^3 b_2^3}{b_1^3 + b_2^3}\right) \qquad \text{(C6.9.4.1.3-1)}$$

where:

b_1, b_2 = individual flange widths (in.)
h = distance between flange centroids (in.)
t_f = flange thickness (in.) Use an average thickness if the flange thicknesses differ.

6.9.4.2—Effects of Local Buckling on the Nominal Compressive Resistance

6.9.4.2.1—Classification of Cross-Section Elements

Longitudinally unstiffened cross-section elements satisfying the following limit shall be defined as nonslender under member axial compression:

$$\frac{b}{t} \le \lambda_r \qquad \text{(6.9.4.2.1-1)}$$

where:

λ_r = corresponding width-to-thickness or slenderness ratio limit as specified in Table 6.9.4.2.1-1
b = element width as specified in Table 6.9.4.2.1-1 (in.)
t = element thickness (in.); for flanges of rolled channels, use the average thickness; for HSS, the provisions of Article 6.12.1.2.4 shall apply.

Local buckling effects shall be neglected for nonslender longitudinally unstiffened cross-section elements. Otherwise, longitudinally unstiffened elements shall be defined as slender under member axial compression and local buckling effects shall be considered according to the provisions of Article 6.9.4.2.2. For longitudinally stiffened cross-section elements, the strength of the stiffener struts, including potential local buckling effects, shall be considered according to the provisions of Article E6.1.

C6.9.4.2.1

Compression members with cross sections composed only of nonslender longitudinally unstiffened elements, i.e., cross sections without any longitudinal stiffeners in which all the component elements satisfy the corresponding width-to-thickness or slenderness ratio limits specified herein, are able to develop their full yield strength under uniform axial compression without any significant impact from local buckling. For compression members with cross sections containing any slender elements, i.e., longitudinally unstiffened elements not satisfying the corresponding width-to-thickness or slenderness ratio limits specified herein, the nominal compressive resistance is instead to be determined according to the provisions of Article 6.9.4.2.2 to account for the effect of potential local buckling of those cross-section elements on the overall compressive resistance of the member. White et al. (2019b) provide a detailed discussion of the background to these limits. For longitudinally stiffened plates, the effects of local buckling of the individual panels are addressed in Article E6.1.3.

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Table 6.9.4.2.1-1—Element Width-to-Thickness or Slenderness Ratio Limits and Element Widths for Axial Compression

Elements Supported along One Longitudinal Edge	λ_r	***b***
Stems of Rolled Tees	$0.75\sqrt{\frac{E}{F_y}}$	• Full depth of tee
Flanges of Rolled I-, Tee, and Channel Sections; Plates Projecting from Rolled I-Sections; and Outstanding Legs of Double Angles in Continuous Contact	$0.56\sqrt{\frac{E}{F_y}}$	• Half-flange width for rolled I- and T-sections • Full-flange width for channel sections • Distance between free edge and first line of bolts or welds for plates projecting from rolled I-sections • Full width of an outstanding leg for double angles in continuous contact
Flanges of Welded and Nonwelded Built-Up I-Sections; and Plates or Angle Legs Projecting from Built-Up I-Sections	$0.64\sqrt{\frac{k_c E}{F_y}}$	• Half-flange width for welded and nonwelded built-up I-sections
Outstanding Legs of Single Angles; Outstanding Legs of Double Angles with Separators; Flange Extensions of Box Sections; Plates or Angle Legs Projecting from Welded and Nonwelded Built-Up I- or Box Sections; and All Other Plates Supported along One Longitudinal Edge	$0.45\sqrt{\frac{E}{F_y}}$	• Full width of outstanding leg for single angle or double angles with separators • Full projecting width for all others
Elements Supported along Two Longitudinal Edges	λ_r	***b***
Perforated Cover Plates	$1.86\sqrt{\frac{E}{F_y}}$	• Clear distance between edge supports; see also the paragraph at the end of Article 6.9.4.3.2
Webs of Rolled I- and Channel Sections; Webs of Nonwelded Built-Up I- and Channel Sections	$1.49\sqrt{\frac{E}{F_y}}$	• Clear distance between flanges minus the fillet or corner radius at each flange for webs of rolled I- and channel sections • Distance between adjacent lines of bolts for webs of nonwelded built-up I- and channel sections
Flanges and Webs of Nonwelded Built-Up Box Sections; Walls of Square and Rectangular Hot-Formed HSS; and Nonperforated Flange Cover Plates	$1.40\sqrt{\frac{E}{F_y}}$	• Distance between adjacent lines of bolts for flanges of nonwelded built-up box sections • Distance between adjacent lines of bolts for webs of nonwelded built-up box sections • Clear distance between walls minus inside corner radius on each side for HSS. Use the outside dimension minus three times the appropriate design wall thickness for HSS specified in Article 6.12.1.2.4 if the corner radius is not known. • Distance between lines of welds or bolts for nonperforated flange cover plates

(continued on next page)

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Elements Supported Along Two Longitudinal Edges	λ_r	*b*
Walls of Square and Rectangular Cold-Formed HSS	$1.28\sqrt{\frac{E}{F_y}}$	• Clear distance between walls minus inside corner radius on each side. Use the outside dimension minus three times the appropriate design wall thickness for HSS specified in Article 6.12.1.2.4 if the corner radius is not known.
All Other Plates Supported along Two Longitudinal Edges	$1.09\sqrt{\frac{E}{F_y}}$	• Clear distance between flanges for webs of welded I, channel, and box sections • Clear distance between webs for flanges of welded box sections • For angle or T-section stiffener legs or stems connected to a stiffened plate, clear distance between the stiffened plate and the inside of the angle leg or T-section stem not connected to the stiffened plate • Clear distance between edge supports for all others
Other Elements	λ_r	*b*
Circular Tubes and Round HSS	$0.11\frac{E}{F_y}$	• Outside diameter of the tube

in which:

k_c = flange local buckling coefficient taken as follows:

- For flanges of welded and nonwelded built-up I-sections:

$$k_c = \frac{4}{\sqrt{\frac{D}{t_w}}} \qquad (6.9.4.2.1\text{-}2)$$

and

$$0.35 \le k_c \le 0.76 \qquad (6.9.4.2.1\text{-}3)$$

where:

D = web depth (in.)
t_w = web thickness (in.)

6.9.4.2.2—Slender Longitudinally Unstiffened Cross-Section Elements

6.9.4.2.2a—General

Compression member cross sections containing one or more longitudinally unstiffened elements, not satisfying the corresponding width-to-thickness or slenderness ratio limits specified in Article 6.9.4.2.1, i.e., slender elements, shall be subject to the requirements specified herein.

C6.9.4.2.2a

For compression members containing any slender longitudinally unstiffened cross-section elements local buckling of the component elements may adversely affect the overall buckling resistance of the member. Hence, the nominal compressive resistance, P_n, based on flexural, torsional, or flexural–torsional buckling, as applicable, may be reduced. For such members, P_n is determined

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For compression member cross sections containing any slender elements, the nominal compressive resistance, P_n, shall be taken as the smallest value based on the applicable modes of flexural buckling, torsional buckling, and flexural–torsional buckling, and shall be computed as follows:

$$P_n = F_{cr} A_{eff} \tag{6.9.4.2.2a-1}$$

in which:

$$F_{cr} = \frac{P_{cr}}{A_g} \tag{6.9.4.2.2a-2}$$

A_{eff} = effective area of the cross section (in.^{2}) determined as specified in Article 6.9.4.2.2c for circular tubes and round HSS. Otherwise, A_{eff} shall be taken as the summation of the effective areas of the cross-section elements determined as follows:

- For rolled-section and HSS members containing slender elements:

$$= A_g - \sum (b - b_e) t \tag{6.9.4.2.2a-3}$$

- Otherwise:

$$= \sum_{lusp} b_e t + \sum_c A_c \tag{6.9.4.2.2a-4}$$

where:

$\sum_{lusp}$ = summation over all longitudinally unstiffened cross-section plate elements

$\sum_c$ = summation over all the corner areas of a noncomposite box-section member

A_c = gross cross-sectional area of the corner pieces of a noncomposite box-section member (in.^{2})

A_g = total gross cross-sectional area of the member (in.^{2})

b = width of the element under consideration determined as specified in Table 6.9.4.2.1-1 (in.)

b_e = effective width of the element under consideration determined as specified in Article 6.9.4.2.2b for slender elements, and taken equal to b for nonslender elements (in.)

P_{cr} = nominal compressive resistance of the member calculated from Eq. 6.9.4.1.1-1 or 6.9.4.1.1-2, as applicable, using A_g (kip)

t = thickness of the element under consideration (in.); for flanges of rolled channels, use the average thickness; for HSS, the provisions of Article 6.12.1.2.4 shall apply.

using a generalized form of the unified effective width/ unified effective area approach (AISC, 2016b; AISI, 2016). Previous specifications utilized a dual philosophy for longitudinally unstiffened plates, commonly referred to as the Q factor method, in which slender elements supported on only one longitudinal edge were assumed to reach their limit of resistance when they attained their theoretical local buckling stress, while slender elements supported on both longitudinal edges utilized an effective width concept to obtain their post-buckling resistance. The unified effective width/unified effective area approach considers the effective width of both of these types of cross-section plate elements. This simplifies the resulting calculations and tends to provide a more accurate characterization of the ultimate strength of all types of slender longitudinally unstiffened plates that recognizes plate postbuckling resistance in all cases. The development and basis of the unified approach is discussed in more detail in the Commentary to Section E7 of AISC (2016b) and in Ziemian (2010).

Two equations are provided for the cross-section effective area to be used in the unified effective width/unified effective area approach. Eq. 6.9.4.2.2a-3 facilitates the inclusion of the fillet areas in rolled-section and square or rectangular HSS members containing any slender elements. Eq. 6.9.4.2.2a-4 addresses general welded and nonwelded built-up sections containing any slender plate elements. The second term of Eq. 6.9.4.2.2a-4 represents the sum of the gross cross-sectional areas of the four corner pieces of a noncomposite box-section member not included in the clear width of the component plates; for all other members, A_c is taken as zero. Flange extensions on box-section members, if present, should be evaluated to determine if they are nonslender or slender elements and included accordingly in Eq. 6.9.4.2.2a-4.

Eqs. 6.9.4.2.2a-1 through 6.9.4.2.2a-4 capture the influence of buckling of individual longitudinally unstiffened plates on the overall member axial compressive resistance in a simple yet accurate to conservative manner. White et al. (2019b) compare this approach to related procedures in AISI (2016) and CEN (2006) and to the results from tests and test simulations.

The original development of the unified effective width/unified effective area method (Peköz, 1986) showed that accurate predictions were obtained for general singly-symmetric and unsymmetric beam-columns, with the exception of slender angle sections, when the moment of the axial loads is taken about the centroidal axis of the effective section determined considering axial load alone. AISI (2016) relaxes the requirement that the bending moment should be defined with respect to the centroidal axis of the effective section. The increased eccentricity due to local buckling can have a measurable impact on the resistance of an ideally pin-ended member; however, this effect tends to become minor in continuous members or members with ends restrained, where the rotations due to these eccentricities are restrained. AISC (2016b) also neglects these effects.

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For all cross-section plate elements supported along two longitudinal edges that are slender as specified in this Article, the provisions of Article 6.9.4.5 also shall be satisfied.

For nonhomogeneous members, F_y may be taken as the smallest specified yield strength of all the cross-section elements for the calculation of F_{cr}. White et al. (2019b) discuss a more rigorous approach for the calculation of an effective F_y for the determination of F_{cr} for nonhomogeneous members with doubly-symmetric I- and box-section profiles.

6.9.4.2.2b—Effective Width of Slender Elements

The effective width, b_e, of slender elements shall be determined as follows:

- If $\frac{b}{t} \le \lambda_r \sqrt{\frac{F_y}{F_{cr}}}$, then:

$$b_e = b \tag{6.9.4.2.2b-1}$$

- If $\frac{b}{t} > \lambda_r \sqrt{\frac{F_y}{F_{cr}}}$, then:

$$b_e = b\left[\left(1 - c_1\sqrt{\frac{F_{el}}{F_{cr}}}\right)\sqrt{\frac{F_{el}}{F_{cr}}} - c_3\right] \tag{6.9.4.2.2b-2}$$

in which:

c_1 = effective width imperfection adjustment factor determined from Table 6.9.4.2.2b-1

c_2 = effective width imperfection adjustment factor determined from Table 6.9.4.2.2b-1

$$= \left(1 - \sqrt{1 - 4c_1\left(1 + c_3\right)}\right) \Big/ \left(2c_1\right) \tag{6.9.4.2.2b-3}$$

c_3 = effective width imperfection adjustment factor determined from Table 6.9.4.2.2b-1

F_{el} = elastic local buckling stress (ksi)

$$= \left(c_2 \frac{\lambda_r}{\left(b/t\right)}\right)^2 F_y \tag{6.9.4.2.2b-4}$$

where:

λ_r = corresponding width-to-thickness ratio limit as specified in Table 6.9.4.2.1-1

b = element width as specified in Table 6.9.4.2.1-1 (in.)

F_{cr} = nominal compressive resistance of the member calculated from Eq. 6.9.4.2.2a-2 (ksi)

t = element thickness (in.); for flanges of rolled channels, use the average thickness; for HSS, the provisions of Article 6.12.1.2.4 shall apply.

C6.9.4.2.2b

Compression member cross-section elements are defined as slender when full yielding of the gross cross section cannot be developed prior to local buckling impacting the resistance. However, if the stress level at overall member buckling, F_{cr}, is small enough relative to F_y, a slender cross-section element will not exhibit any significant local buckling effects prior to the member reaching its axial compressive resistance. This behavioral attribute is accounted for by checking the element width-to-thickness ratio, b/t, versus the modified limit $\lambda_r\sqrt{F_y/F_{cr}}$. If b/t is smaller than $\lambda_r\sqrt{F_y/F_{cr}}$, no significant local buckling effects occur prior to the member reaching its axial compressive resistance. In these cases, the effective width of the slender cross-section element is equal to the full element width as specified by Eq. 6.9.4.2.2b-1.

Eq. 6.9.4.2.2b-2 is a generalized form of the classical equation for plate local buckling effective widths implemented originally in the AISI (1986) cold-formed steel specifications and developed in the seminal work by Winter (1970). For a plate with ideal simply-supported edge conditions, where the theoretical plate local buckling coefficient $k_c = 4.0$, the coefficients in Winter's equation for the plate effective width are $c_1 = 0.22$, $c_2 = 1.485$, and $c_3 = 0.0$. The coefficient $c_3 = 0.075$ for all other plates supported along two longitudinal edges that were not previously mentioned in Table 6.9.4.2.2b-1 shifts Winter's classical plate effective width curve to reflect the results of a wide range of research studies indicating lower local buckling resistances and postbuckling strengths in members composed of general welded plate assemblies, due to the influence of welding residual stresses (White and Lokhande, 2017).

Figure C6.9.4.2.2b-1 shows the compressive resistance, expressed in terms of an average stress on the gross area of the plate, F_n, relative to the minimum specified yield strength F_y, for a welded plate with F_y equal to 50 ksi supported along its two longitudinal edges, for the case where F_{cr} is taken equal to F_y in Eqs. 6.9.4.2.2b-1 and 6.9.4.2.2b-2. One can observe that these types of plates can develop approximately 80 percent of F_y when $b/t = 40$, 60 percent of F_y when $b/t = 60$, and 40 percent of F_y when $b/t = 90$. Traditional AASHTO guidance (AASHTO, 2002) suggested that the b/t of box girder compression flanges should not exceed 60, except in areas of low stress near points of dead load

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contraflexure. Figure C6.9.4.2.2b-1 provides more general guidance for preliminary design selection of longitudinally unstiffened plates. Given the magnitude of force that needs to be developed by a plate of a given width, and the yield strength of the plate, the strength curve in this figure can be employed to estimate the required plate thickness. Similar strength curves can be developed for other types of plates and longitudinal edge conditions.

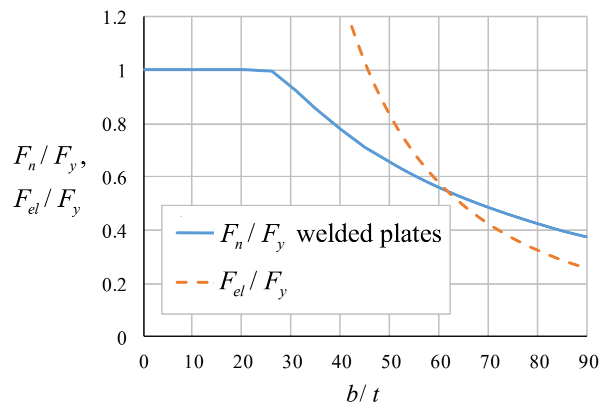

Figure C6.9.4.2.2b-1—Average Stress on the Gross Area of a Welded Plate (F_y = 50 ksi) Supported along Two Longitudinal Edges Relative to the Minimum Specified Yield Stress at the Ultimate Strength Condition, F_n/F_y, and for Theoretical Elastic Plate Buckling, F_{el}/F_y

When calculating the contribution of longitudinally unstiffened plates to the axial compressive resistance of noncomposite steel members, Eqs. 6.9.4.2.2b-1 and 6.9.4.2.2b-2 allow for an increased effectiveness of the plates in longer members where F_{cr} is reduced significantly relative to F_y. This is accomplished by using the unified effective width/unified effective area approach discussed in Article C6.9.4.2.2a.

When calculating the contribution of longitudinally unstiffened compression flange plates in noncomposite steel box-section flexural members, F_{cr} is taken equal to the specified minimum yield strength of the compression flange, F_{yc}, in Eqs. 6.9.4.2.2b-1 and 6.9.4.2.2b-2 based on the provisions of Article 6.12.2.2.2c. Also, when calculating the contribution of panels in longitudinally stiffened plates to a noncomposite member resistance in axial compression and/or flexure, F_{cr} in Eqs. 6.9.4.2.2b-1 and 6.9.4.2.2b-2 is taken equal to the specified minimum yield strength of the stiffened plate, F_{ysp}, based on the provisions of Article E6.1.3. The member resistance in these cases is best represented by considering the development of F_y on the effective area of the longitudinally unstiffened plates, and/or longitudinally stiffened plate panels.

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Table 6.9.4.2.2b-1—Effective Width Imperfection Adjustment Factors, c_1, c_2 and c_3

Slender Element	c_1	c_2	c_3
All Plates Supported along One Longitudinal Edge	0.22	1.49	0.0
Perforated Cover Plates	0.22	1.49	0.0
Webs of Rolled I- and Channel Sections; Webs of Nonwelded Built-Up I- and Channel Sections; Webs of Welded and Nonwelded Built-Up I-Sections Containing Two or More Longitudinal Stiffeners; and Flanges and Webs of Welded and Nonwelded Built-Up Box Sections Containing Two or More Longitudinal Stiffeners	0.18	1.31	0.0
Flanges and Webs of Nonwelded Built-Up Box Sections; Walls of Square and Rectangular Hot-Formed HSS; and Nonperforated Flange Cover Plates	0.20	1.38	0.0
Walls of Square and Rectangular Cold-Formed HSS	0.22	1.49	0.0
All Other Plates Supported along Two Longitudinal Edges	0.22	1.74	0.075

6.9.4.2.2c—Effective Area of Circular Tubes and Round HSS

The effective area, A_{eff}, of circular tubes and round HSS shall be determined as follows:

- If $\frac{D}{t} \leq 0.11\frac{E}{F_y}$, then:

$$A_{eff} = A_g \tag{6.9.4.2.2c-1}$$

- If $0.11\frac{E}{F_y} < \frac{D}{t} \leq 0.45\frac{E}{F_y}$, then:

$$A_{eff} = \left(\frac{0.038E}{F_y\left(D/t\right)} + \frac{2}{3}\right)A_g \tag{6.9.4.2.2c-2}$$

where:

A_g = gross cross-sectional area of the tube (in.^{2})
D = outside diameter of the tube (in.)
t = wall thickness of the tube (in.). For round HSS, the provisions of Article 6.12.1.2.4 shall apply.

C6.9.4.2.2c

An effective area is used for circular tubes and round HSS to account for the interaction between local and column buckling. The effective area is determined based on the ratio between the local buckling stress and the yield stress. The local buckling stress is taken from the AISI (2016) provisions based on inelastic action (Winter, 1970) and is based on tests conducted on fabricated and manufactured cylinders. Subsequent tests on fabricated cylinders confirm that Eq. 6.9.4.2.2c-2 is conservative (Ziemian, 2010).

6.9.4.3—Built-Up Members

6.9.4.3.1—General

C6.9.4.3.1

Two types of built-up members are commonly used for steel bridge construction: closely spaced steel shapes interconnected at intervals using welds or fasteners, and laced or battened members with widely spaced flange components.

The provisions of Article 6.9.4.2 shall apply. For built-up members composed of two or more shapes, the slenderness ratio of each component shape between connecting fasteners or welds shall not be more than 75 percent of the governing slenderness ratio of the built-up member. The least radius of gyration shall be used in computing the slenderness ratio of each component shape between the connectors.

Lacing, including flat bars, angles, channels, or other shapes employed as lacing, or batten plates shall be spaced so that the slenderness ratio of each component shape between the connectors shall not be more than 75 percent of the governing slenderness ratio of the built-up member.

The nominal compressive resistance of built-up members composed of two or more shapes shall be determined as specified in Article 6.9.4.1 subject to the following modification. If the buckling mode involves relative deformations that produce shear forces in the connectors between individual shapes, $K\ell/r$ shall be replaced by $(K\ell/r)_m$ determined as follows for intermediate connectors that are welded or fully-tensioned bolted:

$$\left(\frac{K\ell}{r}\right)_m = \sqrt{\left(\frac{K\ell}{r}\right)_o^2 + 0.82\left(\frac{\alpha^2}{1+\alpha^2}\right)\left(\frac{a}{r_{ib}}\right)^2} \qquad (6.9.4.3.1\text{-}1)$$

where:

$\left(\frac{K\ell}{r}\right)_m$ = modified slenderness ratio of the built-up member

$\left(\frac{K\ell}{r}\right)_o$ = slenderness ratio of the built-up member acting as a unit in the buckling direction being considered

a = distance between connectors (in.)

h = distance between centroids of individual component shapes perpendicular to the member axis of buckling (in.)

r_{ib} = radius of gyration of an individual component shape relative to its centroidal axis parallel to the member axis of buckling (in.)

α = separation ratio = $h/2r_{ib}$

The compressive resistance of built-up members is affected by the interaction between the global buckling mode of the member and the localized component buckling mode between lacing points or intermediate connectors. Duan, Reno, and Uang (2002) refer to this type of buckling as compound buckling. For both types of built-up members, limiting the slenderness ratio of each component shape between connection fasteners or welds or between lacing points, as applicable, to 75 percent of the governing global slenderness ratio of the built-up member effectively mitigates the effect of compound buckling (Duan, Reno, and Uang, 2002).

The compressive resistance of both types of members is also affected by any relative deformation that produces shear forces in the connectors between the individual shapes. Eq. 6.9.4.3.1-1 is adopted from AISC (2005) and provides a modified slenderness ratio taking into account the effect of the shear forces. Eq. 6.9.4.3.1-1 applies for intermediate connectors that are welded or fully-tensioned bolted and was derived from theory and verified by test data (Aslani and Goel, 1991). For other types of intermediate connectors on built-up members, including riveted connectors on existing bridges, Eq. C6.9.4.3.1-1 as follows should instead be applied:

$$\left(\frac{K\ell}{r}\right)_m = \sqrt{\left(\frac{K\ell}{r}\right)_o^2 + \left(\frac{a}{r_i}\right)^2} \qquad (C6.9.4.3.1\text{-}1)$$

where:

r_i = minimum radius of gyration of an individual component shape (in.)

Eq. C6.9.4.3.1-1 is based empirically on test results (Zandonini, 1985). In all cases, the connectors must be designed to resist the shear forces that develop in the buckled member.

Duan, Reno, and Lynch (2000) give an approach for determining the section properties of latticed built-up members, such as the moment of inertia and torsional constant. For additional guidance on the design of lacing and batten plates, refer to AISC (2016b).

6.9.4.3.2—Perforated Plates

Perforated plates shall satisfy the requirements of Articles 6.9.4.2 and 6.8.5.2 and shall be designed for the sum of the shear force due to the factored loads and an additional shear force taken as:

$$V = \frac{P_r}{100}\left(\frac{100}{(\ell/r) + 10} + \frac{8.8\,(\ell/r)\,F_y}{E}\right) \qquad (6.9.4.3.2\text{-}1)$$

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where:

V = additional shear force (kip)
P_r = factored compressive resistance specified in Articles 6.9.2.1 or 6.9.2.2 (kip)
ℓ = member length (in.)
r = radius of gyration about an axis perpendicular to the perforated plate (in.)
F_y = specified minimum yield strength (ksi)

In addition to checking the requirements of Article 6.9.4.2.1 for the clear distance between the two longitudinal edge supports of the perforated cover plate utilizing a width-to-thickness ratio limit λ_r of $1.86\sqrt{E/F_y}$, the requirements of Article 6.9.4.2.1 shall also separately be checked for the projecting width from the edge of the perforation to a single longitudinal edge support utilizing a width-to-thickness ratio limit λ_r of $0.45\sqrt{E/F_y}$.

6.9.4.4—Single-Angle Members

Single angles subject to combined axial compression and flexure about one or both principal axes and satisfying all of the following conditions, as applicable:

- End connections are to a single leg of the angle, and are welded or use a minimum of two bolts;
- The angle is loaded at the ends in compression through the same leg;
- The angle is not subjected to any intermediate transverse loads; and
- If used as web members in trusses, all adjacent web members are attached to the same side of the gusset plate or chord;

may be designed as axially loaded compression members for flexural buckling only according to the provisions of Articles 6.9.2.1, 6.9.4.1.1, and 6.9.4.1.2 provided the following effective slenderness ratio, $(K\ell/r)_{eff}$, is utilized in determining the nominal compressive resistance, P_n:

- For equal-leg angles and unequal-leg angles connected through the longer leg:
  - If $\frac{\ell}{r_x} \leq 80$, then:

$$\left(\frac{K\ell}{r}\right)_{eff} = 72 + 0.75\frac{\ell}{r_x} \qquad (6.9.4.4\text{-}1)$$

  - If $\frac{\ell}{r_x} > 80$, then:

C6.9.4.4

Single angles are commonly used as compression members in cross-frames and lateral bracing for steel bridges. Since the angle is typically connected through one leg only, the member is subject to combined axial compression and flexure, or moments about both principal axes due to the eccentricities of the applied axial load. The angle is also usually restrained by differing amounts about its geometric x- and y-axes. As a result, the prediction of the nominal compressive resistance of these members under these conditions is difficult. The provisions contained herein provide significantly simplified provisions for the design of single-angle members satisfying certain conditions that are subject to combined axial compression and flexure. These provisions are based on the provisions for the design of single-angle members used in latticed transmission towers (ASCE, 2000). Similar provisions are also employed in Section E5 of AISC (2016b).

In essence, these provisions permit the effect of the eccentricities to be neglected when these members are evaluated as axially loaded compression members for flexural buckling only using an appropriate specified effective slenderness ratio, $(K\ell/r)_{eff}$, in place of $(K\ell/r_s)$ in Eq. 6.9.4.1.2-1. The effective slenderness ratio indirectly accounts for the bending in the angles due to the eccentricity of the loading allowing the member to be proportioned according to the provisions of Article 6.9.2.1 as if it were a pinned-end concentrically loaded compression member. Furthermore, when the effective slenderness ratio is used, single angles need not be checked for flexural–torsional buckling. The actual maximum slenderness ratio of the angle, as opposed to $(K\ell/r)_{eff}$, is not to exceed the applicable limiting slenderness ratio specified in Article 6.9.3. Thus, if the

$$\left(\frac{K\ell}{r}\right)_{eff} = 32 + 1.25\frac{\ell}{r_x} \quad (6.9.4.4\text{-}2)$$

- For unequal-leg angles that are connected through the shorter leg with the ratio of the leg lengths less than 1.7:
  - If $\frac{\ell}{r_x} \leq 80$, then:

$$\left(\frac{K\ell}{r}\right)_{eff} = 72 + 0.75\frac{\ell}{r_x} + 4\left[\left(\frac{b_\ell}{b_s}\right)^2 - 1\right] \geq 0.95\frac{\ell}{r_z} \quad (6.9.4.4\text{-}3)$$

  - If $\frac{\ell}{r_x} > 80$, then:

$$\left(\frac{K\ell}{r}\right)_{eff} = 32 + 1.25\frac{\ell}{r_x} + 4\left[\left(\frac{b_\ell}{b_s}\right)^2 - 1\right] \geq 0.95\frac{\ell}{r_z} \quad (6.9.4.4\text{-}4)$$

where:

b_ℓ = length of the longer leg of an unequal-leg angle (in.)
b_s = length of the shorter leg of an unequal-leg angle (in.)
ℓ = distance between the work points of the joints measured along the length of the angle (in.)
r_x = radius of gyration about the geometric axis of the angle parallel to the connected leg (in.)
r_z = radius of gyration about the minor principal axis of the angle (in.)

The actual maximum slenderness ratio of the angle shall not exceed the applicable limiting slenderness ratio specified in Article 6.9.3. Single angles designed using $(K\ell/r)_{eff}$ shall not be checked for flexural–torsional buckling.

actual maximum slenderness ratio of the angle exceeds the limiting ratio, a larger angle section must be selected until the ratio is satisfied. If $(K\ell/r)_{eff}$ exceeds the limiting ratio, but the actual maximum slenderness ratio of the angle does not, the design is satisfactory. The limiting ratios specified in Article 6.9.3 are well below the limiting ratio of 200 specified in AISC (2016b).

The expressions for the effective slenderness ratio presume significant end rotational restraint about the y-axis, or the axis perpendicular to the connected leg and gusset plate, as shown in Figure C6.9.4.4-1.

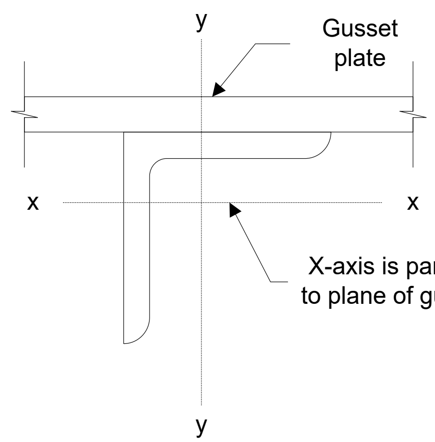

Figure C6.9.4.4-1—Single-angle Geometric Axes Utilized in the Effective Slenderness Ratio Expressions

As a result, the angle tends to buckle primarily about the x-axis due to the eccentricity of the load about the x-axis coupled with the high degree of restraint about the y-axis (Usami and Galambos, 1971; Woolcock and Kitipornchai, 1986; Mengelkoch and Yura, 2002). Therefore, the radius of gyration in the effective slenderness ratio expressions is to be taken as r_x, or the radius of gyration about the geometric axis parallel to the connected leg, and not the minimum radius of gyration r_z about the minor principal axis of the angle. When an angle has significant rotational restraint about the y-axis, the stress along the connected leg will be approximately uniform (Lutz, 1996). Lutz (2006) compared the results from the effective slenderness ratio equations contained herein to test results for single-angle members in compression with essentially pinned-end conditions (Foehl, 1948; Trahair et al., 1969) and found an average value of P_n/P_{test} of 0.998 with a coefficient of variation of 0.109. A separate set of equations provided in AISC (2016b), which assume a higher degree of x-axis rotational restraint and are thus intended for application only to single angles used as web members in box or space trusses, are not provided herein.

For the case of unequal-leg angles connected through the shorter leg, the limited available test data for this case gives lower capacities for comparable ℓ/r_x values than equal-leg angles (Lutz, 2006). Stiffening the shorter leg rotationally tends to force the buckling axis of the angle away from the x-axis and closer to the z-axis. Thus, $(K\ell/r)_{eff}$ for this case is modified by adding an additional term in Eqs. 6.9.4.4-3 and 6.9.4.4-4 along with a governing slenderness limit based on ℓ/r_z for slender unequal-leg angles. The upper limit on b_ℓ/b_s of 1.7 is based on the limits of the available physical tests. For an unequal-leg angle connected through the longer leg, note that r_x should be taken as the smaller value about the angle geometric axes, which is typically listed as r_y in AISC (2017).

Single-angle compression members not meeting one or more of the conditions required in this Article, or with leg length ratios b_ℓ/b_s greater than 1.7, should instead be evaluated for combined axial load and flexure as beam-columns according to Section H2 of AISC (2016b). In computing P_n for these cases, the end restraint conditions should be evaluated in calculating the effective length $K\ell$, with the in-plane effective length factor K taken equal to 1.0. When the effective length factors about both geometric axes have been computed, the procedures given in Lutz (1992) can be used to obtain a minimum effective radius of gyration for the angle. In determining whether the flexural–torsional buckling resistance of the angle needs to be considered in computing P_n, it is recommended that AISC (2000) be consulted. Also, it has been observed that the actual eccentricity in the angle is less than the distance from the centerline of the gusset if there is any restraint present about the x-axis (Lutz, 1998). In this instance, the eccentricity $\bar{y}$ may be reduced by $t/2$, where t is the thickness of the angle, as long as the angle is on one side of the chord or gusset plate (Woolcock and Kitipornchai, 1986). The nominal flexural resistance of the angle M_n for these cases should be determined according to the procedures given in Section F10 of AISC (2016b).

Single-angle members are often employed in X-type configurations in cross-frames. It has been suggested (ASCE, 2000) that for cases in such configurations, where one diagonal is in tension with a force not less than 20 percent of the force in the diagonal compression member, that the crossover or intersection point may be considered as a brace point for out-of-plane buckling. A different approach has been suggested for equally loaded compression and tension diagonals in X-type configurations in which all connections are welded (El-Tayem and Goel, 1986), which also assumes a significant level of restraint at the crossover point. While such approaches could potentially be utilized in determining the effective slenderness ratio, they have not yet received extensive validation and the assumed level of restraint may not actually be present in certain instances. For example, should the members be connected with only a single bolt at the crossover point, the necessary rotational restraint about the y-axis assumed in the effective slenderness ratio equations may not be present at that point. Thus, it is recommended herein in the interim that the effective slenderness ratio

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equations be conservatively applied to single-angle compression members used in X-type bracing configurations by using the full length of the diagonal between the connection work points for ℓ.

6.9.4.5—Plate Buckling under Service and Construction Loads

The provisions contained herein shall not apply at the service limit state or during construction for webs of:

- Composite or noncomposite I-section members subject to flexure only,
- Composite box-section members subject to flexure only, and
- Noncomposite box-section members subject to flexure only, containing longitudinally unstiffened webs or webs with only one longitudinal stiffener.

Such members shall be checked for web bend buckling at these limit states according to the applicable provisions of Articles 6.10 and 6.11, respectively, using the appropriate web bend buckling resistance, F_{crw}, specified in Article 6.10.1.9.

The provisions contained herein also shall not apply for plate elements supported only along one longitudinal edge and for walls of circular tubes.

All other

- Slender plate elements as defined in Article 6.9.4.2.2a, or
- Slender longitudinally stiffened plate panels as defined in Article E6.1.2

subjected to longitudinal compressive stress at one or both of their longitudinal edges shall satisfy the following at the service limit state and for constructibility:

$$f_c \leq \frac{0.9Ek}{\lambda^2} \leq F_y \qquad (6.9.4.5\text{-}1)$$

in which:

λ = slenderness, b_f/t_f, w/t_f, or D/t_w, of the slender flange or web plate element or longitudinally stiffened plate panel under consideration, as applicable

f_c = maximum longitudinal compressive stress (ksi) acting on the gross cross section in the plate element or longitudinally stiffened plate panel under consideration due to:

- The Service II loads;

C6.9.4.5

The checks specified herein are not required at the service limit state and for constructibility for plate elements supported only along one longitudinal edge, webs of I-section and box-section members specified herein that are subject to flexure only, or for the walls of circular tubes. The slenderness or plate buckling response of these types of elements is typically limited by other design provisions, which may be less restrictive in some cases, and/or the postbuckling response is not considered to be of any negative consequence.

Since significant post-buckling resistance is often assumed at the strength limit state in computing the nominal flexural and axial compressive resistance of members with cross sections containing slender plate elements supported along two longitudinal edges and/or longitudinally stiffened plates supported along their two longitudinal edges and containing slender plate panels, such members must satisfy the provisions of this article to ensure that theoretical local buckling of those plate elements or panels does not occur at the service limit state and for constructibility. Eq. 6.9.4.5-1 is used to check for theoretical local buckling at these limit states under the net combined normal stresses acting on each of these types of plate elements or panels subjected to compressive stress.

The influence of combined normal and shear stresses on theoretical plate buckling at the service limit state and for constructibility is not considered in these Specifications. Satisfaction of specified requirements to separately prevent theoretical buckling under normal stresses and under shear stresses is considered sufficient as an approximate technique to control plate bending strains and transverse displacements. In experimental tests, noticeable plate bending deformations and associated transverse displacements can occur from the onset of load application due to initial plate out-of-flatness. Because of stable plate postbuckling behavior, there is no significant change in the rate of increase of the transverse displacements as the theoretical plate buckling stress is exceeded. Due to unavoidable geometric

- The factored load for constructibility as specified in Article 3.4.2.1.

For noncomposite box-section members, the flange areas of the gross cross section shall be reduced to account for shear lag, as applicable, in the calculation of the stresses due to flexure, as specified in Article 6.12.2.2.2g.

k = plate-buckling coefficient considering any gradient in the longitudinal stress calculated as follows:

- If $1.0 \geq \frac{f_1}{f_2} \geq 0.0$, then:

$$k = \frac{8.2}{1.05 + \frac{f_1}{f_2}} \quad (6.9.4.5\text{-}2)$$

- If $0.0 > \frac{f_1}{f_2} \geq -1.0$, then:

$$k = 7.81 - 6.29\frac{f_1}{f_2} + 9.78\left(\frac{f_1}{f_2}\right)^2 \quad (6.9.4.5\text{-}3)$$

- If $-1.0 > \frac{f_1}{f_2} \geq -3.0$, then:

$$k = 5.98\left(1 - \frac{f_1}{f_2}\right)^2 \quad (6.9.4.5\text{-}4)$$

where:

b_f = total inside width between the plate elements providing lateral restraint to the longitudinal edges of the flange plate element under consideration (in.)

D = total inside width between the plate elements providing lateral restraint to the longitudinal

imperfections, the plate buckling behavior is a load-deflection problem rather than a bifurcation problem. Satisfaction of the specified requirements to prevent theoretical buckling helps limit the magnitude of the corresponding transverse displacements.

Plate local buckling is not checked at the fatigue limit state in these provisions because the plate buckling check under the Service II loads will tend to control over a similar check under the unfactored permanent load plus the Fatigue I load combination.

Any box-section member plate element subjected to stress due to bending about a principal axis of the box parallel to the plate is considered as a flange plate element for bending about that axis. Any member plate element subjected to flexural shear orthogonal to the axis of bending, and/or nominally linearly varying normal stresses due to bending about an axis normal to the face of the plate, is considered as a web plate element. It should be noted that in box-section members, the flange plate elements associated with one direction of bending are the web plate elements associated with the other direction of bending, and vice versa. Therefore, for members subjected to biaxial bending, the same elements are designed as a web plate element for bending in one direction, and as a flange plate element for bending in the other direction.

The plate-buckling coefficient, k, given by Eqs. 6.9.4.5-2 through 6.9.4.5-4 is from CEN (2006). These equations address general loading conditions from any combination of axial compression and bending within the plane of the plates for the evaluation of longitudinally unstiffened plates and longitudinally stiffened plate panels. They are based on the idealization of general plates and plate panels as having simply supported boundary edge conditions. The ratio f_1/f_2 is less than or equal to 1.0 in all cases, since f_1 is the smaller edge stress. The stress f_1 may be a smaller compressive value compared to f_2, or it may be a tensile stress, in which case f_1 is taken as a negative value. The ratio f_1/f_2 is positive when both f_1 and f_2 are in compression, and it is negative when f_1 is in tension. For unusual cases where the ratio of f_1/f_2 is smaller than -3, buckling of the plate is unlikely. In this case, it is conservative to use k equal to 96.

A plate-buckling coefficient of k = 4.0 may be employed conservatively in Eq. 6.9.4.5-1 as an initial design check. Web plate elements in noncomposite box-section members subjected predominantly to flexure often may require the larger k values from Eqs. 6.9.4.5-2 through 6.9.4.5-4 to satisfy the requirement of Eq. 6.9.4.5-1.

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edges of the web plate element under consideration (in.)

f_1 = smaller longitudinal stress at the longitudinal edges of the plate element or longitudinally stiffened plate panel at the cross-section under consideration, taken as positive in compression and negative in tension (ksi)

f_2 = larger longitudinal compressive stress at the longitudinal edges of the plate element or longitudinally stiffened plate panel at the cross section under consideration, taken as positive (ksi)

F_y = specified minimum yield strength of the plate element or longitudinally stiffened plate panel under consideration (ksi)

t_f = thickness of the flange plate element or longitudinally stiffened flange plate panel under consideration (in.)

t_w = thickness of the web plate element under consideration (in.)

w = width of the plate between the centerlines of the individual longitudinal stiffeners or between the centerline of the longitudinal stiffener and the inside of the laterally-restrained longitudinal edge of the longitudinally stiffened plate under consideration, as applicable (in.)

Refer to Figure CE6.1.3-1 for further information on the definition of the variable, w, for longitudinally stiffened plates.

For constructibility, the resistance side of Eq. 6.9.4.5-1 shall be multiplied by the resistance factor for flexure, ϕ_f, specified in Article 6.5.4.2.

6.9.5—Composite Members

6.9.5.1—Nominal Compressive Resistance

The provisions of this Article shall apply to composite columns without flexure. The provisions of Article 6.12.2.3 shall apply to composite columns in flexure. The provisions of Articles 6.9.6 and 6.12.2.3.3 provide an alternative for the design of composite concrete-filled steel tubes (CFSTs) subject to axial compression or combined axial compression and flexure.

The nominal compressive resistance of a composite column satisfying the provisions of Article 6.9.5.2 shall be taken as:

- If $\lambda \leq 2.25$, then:

$$P_n = 0.66^{\lambda} F_e A_s \quad (6.9.5.1\text{-}1)$$

- If $\lambda > 2.25$, then:

$$P_n = \frac{0.88 F_e A_s}{\lambda} \quad (6.9.5.1\text{-}2)$$

in which:

λ = normalized column slenderness factor

C6.9.5.1

The procedure for the design of composite columns is the same as that for the design of steel columns, except that the specified minimum yield strength of structural steel, the modulus of elasticity of steel, and the radius of gyration of the steel section are modified to account for the effect of concrete and of longitudinal reinforcing bars. Explanation of the origin of these modifications and comparison of the design procedure, with the results of numerous tests, may be found in SSRC Task Group 20 (1979) and Galambos and Chapuis (1980).

$$\lambda = \left(\frac{K\ell}{r_s \pi}\right)^2 \frac{F_e}{E_e} \qquad (6.9.5.1\text{-}3)$$

F_e = modified yield stress (ksi)

$$F_e = F_y + C_1 F_{yr}\left(\frac{A_r}{A_s}\right) + C_2 f'_c\left(\frac{A_c}{A_s}\right) \qquad (6.9.5.1\text{-}4)$$

E_e = modified modulus of elasticity (ksi)

$$E_e = E\left[1 + \left(\frac{C_3}{n}\right)\left(\frac{A_c}{A_s}\right)\right] \qquad (6.9.5.1\text{-}5)$$

where:

- A_s = cross-sectional area of the steel section (in.^{2})
- A_c = cross-sectional area of the concrete (in.^{2})
- A_r = total cross-sectional area of the longitudinal reinforcement (in.^{2})
- F_y = specified minimum yield strength of the steel section (ksi)
- F_{yr} = specified minimum yield strength of the longitudinal reinforcement (ksi)
- f'_c = specified minimum 28-day compressive strength of the concrete (ksi)
- E = modulus of elasticity of the steel (ksi)
- ℓ = unbraced length of the column (in.)
- K = effective length factor as specified in Article 4.6.2.5
- n = modular ratio of the concrete as specified in Article 6.10.1.1.1b
- r_s = radius of gyration of the steel section in the plane of bending but not less than 0.3 times the width of the composite member in the plane of bending for composite concrete-encased shapes (in.)
- C_1, C_2, C_3 = composite column constant specified in Table 6.9.5.1-1

Table 6.9.5.1-1—Composite Column Constants

	Filled Tubes	Encased Shapes
C_1	1.00	0.70
C_2	0.85	0.60
C_3	0.40	0.20

In determining the moment magnification for composite members subject to combined axial compression and flexure according to the approximate single step adjustment specified in Article 4.5.3.2.2b, the following shall apply:

$$P_e = \frac{A_s F_e}{\lambda} \qquad (6.9.5.1\text{-}6)$$

where:

P_e = Euler buckling load (kip)

6.9.5.2—Limitations

6.9.5.2.1—General

The compressive resistance shall be calculated in accordance with Article 6.9.5.1 if the cross-sectional area of the steel section comprises at least four percent of the total cross-sectional area of the member.

The compressive resistance shall be calculated as a reinforced concrete column under Section 5 if the cross-sectional area of the shape or tube is less than four percent of the total cross-sectional area.

The compressive strength of the concrete shall be between 3.0 ksi and 8.0 ksi.

The specified minimum yield strength of the steel section and the longitudinal reinforcement used to calculate the nominal compressive resistance shall not exceed 60.0 ksi.

The transfer of all load in the composite column shall be considered in the design of supporting components.

The cross section shall have at least one axis of symmetry.

C6.9.5.2.1

Little of the test data supporting the development of the present provisions for design of composite columns involved concrete strengths in excess of 6.0 ksi. Normal weight concrete was believed to have been used in all tests. A lower limit of 3.0 ksi is specified to encourage the use of good-quality concrete.

6.9.5.2.2—Concrete-Filled Tubes

The wall thickness requirements for unfilled tubes specified in Article 6.9.4.2 shall apply to filled composite tubes.

6.9.5.2.3—Concrete-Encased Shapes

Concrete-encased steel shapes shall be reinforced with longitudinal and lateral reinforcement. The reinforcement shall conform to the provisions of Article 5.6.4.6, except that the vertical spacing of lateral ties shall not exceed the least of:

- 16 longitudinal bar diameters,
- 48 tie bar diameters, or
- 0.5 of the least side dimension of the composite member.

Multiple steel shapes in the same cross section of a composite column shall be connected to one another with lacing and tie plates to prevent buckling of individual shapes before hardening of the concrete.

C6.9.5.2.3

Concrete-encased shapes are not subject to the width/thickness limitations specified in Article 6.9.4.2 because it has been shown that the concrete provides adequate support against local buckling.

6.9.6—Composite Concrete-Filled Steel Tubes (CFSTs)

6.9.6.1—General

The provisions of this Article, along with Article 6.12.2.3.3, provide an alternative method for the design of composite CFSTs with or without internal

C6.9.6.1

CFSTs are primarily used for piles, drilled shafts, piers or columns, and other structural members subject to significant compression only or significant compression reinforcement subject to axial compression or for combined axial compression and flexure. CFSTs should not be used as pure flexural members. CFSTs expected to develop full plastic hinging of the composite section as a result of a seismic event shall satisfy the provisions of the *Guide Specifications for LRFD Seismic Bridge Design* (AASHTO, 2011).

and flexure. The provisions for the design of composite CFSTs contained herein may be used as an alternative to the provisions of Articles 6.9.5 and 6.12.2.3.2, which are intended for applications where full composite action is not deemed necessary. Research demonstrates that Articles 6.9.5 and 6.12.2.3.2 do not assure full composite action (Robinson et al., 2012; Lehman and Roeder, 2012). Extensive research on composite CFSTs has been completed since the provisions of Article 6.9.5 were developed (Goode and Lam, 2011; Gourley et al., 2001). The provisions specified in Article 6.9.6 reflect the results of much of that research, reduce uncertainty and increase the accuracy of the prediction of the engineering properties of these members, and are similar to the AISC (2016b) CFST provisions. Seismic design examples demonstrating the application of these provisions are contained in Kenarangi and Bruneau (2017).

Experiments show that the resistance provided by the steel tube is much greater than that provided by internal reinforcement because the tube has a larger moment arm (Roeder and Lehman, 2012).

6.9.6.2—Limitations

The following requirements shall be satisfied:

- Circular steel tubes shall be used.
- Spiral welded tubes formed from coil steel, straight-seam welded tubes formed from flat plates, or seamless pipes shall be used.
- Steel tubes with straight seam welds shall be permitted for CFSTs for all applications where the outside diameter is 24.0 in. or less. Steel tubes with straight seam welds shall also be permitted for tubes larger than 24.0 in. in diameter. For straight seam tubes with diameters over 24.0 in., consideration should be given to the use of a low-shrinkage admixture to achieve a maximum of 0.04 percent shrinkage at 28 days, as tested in accordance with ASTM C157/C157M, Standard Test Method for Length Change of Hardened Hydraulic-Cement Mortar and Concrete, or to providing a shear transfer mechanism such as shear rings.
- The wall thickness of the steel tube shall satisfy:

$$\frac{D}{t} \leq 0.15 \frac{E}{F_{yst}} \qquad (6.9.6.2\text{-}1)$$

where:

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

C6.9.6.2

Circular CFSTs provide continuous confinement of the concrete, which is superior to that achieved with rectangular CFSTs. Rectangular steel tubes are not included in these provisions.

Large diameter tubes are required for most components in bridge applications. These are commonly formed by one of two methods. Coil steel may be unrolled in a helical fashion to form a spirally welded tube. The spiral welds are made as butt joints and formed from both the inside and outside of the tube by the double submerged arc process. The spiral welds are subjected to direct stresses under axial load and flexure, and therefore they are essential for developing CFST resistance. Good performance is assured if proper weld metal and processes are employed and the minimum tensile strength of the weld metal matches the yield strength of the steel tube. The welds may also be inspected by ultrasonic or radiographic methods over the entire length of the weld if increased quality control is needed. This spiral weld process is limited to tubes with wall thickness of about 1.0 in. or less and diameters greater than about 20.0 in. Minimum requirements for these welds are provided in ASTM A252/A252M and API Specification 5L as appropriate.

The limiting diameter-to-thickness ratio given by Eq. 6.9.6.2-1 is commonly used for CFSTs and is the design limit for a compact composite section as specified in AISC (2016b) for CFSTs. This limit has been shown to allow CFSTs to achieve the full plastic capacity while also providing substantial inelastic deformation capacity (Roeder, Lehman, and Bishop, 2010). CFST piles may require a larger thickness than that suggested by Eq. 6.9.6.2-1 as a result of driving requirements.

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- The specified minimum 28-day compressive strength of the concrete shall be the greater of 3.0 ksi and $0.075F_{yst}$.

The limits on the compressive strength of the concrete to the yield stress on the steel are required because research has shown that the benefits of composite action are reduced outside these limits.

Research shows that two mechanisms, mechanical transfer caused by slight projection of the spiral weld into the concrete fill and friction caused by contact stress between the steel and concrete, provide bond transfer between the steel tube and the concrete fill. For large diameter, straight seam tubes, there has been concern that shrinkage of the concrete could compromise the composite behavior although the research results are not definitive. The use of a low-shrinkage admixture is thought to enhance the bond capacity, particularly for larger diameter tubes. The detrimental effects of shrinkage would be less significant with smaller diameter tubes. In addition, binding action resulting from bending enhances bond shear stress transfer between the steel tube and the concrete fill. This binding action assures adequate shear transfer without any internal connectors.

Results of research (Kenarangi and Bruneau, 2017) indicate that composite action between the tube and concrete fill occurs under lateral loading even when the inner diameter surface of the tube was contaminated with mud and bentonite clay. Only a thick layer of grease was found to disrupt the connection sufficiently to prevent composite action at the strength limit state. For the case of large diameter straight seam tubes, where previous research had indicated less robust composite action, adding shear rings or similar positive shear transfer mechanisms provide an alternative to the use of low-shrinkage admixtures. Kenarangi and Bruneau (2017) provide design equations for shear rings.

6.9.6.3—Combined Axial Compression and Flexure

6.9.6.3.1—General

The axial compressive load, P_u, and concurrent moment, M_u, calculated for the factored loadings by elastic analytical procedures shall satisfy the factored stability-based P-M interaction relationship. The factored interaction resistance curve shall be developed by applying the resistance factor, ϕ_c, for combined axial compression and flexure in composite CFSTs specified in Article 6.5.4.2 to the nominal stability-based P-M interaction curve as specified in Article 6.9.6.3.4.

C6.9.6.3.1

The stability-based P-M interaction relationship can be determined by either the plastic stress distribution method (PSDM) or the strain compatibility method (SCM). The PSDM is an analysis method in which the nominal flexural composite resistance of the CFST in the presence of axial load is determined from a cross-sectional analysis using the constituent materials based on equilibrium at full plastification of the section, and is recommended herein. The computed resistance is then later adjusted for stability of the member through the development of a stability-based interaction curve.

An analysis of a database of experiments shows that using standard models, the strength prediction for composite CFSTs has a co-variance of 0.05 when axial loads are less than $0.6P_o$, where P_o is defined in Article 6.9.6.3.2. Further, circular CFST members have higher and more uniformly distributed confining stresses, which provide increased compressive strength and deformability of the fill concrete relative to reinforced concrete sections with discrete spiral confinement.

6.9.6.3.2—Axial Compressive Resistance

The factored resistance, P_r, of a composite CFST column subject to axial compression shall be determined as:

$$P_r = \phi_c P_n \quad (6.9.6.3.2\text{-}1)$$

where:

ϕ_c = resistance factor for axial compression and combined axial compression and flexure in composite CFSTs specified in Article 6.5.4.2
P_n = nominal compressive resistance (kip)

The nominal resistance, P_n, of a composite CFST column subject to axial compression shall be determined using Eqs. 6.9.6.3.2-2 through 6.9.6.3.2-7 as follows:

- If $P_e > 0.44P_o$, then:

$$P_n = \left[0.658^{\left(\frac{P_o}{P_e}\right)} \right] P_o \quad (6.9.6.3.2\text{-}2)$$

- If $P_e \leq 0.44P_o$, then:

$$P_n = 0.877 P_e \quad (6.9.6.3.2\text{-}3)$$

in which:

$$P_o = 0.95 f'_c A_c + F_{yst} A_{st} + F_{yb} A_{sb} \quad (6.9.6.3.2\text{-}4)$$

$$P_e = \frac{\pi^2 EI_{eff}}{(K\ell)^2} \quad (6.9.6.3.2\text{-}5)$$

$$EI_{eff} = EI_{st} + EI_{si} + C'E_c I_c \quad (6.9.6.3.2\text{-}6)$$

$$C' = 0.15 + \frac{P}{P_o} + \frac{A_{st} + A_{sb}}{A_{st} + A_{sb} + A_c} \leq 0.9 \quad (6.9.6.3.2\text{-}7)$$

where:

A_{st} = cross-sectional area of the steel tube (in.^{2})
A_{sb} = total cross-sectional area of the internal reinforcement bars (in.^{2})
A_c = net cross-sectional area of the concrete (in.^{2})
E_c = elastic modulus of the concrete (ksi)
E = elastic modulus of the steel tube and the internal steel reinforcement (ksi)

C6.9.6.3.2

The procedure for designing composite CFST columns subject to axial compression is similar to that used for the design of steel columns, except that a composite resistance and effective flexural stiffness, EI_{eff}, are employed. The effective flexural stiffness increases with increasing compressive load, which suppresses cracking in the concrete fill, and therefore, C' is a function of the axial load. The flexural stiffness values provided by Eqs. 6.9.6.3.2-6 and 6.9.6.3.2-7 correspond to approximately 90 percent of the maximum resistance of the member, which provides a conservative estimate of the buckling capacity. The flexural stiffness equations have been developed by comparison with past experimental results on composite CFSTs where the members have been loaded to loss of lateral load carrying capacity. Experiments show that this estimate is more accurate with smaller variance or standard deviation than that provided by other commonly used methods (Marson and Bruneau, 2004; Roeder, Lehman, and Bishop, 2010).

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EI_{eff} = effective composite flexural cross-sectional stiffness of the CFST (kip-in.^{2})
F_{yb} = specified minimum yield strength of the internal steel reinforcing bars (ksi)
F_{yst} = specified minimum yield strength of the steel tube (ksi)
f'_c = specified minimum 28-day compressive strength of the concrete (ksi)
I_c = uncracked moment of inertia of the concrete about the centroidal axis (in.^{4})
I_{si} = moment of inertia of the internal steel reinforcement about the centroidal axis (in.^{4})
I_{st} = moment of inertia of the steel tube about the centroidal axis (in.^{4})
K = effective length factor as specified in Article 4.6.2.5
l = unbraced length of the column (in.)
P = unfactored axial dead load (kip)
P_e = elastic critical buckling resistance for flexural buckling (kip)
P_n = nominal compressive resistance (kip)
P_o = compressive resistance of the column without consideration of buckling (kip)

6.9.6.3.3—Nominal Flexural Composite Resistance

The nominal flexural composite resistance, M_n, of circular CFSTs as a function of the nominal axial resistance, P_n, shall be determined as specified in Article 6.12.2.3.3.

6.9.6.3.4—Nominal Stability-Based Interaction Curve

The stability-based P-M interaction curve of CFSTs shall be constructed by joining points A', A'', D, and B, as illustrated in Figure 6.9.6.3.4-1, where:

- Point A corresponds to P_o, determined as specified in Article 6.9.6.3.2.
- Point A' corresponds to the axial compression resistance without moment, P_n, determined as specified in Article 6.9.6.3.2.
- Point A'' is the intersection of the material-based interaction curve determined as specified in Article C6.12.2.3.3 and a horizontal line through Point A'.
- Point B corresponds to the composite plastic moment resistance without an axial load, M_o, determined as specified in Article C6.12.2.3.3.
- Point C corresponds to the axial force, P_C, on the material-based interaction curve determined as specified in Article C6.12.2.3.3 that corresponds to the composite plastic moment resistance without axial load, M_o (Point B).
- Point D is located on the material-based interaction curve determined as specified in Article C6.12.2.3.3 and is taken as the axial load, P_D, determined as:

C6.9.6.3.4

The stability-based P-M interaction curve shown in Figure 6.9.6.3.4-1 includes stability effects and is a modified version of the PSDM material interaction curve shown in Figure C6.12.2.3.3-2, where the modification is based on the buckling load computed from Eqs. 6.9.6.3.2-2 or 6.9.6.3.2-3. This interaction curve is used for determining the nominal resistance of the composite CFST for combined axial compression and flexure for all load conditions (Moon et al., 2013).

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$$P_D = 0.5 P_C \frac{P_n}{P_o} \quad (6.9.6.3.4\text{-}1)$$

where P_n is determined as specified in Article 6.9.6.3.2.

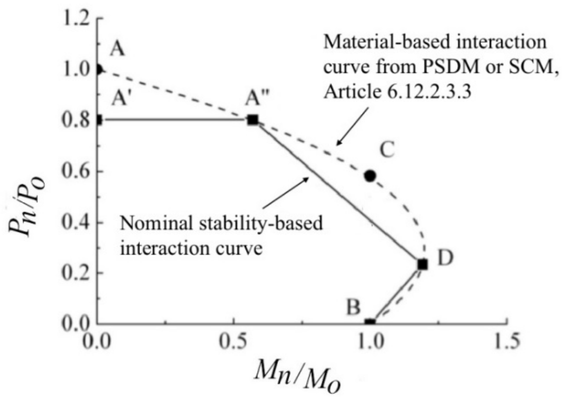

Figure 6.9.6.3.4-1—Construction of the Normalized Stability-based P-M Interaction Curve

6.10—I-SECTION FLEXURAL MEMBERS

6.10.1—General

The provisions of this Article apply to flexure of rolled or fabricated straight, kinked (chorded) continuous, or horizontally-curved steel I-section members symmetrical about the vertical axis in the plane of the web. These provisions cover the design of composite and noncomposite sections, hybrid and nonhybrid sections, and constant and variable web depth members as defined by and subject to the requirements of Articles 6.10.1.1 through 6.10.1.8. The provisions also cover the combined effects of major-axis bending and flange lateral bending from any source.

All types of I-section flexural members shall be designed as a minimum to satisfy:

- The cross-section proportion limits specified in Article 6.10.2;
- The constructibility requirements specified in Article 6.10.3;
- The service limit state requirements specified in Article 6.10.4;
- The fatigue and fracture limit state requirements specified in Article 6.10.5;
- The strength limit state requirements specified in Article 6.10.6.

The web bend-buckling resistance in slender web members shall be determined as specified in Article 6.10.1.9. Flange-strength reduction factors in hybrid and/or slender web members shall be determined as specified in Article 6.10.1.10.

Cross-frames and diaphragms for I-sections shall

C6.10.1

This Article addresses general topics that apply to all types of steel I-sections in either straight bridges, horizontally-curved bridges, or bridges containing both straight and curved segments. For the application of the provisions of Article 6.10, bridges containing both straight and curved segments are to be treated as horizontally-curved bridges since the effects of curvature on the support reactions and girder deflections, as well as the effects of flange lateral bending, usually extend beyond the curved segments. Note that kinked (chorded) girders exhibit the same actions as curved girders, except that the effect of the noncollinearity of the flanges is concentrated at the kinks. Continuous kinked (chorded) girders should be treated as horizontally-curved girders with respect to these Specifications.

The five bullet items in this Article indicate the overarching organization of the subsequent provisions for the design of straight I-section flexural members. Each of the subarticles throughout Article 6.10 are written such that they are largely self-contained, thus minimizing the need for reference to multiple Articles to address any one of the essential design considerations. For the strength limit state, Article 6.10.6 directs the Engineer to the subsequent Articles 6.10.7 through 6.10.12, and optionally for sections in straight I-girder bridges only, to Appendices A6 and B6, for the appropriate design requirements based on the type of I-section. The specific provisions of these Articles and Appendices are discussed in the corresponding Articles of the Commentary.

The provisions of Article 6.10 and the optional Appendices A6 and B6 provide a unified approach for consideration of combined major-axis bending and flange satisfy the provisions of Article 6.7.4. Where required, lateral bracing for I-sections shall satisfy the provisions of Article 6.7.5.

lateral bending from any source. For the majority of straight non-skewed bridges, flange lateral bending effects tend to be most significant during construction and tend to be insignificant in the final constructed condition. Significant flange lateral bending may be caused by wind, by torsion from eccentric concrete deck overhang loads acting on cantilever forming brackets placed along exterior girders, and by the use of discontinuous cross-frames, i.e., not forming a continuous line between multiple girders, in conjunction with skews exceeding 20 degrees. In these cases, the flange lateral bending may be considered at the discretion of the Engineer. Although the use of refined analysis methods is not required in order to fulfill the requirements of these provisions, these methods, when utilized, do allow for consideration of these effects. Some of these effects have not been addressed explicitly in previous Specifications. The intent of the Article 6.10 provisions is to permit the Engineer to consider flange lateral bending effects in the design in a direct and rational manner should they be judged to be significant. In the absence of calculated values of f_ℓ from a refined analysis, a suggested estimate for the total unfactored f_ℓ in a flange at a cross-frame or diaphragm due to the use of discontinuous cross-frame or diaphragm lines at or near supports, but not along the entire length of the bridge, is 10.0 ksi for interior girders and 7.5 ksi for exterior girders. These estimates are based on a limited examination of refined analysis results for bridges with skews approaching 60 degrees from normal and an average D/b_f ratio of approximately 4.0. In regions of the girders with contiguous cross-frames or diaphragms, these values need not be considered. Lateral flange bending in the exterior girders is substantially reduced when cross-frames or diaphragms are placed in discontinuous lines over the entire bridge due to the reduced cross-frame or diaphragm forces. A value of 2.0 ksi is suggested for f_ℓ for the exterior girders in such cases, with the suggested value of 10 ksi retained for the interior girders. In all cases, it is suggested that the recommended values of f_ℓ be proportioned to dead and live load in the same proportion as the unfactored major-axis dead and live load stresses at the section under consideration. An examination of cross-frame or diaphragm forces is also considered prudent in all bridges with skew angles exceeding 20 degrees. When all the above lateral bending effects are judged to be insignificant or incidental, the flange lateral bending term, f_ℓ, is simply set equal to zero in the appropriate equations. The format of the equations then reduces simply to the more conventional and familiar format for checking the nominal flexural resistance of I-sections in the absence of flange lateral bending.

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

White et al. (2012) presents one method of estimating I-girder flange lateral bending stresses in straight-skewed I-girder bridges and curved I-girder bridges with or without skew. This method is particularly useful to calculate these stresses when using a grid or plate and eccentric beam analysis since these results cannot be obtained directly from such analyses.

The fact that new design equations and provisions are provided herein does not imply that existing bridges are unsafe or structurally deficient. It also does not mandate the need to rehabilitate or perform a new load rating of existing structures to satisfy these provisions.

Flowcharts for flexural design of I-section members are provided in Appendix C6. Fundamental calculations for flexural members have been placed in Appendix D6.

6.10.1.1—Composite Sections

Sections consisting of a concrete deck that provides proven composite action and lateral support connected to a steel section by shear connectors designed according to the provisions of Article 6.10.10 shall be considered composite sections.

6.10.1.1.1—Stresses

6.10.1.1.1a—Sequence of Loading

The elastic stress at any location on the composite section due to the applied loads shall be the sum of the stresses caused by the loads applied separately to the:

- steel section,
- short-term composite section, and
- long-term composite section.

For unshored construction, permanent load applied before the concrete deck has hardened or is made composite shall be assumed carried by the steel section alone; permanent load and live load applied after this stage shall be assumed carried by the composite section. For shored construction, all permanent load shall be assumed applied after the concrete deck has hardened or has been made composite and the contract documents shall so indicate.

C6.10.1.1.1a

Previous Specifications indicated that a concrete slab may be considered sufficiently hardened after the concrete attains 75 percent of its specified 28-day compressive strength f'_c. Other indicators may be used based on the judgment of the Engineer.

While shored construction is permitted according to these provisions, its use is not recommended. There has been limited research on the effects of concrete creep on composite steel girders under large dead loads. Also, there have been only a very limited number of demonstration bridges built with shored construction in the U.S. Furthermore, there is an increased likelihood of significant tensile stresses occurring in the concrete deck at permanent interior supports of continuous spans when shored construction is used. These provisions may not be sufficient for shored construction where close tolerances on the girder cambers are important.

6.10.1.1.1b—Stresses for Sections in Positive Flexure

For calculating flexural stresses within sections subjected to positive flexure, the composite section shall consist of the steel section and the transformed area of the effective width of the concrete deck. Concrete on the tension side of the neutral axis shall not be considered effective at the strength limit state.

For transient loads assumed to be applied to the short-term composite section, the concrete deck area shall

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be transformed by using the short-term modular ratio, n. For permanent loads assumed applied to the long-term composite section, the concrete deck area shall be transformed by using the long-term modular ratio, $3n$. Where moments due to the transient and permanent loads are of opposite sign at the strength limit state, the associated composite section may be used with each of these moments if the resulting net stress in the concrete deck due to the sum of the unfactored moments is compressive. Otherwise, the provisions of Article 6.10.1.1.1c shall be used to determine the stresses in the steel section. Stresses in the concrete deck shall be determined as specified in Article 6.10.1.1.1d.

The modular ratio should be taken as:

$$n = \frac{E}{E_c} \tag{6.10.1.1.1b-1}$$

where:

E_c = modulus of elasticity of the concrete determined as specified in Article 5.4.2.4 (ksi)

6.10.1.1.1c—Stresses for Sections in Negative Flexure

For calculating flexural stresses in sections subjected to negative flexure, the composite section for both short-term and long-term moments shall consist of the steel section and the longitudinal reinforcement within the effective width of the concrete deck, except as specified otherwise in Article 6.6.1.2.1, Article 6.10.1.1.1d, or Article 6.10.4.2.1.

6.10.1.1.1d—Concrete Deck Stresses

For calculating longitudinal flexural stresses in the concrete deck due to all permanent and transient loads, the short-term modular ratio, n, shall be used.

C6.10.1.1.1d

Previous Specifications required that the longitudinal flexural stresses in the concrete deck due to permanent load be calculated using the n or the $3n$ section, whichever gives the more critical stress within the deck. When the deck stresses due to short-term and permanent loads are of the same sign, the n section generally governs the deck stress calculation. Also, the maximum combined compression in the deck typically occurs at a section where the permanent and short-term stresses are additive. However, when considering the length of the deck over which the provisions of Article 6.10.1.7 are to be applied, smaller compressive permanent load stresses can result in larger net tensile stresses in the deck in the vicinity of inflection point locations. In these situations, use of the $3n$ section for the permanent load stresses produces the more critical tension stress in the deck. This level of refinement in the calculation of the deck longitudinal tension stresses is considered unjustified.

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6.10.1.1.1e—Effective Width of Concrete Deck

The effective width of the concrete deck shall be determined as specified in Article 4.6.2.6.

6.10.1.2—Noncomposite Sections

Sections where the concrete deck is not connected to the steel section by shear connectors designed in accordance with the provisions of Article 6.10.10 shall be considered noncomposite sections.

C6.10.1.2

Noncomposite sections are not recommended, but are permitted.

6.10.1.3—Hybrid Sections

The specified minimum yield strength of the web should not be less than the larger of 70 percent of the specified minimum yield strength of the higher strength flange and 36.0 ksi.

For members with a higher-strength steel in the web than in one or both flanges, the yield strength of the web shall not be taken greater than 120 percent of the specified minimum yield strength of the lower strength flange in determining the flexural and shear resistance. Composite girders in positive flexure with a higher strength steel in the web than in the compression flange may use the full web strength in determining their flexural and shear resistance.

C6.10.1.3

Hybrid sections consisting of a web with a specified minimum yield strength lower than that of one or both of the flanges may be designed with these Specifications. Although these provisions can be safely applied to all types of hybrid sections (ASCE, 1968), it is recommended that the difference in the specified minimum yield strengths of the web and the higher strength flange preferably be limited to one steel grade. Such sections generally are believed to have greater design efficiency. For these types of sections, the upper limit of F_{yw} on the value of F_{yr}, determined in Articles 6.10.8.2.2, 6.10.8.2.3, A6.3.2 or A6.3.3 as applicable, does not govern. Furthermore, as discussed in Article C6.10.1.9.1, this minimum limit on the web yield strength guards against early inelastic web bend-buckling of slender hybrid webs.

A number of the curved noncomposite I-girders tested by Mozer and Culver (1970) and Mozer et al. (1971) had F_{yw}/F_{yf} between 0.72 and 0.76. The flexural and shear strengths of these hybrid I-girders are predicted adequately by these Specifications, including the development of shear strengths associated with tension field action. The major-axis bending stresses tend to be smaller in curved I-girder webs compared to straight I-girder webs, since part of the flexural resistance is taken up by flange lateral bending. The provisions of Articles 6.10.2 and 6.10.5.3 prevent significant out-of-plane flexing of the web in straight and curved hybrid I-girders (Yen and Mueller, 1966; ASCE, 1968).

Test data for sections with nominally larger yield strengths in the web than in one or both flanges are limited. Nevertheless, in many experimental tests, the actual yield strength of the thinner web is larger than that of the flanges. The nominal yield strength that may be used for the web in determining the flexural and shear resistance for such cases is limited within these Specifications to a range supported by the available test data.

6.10.1.4—Variable Web Depth Members

The effect of bottom flange inclination shall be considered in determining the bottom flange stress caused by bending about the major-axis of the cross section, and

C6.10.1.4

If the normal stress in an inclined bottom flange, calculated without consideration of flange lateral bending, is determined by simply dividing the bending any potential modifications to the vertical web shear. In cases where static equilibrium permits the vertical web shear to be reduced in variable web depth members, only the web dead-load shear may be reduced by the vertical component of the bottom flange force.

At points where the bottom flange becomes horizontal, full- or partial-depth transverse stiffening of the web shall be provided, unless the provisions of Article D6.5.2 are satisfied for the factored vertical component of the inclined flange force using a length of bearing N equal to zero.

moment about the major-axis of the cross section by the elastic section modulus, this stress is generally underestimated. The normal stress within an inclined bottom flange may be determined by first calculating the horizontal component of the flange force required to develop this bending moment as:

$$P_h = MA_f / S_x \qquad \text{(C6.10.1.4-1)}$$

where:

A_f = area of the inclined bottom flange (in.^{2})
M = bending moment about the major-axis of the cross section at the section under consideration (kip-in.)
S_x = elastic section modulus to the inclined bottom flange (in.^{3})

For composite sections, the provisions of Article 6.10.1.1.1a are to be applied in computing P_h. The normal stress in the inclined flange, f_n, may then be determined as (Blodgett, 1982):

$$f_n = P_h / A_f \cos\theta \qquad \text{(C6.10.1.4-2)}$$

where:

θ = angle of inclination of the bottom flange (degrees)

The corresponding vertical component of the flange force, P_v, may be determined as:

$$P_v = P_h \tan\theta \qquad \text{(C6.10.1.4-3)}$$

This component of the flange force affects the vertical web shear. In regions of positive flexure with tapered or parabolic haunches sloping downward toward the supports, the vertical web shear is increased by P_v. For fish belly haunches, $P_v = 0$ near interior supports because the slope of the bottom flange is small in that area. For all other cases, the vertical web shear is reduced by P_v. In cases where the vertical web shear is reduced, the Specifications permit the Engineer to reduce the web dead-load shear accordingly. Calculation of the reduced live-load shear is problematic because numerous sets of concurrent moments and shears must be evaluated in order to determine the critical or smallest shear reduction, and thus is not likely worth the effort. Also, variable depth webs are used most often on longer-span girders where dead load is more predominant. The total modified vertical web shear may be used in the design of the sloping flange-to-web welds.

In fish belly haunches, where the slope of the bottom flange is smaller at positions closer to the interior support, the convex bottom flange in compression produces a uniformly distributed radial tensile stress on the web. In

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parabolic haunches, where the downward slope of the bottom flange is larger at positions closer to the interior support, the concave bottom flange in compression produces a uniformly distributed radial compressive stress on the web. The magnitude of the radial stress in each case is dependent on the radius of curvature of the flange. Blodgett (1982) provides a rational approach for computing and evaluating the effect of the combined stresses on the web, which typically are not of significant concern unless the radius of curvature of the flange is unusually sharp. If the girder web is unstiffened or transversely-stiffened with a stiffener spacing d_o greater than approximately 1.5D within a parabolic haunch adjacent to an interior support, the Engineer should consider checking the stability of the web under the effect of the radial compressive force.

At points where an inclined bottom flange becomes horizontal, the vertical component of the inclined flange force is transferred back into the web as a concentrated load. This concentrated load causes additional stress in the web, and therefore, full- or partial-depth stiffening of the web must be provided at these points, except as discussed below. Full-depth stiffeners should be positively attached to both flanges and partial-depth stiffeners should be positively attached to the bottom flange. At these locations, the web is sufficient without additional stiffening if the provisions of Article D6.5.2 are satisfied for the factored vertical component of the inclined flange force using a length of bearing N equal to zero. At locations where the concentrated load is compressive and N is equal to zero, the provisions of Article D6.5.2 generally govern relative to those of Article D6.5.3; therefore, satisfaction of the provisions of Article D6.5.2 using a length of bearing N equal to zero ensures that the web is adequate without additional stiffening for locations subjected to compressive or tensile concentrated transverse loads.

6.10.1.5—Stiffness

The following stiffness properties shall be used in the analysis of flexural members:

- **For loads applied to noncomposite sections:** the stiffness properties of the steel section alone.
- **For permanent loads applied to composite sections:** the stiffness properties of the long-term composite section, assuming the concrete deck to be effective over the entire span length.
- **For transient loads applied to composite sections:** the stiffness properties of the short-term composite section, assuming the concrete deck to be effective over the entire span length.

The requirement for the modeling of girder torsional stiffness in curved and/or skewed I-girder bridges specified in Article 4.6.3.3.2 shall be satisfied for grid

C6.10.1.5

In line with common practice, it is specified that the stiffness of the steel section alone be used for noncomposite sections, although numerous field tests have shown that considerable unintended composite action occurs in such sections.

Field tests of composite continuous bridges have shown that there is considerable composite action in negative bending regions (Baldwin et al., 1978; Roeder and Eltvik, 1985; Yen et al., 1995). Therefore, the stiffness of the full composite section is to be used over the entire bridge length for the analysis of composite flexural members.

analyses or plate and eccentric beam analyses of steel I-girder bridges.

6.10.1.6—Flange Stresses and Member Bending Moments

For design checks where the flexural resistance is based on lateral–torsional buckling:

- The stress f_{bu} shall be determined as the largest value of the compressive stress throughout the unbraced length in the flange under consideration, calculated without consideration of flange lateral bending.
- The moment M_u shall be determined as the largest value of the major-axis bending moment throughout the unbraced length causing compression in the flange under consideration.
- The stress f_ℓ shall be determined as the largest value of the stress due to lateral bending throughout the unbraced length in the flange under consideration.

For design checks where the flexural resistance is based on yielding, flange local buckling or web bend-buckling, f_{bu}, M_u, and f_ℓ may be determined as the corresponding values at the section under consideration.

The values of f_{bu}, M_u, and f_ℓ shall be determined based on factored loads, and shall be taken as positive in sign in all resistance equations.

Lateral bending stresses in continuously braced flanges shall be taken equal to zero. Lateral bending stresses in discretely braced flanges shall be determined by structural analysis. All discretely braced flanges shall satisfy:

$$f_\ell \leq 0.6F_{yf} \tag{6.10.1.6-1}$$

The flange lateral bending stress, f_ℓ, may be determined directly from first-order elastic analysis in discretely braced compression flanges for which:

$$L_b \leq 1.2L_p\sqrt{\frac{C_bR_b}{f_{bu}/F_{yc}}} \tag{6.10.1.6-2}$$

or equivalently:

$$L_b \leq 1.2L_p\sqrt{\frac{C_bR_b}{M_u/M_{yc}}} \tag{6.10.1.6-3}$$

where:

C6.10.1.6

For checking of lateral–torsional buckling resistance, the correct value of the stress f_{bu} or moment M_u is generally the largest value causing compression in the flange under consideration throughout the unbraced length.

For a discretely braced compression flange also subject to lateral bending, the largest lateral bending stress throughout the unbraced length of the flange under consideration must be used in combination with f_{bu} or M_u when the resistance is based on lateral–torsional buckling. Combined vertical and flange lateral bending is addressed in these Specifications by effectively handling the flanges as equivalent beam-columns. The use of the maximum f_ℓ and f_{bu} or M_u values within the unbraced length, when the resistance is governed by member stability, i.e., lateral–torsional buckling, is consistent with established practice in the proper application of beam-column interaction equations.

Yielding, flange local buckling, and web bend-buckling are considered as cross-section limit states. Hence, the Engineer is allowed to use coincident cross-section values of f_ℓ and f_{bu} or M_u when checking these limit states. Generally, this approach necessitates checking of the limit states at various cross sections along the unbraced length. When the maximum values of f_ℓ and f_{bu} or M_u occur at different locations within the unbraced length, it is conservative to use the maximum values in a single application of the yielding and flange local buckling equations. Flange lateral bending does not enter into the web bend-buckling resistance equations.

In lieu of a more refined analysis, Article C6.10.3.4.1 gives approximate equations for calculation of the maximum flange lateral bending moments due to eccentric concrete deck overhang loads acting on cantilever forming brackets placed along exterior members. Determination of flange wind moments is addressed in Article 4.6.2.7. The determination of flange lateral bending moments due to the effect of discontinuous cross-frames and/or support skew is best handled by a direct structural analysis of the bridge superstructure. The determination of flange lateral bending moments due to curvature is addressed in Article 4.6.1.2.4b.

In all resistance equations, f_{bu}, M_u, and f_ℓ are to be taken as positive in sign. However, when summing dead and live load stresses or moments to obtain the total factored major-axis stresses or moments, f_{bu} or M_u, and total factored lateral bending stresses, f_ℓ, to apply in the equations, the signs of the individual stresses or moments must be considered. Where a dead load effect reduces the magnitude of the net force effect at the strength limit

C_b = moment gradient modifier specified in Article 6.10.8.2.3 or Article A6.3.3, as applicable.

f_{bu} = largest value of the compressive stress throughout the unbraced length in the flange under consideration, calculated without consideration of flange lateral bending (ksi)

L_b = unbraced length (in.)

L_p = limiting unbraced length specified in Article 6.10.8.2.3 (in.)

M_u = largest value of the major-axis bending moment throughout the unbraced length causing compression in the flange under consideration (kip-in.)

M_{yc} = yield moment with respect to the compression flange determined as specified in Article D6.2 (kip-in.)

R_b = web load-shedding factor determined as specified in Article 6.10.1.10.2

If Eq. 6.10.1.6-2, or Eq. 6.10.1.6-3 as applicable, is not satisfied, second-order elastic compression flange lateral bending stresses shall be determined.

Second-order compression flange lateral bending stresses may be approximated by amplifying first-order values as follows:

$$f_\ell = \left(\frac{0.85}{1 - \dfrac{f_{bu}}{F_{cr}}} \right) f_{\ell 1} \geq f_{\ell 1} \tag{6.10.1.6-4}$$

or equivalently:

$$f_\ell = \left(\frac{0.85}{1 - \dfrac{M_u}{F_{cr} S_{xc}}} \right) f_{\ell 1} \geq f_{\ell 1} \tag{6.10.1.6-5}$$

where:

F_{yc} = specified minimum yield strength of the compression flange (ksi)

f_{bu} = largest value of the compressive stress throughout the unbraced length in the flange under consideration, calculated without consideration of flange lateral bending (ksi)

$f_{\ell 1}$ = first-order compression flange lateral bending stress at the section under consideration, or the maximum first-order lateral bending stress in the compression flange under consideration throughout the unbraced length, as applicable (ksi)

F_{cr} = elastic lateral–torsional buckling stress for the flange under consideration determined from Eq. 6.10.8.2.3-8 or Eq. A6.3.3-8. Eq. A6.3.3-8

state, the minimum load factor specified in Table 3.4.1-2 and the corresponding load modifier, η_i, specified in Article 1.3.2.1 should be applied to that dead load effect.

The top flange may be considered continuously braced where it is encased in concrete or anchored to the deck by shear connectors satisfying the provisions of Article 6.10.10. For a continuously braced flange in tension or compression, flange lateral bending effects need not be considered. Additional lateral bending stresses are small once the concrete deck has been placed. Lateral bending stresses induced in a continuously braced flange prior to this stage need not be considered after the deck has been placed. The resistance of the composite concrete deck is generally adequate to compensate for the neglect of these initial lateral bending stresses. The Engineer should consider the non-composite lateral bending stresses in the top flange if the flange is not continuously supported by the deck.

The provisions of Article 6.10 for handling of combined vertical and flange lateral bending are limited to I-sections that are loaded predominantly in major-axis bending. For cases in which the elastically computed flange lateral bending stress is larger than approximately $0.6F_{yf}$, the reduction in the major-axis bending resistance due to flange lateral bending tends to be greater than that determined based on these provisions. The service and strength limit state provisions of these Specifications are sufficient to ensure acceptable performance of I-girders with elastically computed f_ℓ values somewhat larger than this limit.

Eq. 6.10.1.6-2, or equivalently Eq. 6.10.1.6-3 as applicable, gives a maximum value of L_b for which $f_\ell = f_{\ell 1}$ in Eq. 6.10.1.6-4 or 6.10.1.6-5. Eq. 6.10.1.6-4, or equivalently Eq. 6.10.1.6-5 as applicable, is an approximate formula that accounts for the amplification of the first-order compression flange lateral bending stresses due to second-order effects. Eqs. 6.10.1.6-4 and 6.10.1.6-5 are established forms for estimating the maximum second-order elastic moments in braced beam-column members whose ends are restrained by other framing and tend to be significantly conservative for larger unbraced lengths associated with f_{bu} approaching F_{cr} (White et al., 2001). This conservatism exists even when an effective length factor for lateral–torsional buckling and/or a moment gradient factor C_b is considered in the calculation of F_{cr}, and even when one end of the unbraced segment under consideration is not restrained by an adjacent segment. Although Eqs. 6.10.1.6-4 and 6.10.1.6-5 are directed at estimating the maximum second-order lateral bending stress within the unbraced length, by use of the maximum first-order lateral bending stress for $f_{\ell 1}$, they may be applied for estimating the second-order lateral bending stresses at any cross section within the unbraced length under consideration by use of the corresponding value of $f_{\ell 1}$ at that location.

The purpose of Eqs. 6.10.1.6-4 and 6.10.1.6-5 is to guard conservatively against large unbraced lengths in may only be applied for unbraced lengths in straight I-girder bridges in which the web is compact or noncompact.

M_u = largest value of the major-axis bending moment throughout the unbraced length causing compression in the flange under consideration (kip-in.)

S_{xc} = elastic section modulus about the major axis of the section to the compression flange taken as M_{yc}/F_{yc} (in.^{3})

which the flange second-order lateral bending effects are significant. In construction situations where the amplification within these equations is large, the Engineer may wish to consider a direct geometric nonlinear analysis to more accurately determine the second-order effects within the superstructure, or using a lower value of the effective length factor for lateral–torsional buckling to appropriately increase F_{cr} according to the procedure suggested in Article C6.10.8.2.3.

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

Note that the calculated value of F_{cr} for use in Eq. 6.10.1.6-4 is not limited to $R_bR_hF_{yc}$ as specified in Article 6.10.8.2.3, and that the calculated value of $F_{cr}S_{xc}$ for use in Eq. 6.10.1.6-5 is not limited to $R_{pc}M_{yc}$ as specified in Article A6.3.3. The elastic buckling stress is the appropriate stress for use in Eqs. 6.10.1.6-4 and 6.10.1.6-5 to estimate the elastic second-order amplification of the flange lateral bending stresses.

The definitions of a compact web and of a noncompact web are discussed in Article C6.10.6.2.3.

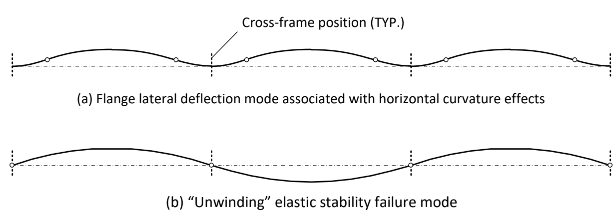

Figure C6.10.1.6-1—Second-order Elastic Lateral Deflections due to Horizontal Curvature Effects versus the Unwinding Stability Failure Mode of the Compression flange

6.10.1.7—Minimum Negative Flexure Concrete Deck Reinforcement

Wherever the longitudinal tensile stress in the concrete deck due to either the factored construction loads or Load Combination Service II in Table 3.4.1-1 exceeds ϕf_r, the total cross-sectional area of the

C6.10.1.7

The use of one percent reinforcement with a size not exceeding No. 6 bars, a yield strength greater than or equal to 60.0 ksi, and spacing at intervals not exceeding 12.0 in. is intended to control concrete deck cracking.

longitudinal reinforcement shall not be less than one percent of the total cross-sectional area of the concrete deck. ϕ shall be taken as 0.9 and f_r shall be taken as the modulus of rupture of the concrete determined as follows:

- **For normal-weight concrete:** $f_r = 0.24\sqrt{f_c'}$
- **For lightweight concrete:** f_r is calculated as specified in Article 5.4.2.6,

The longitudinal stresses in the concrete deck shall be determined as specified in Article 6.10.1.1.1d. The reinforcement used to satisfy this requirement shall have a specified minimum yield strength not less than 60.0 ksi; the size of the reinforcement should not exceed No. 6 bars.

The required reinforcement should be placed in two layers uniformly distributed across the deck width, and two-thirds should be placed in the top layer. The individual bars should be spaced at intervals not exceeding 12.0 in.

Where shear connectors are omitted from the negative flexure region, all longitudinal reinforcement shall be extended into the positive flexure region beyond the additional shear connectors specified in Article 6.10.10.3 a distance not less than the development length specified in Section 5.

Pertinent criteria for concrete crack control are discussed in more detail in AASHTO (1991) and in Haaijer et al. (1987).

Previously, the requirement for one percent longitudinal reinforcement was limited to negative flexure regions of continuous spans, which are often implicitly taken as the regions between points of dead load contraflexure. Under moving live loads, the deck can experience significant tensile stresses outside the points of dead load contraflexure. Placement of the concrete deck in stages can also produce negative flexure during construction in regions where the deck already has been placed, although these regions may be subjected primarily to positive flexure in the final condition. Thermal and shrinkage strains can also cause tensile stresses in the deck in regions where such stresses otherwise might not be anticipated. To address these issues, the one percent longitudinal reinforcement is to be placed wherever the tensile stress in the deck due to either the factored construction loads, including loads during the various phases of the deck placement sequence, or due to Load Combination Service II in Table 3.4.1-1, exceeds ϕf_r. By satisfying the provisions of this Article to control the crack size in regions where adequate shear connection is also provided, the concrete deck may be considered to be effective in tension for computing fatigue stress ranges, as permitted in Article 6.6.1.2.1, and in determining flexural stresses on the composite section due to Load Combination Service II, as permitted in Article 6.10.4.2.1.

In addition to providing one percent longitudinal deck reinforcement, nominal yielding of this reinforcement should be prevented at Load Combination Service II (Carskaddan, 1980; AASHTO, 1991; Grubb, 1993) to control concrete deck cracking. The use of longitudinal deck reinforcement with a specified minimum yield strength not less than 60.0 ksi may be taken to preclude nominal yielding of the longitudinal reinforcement under this load combination in the following cases:

- unshored construction where the steel section utilizes steel with a specified minimum yield strength less than or equal to 70.0 ksi in either flange, or
- shored construction where the steel section utilizes steel with a specified minimum yield strength less than or equal to 50.0 ksi in either flange.

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In these cases, the effects of any nominal yielding within the longitudinal reinforcing steel are judged to be insignificant. Otherwise, the Engineer should check to ensure that nominal yielding of the longitudinal reinforcement does not occur under the applicable Service II loads. The above rules are based on Carskaddan (1980) and apply for members that are designed by the provisions of Article 6.10 or Appendix A6, as well as for members that are designed for redistribution of the pier section moments at the Service II Load Combination using the provisions of Appendix B6.

Where feasible, approximately two-thirds of the required reinforcement should be placed in the top layer. When precast deck panels are used as deck forms, it may not be possible to place the longitudinal reinforcement in two layers. In such cases, the placement requirements may be waived at the discretion of the Engineer.

6.10.1.8—Tension Flanges with Holes

When checking flexural members at the strength limit state or for constructibility, the following additional requirement shall be satisfied at all cross sections containing holes in the tension flange:

$$f_t \leq 0.84\left(\frac{A_n}{A_g}\right)F_u \leq F_{yt} \quad (6.10.1.8\text{-}1)$$

where:

A_n = net area of the tension flange determined as specified in Article 6.8.3 (in.^{2})
A_g = gross area of the tension flange (in.^{2})
f_t = stress on the gross area of the tension flange due to the factored loads calculated without consideration of flange lateral bending (ksi)
F_u = specified minimum tensile strength of the tension flange determined as specified in Table 6.4.1-1 (ksi)

C6.10.1.8

Eq. 6.10.1.8-1 provides a limit on the maximum major-axis bending stress permitted on the gross section of the girder, neglecting the loss of area due to holes in the tension flange. This equation is used in lieu of the 15 percent rule in AASHTO *Standard Specifications* (2002), which allows holes with an area less than or equal to 15 percent of the gross area of the flange to be neglected. For higher-strength steels, with a higher yield-to-ultimate strength ratio than Grade 36 steel, the 15 percent rule is not valid; such steels are better handled using Eq. 6.10.1.8-1. For holes larger than those typically used for connectors such as bolts, refer to Article 6.8.1.

At compact composite sections in positive flexure and at sections designed according to the optional provisions of Appendix A6 with no holes in the tension flange, the nominal flexural resistance is permitted to exceed the moment at first yield at the strength limit state. Pending the results from further research, it is conservatively required that Eq. 6.10.1.8-1 also be satisfied at the strength limit state at any such cross sections containing holes in the tension flange. It has not yet been fully documented that complete plastification of the cross section can occur at these sections prior to fracture on the net section of the tension flange. Furthermore, the splice design provisions of Article 6.13.6.1.3 do not consider the contribution of substantial web yielding to the flexural resistance of these sections. Eq. 6.10.1.8-1 will likely prevent holes from being located in the tension flange at or near points of maximum applied moment where significant yielding of the web, beyond the localized yielding permitted in hybrid sections, may occur.

The factor 0.84 in Eq. 6.10.1.8-1 is approximately equivalent to the ratio of the resistance factor for fracture of tension members, ϕ_u, to the resistance factor for yielding of tension members, ϕ_y, specified in Article 6.5.4.2.

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6.10.1.9—Web Bend-Buckling Resistance

6.10.1.9.1—Webs without Longitudinal Stiffeners

The nominal bend-buckling resistance shall be taken as:

$$F_{crw} = \frac{0.9Ek}{\left(\frac{D}{t_w}\right)^2} \quad (6.10.1.9.1\text{-}1)$$

but not to exceed the smaller of R_hF_{yc} and $F_{yw}/0.7$

in which:

k = bend-buckling coefficient

$$= \frac{9}{(D_c/D)^2} \quad (6.10.1.9.1\text{-}2)$$

where:

D_c = depth of the web in compression in the elastic range (in.). For composite sections, D_c shall be determined as specified in Article D6.3.1.
D = web depth (in.)
R_h = hybrid factor specified in Article 6.10.1.10.1

When both edges of the web are in compression, k shall be taken as 7.2.

C6.10.1.9.1

In subsequent Articles, the web theoretical bend-buckling resistance is checked generally against the maximum compression flange stress due to the factored loads, calculated without consideration of flange lateral bending. The precision associated with making a distinction between the stress in the compression flange and the maximum compressive stress in the web is not warranted. The potential use of a value of F_{crw} greater than the specified minimum yield strength of the web, F_{yw}, in hybrid sections is justified since the flange tends to restrain the longitudinal strains associated with web bend-buckling for nominal compression flange stresses up to R_hF_{yc}. A stable nominally elastic compression flange constrains the longitudinal and plate bending strains in the inelastic web at the web-flange juncture (ASCE, 1968). ASCE (1968) recommends that web bend-buckling does not need to be considered in hybrid sections with F_{yc} up to 100 ksi as long as the web slenderness does not exceed $5.87\sqrt{E/F_{yc}}$. Eq. 6.10.1.9.1-1 predicts $F_{crw} = F_{yc}$ at $2D_c/t_w = 5.7\sqrt{E/F_{yc}}$. For hybrid sections with $F_{yw}/F_{yc} < 0.7$, these provisions adopt a more conservative approach than recommended by ASCE (1968) by limiting F_{crw} to the smaller of R_hF_{yc} and $F_{yw}/0.7$. The flexural resistance equations of these Specifications give somewhat conservative predictions for the strengths of hybrid members without longitudinal stiffeners tested by Lew and Toprac (1968) that had D/t_w and $2D_c/t_w$ values as high as 305 and $F_{yw}/F_{yc} = 0.32$. Therefore, no additional requirements are necessary at the strength limit state for all potential values of F_{yw}/F_{yc} associated with the steels specified in Article 6.4.1.

In many experimental tests, noticeable web plate bending deformations and associated transverse displacements occur from the onset of load application due to initial web out-of-flatness. Because of the stable postbuckling behavior of the web, there is no significant change in the rate of increase of the web transverse displacements as a function of the applied loads as the theoretical web bend-buckling stress is exceeded (Basler et al., 1960). Due to unavoidable geometric imperfections, the web bend-buckling behavior is a load-deflection rather than a bifurcation problem. The theoretical web-buckling load is used in these Specifications as a simple index for controlling the web plate bending strains and transverse displacements.

For a doubly-symmetric I-section without longitudinal web stiffeners, Eq. 6.10.1.9.1-2 gives k = 36.0, which is approximately equal to $k_{ss} + 0.8(k_{sf} - k_{ss})$, where k_{ss} = 23.9 and k_{sf} = 39.6 are the bend-buckling coefficients for simply-supported and fully restrained longitudinal edge conditions, respectively (Timoshenko and Gere, 1961). For I-sections in which $D_c \neq 0.5D$, Eq. 6.10.1.9.1-2 provides a reasonable approximation of theoretical bend-buckling resistance (Ziemian, 2010) consistent with the above.

For composite sections subjected to positive flexure, these Specifications do not require the use of Eq. 6.10.1.9.1-1 after the section is in its final composite condition for webs that do not require longitudinal stiffeners based on Article 6.10.2.1.1. The section must be checked for web bend-buckling during construction while in the noncomposite condition. For loads applied at the fatigue and service limit states after the deck has hardened or is made composite, the increased compressive stresses in the web tend to be compensated for by the increase in F_{crw} resulting from the corresponding decrease in D_c. At the strength limit state, these compensating effects continue. Based on the section proportioning limits specified in Article 6.10.2 and the ductility requirement specified in Article 6.10.7.3, F_{crw} for these sections is generally close to or larger than F_{yc} at the strength limit state.

For composite sections in positive flexure in which longitudinal web stiffeners are required based on Article 6.10.2.1.1, the web slenderness requirement of Article 6.10.2.1.2 is not sufficient in general to ensure that theoretical bend-buckling of the web will not occur. Therefore, the Specifications require the calculation of R_b for these types of sections, as discussed further in Article C6.10.1.10.2.

For composite sections in negative flexure, D_c is to be computed using the section consisting of the steel girder plus the longitudinal deck reinforcement, with the one possible exception noted at the service limit state in Article D6.3.1. This approach limits the potential complications in subsequent load rating resulting from the flexural resistance being a function of D_c and D_c being taken as a function of the applied load. This approach leads to a more conservative calculation of the flexural resistance, but the influence on the resistance is typically inconsequential.

Near points of permanent-load contraflexure, both edges of the web may be in compression when stresses in the steel and composite sections due to moments of opposite sign are accumulated. In this case, the neutral axis lies outside the web. Thus, the specification states that k be taken equal to 7.2 when both edges of the web are in compression, which is approximately equal to the theoretical bend-buckling coefficient for a web plate under uniform compression assuming fully restrained longitudinal edge conditions (Timoshenko and Gere, 1961). Such a case is relatively rare and the accumulated web compressive stresses are typically small when it occurs; however, this case may need to be considered in computer software.

6.10.1.9.2—Webs with Longitudinal Stiffeners

In lieu of an alternative rational analysis, the nominal bend-buckling resistance may be determined as specified in Eq. 6.10.1.9.1-1, with the bend-buckling coefficient taken as follows:

C6.10.1.9.2

Eqs. 6.10.1.9.2-1 and 6.10.1.9.2-2 give an accurate approximation of the bend-buckling coefficient k for webs with a single longitudinal stiffener in any vertical location (Frank and Helwig, 1995). The resulting k depends on the location of the closest longitudinal web

- If $\frac{d_s}{D_c} \geq 0.4$, then:

$$k = \frac{5.17}{\left(d_s / D\right)^2} \geq \frac{9}{\left(D_c / D\right)^2} \tag{6.10.1.9.2-1}$$

- If $\frac{d_s}{D_c} < 0.4$, then:

$$k = \frac{11.64}{\left(\frac{D_c - d_s}{D}\right)^2} \tag{6.10.1.9.2-2}$$

where:

d_s = distance from the centerline of the closest plate longitudinal stiffener or from the gauge line of the closest angle longitudinal stiffener to the inner surface or leg of the compression flange element (in.)

When both edges of the web are in compression, k shall be taken as 7.2.

stiffener to the compression–flange with respect to its optimum location at d_s/D_c = 0.4 (Vincent, 1969) and is used to determine the bend-buckling resistance from Eq. 6.10.1.9.1-1.

Changes in flange size cause D_c to vary along the length of a girder. In a composite girder, D_c is also a function of the applied load. If the longitudinal stiffener is located a fixed distance from the compression–flange, which is normally the case, the stiffener cannot be at its optimum location throughout the girder length. In composite girders with longitudinally-stiffened webs subjected to positive flexure, D_c tends to be large for noncomposite loadings during construction and therefore web bend-buckling must be checked. Furthermore, D_c can be sufficiently large for the composite girder at the service limit state such that web bend-buckling may still be a concern. Therefore, the value of D_c for checking web bend-buckling of these sections in regions of positive flexure at the service limit state is to be determined based on the accumulated flexural stresses due to the factored loads, as specified in Article D6.3.1.

For composite sections in negative flexure, D_c is to be computed in the same manner as discussed in Article C6.10.1.9.1.

Eqs. 6.10.1.9.2-1 and 6.10.1.9.2-2 and the associated optimum stiffener location assume simply-supported boundary conditions at the flanges. These equations for k allow the Engineer to compute the web bend-buckling resistance for any position of the longitudinal stiffener with respect to D_c. When the distance from the closest longitudinal stiffener to the compression–flange, d_s, is less than 0.4D_c, the stiffener is above its optimum location and web bend-buckling occurs in the panel between the stiffener and the tension flange. When d_s is greater than 0.4D_c, web bend-buckling occurs in the panel between the stiffener and the compression–flange. When d_s is equal to 0.4D_c, the stiffener is at its optimum location and bend-buckling occurs in both panels. For this case, both equations yield a k value equal to 129.3 for a symmetrical girder (Dubas, 1948). Further information on locating longitudinal stiffeners on the web may be found in Article C6.10.11.3.1.

Since bend-buckling of a longitudinally-stiffened web must be investigated for both noncomposite and composite stress conditions and at various locations along the girder, it is possible that the stiffener might be located at an inefficient position for a particular condition, resulting in a small bend-buckling coefficient. Because simply-supported boundary conditions were assumed in the development of Eqs. 6.10.1.9.2-1 and 6.10.1.9.2-2, the computed web bend-buckling resistance for the longitudinally-stiffened web may be less than that computed for a web of the same dimensions without longitudinal stiffeners where some rotational restraint from the flanges has been assumed. To prevent this anomaly, the Specifications state that the k value for a longitudinally-stiffened web from Eq. 6.10.1.9.2-1 must equal or exceed a value of $9.0/(D_c/D)^2$, which is the k

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value for a web without longitudinal stiffeners from Eq. 6.10.1.9.1-2 computed assuming partial rotational restraint from the flanges. Note this limit only need be checked when Eq. 6.10.1.9.2-1 controls.

As discussed further in Article C6.10.1.9.1, when both edges of the web are in compression, the bend-buckling coefficient is taken equal to 7.2.

Eqs. 6.10.1.9.2-1 and 6.10.1.9.2-2 neglect the benefit of placing more than one longitudinal stiffener on the web. Therefore, they may be used conservatively for webs with multiple longitudinal stiffeners. Alternatively, the Engineer is permitted to determine F_{crw} of Eq. 6.10.1.9.1-1 or the corresponding k value for use within this equation by a direct buckling analysis of the web panel. The boundary conditions at the flanges and at the stiffener locations should be assumed as simply-supported in this analysis.

6.10.1.10—Flange-Strength Reduction Factors

6.10.1.10.1—Hybrid Factor, R_h

For rolled shapes, homogenous built-up sections and built-up sections with a higher-strength steel in the web than in both flanges, R_h shall be taken as 1.0. Otherwise, in lieu of an alternative rational analysis, the hybrid factor shall be taken as:

$$R_h = \frac{12+\beta\left(3\rho-\rho^3\right)}{12+2\beta} \qquad (6.10.1.10.1\text{-}1)$$

in which:

$$\beta = \frac{2D_n t_w}{A_{fn}} \qquad (6.10.1.10.1\text{-}2)$$

ρ = the smaller of F_{yw}/f_n and 1.0

where:

A_{fn} = sum of the flange area and the area of any cover plates on the side of the neutral axis corresponding to D_n (in.^{2}). For composite sections in negative flexure, the area of the longitudinal reinforcement may be included in calculating A_{fn} for the top flange.

D_n = larger of the distances from the elastic neutral axis of the cross section to the inside face of either flange (in.). For sections where the neutral axis is at the mid-depth of the web, the distance from the neutral axis to the inside face of the flange on the side of the neutral axis where yielding occurs first.

f_n = for sections where yielding occurs first in the flange, a cover plate or the longitudinal reinforcement on the side of the neutral axis corresponding to D_n, the largest of the specified

C6.10.1.10.1

The R_h factor accounts for the reduced contribution of the web to the nominal flexural resistance at first yield in any flange element, due to earlier yielding of the lower strength steel in the web of a hybrid section. As used herein, the term flange element is defined as a flange or cover plate or the longitudinal reinforcement.

Eq. 6.10.1.10.1-1 represents a condensation of the formulas for R_h in previous AASHTO Specifications and considers all possible combinations associated with different positions of the elastic neutral axis and different yield strengths of the top and bottom flange elements. The fundamental equation, originally derived for a doubly-symmetric I-section (ASCE, 1968; Schilling, 1968; and Frost and Schilling, 1964), is adapted in these provisions to handle singly-symmetric and composite sections by focusing on the side of the neutral axis where yielding occurs first. This side of the neutral axis has the most extensive web yielding prior to first yielding of any flange element. All flange elements on this side of the neutral axis are conservatively assumed to be located at the edge of the web. The equation is also adapted by assuming that the shift in the neutral axis due to the onset of web yielding is negligible. These assumptions are similar to those used in the development of a separate R_h equation for composite sections in prior AASHTO Specifications. In lieu of the approximate Eq. 6.10.1.10.1-1, the Engineer may determine R_h based on a direct iterative strain–compatibility analysis. Since the computed R_h values by any approach are typically close to 1.0, the conservative assumptions made in the derivation of the simplified single noniterative Eq. 6.10.1.10.1-1 should not result in a significant economic penalty.

For hybrid longitudinally-stiffened sections in negative flexure that do not satisfy Eq. 6.10.1.10.2-1, that is, for which the web load-shedding factor, R_b, is minimum yield strengths of each component included in the calculation of A_{fn} (ksi). Otherwise, the largest of the elastic stresses in the flange, cover plate or longitudinal reinforcement on the side of the neutral axis corresponding to D_n at first yield on the opposite side of the neutral axis.

less than 1.0, Article C6.10.1.10.2 suggests the potential use of an alternative strain–compatibility analysis, using an effective-width cross-section model, to perform a combined R_b and R_h calculation in cases with highly slender longitudinally-stiffened webs with a large fraction of the web depth in compression. The need for separate calculation of the hybrid factor, R_h, is eliminated for such sections when this analysis is employed.

For composite sections in positive flexure, D_n may be taken conservatively as the distance from the neutral axis of the short-term composite section to the inside face of the bottom flange. This approach is strongly recommended to prevent possible complications in subsequent load rating resulting from the flexural resistance being a function of D_n and D_n being a function of the applied load.

For composite sections where the neutral axis is at the mid-depth of the web and where first yield occurs simultaneously in both flange elements, D_n should be taken as the distance to the flange element with the smaller A_{fn}.

For longitudinally-stiffened sections, all longitudinal stiffeners should be included in the calculation of D_c and in the section properties associated with the calculation of f_n.

6.10.1.10.2—Web Load-Shedding Factor, R_b

When checking constructibility according to the provisions of Article 6.10.3.2, or:

- the section is composite and is in positive flexure and the web satisfies the requirement of Article 6.10.2.1.1 or 6.11.2.1.2, as applicable, or:

- the web satisfies:

$$\frac{2D_c}{t_w} \leq \lambda_{rw} \quad (6.10.1.10.2\text{-}1)$$

then, R_b shall be taken equal to 1.0.

Otherwise:

- in lieu of a strain–compatibility analysis considering the web effective widths, for longitudinally-stiffened sections in which one or more continuous longitudinal stiffeners are provided that satisfy d_s/D_c < 0.76:

$$R_b = 1.07 - 0.12\frac{D_c}{D} - \frac{a_{wc}}{1200 + 300a_{wc}}\left[\frac{D}{t_w} - \lambda_{rwD}\right] \leq 1.0 \quad (6.10.1.10.2\text{-}2)$$

- for all other cases:

C6.10.1.10.2

The term R_b is a postbuckling strength reduction factor that accounts for the nonlinear variation of stresses subsequent to local bend-buckling of slender webs. This factor accounts for the reduction in the section flexural resistance caused by the shedding of compressive stresses from a slender web and the corresponding increase in the flexural stress within the compression flange. Article 6.10.1.10.2 provides two equations for R_b. Eq. 6.10.1.10.2-2 is a fit to data from Subramanian and White (2016a and 2017b) for the bend buckling strength reduction in longitudinally-stiffened girders. The R_b factor given by Eq. 6.10.1.10.2-3 is based on extensive experimental and theoretical studies (Ziemian, 2010) of nonlongitudinally stiffened girders and is the more refined of two equations developed by Basler and Thurlimann (1961). Both of these equations depend on the difference between the web slenderness, expressed as D/t_w and $2D_c/t_w$ respectively, and the corresponding web slenderness limits at which the strength reduction due to bend buckling becomes significant, expressed as λ_{rwD} and λ_{rw} respectively. Both equations also depend significantly on the ratio of the area or equivalent area of two times the area of the girder web in compression to the area of the compression flange, a_{wc}. For homogeneous longitudinally-stiffened girders, and for all nonlongitudinally stiffened girders, the λ_{rwD} and λ_{rw} limits correspond to the state at which the web theoretical elastic bend buckling stress is equal to the compression flange yield strength, F_{yc}. These limits are quantified by

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$$R_b = 1.0 - \frac{a_{wc}}{1200 + 300 a_{wc}} \left(\frac{2D_c}{t_w} - \lambda_{rw} \right) \leq 1.0$$

(6.10.1.10.2-3)

in which:

λ_{rw} = limiting slenderness ratio for a noncompact web, expressed in terms of $2D_c/t_w$, calculated as follows:

- for longitudinally-stiffened sections:

$$= \left(\frac{2D_c}{D} \right) \lambda_{rwD} \quad (6.10.1.10.2\text{-}4)$$

- for all other cases:

$$4.6\sqrt{\frac{E}{F_{yc}}} \leq \lambda_{rw} = \left(3.1 + \frac{5.0}{a_{wc}} \right) \sqrt{\frac{E}{F_{yc}}} \leq 5.7\sqrt{\frac{E}{F_{yc}}}$$

(6.10.1.10.2-5)

Eq. 6.10.1.10.2-6 for homogeneous longitudinally-stiffened girders and by Eq. 6.10.1.10.2-5 for nonlongitudinally stiffened girders. Hybrid longitudinally-stiffened girders exhibit significant strength reductions due to combined early web yielding and web bend buckling at smaller values of the web slenderness than that corresponding to theoretical elastic bend buckling at F_{yc} (Subramanian and White, 2017c). This behavior is captured by Eq. 6.10.1.10.2-7, which sets this limit effectively at the same web slenderness as that for a non-longitudinally stiffened girder. However, the strength reduction given by Eq. 6.10.1.10.2-2 is generally less than that given by Eq. 6.10.1.10.2-3. That is, the R_b values from Eq. 6.10.1.10.2-2 are generally larger for girders with the same $2D_c/t_w$ and a_{wc} values, due to the use of D/t_w and λ_{rwD} rather than $2D_c/t_w$ and λ_{rw} in the last term of the equation. For both hybrid longitudinally-stiffened and nonlongitudinally stiffened girders, the combined effects of load shedding to the compression flange due to web bend buckling and early web yielding are quantified by the product of R_b with the hybrid strength reduction factor, R_h, from Article 6.10.1.10.1. In all cases, the R_b factor is not applied in determining the nominal flexural resistance of the tension flange since the tension flange stress is not increased significantly by the shedding of the web compressive stresses (Basler and Thurlimann, 1961).

When computing the nominal flexural resistance of the compression flange for checking constructibility according to the provisions of Article 6.10.3.2, R_b is always to be taken equal to 1.0. This condition is ensured in these Specifications for all homogeneous slender-web sections by limiting the compression flange flexural stresses under the factored loads during construction to the elastic bend-buckling resistance of the web, F_{crw}. Hybrid girders may experience some local web buckling prior to reaching the theoretical elastic bend buckling stress, due to the early onset of yielding in the web. This condition has not been found to impact the girder performance.

For composite sections in positive flexure at the strength limit state, R_b is generally equal to or close to 1.0 for sections that satisfy the requirements of Articles 6.10.2.2 and 6.10.7.3, as long as the requirement of Article 6.10.2.1.1 is also met such that longitudinal stiffeners are not required. This is particularly true when a transformed area of the concrete deck is taken as part of the compression flange area as implemented for longitudinally-stiffened sections in Eq. 6.10.1.10.2-9. Therefore, the reduction in the flexural resistance due to web bend-buckling is zero or negligible and R_b is simply taken equal to 1.0 for these sections.

For sections in positive or negative flexure with one or more longitudinal web stiffeners that satisfy Eq. 6.10.1.10.2-1, R_b is taken equal to 1.0. For homogeneous sections with these characteristics, the web slenderness, D/t_w, is at or below the value at which the theoretical elastic bend-buckling stress at the strength limit state is equal to F_{yc} given by Eq. 6.10.1.10.2-6. For

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λ_{rwD} = limiting slenderness ratio for a noncompact web, expressed in terms of D/t_w, calculated as follows:

- for homogeneous longitudinally-stiffened sections:

$$=0.95\sqrt{\frac{Ek}{F_{yc}}} \quad (6.10.1.10.2\text{-}6)$$

- for hybrid longitudinally-stiffened sections:

$$=\left(\frac{1}{2D_c/D}\right)5.7\sqrt{\frac{E}{F_{yc}}} \quad (6.10.1.10.2\text{-}7)$$

a_{wc} = for all sections except as noted below, ratio of two times the web area in compression to the area of the compression flange

$$=\frac{2D_c t_w}{b_{fc}t_{fc}} \quad (6.10.1.10.2\text{-}8)$$

for composite longitudinally-stiffened sections in positive flexure:

$$=\frac{2D_c t_w}{b_{fc}t_{fc}+b_s t_s(1-f_{DC1}/F_{yc})/3n} \quad (6.10.1.10.2\text{-}9)$$

where:

b_{fc} = full width of the compression flange
b_s = effective width of the concrete deck (in.)
d_s = distance from the centerline of the closest plate longitudinal stiffener or from the gauge line of the closest angle longitudinal stiffener to the inner surface or leg of the compression flange element (in.)
D = web depth (in.)
D_c = depth of the web in compression in the elastic range (in.). For composite sections, D_c shall be determined as specified in Article D6.3.1. For longitudinally-stiffened sections, all longitudinal stiffeners including any longitudinal stiffeners in the tension zone of the web, shall be included in the calculation of D_c.
f_{DC1} = compression flange stress at the section under consideration, calculated without consideration of flange lateral bending and caused by the factored permanent load applied before the concrete deck has hardened or is made composite (ksi)
k = bend-buckling coefficient for webs with longitudinal stiffeners determined as specified in Article 6.10.1.9.2

a homogeneous doubly-symmetric girder, i.e., $D_c = 0.5D$, with a single longitudinal stiffener located at the optimum position on the web, this limit is as follows for different grades of steel:

Table C6.10.1.10.2-1—Limiting Slenderness Ratio for R_b = 1.0 in a Homogeneous Longitudinally-Stiffened Girder with the Stiffener at the Optimum Location and D_c/D = 0.5

F_{yc} (ksi)	$0.95\sqrt{\frac{Ek}{F_{yc}}}$
36.0	300
50.0	260
70.0	220
90.0	194
100.0	184

For homogeneous singly symmetric girders with $D_c/D > 0.5$ and/or where a single longitudinal stiffener is not located at its optimum position, the limiting D/t_w from Eq. 6.10.1.10.2-6 generally will be less than the value shown in Table C6.10.1.10.2-1.

For composite sections in regions of positive flexure, the concrete deck typically contributes a large fraction of the flexural resistance as a compression flange element. For longitudinally-stiffened sections of this type, Eq. 6.10.1.10.2-9 accounts for this contribution conservatively in the calculation of R_b by including a fraction of the transformed deck area based on the $3n$ section with the steel compression flange area in computing the a_{wc} term. D_c in Eq. 6.10.1.10.2-9 is to be computed as specified for composite sections in positive flexure in Article D6.3.1 and is a function of the applied loads. The relationship of the position of the longitudinal stiffener to D_c and the resulting effect on the web bend-buckling coefficient, k, is discussed further in Articles C6.10.1.9.2 and C6.10.11.3.1. In cases where R_b is equal to 1.0 for these sections, potential difficulties during load rating associated with the dependency of the flexural resistance on D_c and the dependency of D_c on the applied loading are avoided.

Eq. 6.10.1.10.2-6 ignores the beneficial effect of placing more than one longitudinal stiffener on the web. For webs with more than one longitudinal stiffener, homogeneous girders may be proportioned for R_b = 1.0 if F_{crw}, determined by an alternative rational analysis conducted as specified in Article C6.10.1.9.2, is greater than or equal to F_{yc}.

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n = modular ratio determined as specified in Article 6.10.1.1.1b

t_{fc} = thickness of the compression flange (in.)

t_s = thickness of concrete deck (in.)

t_w = web thickness (in.)

For nonlongitudinally stiffened composite sections in negative flexure and nonlongitudinally stiffened noncomposite sections that satisfy Eq. 6.10.1.10.2-1, R_b is also taken equal to 1.0 since the web slenderness, $2D_c/t_w$, is at or below the value given by Eq. 6.10.1.10.2-5 at which the theoretical elastic bend-buckling stress is equal to F_{yc} at the strength limit state. Eq. 6.10.1.10.2-5 also defines the slenderness limit for a noncompact web for sections with these characteristics. Nonlongitudinally stiffened webs with slenderness ratios exceeding Eq. 6.10.1.10.2-5 are termed slender. For different grades of steel, the lower and upper values of this slenderness limit are as follows:

Table C6.10.1.10.2-2—Lower and Upper Values of the Limiting Slenderness Ratio for a Noncompact Web and R_b = 1.0 in Girders without Web Longitudinal Stiffeners

F_{yc} (ksi)	$\lambda_{rw} = 4.6\sqrt{\frac{E}{F_{yc}}}$	$\lambda_{rw} = 5.7\sqrt{\frac{E}{F_{yc}}}$
36.0	130	162
50.0	111	137
70.0	94	116
90.0	82	102
100.0	78	97

Previous Specifications defined sections as compact or noncompact and did not explicitly distinguish between a noncompact and a slender web. The classification of webs as compact, noncompact, or slender in these Specifications applies to composite sections in negative flexure and noncomposite sections. These classifications are consistent with those in AISC (2016b). For composite sections in positive flexure, these Specifications still classify the entire cross section as compact or noncompact based on the criteria in Article 6.10.6.2.2. The Article 6.10.6.2.2 classification includes consideration of the web slenderness as well as other cross-section characteristics.

The factor 5.7 in Eq. 6.10.1.10.2-5 is based on a bend-buckling coefficient k = 36.0, which is approximately equal to $k_{ss} + 0.8(k_{sf} - k_{ss})$, where k_{ss} = 23.9 and k_{sf} = 39.6 are the bend-buckling coefficients for simply-supported and fully restrained longitudinal edge conditions, respectively, in webs without longitudinal stiffeners (Timoshenko and Gere, 1961). The factor 4.6 in this equation is based on idealized simply-supported edge conditions. Subramanian and White (2016a) show that 4.6 is representative of the strength limit state behavior of girders with relatively large webs compared to the compression flange area, i.e., a_{wc} > 3.33, and 5.7 is more representative for girders with a_{wc} < 1.92. The related Eq. 6.10.1.9.1-1 gives the web flexural stress at the theoretical onset of web bend buckling whereas Eq. 6.10.1.10.2-3 quantifies the effect of the post-buckled web on the overall girder flexural resistance. Eq. 6.10.1.9.1-1 uses a constant k = 36.0 for doubly-symmetric sections in all cases, and a constant k = 36.0 based on an equivalent web slenderness of $2D_c/t_w$ in all singly-symmetric cases. This is a simplification recognizing the acceptable service, construction and fatigue performance of girders proportioned using this simplified estimate.

For nonlongitudinally stiffened composite sections in negative flexure and general nonlongitudinally stiffened noncomposite sections that do not satisfy Eq. 6.10.1.10.2-1, R_b is to be computed from Eq. 6.10.1.10.2-3. For composite sections in negative flexure, D_c is to be computed for the section consisting of the steel girder plus the longitudinal deck reinforcement when determining R_b for reasons discussed in Article C6.10.1.9.1. For compression flanges with cover plates, the cover plate area may be added to the flange area $b_{fc}t_{fc}$ in the denominator of Eq. 6.10.1.10.2-8.

While previous specifications have suggested that it is acceptable to substitute the actual compression flange stress due to the factored loads, f_{bu}, calculated without consideration of flange lateral bending, for F_{yc} in the web-slenderness limit equations, such a refinement is not likely to lead to a significant increase in the value of R_b. In addition, this practice does not necessarily result in improvements in the accuracy of the strength predictions (Subramanian and White, 2017b). Use of the actual flange stress to compute the flexural resistance can also lead to subsequent difficulties in load rating since the flexural resistance then becomes a function of the applied load. Therefore, it is recommended that R_b should always be calculated based on the compression flange yield strength, F_{yc}.

The studies by Subramanian and White (2016a and 2017c) illustrate the significant contribution of a continuous longitudinal stiffener to the flexural resistance of sections in negative flexure that are loaded into the web postbuckling range at the strength limit state. These studies found that, for girders that do not satisfy Eq. 6.10.1.10.2-1, the most accurate prediction of the web postbuckling, or R_b effects, in longitudinally-stiffened homogeneous girders, and the most accurate prediction of the combined web yielding and web postbuckling, or R_bR_h effects, in longitudinally-stiffened hybrid sections in negative flexure is accomplished by calculating the ultimate moment capacity from an iterative strain–compatibility analysis considering specified effective widths of the web panels. For homogeneous longitudinally-stiffened girders, this type of strain–compatibility analysis gives acceptable estimates of test simulation resistances that range from only 4 percent smaller to 11 percent larger than the R_b values obtained from Eq. 6.10.1.10.2-2. For hybrid longitudinally-stiffened girders, the strain–compatibility analysis gives acceptable estimates of test simulation resistances that range from 7 percent smaller to 17 percent larger than the R_hR_b values obtained from Eqs. 6.10.1.10.1-1 and 6.10.1.10.2-2. This iterative strain–compatibility analysis is recommended for longitudinally-stiffened girders where additional calculated capacities are needed, and where D_c/D is greater than 0.5 and (D/t_w - λ_{rwD}) is larger than 110. The additional calculated capacity from the strain–compatibility analysis model tends to be marginal to none in other cases. The recommended cross-section models for performing the strain–compatibility analysis for homogeneous and hybrid longitudinally-stiffened girders are given in Subramanian and White (2017b and 2017c). Subramanian and White (2016a, 2016b, 2017a, and 2017b) demonstrate the applicability of the recommended strain–compatibility analysis models with the corresponding Specification provisions for the lateral–torsional buckling, flange local buckling and tension flange yielding limit states in straight and curved girders.

Eq. 6.10.1.10.2-2 and the recommended strain–compatibility analysis models assume that the longitudinal stiffener is continuous at the ends of the web panel under consideration. This may be achieved by continuing the longitudinal stiffener across the transverse stiffener locations on the side of the web opposite from the transverse stiffeners, or by positively attaching the longitudinal stiffener on each side of the transverse stiffener at both ends of the panel when the stiffeners are on the same side of the web. The longitudinally stiffened web panel must be an interior panel, adjacent to at least one additional panel into which the longitudinal stiffener is continued, for the longitudinal stiffening to be considered effective for calculation of the postbuckling flexural resistance at the strength limit state. The contribution of the longitudinal stiffener to the postbuckling flexural resistance of the girder at the strength limit state is to be neglected when the panel has longitudinal stiffeners that are not continuous; that is, Eq. 6.10.1.10.2-3 must be used to compute R_b and Eq. 6.10.1.10.2-5 must be used to compute λ_{rw}. Provision of a longitudinal stiffener that is discontinued at one end of the web panel is still deemed to satisfy the Article 6.10.2.1.2 requirements. The contribution of any longitudinal stiffeners located in the tension zone of the web, or with $d_s/D_c > 0.76$, is ignored in Eq. 6.10.1.10.2-2 and in the recommended cross-section strain–compatibility analysis models.

The requirements for proportioning of longitudinal stiffeners in Article 6.10.11.3 ensure the development of the elastic web bend-buckling resistance specified in Article 6.10.1.9. Elastic bend buckling of longitudinally-stiffened webs is theoretically prevented up through the service limit state in these Specifications, but is permitted at the strength limit state. The minimum stiffener proportioning requirements do not ensure that a horizontal line of near zero lateral deflection will be maintained for the subsequent post-bend-buckling response of the web, particularly at web slenderness values significantly larger than 200 (Subramanian and White, 2017b). However, satisfaction of the minimum stiffener proportioning requirements allows R_b to be taken as 1.0 up through the service limit state. While it may be beneficial to increase the stiffener rigidity to some extent from the minimum requirements in order to increase the postbuckling strength, the increase in strength is minor and there are generally diminishing returns as the stiffener size is increased beyond twice the minimum proportioning requirements (Subramanian and White, 2016b).

6.10.2—Cross-Section Proportion Limits

6.10.2.1—Web Proportions

6.10.2.1.1—Webs without Longitudinal Stiffeners

Webs shall be proportioned such that:

$$\frac{D}{t_w} \leq 150 \qquad (6.10.2.1.1\text{-}1)$$

C6.10.2.1.1

Eq. 6.10.2.1.1-1 is a practical upper limit on the slenderness of webs without longitudinal stiffeners expressed in terms of the web depth, D. This equation allows for easier proportioning of the web in preliminary design relative to previous Specifications. In previous Specifications, Eq. 6.10.2.1.1-1 was the upper limit for unstiffened webs. By also limiting the slenderness of transversely-stiffened webs to this value, maximum transverse stiffener spacings up to $3D$ are permitted; the requirement in previous Specifications to provide additional transverse stiffeners for handling in girders with more slender webs, beyond those required for shear, is eliminated. Furthermore, satisfaction of Eq. 6.10.2.1.1-1 allows web bend-buckling to be disregarded in the design of composite sections in positive flexure, as discussed further in Article C6.10.1.9.1. The limit in Eq. 6.10.2.1.1-1 is valid for sections with specified minimum yield strengths up to and including 100.0 ksi designed according to these Specifications.

The vertical flange buckling limit-state equations in AISC (2016b), which are based in large part on ASCE (1968), are not considered in these Specifications. These equations specify a limit on the web slenderness to prevent theoretical elastic buckling of the web as a column subjected to a radial transverse compression due to the curvature of the flanges. For girders that satisfy Eq. 6.10.2.1.1-1, these equations do not govern the web slenderness unless F_{yc} is greater than 85.0 ksi. Furthermore, tests conducted by Lew and Toprac (1968), Cooper (1967), and others, in which the final failure mode involved vertical flange buckling, or a folding of the compression–flange vertically into the web, indicate that the influence of this failure mode on the predicted girder flexural resistances is small. This is the case even for girders with parameters that

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significantly violate the vertical flange buckling limit-state equations.

6.10.2.1.2—Webs with Longitudinal Stiffeners

Webs shall be proportioned such that:

$$\frac{D}{t_w} \leq 300 \qquad (6.10.2.1.2\text{-}1)$$

C6.10.2.1.2

Eq. 6.10.2.1.2-1 is a practical upper limit on the slenderness of webs with longitudinal stiffeners expressed in terms of the web depth, D. This limit allows for easier proportioning of the web for preliminary design than comparable limits in previous Specifications. The limit in Eq. 6.10.2.1.2-1 is valid for sections with specified minimum yield strengths up to and including 100.0 ksi designed according to these Specifications.

Cooper (1967) discusses the conservatism of vertical flange buckling limit-state equations and the justification for not considering this limit state in longitudinally-stiffened I-girders. Tests by Cooper (1967), Owen et al. (1970) and others have demonstrated that the flexural resistance is not adversely affected by final failure modes involving vertical flange buckling, even for longitudinally-stiffened girders that significantly exceed the limit of Eq. 6.10.2.1.2-1. In all cases involving a vertical flange buckling type of failure, extensive flexural yielding of the compression flange preceded the failure. However, webs that have larger D/t_w values than specified by Eq. 6.10.2.1.2-1 are relatively inefficient, are likely to be more susceptible to distortion-induced fatigue, and are more susceptible to the limit states of web crippling and web yielding of Article D6.5.

6.10.2.2—Flange Proportions

Compression and tension flanges shall be proportioned such that:

$$\frac{b_f}{2t_f} \leq 12.0, \qquad (6.10.2.2\text{-}1)$$

$$b_f \geq D/6, \qquad (6.10.2.2\text{-}2)$$

C6.10.2.2

Eq. 6.10.2.2-1 is a practical upper limit to ensure the flange will not distort excessively when welded to the web.

White and Barth (1998) observe that the cross-section aspect ratio D/b_f is a significant parameter affecting the strength and moment-rotation characteristics of I-sections. Eq. 6.10.2.2-2 limits this ratio to a maximum value of 6. Experimental test data are limited for sections with very narrow flanges. A significant number of the limited tests that have been conducted have indicated relatively low nominal flexural and shear resistances relative to the values determined using these and previous Specifications. Limiting this ratio to a maximum value of 6 for both the compression and tension flanges ensures that stiffened interior web panels, with the section along the entire panel proportioned to satisfy Eq. 6.10.9.3.2-1, can develop postbuckling shear resistance due to tension-field action (White et al., 2004). Eq. 6.10.2.2-2 provides a lower limit on the flange width. In most practical cases, a wider flange will be required, particularly for horizontally-curved girders.

To help ensure that individual girder field sections will be stable and easier to handle for lifting, erection, and shipping without the need for special stiffening trusses or

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falsework, the following guideline should also be satisfied:

$$b_{tfs} \geq \frac{L_{fs}}{85} \qquad \text{(C6.10.2.2-1)}$$

where:

b_{tfs} = smallest top-flange width within the unspliced individual girder field section under consideration (in.)

L_{fs} = length of the unspliced individual girder field section under consideration (in.)

Eq. C6.10.2.2-1 should be used during the design, in conjunction with the flange proportioning limits specified in Article 6.10.2.2, to establish a minimum required top-flange width for each individual unspliced girder field section. It should be emphasized that Eq. C6.10.2.2-1 is provided merely as a guideline and is not an absolute requirement. It is also not intended that the Engineer attempt to anticipate how the individual field sections may eventually be assembled or spliced together and/or stabilized or supported for shipping or erection in the application of the preceding guideline; the guideline is only to be applied to individual unspliced girder field sections for design. The stability of steel girders for shipping and erection either as individual shipping pieces or after being spliced together to form larger pieces should be considered the responsibility of the Contractor (NHI, 2015a).

$$t_f \geq 1.1 t_w, \qquad \text{(6.10.2.2-3)}$$

and:

$$0.1 \leq \frac{I_{yc}}{I_{yt}} \leq 10 \qquad \text{(6.10.2.2-4)}$$

where:

I_{yc} = moment of inertia of the compression flange of the steel section about the vertical axis in the plane of the web (in.^{4})

I_{yt} = moment of inertia of the tension flange of the steel section about the vertical axis in the plane of the web (in.^{4})

Eq. 6.10.2.2-3 ensures that some restraint will be provided by the flanges against web shear buckling, and also that the boundary conditions assumed at the web-flange juncture in the web bend-buckling and compression flange local buckling formulations within these Specifications are sufficiently accurate. The ratio of the web area to the compression–flange area is always less than or equal to 5.45 for members that satisfy Eqs. 6.10.2.2-2 and 6.10.2.2-3. Therefore, the AISC (2016b) limit of 10 on this ratio is not required.

An I-section with a ratio of I_{yc}/I_{yt} outside the limits specified in Eq. 6.10.2.2-4 is more like a T-section with the shear center located at the intersection of the larger flange and the web. The limits of Eq. 6.10.2.2-4 are similar to the limits specified in previous Specifications, but are easier to apply since they are based on the ratio of I_{yc} to I_{yt} rather than to I_y of the entire steel section. Eq. 6.10.2.2-4 ensures more efficient flange proportions and prevents the use of sections that may be particularly difficult to handle during construction. Also, Eq. 6.10.2.2-4 ensures the validity of the equations for $C_b > 1$ in cases involving moment gradients. Furthermore, these limits tend to prevent the use of extremely monosymmetric sections for which the larger of the yield moments, M_{yc} or M_{yt}, may be greater than the plastic moment, M_p. If the flanges are composed of plates of

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equal thickness, these limits are equivalent to $b_{fc} \geq 0.46 b_{ft}$ and $b_{fc} \leq 2.15\ b_{ft}$.

6.10.3—Constructibility

6.10.3.1—General

The provisions of Article 2.5.3 shall apply. In addition to providing adequate strength, nominal yielding or reliance on post-buckling resistance shall not be permitted for main load-carrying members during critical stages of construction, except for yielding of the web in hybrid sections. This shall be accomplished by satisfying the requirements of Articles 6.10.3.2 and 6.10.3.3 at each critical construction stage. For sections in positive flexure that are composite in the final condition, but are noncomposite during construction, the provisions of Article 6.10.3.4 shall apply. For investigating the constructibility of flexural members, all loads shall be factored as specified in Article 3.4.2. For the calculation of deflections, the load factors shall be taken as 1.0.

Potential uplift at bearings shall be investigated at each critical construction stage.

Webs without bearing stiffeners at locations subjected to concentrated loads not transmitted through a deck or deck system shall satisfy the provisions of Article D6.5.

If there are holes in the tension flange at the section under consideration, the tension flange shall also satisfy the requirement specified in Article 6.10.1.8.

Load-resisting bolted connections either in or to flexural members shall be proportioned to prevent slip under the factored loads at each critical construction stage. The provisions of Article 6.13.2.8 shall apply for investigation of connection slip.

C6.10.3.1

If uplift is indicated at any critical stage of construction, temporary load may be placed to prevent lift-off. The magnitude and position of any required temporary load should be provided in the contract documents.

Factored forces at high-strength bolted joints of load carrying members are limited to the slip resistance of the connection during each critical construction state to ensure that the correct geometry of the structure is maintained.

6.10.3.2—Flexure

6.10.3.2.1—Discretely Braced Flanges in Compression

For critical stages of construction, each of the following requirements shall be satisfied. For sections with slender webs, Eq. 6.10.3.2.1-1 shall not be checked when f_ℓ is equal to zero. For sections with compact or noncompact webs, Eq. 6.10.3.2.1-3 shall not be checked.

$$f_{bu} + f_\ell \leq \phi_f R_h F_{yc}, \qquad (6.10.3.2.1\text{-}1)$$

$$f_{bu} + \frac{1}{3} f_\ell \leq \phi_f F_{nc}, \qquad (6.10.3.2.1\text{-}2)$$

and

$$f_{bu} \leq \phi_f F_{crw} \qquad (6.10.3.2.1\text{-}3)$$

where:

C6.10.3.2.1

A distinction is made between discretely and continuously braced compression and tension flanges because for a continuously braced flange, flange lateral bending need not be considered.

This Article gives constructibility requirements for discretely braced compression flanges, expressed by Eqs. 6.10.3.2.1-1, 6.10.3.2.1-2, and 6.10.3.2.1-3 in terms of the combined factored vertical and flange lateral bending stresses during construction. In making these checks, the stresses f_{bu} and f_ℓ must be determined according to the procedures specified in Article 6.10.1.6.

Eq. 6.10.3.2.1-1 ensures that the maximum combined stress in the compression flange will not exceed the specified minimum yield strength of the flange times the hybrid factor; that is, it is a yielding limit state check.

Eq. 6.10.3.2.1-2 ensures that the member has sufficient strength with respect to lateral–torsional and

ϕ_f = resistance factor for flexure specified in Article 6.5.4.2.

f_{bu} = flange stress calculated without consideration of flange lateral bending determined as specified in Article 6.10.1.6 (ksi)

f_ℓ = flange lateral bending stress determined as specified in Article 6.10.1.6 (ksi)

F_{crw} = nominal bend-buckling resistance for webs specified in Article 6.10.1.9 (ksi)

F_{nc} = nominal flexural resistance of the flange (ksi). F_{nc} shall be determined as specified in Article 6.10.8.2. For sections in straight I-girder bridges with compact or noncompact webs, the lateral–torsional buckling resistance may be taken as M_{nc} determined as specified in Article A6.3.3 divided by S_{xc}. In computing F_{nc} for constructibility, the web load-shedding factor, R_b, shall be taken as 1.0.

M_{yc} = yield moment with respect to the compression flange determined as specified in Article D6.2 (kip-in.)

R_h = hybrid factor specified in Article 6.10.1.10.1. For hybrid sections in which f_{bu} does not exceed the specified minimum yield strength of the web, the hybrid factor shall be taken equal to 1.0.

S_{xc} = elastic section modulus about the major axis of the section to the compression flange taken as M_{yc}/F_{yc} (in.^{3})

flange local buckling based limit states, including the consideration of flange lateral bending where these effects are judged to be significant. For horizontally-curved bridges, flange lateral bending effects due to curvature must always be considered in discretely braced flanges during construction.

Eq. 6.10.3.2.1-3 ensures that theoretical web bend-buckling will not occur during construction.

Eq. 6.10.3.2.1-2 addresses the resistance of the compression flange by considering this element as an equivalent beam-column. This equation is effectively a beam-column interaction equation, expressed in terms of the flange stresses computed from elastic analysis (White and Grubb, 2005). The f_{bu} term is analogous to the axial load and the f_ℓ term is analogous to the bending moment within the equivalent beam-column member. The factor of 1/3 in front of the f_ℓ term in Eq. 6.10.3.2.1-2 gives an accurate linear approximation of the equivalent beam-column resistance within the limits on f_ℓ specified in Article 6.10.1.6 (White and Grubb, 2005).

Eq. 6.10.3.2.1-1 often controls relative to Eq. 6.10.3.2.1-2, particularly for girders with large f_ℓ and for members with compact or noncompact webs. However, for members with noncompact flanges or large unsupported lengths during construction combined with small or zero values for f_ℓ, Eq. 6.10.3.2.1-2 will typically control. During construction before the hardening of the deck, most flanges are discretely braced. The compact, noncompact and slender web definitions are discussed in Article C6.10.6.2.3. For making these checks with the section in its noncomposite condition, the categorization of the web is to be based on the properties of the noncomposite section. The meanings assigned to the compact and noncompact flange categorizations are discussed in Article C6.10.8.2.2. When $f_\ell = 0$, Eq. 6.10.3.2.1-1 will not control and need not be checked for sections with slender webs. For sections with compact or noncompact webs, Eq. 6.10.3.2.1-1 should still be checked. However, web bend-buckling is not a consideration for these types of members, and therefore, Eq. 6.10.3.2.1-3 need not be checked for these sections.

In checking Eq. 6.10.3.2.1-2 for sections in straight I-girder bridges with compact or noncompact webs, the lateral–torsional buckling resistance of the flange may be determined from the provisions of Article A6.3.3, which include the beneficial contribution of the St. Venant torsional constant J. This may be useful for sections in such bridges with compact or noncompact webs having larger unbraced lengths, if additional lateral–torsional buckling resistance is required beyond that calculated based on the provisions of Article 6.10.8.2. The resulting lateral–torsional buckling resistance, M_{nc}, is then divided by S_{xc} to express the resistance in terms of stress for direct application in Eq. 6.10.3.2.1-2. In some cases, the calculated resistance will exceed F_{yc} since Appendix A6 accounts in general for flexural resistances greater than the yield moment resistance, M_{yc} or M_{yt}. However, Eq. 6.10.3.2.1-1 will control in these cases, thus ensuring

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that the combined factored stress in the flange will not exceed F_{yc} times the hybrid factor during construction.

The rationale for calculation of S_{xc}, as defined in this Article for use in determining F_{nc} for sections with noncompact or compact webs, is discussed in Article CA6.1.1.

For sections that are composite in the final condition, but are noncomposite during construction, different values of the hybrid factor, R_h, must be calculated for checks in which the member is noncomposite and for checks in which the member is composite.

Because the flange stress is limited to the web bend-buckling stress according to Eq. 6.10.3.2.1-3, the R_b factor is always to be taken equal to 1.0 in computing the nominal flexural resistance of the compression flange for constructibility.

Should the web bend-buckling resistance be exceeded for the construction condition, the Engineer has several options to consider. These options include providing a larger compression flange or a smaller tension flange to decrease the depth of the web in compression, adjusting the deck-placement sequence to reduce the compressive stress in the web, or providing a thicker web. Should these options not prove to be practical or cost-effective, a longitudinal web stiffener can be provided. As specified in Article 6.10.11.3.1, the longitudinal stiffener must be located vertically on the web to satisfy Eq. 6.10.3.2.1-3 for the construction condition, Eq. 6.10.4.2.2-4 at the service limit state and all the appropriate design requirements at the strength limit state. Further discussions of procedures for locating a longitudinal stiffener are provided in Article C6.10.11.3.1.

6.10.3.2.2—Discretely Braced Flanges in Tension

For critical stages of construction, the following requirement shall be satisfied:

$$f_{bu} + f_{\ell} \leq \phi_f R_h F_{yt} \qquad (6.10.3.2.2\text{-}1)$$

C6.10.3.2.2

For a discretely braced flange in tension, Eq. 6.10.3.2.2-1 ensures that the stress in the flange will not exceed the specified minimum yield strength of the flange times the hybrid factor during construction under the combination of the major-axis bending and lateral bending stresses due to the factored loads.

6.10.3.2.3—Continuously Braced Flanges in Tension or Compression

For critical stages of construction, the following requirement shall be satisfied:

$$f_{bu} \leq \phi_f R_h F_{yf} \qquad (6.10.3.2.3\text{-}1)$$

For noncomposite sections with slender webs, flanges in compression shall also satisfy Eq. 6.10.3.2.1-3.

C6.10.3.2.3

This Article assumes that a continuously braced flange in compression is not subject to local or lateral–torsional buckling. Article C6.10.1.6 states the conditions for which a flange may be considered to be continuously braced. By encasing the flange in concrete or by attaching the flange to the concrete deck by shear connectors that satisfy the requirements of Article 6.10.10, one side of the flange is effectively prevented from local buckling, or both sides of the flange must buckle in the direction away from the concrete deck. Therefore, highly restrained boundary conditions are provided in effect at the web-flange juncture. Also, the flange lateral bending deflections, required to obtain a significant reduction in strength associated with flange local buckling, are effectively prevented by the concrete deck. Therefore, neither flange local nor lateral–torsional buckling need to be checked for compression flanges that satisfy the proportioning limits of Article 6.10.2.2 and are continuously braced according to the conditions stated in Article C6.10.1.6.

6.10.3.2.4—Concrete Deck

The longitudinal tensile stress in a composite concrete deck due to the factored loads shall not exceed ϕf_r during critical stages of construction, unless longitudinal reinforcement is provided according to the provisions of Article 6.10.1.7. The concrete stress shall be determined as specified in Article 6.10.1.1.1d. ϕ and f_r shall be taken as specified in Article 6.10.1.7.

C6.10.3.2.4

This Article is intended to address primarily the situation when the concrete deck is placed in a span adjacent to a span where the concrete has already been placed. Negative moment in the adjacent span causes tensile stresses in the previously placed concrete. Also, if long placements are made such that a negative flexure region is included in the first placement, it is possible that the concrete in this region will be stressed in tension during the remainder of the deck placement, which may lead to early cracking of the deck. When the longitudinal tensile stress in the deck exceeds the factored modulus of rupture of the concrete, longitudinal reinforcement is to be provided according to the provisions of Article 6.10.1.7 to control the cracking. Stresses in the concrete deck are to be computed using the short-term modular ratio, *n*, per Article 6.10.1.1.1d.

6.10.3.3—Shear

Webs shall satisfy the following requirement during critical stages of construction:

$$V_u \leq \phi_v V_{cr} \tag{6.10.3.3-1}$$

where:

ϕ_v = resistance factor for shear specified in Article 6.5.4.2

V_u = shear in the web acting on the noncomposite section under consideration due to the factored load for constructibility specified in Article 3.4.2.1 (kip)

V_{cr} = shear-yielding or shear-buckling resistance determined from Eq. 6.10.9.3.3-1 (kip)

C6.10.3.3

The web is to be investigated for the sum of the factored permanent loads and factored construction loads applied to the noncomposite section during construction. The nominal shear resistance for this check is limited to the shear yielding or shear-buckling resistance per Eq. 6.10.9.3.3-1. The use of tension-field action per Eq. 6.10.9.3.2-2 is not permitted under these loads during construction. Use of tension-field action is permitted after the deck has hardened or is made composite, if the section along the entire panel is proportioned to satisfy Eq. 6.10.9.3.2-1.

6.10.3.4—Deck Placement

6.10.3.4.1—General

Sections in positive flexure that are composite in the final condition, but are noncomposite during construction, shall be investigated for flexure according to the provisions of Article 6.10.3.2 during the various stages of the deck placement.

Geometric properties, bracing lengths, and stresses used in calculating the nominal flexural resistance shall be for the steel section only. Changes in load, stiffness,

C6.10.3.4.1

The entire concrete deck may not be placed in one stage; thus, parts of the girders may become composite in sequential stages. If certain deck placement sequences are followed, the temporary moments induced in the girders during the deck placement can be considerably higher than the final noncomposite dead load moments after the sequential placement is complete.

and bracing during the various stages of the deck placement shall be considered.

The effects of forces from deck overhang brackets acting on the fascia girders shall be considered.

Sequentially staged concrete placement can also result in significant tensile strains in the previously placed deck in adjacent spans. When cracking is predicted, longitudinal deck reinforcement as specified in Article 6.10.3.2.4 is required to control the cracking. Temporary dead load deflections during sequential deck placement can also be different from final noncomposite dead load deflections. If the differences are deemed significant, this should be considered when establishing girder cambers that are subsequently considered in setting screed requirements for construction, as specified in Article 6.7.2. These constructibility concerns apply to deck replacement as well as initial construction.

During construction of steel girder bridges, concrete deck overhang loads are typically supported by cantilever forming brackets typically placed at 3.0 to 4.0 ft spacings along the exterior members. The eccentricity of the deck weight and other loads acting on the overhang brackets creates applied torsional moments on the exterior members. As a result, the following issues must be considered in the design of the exterior members:

- The applied torsional moments bend the exterior girder top flanges outward. The resulting flange lateral bending stresses tend to be largest at the brace points at one or both ends of the unbraced length. The lateral bending stress in the top flange is tensile at the brace points on the side of the flange opposite from the brackets. These lateral bending stresses should be considered in the design of the flanges.
- The horizontal components of the reactions on the cantilever-forming brackets are often transmitted directly onto the exterior girder web. The girder web may exhibit significant plate bending deformations due to these loads. The effect of these deformations on the vertical deflections at the outside edge of the deck should be considered. The effect of the reactions from the brackets on the cross-frame forces should also be considered.
- Excessive deformation of the web or top flange may lead to excessive deflection of the bracket supports causing the deck finish to be problematic.

Where practical, forming brackets should be carried to the intersection of the bottom flange and the web. Alternatively, the brackets may bear on the girder webs if means are provided to ensure that the web is not damaged. The provisions of Article 6.10.3.2 allow for the consideration of the flange lateral bending stresses in the design of the flanges. In the absence of a more refined analysis, either of the following equations may be used to estimate the maximum flange lateral bending moments due to the eccentric loadings depending on how the lateral load is assumed applied to the top flange:

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$$M_\ell = \frac{F_\ell L_b^2}{12} \quad \text{(C6.10.3.4.1-1)}$$

where:

M_ℓ = lateral bending moment in the flanges due to the eccentric loadings from the forming brackets (kip-in.)
F_ℓ = statically equivalent uniformly distributed lateral force from the brackets due to the factored loads (kip/in.)
L_b = unbraced length (in.)

$$M_\ell = \frac{P_\ell L_b}{8} \quad \text{(C6.10.3.4.1-2)}$$

where:

P_ℓ = statically equivalent concentrated lateral bracket force placed at the middle of the unbraced length (kip)

Eqs. C6.10.3.4.1-1 and C6.10.3.4.1-2 are both based on the assumption of interior unbraced lengths in which the flange is continuous with adjacent unbraced lengths, as well as equal adjacent unbraced lengths such that due to approximate symmetry boundary conditions, the ends of the unbraced length are effectively torsionally fixed. The Engineer should consider other more appropriate idealizations when these assumptions do not approximate the actual conditions.

Construction dead loads, such as those acting on the deck overhangs, are often applied to the noncomposite section and removed when the bridge has become composite. Typically, the major-axis bending moments due to these loads are small relative to other design loads. However, the Engineer may find it desirable in some cases to consider the effect of these moments, particularly in computing deflections for cambers. The lateral bending moments due to overhang loads not applied through the shear center of the girder are often more critical. Refined analysis of the noncomposite bridge for these loads provides more accurate lateral moments and may identify any rotation of the overhang that could potentially affect the elevation of the screed when finishing the deck.

The magnitude and application of the overhang loads assumed in the design should be shown in the contract documents.

6.10.3.4.2—Global Displacement Amplification in Narrow I-Girder Bridge Units

The provisions of this Article shall apply to spans of straight I-girder bridge units with three or fewer girders, interconnected by cross-frames or diaphragms, that also meet both of the following conditions in their

C6.10.3.4.2

The recommendations in this Article are intended to avoid excessive amplification of the lateral and vertical displacements of narrow straight I-girder bridge units during the deck placement operation before the concrete

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noncomposite condition during the deck placement operation:

- the unit is not braced by other structural units and/or by external bracing within the span; and
- the unit does not contain any flange level lateral bracing or lateral bracing from a hardened composite deck within the span.

Considering all of the girders across the width of the unit within the span under consideration, the sum of the largest total factored girder moments during the deck placement within the span under consideration should not exceed 70 percent of the elastic global lateral–torsional buckling resistance of the span acting as a system. The elastic global lateral–torsional buckling resistance of the span acting as a system, M_{gs}, may be calculated as follows:

$$M_{gs} = C_{bs}\frac{\pi^2 w_g E}{L^2}\sqrt{I_{eff} I_x} \qquad (6.10.3.4.2\text{-}1)$$

in which:

C_{bs} = system moment gradient modifier
= 1.1 for simply-supported units
= 2.0 for continuous-span units

- For doubly symmetric girders:

$$I_{eff} = I_y \qquad (6.10.3.4.2\text{-}2)$$

- For singly symmetric girders:

$$I_{eff} = I_{yc} + \left(\frac{t}{c}\right) I_{yt} \qquad (6.10.3.4.2\text{-}3)$$

where:

c = distance from the centroid of the noncomposite steel section under consideration to the centroid of the compression flange (in.). The distance shall be taken as positive.
I_x = noncomposite moment of inertia about the horizontal centroidal axis of a single girder within the span under consideration (in.^4)
I_{yc}, I_{yt} = moments of inertia of the compression and tension flange, respectively, about the vertical centroidal axis of a single girder within the span under consideration (in.^4)
I_y = noncomposite moment of inertia about the vertical centroidal axis of a single girder within the span under consideration (in.^4)
L = length of the span under consideration (in.)

deck has hardened. The global buckling mode in this case refers to buckling of the bridge unit as a structural unit, and not buckling of the girders between intermediate braces. Limiting the sum of the largest total factored girder moments across the width of the unit within the span under consideration to 70 percent of the elastic global buckling resistance of the span acting as a system theoretically limits the amplification under the corresponding nominal loads to a maximum value of approximately 2.0.

Eq. 6.10.3.4.2-1 (Yura et al., 2008) provides one method of estimating the elastic global lateral–torsional buckling resistance of a given straight I-girder bridge span under noncomposite loading conditions. The system moment gradient modifier, C_{bs}, in Eq. 6.10.3.4.2-1 accounts for the beneficial effect of the moment gradient within the span on the elastic global lateral–torsional buckling resistance of the span acting as a system, which is particularly significant for continuous-span units. A C_{bs} value of 1.1 applies to simply-supported units, and should also be applied if investigating continuous-span units that are in a partially erected condition. A C_{bs} value of 2.0 applies to fully erected continuous-span units. Two-girder units are particularly susceptible to excessive global lateral–torsional amplification during the deck placement; however, units with large span/width ratios having up to three girders also may be susceptible to significant global amplification in some cases. Other methods, such as an eigenvalue buckling analysis or a global second-order load-deflection analysis, may also be used to determine the response of the system. Once a concrete deck is acting compositely with the steel girders, a given span of a bridge unit is practically always stable as an overall system; Eq. 6.10.3.4.2-1 is not intended for application to I-girder bridge spans in their composite condition. Eq. 6.10.3.4.2-1 is also not applicable to I-girder bridge units with more than three girders, which are typically not susceptible to excessive global lateral–torsional amplification during the deck placement.

Eq. 6.10.3.4.2-1 was derived assuming prismatic girders and that all girder cross sections in the unit are the same. For cases where the girders are nonprismatic and/or the girder cross sections vary across the unit, it is recommended herein that length-weighted average moments of inertia within the positive-moment sections of all the girders in the span under consideration be used for I_x, I_y, I_{yc} and I_{yt}, as applicable, in calculating the elastic global lateral–torsional buckling resistance from Eq. 6.10.3.4.2-1. Also, in cases where the girder spacing is less than the girder depth, it is recommended that the more general elastic global lateral–torsional buckling equation provided in Yura et. al. (2008) be used, as Eq. 6.10.3.4.2-1 becomes more conservative in this case. Yura et al. (2008) further indicates the adjustments that need to be made to the more general buckling equation for singly symmetric girders and/or for three-girder systems.

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t = distance from the centroid of the noncomposite steel section under consideration to the centroid of the tension flange (in.). The distance shall be taken as positive.

w_g = girder spacing for a two-girder system or the distance between the two exterior girders of the unit for a three-girder system (in.)

Should the sum of the largest total factored girder moments across the width of the unit within the span under consideration exceed 70 percent of M_{gs}, the following alternatives may be considered:

- The addition of flange level lateral bracing adjacent to the supports of the span may be considered as discussed in Article 6.7.5.2;
- The unit may be revised to increase the system stiffness; or
- The amplified girder second-order displacements of the span during the deck placement may be evaluated to verify that they are within tolerances permitted by the Owner.

Large global torsional rotations signified by large differential vertical deflections between the girders and also large lateral deflections, as determined from a first-order analysis, are indicative of the potential for significant second-order global amplification. Situations exhibiting potentially significant global second-order amplification include phased construction involving narrow unsupported units with only two or three girders and possibly unevenly applied deck weight. One suggested method of increasing the global buckling resistance in such cases is to consider the addition of flange level lateral bracing to the system. Yura et al. (2008) suggest adjustments to be made when estimating the elastic global lateral–torsional buckling resistance of the system where a partial top-flange lateral bracing system is present at the ends of the span, along with some associated bracing design recommendations.

The elastic global buckling resistance should only be used as a general indicator of the susceptibility of horizontally-curved I-girder systems to second-order amplification under noncomposite loading conditions. Narrow horizontally-curved I-girder bridge units that meet both of the conditions stated in this Article in their noncomposite condition during the deck placement may be subject to significant second-order amplification and should instead be analyzed using a global second-order load-deflection analysis to evaluate the behavior. As an alternative, the addition of flange level lateral bracing adjacent to the supports of the span may be considered as discussed in Article 6.7.5.2, or the unit can be braced to other structural units or by external bracing within the span.

6.10.3.5—Dead Load Deflections

The provisions of Article 6.7.2 shall apply, as applicable.

C6.10.3.5

If staged construction is specified, the sequence of load application should be recognized in determining the camber and stresses.

6.10.4—Service Limit State

6.10.4.1—Elastic Deformations

The provisions of Article 2.5.2.6 shall apply, as applicable.

C6.10.4.1

The provisions of Article 2.5.2.6 contain optional live load deflection criteria and criteria for span-to-depth ratios. In the absence of depth restrictions, the span-to-depth ratios should be used to establish a reasonable minimum web depth for the design.

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6.10.4.2—Permanent Deformations

6.10.4.2.1—General

For the purposes of this Article, the Service II load combination specified in Table 3.4.1-1 shall apply.

The following methods may be used to calculate stresses in structural steel at the Service II limit state:

- For members with shear connectors provided throughout their entire length that also satisfy the provisions of Article 6.10.1.7, flexural stresses in the structural steel caused by Service II loads applied to the composite section may be computed using the short-term or long-term composite section, as appropriate. The concrete deck may be assumed to be effective for both positive and negative flexure, provided that the maximum longitudinal tensile stresses in the concrete deck at the section under consideration caused by the Service II loads are smaller than $2f_r$, where f_r is the modulus of rupture of the concrete specified in Article 6.10.1.7.
- For sections that are composite for negative flexure with maximum longitudinal tensile stresses in the concrete deck greater than or equal to $2f_r$, the flexural stresses in the structural steel caused by Service II loads shall be computed using the section consisting of the steel section and the longitudinal reinforcement within the effective width of the concrete deck.
- For sections that are noncomposite for negative flexure, the properties of the steel section alone shall be used for calculation of the flexural stresses in the structural steel.

The longitudinal stresses in the concrete deck shall be determined as specified in Article 6.10.1.1.1d.

C6.10.4.2.1

These provisions are intended to apply to the design live load specified in Article 3.6.1.1. If this criterion were to be applied to a design permit load, a reduction in the load factor for live load should be considered.

Article 6.10.1.7 requires that one percent longitudinal deck reinforcement be placed wherever the tensile stress in the concrete deck due to either factored construction loads or due to Load Combination Service II exceeds the factored modulus of rupture of the concrete. By controlling the crack size in regions where adequate shear connection is also provided, the concrete deck may be considered effective in tension for computing flexural stresses on the composite section due to Load Combination Service II.

The cracking behavior and the partial participation of the physically cracked slab in transferring forces in tension is very complex. Article 6.10.4.2.1 provides specific guidance that the concrete slab may be assumed to be uncracked when the maximum longitudinal concrete tensile stress is smaller than $2f_r$. This limit between the use of an uncracked or cracked section for calculation of flexural stresses in the structural steel is similar to a limit suggested in CEN (2004) beyond which the effects of concrete cracking should be considered.

6.10.4.2.2—Flexure

Flanges shall satisfy the following requirements:

- For the top steel flange of composite sections:

$$f_f \leq 0.95 R_h F_{yf} \qquad (6.10.4.2.2\text{-}1)$$

- For the bottom steel flange of composite sections:

$$f_f + \frac{f_\ell}{2} \leq 0.95 R_h F_{yf} \qquad (6.10.4.2.2\text{-}2)$$

- For both steel flanges of noncomposite sections:

$$f_f + \frac{f_\ell}{2} \leq 0.80 R_h F_{yf} \qquad (6.10.4.2.2\text{-}3)$$

where:

C6.10.4.2.2

Eqs. 6.10.4.2.2-1 through 6.10.4.2.2-3 are intended to prevent objectionable permanent deflections due to expected severe traffic loadings that would impair rideability. For homogeneous sections with zero flange lateral bending, they correspond to the overload check in the AASHTO *Standard Specifications* (2002) and are based on successful past practice. Their development is described in Vincent (1969). A resistance factor is not applied in these equations because the specified limits are serviceability criteria for which the resistance factor is 1.0.

Eqs. 6.10.4.2.2-1 through 6.10.4.2.2-3 address the increase in flange stresses caused by early web yielding in hybrid sections by including the hybrid factor R_h.

For continuous-span members in which noncomposite sections are utilized in negative flexure regions only, it is recommended that Eqs. 6.10.4.2.2-1

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f_f = flange stress at the section under consideration due to the Service II loads calculated without consideration of flange lateral bending (ksi)

f_ℓ = flange lateral bending stress at the section under consideration due to the Service II loads determined as specified in Article 6.10.1.6 (ksi)

R_h = hybrid factor determined as specified in Article 6.10.1.10.1

For continuous span flexural members in straight I-girder bridges that satisfy the requirements of Article B6.2, a calculated percentage of the negative moment due to the Service II loads at the pier section under consideration may be redistributed using the procedures of either Article B6.3 or B6.6.

For compact composite sections in positive flexure utilized in shored construction, the longitudinal compressive stress in the concrete deck due to the Service II loads, determined as specified in Article 6.10.1.1.1d, shall not exceed $0.6f'_c$.

Except for composite sections in positive flexure in which the web satisfies the requirement of Article 6.10.2.1.1, all sections shall also satisfy the following requirement:

$$f_c \leq F_{crw} \tag{6.10.4.2.2-4}$$

where:

f_c = compression flange stress at the section under consideration due to the Service II loads calculated without consideration of flange lateral bending (ksi)

F_{crw} = nominal bend-buckling resistance for webs with or without longitudinal stiffeners, as applicable, determined as specified in Article 6.10.1.9 (ksi)

and 6.10.4.2.2-2, as applicable, be applied in those regions.

Under the load combinations specified in Table 3.4.1-1, Eqs. 6.10.4.2.2-1 through 6.10.4.2.2-3, as applicable, do not control and need not be checked for the following sections:

- Composite sections in negative flexure for which the nominal flexural resistance under the Strength load combinations is determined according to the provisions of Article 6.10.8;
- Noncomposite sections with $f_\ell = 0$ and for which the nominal flexural resistance under the Strength load combinations is determined according to the provisions of Article 6.10.8;
- Noncompact composite sections in positive flexure.

However, Eq. 6.10.4.2.2-4 must still be checked for these sections where applicable.

The 1/2 factor in Eqs. 6.10.4.2.2-2 and 6.10.4.2.2-3 comes from Schilling (1996) and Yoo and Davidson (1997). Eqs. 6.10.4.2.2-2 and 6.10.4.2.2-3 with a limit of F_{yf} on the right-hand side are a close approximation to rigorous yield interaction equations for the load level corresponding to the onset of yielding at the web-flange juncture, including the effect of flange tip yielding that occurs prior to this stage, but not considering flange residual stress effects. If the flanges are nominally elastic at the web-flange juncture and the elastically computed flange lateral bending stresses are limited as required by Eq. 6.10.1.6-1, the permanent deflections will be small. The $0.95R_h$ and $0.80R_h$ factors are included on the right-hand side of Eqs. 6.10.4.2.2-2 and 6.10.4.2.2-3 to make them compatible with the corresponding equations in the prior Specifications when f_ℓ = 0, and to provide some additional conservatism for control of permanent deformations when the flange lateral bending is significant. The sign of f_f and f_ℓ should always be taken as positive in Eqs. 6.10.4.2.2-2 and 6.10.4.2.2-3. However, when summing dead and live load stresses to obtain the total factored major-axis and lateral bending stresses, f_f and f_ℓ, to apply in the equations, the signs of the individual stresses must be considered.

f_ℓ is not included in Eq. 6.10.4.2.2-1 because the top flange is continuously braced by the concrete deck. For continuously braced top flanges of noncomposite sections, the f_ℓ term in Eq. 6.10.4.2.2-3 may be taken equal to zero.

Lateral bending in the bottom flange is only a consideration at the service limit state for all horizontally-curved I-girder bridges and for straight I-girder bridges with discontinuous cross-frame or diaphragm lines in conjunction with skews exceeding 20 degrees. Wind load and deck overhang effects are not considered at the service limit state.

Localized yielding in negative-flexural sections at interior piers results in redistribution of the elastic moments. For continuous-span flexural members in straight I-girder bridges that satisfy the provisions of Article B6.2, the procedures of either Article B6.3 or B6.6 may be used to calculate the redistribution moments at the service limit state. These procedures represent an improvement on the former ten-percent redistribution rule. When the redistribution moments are calculated according to these procedures, Eqs. 6.10.4.2.2-1 through 6.10.4.2.2-3, as applicable, need not be checked within the regions extending from the pier section under consideration to the nearest flange transition or point of permanent-load contraflexure, whichever is closest, in each adjacent span. Eq. 6.10.4.2.2-4 must still be considered within these regions using the elastic moments prior to redistribution. At all locations outside of these regions, Eqs. 6.10.4.2.2-1 through 6.10.4.2.2-4, as applicable, must be satisfied after redistribution. Research has not yet been conducted to extend the provisions of Appendix B6 to kinked (chorded) continuous or horizontally-curved steel I-girder bridges.

For compact composite sections utilized in shored construction, the longitudinal stresses in the concrete deck are limited to $0.6f'_c$ to ensure linear behavior of the concrete. In unshored construction, the concrete stress near first yielding of either steel flange is generally significantly less than f'_c thereby eliminating the need to check the concrete stress in this case.

With the exception of composite sections in positive flexure in which the web satisfies the requirement of Article 6.10.2.1.1 such that longitudinal stiffeners are not required, and web bend-buckling effects are negligible, web bend-buckling of all sections must be checked under the Service II Load Combination according to Eq. 6.10.4.2.2-4. Article C6.10.1.9.1 explains why web bend-buckling does not need to be checked for the above exception. Options to consider should the web bend-buckling resistance be exceeded are similar to those discussed for the construction condition at the end of Article C6.10.3.2.1, except of course for adjusting the deck-placement sequence.

If the concrete deck is assumed effective in tension in regions of negative flexure, as permitted at the service limit state for composite sections satisfying the requirements specified in Article 6.10.4.2.1, more than half of the web may be in compression thus increasing the susceptibility to web bend-buckling. As specified in Article D6.3.1, for composite sections in negative flexure, the appropriate value of D_c to be used at the service limit state depends on whether or not the concrete deck is assumed effective in tension. For noncomposite sections, D_c of the steel section alone should always be used.

6.10.5—Fatigue and Fracture Limit State

6.10.5.1—Fatigue

Details shall be investigated for fatigue as specified in Article 6.6.1. The applicable Fatigue load combination specified in Table 3.4.1-1 and the fatigue live load specified in Article 3.6.1.4 shall apply.

For horizontally-curved I-girder bridges, the fatigue stress range due to major-axis bending plus lateral bending shall be investigated.

The provisions for fatigue in shear connectors specified in Articles 6.10.10.2 and 6.10.10.3 shall apply.

C6.10.5.1

In horizontally-curved I-girder bridges, the base metal adjacent to butt welds and welded attachments on discretely braced flanges subject to a net applied tensile stress must be checked for the fatigue stress range due to major-axis bending, plus flange lateral bending, at the critical transverse location on the flange. Examples of welded attachments for which this requirement applies include transverse stiffeners and gusset plates receiving lateral bracing members. The base metal adjacent to flange-to-web welds need only be checked for the stress range due to major-axis bending since the welds are located near the center of the flange. Flange lateral bending need not be considered for details attached to continuously braced flanges.

6.10.5.2—Fracture

Fracture toughness requirements specified in the contract documents shall be in conformance with the provisions of Article 6.6.2.1.

6.10.5.3—Special Fatigue Requirement for Webs

For the purposes of this Article, the factored fatigue load shall be determined using the Fatigue I load combination specified in Table 3.4.1-1, with the fatigue live load taken as specified in Article 3.6.1.4.

Interior panels of webs with transverse stiffeners, with or without longitudinal stiffeners, shall satisfy the following requirement:

$$V_u \leq V_{cr} \qquad (6.10.5.3\text{-}1)$$

where:

V_u = shear in the web at the section under consideration due to the unfactored permanent load plus the factored fatigue load (kip)

V_{cr} = shear-yielding or shear-buckling resistance determined from Eq. 6.10.9.3.3-1 (kip)

C6.10.5.3

If Eq. 6.10.5.3-1 is satisfied, significant elastic flexing of the web due to shear is not expected to occur, and the member is assumed able to sustain an infinite number of smaller loadings without fatigue cracking due to this effect.

This provision is included here, rather than in Article 6.6, because it involves a check of the maximum web shear-buckling stress instead of a check of the stress ranges caused by cyclic loading.

The live load stress due to the passage of the specified fatigue live load for this check is that of the heaviest truck expected to cross the bridge in 75 years.

The check for bend-buckling of webs given in AASHTO (1998) due to the load combination specified in this Article is not included in these Specifications. For all sections, except for composite sections in positive flexure in which the web satisfies Article 6.10.2.1.1, a web bend-buckling check is required under the Service II Load Combination according to the provisions of Article 6.10.4.2.2. As discussed further in Article C6.10.1.9.1, web bend-buckling of composite sections in positive flexure is not a concern at any limit state after the section is in its final composite condition for sections with webs that satisfy Article 6.10.2.1.1. For all other sections, the web bend-buckling check under the Service II loads will control over a similar check under the load combination specified in this Article. For composite sections in positive flexure with webs that do not satisfy Article 6.10.2.1.1, the smaller value of F_{crw} resulting from the larger value of D_c at the fatigue limit state tends to be compensated for by the lower web compressive stress due to the load combination specified in this Article. Web bend-buckling of these sections is also checked under the construction condition according to Eq. 6.10.3.2.1-3.

The shear in unstiffened webs is already limited to either the shear-yielding or shear-buckling resistance at the strength limit state according to the provisions of Article 6.10.9.2. The shear in end panels of stiffened webs is also limited to the shear-yielding or shear-buckling resistance at the strength limit state according to the provisions of Article 6.10.9.3.3. Consequently, the requirement in this Article need not be checked for unstiffened webs or the end panels of stiffened webs.

6.10.6—Strength Limit State

6.10.6.1—General

For the purposes of this Article, the applicable Strength load combinations specified in Table 3.4.1-1 shall apply.

C6.10.6.1

At the strength limit state, Article 6.10.6 directs the Engineer to the appropriate Articles for the design of composite or noncomposite I-sections in regions of positive or negative flexure.

For sections in which the flexural resistance is expressed in terms of stress, the elastically computed flange stress is strictly not an estimate of the actual flange stress because of limited partial yielding within the cross section due to the combination of applied load effects with initial residual stresses and various other incidental stress contributions not included within the design analysis calculations. The effects of partial yielding within the cross section on the distribution of internal forces within the system prior to reaching the maximum resistances as defined in these Specifications are minor and may be neglected in the calculation of the applied stresses and/or moments.

The use of stresses is considered to be more appropriate in members within which the maximum resistance is always less than or equal to the yield moment M_y in major-axis bending. This is due to the nature of the different types of loadings that contribute to the member flexural stresses: noncomposite, long-term composite and short-term composite. The combined effects of the loadings on these different states of the member cross section are better handled by working with flange stresses rather than moments. Bridge engineers typically are also more accustomed to working with stresses rather than moments. Therefore, although the provisions can be written equivalently in terms of bending moment, the provisions of Article 6.10 are written in terms of stress whenever the maximum potential resistance in terms of f_{bu} is less than or equal to F_y.

Conversely, for members in which the resistance is potentially greater than M_y, significant yielding within the cross section makes the handling of the capacities in terms of stress awkward. Although the provisions that are written in terms of moment can be written equivalently in terms of elastic stress quantities, the corresponding elastic stress limits will be generally greater than the yield stress since the moments are greater than the yield moment. Also, the calculation of the resistance where it is generally greater than M_y is fundamentally based on stress resultants. For example, M_p for a compact composite section in positive flexure is based on a plastic analysis of the composite cross section. Therefore, it is more natural to write the resistance equations in terms of bending moments for these types of sections. This is also the practice in AASHTO (1998).

For sections in which the flexural resistance is expressed in terms of moment, the moments acting on the noncomposite, long-term composite and short-term composite sections may be directly summed for comparison to the nominal flexural resistance. That is, the effect of the sequence of application of the different types of loads on the stress states and of partial yielding within the cross section on the maximum resistance need not be considered.

If the Engineer uses analysis software in which the webs of I-section members or the composite deck, or both, are represented as shell or solid elements, then the flange stresses may be obtained directly from the software. Adjustments to these stresses will be required however, for cases where the effective flange width of the deck is less than the full tributary width perpendicular to the axis of the member, and for cases in regions of negative flexure where the concrete deck may not be considered effective in tension. For specification checks involving major-axis bending moments supported by composite sections, the neutral axis of the composite section can be determined from the ratio of the flange stresses over the steel section depth. The amount of effective deck is then computable, from which a moment of inertia of the effective composite section can then be calculated. From the moment of inertia and the available flange stresses, a major-axis bending moment acting on the composite girder can then be determined.

In subsequent Articles, a continuously braced flange in compression is assumed not to be subject to local or lateral–torsional buckling. The rationale for excluding these limit state checks is discussed in Article C6.10.3.2.3.

These provisions assume low or zero levels of axial force in the member and uniaxial flexure. For members that are also subject to a factored concentrically-applied axial force, P_u, in excess of 5 percent of the factored axial resistance of the member, P_r or P_{ry} as applicable, at the strength limit state, and/or if the member is subject to biaxial bending, the member should instead be checked according to the provisions of Article 6.8.2.3 or 6.9.2.2, as applicable. The level of 5 percent is based conservatively on the linear interaction equations given in these Articles, which apply in the majority of cases. Below this level, it is reasonable to ignore the effect of the axial force in the design of the member.

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6.10.6.2—Flexure

6.10.6.2.1—General

If there are holes in the tension flange at the section under consideration, the tension flange shall satisfy the requirement specified in Article 6.10.1.8.

6.10.6.2.2—Composite Sections in Positive Flexure

Composite sections in kinked (chorded) continuous or horizontally-curved steel girder bridges shall be considered as noncompact sections and shall satisfy the requirements of Article 6.10.7.2.

Composite sections in straight bridges that satisfy the following requirements shall qualify as compact composite sections:

- The specified minimum yield strengths of the flanges do not exceed 70.0 ksi,
- The web satisfies the requirement of Article 6.10.2.1.1, and
- The section satisfies the web slenderness limit:

$$\frac{2D_{cp}}{t_w} \leq 3.76\sqrt{\frac{E}{F_{yc}}} \qquad (6.10.6.2.2\text{-}1)$$

where:

D_{cp} = depth of the web in compression at the plastic moment determined as specified in Article D6.3.2 (in.)

Compact sections shall satisfy the requirements of Article 6.10.7.1. Otherwise, the section shall be considered noncompact and shall satisfy the requirements of Article 6.10.7.2.

Compact and noncompact sections shall satisfy the ductility requirement specified in Article 6.10.7.3.

C6.10.6.2.2

The nominal flexural resistance of composite sections in positive flexure in straight bridges satisfying specific steel grade, web slenderness, and ductility requirements is permitted to exceed the moment at first yield according to the provisions of Article 6.10.7. The nominal flexural resistance of these sections, termed compact sections, is therefore more appropriately expressed in terms of moment. For composite sections in positive flexure in straight bridges not satisfying one or more of these requirements, or for composite sections in positive flexure in horizontally-curved bridges, termed noncompact sections, the nominal flexural resistance is not permitted to exceed the moment at first yield. The nominal flexural resistance in these cases is therefore more appropriately expressed in terms of the elastically computed flange stress.

Composite sections in positive flexure in straight bridges with flange yield strengths greater than 70.0 ksi or with webs that do not satisfy Article 6.10.2.1.1 are to be designed at the strength limit state as noncompact sections as specified in Article 6.10.7.2. For concrete compressive strengths typically employed for deck construction, the use of larger steel yield strengths may result in significant nonlinearity and potential crushing of the deck concrete prior to reaching the flexural resistance specified for compact sections in Article 6.10.7.1. Longitudinal stiffeners generally must be provided in sections with webs that do not satisfy Article 6.10.2.1.1. Since composite longitudinally-stiffened sections tend to be deeper and used in longer spans with corresponding larger noncomposite dead load stresses, they tend to have D_c/t_w values that would preclude the development of substantial inelastic flexural strains within the web prior to bend-buckling at moment levels close to R_hM_y. Therefore, although the depth of the web in compression typically reduces as plastic strains associated with moments larger than R_hM_y are incurred, and D_{cp} may indeed satisfy Eq. 6.10.6.2.2-1 at the plastic moment resistance, sufficient test data do not exist to support the design of these types of sections for M_p. Furthermore, because of the relative size of the steel section to the concrete deck typical for these types of sections, M_p often is not substantially larger than R_hM_y. Due to these factors, composite sections in positive flexure in which the web does not satisfy Article 6.10.2.1.1 are categorized as noncompact sections. Composite sections in positive flexure in kinked (chorded) continuous or horizontally-curved steel bridges are also to be designed at the strength limit state as noncompact sections as specified in Article 6.10.7.2. Research has not yet been conducted to support the design of these sections for a nominal flexural resistance exceeding the moment at first yield.

The web slenderness requirement of this Article is adopted from AISC (2016b) and gives approximately the same allowable web slenderness as specified for compact sections in AASHTO *Standard Specifications* (2002). Most composite sections in positive flexure without longitudinal web stiffeners will qualify as compact according to this criterion since the concrete deck causes an upward shift in the neutral axis, which reduces the depth of the web in compression. Also, D/t_w for these sections is limited to a maximum value of 150 based on the requirement of Article 6.10.2.1.1. The location of the neutral axis of the composite section at the plastic moment may be determined using the equations listed in Table D6.1-1.

Compact composite sections in positive flexure must also satisfy the provisions of Article 6.10.7.3 to ensure a ductile mode of failure. Noncompact sections must also satisfy the ductility requirement specified in Article 6.10.7.3 to ensure a ductile failure. Satisfaction of this requirement ensures an adequate margin of safety against premature crushing of the concrete deck for sections utilizing up to 100-ksi steels and/or for sections utilized in shored construction. This requirement is also a key limit in allowing web bend-buckling to be disregarded in the design of composite sections in positive flexure when the web also satisfies Article 6.10.2.1.1, as discussed in Article C6.10.1.9.1.

6.10.6.2.3—Composite Sections in Negative Flexure and Noncomposite Sections

Sections in all kinked (chorded) continuous or horizontally-curved steel girder bridges shall be proportioned according to the provisions specified in Article 6.10.8.

Sections in straight bridges whose supports are normal or skewed not more than 20 degrees from normal, and with intermediate diaphragms or cross-frames placed in contiguous lines parallel to the supports may be proportioned according to the provisions for compact or noncompact web sections specified in Appendix A6. For these sections:

- The specified minimum yield strengths of the flanges do not exceed 70.0 ksi,
- The web satisfies the noncompact slenderness limit:

$$\frac{2D_c}{t_w} \leq \lambda_{rw} \quad (6.10.6.2.3\text{-}1)$$

and:

C6.10.6.2.3

For composite sections in negative flexure and noncomposite sections, the provisions of Article 6.10.8 limit the nominal flexural resistance to be less than or equal to the moment at first yield. As a result, the nominal flexural resistance for these sections is conveniently expressed in terms of the elastically computed flange stress.

For composite sections in negative flexure or noncomposite sections in straight bridges without skewed supports or with limited skews that satisfy the specified steel grade requirements and with webs that satisfy Eq. 6.10.6.2.3-1 and flanges that satisfy Eq. 6.10.6.2.3-2, the optional provisions of Appendix A6 may be applied to determine the nominal flexural resistance, which may exceed the moment at first yield. Therefore, the nominal flexural resistance determined from the provisions of Appendix A6 is expressed in terms of moment. Because these types of sections are less commonly used, the provisions for their design have been placed in an appendix in order to simplify and streamline the main design provisions. The provisions of Article 6.10.8 may be used for these types of sections to obtain an accurate to somewhat

- The flanges satisfy the following ratio:

$$\frac{I_{yc}}{I_{yt}} \geq 0.3 \tag{6.10.6.2.3-2}$$

in which:

λ_{rw} = limiting slenderness ratio for a noncompact web

$$= 4.6\sqrt{\frac{E}{F_{yc}}} \leq \lambda_{rw} = \left(3.1 + \frac{5.0}{a_{wc}}\right)\sqrt{\frac{E}{F_{yc}}} \leq 5.7\sqrt{\frac{E}{F_{yc}}} \tag{6.10.6.2.3-3}$$

a_{wc} = ratio of two times the web area in compression to the area of the compression flange

$$= \frac{2D_c t_w}{b_{fc} t_{fc}} \tag{6.10.6.2.3-4}$$

where:

D_c = depth of the web in compression in the elastic range (in.). For composite sections, D_c shall be determined as specified in Article D6.3.1.

I_{yc} = moment of inertia of the compression flange of the steel section about the vertical axis in the plane of the web (in.^{4})

I_{yt} = moment of inertia of the tension flange of the steel section about the vertical axis in the plane of the web (in.^{4})

Otherwise, the section shall be proportioned according to provisions specified in Article 6.10.8.

For continuous span flexural members in straight bridges that satisfy the requirements of Article B6.2, a calculated percentage of the negative moments due to the factored loads at the pier section under consideration may be redistributed using the procedures of either Article B6.4 or B6.6.

conservative determination of the nominal flexural resistance than would be obtained using Appendix A6.

For composite sections in negative flexure or noncomposite sections in straight bridges not satisfying one or more of these requirements, or for these sections in horizontally-curved bridges, the provisions of Article 6.10.8 must be used. Research has not yet been conducted to extend the provisions of Appendix A6 either to sections in kinked (chorded) continuous or horizontally-curved steel bridges or to bridges with supports skewed more than 20 degrees from normal. Severely skewed bridges with contiguous cross-frames have significant transverse stiffness and thus already have large cross-frame forces in the elastic range. As interior-pier sections yield and begin to lose stiffness and shed their load, the forces in the adjacent cross-frames will increase. There is currently no established procedure to predict the resulting increase in the forces without performing a refined nonlinear analysis. With discontinuous cross-frames, significant lateral flange bending effects can occur. The resulting lateral bending moments and stresses are amplified in the bottom compression flange adjacent to the pier as the flange deflects laterally. There is currently no means to accurately predict these amplification effects as the flange is also yielding. Skewed supports also result in twisting of the girders, which is not recognized in plastic-design theory. The relative vertical deflections of the girders create eccentricities that are also not recognized in the theory. Thus, until further research is done to examine these effects in greater detail, a conservative approach has been taken in the specification.

Eq. 6.10.6.2.3-1 defines the slenderness limit for a noncompact web. A web with a slenderness ratio exceeding this limit is termed slender. The previous Specifications defined sections as compact or noncompact and did not explicitly distinguish between a noncompact and a slender web. For noncompact webs, theoretical web bend-buckling does not occur for elastic stress values, computed according to beam theory, smaller than the limit of the flexural resistance. Sections with slender webs rely upon the significant web post bend-buckling resistance under Strength Load Combinations. Specific values for the noncompact web slenderness limit for different grades of steel are listed in Table C6.10.1.10.2-2.

A compact web is one that satisfies the slenderness limit given by Eq. A6.2.1-1. Sections with compact webs and $I_{yc}/I_{yt} \geq 0.3$ are able to develop their full plastic moment capacity M_p provided that other steel grade, ductility, flange slenderness and/or lateral bracing requirements are satisfied. The web-slenderness limit given by Eq. A6.2.1-1 is significantly smaller than the limit shown in Table C6.10.1.10.2-2. It is generally satisfied by rolled I-shapes, but typically not by the most efficient built-up section proportions.

The flange yield stress, F_{yc}, is more relevant to the web buckling behavior and its influence on the flexural

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resistance than F_{yw}. For a section that has a web proportioned at the noncompact limit, a stable nominally elastic compression flange tends to constrain a lower-strength hybrid web at stress levels less than or equal to R_hF_{yc}. For a section that has a compact web, the inelastic strains associated with development of the plastic flexural resistance are more closely related to the flange rather than the web yield strength.

The majority of steel-bridge I-sections utilize either slender webs or noncompact webs that approach the slenderness limit of Eq. 6.10.6.2.3-1 represented by the values listed in Table C6.10.1.10.2-2. For these sections, the simpler and more streamlined provisions of Article 6.10.8 are the most appropriate for determining the nominal flexural resistance of composite sections in negative flexure and noncomposite sections. These provisions may also be applied to sections with compact webs or to sections with noncompact webs that are nearly compact, but at the expense of some economy. Such sections are typically used in bridges with shorter spans. The potential loss in economy increases with decreasing web slenderness. The Engineer should give strong consideration to utilizing the provisions of Appendix A6 to compute the nominal flexural resistance of these sections in straight bridges, in particular, sections with compact webs.

Eq. 6.10.6.2.3-2 is specified to guard against extremely monosymmetric noncomposite I-sections, in which analytical studies indicate a significant loss in the influence of the St. Venant torsional rigidity GJ on the lateral–torsional buckling resistance due to cross-section distortion. The influence of web distortion on the lateral–torsional buckling resistance is larger for such members. If the flanges are of equal thickness, this limit is equivalent to $b_{fc} \geq 0.67b_{ft}$.

Yielding in negative-flexural sections at interior piers at the strength limit state results in redistribution of the elastic moments. For continuous-span flexural members in straight bridges that satisfy the provisions of Article B6.2, the procedures of either Article B6.4 or B6.6 may be used to calculate redistribution moments at the strength limit state. These provisions replace the former ten-percent redistribution allowance and provide a more rational approach for calculating the percentage redistribution from interior-pier sections. When the redistribution moments are calculated according to these procedures, the flexural resistances at the strength limit state within the unbraced lengths immediately adjacent to interior-pier sections satisfying the requirements of Article B6.2 need not be checked. At all other locations, the provisions of Articles 6.10.7, 6.10.8.1 or A6.1, as applicable, must be satisfied after redistribution. The provisions of Article B6.2 are often satisfied by compact-flange unstiffened or transversely-stiffened pier sections that are otherwise designed by Article 6.10.8 or Appendix A6 using $C_b = 1.0$. Research has not yet been conducted to extend the provisions of Appendix B6 to kinked (chorded) continuous or horizontally-curved steel bridges.

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6.10.6.3—Shear

The provisions of Article 6.10.9 shall apply.

6.10.6.4—Shear Connectors

The provisions of Article 6.10.10.4 shall apply.

6.10.7—Flexural Resistance—Composite Sections in Positive Flexure

6.10.7.1—Compact Sections

6.10.7.1.1—General

At the strength limit state, the section shall satisfy:

$$M_u + \frac{1}{3} f_\ell S_{xt} \leq \phi_f M_n \qquad (6.10.7.1.1\text{-}1)$$

where:

ϕ_f = resistance factor for flexure specified in Article 6.5.4.2
f_ℓ = flange lateral bending stress determined as specified in Article 6.10.1.6 (ksi)
M_n = nominal flexural resistance of the section determined as specified in Article 6.10.7.1.2 (kip-in.)
M_u = bending moment about the major-axis of the cross section determined as specified in Article 6.10.1.6 (kip-in.)
M_{yt} = yield moment with respect to the tension flange determined as specified in Article D6.2 (kip-in.)
S_{xt} = elastic section modulus about the major axis of the section to the tension flange taken as M_{yt}/F_{yt} (in.^{3})

C6.10.7.1.1

For composite sections in positive flexure, lateral bending does not need to be considered in the compression–flange at the strength limit state because the flange is continuously supported by the concrete deck.

Eq. 6.10.7.1.1-1 is an interaction equation that addresses the influence of lateral bending within the tension flange, represented by the elastically computed flange lateral bending stress, f_ℓ, combined with the major-axis bending moment, M_u. This equation is similar to the subsequent Eqs. 6.10.7.2.1-2 and 6.10.8.1.2-1, the basis of which is explained in Article C6.10.8.1.2. However, these other equations are expressed in an elastically computed stress format, and the resistance term on their right-hand side is generally equal to $\phi_f R_h F_{yt}$. Eq. 6.10.7.1.1-1 is expressed in a bending moment format, but alternatively can be considered in a stress format by dividing both sides of the equation by the elastic section modulus, S_{xt}.

The term M_n on the right-hand side of Eq. 6.10.7.1.1-1 is generally greater than the yield moment capacity, M_{yt}. Therefore, the corresponding resistance, written in the format of an elastically computed stress, is generally greater than F_{yt}. These Specifications use a moment format for all resistance equations which, if written in terms of an elastically computed stress, can potentially assume resistance values greater than the specified minimum yield strength of the steel. In these types of sections, the major-axis bending moment is physically a more meaningful quantity than the corresponding elastically computed bending stress.

Eq. 6.10.7.1.1-1 gives a reasonably accurate but conservative representation of the results from an elastic-plastic section analysis in which a fraction of the width from the tips of the tension flange is deducted to accommodate flange lateral bending. The rationale for calculation of S_{xt}, as defined in this Article for use in Eq. 6.10.7.1.1-1, is discussed in Article CA6.1.1.

6.10.7.1.2—Nominal Flexural Resistance

The nominal flexural resistance of the section shall be taken as:

C6.10.7.1.2

Eq. 6.10.7.1.2-2 implements the philosophy introduced by Wittry (1993) that an additional margin of safety should be applied to the theoretical nominal flexural

If $D_p \le 0.1\, D_t$, then:

$$M_n = M_p \tag{6.10.7.1.2-1}$$

Otherwise:

$$M_n = M_p\left(1.07 - 0.7\frac{D_p}{D_t}\right) \tag{6.10.7.1.2-2}$$

where:

D_p = distance from the top of the concrete deck to the neutral axis of the composite section at the plastic moment (in.)
D_t = total depth of the composite section (in.)
M_p = plastic moment of the composite section determined as specified in Article D6.1 (kip-in.)

In a continuous span, the nominal flexural resistance of the section shall satisfy:

$$M_n \le 1.3\, R_h M_y \tag{6.10.7.1.2-3}$$

where:

M_n = nominal flexural resistance determined from Eq. 6.10.7.1.2-1 or 6.10.7.1.2-2, as applicable (kip-in.)
M_y = yield moment determined as specified in Article D6.2 (kip-in.)
R_h = hybrid factor determined as specified in Article 6.10.1.10.1

unless:

- the span under consideration and all adjacent interior-pier sections satisfy the requirements of Article B6.2,

and:

- the appropriate value of θ_{RL} from Article B6.6.2 exceeds 0.009 radians at all adjacent interior-pier sections,

in which case the nominal flexural resistance of the section is not subject to the limitation of Eq. 6.10.7.1.2-3.

resistance of compact composite sections in positive flexure when the depth of the plastic neutral axis below the top of the deck, D_p, exceeds a certain value. This additional margin of safety, which increases approximately as a linear function of D_p/D_t, is intended to protect the concrete deck from premature crushing, thereby ensuring adequate ductility of the composite section. Sections with D_p/D_t less than or equal to 0.1 can reach as a minimum the plastic moment, M_p, of the composite section without any ductility concerns.

Eq. 6.10.7.1.2-2 gives approximately the same results as the comparable equation in previous Specifications, but is a simpler form that depends only on the plastic moment resistance M_p and on the ratio D_p/D_t, as also suggested in Yakel and Azizinamini (2005). Both equations implement the above philosophy justified by Wittry (1993). Eq. 6.10.7.1.2-2 is somewhat more restrictive than the equation in previous Specifications for sections with small values of M_p/M_y, such as sections with hybrid webs, a relatively small deck area and a high-strength tension flange. It is somewhat less restrictive for sections with large values of M_p/M_y. Wittry (1993) considered various experimental test results and performed a large number of parametric cross-section analyses. The smallest experimental or theoretical resistance of all the cross sections considered in this research and in other subsequent studies is $0.96M_p$. Eq. 6.10.7.1.2-2 is based on the target additional margin of safety of 1.28 specified by Wittry at the maximum allowed value of D_p combined with an assumed theoretical resistance of $0.96M_p$ at this limit. At the maximum allowed value of D_p specified by Eq. 6.10.7.3-1, the resulting nominal design flexural resistance is $0.78M_p$.

The limit of $D_p < 0.1D_t$ for the use of Eq. 6.10.7.1.2-1 is obtained by use of a single implicit β value of 0.75 in the comparable equations from AASHTO (1998). AASHTO (1998) specifies β = 0.7 for F_y = 50 and 70.0 ksi and β = 0.9 for F_y = 36.0 ksi. The value of β = 0.75 is justifiable for all cases based on the scatter in strain-hardening data. The derived β values are sensitive to the assumed strain-hardening characteristics.

The shape factor, M_p/M_y, for composite sections in positive flexure can be somewhat larger than 1.5 in certain cases. Therefore, a considerable amount of yielding and resulting inelastic curvature is required to reach M_p in these situations. This yielding reduces the effective stiffness of the positive flexural section. In continuous spans, the reduction in stiffness can shift moment from the positive to the negative flexural regions. If the interior-pier sections in these regions do not have additional capacity to sustain these larger moments and are not designed to have ductile moment-rotation characteristics according to the provisions of Appendix B6, the shedding of moment to these sections could result in incremental collapse under repeated live load applications. Therefore, for cases where the span or either of the adjacent interior-pier sections do not satisfy the provisions of Article B6.2, or where the appropriate value of θ_{RL} from Article B6.6.2 at either adjacent pier section is less than or equal to 0.009 radians, the positive flexural sections must satisfy Eq. 6.10.7.1.2-3.

It is possible to satisfy the above concerns by ensuring that the pier section flexural resistances are not exceeded if the positive flexural section moments above R_hM_y are redistributed and combined with the concurrent negative moments at the pier sections determined from an elastic analysis. This approach is termed the Refined Method in AASHTO (1998). However, concurrent moments are not typically tracked in the analysis and so this method is not included in these Specifications.

Eq. 6.10.7.1.2-3 is provided to limit the amount of additional moment allowed above R_hM_y at composite sections in positive flexure to 30 percent of R_hM_y in continuous spans where the span or either of the adjacent pier sections do not satisfy the requirements of Article B6.2. The $1.3R_hM_y$ limit is the same as the limit specified for the Approximate Method in AASHTO (1998). The nominal flexural resistance determined from Eq. 6.10.7.1.2-3 is not to exceed the resistance determined from either Eq. 6.10.7.1.2-1 or 6.10.7.1.2-2, as applicable, to ensure adequate strength and ductility of the composite section. In cases where D_p/D_t is relatively large and M_p/M_y is relatively small, Eq. 6.10.7.1.2-2 may govern relative to Eq. 6.10.7.1.2-3. However, for most practical cases, Eq. 6.10.7.1.2-3 will control.

Interior-pier sections satisfying the requirements of Article B6.2 and for which the appropriate value of θ_{RL} from Article B6.6.2 exceeds 0.009 radians have sufficient ductility and robustness such that the redistribution of moments caused by partial yielding within the positive flexural regions is inconsequential. The value of 0.009 radians is taken as an upper bound for the potential increase in the inelastic rotations at the interior-pier sections due to positive-moment yielding. Thus, the nominal flexural resistance of positive flexural sections in continuous spans that meet these requirements is not limited due to the effect of potential moment shifting. These restrictions are often satisfied by compact-flange unstiffened or transversely-stiffened pier sections designed by Article 6.10.8 or Appendix A6 using C_b = 1.0. All current ASTM A6/A6M rolled I-shapes satisfying Eqs. B6.2.1-3, B6.2.2-1, and B6.2.4-1 meet these restrictions. All built-up sections satisfying Article B6.2 that also either have $D/b_{fc} < 3.14$ or satisfy the additional requirements of Article B6.5.1 meet these restrictions.

The Engineer is not required to redistribute moments from the pier sections in order to utilize the additional resistance in positive flexure, but only to satisfy the stated restrictions from Appendix B6 that ensure significant ductility and robustness of the adjacent pier sections. Redistribution of the pier moments is permitted in these cases, if desired, according to the provisions of Appendix B6.

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Assuming the fatigue and fracture limit state does not control, under the load combinations specified in Table 3.4.1-1 and in the absence of flange lateral bending, the permanent deflection service limit state criterion given by Eq. 6.10.4.2.2-2 will often govern the design of the bottom flange of compact composite sections in positive flexure wherever the nominal flexural resistance at the strength limit state is based on either Eq. 6.10.7.1.2-1, 6.10.7.1.2-2, or 6.10.7.1.2-3. Thus, it is prudent and expedient to initially design these types of sections to satisfy this permanent deflection service limit state criterion and then to subsequently check the nominal flexural resistance at the strength limit state according to the applicable Eq. 6.10.7.1.2-1, 6.10.7.1.2-2, or 6.10.7.1.2-3.

6.10.7.2—Noncompact Sections

6.10.7.2.1—General

At the strength limit state, the compression flange shall satisfy:

$$f_{bu} \le \phi_f F_{nc} \qquad (6.10.7.2.1\text{-}1)$$

where:

ϕ_f = resistance factor for flexure specified in Article 6.5.4.2
f_{bu} = flange stress calculated without consideration of flange lateral bending determined as specified in Article 6.10.1.6 (ksi)
F_{nc} = nominal flexural resistance of the compression–flange determined as specified in Article 6.10.7.2.2 (ksi)

The tension flange shall satisfy:

$$f_{bu} + \frac{1}{3} f_\ell \le \phi_f F_{nt} \qquad (6.10.7.2.1\text{-}2)$$

where:

f_ℓ = flange lateral bending stress determined as specified in Article 6.10.1.6 (ksi)
F_{nt} = nominal flexural resistance of the tension flange determined as specified in Article 6.10.7.2.2 (ksi)

The maximum longitudinal compressive stress in the concrete deck at the strength limit state, determined as specified in Article 6.10.1.1.1d, shall not exceed $0.6f'_c$.

C6.10.7.2.1

For noncompact sections, the compression flange must satisfy Eq. 6.10.7.2.1-1 and the tension flange must satisfy Eq. 6.10.7.2.1-2 at the strength limit state. The basis for Eq. 6.10.7.2.1-2 is explained in Article C6.10.8.1.2. For composite sections in positive flexure, lateral bending does not need to be considered in the compression–flange at the strength limit state because the flange is continuously supported by the concrete deck.

For noncompact sections, the longitudinal stress in the concrete deck is limited to $0.6f'_c$ to ensure linear behavior of the concrete, which is assumed in the calculation of the steel flange stresses. This condition is unlikely to govern except in cases involving: (1) shored construction, or unshored construction where the noncomposite steel dead load stresses are low, combined with (2) geometries causing the neutral axis of the short-term and long-term composite section to be significantly below the bottom of the concrete deck.

6.10.7.2.2—Nominal Flexural Resistance

The nominal flexural resistance of the compression flange shall be taken as:

C6.10.7.2.2

The nominal flexural resistance of noncompact composite sections in positive flexure is limited to the moment at first yield. Thus, the nominal flexural

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$$F_{nc} = R_b R_h F_{yc} \quad (6.10.7.2.2\text{-}1)$$

where:

R_b = web load-shedding factor determined as specified in Article 6.10.1.10.2
R_h = hybrid factor determined as specified in Article 6.10.1.10.1

The nominal flexural resistance of the tension flange shall be taken as:

$$F_{nt} = R_h F_{yt} \quad (6.10.7.2.2\text{-}2)$$

resistance is expressed simply in terms of the flange stress. For noncompact sections, the elastically computed stress in each flange due to the factored loads, determined in accordance with Article 6.10.1.1.1a, is compared with the yield stress of the flange times the appropriate flange-strength reduction factors.

6.10.7.3—Ductility Requirement

Compact and noncompact sections shall satisfy:

$$D_p \le 0.42\, D_t \quad (6.10.7.3\text{-}1)$$

where:

D_p = distance from the top of the concrete deck to the neutral axis of the composite section at the plastic moment (in.)
D_t = total depth of the composite section (in.)

C6.10.7.3

The ductility requirement specified in this Article is intended to protect the concrete deck from premature crushing. The limit of $D_p < 5D'$ in AASHTO (1998) corresponds to $D_p/D_t < 0.5$ for $\beta = 0.75$. The D_p/D_t ratio is lowered to 0.42 in Eq. 6.10.7.3-1 to ensure significant yielding of the bottom flange when the crushing strain is reached at the top of concrete deck for all potential cases. In checking this requirement, D_t should be computed using a lower bound estimate of the actual thickness of the concrete haunch, or may be determined conservatively by neglecting the thickness of the haunch.

6.10.8—Flexural Resistance—Composite Sections in Negative Flexure and Noncomposite Sections

6.10.8.1—General

6.10.8.1.1—Discretely Braced Flanges in Compression

At the strength limit state, the following requirement shall be satisfied:

$$f_{bu} + \frac{1}{3} f_\ell \le \phi_f F_{nc} \quad (6.10.8.1.1\text{-}1)$$

where:

ϕ_f = resistance factor for flexure specified in Article 6.5.4.2
f_{bu} = flange stress calculated without consideration of flange lateral bending determined as specified in Article 6.10.1.6 (ksi)
f_ℓ = flange lateral bending stress determined as specified in Article 6.10.1.6 (ksi)
F_{nc} = nominal flexural resistance of the flange determined as specified in Article 6.10.8.2 (ksi)

C6.10.8.1.1

Eq. 6.10.8.1.1-1 addresses the resistance of the compression flange by considering this element as an equivalent beam-column. This equation is effectively a beam-column interaction equation, expressed in terms of the flange stresses computed from elastic analysis (White and Grubb, 2005). The f_{bu} term is analogous to the axial load and the f_ℓ term is analogous to the bending moment within the equivalent beam-column member. The factor of one-third in front of the f_ℓ term in Eq. 6.10.8.1.1-1 gives an accurate linear approximation of the equivalent beam-column resistance within the limits on f_ℓ specified in Article 6.10.1.6 (White and Grubb, 2005).

Eqs. 6.10.8.1.1-1, 6.10.8.1.2-1, and 6.10.8.1.3-1 are developed specifically for checking of slender-web noncomposite sections and slender-web composite sections in negative flexure. These equations may be used as a simple conservative resistance check for other types of composite sections in negative flexure and noncomposite sections. The provisions specified in Appendix A6 may be used for composite sections in negative flexure and for noncomposite sections with compact or noncompact webs in straight bridges for which the specified minimum yield strengths of the flanges and web do not exceed 70 ksi and for which the flanges satisfy Eq. 6.10.6.2.3-2. The Engineer should give consideration to utilizing the provisions of Appendix A6 for such sections in straight bridges with compact webs; however, Appendix A6 provides only minor increases in the nominal resistance for sections in which the web slenderness approaches the noncompact web limit of Eq. 6.10.6.2.3-1.

6.10.8.1.2—Discretely Braced Flanges in Tension

At the strength limit state, the following requirement shall be satisfied:

$$f_{bu} + \frac{1}{3} f_\ell \leq \phi_f F_{nt} \qquad (6.10.8.1.2\text{-}1)$$

where:

F_{nt} = nominal flexural resistance of the flange determined as specified in Article 6.10.8.3 (ksi)

C6.10.8.1.2

Eq. 6.10.8.1.2-1 is an accurate approximation of the full plastic strength of a rectangular flange cross section subjected to combined vertical and lateral bending within the limit of Eq. 6.10.1.6-1, originally proposed by NCHRP (1999).

6.10.8.1.3—Continuously Braced Flanges in Tension or Compression

At the strength limit state, the following requirement shall be satisfied:

$$f_{bu} \leq \phi_f R_h F_{yf} \qquad (6.10.8.1.3\text{-}1)$$

6.10.8.2 Compression–Flange Flexural Resistance

6.10.8.2.1—General

Eq. 6.10.8.1.1-1 shall be satisfied for both local buckling and lateral–torsional buckling using the appropriate value of F_{nc} determined for each case as specified in Articles 6.10.8.2.2 and 6.10.8.2.3, respectively.

C6.10.8.2.1

All of the I-section compression flange flexural resistance equations of these Specifications are based consistently on the logic of identifying the two anchor points shown in Figure C6.10.8.2.1-1 for the case of uniform major-axis bending. Anchor point 1 is located at the length $L_b = L_p$ for lateral–torsional buckling or flange slenderness $b_{fc}/2t_{fc} = \lambda_{pf}$ for flange local buckling corresponding to development of the maximum potential flexural resistance, labeled as F_{max} or M_{max} in the figure, as applicable. Anchor point 2 is located at the length L_r or flange slenderness λ_{rf} for which the inelastic and elastic lateral–torsional buckling or flange local buckling resistances are the same. In Article 6.10.8, this resistance is taken as R_bF_{yr}, where F_{yr} is taken as the smaller of $0.7F_{yc}$ and F_{yw}, but not less than $0.5F_{yc}$. With the exception of hybrid sections with F_{yw} significantly smaller than F_{yc}, $F_{yr} = 0.7F_{yc}$. This limit corresponds to a nominal compression flange residual stress effect of $0.3F_{yc}$. The $0.5F_{yc}$ limit on F_{yr} avoids anomalous situations for some types of cross sections in which the inelastic buckling equation gives a larger resistance than the corresponding elastic buckling curve. Also, the $0.5F_{yc}$

limit is equivalent to the implicit value of F_{yr} used in AASHTO (1998). For $L_b > L_r$ or $b_{fc}/2t_{fc} > \lambda_{rf}$, the lateral–torsional buckling and flange local buckling resistances are governed by elastic buckling. However, the elastic flange local buckling resistance equations are not specified explicitly in these provisions since the limits of Article 6.10.2.2 preclude elastic flange local buckling for specified minimum yield strengths up to and including F_{yc} = 90 ksi. Use of the inelastic flange local buckling Eq. 6.10.8.2.2-2 is permitted for rare cases in which $b_{fc}/2t_{fc}$ can potentially exceed λ_{rf} for F_{yc} > 90 ksi.

For unbraced lengths subjected to moment gradient, the lateral–torsional buckling resistances for the case of uniform major-axis bending are simply scaled by the moment gradient modifier C_b, with the exception that the lateral–torsional buckling resistance is capped at F_{max} or M_{max}, as illustrated by the dashed line in Figure C6.10.8.2.1-1. The maximum unbraced length at which the lateral–torsional buckling resistance is equal to F_{max} or M_{max} under a moment gradient may be determined from Article D6.4.1 or D6.4.2, as applicable. The flange local buckling resistance for moment gradient cases is the same as that for the case of uniform major-axis bending, neglecting the relatively minor influence of moment gradient effects.

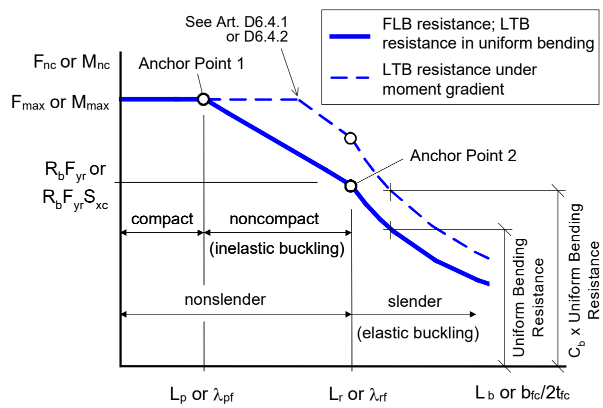

Figure C6.10.8.2.1-1—Basic Form of All I-section Compression–Flange Flexural Resistance Equations

6.10.8.2.2—Local Buckling Resistance

The local buckling resistance of the compression flange shall be taken as:

- If $\lambda_f \leq \lambda_{pf}$, then:

$$F_{nc} = R_b R_h F_{yc} \quad (6.10.8.2.2\text{-}1)$$

- Otherwise:

$$F_{nc} = \left[1 - \left(1 - \frac{F_{yr}}{R_h F_{yc}}\right)\left(\frac{\lambda_f - \lambda_{pf}}{\lambda_{rf} - \lambda_{pf}}\right)\right] R_b R_h F_{yc} \quad (6.10.8.2.2\text{-}2)$$

in which:

λ_f = slenderness ratio for the compression flange

$$= \frac{b_{fc}}{2t_{fc}} \quad (6.10.8.2.2\text{-}3)$$

λ_{pf} = limiting slenderness ratio for a compact flange

$$= 0.38\sqrt{\frac{E}{F_{yc}}} \quad (6.10.8.2.2\text{-}4)$$

λ_{rf} = limiting slenderness ratio for a noncompact flange

$$= 0.56\sqrt{\frac{E}{F_{yr}}} \quad (6.10.8.2.2\text{-}5)$$

where:

F_{yr} = compression flange stress at the onset of nominal yielding within the cross section, including residual stress effects, but not including compression flange lateral bending, taken as the smaller of $0.7F_{yc}$ and F_{yw}, but not less than $0.5F_{yc}$

R_b = web load-shedding factor determined as specified in Article 6.10.1.10.2

R_h = hybrid factor determined as specified in Article 6.10.1.10.1

C6.10.8.2.2

Eq. 6.10.8.2.2-4 defines the slenderness limit for a compact flange whereas Eq. 6.10.8.2.2-5 gives the slenderness limit for a noncompact flange. The nominal flexural resistance of a section with a compact flange is independent of the flange slenderness, whereas the flexural resistance of a section with a noncompact flange is expressed as a linear function of the flange slenderness as illustrated in Figure C6.10.8.2.1-1. The compact flange slenderness limit is the same as specified in AISC (2016b) and in AASHTO (1998). For different grades of steel, this slenderness limit is as follows:

Table C6.10.8.2.2-1—Limiting Slenderness Ratio for a Compact Flange

F_{yc} (ksi)	λ_{pf}
36.0	10.8
50.0	9.2
70.0	7.7
90.0	6.8
100.0	6.5

Eq. 6.10.8.2.2-5 is based conservatively on the more general limit given by Eq. A6.3.2-5, but with a flange local buckling coefficient of $k_c = 0.35$. With the exception of hybrid sections with $F_{yw} < 0.7F_{yc}$, the term F_{yr} in Eq. 6.10.8.2.2-5 is always equal to $0.7F_{yc}$.

6.10.8.2.3—Lateral–Torsional Buckling Resistance

For unbraced lengths in which the member is prismatic, the lateral–torsional buckling resistance of the compression flange shall be taken as:

- If $L_b \leq L_p$, then:

$$F_{nc} = R_b R_h F_{yc} \tag{6.10.8.2.3-1}$$

- If $L_p < L_b \leq L_r$, then:

$$F_{nc} = C_b \left[1 - \left(1 - \frac{F_{yr}}{R_h F_{yc}}\right)\left(\frac{L_b - L_p}{L_r - L_p}\right)\right] R_b R_h F_{yc} \leq R_b R_h F_{yc} \tag{6.10.8.2.3-2}$$

- If $L_b > L_r$, then:

$$F_{nc} = F_{cr} \leq R_b R_h F_{yc} \tag{6.10.8.2.3-3}$$

in which:

L_b = unbraced length (in.)

L_p = limiting unbraced length to achieve the nominal flexural resistance of $R_b R_h F_{yc}$ under uniform bending (in.)

$$= 1.0 r_t \sqrt{\frac{E}{F_{yc}}} \tag{6.10.8.2.3-4}$$

L_r = limiting unbraced length to achieve the onset of nominal yielding in either flange under uniform bending with consideration of compression flange residual stress effects (in.)

$$= \pi r_t \sqrt{\frac{E}{F_{yr}}} \tag{6.10.8.2.3-5}$$

C_b = moment gradient modifier. In lieu of an alternative rational analysis, C_b may be calculated as follows:

C6.10.8.2.3

Eq. 6.10.8.2.3-4 defines the compact unbraced length limit for a member subjected to uniform major-axis bending, whereas Eq. 6.10.8.2.3-5 gives the corresponding noncompact unbraced length limit. The nominal flexural resistance of a member braced at or below the compact limit is independent of the unbraced length, whereas the flexural resistance of a member braced at or below the noncompact limit is expressed as a linear function of the unbraced length as illustrated in Figure C6.10.8.2.1-1. The compact bracing limit of Eq. 6.10.8.2.3-4 is similar to the bracing requirement for use of the general compact-section flexural resistance equations and/or the *Q* formula equations in AASHTO (1998) for F_{yc} = 50 ksi. For larger F_{yc} values, it is somewhat less restrictive than the previous requirement. The limit given by Eq. 6.10.8.2.3-4 is generally somewhat more restrictive than the limit given by the corresponding L_p equation in AASHTO (1998) and AISC (2016b). The limit given by Eq. 6.10.8.2.3-4 is based on linear regression analysis within the region corresponding to the inelastic lateral–torsional buckling equation, shown qualitatively in Figure C6.10.8.2.1-1, for a wide range of data from experimental flexural tests involving uniform major-axis bending and in which the physical effective length for lateral–torsional buckling is effectively 1.0.

Note that the most economical solution is not necessarily achieved by limiting the unbraced length to L_p in order to reach the maximum flexural resistance, F_{max}, particularly if the moment gradient modifier, C_b, is taken equal to 1.0.

Eq. 6.10.8.2.3-8 is a conservative simplification of Eq. A6.3.3-8, which gives the exact beam-theory based solution for the elastic lateral–torsional buckling resistance of a doubly-symmetric I-section (Timoshenko and Gere, 1961) for the case of uniform major-axis bending when C_b is equal to 1.0 and when r_t is defined as:

$$r_t = \frac{b_{fc}}{\sqrt{12\left(\frac{h}{d} + \frac{1}{3}\frac{D_c t_w}{b_{fc} t_{fc}}\frac{D^2}{hd}\right)}} \tag{C6.10.8.2.3-1}$$

Eq. 6.10.8.2.3-8 provides an accurate to conservative estimate of the compression flange elastic lateral–torsional buckling resistance, including the effect of the distorsional flexibility of the web (White, 2008). Eq. 6.10.8.2.3-9 is a simplification of the above r_t equation obtained by assuming $D = h = d$. For sections with thick flanges, Eq. 6.10.8.2.3-9 gives an r_t value that can be as much as three to four percent conservative relative to the exact equation. Use of Eq. C6.10.8.2.3-1 is permitted for software calculations or if the Engineer requires a more precise calculation of the elastic lateral–torsional buckling resistance. The other key simplification in Eq. 6.10.8.2.3-8 is that the St. Venant torsional constant, *J,* is assumed equal to zero. This simplification is prudent for cases such as longitudinally-stiffened girders with web slenderness values approaching the maximum limit of Eq. 6.10.2.1.2-1. For these types of sections, the contribution of J to the elastic lateral–torsional buckling resistance is generally small and is likely to be reduced due to distortion of the web into an S shape and the corresponding raking of the compression–flange relative to the tension flange. However, for sections that have web slenderness values approaching the noncompact limit given by Eq. 6.10.6.2.3-1 and listed for different yield strengths in Table C6.10.1.10.2-2, the assumption of $J = 0$ is convenient but tends to be conservative. For sections below the noncompact limit, the use of the equations given in Article A6.3.3 should be considered. For typical flexural I-sections with $D/b_{fc} > 2$ and $I_{yc}/I_{yt} \geq 0.3$, the effect of this assumption on the magnitude of the noncompact bracing limit, L_r, is usually smaller than ten percent (White et al., 2001).

Eqs. 6.10.8.2.3-8 and A6.3.3-8 provide one single consistent representation of the elastic lateral–torsional buckling resistance for all types of I-section members. These equations give a conservative representation of the elastic lateral–torsional buckling resistance of composite I-section members in negative flexure since they neglect the restraint provided to the bottom compression–flange by the lateral and torsional stiffness of the deck. The effects of this restraint are reduced in general by web distortion. The benefits of this restraint are judged to not be worth the additional complexity associated with a general distorsional buckling solution, particularly if it is suspected that less than effectively fixed torsional restraint is provided to a relatively large bridge I-girder by the deck.

The Engineer should note the importance of the web term $D_c t_w$ within Eq. 6.10.8.2.3-9. Prior specifications have often used the radius of gyration of only the compression flange, $r_{yc} = b_{fc} / \sqrt{12}$, within the design equations for lateral–torsional buckling. This approximation can lead to significant unconservative predictions relative to experimental and refined finite-element results. The web term in Eq. 6.10.8.2.3-9 accounts for the destabilizing effects of the flexural compression within the web.

If $D_c t_w / b_{fc} t_{fc}$ in Eq. 6.10.8.2.3-9 is taken as a representative value of 2.0, this equation reduces to $0.22 b_{fc}$. Based on this assumption and F_{yc} = 50 ksi, the compact bracing limit is $L_p = 5.4 b_{fc}$ and the noncompact bracing limit given by Eq. 6.10.8.2.3-5 simplifies to $L_r = 20 b_{fc}$. Based on these same assumptions, the equations of Articles B6.2.4 and D6.4 give corresponding limits on L_b that are generally larger than 5.4 b_{fc}. The limit given in Article B6.2.4 is sufficient to permit moment redistribution at interior-pier sections of continuous-span members. The limit given in Article D6.4 is sufficient to develop F_{max} or M_{max} shown in Figure C6.10.8.2.1-1 in cases involving a moment gradient along the unbraced length for which $C_b > 1.0$.

The effect of the variation in the moment along the length between brace points is accounted for by the moment gradient modifier, C_b. C_b has a base value of 1.0 when the moment and the corresponding flange

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compressive major-axis bending stress are constant over the unbraced length. C_b may be conservatively taken equal to 1.0 for all cases, with the exception of some unusual circumstances involving no cross-bracing within the span or cantilever beams with significant top-flange loading as discussed below.

The procedure for calculation of C_b retains Eq. 6.10.8.2.3-7 from the previous specifications; however, the definition of when C_b is to be taken equal to 1.0 and the specific calculation of the terms f_1 and f_2 in Eq. 6.10.8.2.3-7 have been modified to remove ambiguities and to address a number of potentially important cases where the prior C_b calculations are significantly unconservative relative to more refined solutions. One specific example is a simply-supported member supporting its own weight as well as a uniform transverse load, but braced only at its ends and its mid-span. This ideal case is representative of potential erection conditions in which the number of cross-frames within the superstructure is minimal and the superstructure is being considered in its noncomposite condition prior to hardening of a cast-in-place concrete slab. For this case, the prior specifications give a C_b value of 1.75 whereas the more accurate equations from AISC (1999) give a C_b value of 1.30. The smaller C_b value of 1.30 is due to the parabolic shape of the moment diagram, and the fact that the flange compression is significantly larger within the unbraced lengths than the linear variation implicitly assumed in the prior application of Eq. 6.10.8.2.3-7.

The procedure for calculation of C_b in these provisions addresses the above issues by utilizing the stress due to the factored loads at the middle of the unbraced length of the flange under consideration, f_{mid}. If f_{mid} is greater than or equal to the largest compressive stress in the flange due to the factored loads at either end of the unbraced length, f_2, C_b is taken equal to 1.0. Also, in rare situations where the flange stress is zero or tensile at both ends of its unbraced length, for which f_2 is defined as zero, C_b is taken equal to 1.0. This type of situation occurs only for members with very large unbraced lengths such as simply-supported or continuous spans with no cross-bracing within the span. For unbraced cantilevers, C_b is also taken equal to 1.0, consistent with AASHTO (1998) and AISC (2016b).

For all other cases, significant beneficial and calculable moment gradient effects exist. In these cases, Eq. 6.10.8.2.3-7 requires the approximation of the stress variation along the unbraced length as the most critical of: (1) a line that passes through f_2 and f_{mid} or (2) a line that passes between f_2 and the calculated stress in the flange under consideration at the opposite end of the unbraced length, f_0, whichever produces the smaller value of C_b. The intercept of this most critical assumed linear stress variation at the opposite end from f_2 is denoted as f_1. For the specific example cited above, this procedure gives a C_b value of 1.30, which is identical to the C_b value predicted by the more refined AISC (2016b) equation. In all cases

- For unbraced cantilevers and for members where

  $f_{mid}/f_2 > 1$ or $f_2 = 0$

$$C_b = 1.0 \quad (6.10.8.2.3\text{-}6)$$

- For all other cases:

$$C_b = 1.75 - 1.05\left(\frac{f_1}{f_2}\right) + 0.3\left(\frac{f_1}{f_2}\right)^2 \leq 2.3 \quad (6.10.8.2.3\text{-}7)$$

F_{cr} = elastic lateral–torsional buckling stress (ksi)

$$= \frac{C_b R_b \pi^2 E}{\left(\frac{L_b}{r_t}\right)^2} \quad (6.10.8.2.3\text{-}8)$$

r_t = effective radius of gyration for lateral–torsional buckling (in.)

$$= \frac{b_{fc}}{\sqrt{12\left(1+\frac{1}{3}\frac{D_c t_w}{b_{fc} t_{fc}}\right)}} \quad (6.10.8.2.3\text{-}9)$$

where:

F_{yr} = compression flange stress at the onset of nominal yielding within the cross section, including residual stress effects, but not including compression flange lateral bending, taken as the smaller of $0.7F_{yc}$ and F_{yw}, but not less than $0.5F_{yc}$

D_c = depth of the web in compression in the elastic range (in.). For composite sections, D_c shall be determined as specified in Article D6.3.1.

f_{mid} = stress without consideration of lateral bending at the middle of the unbraced length of the flange under consideration, calculated from the moment envelope value that produces the largest compression at this point, or the smallest tension if this point is never in compression (ksi). f_{mid} shall be due to the factored loads and shall be taken as positive in compression and negative in tension.

f_0 = stress without consideration of lateral bending at the brace point opposite to the one corresponding to f_2, calculated from the moment envelope value that produces the largest

where f_{mid} is smaller in magnitude than the average of f_0 and f_2, or when the moment diagram or envelope along the entire length between brace points is concave in shape, f_1 and f_2 in Eq. 6.10.8.2.3-7 are always equal to the stresses at the ends of the unbraced length in the flange under consideration; that is, $f_1 = f_0$. Sample illustrations of the calculation of the C_b factor for various cases are provided at the end of Appendix C6.

For unbraced lengths where the member consists of monosymmetric noncomposite I-sections and is subject to reverse curvature bending, the lateral–torsional buckling resistance must be checked in general for both flanges, unless the top flange is considered to be continuously braced. Since the flanges are of different sizes in these types of sections, the lateral–torsional buckling resistance may be governed by compression in the smaller flange, even though this compressive stress may be smaller than the maximum compression in the larger flange. The specified approach generally produces accurate to conservative values of C_b for these cases. For highly monosymmetric sections and reverse curvature bending, the values of C_b between 1.75 and 2.3 obtained using these provisions are often significantly conservative relative to refined calculations of the lateral–torsional buckling resistance, such as those provided by Kitipornchai and Trahair (1986). However, these provisions are less conservative than the resistances estimated by a refinement of the AISC (2016b) C_b equation given by Helwig et al. (1997) when the transverse loading effects are small and the variation of the moment along the unbraced length is approximately linear. For other cases involving significant transverse loading effects, the refined AISC equation recommended by Helwig et al. (1997) gives more accurate and less conservative results for unbraced lengths where the member is subjected to reverse curvature bending. The top flange of composite I-sections in unbraced lengths where the member is subject to reverse curvature bending need not be checked for lateral–torsional buckling since the flange is continuously braced.

Strict application of the C_b provisions would require the consideration of the concurrent moments along the unbraced length. This would necessitate the calculation of:

(1) the maximum possible value of f_2 at the brace point with the higher compressive stress using the critical moment envelope value, along with calculation of f_{mid} and f_0 using the concurrent moments, and

(2) the maximum possible compressive value of f_{mid} using the critical moment envelope value, along with the calculation of f_0 and f_2 using the concurrent moments.

However, since concurrent moments are normally not tracked in the analysis, it is convenient and always conservative to use the worst-case moment values to compute the above stresses. The worst-case moment for calculation of f_2 is the critical envelope value, or the moment causing the largest value of f_2 in the flange under compression at this point in the flange under consideration, or the smallest tension if this point is never in compression (ksi). f_0 shall be due to the factored loads and shall be taken as positive in compression and negative in tension.

f_1 = stress without consideration of lateral bending at the brace point opposite to the one corresponding to f_2, calculated as the intercept of the most critical assumed linear stress variation passing through f_2 and either f_{mid} or f_0, whichever produces the smaller value of C_b (ksi). f_1 may be determined as follows:

- When the variation in the moment along the entire length between the brace points is concave in shape:

$$f_1 = f_0 \quad (6.10.8.2.3\text{-}10)$$

- Otherwise:

$$f_1 = 2f_{mid} - f_2 \geq f_0 \quad (6.10.8.2.3\text{-}11)$$

f_2 = except as noted below, largest compressive stress without consideration of lateral bending at either end of the unbraced length of the flange under consideration, calculated from the critical moment envelope value (ksi). f_2 shall be due to the factored loads and shall be taken as positive. If the stress is zero or tensile in the flange under consideration at both ends of the unbraced length, f_2 shall be taken as zero.

R_b = web load-shedding factor determined as specified in Article 6.10.1.10.2

R_h = hybrid factor determined as specified in Article 6.10.1.10.1

For unbraced lengths where the member consists of noncomposite monosymmetric sections and is subject to reverse curvature bending, the lateral–torsional buckling resistance shall be checked for both flanges, unless the top flange is considered to be continuously braced.

For unbraced lengths in which the member is nonprismatic, the lateral–torsional buckling resistance of the compression flange F_{nc} at each section within the unbraced length may be taken as the smallest resistance within the unbraced length under consideration determined from Eq. 6.10.8.2.3-1, 6.10.8.2.3-2, or 6.10.8.2.3-3, as applicable, assuming the unbraced length is prismatic. The moment gradient modifier, C_b, shall be taken equal to 1.0 in this case and L_b shall not be modified by an effective length factor.

For unbraced lengths containing a transition to a smaller section at a distance less than or equal to 20 percent of the unbraced length from the brace point with the smaller moment, the lateral–torsional buckling resistance may be determined assuming the transition to the smaller section does not exist provided the lateral

consideration. The worst-case moments used to compute f_0 and f_{mid} are the values obtained from the moment envelopes that produce the largest compressive stress, or the smallest tensile stress if the point is never in compression, within the flange under consideration at each of these locations. The use of the worst-case moments to compute f_2, f_{mid} and f_0 is always conservative since it can be shown that a more critical stress distribution along the unbraced length can never exist for all possible concurrent loadings. This includes any potential condition in which the stress is smaller at the f_2 or f_{mid} locations, but in which the moment gradient is also smaller thus producing a smaller value of C_b. Furthermore, the use of the concurrent moments to compute f_0 and f_{mid} for the loading that gives the largest value of f_2 always would result in a larger value of C_b for this specific loading. Similarly, the use of the concurrent moments to compute f_2 and f_0 for the loading that produces the largest compressive value of f_{mid} always would result in a larger value of C_b for this specific loading.

The preceding guidelines are also applicable when calculating C_b for compact and noncompact web sections designed by Article A6.3.3. The use of the compression flange major-axis bending stresses for calculating C_b is strongly recommended for sections designed by Article 6.10.8 since this practice better reflects the fact that the dead and live load bending moments due to the factored loads are applied to different sections in composite girders. However, for convenience, the ratio of the major-axis bending moments at the brace points may be used in lieu of the ratio of the compression flange stresses if it is felt in the judgment of the Engineer that the use of these alternative ratios does not have a significant effect on the final calculated value of C_b. For compact and noncompact web sections designed by Article A6.3.3, it is specified that the major-axis bending moments be used when computing C_b. Moments are used in Eq. A6.3.3-7 because the overall effect of applying the moments to the different sections is less critical for these types of sections.

Where C_b is greater than 1.0, indicating the presence of a significant beneficial moment gradient effect, the lateral–torsional buckling resistances may alternatively be calculated by the equivalent procedures specified in Article D6.4.1. Both the equations in this Article and in Article D6.4.1 permit F_{max} in Figure C6.10.8.2.1-1 to be reached at larger unbraced lengths when C_b is greater than 1.0. The procedures in Article D6.4.1 allow the Engineer to focus directly on the maximum unbraced length at which the flexural resistance is equal to F_{max}. The use of these equivalent procedures is strongly recommended when C_b values greater than 1.0 are utilized in the design.

Although the calculation of C_b greater than 1.0 in general can result in a dependency of the flexural resistance on the applied loading, and hence subsequent difficulties in load rating, a C_b value only slightly greater than 1.0 is sufficient in most cases to develop the

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moment of inertia of the flange or flanges of the smaller section is equal to or larger than one-half the corresponding value in the larger section.

maximum flexural resistance F_{max}. As long as the combination of the brace spacing and $C_b > 1.0$ is sufficient to develop F_{max}, the flexural resistance is independent of the applied loading. Therefore, when $C_b >$ 1.0 is used, it is recommended that the unbraced lengths, L_b, at critical locations be selected such that this condition is satisfied in the final constructed condition. The provisions in this Article tend to give values of C_b that are accurate to significantly conservative. Therefore, if the above guidelines are followed in design, it is unlikely that the flexural resistance would differ from F_{max} in any rating situation, particularly if the Engineer was to use a more refined calculation of C_b for the rating calculations. Other more refined formulations for C_b may be found in Ziemian (2010).

The C_b equations in these provisions and in AISC (2016b) both neglect the effect of the location of the applied load relative to the mid-height of the section. For unusual situations with no intermediate cross-bracing and for unbraced cantilevers with significant loading applied at the level of the top flange, the Engineer should consider including load-height effects within the calculation of C_b. In these cases, the associated C_b values can be less than 1.0. Ziemian (2010) gives equations for consideration of load-height effects in simple or continuous spans, and Dowswell (2002) gives solutions considering these effects in unbraced cantilevers. When $C_b < 1.0$, F_n can be smaller than F_{max} in Figure C6.10.8.2.1-1 even when L_b is less than or equal to L_p. Therefore, for $C_b < 1.0$, the resistance should be calculated from Eq. 6.10.8.2.3-2 for L_b less than or equal to L_r.

For rehabilitation design or in extraordinary circumstances, the Engineer may consider modifying L_b by an elastic effective length factor for lateral–torsional buckling. Ziemian (2010) and Nethercot and Trahair (1976) present a simple hand method that may be used for this calculation.

Ziemian (2010) provides general guidelines for stability design of bracing systems. In past practice, points of contraflexure sometimes have been considered as brace points when the influence of moment gradient was not included in the lateral–torsional buckling resistance equations. In certain cases, this practice can lead to a substantially unconservative estimate of the flexural resistance. These Specifications do not intend for points of contraflexure to be considered as brace points. The influence of moment gradient may be accounted for correctly through the use of C_b and the effect of restraint from adjacent unbraced segments may be accounted for by using an effective length factor less than 1.0. Suggested values of C_b for compact web sections, subject to reverse curvature bending with no intermediate bracing or bracing on only one flange, are provided in Yura and Helwig (2010), and in the commentary to Article F1 of AISC (2016b).

For the case of uniform bending, the reduction in the elastic lateral–torsional buckling resistance due to a transition to a smaller section is approximately five percent when the transition is placed at 20 percent of the unbraced length from one of the brace points and the lateral moment of inertia of the flange in the smaller section is set at one-half of the corresponding value in the larger section (Carskaddan and Schilling, 1974). For moment gradient cases in which the larger bending moment occurs within the larger section, and/or where the section transition is placed closer to the brace point, and/or where the lateral moment of inertia of the flange of the smaller section is larger than one-half of the corresponding value in the larger section, the reduction in the lateral–torsional buckling resistance is less than five percent. Since section transitions are typically placed within regions having a significant moment gradient, the effect of the section transition on the lateral–torsional buckling resistance may be neglected whenever the stated conditions are satisfied. For a case with more than one transition, any transition located within 20 percent of the unbraced length from the brace point with the smaller moment, and with the lateral moment of inertia of the flange or flanges of the smaller section equal to or larger than one-half the corresponding value in the larger section, may be ignored and the lateral–torsional buckling resistance of the remaining nonprismatic unbraced length may then be computed as the smallest resistance based on the remaining sections.

For unbraced lengths containing a transition to a smaller section at a distance greater than 20 percent of the unbraced length from the brace point with the smaller moment, the lateral–torsional buckling resistance should be taken as the smallest resistance, F_{nc}, within the unbraced length under consideration. This approximation is based on replacing the nonprismatic member with an equivalent prismatic member. The cross section of the equivalent member that gives the correct lateral–torsional buckling resistance is generally some weighted average of all the cross sections along the unbraced length. If the cross section within the unbraced length that gives the smallest uniform bending resistance is used, and the calculated resistance is not exceeded at any section along the unbraced length, a conservative solution is obtained. A suggested procedure to provide a more refined estimate of the lateral–torsional buckling resistance for this case is presented in Grubb and Schmidt (2012).

To avoid a significant reduction in the lateral–torsional buckling resistance, flange transitions can be located within 20 percent of the unbraced length from the brace point with the smaller moment, given that the lateral moment of inertia of the flange or flanges of the smaller section is equal to or larger than one-half of the corresponding value in the larger section.

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6.10.8.3—Flexural Resistance Based on Tension Flange Yielding

The nominal flexural resistance based on tension flange yielding shall be taken as:

$$F_{nt} = R_h F_{yt} \quad (6.10.8.3\text{-}1)$$

where:

R_h = hybrid factor determined as specified in Article 6.10.1.10.1

C6.10.8.3

For sections in which $M_{yt} > M_{yc}$, Eq. 6.10.8.3-1 does not control and tension flange yielding need not be checked, where M_{yc} and M_{yt} are the yield moments with respect to the compression and tension flange, respectively, determined as specified in Article D6.2.

6.10.9—Shear Resistance

6.10.9.1—General

At the strength limit state, straight and curved web panels shall satisfy:

$$V_u \leq \phi_v V_n \quad (6.10.9.1\text{-}1)$$

where:

ϕ_v = resistance factor for shear specified in Article 6.5.4.2
V_n = nominal shear resistance determined as specified in Articles 6.10.9.2 and 6.10.9.3 for unstiffened and stiffened webs, respectively (kip)
V_u = factored shear in the web at the section under consideration (kip)

Transverse intermediate stiffeners shall be designed as specified in Article 6.10.11.1. Longitudinal stiffeners shall be designed as specified in Article 6.10.11.3.

Transverse intermediate stiffeners shall be required wherever the factored shear in the web, V_u, exceeds the factored shear resistance, $\phi_v V_n$, for an unstiffened web, where V_n is determined as specified in Article 6.10.9.2.

Interior web panels of nonhybrid and hybrid I-shaped members:

- without a longitudinal stiffener and with a transverse stiffener spacing not exceeding 3*D*, or
- with one or more longitudinal stiffeners and with a transverse stiffener spacing not exceeding 2*D*

shall be considered stiffened, and the provisions of Article 6.10.9.3 shall apply. Otherwise, the interior web panel shall be considered unstiffened, and the provisions of Article 6.10.9.2 shall apply.

End web panels of nonhybrid and hybrid I-shaped members:

- With or without one or more longitudinal stiffeners and with a transverse stiffener spacing not exceeding 1.5*D*

C6.10.9.1

This Article applies to:

- sections without stiffeners,
- sections with transverse stiffeners only, and
- sections with both transverse and longitudinal stiffeners.

A flowchart for determining the shear resistance of I-sections is shown below.

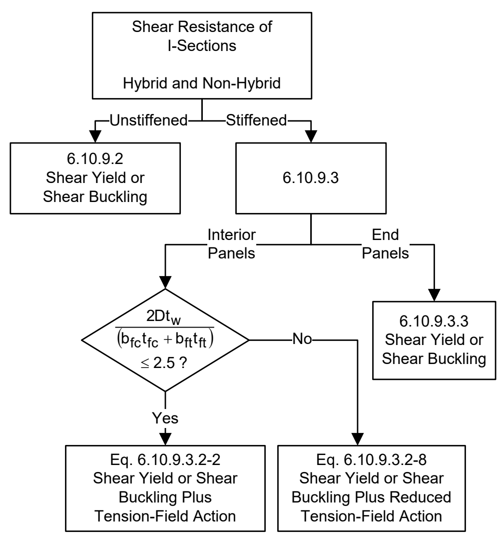

Figure C6.10.9.1-1—Flowchart for Shear Design of I-Sections

Unstiffened and stiffened interior web panels are defined according to the maximum transverse stiffener spacing requirements specified in this Article.

The nominal shear resistance of unstiffened web panels in both nonhybrid and hybrid members is defined by either shear yielding or shear buckling, depending on the web slenderness ratio, as specified in Article 6.10.9.2.

shall be considered stiffened, and the provisions of Article 6.10.9.3.3 shall apply. Otherwise, the end web panel shall be considered unstiffened, and the provisions of Article 6.10.9.2 shall apply.

The nominal shear resistance of stiffened interior web panels of both nonhybrid and hybrid members, where the section along the entire panel is proportioned to satisfy Eq. 6.10.9.3.2-1, is defined by the shear-yielding resistance or the sum of the shear-buckling resistance and the postbuckling resistance from tension-field action, as specified in Article 6.10.9.3.2. Otherwise, the shear resistance is taken as the shear resistance given by Eq. 6.10.9.3.2-8. Previous Specifications did not recognize the potential for web panels of hybrid members to develop postbuckling resistance due to tension-field action. The applicability of these provisions to the shear strength of curved nonhybrid and hybrid webs is addressed by Zureick et al. (2002), White et al. (2001), White and Barker (2008), White et al. (2008), and Jung and White (2006).

For nonhybrid and hybrid members, the nominal shear resistance of end panels in stiffened webs is defined by either shear yielding or shear buckling, as specified in Article 6.10.9.3.3.

6.10.9.2—Nominal Resistance of Unstiffened Webs

The nominal shear resistance of unstiffened webs shall be taken as the shear-yielding or shear-buckling resistance as follows:

$$V_n = V_{cr} = CV_p \qquad (6.10.9.2\text{-}1)$$

in which:

$$V_p = 0.58\,F_{yw}Dt_w \qquad (6.10.9.2\text{-}2)$$

where:

C = ratio of the shear-buckling resistance to the shear yield strength determined by Eqs. 6.10.9.3.2-4, 6.10.9.3.2-5 or 6.10.9.3.2-6 as applicable, with the shear-buckling coefficient, k, taken equal to 5.0
V_{cr} = shear-yielding or shear-buckling resistance (kip)
V_n = nominal shear resistance (kip)
V_p = plastic shear force (kip)

C6.10.9.2

The consideration of tension-field action (Basler, 1961) is not permitted for unstiffened web panels. The shear-yielding or shear-buckling resistance is calculated as the product of the constant C specified in Article 6.10.9.3.2 times the plastic shear force, V_p, given by Eq. 6.10.9.2-2. The plastic shear force is equal to the web area times the assumed shear yield strength of $F_{yw}/\sqrt{3}$. The shear-buckling coefficient, k, to be used in calculating the constant C is defined as 5.0 for unstiffened web panels, which is a conservative approximation of the exact value of 5.35 for an infinitely long strip with simply-supported edges (Timoshenko and Gere, 1961). The nominal shear resistance is equal to the shear-yielding resistance when C is equal to 1.0; otherwise, the nominal shear resistance is equal to the shear-buckling resistance.

6.10.9.3—Nominal Resistance of Stiffened Webs

6.10.9.3.1—General

The nominal shear resistance of transversely or transversely and longitudinally-stiffened interior web panels shall be taken as the shear-yielding resistance or the sum of the shear-buckling resistance and the postbuckling shear resistance due to tension-field action as specified in Article 6.10.9.3.2. The nominal shear resistance of transversely or transversely and

C6.10.9.3.1

Longitudinal stiffeners divide a web panel into subpanels. In Cooper (1967), the shear resistance of the entire panel is taken as the sum of the shear resistance of the subpanels. However, the contribution to the shear resistance of a single longitudinal stiffener located at its optimum position for flexure is relatively small. Thus, it is conservatively specified that the influence of the longitudinally-stiffened end web panels shall be taken as the shear-yielding or shear-buckling resistance as specified in Article 6.10.9.3.3. The total web depth, D, shall be used in determining the nominal shear resistance of web panels with longitudinal stiffeners. The required transverse stiffener spacing shall be calculated using the maximum shear in a panel.

Stiffeners shall satisfy the requirements specified in Article 6.10.11.

longitudinal stiffener be neglected in computing the nominal shear resistance of the web plate.

6.10.9.3.2—Interior Panels

The nominal shear resistance of an interior web panel complying with the provisions of Article 6.10.9.1, and with the section along the entire panel proportioned such that:

$$\frac{2Dt_w}{\left(b_{fc}t_{fc}+b_{ft}t_{ft}\right)} \leq 2.5 \qquad (6.10.9.3.2\text{-}1)$$

shall be taken as:

$$V_n = V_p\left[C + \frac{0.87(1-C)}{\sqrt{1+\left(\frac{d_o}{D}\right)^2}}\right] \qquad (6.10.9.3.2\text{-}2)$$

in which:

$$V_p = 0.58F_{yw}Dt_w \qquad (6.10.9.3.2\text{-}3)$$

where:

b_{fc} = width of the compression flange (in.)
b_{ft} = width of the tension flange (in.)
t_{fc} = thickness of the compression flange (in.)
t_{ft} = thickness of the tension flange (in.)
t_w = web thickness (in.)
d_o = transverse stiffener spacing (in.)
V_n = nominal shear resistance of the web panel (kip)
V_p = plastic shear force (kip)
C = ratio of the shear-buckling resistance to the shear yield strength

The ratio, C, shall be determined as specified below:

- If $\frac{D}{t_w} \leq 1.12\sqrt{\frac{Ek}{F_{yw}}}$, then:

$$C = 1.0 \qquad (6.10.9.3.2\text{-}4)$$

C6.10.9.3.2

Stiffened interior web panels of nonhybrid and hybrid members satisfying Eq. 6.10.9.3.2-1 are capable of developing postbuckling shear resistance due to tension-field action (Basler, 1961; White et al., 2004). This action is analogous to that of the tension diagonals of a Pratt truss. The nominal shear resistance of these panels can be computed by summing the contributions of beam action and post-buckling tension-field action. The resulting expression is given in Eq. 6.10.9.3.2-2, where the first term in the bracket relates to either the shear yield or shear-buckling force and the second term relates to the postbuckling tension-field force. If Eq. 6.10.9.3.2-1 is not satisfied, the total area of the flanges within the panel is small relative to the area of the web and the full postbuckling resistance generally cannot be developed (White et al., 2004). However, it is conservative in these cases to use the postbuckling resistance given by Eq. 6.10.9.3.2-8. Eq. 6.10.9.3.2-8 gives the solution neglecting the increase in stress within the wedges of the web panel outside of the tension band implicitly included within the Basler model (Gaylord, 1963; Salmon and Johnson, 1996).

Within the restrictions specified by Eqs. 6.10.9.3.2-1 and 6.10.2.2-2 in general, and Article 6.10.9.3.1 for longitudinally-stiffened I-girders in particular, and provided that the maximum moment within the panel is utilized in checking the flexural resistance, White et al. (2004) shows that the equations of these Specifications sufficiently capture the resistance of a reasonably comprehensive body of experimental test results without the need to consider moment–shear interaction. In addition, the additional shear resistance and anchorage of tension field action provided by a composite deck are neglected within the shear resistance provisions of these Specifications. Also, the maximum moment and shear envelope values are typically used for design, whereas the maximum concurrent moment and shear values tend to be less critical. These factors provide some additional margin of conservatism beyond the sufficient level of safety obtained if these factors do not exist. Therefore, previous provisions related to the effects of moment–shear interaction are not required in these Specifications.

The coefficient, C, is equal to the ratio of the elastic buckling stress of the panel, computed assuming simply-supported boundary conditions, to the shear yield

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- If $1.12\sqrt{\frac{Ek}{F_{yw}}} < \frac{D}{t_w} \leq 1.40\sqrt{\frac{Ek}{F_{yw}}}$, then:

$$C = \frac{1.12}{\frac{D}{t_w}}\sqrt{\frac{Ek}{F_{yw}}} \quad (6.10.9.3.2\text{-}5)$$

- If $\frac{D}{t_w} > 1.40\sqrt{\frac{Ek}{F_{yw}}}$, then:

$$C = \frac{1.57}{\left(\frac{D}{t_w}\right)^2}\left(\frac{Ek}{F_{yw}}\right) \quad (6.10.9.3.2\text{-}6)$$

in which:

k = shear-buckling coefficient

$$= 5 + \frac{5}{\left(\frac{d_o}{D}\right)^2} \quad (6.10.9.3.2\text{-}7)$$

Otherwise, the nominal shear resistance shall be taken as follows:

$$V_n = V_p\left[C + \frac{0.87(1-C)}{\left(\sqrt{1+\left(\frac{d_o}{D}\right)^2} + \frac{d_o}{D}\right)}\right] \quad (6.10.9.3.2\text{-}8)$$

strength assumed to equal $F_{yw}/\sqrt{3}$. Eq. 6.10.9.3.2-6 is applicable only for C values not exceeding 0.8 (Basler, 1961). Above 0.8, C values are given by Eq. 6.10.9.3.2-5 until a limiting slenderness ratio is reached where the shear-buckling stress is equal to the shear yield strength and C = 1.0. Eq. 6.10.9.3.2-7 for the shear-buckling coefficient is a simplification of two exact equations for k that depend on the panel aspect ratio. The coefficients within Eqs. 6.10.9.3.2-4 through 6.10.9.3.2-6 have been modified slightly from the values given in previous Specifications to correct minor round-off errors.

Because the slenderness of webs without longitudinal stiffeners is limited to 150 according to the provisions of Article 6.10.2.1.1, the separate handling requirement given in previous Specifications for web panels without longitudinal stiffeners is not required and is omitted in these Specifications.

6.10.9.3.3—End Panels

The nominal shear resistance of a stiffened web end panel shall be taken as:

$$V_n = V_{cr} = CV_p \quad (6.10.9.3.3\text{-}1)$$

in which:

$$V_p = 0.58F_{yw}Dt_w \quad (6.10.9.3.3\text{-}2)$$

where:

C = ratio of the shear-buckling resistance to the shear yield strength determined by Eqs. 6.10.9.3.2-4, 6.10.9.3.2-5, or 6.10.9.3.2-6 as applicable

V_{cr} = shear-yielding or shear-buckling resistance (kip)

V_p = plastic shear force (kip)

C6.10.9.3.3

The shear in stiffened end panels adjacent to simple supports is limited to either the shear-yielding or shear-buckling resistance given by Eq. 6.10.9.3.3-1 in order to provide an anchor for the tension field in adjacent interior panels. The shear-buckling coefficient, k, to be used in determining the constant C in Eq. 6.10.9.3.3-1 is to be calculated based on the spacing from the support to the first stiffener adjacent to the support, which may not exceed $1.5D$.

The transverse stiffener spacing for stiffened end panels with or without longitudinal stiffeners shall not exceed $1.5D$.

6.10.10—Shear Connectors

6.10.10.1—General

In composite sections, stud or channel shear connectors shall be provided at the interface between the concrete deck and the steel section to resist the interface shear.

Simple span composite bridges shall be provided with shear connectors throughout the length of the span.

Straight continuous composite bridges should normally be provided with shear connectors throughout the length of the bridge. In the negative flexure regions, shear connectors shall be provided where the longitudinal reinforcement is considered to be a part of the composite section. Otherwise, shear connectors need not be provided in negative flexure regions, but additional connectors shall be placed in the region of the points of permanent load contraflexure as specified in Article 6.10.10.3.

Where shear connectors are omitted in negative flexure regions, the longitudinal reinforcement shall be extended into the positive flexure region as specified in Article 6.10.1.7.

Curved continuous composite bridges shall be provided with shear connectors throughout the length of the bridge.

C6.10.10.1

Shear connectors help control cracking in regions of negative flexure where the deck is subject to tensile stress and has longitudinal reinforcement.

Shear connectors are to be provided in regions of negative flexure in curved continuous bridges because torsional shear exists and is developed in the full composite section along the entire bridge. For bridges containing one or more curved segments, the effects of curvature usually extend beyond the curved segment. Therefore, it is conservatively specified that shear connectors be provided along the entire length of the bridge in this case as well.

6.10.10.1.1—Types

Stud and channel shear connectors shall be designed by the provisions of this Article.

Shear connectors should be of a type that permits a thorough compaction of the concrete to ensure that their entire surfaces are in contact with the concrete. The connectors shall be capable of resisting both horizontal and vertical movement between the concrete and the steel.

The ratio of the height to the diameter of a stud shear connector shall not be less than 4.0.

Channel shear connectors shall have fillet welds not smaller than 0.1875 in. placed along the heel and toe of the channel.

6.10.10.1.2—Pitch

The pitch of the shear connectors shall be determined to satisfy the fatigue limit state, as specified in Article 6.10.10.2 and 6.10.10.3. The resulting number of

C6.10.10.1.2

At the fatigue limit state, shear connectors are designed for the range of live load shear between the deck and top flange of the girder. In straight girders, the shear range shear connectors shall not be less than the number required to satisfy the strength limit state as specified in Article 6.10.10.4.

The pitch, p, of shear connectors shall satisfy:

$$p \leq \frac{nZ_r}{V_{sr}} \quad (6.10.10.1.2\text{-}1)$$

in which:

V_{sr} = horizontal fatigue shear range per unit length (kip/in.)

$$= \sqrt{\left(V_{fat}\right)^2 + \left(F_{fat}\right)^2} \quad (6.10.10.1.2\text{-}2)$$

V_{fat} = longitudinal fatigue shear range per unit length (kip/in.)

$$= \frac{V_f Q}{I} \quad (6.10.10.1.2\text{-}3)$$

F_{fat} = radial fatigue shear range per unit length (kip/in.) taken as the larger of either:

$$F_{fat1} = \frac{A_{bot}\sigma_{flg}\ell}{wR} \quad (6.10.10.1.2\text{-}4)$$

or:

$$F_{fat2} = \frac{F_{rc}}{w} \quad (6.10.10.1.2\text{-}5)$$

where:

σ_{flg} = range of longitudinal fatigue stress in the bottom flange without consideration of flange lateral bending (ksi)
A_{bot} = area of the bottom flange (in.^{2})
F_{rc} = net range of cross-frame or diaphragm force at the top flange (kip)
I = moment of inertia of the short-term composite section (in.^{4})
ℓ = distance between brace points (ft)
n = number of shear connectors in a cross section
p = pitch of shear connectors along the longitudinal axis (in.)
Q = first moment of the transformed short-term area of the concrete deck about the neutral axis of the short-term composite section (in.^{3})
R = minimum girder radius within the panel (ft)
V_f = vertical shear force range under the applicable fatigue load combination specified in Table 3.4.1-1 with the fatigue live load taken as specified in Article 3.6.1.4 (kip)

normally is due to only major-axis bending if torsion is ignored. Curvature, skew and other conditions may cause torsion, which introduces a radial component of the horizontal shear. These provisions provide for consideration of both of the components of the shear to be added vectorially according to Eq. 6.10.10.1.2-2.

The parameters I and Q should be determined using the deck within the effective flange width. However, in negative flexure regions of straight girders only, the parameters I and Q may be determined using the longitudinal reinforcement within the effective flange width for negative moment, unless the concrete deck is considered to be effective in tension for negative moment in computing the range of the longitudinal stress, as permitted in Article 6.6.1.2.1.

The maximum longitudinal fatigue shear range, V_{fat}, is produced by placing the fatigue live load immediately to the left and to the right of the point under consideration. For the load in these positions, positive moments are produced over significant portions of the girder length. Thus, the use of the full composite section, including the concrete deck, is reasonable for determining the stiffness used to determine the shear range along the entire span. Also, the horizontal shear force in the deck is most often considered to be effective along the entire span in the analysis. To satisfy this assumption, the shear force in the deck should be developed along the entire span. For straight girders, an option is permitted to ignore the concrete deck in computing the shear range in regions of negative flexure, unless the concrete is considered to be effective in tension in computing the range of the longitudinal stress, in which case the shear force in the deck must be developed. If the concrete is ignored in these regions, the maximum pitch specified at the end of this Article must not be exceeded.

The radial shear range, F_{fat}, typically is determined for the fatigue live load positioned to produce the largest positive and negative major-axis bending moments in the span. Therefore, vectorial addition of the longitudinal and radial components of the shear range is conservative because the longitudinal and radial shears are not produced by concurrent loads.

Eq. 6.10.10.1.2-4 may be used to determine the radial fatigue shear range resulting from the effect of any curvature between brace points. The shear range is taken as the radial component of the maximum longitudinal range of force in the bottom flange between brace points, which is used as a measure of the major-axis bending moment. The radial shear range is distributed over an effective length of girder flange, w. At end supports, w is halved. Eq. 6.10.10.1.2-4 gives the same units as V_{fat}.

Eq. 6.10.10.1.2-5 will typically govern the radial fatigue shear range where torsion is caused by effects other than curvature, such as skew. Eq. 6.10.10.1.2-5 is most likely to control when discontinuous cross-frame or diaphragm lines are used in conjunction with skew angles exceeding 20 degrees in either a straight or horizontally-curved bridge. For all other cases, F_{rc} can be taken equal

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w = effective length of deck (in.) taken as 48.0 in., except at end supports where w may be taken as 24.0 in.

Z_r = shear fatigue resistance of an individual shear connector determined as specified in Article 6.10.10.2 (kip)

For straight spans or segments, the radial fatigue shear range from Eq. 6.10.10.1.2-4 may be taken equal to zero. For straight or horizontally-curved bridges with skews not exceeding 20 degrees, the radial fatigue shear range from Eq. 6.10.10.1.2-5 may be taken equal to zero.

The center-to-center pitch of shear connectors shall not exceed 48.0 in. for members having a web depth greater than or equal to 24.0 in. For members with a web depth less than 24.0 in., the center-to-center pitch of shear connectors shall not exceed 24.0 in. The center-to-center pitch of shear connectors shall also not be less than six stud diameters.

to zero. Eqs. 6.10.10.1.2-4 and 6.10.10.1.2-5 yield approximately the same value if the span or segment is curved and there are no other sources of torsion in the region under consideration. Note that F_{rc} represents the resultant range of horizontal force from all cross-frames or diaphragms at the point under consideration due to the factored fatigue load plus impact that is resisted by the shear connectors. In lieu of a refined analysis, F_{rc} may be taken as 25.0 kips for an exterior girder, which is typically the critical girder.

Eqs. 6.10.10.1.2-4 and 6.10.10.1.2-5 are provided to ensure that a load path is provided through the shear connectors to satisfy equilibrium at a transverse section through the girders, deck, and cross-frame or diaphragm.

The basis for previous 24.0 in. maximum center-to-center spacing for all web depths was based on scaled experiments in the 1940s that recommended a maximum pitch of three to four slab thicknesses, as described further in Yura et. al. (2008). More recent test results (Badie and Tadros, 2008; Provines and Ocel, 2014) have shown that placing shear connectors at a pitch of up to 48.0 in. has no negative effect on the global flexural resistance of composite steel members. The research did not test very shallow web depths with long pitches and limiting the pitch to 24.0 in. for web depth less than 24.0 in. was a decision to ensure that designs stay within the bounds that have proven satisfactory in experiments.

6.10.10.1.3—Transverse Spacing

Shear connectors shall be placed transversely across the top flange of the steel section and may be spaced at regular or variable intervals.

Stud shear connectors shall not be closer than 4.0 stud diameters center-to-center transverse to the longitudinal axis of the supporting member.

The clear distance between the edge of the top flange and the edge of the nearest shear connector shall not be less than 1.0 in.

6.10.10.1.4—Cover and Penetration

The clear depth of concrete cover over the tops of the shear connectors should not be less than 2.0 in. Shear connectors should penetrate at least 2.0 in. into the concrete deck.

C6.10.10.1.4

Stud shear connectors should penetrate through the haunch between the bottom of the deck and the top flange, if present, and into the deck. Otherwise, the haunch should be reinforced to contain the stud connector and develop its load in the deck.

6.10.10.2—Fatigue Resistance

The fatigue shear resistance of an individual stud shear connector, Z_r, shall be taken as:

For stud-type shear connectors:

- Where the projected 75-year single lane Average Daily Truck Traffic $(ADTT)_{SL}$ is greater than or equal to 1090 trucks per day, the Fatigue I load

C6.10.10.2

For the development of this information, see Slutter and Fisher (1966).

The values of $(ADTT)_{SL}$ specified in this Article to determine which Fatigue load combination and fatigue shear resistance should be used were determined by equating infinite and finite life resistances with due regard to the difference in load factors used with the

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combination shall be used and the fatigue shear resistance for infinite life shall be taken as:

$$Z_r = 5.5d^2 \quad (6.10.10.2\text{-}1)$$

- Otherwise, the Fatigue II load combination shall be used and the fatigue shear resistance for finite life shall be taken as:

$$Z_r = \alpha d^2 \quad (6.10.10.2\text{-}2)$$

in which:

$$\alpha = 34.5 - 4.28 \log N \quad (6.10.10.2\text{-}3)$$

For channel-type shear connectors:

- Where the projected 75-year single lane Average Daily Truck Traffic $(ADTT)_{SL}$ is greater than or equal to 2240 trucks per day, the Fatigue I load combination shall be used and the fatigue shear resistance for infinite life shall be taken as:

$$Z_r = 2.1w \quad (6.10.10.2\text{-}4)$$

- Otherwise, the Fatigue II load combination shall be used and the fatigue shear resistance for finite life shall be taken as:

$$Z_r = Bw \quad (6.10.10.2\text{-}5)$$

in which:

$$B = 9.37 - 1.08 \log N \quad (6.10.10.2\text{-}6)$$

where:

$(ADTT)_{SL}$ = single-lane $ADTT$ as specified in Article 3.6.1.4.2
d = diameter of the stud (in.)
N = number of cycles specified in Article 6.6.1.2.5
w = length of the channel measured transverse to the direction of the flange (in.)

The pitch shall be determined from Eq. 6.10.10.1.2-1 using the value of Z_r and the shear force range V_{sr}.

The effect of the shear connector on the fatigue resistance of the flange shall be investigated using the provisions of Article 6.6.1.2.

Fatigue I and Fatigue II load combinations. A fatigue design life of 75 years and a number of stress range cycles per truck passage, n, equal to 1.0 were also assumed. For other values of the fatigue design life, the value of $(ADTT)_{SL}$ for making this determination for stud shear connectors should instead be taken as the ratio of 81,472 divided by the fatigue life sought in years; the value of $(ADTT)_{SL}$ for making this determination for channel shear connectors should instead be taken as the ratio of 167,767 divided by the fatigue life sought in years. For other values of n, the values of $(ADTT)_{SL}$ should be modified by dividing by the appropriate value of n taken from Table 6.6.1.2.5-2.

6.10.10.3—Special Requirements for Points of Permanent Load Contraflexure

For members that are noncomposite for negative flexure in the final condition, additional shear connectors shall be provided in the region of points of permanent load contraflexure.

C6.10.10.3

The purpose of the additional connectors is to develop the reinforcing bars used as part of the negative flexural composite section.

The number of additional connectors, n_{ac}, shall be taken as:

$$n_{ac} = \frac{A_s f_{sr}}{Z_r} \quad (6.10.10.3\text{-}1)$$

where:

A_s = total area of longitudinal reinforcement over the interior support within the effective concrete deck width (in.^{2})

f_{sr} = stress range in the longitudinal reinforcement over the interior support under the applicable Fatigue load combination specified in Table 3.4.1-1 with the fatigue live load taken as specified in Article 3.6.1.4 (ksi)

Z_r = fatigue shear resistance of an individual shear connector determined as specified in Article 6.10.10.2 (kip)

The additional shear connectors shall be placed within a distance extending one-third of the effective flange width specified in Article 4.6.2.6 from each side of the point of steel dead load contraflexure. The center-to-center pitch of all connectors, including the additional connectors, within that distance shall satisfy the maximum and minimum pitch requirements specified in Article 6.10.10.1.2. Field splices should be placed so as not to interfere with the shear connectors.

6.10.10.4—Strength Limit State

6.10.10.4.1—General

The factored shear resistance of a single shear connector, Q_r, at the strength limit state shall be taken as:

$$Q_r = \phi_{sc} Q_n \quad (6.10.10.4.1\text{-}1)$$

where:

Q_n = nominal shear resistance of a single shear connector determined as specified in Article 6.10.10.4.3 (kip)

ϕ_{sc} = resistance factor for shear connectors specified in Article 6.5.4.2

At the strength limit state, the minimum number of shear connectors, n, over the region under consideration shall be taken as:

$$n = \frac{P}{Q_r} \quad (6.10.10.4.1\text{-}2)$$

where:

P = total nominal shear force determined as specified in Article 6.10.10.4.2 (kip)

Q_r = factored shear resistance of one shear connector determined from Eq. 6.10.10.4.1-1 (kip)

6.10.10.4.2—Nominal Shear Force

For simple spans and for continuous spans that are noncomposite for negative flexure in the final condition, the total nominal shear force, P, between the point of maximum positive design live load plus impact moment and each adjacent point of zero moment shall be taken as:

$$P = \sqrt{P_p^{\,2} + F_p^{\,2}} \qquad (6.10.10.4.2\text{-}1)$$

in which:

P_p = total longitudinal force in the concrete deck at the point of maximum positive live load plus impact moment (kip) taken as the lesser of either:

$$P_{1p} = 0.85 f'_c b_s t_s \qquad (6.10.10.4.2\text{-}2)$$

or

$$P_{2p} = F_{yw} D t_w + F_{yt} b_{ft} t_{ft} + F_{yc} b_{fc} t_{fc} \qquad (6.10.10.4.2\text{-}3)$$

F_p = total radial force in the concrete deck at the point of maximum positive live load plus impact moment (kip) taken as:

$$F_p = P_p \frac{L_p}{R} \qquad (6.10.10.4.2\text{-}4)$$

where:

b_s = effective width of the concrete deck (in.)
L_p = arc length between an end of the girder and an adjacent point of maximum positive live load plus impact moment (ft)
R = minimum girder radius over the length, L_p (ft)
t_s = thickness of the concrete deck (in.)

For straight spans or segments, F_p may be taken equal to zero.

For continuous spans that are composite for negative flexure in the final condition, the total nominal shear force, P, between the point of maximum positive design live load plus impact moment and an adjacent end of the member shall be determined from Eq. 6.10.10.4.2-1. The total nominal shear force, P, between the point of maximum positive design live load plus impact moment and the centerline of an adjacent interior support shall be taken as:

C6.10.10.4.2

Composite beams in which the longitudinal spacing of shear connectors has been varied according to the intensity of shear and duplicate beams where the number of connectors were essentially uniformly spaced have exhibited essentially the same ultimate strength and the same amount of deflection at service loads. Only a slight deformation in the concrete and the more heavily stressed connectors are needed to redistribute the horizontal shear to other less heavily stressed connectors. The important consideration is that the total number of connectors be sufficient to develop the nominal longitudinal force, P_n, on either side of the point of maximum design live load plus impact moment.

The point of maximum design live load plus impact moment is specified because it applies to the composite section and is easier to locate than a maximum of the sum of the moments acting on the composite section.

For continuous spans that are noncomposite for negative flexure in the final condition, points of zero moment within the span should be taken as the points of steel dead load contraflexure.

For continuous spans that are composite for negative flexure in the final condition, sufficient shear connectors are required to transfer the ultimate tensile force in the reinforcement from the concrete deck to the steel section. The number of shear connectors required between points of maximum positive design live load plus impact moment and the centerline of an adjacent interior support is computed from the sum of the critical forces at the maximum positive and negative moment locations. Since there is no point where moment always changes sign, many shear connectors resist reversing action in the concrete deck depending on the live load position. However, the required number of shear connectors is conservatively determined from the sum of the critical forces at the maximum moment locations to provide adequate shear resistance for any live load position.

The tension force in the deck given by Eq. 6.10.10.4.2-8 is defined as 45 percent of the specified 28-day compressive strength of the concrete. This is a conservative approximation to account for the combined contribution of both the longitudinal reinforcement and also the concrete that remains effective in tension based on its modulus of rupture. A more precise value may be substituted.

The radial effect of curvature is included in Eqs. 6.10.10.4.2-4 and 6.10.10.4.2-9. For curved spans or segments, the radial force is required to bring into equilibrium the smallest of the longitudinal forces in

$$P = \sqrt{P_T^{\,2} + F_T^{\,2}} \tag{6.10.10.4.2-5}$$

in which:

P_T = total longitudinal force in the concrete deck between the point of maximum positive live load plus impact moment and the centerline of an adjacent interior support (kip) taken as:

$$P_T = P_p + P_n \tag{6.10.10.4.2-6}$$

P_n = total longitudinal force in the concrete deck over an interior support (kip) taken as the lesser of either:

$$P_{1n} = F_{yw}Dt_w + F_{yt}b_{ft}t_{ft} + F_{yc}b_{fc}t_{fc} \tag{6.10.10.4.2-7}$$

or:

$$P_{2n} = 0.45 f'_c b_s t_s \tag{6.10.10.4.2-8}$$

F_T = total radial force in the concrete deck between the point of maximum positive live load plus impact moment and the centerline of an adjacent interior support (kip) taken as:

$$F_T = P_T \frac{L_n}{R} \tag{6.10.10.4.2-9}$$

where:

L_n = arc length between the point of maximum positive live load plus impact moment and the centerline of an adjacent interior support (ft)
R = minimum girder radius over the length, L_n (ft)

For straight spans or segments, F_T may be taken equal to zero.

either the deck or the girder. When computing the radial component, the longitudinal force is conservatively assumed to be constant over the entire length L_p or L_n, as applicable.

6.10.10.4.3—Nominal Shear Resistance

The nominal shear resistance of one stud shear connector embedded in a concrete deck shall be taken as:

$$Q_n = 0.5A_{sc}\sqrt{f'_c E_c} \le A_{sc}F_u \tag{6.10.10.4.3-1}$$

where:

A_{sc} = cross-sectional area of a stud shear connector (in.^{2})
E_c = modulus of elasticity of the deck concrete determined as specified in Article 5.4.2.4 (ksi)
F_u = specified minimum tensile strength of a stud shear connector determined as specified in Article 6.4.4 (ksi)

C6.10.10.4.3

Studies have defined stud shear connector strength as a function of both the concrete modulus of elasticity and concrete strength (Ollgaard et al., 1971). Note that an upper bound on stud shear strength is the product of the cross-sectional area of the stud times its ultimate tensile strength. Eq. 6.10.10.4.3-2 is a modified form of the formula for the resistance of channel shear connectors developed in Slutter and Driscoll (1965) that extended its use to lightweight as well as normal-weight concrete.

The nominal shear resistance of one channel shear connector embedded in a concrete deck shall be taken as:

$$Q_n = 0.3\left(t_f + 0.5t_w\right) L_c \sqrt{f'_c E_c} \qquad (6.10.10.4.3\text{-}2)$$

where:

t_f = flange thickness of channel shear connector (in.)
t_w = web thickness of channel shear connector (in.)
L_c = length of channel shear connector (in.)

6.10.11—Web Stiffeners

6.10.11.1—Web Transverse Stiffeners

6.10.11.1.1—General

Transverse stiffeners shall consist of plates or angles welded or bolted to either one or both sides of the web.

Stiffeners in straight girders not used as connection plates shall be tight fit or attached at the compression flange, but need not be in bearing with the tension flange. Single-sided stiffeners on horizontally-curved girders should be attached to both flanges. When pairs of transverse stiffeners are used on horizontally-curved girders, they shall be fitted tightly or attached to both flanges.

Stiffeners used as connecting plates for diaphragms or cross-frames shall be attached to both flanges.

The distance between the end of the web-to-stiffener weld and the near edge of the adjacent web-to-flange weld shall not be less than $4t_w$, but shall not exceed the lesser of $6t_w$ and 4.0 in.

Transverse stiffeners with a spacing not exceeding $2D$ shall be provided in web panels with longitudinal stiffeners.

C6.10.11.1.1

When single-sided transverse stiffeners are used on horizontally-curved girders, they should be attached to both flanges to help retain the cross-sectional configuration of the girder when subjected to torsion and to avoid high localized bending within the web. This is particularly important at the top flange due to the torsional restraint from the slab. The fitting of pairs of transverse stiffeners against the flanges, or attachment to both flanges, is required for the same reason.

The minimum distance between the end of the web-to-stiffener weld to the adjacent web-to-flange weld is specified to relieve flexing of the unsupported segment of the web to avoid fatigue-induced cracking of the stiffener-to-web welds. The $6t_w$-criterion for maximum distance is specified to avoid vertical buckling of the unsupported web. The 4.0-in. criterion was arbitrarily selected to avoid a large unsupported length where the web thickness has been selected for reasons other than stability, e.g., webs of bascule girders at trunions.

Web panels with longitudinal stiffeners must include transverse stiffeners at a spacing of $2D$ or less, whether or not they are required for shear, in order to provide support to the longitudinal stiffeners along their length, and because all available supporting experimental and simulation data for the design of longitudinally stiffened girders is from specimens that included transverse stiffeners, with panel aspect ratios generally less than or equal to 2.0 (Subramanian and White, 2016a and 2017b).

The size of intermediate stiffeners should be kept the same along the length of the girders to eliminate multiple plate sizes and eliminate the possibility of placement errors.

6.10.11.1.2—Projecting Width

The width, b_t, of each projecting stiffener element shall satisfy:

C6.10.11.1.2

$$b_t \geq 2.0 + \frac{D}{30} \quad (6.10.11.1.2\text{-}1)$$

and:

$$16t_p \geq b_t \geq b_f / 4 \quad (6.10.11.1.2\text{-}2)$$

where:

b_f = for I-sections, full width of the widest compression flange within the field section under consideration; for tub sections, full width of the widest top flange within the field section under consideration; for closed box sections, the limit of $b_f/4$ does not apply (in.)

t_p = thickness of the projecting stiffener element (in.)

Eq. 6.10.11.1.2-1 is taken from Ketchum (1920). This equation tends to govern relative to Eq. 6.10.11.1.2-2 in I-girders with large D/b_f.

The full width of the widest compression flange within the field section under consideration is used for b_f in Eq. 6.10.11.1.2-2 to ensure a minimum stiffener width that will help restrain the widest compression flange. This requirement also conveniently allows for the use of the same minimum stiffener width throughout the entire field section, if desired. The widest top flange is used in Eq. 6.10.11.1.2-2 for tub sections since the bottom flange is restrained by a web along both of its edges. The limit of $b_f/4$ does not apply for closed box sections for the same reason.

6.10.11.1.3—Moment of Inertia

For transverse stiffeners adjacent to web panels not subject to postbuckling tension-field action, the moment of inertia, I_t, of the transverse stiffener shall satisfy the smaller of the following limits:

$$I_t \geq I_{t1} \quad (6.10.11.1.3\text{-}1)$$

and:

$$I_t \geq I_{t2} \quad (6.10.11.1.3\text{-}2)$$

in which:

$$I_{t1} = b t_w^{\,3} J \quad (6.10.11.1.3\text{-}3)$$

$$I_{t2} = \frac{D^4 \rho_t^{1.3}}{40}\left(\frac{F_{yw}}{E}\right)^{1.5} \quad (6.10.11.1.3\text{-}4)$$

$$J = \frac{2.5}{\left(d_o / D\right)^2} - 2.0 \geq 0.5 \quad (6.10.11.1.3\text{-}5)$$

$$F_{crs} = \frac{0.31E}{\left(\dfrac{b_t}{t_p}\right)^2} \leq F_{ys} \quad (6.10.11.1.3\text{-}6)$$

where:

I_t = moment of inertia of the transverse stiffener taken about the edge in contact with the web for single stiffeners and about the mid-thickness of the web for stiffener pairs (in.^{4})

b = the smaller of d_o and D (in.)

C6.10.11.1.3

For the web to adequately develop the shear-buckling resistance or the combined shear-buckling and postbuckling tension-field resistance as determined in Article 6.10.9, the transverse stiffener must have sufficient rigidity to maintain a vertical line of near zero lateral deflection along the line of the stiffener. For ratios of (d_o/D) less than 1.0, much larger values of I_t are required to develop the shear-buckling resistance, as discussed in Bleich (1952) and represented by Eq. 6.10.11.1.3-1. For single stiffeners, a significant portion of the web is implicitly assumed to contribute to the bending rigidity such that the neutral axis of the stiffener is located close to the edge in contact with the web. Therefore, for simplicity, the neutral axis is assumed to be located at this edge and the contribution of the web to the moment of inertia about this axis is neglected. The term b in Eq. 6.10.11.1.3-1 replaces d_o in prior Specifications. This term and Eq. 6.10.11.1.3-5 give a constant value for the I_t required to develop the shear-buckling resistance for web panels with $d_o > D$ (Kim and White, 2014).

Eq. 6.10.11.1.3-1 requires excessively large stiffener sizes as D/t_w is reduced below $1.12\sqrt{Ek/F_{yw}}$, the web slenderness required for $C = 1$, since Eq. 6.10.11.1.3-1 is based on developing the web elastic shear-buckling resistance. Inelastic buckling solutions using procedures from Bleich (1952) show that larger stiffeners are not required as D/t_w is reduced below this limit. These results are corroborated by refined finite element analysis (FEA) solutions (Kim and White, 2014). k is the shear-buckling coefficient defined in Article 6.10.9.

To develop the web shear postbuckling resistance associated with tension-field action, the transverse stiffeners generally must have a larger I_t than defined by Eq. 6.10.11.1.3-1. The I_t defined by Eq. 6.10.11.1.3-2, which for ρ_t = 1 is approximately equal to the value required by Eq. 6.10.11.1.3-1 for a web with D/t_w =

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d_o = the smaller of the adjacent web panel widths (in.)
J = transverse stiffener bending rigidity parameter
ρ_t = the larger of F_{yw}/F_{crs} and 1.0
F_{crs} = local buckling stress for the stiffener (ksi)
F_{ys} = specified minimum yield strength of the stiffener (ksi)
b_t = projecting width of the transverse stiffener (in.)
F_{yw} = specified minimum yield strength of the web (ksi)
I_{t1} = minimum moment of inertia of the transverse stiffener required for the development of the web shear-buckling resistance (in.^{4})
I_{t2} = minimum moment of inertia of the transverse stiffener required for the development of the full web shear buckling plus postbuckling tension-field action resistance (in.^{4})
t_p = thickness of the projecting stiffener element (in.)

For transverse stiffeners adjacent to web panels subject to postbuckling tension-field action, the moment of inertia, I_t, of the transverse stiffeners shall satisfy:

- If $I_{t2} > I_{t1}$, then:

$$I_t \geq I_{t1} + \left(I_{t2} - I_{t1}\right)\rho_w \quad (6.10.11.1.3\text{-}7)$$

- Otherwise:

$$I_t \geq I_{t2} \quad (6.10.11.1.3\text{-}8)$$

in which:

- If both web panels adjacent to the stiffener are subject to postbuckling tension-field action, then:

ρ_w = maximum ratio of $\left(\dfrac{V_u - \phi_v V_{cr}}{\phi_v V_n - \phi_v V_{cr}}\right)$ within the two web panels

- Otherwise:

ρ_w = ratio of $\left(\dfrac{V_u - \phi_v V_{cr}}{\phi_v V_n - \phi_v V_{cr}}\right)$ within the one panel subject to postbuckling tension-field action

V_{cr} = shear-yielding or shear-buckling resistance of the web panel under consideration (kip)

$$= CV_p \quad (6.10.11.1.3\text{-}9)$$

V_p = plastic shear force (kip)

$$= 0.58F_{yw}Dt_w \quad (6.10.11.1.3\text{-}10)$$

$1.12\sqrt{Ek/F_{yw}}$, provides an accurate to slightly conservative stiffener size relative to refined FEA solutions for straight and curved I-girders at all values of D/t_w permitted by these Specifications (Kim and White, 2014). Eq. 6.10.11.1.3-2 is an approximate upper bound to the results for all values of d_o/D from an equation recommended by Kim and White (2014) recognizing that the stiffener demands are insensitive to this parameter.

Multiple research studies have shown that transverse stiffeners in I-girders designed for tension-field action are loaded predominantly in bending due to the restraint they provide to lateral deflection of the web. Generally, there is evidence of some axial compression in the transverse stiffeners due to the tension field, but even in the most slender web plates permitted by these Specifications, the effect of the axial compression transmitted from the postbuckled web plate is typically minor compared to the lateral loading effect. Therefore, the transverse stiffener area requirement from prior Specifications is no longer specified.

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where:

ϕ_v = resistance factor for shear specified in Article 6.5.4.2
C = ratio of the shear-buckling resistance to the shear yield strength determined by Eq. 6.10.9.3.2-4, 6.10.9.3.2-5, or 6.10.9.3.2-6, as applicable
V_n = nominal shear-yielding or shear-buckling plus postbuckling tension-field action resistance of the web panel under consideration determined as specified in Article 6.10.9.3.2 (kip)
V_u = maximum shear due to the factored loads in the web panel under consideration (kip)

Transverse stiffeners used in web panels with longitudinal stiffeners shall also satisfy:

$$I_t \geq \left(\frac{b_t}{b_\ell}\right)\left(\frac{D}{3.0d_o}\right)I_\ell \qquad (6.10.11.1.3\text{-}11)$$

where:

b_t = projecting width of the transverse stiffener (in.)
b_ℓ = projecting width of the longitudinal stiffener (in.)
I_ℓ = moment of inertia of the longitudinal stiffener determined as specified in Article 6.10.11.3.3 (in.^{4})

For girders with single-sided stiffeners, Eq. 6.10.11.1.3-2 typically requires slightly larger stiffeners than in previous Specifications for small D/t_w slightly exceeding $1.12\sqrt{Ek/F_{yw}}$, where the I_t requirement comparable to Eq. 6.10.11.1.3-1 governs relative to the area requirement for single-sided stiffeners given in previous Specifications. For larger D/t_w values, Eq. 6.10.11.1.3-2 typically gives comparable or smaller single-sided stiffeners compared to the area requirement in previous Specifications at $V_u = \phi_v V_n$. For girders with stiffener pairs, the previous Specifications substantially underestimated the required stiffener size for increasing $D/t_w > 1.12\sqrt{Ek/F_{yw}}$. Eq. 6.10.11.1.3-2 recognizes the fact that single- and double-sided transverse stiffeners with the same I_t exhibit essentially identical performance (Horne and Grayson, 1983; Rahal and Harding, 1990; Stanway et al., 1996; Lee et al., 2003; Kim and White, 2014).

The term ρ_t in Eq. 6.10.11.1.3-2 accounts conservatively for the effect of early yielding in transverse stiffeners with $F_{ys} < F_{yw}$ and for the effect of potential local buckling of stiffeners having a relatively large width-to-thickness ratio b_t/t_p. The definition of the stiffener local buckling stress F_{crs} is retained from AASHTO (1998).

Eq. 6.10.11.1.3-7 accounts for the fact that the I_t necessary to develop a shear resistance greater than or equal to V_u is smaller when V_u is smaller than the full factored combined web shear buckling and post-buckling resistance, $\phi_v V_n$ (Kim and White, 2014). For large girder depths, the philosophy of providing a stiffener flexural rigidity sufficient to develop $V_u = \phi_v V_n$ leads to stiffener sizes that are significantly larger than typically selected using prior AASHTO Specifications, where the former area requirement for the stiffeners was reduced when V_u was less than $\phi_v V_n$. Eq. 6.10.11.1.3-7 allows the calculation of a conservative but more economical stiffener size for these larger girder depths sufficient to develop a girder shear resistance greater than or equal to V_u. Eq. 6.10.11.1.3-8 addresses a small number of cases with stocky webs where V_n is approximately equal to V_p.

Lateral loads along the length of a longitudinal stiffener are transferred to the adjacent transverse stiffeners as concentrated reactions (Cooper, 1967). Eq. 6.10.11.1.3-11 gives a relationship between the moments of inertia of the longitudinal and transverse stiffeners to ensure that the latter does not fail under the concentrated reactions. This equation applies whether the stiffeners are on the same or opposite side of the web.

6.10.11.2—Bearing Stiffeners

6.10.11.2.1—General

Bearing stiffeners shall be placed on the webs of built-up sections at all bearing locations. At bearing locations on rolled shapes and at other locations on built-up sections or rolled shapes subjected to concentrated loads, where the loads are not transmitted through a deck or deck system, either bearing stiffeners shall be provided or the web shall satisfy the provisions of Article D6.5.

Bearing stiffeners shall consist of one or more plates or angles welded or bolted to both sides of the web. The connections to the web shall be designed to transmit the full bearing force due to the factored loads.

The stiffeners shall extend the full depth of the web and as closely as practical to the outer edges of the flanges.

Each stiffener should be finished to bear against the flange through which it receives its load. Bearing stiffeners serving as connection plates shall be attached to both flanges of the cross section as specified in Article 6.6.1.3.1.

C6.10.11.2.1

Webs of built-up sections and rolled shapes without bearing stiffeners at the indicated locations must be investigated for the limit states of web local yielding and web crippling according to the procedures specified in Article D6.5. The section should either be modified to comply with these requirements or else bearing stiffeners designed according to these Specifications should be placed on the web at the location under consideration.

In particular, inadequate provisions to resist temporary concentrated loads during construction that are not transmitted through a deck or deck system can result in failures. The Engineer should be especially cognizant of this issue when girders are incrementally launched over supports.

The use of full penetration groove welds to attach each stiffener to the flange though which it receives its load is permitted, but is not recommended in order to significantly reduce the welding deformation of the flange.

6.10.11.2.2—Minimum Thickness

The thickness, t_p, of each projecting stiffener element shall satisfy:

$$t_p \geq \frac{b_t}{0.48\sqrt{\frac{E}{F_{ys}}}} \qquad (6.10.11.2.2\text{-}1)$$

where:

F_{ys} = specified minimum yield strength of the stiffener (ksi)
b_t = width of the projecting stiffener element (in.)

C6.10.11.2.2

The provision specified in this Article is intended to prevent local buckling of the bearing stiffener plates.

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6.10.11.2.3—Bearing Resistance

The factored bearing resistance for the fitted ends of bearing stiffeners shall be taken as:

$$\left(R_{sb}\right)_r = \phi_b \left(R_{sb}\right)_n \quad (6.10.11.2.3\text{-}1)$$

in which:

$(R_{sb})_n$ = nominal bearing resistance for the fitted ends of bearing stiffeners (kip)

$$= 1.4 A_{pn} F_{ys} \quad (6.10.11.2.3\text{-}2)$$

where:

ϕ_b = resistance factor for bearing specified in Article 6.5.4.2
A_{pn} = area of the projecting elements of the stiffener outside of the web-to-flange fillet welds but not beyond the edge of the flange (in.^{2})
F_{ys} = specified minimum yield strength of the stiffener (ksi)

C6.10.11.2.3

To bring bearing stiffener plates tight against the flanges, part of the stiffener must be clipped to clear the web-to-flange fillet weld. Thus, the area of direct bearing is less than the gross area of the stiffener. The bearing resistance is based on this bearing area and the yield strength of the stiffener.

The specified factored bearing resistance is approximately equivalent to the bearing strength given in AISC (2016b). The nominal bearing resistance given by Eq. 6.10.11.2.3-2 is reduced from the nominal bearing resistance of $1.8A_{pn}F_{ys}$ specified in AISC (2016b) to reflect the relative difference in the resistance factors for bearing given in the AISC and *AASHTO LRFD Bridge Design Specifications*.

6.10.11.2.4—Axial Resistance of Bearing Stiffeners

6.10.11.2.4a—General

The factored axial resistance, P_r, shall be determined as specified in Article 6.9.2.1 using the specified minimum yield strength of the stiffener plates F_{ys}. The radius of gyration shall be computed about the mid-thickness of the web and the effective length shall be taken as $0.75D$, where D is the web depth.

C6.10.11.2.4a

The end restraint against column buckling provided by the flanges allows for the use of a reduced effective length. The specified minimum yield strength of the stiffener plates, F_{ys}, is to be used in the calculation of the axial resistance to account for the early yielding of the lower strength stiffener plates.

6.10.11.2.4b—Effective Section

For stiffeners bolted to the web, the effective column section shall consist of the stiffener elements only.

Except as noted herein, for stiffeners welded to the web, a portion of the web shall be included as part of the effective column section. For stiffeners consisting of two plates welded to the web, the effective column section shall consist of the two stiffener elements, plus a centrally located strip of web extending not more than $9t_w$ on each side of the stiffeners. If more than one pair of stiffeners is used, the effective column section shall consist of all stiffener elements, plus a centrally located strip of web extending not more than $9t_w$ on each side of the outer projecting elements of the group.

The strip of the web shall not be included in the effective section at interior supports of continuous-span hybrid members for which the specified minimum yield strength of the web is less than 70 percent of the specified minimum yield strength of the higher strength flange.

If the specified minimum yield strength of the web is less than that of the stiffener plates, the strip of the web

C6.10.11.2.4b

A portion of the web is assumed to act in combination with the bearing stiffener plates. This portion of the web is not included for the stated case at interior supports of continuous-span hybrid members with F_{yw} less than the specified value because of the amount of web yielding that may be expected due to longitudinal flexural stress in this particular case. At end supports of hybrid members, the web may be included regardless of the specified minimum yield strength of the web.

If more than one pair of bearing stiffeners is used, AASHTO/NSBA (2016) recommends that a minimum spacing of 8.0 in. or 1.5 times the plate width be provided between the stiffeners for welding access. If the stiffeners are skewed, the spacing should be measured from the closest edge of the plate and not necessarily from the intersection of the plate with the web; more space will be required than for stiffeners perpendicular to the web. It is recommended that the maximum spacing between the stiffeners not exceed $1.09t_w\sqrt{E/F_{ys}}$ to prevent an included in the effective section shall be reduced by the ratio F_{yw}/F_{ys}.

effective width reduction on the web of the section between the stiffeners.

For unusual cases in which F_{ys} is larger than F_{yw}, the yielding of the lower strength web is accounted for in the stiffener axial resistance by adjusting the width of the web strip included in the effective section by F_{yw}/F_{ys}.

6.10.11.3—Web Longitudinal Stiffeners

6.10.11.3.1—General

Where required, longitudinal stiffeners should consist of either a plate welded to one side of the web, or a bolted angle. Longitudinal stiffeners shall be located at a vertical position on the web such that Eq. 6.10.3.2.1-3 is satisfied when checking constructibility, Eq. 6.10.4.2.2-4 is satisfied at the service limit state, and all theappropriate design requirements are satisfied at the strength limit state.

Longitudinal stiffeners should be included in calculating the section properties of the member gross cross section.

Longitudinal stiffeners should be placed on the side of the web with the fewest transverse intermediate stiffeners and connection plates. Except as specified herein, at the intersection of a welded longitudinal stiffener and a welded transverse intermediate stiffener or connection plate, the vertical stiffener shall be interrupted and attached to the continuous longitudinal stiffener with fillet welds on both sides of the vertical and longitudinal stiffeners as shown in Table 6.6.1.2.4-1. The welds shall satisfy the minimum size requirements specified in Table 6.13.3.4-1. At bearing stiffeners, the longitudinal stiffener should be interrupted and similarly attached to the continuous bearing stiffener as shown in Table 6.6.1.2.4-1. Where more than one pair of bearing stiffeners is used, the longitudinal stiffener shall not be continued in-between the stiffener pair. The longitudinal stiffener may be similarly interrupted and attached to a continuous transverse intermediate stiffener or connection plate, but only at locations where the intersection is subject to a net compressive stress under Strength Load Combination I. The flexural stress in the longitudinal stiffener, f_s, due to the factored loads at the strength limit state and when checking constructibility shall satisfy:

$$f_s \leq \phi_f R_h F_{ys} \tag{6.10.11.3.1-1}$$

where:

- ϕ_f = resistance factor for flexure specified in Article 6.5.4.2
- F_{ys} = specified minimum yield strength of the stiffener (ksi)
- R_h = hybrid factor determined as specified in Article 6.10.1.10.1

C6.10.11.3.1

For composite sections in regions of positive flexure, the depth of the web in compression D_c changes relative to the vertical position of a longitudinal web stiffener, which is usually a fixed distance from the compression flange, after the concrete deck has been placed. Thus, the computed web bend-buckling resistance is different before and after placement of the deck and is dependent on the loading. As a result, an investigation of several trial locations of the stiffener may be necessary to determine a location of the stiffener to satisfy Eq. 6.10.3.2.1-3 for constructibility, Eq. 6.10.4.2.2-4 at the service limit state and the appropriate design requirements at the strength limit state along the girder. The following equation may be used to determine an initial trial stiffener location for composite sections in regions of positive flexure:

$$\frac{d_s}{D_c} = \frac{1}{1 + 1.5\sqrt{\dfrac{f_{DC1} + f_{DC2} + f_{DW} + f_{LL+IM}}{f_{DC1}}}} \tag{C6.10.11.3.1-1}$$

where:

- d_s = distance from the centerline of a plate longitudinal stiffener, or the gauge line of an angle longitudinal stiffener, to the inner surface or leg of the compression flange element (in.)
- D_c = depth of the web of the noncomposite steel section in compression in the elastic range (in.)
- f_{xx} = compression flange stresses at the strength limit state caused by the different factored loads at the section with the maximum compressive flexural stress; i.e., *DC1*, the permanent load acting on the noncomposite section; *DC2*, the permanent load acting on the long-term composite section; *DW*, the wearing surface load; and *LL+IM*; acting on their respective sections (ksi). Flange lateral bending is to be disregarded in this calculation.

The stiffener may need to be moved vertically up or down from this initial trial location in order to satisfy all the specified limit-state criteria.

For composite sections in regions of negative flexure and for noncomposite sections, it is suggested than an initial trial stiffener location of $2D_c/5$ from the inner surface of the compression flange be examined at the section with the maximum flexural compressive stress due to the factored loads at the strength limit state. Furthermore, for composite sections, D_c should be computed for the section consisting of the steel girder plus the longitudinal deck reinforcement. The stiffener may need to be moved vertically up or down from the initial trial location in order to satisfy all the specified limit-state criteria, in particular for cases where the concrete deck is assumed effective in tension in regions of negative flexure at the service limit state, as permitted for composite sections satisfying the requirements specified in Article 6.10.4.2.1.

Theoretical and experimental studies on noncomposite girders have indicated that the optimum location of one longitudinal stiffener is $2D_c/5$ for bending and $D/2$ for shear. Tests have also shown that longitudinal stiffeners located at $2D_c/5$ on these sections can effectively control lateral web deflections under flexure (Cooper, 1967). The distance $2D_c/5$ is recommended because shear is always accompanied by moment and because a properly proportioned longitudinal stiffener also reduces the web lateral deflections caused by shear. Also, because D_c may vary along the length of the span, it is recommended that the stiffener be located based on D_c computed at the section with the largest compressive flexural stress. Thus, the stiffener may not be located at its optimum location at other sections with a lower stress and a different D_c. These sections should also be examined to ensure that they satisfy the specified limit states.

In regions where the web undergoes stress reversal, it may be necessary, or desirable, to use two longitudinal stiffeners on the web.

Detailing of the intersections of welded longitudinal and vertical web stiffeners is to be done as shown in Table 6.6.1.2.4-1 for the detail under consideration. In regions subject to net tension, the longitudinal stiffener should be continuous to avoid conditions susceptible to fatigue and also conditions susceptible to constraint-induced fracture due to the build-up of force that would occur in the gap should the longitudinal stiffener be interrupted. Longitudinal stiffeners are also designed as continuous members to support the web against bend-buckling, and therefore, should not be interrupted and left unattached to the vertical web stiffeners in regions where the intersection is subject to net compression. A longitudinal stiffener should be interrupted at intersections with a bearing stiffener and attached to the bearing stiffener as shown in Table 6.6.1.2.4-1. A longitudinal stiffener may also be interrupted at intersections with transverse intermediate stiffeners or connection plates, if desired, but only if such intersections are subject to a net compressive stress under Strength Load Combination I and the longitudinal stiffener is attached to the vertical

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web stiffeners as shown in Table 6.6.1.2.4-1. Longitudinal stiffeners should only be interrupted and left unattached to vertical web stiffeners if it is desired to terminate the longitudinal stiffener at that location, and either the location is subject to a net compressive stress or the nominal fatigue resistance of the longitudinal stiffener end detail is shown to be satisfactory should that location be subject to a net tensile stress determined as specified in Article 6.6.1.2.1. All stiffener intersections subject to a net tensile stress determined as specified in Article 6.6.1.2.1 must be designed with respect to fatigue. Copes or snipes should always be provided in the discontinuous element at the intersection to allow for continuous welds of the continuous element to the web.

For determining the nominal fatigue resistance of various longitudinal stiffener end details, refer to Condition 4.3 in Table 6.6.1.2.3-1. While the use of complete- or partial-penetration groove welds to attach the stiffener to the web or flange is permitted, the use of fillet welds is strongly encouraged as shown in Table 6.6.1.2.3-1. The use of groove welds does not enhance the nominal fatigue resistance of the end detail. Consideration should be given to wrapping the weld around the end of the stiffener for sealing. The weld and stiffener material should be ground to a smooth contour where the radiused stiffener end becomes tangent to the web or flange. Welded shop splices in longitudinal stiffeners should be complete-penetration groove welded. Where longitudinal stiffeners are discontinued at bolted field splices, consideration should be given to taking the stiffener to the free edge of the web where the normal stress is zero.

Longitudinal stiffeners are subject to the same flexural strain as the web at their vertical position on the web. Therefore, they must have sufficient rigidity and strength to resist bend-buckling of the web, where required to do so, and to transmit the stresses in the stiffener and a portion of the web as an equivalent column (Cooper, 1967). Thus, full nominal yielding of the stiffeners is not permitted at the strength limit state and when checking constructibility as an upper bound. Eq. 6.10.11.3.1-1 serves as a limit on the validity of Eq. 6.10.11.3.3-2, which is in turn based on the axial resistance of an equivalent column section composed of the stiffener and a portion of the web plate. To account for the influence of web yielding on the longitudinal stiffener stress in hybrid members, the elastically computed stress in the stiffener is limited to $\phi_f R_h F_{ys}$ in Eq. 6.10.11.3.1-1. For the strength limit state and constructibility checks, the corresponding value of R_h at the section under consideration should be applied in Eq. 6.10.11.3.1-1.

6.10.11.3.2—Projecting Width

The projecting width, b_ℓ, of the stiffener shall satisfy:

C6.10.11.3.2

This requirement is intended to prevent local buckling of the longitudinal stiffener.

$$b_\ell \le 0.48 t_s \sqrt{\frac{E}{F_{ys}}} \quad (6.10.11.3.2\text{-}1)$$

where:

t_s = thickness of the stiffener (in.)

6.10.11.3.3—Moment of Inertia and Radius of Gyration

Longitudinal stiffeners shall satisfy:

$$I_\ell \ge D t_w^3 \left[2.4 \left(\frac{d_o}{D} \right)^2 - 0.13 \right] \beta \quad (6.10.11.3.3\text{-}1)$$

and:

$$r \ge \frac{0.16 d_o \sqrt{\frac{F_{ys}}{E}}}{\sqrt{1 - 0.6 \frac{F_{yc}}{R_h F_{ys}}}} \quad (6.10.11.3.3\text{-}2)$$

in which:

β = curvature correction factor for longitudinal stiffener rigidity calculated as follows:

- For cases where the longitudinal stiffener is on the side of the web away from the center of curvature:

$$\beta = \frac{Z}{6} + 1 \quad (6.10.11.3.3\text{-}3)$$

- For cases where the longitudinal stiffener is on the side of the web toward the center of curvature:

$$\beta = \frac{Z}{12} + 1 \quad (6.10.11.3.3\text{-}4)$$

Z = curvature parameter:

$$= \frac{0.95 d_o^2}{R t_w} \le 12 \quad (6.10.11.3.3\text{-}5)$$

where:

d_o = transverse stiffener spacing (in.)

I_ℓ = moment of inertia of the longitudinal stiffener including an effective width of the web equal to $18t_w$ taken about the neutral axis of the combined section (in.[4]). If F_{yw} is smaller than F_{ys}, the strip of the web included in the effective section shall be reduced by the ratio F_{yw}/F_{ys}

C6.10.11.3.3

Eq. 6.10.11.3.3-1 ensures that the stiffener will have adequate rigidity to maintain a horizontal line of near zero lateral deflection in the web panel when necessary to resist bend-buckling of the web (Ziemian, 2010). Eq. 6.10.11.3.3-2 ensures that the longitudinal stiffener acting in combination with an adjacent strip of the web will withstand the axial compressive stress without lateral buckling. The moment of inertia, I_ℓ, and radius of gyration, r, are taken about the neutral axis of an equivalent column cross section composed of the stiffener and an adjacent strip of the web, as suggested by Cooper (1967). Previous Specifications required that these quantities be calculated about the edge of the stiffener in contact with the web plate. The values for I_ℓ and for r calculated as suggested by Cooper (1967) are generally smaller than the corresponding values determined as suggested in the previous Specifications. The specified procedure for calculation of I_ℓ and r is consistent with the horizontally-curved I-girder provisions of AASHTO (2003) in the limit that the girder is straight. The effect of the web plate having a lower yield strength than that of the longitudinal stiffener is accommodated by adjusting the web strip that contributes to the effective column section by F_{yw}/F_{ys} in the calculation of the moment of inertia of the longitudinal stiffener.

The rigidity required of longitudinal stiffeners on curved webs is greater than the rigidity required on straight webs because of the tendency of curved webs to bow. The factor β in Eq. 6.10.11.3.3-1 is a simplification of the requirement in the Hanshin (1988) provisions for longitudinal stiffeners used on curved girders. For longitudinal stiffeners on straight webs, Eq. 6.10.11.3.3-5 leads to β = 1.0.

Eq. 6.10.11.3.3-2 is based on the model described by Cooper (1967), except that the possibility of different specified minimum yield strengths for the stiffener and compression flange is accommodated. Also, the influence of a hybrid web is approximated by including the hybrid factor, R_h, within this equation. For a nonhybrid I-section, the required radius of gyration from Eq. 6.10.11.3.3-2 is slightly larger than that required in previous Specifications. For an I-section in which F_{yc}/F_{ys} is greater than one, the required radius of gyration from Eq. 6.10.11.3.3-2 is significantly larger than required in previous Specifications. This is necessary because in these cases, the longitudinal stiffener is subjected to larger stresses compared to its resistance as

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R = minimum girder radius in the panel (in.)

r = radius of gyration of the longitudinal stiffener including an effective width of the web equal to $18t_w$ taken about the neutral axis of the combined section (in.)

an equivalent column than in an equivalent homogeneous section.

Article 6.10.9.3.1 requires that the shear resistance of the web panel be determined based on the total web depth D. Therefore, no area requirement is given for the longitudinal stiffeners to anchor the tension field.

6.10.12—Cover Plates

6.10.12.1—General

The length of any cover plate, L_{cp}, in ft, added to a member shall satisfy:

$$L_{cp} \geq \frac{d}{6.0} + 3.0 \quad (6.10.12.1\text{-}1)$$

where:

d = total depth of the steel section (in.)

Partial length welded cover plates shall not be used on flanges more than 0.8 in. thick for nonredundant load path structures subjected to repetitive loadings that produce tension or reversal of stress in the flange.

The maximum thickness of a single cover plate on a flange shall not be greater than two times the thickness of the flange to which the cover plate is attached. Multiple welded cover plates shall not be permitted.

Cover plates may either be wider or narrower than the flange to which they are attached.

6.10.12.2—End Requirements

6.10.12.2.1—General

The theoretical end of the cover plate shall be taken as the section where the moment, M_u, or flexural stress, f_{bu}, due to the factored loads equals the factored flexural resistance of the flange. The cover plate shall be extended beyond the theoretical end far enough so that:

- the stress range at the actual end satisfies the appropriate fatigue requirements specified in Article 6.6.1.2, and
- the longitudinal force in the cover plate due to the factored loads at the theoretical end can be developed by welds and/or bolts placed between the theoretical and actual ends.

The width at ends of tapered cover plates shall not be less than 3.0 in.

6.10.12.2.2—Welded Ends

The welds connecting the cover plate to the flange between the theoretical and actual ends shall be adequate to develop the computed force in the cover plate at the theoretical end.

Where cover plates are wider than the flange, welds shall not be wrapped around the ends of the cover plate.

6.10.12.2.3—Bolted Ends

The bolts in the slip-critical connections of the cover plate to the flange between the theoretical and actual ends shall be adequate to develop the force due to the factored loads in the cover plate at the theoretical end.

The slip resistance of the end-bolted connection shall be determined in accordance with Article 6.13.2.8. The longitudinal welds connecting the cover plate to the flange shall be continuous and shall stop a distance equal to one bolt spacing before the first row of bolts in the end-bolted portion. Where end-bolted cover plates are used, the contract documents shall specify that they be installed in the following sequence:

- drill holes,
- clean faying surfaces,
- install bolts, and
- weld plates.

C6.10.12.2.3

Research on end-bolted cover plates is discussed in Wattar et al. (1985).

6.11—COMPOSITE BOX-SECTION FLEXURAL MEMBERS

6.11.1—General

The provisions of this Article apply to flexure of straight or horizontally-curved steel single or multiple closed-box or tub sections in simple or continuous bridges of moderate length. The provisions cover the design of composite, hybrid and nonhybrid, and constant and variable web depth members as defined by and subject to the requirements of Article 6.10.1.1, Articles 6.10.1.3 through 6.10.1.8, and Articles 6.11.1.1 through 6.11.1.4. The provisions of Article 6.10.1.6 shall apply only to the top flanges of tub sections.

Single box sections shall be positioned in a central position with respect to the cross section, and the center of gravity of the dead load shall be as close to the shear center of the box as is practical. These provisions shall not be applied to multiple cell single box sections, or to composite box flanges used as bottom flanges.

All types of box-section flexural members shall be designed as a minimum to satisfy:

- The cross-section proportion limits specified in Article 6.11.2;

C6.11.1

Article 6.11.1 addresses general topics that apply to closed-box and tub sections used as flexural members in either straight bridges, horizontally-curved bridges, or bridges containing both straight and curved segments. For the application of the provisions of Article 6.11, bridges containing both straight and curved segments are to be treated as horizontally-curved bridges since the effects of curvature on the support reactions and girder deflections, as well as the effects of flange lateral bending and torsional shear, usually extend beyond the curved segments. The term moderate length as used herein refers to bridges of spans up to approximately 350 ft. The provisions may be applied to larger spans based on a thorough evaluation of the application of the bridge under consideration consistent with basic structural fundamentals. Alternative information regarding the design of long-span steel box-girder bridges is contained in FHWA (1980). For general overview on box-girder bridges, refer to Wolchuk (1997).

The five bullet items in this Article indicate the overarching organization of the subsequent provisions for

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- The constructibility requirements specified in Article 6.11.3;
- The service limit state requirements specified in Article 6.11.4;
- The fatigue and fracture limit state requirements specified in Article 6.11.5;
- The strength limit state requirements specified in Article 6.11.6.

The web bend-buckling resistance in slender web members shall be determined as specified in Article 6.10.1.9. Flange-strength reduction factors in hybrid and/or slender web members shall be determined as specified in Article 6.10.1.10.

Internal and external cross-frames and diaphragms for box sections shall satisfy the provisions of Article 6.7.4. Top flange bracing for tub sections shall satisfy the provisions of Article 6.7.5.

the design of box-section flexural members. To avoid repetition, some of the general topics in this Article refer back to the general provisions of Article 6.10.1 for I-sections, which apply equally well to box sections. Where necessary, other Articles in Article 6.10 are referred to at appropriate points within Article 6.11.

Within these provisions, the term box flange refers to a flange plate connected to two webs.

These provisions do not apply to the use of box sections which are noncomposite in the final condition, as defined in Article 6.10.1.2, as flexural members. The concrete deck is to be assumed effective over the entire span length in the analysis for loads applied to the composite section according to the provisions of Article 6.10.1.5. Therefore, shear connectors must be present along the entire span to resist the torsional shear that exists along the entire span in all types of composite box sections in order to avoid possible debonding of the deck. Shear connectors must also be present in regions of negative flexure in order to be consistent with the prototype and model bridges that were studied in the original development of the live-load distribution provisions for box sections (Johnston and Mattock, 1967). For considerations while a composite box section is under construction, applicable provisions of Articles 6.10 and 6.11 may be utilized depending on whether the section is thought to be effectively open or quasi-box in behavior, respectively. The flexural resistance of noncomposite closed-box sections used as compression or tension members is specified in Article 6.12.2.2.2.

These provisions may be applied to the use of composite closed-box sections, or sections utilizing a steel plate for the top flange that is composite with the concrete deck, as flexural members. The use of such sections has been relatively rare in the U.S. to date due to cost considerations related to the implementation of necessary safety requirements for working inside of closed boxes. These Specifications do not apply to the use of composite concrete on bottom box flanges in order to stiffen the flanges in regions of negative flexure.

The use of single box sections is permitted in these Specifications because torsional equilibrium can be established with two bearings at some supports. Placing the center of gravity of the dead load near the shear center of single box sections ensures minimal torsion. Items such as sound barriers on one side of the bridge may be critical on single box sections.

These Specifications do not apply to multiple-cell single box sections because there has been little published research in the U.S. regarding these members. Analysis of this bridge type involves consideration of shear flow in each cell.

In variable web depth box members with inclined webs, the inclination of the webs should preferably remain constant in order to simplify the analysis and the fabrication. For a constant distance between the webs at the top of the box, which is also preferred, this requires that the width of the bottom flange vary along the length and that the web heights at a given cross section be kept

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equal. If the bridge is to incrementally launched, a constant depth box is recommended.

The provisions of Article 6.11 provide a unified approach for consideration of combined major-axis bending and flange lateral bending from any source in the design of top flanges of tub sections during construction. These provisions also provide a unified approach for consideration of the combined effects of normal stress and St. Venant torsional shear stress in closed-box and tub sections both during construction and in the final constructed condition. General design equations are provided for determining the nominal flexural resistance of box flanges under the combined effects of normal stress and torsional shear stress. The provisions also allow for the consideration of torsional shear in the design of the box-section webs and shear connectors. For straight boxes, the effects of torsional shear are typically relatively small unless the bridge is subjected to large torques. For example, boxes resting on skewed supports are usually subjected to large torques. For horizontally-curved boxes, flange lateral bending effects due to curvature and the effects of torsional shear must always be considered at all limit states and also during construction.

For cases where the effects of the flange lateral bending and/or torsional shear are judged to be insignificant or incidental, or are not to be considered, the terms related to these effects are simply set equal to zero in the appropriate equations. The format of the equations then simply reduces to the format of the more familiar equations given in previous Specifications for checking the nominal flexural resistance of box sections in the absence of flange lateral bending and St. Venant torsion.

Fundamental calculations for flexural members have been placed in Appendix D6.

6.11.1.1—Stress Determinations

Box flanges in multiple and single box sections shall be considered fully effective in resisting flexure if the width of the flange does not exceed one-fifth of the effective span. For simple spans, the effective span shall be taken as the span length. For continuous spans, the effective span shall be taken equal to the distance between points of permanent load contraflexure, or between a simple support and a point of permanent load contraflexure, as applicable. If the flange width exceeds one-fifth of the effective span, only a width equal to one-fifth of the effective span shall be considered effective in resisting flexure.

For multiple box sections in straight bridges satisfying the requirements of Article 6.11.2.3, the live-load flexural moment in each box may be determined in accordance with the applicable provisions of Article 4.6.2.2.2b. Shear due to St. Venant torsion and transverse bending and longitudinal warping stresses due to cross-

C6.11.1.1

Stress analyses of actual box girder bridge designs were carried out to evaluate the effective width of a box flange using a series of folded plate equations (Goldberg and Leve, 1957). Bridges for which the span-to-flange width ratio varied from 5.65 to 35.3 were included in the study. The effective flange width as a ratio of the total flange width covered a range from 0.89 for the bridge with the smallest span-to-width ratio to 0.99 for the bridge with the largest span-to-width ratio. On this basis, it is reasonable to permit a box flange to be considered fully effective and subject to a uniform longitudinal stress, provided that its width does not exceed one-fifth of the span of the bridge. For extremely wide box flanges, a special investigation for shear lag effects may be required.

Although the results quoted above were obtained for simply-supported bridges, this criterion would apply equally to continuous bridges using the appropriate

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section distortion may also be neglected for sections within these bridges that have fully effective box flanges. The section of an exterior member assumed to resist horizontal factored wind loading within these bridges may be taken as the bottom box flange acting as a web and 12 times the thickness of the web acting as flanges.

The provisions of Article 4.6.2.2.2b shall not apply to:

- Single box sections in straight or horizontally-curved bridges,
- Multiple box sections in straight bridges not satisfying the requirements of Article 6.11.2.3, or
- Multiple box sections in horizontally-curved bridges.

For these sections, and for sections that do not have fully effective box flanges, the effects of both flexural and St. Venant torsional shear shall be considered. The St. Venant torsional shear stress in box flanges due to the factored loads at the strength limit state shall not exceed the factored torsional shear resistance of the flange, F_{vr}, taken as:

$$F_{vr} = 0.75\phi_v \frac{F_{yf}}{\sqrt{3}} \quad (6.11.1.1\text{-}1)$$

where:

ϕ_v = resistance factor for shear specified in Article 6.5.4.2

In addition, transverse bending stresses due to cross-section distortion shall be considered for fatigue as specified in Article 6.11.5, and at the strength limit state. Transverse bending stresses due to the factored loads shall not exceed 20.0 ksi at the strength limit state. Longitudinal warping stresses due to cross-section distortion shall be considered for fatigue as specified in Article 6.11.5, but may be ignored at the strength limit state. Transverse bending and longitudinal warping stresses shall be determined by rational structural analysis in conjunction with the application of strength-of-materials principles. Transverse stiffeners attached to the webs or box flanges should be considered effective in resisting transverse bending.

effective span defined in this Article for the section under consideration.

The effective box-flange width should be used when calculating the flexural stresses in the section due to the factored loads. The full flange width should be used to calculate the nominal flexural resistance of the box flange.

Closed-box sections are capable of resisting torsion with limited distortion of the cross section. Since distortion is generally limited by providing sufficient internal bracing in accordance with Article 6.7.4.3, torsion is resisted mainly by St. Venant torsional shear flow. The warping constant for closed-box sections is approximately equal to zero. Thus, warping shear and normal stresses due to warping torsion are typically quite small and are usually neglected.

Transverse bending stresses in box flanges and webs due to distortion of the box cross section occur due to changes in direction of the shear flow vector. The transverse bending stiffness of the webs and flanges alone is not sufficient to retain the box shape so internal cross bracing is required. Longitudinal warping stresses due to cross-section distortion are also best controlled by internal cross bracing, as discussed further in Article C6.7.4.3.

Top flanges of tub girders subject to torsional loads need to be braced so that the section acts as a pseudo-box for noncomposite loads applied before the concrete deck hardens or is made composite. Top-flange bracing working with internal cross bracing retains the box shape and resists lateral force induced by inclined webs and torsion.

As discussed further in Article C6.11.2.3, for multiple box sections in straight bridges that conform to the restrictions specified in Article 6.11.2.3, the effects of St. Venant torsional shear and secondary distorsional stresses may be neglected unless the box flange is very wide. The live-load distribution factor specified in Article 4.6.2.2.2b for straight multiple steel box sections may also be applied in the analysis of these bridges. Bridges not satisfying one or more of these restrictions must be investigated using one of the available methods of refined structural analysis, or other acceptable methods of approximate structural analysis as specified in Articles 4.4 or 4.6.2.2.4, since the specified live-load distribution factor does not apply to such bridges. The effects of St. Venant torsional shear and secondary distorsional stresses are also more significant and must therefore be considered for sections in these bridges. Included in this category are all types of bridges containing single box sections, and horizontally-curved bridges containing multiple box sections. Transverse bending stresses are of particular concern in boxes that may be subjected to large torques; e.g., single box sections, sharply curved boxes, and boxes resting on skewed supports. For other cases, the distorsional stresses may be ignored if it can be demonstrated that the torques are of comparable magnitude to the torques for cases in which research has shown that these stresses are small enough to be neglected (Johnston and Mattock, 1967), e.g., a straight bridge of similar proportion satisfying the requirements of Article 6.11.2.3 or if the torques are deemed small enough in the judgment of the Owner and the Engineer. In such cases, it is recommended that all web stiffeners be attached to both flanges to enhance fatigue performance.

In single box sections in particular, significant torsional loads may occur during construction and under live loads. Live loads at the extreme of the deck can cause critical torsional loads without causing critical flexural moments. In the analysis, live load positioning should be done for flexure and torsion. The position of the bearings should be recognized in the analysis in sufficient completeness to permit direct computation of the reactions.

Where required, the St. Venant torsional shear and shear stress in web and flange elements can be calculated from the shear flow, which is determined as follows:

$$f = \frac{T}{2A_o} \qquad \text{(C6.11.1.1-1)}$$

where:

A_o = enclosed area within the box section (in.^{2})
f = shear flow (kip/in.)
T = internal torque due to the factored loads (kip-in.)

For torques applied to the noncomposite section, A_o is to be computed for the noncomposite box section. As specified in Article 6.7.5.3, if top lateral bracing in a tub section is attached to the webs, A_o is to be reduced to reflect the actual location of the bracing. Because shear connectors are required along the entire length of box sections according to these provisions, the concrete deck can be considered effective in resisting torsion at any point along the span. Therefore, for torques applied to the composite section in regions of positive or negative flexure, A_o is to be computed for the composite section using the depth from the bottom flange to the mid-thickness of the concrete deck. The depth may be computed using a lower bound estimate of the actual thickness of the concrete haunch, or may be determined conservatively by neglecting the thickness of the haunch.

The torsion acting on the composite section also introduces horizontal shear in the concrete deck that should be considered when designing the reinforcing steel. Article C6.11.10 suggests a procedure for determining the torsional shear in the concrete deck for closed-box sections. For tub sections, the deck should be assumed to resist all the torsional shear acting on top of the composite box section.

Previous editions of AASHTO's *Guide Specifications for Horizontally Curved Highway Bridges* (AASHTO, 1993) limited the nominal St. Venant torsional shear resistance of box flanges to the shear yield stress, $F_{yf}/\sqrt{3}$. However, at this level of shear stress, there is a significant reduction in the nominal flexural resistance of the flange. Therefore, the nominal shear resistance is limited to 0.75 $F_{yf}/\sqrt{3}$ in these provisions. Such a level of torsional shear stress is rarely, if ever, encountered in practical box-girder designs.

Where required, transverse or through-thickness bending stresses and stress ranges in the webs and flanges due to cross-section distortion can be determined using the beam-on-elastic-foundation or BEF analogy presented by Wright and Abdel-Samad (1968). In this method, the internal diaphragms or cross-frames are analogous to intermediate supports in the BEF, and the resistance to distortion provided by the box cross section is analogous to a continuous elastic foundation. The deflection of the BEF is analogous to the transverse bending stress. Transverse stiffeners should be considered effective with the web or box flange, as applicable, in computing the flexural rigidities of these elements. Sample calculations based on the BEF analogy are presented in Heins and Hall (1981) and in AASHTO (2003). The use of finite-element analysis to determine through-thickness bending stresses as part of the overall analysis of box sections is impractical due to the mesh refinement necessary for the accurate calculation of these stresses.

Longitudinal warping stresses due to cross-section distortion can also be determined using the BEF analogy. The warping stress is analogous to the moment in the BEF. The warping stresses are largest at the corners of the box where critical welded details are often located and should be considered for fatigue (Wright and Abdel-Samad, 1968). Tests have indicated that these warping stresses do not affect the ultimate strength of box girders of typical proportions.

Since top lateral bracing contributes to the flexural stiffness of tub sections, consideration should be given to including the longitudinal component of the top-flange bracing area when computing the section properties of the tub. Where used, longitudinal flange stiffeners should also be included in the section properties of the box or tub.

6.11.1.2—Bearings

Single or double bearings may be used at supports. Double bearings may be placed either inboard or outboard of the box section webs. If single bearings narrower than the bottom flange are used, they shall be aligned with the shear center of the box, and other supports shall have adequate bearings to ensure against overturning under any load combination. If tie-down bearings are used, the resulting force effects shall be considered in the design.

C6.11.1.2

The bearing arrangement dictates how torsion is resisted at supports and is especially critical for single box sections. When a single bearing arrangement is used, torque may be removed from multiple box sections through cross-frames or diaphragms between the boxes. Two bearings under each box provide a couple to resist the torque in each box. Double bearings can be placed between the box webs or outboard of the box. Placing bearings outboard of the box reduces overturning loads on the bearings and may eliminate uplift. For the case of double bearings, uplift may be especially critical when deck overhangs are large and heavy parapets or sound barriers

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are placed at the edges of the overhangs. Uplift should be checked ignoring the effect of the future wearing surface.

Integral cap beams of steel or concrete are often used with box sections in lieu of bearings.

6.11.1.3—Flange-to-Web Connections

Except as specified herein, the total effective thickness of flange-to-web welds shall not be less than the smaller of the web or flange thickness.

Where two or more intermediate internal diaphragms are provided in each span, fillet welds may be used for the flange-to-web connections. The weld size shall not be less than the size consistent with the requirements of Article 6.13.3.4. If fillet welds are used, they shall be placed on both sides of the connecting flange or web plate.

C6.11.1.3

If at least two intermediate internal cross-frames or diaphragms are not provided in each span, it is essential that the web-to-flange welds be of sufficient size to develop the smaller of the full web or flange section. Full-thickness welds should be provided in this case because of the possibility of secondary flexural stresses developing in the box section as a result of vibrations and/or distortions of the cross section. Haaijer (1981) demonstrated that the transverse secondary distorsional stress range at the web-to-flange welded joint is reduced more than 50 percent in such sections when one intermediate internal cross-frame per span is introduced and more than 80 percent when two intermediate internal cross-frames per span are introduced. Thus, when two or more intermediate internal cross-frames or diaphragms are provided in each span, fillet welds on both sides of the web designed according to the requirements of Article 6.13.3.4 may be assumed to be adequate.

It is essential that the welds be placed on both sides of the connecting flange or web plate whether full penetration or fillet welds are used. This will help minimize the possibility of a fatigue failure resulting from the transverse bending stresses.

6.11.1.4—Access and Drainage

Access holes in box sections should be located in the bottom flange in areas of low stress. The effect of access holes on the stresses in the flange should be investigated at all limit states to determine if reinforcement of the holes is required. At access holes in box flanges subject to compression, the nominal flexural resistance of the remaining flange on each side of the hole at the strength limit state shall be determined according to the provisions of Article 6.10.8.2.2, with λ_f taken as the projecting width of the flange on that side of the hole divided by the flange thickness, including any reinforcement. Provisions should be made for ventilation and drainage of the interior of box sections.

C6.11.1.4

At access holes in box flanges subject to compression, the nominal flexural resistance of the remaining flange on each side of the hole is determined using the local buckling resistance equations for I-girder compression flanges, with the flange slenderness based on the projecting width of the flange on that side of the hole.

Outside access holes should be large enough to provide easy access for inspection. Doors for exterior access holes should be hinged and provided with locks. All outside openings in box sections should be screened to exclude unauthorized persons, birds, and vermin.

Consideration should be given to painting the interior of box sections a light color. Painting the interior of these sections is primarily done to facilitate inspections, and for tub sections, to prevent solar gain and to offer a minimum level of protection to the steel from the elements while the tub is temporarily open during construction. The paint quality need not match that normally used for exterior surfaces. A single-coat system should be sufficient in most cases, particularly when provisions are made for ventilation and drainage of the interior of the box.

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6.11.2—Cross-Section Proportion Limits

6.11.2.1—Web Proportions

6.11.2.1.1—General

Webs may be inclined or vertical. The inclination of the web plates to a plane normal to the bottom flange should not exceed 1 to 4. For the case of inclined webs, the distance along the web shall be used for checking all design requirements. Webs attached to top flanges of tub sections shall be attached at mid-width of the flanges.

C6.11.2.1.1

Inclined webs are advantageous in reducing the width of the bottom flange.

Top flanges of tub sections with webs located at other than mid-width of the flange are not to be used because additional lateral flange bending effects are introduced that would require special investigation.

6.11.2.1.2—Webs without Longitudinal Stiffeners

Webs shall be proportioned such that:

$$\frac{D}{t_w} \leq 150 \qquad (6.11.2.1.2\text{-}1)$$

C6.11.2.1.2

Eq. 6.11.2.1.2-1 is discussed in Article C6.10.2.1.1.

6.11.2.1.3—Webs with Longitudinal Stiffeners

Webs shall be proportioned such that:

$$\frac{D}{t_w} \leq 300 \qquad (6.11.2.1.3\text{-}1)$$

C6.11.2.1.3

Eq. 6.11.2.1.3-1 is discussed in Article C6.10.2.1.2.

6.11.2.2—Flange Proportions

Top flanges of tub sections subject to compression or tension shall be proportioned such that:

$$\frac{b_f}{2t_f} \leq 12.0, \qquad (6.11.2.2\text{-}1)$$

$$b_f \geq D/6, \qquad (6.11.2.2\text{-}2)$$

and:

$$t_f \geq 1.1t_w \qquad (6.11.2.2\text{-}3)$$

C6.11.2.2

Eqs. 6.11.2.2-1 through 6.11.2.2-3 apply to flanges of I-sections and are also applied to a single top flange of a tub section. Eqs. 6.11.2.2-1 through 6.11.2.2-3 are discussed in Article C6.10.2.2.

Box flanges should extend at least one inch beyond the outside of each web to allow for welding of the webs to the flange. The Engineer should consider providing an option on the design plans for the fabricator to increase this distance, if necessary, to provide for greater welding access.

6.11.2.3—Special Restrictions on Use of Live Load Distribution Factor for Multiple Box Sections

Cross sections of straight bridges consisting of two or more single-cell box sections, for which the live load flexural moment in each box is determined in accordance with the applicable provisions of Article 4.6.2.2.2b, shall satisfy the geometric restrictions specified herein. In addition, the bearing lines shall not be skewed.

The distance center-to-center of flanges of adjacent boxes, *a*, taken at the midspan, shall neither be greater than 120 percent nor less than 80 percent of the distance center-to-center of the flanges of each adjacent box, *w*, as illustrated in Figure 6.11.2.3-1. In addition to the midspan requirement, where nonparallel box sections are used, the

C6.11.2.3

Restrictions specified in this Article for straight bridges utilizing multiple box sections are necessary in order to employ the lateral live-load distribution factor given in Article 4.6.2.2.2b for straight multiple steel box sections. The development of this distribution factor is based on an extensive study of bridges that conform to these limitations (Johnston and Mattock, 1967). The study assumed an uncracked stiffness for the composite section along the entire span.

Further, it was determined that when these restrictions are satisfied, shear due to St. Venant torsion and secondary distorsional bending stress effects may be

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distance center-to-center of adjacent flanges at supports shall neither be greater than 135 percent nor less than 65 percent of the distance center-to-center of the flanges of each adjacent box. The distance center-to-center of flanges of each individual box shall be the same.

The inclination of the web plates to a plane normal to the bottom flange shall not exceed 1 to 4.

The cantilever overhang of the concrete deck, including curb and parapet, shall not be greater than either 60 percent of the average distance between the centers of the top steel flanges of adjacent box sections, *a*, or 6.0 ft.

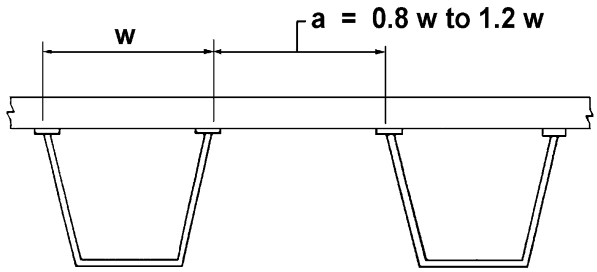

Figure 6.11.2.3-1—Center-to-Center Flange Distance

neglected if the width of the box flange does not exceed one-fifth of the effective span defined in Article 6.11.1.1. It was found from an analytical study of bridges of this type that when such bridges were loaded so as to produce maximum moment in a particular girder, and hence maximum compression in the flange plate near an intermediate support, the amount of twist in that girder was negligible. It therefore appears reasonable that, for bridges conforming to the restrictions set forth in this Article and with fully effective box flanges, shear due to torsion need not be considered in the design of box flanges for maximum compression or tension loads.

In the case of bridges with support skew, additional torsional effects occur in the box sections and the lateral distribution of loads is also affected. Although the bridge may satisfy the cross-section restrictions of this Article, these effects are not comprehended by the lateral distribution factor specified in Article 4.6.2.2.2b. Therefore, in these cases, a more rigorous analysis of stresses is necessary using one of the available methods of refined structural analysis. For straight portions of bridges that satisfy these restrictions, but that also contain horizontally-curved segments, a refined analysis is also recommended. Although not required, refined structural analysis methods may also be used for bridges satisfying the restrictions of this Article, if desired.

Some limitations are placed on the variation of the distance *a* with respect to the distance *w* shown in Figure 6.11.2.3-1 when the distribution factor is used because the studies on which the live load distribution provisions are based were made on bridges in which *a* and *w* were equal. The limitations given for nonparallel box sections will allow some flexibility of layout in design while generally maintaining the validity of the provisions. For cases with nonparallel box sections where the live load distribution factor is employed, refer to the provisions of Article 4.6.2.2.2b.

6.11.3—Constructibility

6.11.3.1—General

Except as specified herein, the provisions of Article 6.10.3 shall apply.

The individual box section geometry shall be maintained throughout all stages of construction. The need for temporary or permanent intermediate internal diaphragms or cross-frames, external diaphragms or cross-frames, top lateral bracing, or other means shall be investigated to ensure that deformations of the box section are controlled.

C6.11.3.1

The Engineer should consider possible eccentric loads that may occur during construction. These may include uneven placement of concrete and equipment. Temporary cross-frames or diaphragms that are not part of the original design should be removed because the structural behavior of the box section, including load distribution, may be significantly affected if these members are left in place.

Additional information on construction of composite box sections may be found in NSBA (1996) and United States Steel (1978).

For painted box sections, the Engineer should consider making an allowance for the weight of the paint. For typical structures, three percent of the steel weight is a reasonable allowance.

6.11.3.2—Flexure

For critical stages of construction, the provisions of Articles 6.10.3.2.1 through 6.10.3.2.3 shall be applied only to the top flanges of tub sections. The unbraced length should be taken as the distance between interior cross-frames or diaphragms. The provisions of Article A6.3.3 shall not be applied in determining the lateral–torsional buckling resistance of top flanges of tub sections with compact or noncompact webs.

For critical stages of construction, noncomposite box flanges in compression shall satisfy the following requirements:

$$f_{bu} \leq \phi_f F_{nc} \tag{6.11.3.2-1}$$

and:

$$f_{bu} \leq \phi_f F_{crw} \tag{6.11.3.2-2}$$

where:

- ϕ_f = resistance factor for flexure specified in Article 6.5.4.2
- f_{bu} = longitudinal flange stress due to the factored loads at the section under consideration calculated without consideration of longitudinal warping (ksi)
- F_{crw} = nominal bend-buckling resistance for webs specified in Article 6.10.1.9 (ksi)
- F_{nc} = nominal flexural resistance of box flanges in compression determined as specified in Article 6.11.8.2 (ksi). In computing F_{nc} for constructibility, the web load-shedding factor, R_b, shall be taken as 1.0.

For sections with compact or noncompact webs, Eq. 6.11.3.2-2 shall not be checked.

For critical stages of construction, noncomposite box flanges in tension and continuously braced box flanges in tension or compression shall satisfy the following requirement:

$$f_{bu} \leq \phi_f R_h F_{yf} \Delta \tag{6.11.3.2-3}$$

in which:

$$\Delta = \sqrt{1-3\left(\frac{f_v}{F_{yf}}\right)^2} \tag{6.11.3.2-4}$$

- f_v = St. Venant torsional shear stress in the flange due to the factored loads at the section under consideration (ksi)

C6.11.3.2

Although the equations of Articles 6.10.3.2.1 through 6.10.3.2.3 apply to flanges of I-sections, they may also safely be applied to a single top flange of a tub section. The provisions of Article 6.10.1.6 also apply when these equations are used.

Top lateral bracing attached to the flanges at points where only struts exist between the flanges may be considered as brace points at the discretion of the Engineer.

For straight girders, lateral bending in discretely braced top flanges of tub sections, before the concrete deck has hardened or is made composite, is caused by wind and by torsion from various origins. The equations of Articles 6.10.3.2.1 and 6.10.3.2.2 allow the Engineer to directly consider the effects of the flange lateral bending, if deemed significant. When the flange lateral bending effects are judged to be insignificant or incidental, the lateral bending term, f_ℓ, is simply set equal to zero in these equations. The format of the equations then reduces simply to the more conventional format for checking the flanges for the limit states of yielding, lateral–torsional buckling or local buckling, as applicable, in the absence of flange lateral bending. For horizontally-curved girders, flange lateral bending effects due to curvature must always be considered during construction. For loads applied during construction once the top flanges are continuously braced, the provisions of Article 6.10.3.2.3 apply. A distinction is made between discretely and continuously braced flanges in Article 6.10.3.2 because for a continuously braced flange, lateral flange bending need not be considered. Article C6.10.1.6 states the conditions for which top flanges may be considered continuously braced. St. Venant torsional shears are also typically neglected in continuously braced top flanges of tub sections. In checking the requirements of Articles 6.10.3.2.1 through 6.10.3.2.3 for a single top flange of a tub, it is recommended that the checks be made for half of the tub section.

In checking Eq. 6.10.3.2.1-2 for I-sections in straight bridges with compact or noncompact webs, Article A6.3.3 optionally permits the lateral–torsional buckling resistance of the compression flange to be determined including the beneficial contribution of the St. Venant torsional constant J. The use of these provisions is conservatively prohibited in checking top flanges of tub sections with compact or noncompact webs. The compact, noncompact, and slender web definitions are discussed in Article C6.10.6.2.3. For making these checks with the section in its noncomposite condition, the categorization of the web is to be based on the properties of the noncomposite section.

One potential source of flange lateral bending due to torsion is the effect of eccentric concrete deck overhang loads acting on cantilever forming brackets placed along exterior tub sections. In lieu of a more refined analysis, the maximum flange lateral bending moments in the outermost top flange of a tub due to these eccentric loadings

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$$= \frac{T}{2A_o t_f} \tag{6.11.3.2-5}$$

where:

A_o = enclosed area within the box section (in.^{2})
R_h = hybrid factor determined as specified in Article 6.10.1.10.1
T = internal torque due to the factored loads (kip-in.)

For loads applied to a composite box flange before the concrete has hardened or is made composite, the flange shall be designed as a noncomposite box flange. The maximum vertical deflection of the noncomposite box flange due to the unfactored permanent loads, including the self-weight of the flange plus the unfactored construction loads, shall not exceed 1/360 times the transverse span between webs. The through-thickness bending stress in the noncomposite box flange due to the factored permanent loads and factored construction loads shall not exceed 20.0 ksi. The weight of wet concrete and other temporary or permanent loads placed on a noncomposite box flange may be considered by assuming the box flange acts as a simple beam spanning between webs. Stiffening of the flange may be used where required to control flange deflection and stresses due to loads applied before the concrete deck has hardened or is made composite.

may be estimated using either Eq. C6.10.3.4.1-1 or C6.10.3.4.1-2 depending on how the lateral load is assumed applied to the flange.

In box sections with inclined webs, the change in the horizontal component of the web dead load shear plus the change in the St. Venant torsional dead load shear per unit length along the member acts as a uniformly distributed transverse load on the girder flanges. Additional intermediate internal cross-frames, diaphragms or struts may be required to reduce the lateral bending in discretely braced top flanges of tub sections resulting from this transverse load. This may be particularly true for cases where the inclination of the web plates to a plane normal to the bottom flange is permitted to exceed 1 to 4, and/or where the unbraced length of the top flanges exceeds 30 ft. Otherwise, this transverse load can typically be ignored. The maximum lateral flange bending moments due to this transverse load can be estimated using Eq. C6.10.3.4.1-1 in lieu of a more refined analysis, where F_ℓ is taken as the magnitude of the factored uniformly distributed transverse load. The entire transverse load should be assumed applied to the top flanges (Fan and Helwig, 1999). The cross-frame or strut can be assumed to carry the entire transverse load within the panel under consideration.

Another potential source of flange lateral bending is due to the forces that develop in Warren-type single-diagonal top lateral bracing systems due to flexure of the tub section. Refer to Article C6.7.5.3 for further discussion regarding this topic.

In cases where a full-length lateral bracing system is not employed within a tub section, as discussed further in Article C6.7.5.3, the minimum width of the top flanges within each individual unspliced field section should satisfy the guideline given by Eq. C6.10.2.2-1, in conjunction with the flange proportion limits specified in Article 6.11.2.2. In this case, L_{fs} in Eq. C6.10.2.2-1 is to be taken as the larger of the distances along the field section between panels of lateral bracing or between a panel of lateral bracing and the end of the piece. For cases where a full-length lateral bracing system is employed, Eq. C6.10.2.2-1 need not be considered for top flanges of tub sections.

For noncomposite box flanges in compression, local buckling of the flange during critical stages of construction is checked according to Eq. 6.11.3.2-1. Flange lateral bending and lateral–torsional buckling are not a consideration for box flanges.

Eq. 6.11.3.2-2 ensures that theoretical web bend-buckling will not occur during construction at sections where noncomposite box flanges are subject to compression. Eq. 6.10.3.2.1-3 serves a similar function at sections where top flanges of tub sections are subject to compression. For box sections with inclined webs, D_c should be taken as depth of the web in compression measured along the slope in determining the web bend-buckling resistance, F_{crw}, in either case. Because the flange stress is limited to the web bend-buckling stress, the R_b factor is always to be taken equal to 1.0 in computing the nominal flexural resistance of the compression flange for constructibility. Options to consider should the flange not satisfy Eq. 6.11.3.2-2 or Eq. 6.10.3.2.1-3, as applicable, for the construction condition are discussed in Article C6.10.3.2.1. For sections with compact or noncompact webs, web bend-buckling is not a consideration, and therefore, need not be checked for these sections.

For noncomposite box flanges in tension, or for continuously braced box flanges in tension or compression, the von Mises yield criterion (Boresi et al., 1978) is used in Eq. 6.11.3.2-3 to consider the effect of the torsional shear.

Longitudinal warping stresses due to cross-section distortion typically need not be considered in checking Eqs. 6.11.3.2-1 and 6.11.3.2-3, but are required to be considered when checking slip of the connections in bolted flange splices for the construction condition as specified in Article 6.13.6.1.3b.

In closed-box sections, noncomposite box flanges on top of the box receive the weight of wet concrete and other loads during construction before the deck hardens or is made composite. Transverse and/or longitudinal stiffening of the box flange may be required to control box-flange deflection and stresses.

6.11.3.3—Shear

When checking the shear requirement specified in Article 6.10.3.3, the provisions of Article 6.11.9 shall also apply, as applicable.

6.11.4—Service Limit State

Except as specified herein, the provisions of Article 6.10.4 shall apply.

The f_ℓ term in Eq. 6.10.4.2.2-2 shall be taken equal to zero. Eq. 6.10.4.2.2-3 shall not apply. Except for sections in positive flexure in which the web satisfies the requirement of Article 6.11.2.1.2, all sections shall satisfy Eq. 6.10.4.2.2-4.

Redistribution of the negative moment due to the Service II loads at interior-pier sections in continuous-span flexural members using the procedures specified in Appendix B6 shall not apply.

C6.11.4

Article 6.10.4.1 refers to the provisions of Article 2.5.2.6, which contain optional live-load deflection criteria and criteria for span-to-depth ratios. In the absence of depth restrictions, the span-to-depth ratios listed for I-sections can be used to establish a reasonable minimum web depth for the design. However, because of the inherent torsional stiffness of a box section, the optimum depth for a box section will typically be slightly less than the optimum depth for an I-section of the same span. Because the size of box flanges can typically be varied less over the bridge length, establishing a sound optimum depth for box sections is especially important. Boxes that are overly shallow may be subject to larger torsional shears.

Under the load combinations specified in Table 3.4.1-1, Eqs. 6.10.4.2.2-1 and 6.10.4.2.2-2 need only be checked for compact sections in positive flexure. For sections in negative flexure and noncompact sections in positive flexure, these equations do not control and need not be checked. However, Eq. 6.10.4.2.2-4 must still be checked for these sections where applicable.

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Flange lateral bending is not a consideration for box flanges, and therefore, need not be considered when checking Eq. 6.10.4.2.2-2. Flange lateral bending is not considered in Eq. 6.10.4.2.2-1 because the top flanges are continuously braced at the service limit state. Longitudinal warping stresses due to cross-section distortion need not be considered in checking the equations of Article 6.10.4.2.2, but are required to be considered when checking slip of the connections in bolted flange splices at the service limit state as specified in Article 6.13.6.1.3b. St. Venant torsional shear stresses are also not considered in checking the equations of Article 6.10.4.2.2 for box flanges. The effects of longitudinal warping stresses and torsional shear on the overall permanent deflections at the service limit state are considered to be relatively insignificant.

For box sections with inclined webs, D_c should be taken as the depth of the web in compression measured along the slope in determining the web bend-buckling resistance, F_{crw}, for checking Eq. 6.10.4.2.2-4, where applicable.

The applicability of the optional provisions of Appendix B6 to box sections has not been demonstrated. Therefore, these provisions may not be used in the design of box sections.

6.11.5—Fatigue and Fracture Limit State

Except as specified herein, the provisions of Article 6.10.5 shall apply.

For fatigue in shear connectors, the provisions of Article 6.11.10 shall also apply, as applicable. The provisions for fatigue in shear connectors specified in Article 6.10.10.3 shall not apply.

When checking the shear requirement specified in Article 6.10.5.3, the provisions of Article 6.11.9 shall also apply, as applicable.

Longitudinal warping stresses and transverse bending stresses due to cross-section distortion shall be considered for:

- Single box sections in straight or horizontally-curved bridges,
- Multiple box sections in straight bridges not satisfying the requirements of Article 6.11.2.3,
- Multiple box sections in horizontally-curved bridges, or
- Any single or multiple box section with a box flange that is not fully effective according to the provisions of Article 6.11.1.1.

The stress range due to longitudinal warping shall be considered in checking the fatigue resistance of the base metal at all details on the box section according to the provisions specified in Article 6.6.1. The transverse bending stress range shall be considered separately in evaluating the fatigue resistance of the base metal

C6.11.5

When a box section is subjected to a torsional load, its cross section distorts and is restored at diaphragms or cross-frames. This distortion gives rise to secondary bending stresses. A torsional loading in the opposite direction produces a reversal of these distorsional secondary bending stresses. In certain cases, as defined herein, these distorsional stresses are to be considered when checking fatigue. Situations for which these stresses are of particular concern and for which these stresses may potentially be ignored are discussed in Article C6.11.1.1.

Where force effects in the cross-frames or diaphragms are computed from a refined analysis, stress ranges for checking load-induced fatigue and torque ranges for checking fatigue due to cross-section distortion in cross-frame and diaphragm members should be determined as recommended in Article C6.6.1.2.1. Transverse bending and longitudinal warping stress ranges due to cross-section distortion can be determined using the BEF analogy, as discussed in Article C6.11.1.1. Longitudinal warping stresses are considered additive to the longitudinal major-axis bending stresses

The most critical case for transverse bending is likely to be the base metal at the termination of fillet welds connecting transverse elements to webs and box flanges. A stress concentration occurs at the termination of these welds as a result of the transverse bending. The fatigue resistance of this detail when subject to transverse

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adjacent to flange-to-web fillet welds and adjacent to the termination of fillet welds connecting transverse elements to webs and box flanges. The need for a bottom transverse member within the internal cross-frames to resist the transverse bending stress range in the bottom box flange at the termination of fillet welds connecting cross-frame connection plates to the flange shall be considered. Transverse cross-frame members next to box flanges shall be attached to the box flange unless longitudinal flange stiffeners are used, in which case the transverse members shall be attached to the longitudinal stiffeners by bolting. The moment of inertia of these transverse cross-frame members shall not be less than the moment of inertia of the largest connection plate for the internal cross-frame under consideration taken about the edge in contact with the web.

bending is not currently quantified, but is anticipated to be perhaps as low as Category E.

Should this situation be found critical in the web at transverse web stiffeners not serving as connection plates, the transverse bending stress range may be reduced by welding the stiffeners to the flanges. Attaching transverse stiffeners to the flanges reduces the sharp through-thickness bending stresses within the unstiffened portions of the web at the termination of the stiffener-to-web welds, which is usually the most critical region for this check. Cross-frame connection plates already are required to be attached to the flanges according to the provisions of Article 6.6.1.3.1 for this reason.

Should it become necessary to reduce the transverse bending stress range in the box flange adjacent to the cross-frame connection plate welds to the flange, the provision of transverse cross-frame members across the bottom of the box or tub as part of the internal cross-bracing significantly reduces the transverse bending stress range at the welds and ensures that the cross-section shape is retained. Closer spacing of cross-frames also leads to lower transverse bending stresses. Where bottom transverse cross-frame members are provided, they are to be attached to the box flange or to the longitudinal flange stiffeners, as applicable. For closed-box sections, the top transverse cross-frame members should be similarly attached. Where transverse bracing members are welded directly to the box flange, the stress range due to transverse bending should also be considered in checking the fatigue resistance of the base metal adjacent to the termination of these welds. Where transverse bracing members are connected to longitudinal flange stiffeners, the box flange may be considered stiffened when computing the transverse bending stresses. In such cases, the transverse connection plates must still be attached to both flanges as specified in Article 6.6.1.3.1.

Load-induced fatigue is usually not critical for top lateral bracing in tub sections since the concrete deck is much stiffer and resists more of the load than does the bracing. Since the deck resists the majority of the torsional shear in these cases, it is advisable to check the reinforcement in the deck for the additional horizontal shear. Severely skewed supports may cause critical horizontal deck shear.

It is advisable to connect the lateral bracing to the top flanges to eliminate a load path through the web. Although removable deck forms are problematic in tub girders, they are sometimes required by the Owner. In such cases, it may be necessary to lower the lateral bracing by attaching it to the box webs. In these cases, connections to the webs must be made according to the requirements of Article 6.6.1.3.2 to prevent potential problems resulting from fatigue. An adequate load path, with fatigue considered, must be provided between the bracing-to-web connections and the top flanges. Connections of the lateral bracing to the web can be avoided by using metal stay-in-place deck forms.

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Fatigue of the base metal at the net section of access holes or manholes should be considered. The fatigue resistance at the net section of large access holes satisfying specified geometric conditions and made to the requirements of the *AASHTO/AWS D1.5M/D1.5 Bridge Welding Code* (AASHTO/AWS, 2015) is provided in Condition 1.6 of Table 6.6.1.2.3-1. The classification of large access holes satisfying the specified conditions as fatigue detail Category C assumes a stress concentration, or ratio of the elastic tensile stress adjacent to the hole to the average stress on the net area, of less than 2.4.

6.11.6—Strength Limit State

6.11.6.1—General

For the purposes of this Article, the applicable Strength load combinations specified in Table 3.4.1-1 shall apply.

C6.11.6.1

At the strength limit state, Article 6.11.6 directs the Engineer to the appropriate Articles for the design of box sections in regions of positive or negative flexure.

6.11.6.2—Flexure

6.11.6.2.1—General

If there are holes in the tension flange at the section under consideration, the tension flange shall satisfy the requirement specified in Article 6.10.1.8.

C6.11.6.2.1

Where an access hole is provided in the tension flange, the hole should be deducted in determining the gross section for checking the requirement of Article 6.10.1.8, as specified in Article 6.8.1.

A continuously braced flange in compression is assumed not to be subject to local or lateral–torsional buckling, as applicable. The rationale for excluding these limit state checks is discussed in Article C6.10.3.2.3.

These provisions assume low or zero levels of axial force in the member and uniaxial flexure. For members that are also subject to a factored concentrically-applied axial force, P_u, in excess of 5 percent of the factored axial resistance of the member, P_r or P_{ry} as applicable, at the strength limit state, and/or if the member is subject to biaxial bending, the member should instead be checked according to the provisions of Article 6.8.2.3 or 6.9.2.2, as applicable. The level of 5 percent is based conservatively on the linear interaction equations given in these Articles, which apply in the majority of cases. Below this level, it is reasonable to ignore the effect of the axial force in the design of the member.

6.11.6.2.2—Sections in Positive Flexure

Sections in horizontally-curved steel girder bridges shall be considered as noncompact sections and shall satisfy the requirements of Article 6.11.7.2.

Sections in straight bridges that satisfy the following requirements shall qualify as compact sections:

- The specified minimum yield strengths of the flanges and web do not exceed 70.0 ksi,

C6.11.6.2.2

The nominal flexural resistance of sections in positive flexure within straight bridges satisfying the requirements of Article 6.11.2.3 and that also satisfy specific steel grade, web slenderness, effective flange width and ductility requirements is permitted to exceed the moment at first yield according to the provisions of Article 6.10.7. The nominal flexural resistance of these sections, termed compact sections, is therefore more appropriately expressed in terms of moment. For sections in positive flexure in straight bridges not satisfying

- The web satisfies the requirement of Article 6.11.2.1.2,
- The section is part of a bridge that satisfies the requirements of Article 6.11.2.3,
- The box flange is fully effective as specified in Article 6.11.1.1,

and:

- The section satisfies the web slenderness limit:

$$\frac{2D_{cp}}{t_w} \leq 3.76\sqrt{\frac{E}{F_{yc}}} \qquad (6.11.6.2.2\text{-}1)$$

where:

D_{cp} = depth of the web in compression at the plastic moment determined as specified in Article D6.3.2 (in.)

Compact sections shall satisfy the requirements of Article 6.11.7.1. Otherwise, the section shall be considered noncompact and shall satisfy the requirements of Article 6.11.7.2.

Compact and noncompact sections shall satisfy the ductility requirement specified in Article 6.10.7.3.

one or more of these requirements, or for composite sections in positive flexure in horizontally-curved bridges, termed noncompact sections, the nominal flexural resistance is not permitted to exceed the moment at first yield. The nominal flexural resistance in these cases is therefore more appropriately expressed in terms of the elastically computed flange stress.

For reasons discussed in Article C6.10.6.2.2, composite sections in positive flexure in straight bridges with flange yield strengths greater than 70.0 ksi or with webs that do not satisfy Article 6.11.2.1.2 or Eq. 6.11.6.2.2-1 are to be designed at the strength limit state as noncompact sections as specified in Article 6.11.7.2. Furthermore, if the section is not part of a straight bridge that satisfies the restrictions specified in Article 6.11.2.3, or is part of a horizontally-curved bridge, or if the box flange is not fully effective as defined in Article 6.11.1.1, the section must be designed as a noncompact section. The ability of such sections to develop a nominal flexural resistance greater than the moment at first yield in the presence of potentially significant St. Venant torsional shear and cross-sectional distortion stresses has not been demonstrated.

Compact sections in positive flexure must satisfy the provisions of Article 6.10.7.3 to ensure a ductile mode of failure. Noncompact sections must also satisfy the ductility requirement specified in Article 6.10.7.3 to ensure a ductile failure. Satisfaction of this requirement ensures an adequate margin of safety against premature crushing of the concrete deck for sections utilizing up to 100-ksi steels and/or for sections utilized in shored construction. This requirement is also a key limit in allowing web bend-buckling to be disregarded in the design of composite sections in positive flexure when the web also satisfies Article 6.11.2.1.2, as discussed in Article C6.10.1.9.1.

6.11.6.2.3—Sections in Negative Flexure

The provisions of Article 6.11.8 shall apply. The provisions of Appendix A6 shall not apply. Redistribution of the negative moment due to the factored loads at interior-pier sections in continuous-span flexural members using the procedures specified in Appendix B6 shall not apply.

C6.11.6.2.3

For sections in negative flexure, the provisions of Article 6.11.8 limit the nominal flexural resistance to be less than or equal to the moment at first yield for all types of box girder bridges. As a result, the nominal flexural resistance for these sections is conveniently expressed in terms of the elastically computed flange stress.

The applicability of the optional provisions of Appendices A6 and B6 to box sections has not been demonstrated. Therefore, these provisions may not be used in the design of box sections.

6.11.6.3—Shear

The provisions of Article 6.11.9 shall apply.

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6.11.6.4—Shear Connectors

The provisions of Article 6.10.10.4 shall apply. The provisions of Article 6.11.10 shall also apply, as applicable.

6.11.7—Flexural Resistance—Sections in Positive Flexure

6.11.7.1—Compact Sections

6.11.7.1.1—General

At the strength limit state, the section shall satisfy:

$$M_u \leq \phi_f M_n \tag{6.11.7.1.1-1}$$

where:

ϕ_f = resistance factor for flexure specified in Article 6.5.4.2

M_n = nominal flexural resistance of the section determined as specified in Article 6.11.7.1.2 (kip-in.)

M_u = bending moment about the major-axis of the cross section due to the factored loads at the section under consideration (kip-in.)

C6.11.7.1.1

For composite sections in positive flexure, lateral bending does not need to be considered in the compression flanges of tub sections at the strength limit state because the flanges are continuously supported by the concrete deck. Flange lateral bending is also not a consideration for box flanges.

6.11.7.1.2—Nominal Flexural Resistance

The nominal flexural resistance of the section shall be taken as specified in Article 6.10.7.1.2, except that for continuous spans, the nominal flexural resistance shall always be subject to the limitation of Eq. 6.10.7.1.2-3.

C6.11.7.1.2

The equations of Article 6.10.7.1.2 are discussed in detail in Article C6.10.7.1.2.

For box sections, Eq. 6.10.7.1.2-3 is to always be used for determining the limiting nominal flexural resistance of compact sections in positive flexure in straight continuous spans. The provisions of Appendix B6, which ensure that interior-pier sections will have sufficient ductility and robustness such that the redistribution of moments caused by partial yielding within the positive flexural regions is inconsequential, are not presently applicable to box sections.

6.11.7.2—Noncompact Sections

6.11.7.2.1—General

At the strength limit state, compression flanges shall satisfy:

$$f_{bu} \leq \phi_f F_{nc} \tag{6.11.7.2.1-1}$$

where:

ϕ_f = resistance factor for flexure specified in Article 6.5.4.2

f_{bu} = longitudinal flange stress at the section under consideration calculated without consideration

C6.11.7.2.1

For noncompact sections, the compression flange must satisfy Eq. 6.11.7.2.1-1 and the tension flange must satisfy Eq. 6.11.7.2.1-2 at the strength limit state. For composite sections in positive flexure, lateral bending does not need to be considered in the compression flanges at the strength limit state because the flanges are continuously supported by the concrete deck. Lateral bending is also not a consideration for the tension flange, which is always a box flange in this case.

For noncompact sections, the longitudinal stress in the concrete deck is limited to $0.6f'_c$ to ensure linear behavior of the concrete, which is assumed in the of flange lateral bending or longitudinal warping, as applicable (ksi)

F_{nc} = nominal flexural resistance of the compression flange determined as specified in Article 6.11.7.2.2 (ksi)

The tension flange shall satisfy:

$$f_{bu} \leq \phi_f F_{nt} \qquad (6.11.7.2.1\text{-}2)$$

where:

F_{nt} = nominal flexural resistance of the tension flange determined as specified in Article 6.11.7.2.2 (ksi)

The maximum longitudinal compressive stress in the concrete deck at the strength limit state, determined as specified in Article 6.10.1.1.1d, shall not exceed $0.6f'_c$.

calculation of the steel flange stresses. This condition is unlikely to govern except in cases involving: (1) shored construction, or unshored construction where the noncomposite steel dead load stresses are low, combined with (2) geometries causing the neutral axis of the short-term and long-term composite section to be significantly below the bottom of the concrete deck.

6.11.7.2.2—Nominal Flexural Resistance

The nominal flexural resistance of the compression flanges of tub sections shall be taken as:

$$F_{nc} = R_b R_h F_{yc} \qquad (6.11.7.2.2\text{-}1)$$

where:

R_b = web load-shedding factor determined as specified in Article 6.10.1.10.2

R_h = hybrid factor determined as specified in Article 6.10.1.10.1

The nominal flexural resistance of the compression flange of closed-box sections shall be taken as:

$$F_{nc} = R_b R_h F_{yc} \Delta \qquad (6.11.7.2.2\text{-}2)$$

in which:

$$\Delta = \sqrt{1 - 3\left(\frac{f_v}{F_{yc}}\right)^2} \qquad (6.11.7.2.2\text{-}3)$$

f_v = St. Venant torsional shear stress in the flange due to the factored loads at the section under consideration (ksi)

$$= \frac{T}{2A_o t_{fc}} \qquad (6.11.7.2.2\text{-}4)$$

where:

A_o = enclosed area within the box section (in.^{2})

T = internal torque due to the factored loads (kip-in.)

C6.11.7.2.2

The nominal flexural resistance of noncompact sections in positive flexure is limited to the moment at first yield. Thus, the nominal flexural resistance is expressed simply in terms of the flange stress. For noncompact sections, the elastically computed stress in each flange due to the factored loads, determined in accordance with Article 6.10.1.1.1a, is compared with the yield stress of the flange times the appropriate flange-stress reduction factors.

For box flanges, the effect of the St. Venant torsional shear stress in the flange must also be considered where necessary. The computation of the flange torsional shear stress from Eq. 6.11.7.2.2-4 or 6.11.7.2.2-7, as applicable, due to torques applied separately to the noncomposite and composite sections is discussed in Article C6.11.1.1.

The nominal flexural resistance of the tension flange of closed-box and tub sections shall be taken as:

$$F_{nt} = R_h F_{yt} \Delta \qquad (6.11.7.2.2\text{-}5)$$

in which:

$$\Delta = \sqrt{1 - 3\left(\frac{f_v}{F_{yt}}\right)^2} \qquad (6.11.7.2.2\text{-}6)$$

f_v = St. Venant torsional shear stress in the flange due to the factored loads at the section under consideration (ksi)

$$= \frac{T}{2A_o t_{ft}} \qquad (6.11.7.2.2\text{-}7)$$

6.11.8—Flexural Resistance—Sections in Negative Flexure

6.11.8.1—General

6.11.8.1.1—Box Flanges in Compression

At the strength limit state, the following requirement shall be satisfied:

$$f_{bu} \le \phi_f F_{nc} \qquad (6.11.8.1.1\text{-}1)$$

where:

ϕ_f = resistance factor for flexure specified in Article 6.5.4.2

f_{bu} = longitudinal flange stress due to the factored loads at the section under consideration calculated without consideration of longitudinal warping (ksi)

F_{nc} = nominal flexural resistance of the flange determined as specified in Article 6.11.8.2 (ksi)

C6.11.8.1.1

Eq. 6.11.8.1.1-1 ensures that box flanges in compression have sufficient strength with respect to flange local buckling. Flange lateral bending and lateral–torsional buckling are not a consideration for box flanges.

In general, bottom box flanges at interior-pier sections are subjected to biaxial stresses due to major-axis bending of the box section and major-axis bending of the internal diaphragm over the bearing sole plate. The flange is also subject to shear stresses due to the internal diaphragm vertical shear, and in cases where it must be considered, the St. Venant torsional shear. Bending of the internal diaphragm over the bearing sole plate can be particularly significant for boxes supported on single bearings. For cases where the shear stresses and/or bending of the internal diaphragm are deemed significant, the following equation may be used to check this combined stress state in the box flange at the strength limit state:

$$\sqrt{f_{bu}^2 - f_{bu} f_{by} + f_{by}^2 + 3\left(f_d + f_v\right)^2} \le \phi_f R_b R_h F_{yc} \qquad (C6.11.8.1.1\text{-}1)$$

where:

f_{by} = stress in the flange due to the factored loads caused by major-axis bending of the internal diaphragm over the bearing sole plate (ksi)

f_d = shear stress in the flange caused by the internal diaphragm vertical shear due to the factored loads (ksi)

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f_v = St. Venant torsional shear stress in the flange due to the factored loads (ksi)
R_b = web load-shedding factor determined as specified in Article 6.10.1.10.2
R_h = hybrid factor determined as specified in Article 6.10.1.10.1

Eq. C6.11.8.1.1-1 represents the general form of the Huber-von Mises-Hencky yield criterion (Ugural and Fenster, 1978).

For a box supported on two bearings, f_{by} in Eq. C6.11.8.1.1-1 is typically relatively small and can often be neglected.

The box flange may be considered effective with the internal diaphragm at interior-pier sections in making this check. A flange width equal to six times its thickness may be considered effective with the internal diaphragm. The shear stress in the flange, f_d, caused by the internal diaphragm vertical shear due to the factored loads can then be estimated as:

$$f_d = \frac{VQ}{It_{fc}} \quad \text{(C6.11.8.1.1-2)}$$

where:

V = vertical shear in the internal diaphragm due to flexure plus St. Venant torsion (kip)
Q = first moment of one-half the effective box-flange area about the neutral axis of the effective internal diaphragm section (in.^{3})
I = moment of inertia of the effective internal diaphragm section (in.^{4})

Wherever an access hole is provided within the internal diaphragm, the effect of the hole should be considered in computing the section properties of the effective diaphragm section.

6.11.8.1.2—Continuously Braced Flanges in Tension

At the strength limit state, the following requirement shall be satisfied:

$$f_{bu} \leq \phi_f F_{nt} \quad \text{(6.11.8.1.2-1)}$$

where:

F_{nt} = nominal flexural resistance of the flange determined as specified in Article 6.11.8.3 (ksi)

C6.11.8.1.2

For continuously braced top flanges of tub sections, lateral flange bending need not be considered. St. Venant torsional shears are also typically neglected. The torsional shears may not be neglected, however, in a continuously braced box flange.

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6.11.8.2—Flexural Resistance of Box Flanges in Compression

6.11.8.2.1—General

The nominal flexural resistance of box flanges in compression without flange longitudinal stiffeners shall be determined as specified in Article 6.11.8.2.2. The nominal flexural resistance of box flanges in compression with flange longitudinal stiffeners shall be determined as specified in Article 6.11.8.2.3.

6.11.8.2.2—Unstiffened Flanges

The nominal flexural resistance of the compression flange, F_{nc}, shall be taken as:

$$F_{nc} = F_{cb}\sqrt{1-\left(\frac{f_v}{\phi_v F_{cv}}\right)^2} \qquad (6.11.8.2.2\text{-}1)$$

in which:

F_{cb} = nominal axial compression buckling resistance of the flange under compression alone calculated as follows (ksi):

- If $\lambda_f \leq \lambda_p$, then:

$$F_{cb} = R_b R_h F_{yc}\Delta \qquad (6.11.8.2.2\text{-}2)$$

- If $\lambda_p < \lambda_f \leq \lambda_r$, then:

$$F_{cb} = R_b R_h F_{yc}\left[\Delta - \left(\Delta - \frac{\Delta - 0.3}{R_h}\right)\left(\frac{\lambda_f - \lambda_p}{\lambda_r - \lambda_p}\right)\right] \qquad (6.11.8.2.2\text{-}3)$$

- If $\lambda_f > \lambda_r$, then:

$$F_{cb} = \frac{0.9ER_b k}{\lambda_f^2} \qquad (6.11.8.2.2\text{-}4)$$

F_{cv} = nominal shear buckling resistance of the flange under shear alone calculated as follows (ksi):

- If $\lambda_f \leq 1.12\sqrt{\frac{Ek_s}{F_{yc}}}$, then:

$$F_{cv} = 0.58F_{yc} \qquad (6.11.8.2.2\text{-}5)$$

- If $1.12\sqrt{\frac{Ek_s}{F_{yc}}} < \lambda_f \leq 1.40\sqrt{\frac{Ek_s}{F_{yc}}}$, then:

C6.11.8.2.2

For unstiffened flanges, the slenderness is based on the full flange width between webs, b_{fc}.

For flanges under combined normal stress and torsional shear stress, the following nonlinear interaction curve is used to derive the resistance of the flange (NHI, 2011a):

$$\left(\frac{f_v}{\phi_v F_{cv}}\right)^2 + \left(\frac{f_c}{\phi_f F_{cb}}\right)^2 \leq 1.0 \qquad (C6.11.8.2.2\text{-}1)$$

Rearranging Eq. C6.11.8.2.2-1 in terms of f_c and substituting F_{nc} for f_c facilitates the definition of the nominal flexural resistance of the compression flange as provided in Eq. 6.11.8.2.2-1. A general discussion of the problem of reduction of critical local buckling stresses due to the presence of torsional shear may be found in Ziemian (2010).

The nominal axial compression buckling resistance of the flange under compression alone, F_{cb}, is defined for three distinct regions based on the slenderness of the flange. The elastic buckling resistance of the flange given by Eq. 6.11.8.2.2-4 is based on the theoretical elastic Euler buckling equation for an infinitely long plate under a uniform normal stress (Timoshenko and Gere, 1961). For stocky plates, full yielding of the plate as defined by the von Mises yield criterion for combined normal and shear stress (Boresi et al., 1978) can be achieved. For such plates, F_{cb} is defined by Eq. 6.11.8.2.2-2. In between these two regions is a transition region that reflects the fact that partial yielding due to residual stresses and initial imperfections does not permit the attainment of the elastic buckling stress. The nominal flexural resistance of the flange in this region is expressed in Eq. 6.11.8.2.2-3 as a linear function of the flange slenderness. A residual stress level equal to $0.3F_{yc}$ is assumed in the presence of no shear.

The limiting flange slenderness, λ_p, defining whether to use Eq. 6.11.8.2.2-2 or 6.11.8.2.2-3 is defined as 0.6 times the flange slenderness at which the elastic buckling stress given by Eq. 6.11.8.2.2-4 equals $R_bF_{yc}\Delta$. The limiting flange slenderness, λ_r, defining whether to use Eq. 6.11.8.2.2-3 or 6.11.8.2.2-4 is defined as the flange slenderness at which the elastic buckling stress

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$$F_{cv} = \frac{0.65\sqrt{F_{yc}Ek_s}}{\lambda_f} \quad (6.11.8.2.2\text{-}6)$$

- If $\lambda_f > 1.40\sqrt{\frac{Ek_s}{F_{yc}}}$, then:

$$F_{cv} = \frac{0.9Ek_s}{\lambda_f^2} \quad (6.11.8.2.2\text{-}7)$$

λ_f = slenderness ratio for the compression flange

$$= \frac{b_{fc}}{t_{fc}} \quad (6.11.8.2.2\text{-}8)$$

$$\lambda_p = 0.57\sqrt{\frac{Ek}{F_{yc}\Delta}} \quad (6.11.8.2.2\text{-}9)$$

$$\lambda_r = 0.95\sqrt{\frac{Ek}{F_{yr}}} \quad (6.11.8.2.2\text{-}10)$$

$$\Delta = \sqrt{1-3\left(\frac{f_v}{F_{yc}}\right)^2} \quad (6.11.8.2.2\text{-}11)$$

f_v = St. Venant torsional shear stress in the flange due to the factored loads at the section under consideration (ksi)

$$= \frac{T}{2A_o t_{fc}} \quad (6.11.8.2.2\text{-}12)$$

F_{yr} = smaller of the compression flange stress at the onset of nominal yielding, with consideration of residual stress effects, or the specified minimum yield strength of the web (ksi)

$$= (\Delta - 0.3)F_{yc} \quad (6.11.8.2.2\text{-}13)$$

k = plate-buckling coefficient for uniform normal stress

= 4.0

k_s = plate-buckling coefficient for shear stress

= 5.34

given by Eq. 6.11.8.2.2-4 equals R_bF_{yr}, where F_{yr} is given by Eq. 6.11.8.2.2-13.

The equations for the nominal shear buckling resistance of the flange under shear alone, F_{cv}, are determined from the equations for the constant, C, given in Article 6.10.9.3.2, where C is the ratio of the shear buckling resistance to the shear yield strength of the flange taken as $F_{yc}/\sqrt{3}$.

The computation of the flange torsional shear stress, f_v, from Eq. 6.11.8.2.2-12 due to torques applied separately to the noncomposite and composite sections is discussed in Article C6.11.1.1. In cases where f_v is relatively small, consideration might be given to assuming Δ equal to 1.0 and F_{nc} equal to F_{cb} for preliminary design.

The specified plate-buckling coefficient for uniform normal stress, k, and shear-buckling coefficient, k_s, assume simply-supported boundary conditions at the edges of the flanges (Timoshenko and Gere, 1961).

The term R_b is a postbuckling strength reduction factor that accounts for the reduction in the section flexural resistance caused by the shedding of compressive stresses from a slender web and the corresponding increase in the flexural stress within the compression flange. The R_h factor accounts for the reduced contribution of the web to the nominal flexural resistance at first yield in any flange element, due to earlier yielding of the lower strength steel in the web of a hybrid section. The R_b and R_h factors are discussed in greater detail in Articles C6.10.1.10.2 and C6.10.1.10.1, respectively. In calculating R_b and R_h for a tub section, use one-half of the effective box flange width in conjunction with one top flange and a single web, where the effective box flange width is defined in Article 6.11.1.1. For a closed-box section, use one-half of the effective top and bottom box flange width in conjunction with a single web.

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where:

ϕ_f = resistance factor for flexure specified in Article 6.5.4.2
ϕ_v = resistance factor for shear specified in Article 6.5.4.2
b_{fc} = compression flange width between webs (in.)
A_o = enclosed area within the box section (in.^{2})
R_b = web load-shedding factor determined as specified in Article 6.10.1.10.2
R_h = hybrid factor determined as specified in Article 6.10.1.10.1
T = internal torque due to the factored loads (kip-in.)

6.11.8.2.3—Longitudinally Stiffened Flanges

The nominal flexural resistance of the compression flange shall be taken as equal to the nominal flexural resistance for the compression flange without longitudinal stiffeners, determined as specified in Article 6.11.8.2.2, with the following substitutions:

- w shall be substituted for b_{fc},
- The plate-buckling coefficient for uniform normal stress, k, shall be taken as:
- If $n = 1$, then:

$$k = \left(\frac{8I_s}{wt_{fc}^3}\right)^{\frac{1}{3}} \qquad (6.11.8.2.3\text{-}1)$$

- If $n = 2$, then:

$$k = \left(\frac{0.894I_s}{wt_{fc}^3}\right)^{\frac{1}{3}} \qquad (6.11.8.2.3\text{-}2)$$

$1.0 \le k \le 4.0$ and:

- the plate-buckling coefficient for shear stress, k_s, shall be taken as:

$$k_s = \frac{5.34 + 2.84\left(\frac{I_s}{wt_{fc}^3}\right)^{\frac{1}{3}}}{(n+1)^2} \le 5.34 \qquad (6.11.8.2.3\text{-}3)$$

where:

I_s = moment of inertia of a single longitudinal flange stiffener about an axis parallel to the flange and taken at the base of the stiffener (in.^{4})

n = number of equally spaced longitudinal flange stiffeners
w = larger of the width of the flange between longitudinal flange stiffeners or the distance

C6.11.8.2.3

When a noncomposite unstiffened box flange becomes so slender that nominal flexural resistance of the flange decreases to an impractical level, longitudinal stiffeners can be added to the flange.

The nominal flexural resistance of a longitudinally-stiffened box flange is determined using the same basic equations specified for unstiffened box flanges in Article 6.11.8.2.2. The width, w, must be substituted for b_{fc} in the equations. The shear-buckling coefficient, k_s, for a stiffened plate to be used in the equations is given by Eq. 6.11.8.2.3-3, which comes from Culver (1972). The plate-buckling coefficient for uniform normal stress, k, to be used in the equations is related to the stiffness of the longitudinal flange stiffeners and is derived from Eq. 6.11.11.2-2. k can take any value ranging from 1.0 to 4.0. However, a value of k ranging from 2.0 to 4.0 generally should be assumed. Eq. 6.11.8.2.3-1 applies for one longitudinal flange stiffener; i.e., n = 1, and Eq. 6.11.8.2.3-2 applies for two longitudinal flange stiffeners; i.e., n = 2. As discussed further in Article C6.11.11.2, as the number of stiffeners is increased beyond one, the required moment of inertia from Eq. 6.11.11.2-2 to achieve the desired k value begins to increase dramatically and eventually becomes nearly impractical. Therefore, for boxes of typical proportions, it is strongly recommended that the number of longitudinal flange stiffeners not exceed one for maximum economy.

Note that Eq. 6.11.11.2-2 is automatically satisfied by the longitudinal flange stiffener moment of inertia that is assumed in determining the k value from Eq. 6.11.8.2.3-1 or 6.11.8.2.3-2, as applicable, since Eqs. 6.11.8.2.3-1 and 6.11.8.2.3-2 are derived directly from Eq. 6.11.11.2-2. Another option in lieu of using Eq. 6.11.8.2.3-1 or 6.11.8.2.3-2 is to assume a k value and then determine the minimum required moment of inertia for each longitudinal flange stiffener from Eq. 6.11.11.2-2 that will provide the assumed value of k.

If the longitudinal flange stiffeners are very rigid, k will be at or near a value of 4.0 and plate buckling will be forced to occur between the stiffeners. Less rigid stiffeners will yield a lower value of k and a corresponding lower value of the flange nominal flexural

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from a web to the nearest longitudinal flange stiffener (in.)

Compression flange longitudinal stiffeners shall satisfy the requirements specified in Article 6.11.11.2.

resistance. Eqs. 6.11.8.2.3-1 and 6.11.8.2.3-2, or alternatively Eq. 6.11.11.2-2, allow the Engineer to match the required stiffener size to the required flange resistance rather than always providing the largest stiffener(s) required to obtain a k value equal to 4.0.

Longitudinal flange stiffeners are best discontinued at field splice locations, particularly when the span balance is such that the box flange on the other side of the field splice need not be stiffened. To accomplish this successfully, the flange splice plates must be split to allow the stiffener to be taken to the free edge of the flange where the flange normal stress is zero. The compressive resistance of the unstiffened box flange on the other side of the splice should be checked. Otherwise, if the stiffener must be discontinued in a region subject to a net tensile stress, determined as specified in Article 6.6.1.2.1, the termination of the stiffener-to-flange weld must be checked for fatigue according to the terminus detail. Where it becomes necessary to run the stiffener beyond the field splice, splicing the stiffener across the field splice is recommended.

6.11.8.3—Flexural Resistance Based on Tension Flange Yielding

The nominal flexural resistance of tub sections based on tension flange yielding shall be taken as:

$$F_{nt} = R_h F_{yt} \tag{6.11.8.3-1}$$

where:

R_h = hybrid factor determined as specified in Article 6.10.1.10.1

The nominal flexural resistance of closed-box sections based on tension flange yielding shall be determined from Eq. 6.11.7.2.2-5.

C6.11.8.3

For sections in which $M_{yt} > M_{yc}$, Eq. 6.11.8.3-1 or Eq. 6.11.7.2.2-5, as applicable, does not control and tension flange yielding need not be checked, where M_{yc} and M_{yt} are the yield moments with respect to the compression and tension flange, respectively, determined as specified in Article D6.2.

6.11.9—Shear Resistance

Except as specified herein, the provisions of Article 6.10.9 shall apply for determining the factored shear resistance of a single web. For the case of inclined webs, D in Article 6.10.9 shall be taken as the depth of the web plate measured along the slope.

For the case of inclined webs, each web shall be designed for a shear, V_{ui}, due to the factored loads taken as:

$$V_{ui} = \frac{V_u}{\cos\theta} \tag{6.11.9-1}$$

C6.11.9

For boxes with inclined webs, the web must be designed for the component of the vertical shear in the plane of the web.

Usually, the box webs are detailed with equal height webs. If the deck is superelevated, the box may be rotated to match the deck slope, which is generally preferred to simplify fabrication by maintaining symmetry of the girder sections. The result is that the inclination of one web is increased over what it would have been if the box were not rotated. The computed shear in that web due to vertically applied loads should be adjusted accordingly.

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where:

V_u = vertical shear due to the factored loads on one inclined web (kip)
θ = the angle of inclination of the web plate to the vertical (degrees)

For all single box sections, horizontally-curved sections, and multiple box sections in bridges not satisfying the requirements of Article 6.11.2.3, or with box flanges that are not fully effective according to the provisions of Article 6.11.1.1, V_u shall be taken as the sum of the flexural and St. Venant torsional shears.

For box flanges, b_{fc} or b_{ft}, as applicable, shall be taken as one-half of the effective flange width between webs in checking Eq. 6.10.9.3.2-1, where the effective flange width shall be taken as specified in Article 6.11.1.1, but not to exceed $18t_f$ where t_f is the thickness of the box flange.

Web stiffeners shall satisfy the requirements of Article 6.11.11.1.

For the box sections specifically cited in this Article, including sections in horizontally-curved bridges, St. Venant torsional shear must be considered in the design of the webs. The total shear in one web is greater than in the other web at the same section since the torsional shear is of opposite sign in the two webs. As a matter of practicality, both webs can be designed for the critical shear.

Although shear and longitudinal stresses in the webs due to warping are not zero, these effects are typically quite small and can be ignored in the design of the webs.

For multiple box sections in straight bridges satisfying the requirements of Article 6.11.2.3 for which a live load distribution factor for moment is employed, one-half the distribution factor for moment should be used in the calculation of the live load vertical shear in each web.

6.11.10—Shear Connectors

Except as specified herein, shear connectors shall be designed according to the provisions of Article 6.10.10.

Shear connectors shall be provided in negative flexure regions.

For all single box sections, horizontally-curved sections, and multiple box sections in bridges not satisfying the requirements of Article 6.11.2.3, or with box flanges that are not fully effective according to the provisions of Article 6.11.1.1, shear connectors shall be designed for the sum of the flexural and St. Venant torsional shears. The longitudinal fatigue shear range per unit length, V_{fat}, for one top flange of a tub girder shall be computed for the web subjected to additive flexural and torsional shears. The resulting shear connector pitch shall also be used for the other top flange. The radial fatigue shear range due to curvature, F_{fat1}, given by Eq. 6.10.10.1.2-4 may be ignored in the design of box sections in straight or horizontally-curved spans or segments.

For checking the resulting number of shear connectors to satisfy the strength limit state, the cross-sectional area of the steel box section under consideration and the effective area of the concrete deck associated with that box shall be used in determining P by Eqs. 6.10.10.4.2-2, 6.10.10.4.2-3, 6.10.10.4.2-7, and 6.10.10.4.2-8.

Shear connectors on composite box flanges shall be uniformly distributed across the width of the flange. The maximum transverse spacing, s_t, between shear connectors on composite box flanges shall satisfy the following requirement:

C6.11.10

Shear connectors must be present in regions of negative flexure to resist the torsional shear that exists along the entire span in all types of composite box sections. Also, the prototype and model bridges that were studied in the original development of the live-load distribution provisions for straight box sections had shear connectors throughout the negative flexure region.

Maximum flexural and torsional shears are typically not produced by concurrent loads. However, the interaction between flexure and torsion due to moving loads is too complex to treat practically. Instead, for cases where the torsional shear must be considered, these provisions allow the longitudinal shear range for fatigue to be computed from Eq. 6.10.10.1.2-3 using the sum of the maximum flexural and torsional shears in the web subjected to additive shears. The shear range and the resulting pitch should be computed using one-half the moment of inertia of the composite box section. The top flange over the other web, or the other half of the flange for a closed-box section, should contain an equal number of shear connectors. Because of the inherent conservatism of these requirements, the radial fatigue shear range due to curvature need not be included when computing the horizontal fatigue shear range for box sections in either straight or horizontally-curved spans or segments.

Shear connectors on box flanges are best distributed uniformly across the flange width to ensure composite action of the entire flange with the concrete. The shear connectors are to be spaced transversely to satisfy Eq. 6.11.10-1 in order to help prevent local buckling of the flange plate when subject to compression. The torsional shear or shear range resisted by the concrete

$$\frac{s_t}{t_f}\sqrt{\frac{F_{yf}}{kE}} \leq \lambda_p \quad (6.11.10\text{-}1)$$

where:

k = plate-buckling coefficient for uniform normal stress determined as specified in Article 6.11.8.2

λ_p = limiting slenderness ratio for the box flange determined from Eq. 6.11.8.2.2-9

For composite box flanges at the fatigue limit state, V_{sr} in Eq. 6.10.10.1.2-1 shall be determined as the vector sum of the longitudinal fatigue shear range given by Eq. 6.10.10.1.2-3 and the torsional fatigue shear range in the concrete deck. The number of shear connectors required to satisfy the strength limit state shall be determined according to the provisions of Article 6.10.10.4. In addition, the vector sum of the longitudinal and torsional shears due to the factored loads in the concrete deck per connector shall not exceed Q_r determined from Eq. 6.10.10.4.1-1.

deck can be determined by multiplying the torsional shear or shear range acting on the top of the composite box section by the ratio of the thickness of the transformed concrete deck to the total thickness of the top flange plus the transformed deck. Adequate transverse reinforcement should be provided in the deck to resist this torsional shear.

6.11.11—Stiffeners

6.11.11.1—Web Stiffeners

Transverse intermediate web stiffeners shall be designed according to the provisions of Article 6.10.11.1.

Longitudinal web stiffeners shall be designed according to the provisions of Article 6.10.11.3.

Except as specified herein, bearing stiffeners shall be designed according to the provisions of Article 6.10.11.2. Bearing stiffeners should be attached to diaphragms rather than inclined webs. For bearing stiffeners attached to diaphragms, the provisions of Article 6.10.11.2.4b shall apply to the diaphragm rather than to the web. At expansion bearings, bearing stiffeners and diaphragms should be designed for eccentricity due to thermal movement.

C6.11.11.1

When inclined webs are used, bearing stiffeners should be attached to either an internal or external diaphragm rather than to the webs so that the bearing stiffeners are perpendicular to the sole plate. Thermal movements of the bridge may cause the diaphragm to be eccentric with respect to the bearings. This eccentricity should be recognized in the design of the diaphragm and bearing stiffeners. The effects of the eccentricity are usually most critical when the bearing stiffeners are attached to diaphragms. The effects of the eccentricity can be recognized by designing the bearing stiffener assembly as a beam-column according to the provisions of Articles 6.10.11.2 and 6.9.2.2.

6.11.11.2—Longitudinal Compression Flange Stiffeners

Longitudinal compression flange stiffeners on box flanges shall be equally spaced across the flange width. The specified minimum yield strength of the stiffeners shall not be less than the specified minimum yield strength of the box flange to which they are attached.

The projecting width, b_ℓ, of a flange longitudinal stiffener element shall satisfy:

$$b_\ell \leq 0.48 t_s \sqrt{\frac{E}{F_{yc}}} \quad (6.11.11.2\text{-}1)$$

where:

C6.11.11.2

Eq. 6.11.11.2-1 is intended to prevent local buckling of the projecting elements of the longitudinal flange stiffener. For structural tees, b_ℓ should be taken as one-half the width of the flange.

Eq. 6.11.11.2-2 for the required longitudinal flange stiffener moment of inertia, I_ℓ, is an approximate expression that within its range of applicability yields values of the elastic critical flange buckling stress close to those obtained by use of the exact but cumbersome equations of elastic stability (Timoshenko and Gere, 1961). The required size of the stiffener increases as the panel becomes smaller since the buckling resistance of the panels increases as the panels become smaller.

t_s = thickness of the projecting longitudinal stiffener element (in.)

The moment of inertia, I_ℓ, of each stiffener about an axis parallel to the flange and taken at the base of the stiffener shall satisfy:

$$I_\ell \geq \psi w t_{fc}^3 \quad (6.11.11.2\text{-}2)$$

where:

ψ = $0.125k^3$ for $n = 1$
= $1.120k^3$ for $n = 2$
k = plate-buckling coefficient for uniform normal stress
= $1.0 \leq k \leq 4.0$
n = number of equally spaced longitudinal flange stiffeners
w = larger of the width of the flange between longitudinal flange stiffeners or the distance from a web to the nearest longitudinal flange stiffener (in.)

The actual longitudinal flange stiffener moment of inertia, I_s, used in determining the plate-buckling coefficient for uniform normal stress, k, from either Eq. 6.11.8.2.3-1 or 6.11.8.2.3-2, as applicable, automatically satisfies Eq. 6.11.11.2-2 when that value of k is used since the equations for k are derived directly from Eq. 6.11.11.2-2. Alternatively, a k value can be assumed in lieu of using Eq. 6.11.8.2.3-1 or 6.11.8.2.3-2. k can take any value ranging from 1.0 to 4.0. However, a value of k ranging from 2.0 to 4.0 generally should be assumed. The minimum required moment of inertia for each longitudinal flange stiffener that will provide the assumed value of k can then be determined from Eq. 6.11.11.2-2.

Where required, the number of longitudinal flange stiffeners should preferably not exceed one for maximum economy in boxes of typical proportions. Eq. 6.11.11.2-2 assumes that the box flange plate and the stiffeners are infinitely long and ignores the effect of any transverse bracing or stiffening. Thus, when n exceeds 1, the required moment of inertia from Eq. 6.11.11.2-2 begins to increase dramatically. When n exceeds 2, for which the value of ψ equals $0.07k^3n^4$, the required moment of inertia from Eq. 6.11.11.2-2 becomes nearly impractical.

For rare cases where an exceptionally wide box flange is required and n may need to exceed 2, it is suggested that transverse flange stiffeners be considered to reduce the required size of the longitudinal flange stiffeners to a more practical value. The use of transverse flange stiffeners might also be considered for the case where n equals 2 if a k value greater than about 2.5 is needed and it is desired to reduce the required size of the longitudinal stiffeners over that given by Eq. 6.11.11.2-2. The specified minimum yield strength of the transverse flange stiffeners should not be less than the specified minimum yield strength of the box flange. Individual structural tees can be used as transverse flange stiffeners, and/or a bottom strut, provided within the internal transverse bracing of the box and satisfying the requirements of Article 6.11.5, can serve as a transverse flange stiffener if the strut also satisfies the stiffness requirement given by Eq. C6.11.11.2-4. In either case, the transverse flange stiffeners should be attached to the longitudinal flange stiffeners by bolting. The connection to each longitudinal stiffener should be designed to resist the following vertical force:

$$F_s = \frac{\phi_f F_{ys} S_s}{n b_{fc}} \quad (C6.11.11.2\text{-}1)$$

where:

ϕ_f = resistance factor for flexure specified in Article 6.5.4.2
F_{ys} = specified minimum yield strength of the transverse flange stiffener (ksi)
S_s = section modulus of the transverse flange stiffener (in.^{3})

Individual structural tees serving as transverse flange stiffeners should also be attached to the webs of the box. The connection of transverse flange stiffeners to each web should be designed to resist the following vertical force:

$$F_w = \frac{\phi_f F_{ys} S_s}{2b_{fc}} \tag{C6.11.11.2-2}$$

For the exceptional case where transverse flange stiffeners are deemed necessary, the constant ψ in Eq. 6.11.11.2-2 is to be taken as 8.0 in determining the required moment of inertia of the longitudinal flange stiffeners. n in this case should preferably not exceed five. The longitudinal spacing of the transverse flange stiffeners should not exceed three times the full width of the box flange, b_{fc}, in order for the transverse stiffeners to be considered effective. The plate-buckling coefficient, k, for uniform normal stress to be used in the equations of Article 6.11.8.2.2 in lieu of k determined from Eqs. 6.11.8.2.3-1 or 6.11.8.2.3-2 may then be taken as follows:

$$k = \frac{\left[1+\left(a/b_{fc}\right)^2\right]^2 + 87.3}{(n+1)^2\left(a/b_{fc}\right)^2\left[1+0.1(n+1)\right]} \leq 4.0 \tag{C6.11.11.2-3}$$

where:

a = longitudinal spacing of the transverse flange stiffeners (in.)

Transverse flange stiffeners spaced at a distance less than or equal to $4w$ will provide a k of approximately 4.0 according to Eq. C6.11.11.2-3 when n does not exceed 5. When the k value from Eq. C6.11.11.2-3 is used, the moment of inertia, I_t, of each transverse flange stiffener about an axis through its centroid and parallel to its bottom edge must satisfy:

$$I_t \geq 0.05(n+1)^3 w^3 \left(\frac{f_s}{E}\right)\left(\frac{A_f}{a}\right) \tag{C6.11.11.2-4}$$

where:

A_f = area of the box flange including the longitudinal flange stiffeners (in.^{2})

f_s = largest of the longitudinal flange stresses due to the factored loads in the panels on either side of the transverse flange stiffener under consideration (ksi)

Structural tees are preferred for longitudinal flange stiffeners because a tee provides a high ratio of stiffness to cross-sectional area. Tees also minimize the potential for lateral–torsional buckling of the stiffeners. Using less efficient flat bars as stiffeners is an undesirable alternative. Since the longitudinal flange stiffeners are primary load carrying members, the specified minimum yield strength of the stiffeners must not be less than the specified minimum yield strength of the box flange to which they are attached. Tees may not be available in higher grades of steel. In these cases, tees can be fabricated from plates or bars cut from plate.

Longitudinal flange stiffeners should be continuous through internal diaphragms. Consideration should be given to attaching longitudinal flange stiffeners to the internal diaphragms. Tees may be conveniently attached to the diaphragms with a pair of clip angles.

For the cases specified in Article 6.11.5 where transverse bending stresses due to cross-section distortion are to be considered for fatigue, it may be necessary in certain situations to consider providing bottom transverse bracing members as part of the internal cross-frames to control distortion of the box flange and reduce the transverse bending stress ranges in the flange. Where longitudinal flange stiffeners are used, the transverse member is to be attached to the longitudinal stiffeners by bolting. As discussed previously in this Article, bottom transverse bracing members and/or individual transverse flange stiffeners attached to the longitudinal flange stiffeners may also be necessary in the unusual case of an exceptionally wide box flange. For all other cases, additional transverse stiffening of box flanges is not required. It should be emphasized that bottom transverse bracing members and their connections, where provided, need not satisfy the requirements of Eqs. C6.11.11.2-1, C6.11.11.2-2 and C6.11.11.2-4, unless the k value from Eq. C6.11.11.2-3 is utilized in the design of the box flange.

6.12—MISCELLANEOUS FLEXURAL MEMBERS

6.12.1—General

6.12.1.1—Scope

The provisions of this Article shall apply to:

- noncomposite H-shaped members bent about either axis of the cross section, and noncomposite I-shaped members bent about their weak axis;
- noncomposite single-cell rectangular box-section members with or without longitudinal stiffeners, including square and rectangular HSS, bent about either principal axis;
- noncomposite circular tubes, including round HSS;
- channels, angles, tees, rectangular bars, and solid rounds;
- concrete-encased rolled shapes; and
- circular concrete-filled steel tubes (CFSTs).

C6.12.1.1

This Article covers miscellaneous rolled or built-up noncomposite or composite members subject to flexure, often in combination with axial loads; that is, flexural members not covered by the provisions of Article 6.10 or 6.11. Included are doubly- and singly-symmetric noncomposite single-cell rectangular box-section members with or without longitudinal stiffeners, and with cross-section principal axes parallel to the cross-section component plates. These sections are often utilized in trusses, frames, and arches. In addition, angles, tees, and channels are addressed. These sections are often utilized as cross-frame, diaphragm, and lateral bracing members.

Noncomposite circular tubes or pipes may be designed using the provisions specified herein for round Hollow Structural Sections (HSS) provided that they conform to ASTM A53/A53M, Grade B and the

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appropriate parameters are used in the design. Additional information on connection design for square, rectangular, and round HSS may be found in Chapter K of AISC (2016b). Resistances for fatigue design for square, rectangular and round HSS may be found in Section 9.2.7 of the ANSI/AWS D1.1 *Structural Welding Code* or in Section 11 of the *AASHTO Standard Specifications for Structural Supports for Highway Signs, Luminaries and Traffic Signals*. Where these members are used in fracture-critical applications, refer to Article 8.2.3 of the *Guide Specifications for the Design of Pedestrian Bridges*.

6.12.1.2—Strength Limit State

6.12.1.2.1—Flexure

At the strength limit state, the section shall satisfy:

$$M_u \leq M_r \tag{6.12.1.2.1-1}$$

Except as specified herein, the factored flexural resistance, M_r, shall be taken as:

$$M_r = \phi_f M_n \tag{6.12.1.2.1-2}$$

where:

ϕ_f = resistance factor for flexure specified in Article 6.5.4.2
M_n = nominal flexural resistance of the section determined as specified in Articles 6.12.2.2 and 6.12.2.3 for noncomposite and composite members, respectively (kip-in.)
M_u = factored bending moment about the axis of bending under consideration (kip-in.)

The material-based P-M interaction curve of composite circular CFSTs shall be determined as specified in Article 6.12.2.3.3.

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6.12.1.2.2—Combined Flexure, Axial Load, and Flexural and/or Torsional Shear

The provisions of Article 6.8.2.3 for combined axial tension, flexure, and flexural and/or torsional shear, or the provisions of Article 6.9.2.2 for combined axial compression, flexure, and flexural and/or torsional shear shall apply, as applicable.

For noncomposite circular tubes, including round HSS, that are subject to combined flexure, axial load, and flexural and/or torsional shear, Eq. 6.12.1.2.3a-5 shall be checked in addition to the interaction of the flexural or torsional shear resistances with the member axial and flexural resistances as specified in Articles 6.8.2.3.2 or 6.9.2.2.2, as applicable.

C6.12.1.2.2

The provisions of Article 6.12.1.2.1 assume low or zero levels of axial force in the member and uniaxial flexure. For members that are also subject to a factored concentrically-applied axial force, P_u, in excess of 5 percent of the factored axial resistance of the member, P_r or P_{ry} as applicable, at the strength limit state, and/or if the member is subject to biaxial bending, the member should instead be checked according to the provisions of Article 6.8.2.3 or 6.9.2.2, as applicable. The level of 5 percent is based conservatively on the linear interaction equations given in these articles, which apply in the majority of cases. Below this level, it is reasonable to ignore the effect of the axial force in the design of the member.

6.12.1.2.3—Flexural Shear and/or Torsion

6.12.1.2.3a—General

At the strength limit state, the section shall satisfy:

$$V_u \leq V_r \tag{6.12.1.2.3a-1}$$

The factored flexural shear resistance, V_r, shall be taken as:

$$V_r = \phi_v V_n \tag{6.12.1.2.3a-2}$$

At the strength limit state, noncomposite circular tubes, including round HSS, subject to torsion only shall satisfy:

$$T_u \leq T_r \tag{6.12.1.2.3a-3}$$

The factored torsional resistance, T_r, of noncomposite circular tubes, including round HSS, shall be taken as:

$$T_r = \phi_T T_n \tag{6.12.1.2.3a-4}$$

where:

ϕ_T = resistance factor for torsion specified in Article 6.5.4.2

ϕ_v = resistance factor for shear specified in Article 6.5.4.2

T_n = nominal torsional resistance for noncomposite circular tubes and round HSS (kip-in.) calculated as specified in Article 6.12.1.2.3b

T_u = factored torque at the section under consideration (kip-in.)

V_n = nominal flexural shear resistance (kip) calculated as follows:

C6.12.1.2.3a

These provisions are used to check the factored flexural shear resistance of the web elements of noncomposite rectangular box-section members, including square and rectangular HSS, webs of composite members and other noncomposite members covered by the provisions of Article 6.12, noncomposite circular tubes, including round HSS, and concrete-filled tubes, including CFSTs, that are subject to flexural shear only.

These provisions are also used to check the factored torsional resistance of noncomposite circular tubes, including round HSS, that are subject to torsion only, or the resistance of such members that are subject to combined flexural shear and torsion.

- For each web element of a noncomposite rectangular box-section member, including square and rectangular HSS, the provisions of Articles 6.10.9 and 6.12.1.2.4 shall apply, as applicable.

  - In checking Eq. 6.10.9.3.2-1, b_{fc} shall be taken as one-half the effective width between webs determined as specified in Article 6.9.4.2.2b for longitudinally unstiffened compression flanges, and as one-half the effective area between webs determined as specified in Article 6.12.2.2.2d, including the longitudinal stiffeners, divided by the thickness of the plate for longitudinally stiffened compression flanges. b_{ft} shall be taken as one-half the gross area of the tension flange, including any longitudinal stiffeners, divided by the thickness of the plate. Shear lag effects shall be considered, as applicable, in the determination of b_{fc} and b_{ft}, as specified in Article 6.12.2.2.2g.

- For noncomposite circular tubes, including round HSS, the provisions of Article 6.12.1.2.3b shall apply.

- For webs of all other noncomposite members, the provisions of Article 6.10.9 shall apply, as applicable.

- For webs of composite members and for concrete-filled tubes, including composite circular CFSTs, the provisions of Article 6.12.3 shall apply, as applicable.

V_u = for noncomposite circular tubes, including round HSS, factored flexural shear at the section under consideration. For noncomposite rectangular box-section members subject to torsion, including square and rectangular HSS, the factored shear in each web element shall be taken as the sum of the flexural and St. Venant torsional shears (kip)

For noncomposite circular tubes, including round HSS, subject to both flexural shear and torsion, the following relationship shall be satisfied at the strength limit state:

$$\frac{V_u}{V_r} + \frac{T_u}{T_r} \leq 1.0 \qquad (6.12.1.2.3a\text{-}5)$$

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For stems of tees and for elements of noncomposite I- and H-shapes loaded about their weak axis, the shear buckling coefficient, k, shall be taken as 1.2.

6.12.1.2.3b—Circular Tubes and Round HSS

For noncomposite circular tubes, including round HSS, the nominal flexural shear resistance, V_n, shall be taken as:

$$V_n = 0.5 F_{cr} A_g \tag{6.12.1.2.3b-1}$$

in which:

F_{cr} = flexural shear buckling resistance (ksi) taken as the larger of either:

$$F_{cr1} = \frac{1.60E}{\sqrt{\dfrac{L_v}{D}}\left(\dfrac{D}{t}\right)^{\frac{5}{4}}} \leq 0.58F_y \tag{6.12.1.2.3b-2}$$

or:

$$F_{cr2} = \frac{0.78E}{\left(\dfrac{D}{t}\right)^{\frac{3}{2}}} \leq 0.58F_y \tag{6.12.1.2.3b-3}$$

where:

A_g = gross area of the section based on the design wall thickness (in.)
D = outside diameter of the tube (in.)
L_v = distance between points of maximum and zero shear, or the full length of the member if the shear does not go to zero within the member length (in.)
t = wall thickness of the tube (in.). For round HSS, the provisions of Article 6.12.1.2.4 shall apply.

The nominal torsional resistance, T_n, shall be taken as:

$$T_n = F_{cv} C \tag{6.12.1.2.3b-4}$$

in which:

C = torsional constant (in.^{3})

$$= \frac{\pi (D-t)^2 t}{2} \tag{6.12.1.2.3b-5}$$

F_{cv} = torsional shear buckling resistance (ksi) taken as the larger of either:

C6.12.1.2.3b

The provisions for noncomposite circular tubes, including round Hollow Structural Sections (HSS), subject to flexural shear are based on the provisions for local buckling of cylinders due to torsion. However, since torsion is generally constant along the member length and flexural shear typically has a gradient, the critical buckling stress for flexural shear is taken as 1.3 times the critical stress for torsion (Brockenbrough and Johnston 1981; Ziemian, 2010). The torsional shear equations apply over the full length of the member, but for flexural shear, it is reasonable to use the length between points of maximum and zero shear. The nominal flexural shear resistance is computed assuming that the shear stress at the neutral axis is at F_{cr}. The resulting stress at the neutral axis is $V/\pi Rt$, in which the denominator is half the area of the circular tube.

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$$F_{cv1} = \frac{1.23E}{\sqrt{\frac{L}{D}\left(\frac{D}{t}\right)^{\frac{5}{4}}}} \leq 0.58F_y \qquad (6.12.1.2.3b\text{-}6)$$

or:

$$F_{cv2} = \frac{0.60E}{\left(\frac{D}{t}\right)^{\frac{3}{2}}} \leq 0.58F_y \qquad (6.12.1.2.3b\text{-}7)$$

where:

L = length of the member (in.)

6.12.1.2.4—Special Provisions for HSS Members

For square and rectangular HSS members, the web depth, D, shall be taken as the clear distance between flanges less the inside corner radius on each side, and the area of both webs shall be considered effective in resisting the shear.

For square and rectangular HSS members, the inside width of each flange, b_{fi}, shall be taken as the clear width of the flange between the webs less the inside corner radius on each side.

For square, rectangular, and round HSS members, the design wall thickness, t, shall be taken as the nominal wall thickness for HSS produced according to ASTM A1085/A1085M. For HSS produced according to other standards specified in Article 6.4.1, t shall be taken as 0.93 times the nominal wall thickness.

C6.12.1.2.4

These provisions are used to define the web depth, D, and inside flange width, b_{fi}, for the design of square and rectangular HSS members, and the design wall thickness, t, for the design of square, rectangular, and round HSS members. These provisions are referred to in Articles 6.9.4.2, 6.12.1.2.3, 6.12.2.2.2, and 6.12.2.2.3.

For square and rectangular HSS members, if the inside corner radius is not known, use the outside dimension minus three times the appropriate design wall thickness specified herein.

6.12.2—Nominal Flexural Resistance

6.12.2.1—General

Except as specified herein, provisions for lateral–torsional buckling need not be applied to composite members, noncomposite I- and H-shaped members bent about their weak axis, and noncomposite circular tubes.

6.12.2.2—Noncomposite Members

6.12.2.2.1—I- and H-Shaped Members

The provisions of this Article apply to I- and H-shaped members and members consisting of two channel flanges connected by a web plate.

The provisions of Article 6.10 shall apply to flexure about an axis perpendicular to the web.

The nominal flexural resistance for flexure about the weak axis shall be taken as:

- If $\lambda_f \leq \lambda_{pf}$, then:

C6.12.2.2.1

Eqs. 6.12.2.2.1-1 and 6.12.2.2.1-2 are taken from Appendix F of AISC (1999), except that the flange slenderness λ_{rf} corresponding to the transition from inelastic to elastic flange local buckling is consistently set based on the yield moment in weak-axis bending $F_{yf}S_y$. AISC (1999) uses $F_{yf}S_y$ as the moment corresponding to the inelastic-to-elastic flange local buckling transition, but then specifies λ_{rf} based on a smaller moment level. The approach adopted in these provisions is interpreted as a corrected form of the AISC (1999) equations and is

$$M_n = M_p \qquad (6.12.2.2.1\text{-}1)$$

- If $\lambda_{pf} < \lambda_f \leq \lambda_{rf}$, then:

$$M_n = \left[1 - \left(1 - \frac{S_y}{Z_y}\right)\left(\frac{\lambda_f - \lambda_{pf}}{0.45\sqrt{\dfrac{E}{F_{yf}}}}\right)\right] F_{yf} Z_y \qquad (6.12.2.2.1\text{-}2)$$

in which:

λ_f = slenderness ratio for the flange

$$= \frac{b_f}{2t_f} \qquad (6.12.2.2.1\text{-}3)$$

λ_{pf} = limiting slenderness ratio for a compact flange

$$= 0.38\sqrt{\frac{E}{F_{yf}}} \qquad (6.12.2.2.1\text{-}4)$$

λ_{rf} = limiting slenderness ratio for a noncompact flange

$$= 0.83\sqrt{\frac{E}{F_{yf}}} \qquad (6.12.2.2.1\text{-}5)$$

where:

F_{yf} = specified minimum yield strength of the lower-strength flange (ksi)
M_p = plastic moment about the weak axis (kip-in.)
S_y = elastic section modulus about the weak axis (in.^{3})
Z_y = plastic section modulus about the weak axis (in.^{3})

conservative relative to the AISC (1999) equations as printed. The yield moment $F_{yf}S_y$ may be taken conservatively as the moment at the inelastic-to-elastic flange local buckling transition because of the beneficial effects of the stress gradient in the flange associated with weak-axis bending.

For H-shaped members $M_p = 1.5F_yS$, where S is the elastic section modulus about this axis.

6.12.2.2.2—Rectangular Box-Section Members

6.12.2.2.2a—General

For doubly- and singly-symmetric single-cell rectangular box-section members with or without longitudinal stiffeners, bent about either principal axis, the nominal flexural resistance shall be based on the combined influence of general yielding, compression flange local buckling, and/or lateral–torsional buckling. The nominal flexural resistance corresponding to these limit states shall be determined as specified in Article 6.12.2.2.2e. In addition, for all box-section members, the provisions of Article 6.12.2.2.2f shall be considered at the fatigue and service limit states, and for constructibility.

For box sections with different thickness webs, the smaller web thickness shall be used in the calculation of all section properties.

C6.12.2.2.2a

Article 6.12.2.2.2 addresses the flexural design of homogeneous and hybrid doubly- and singly-symmetric single-cell rectangular box-section members with or without longitudinal stiffeners, including square and rectangular HSS, bent about either principal axis in which the cross-section principal axes are parallel to the cross-section component plates.

The concepts and procedures specified in Article 6.12.2.2.2 for rectangular box-section members are also largely applicable to other noncomposite box cross-section profiles, including trapezoidal geometries, which may be used for certain flexural members, parallelogram geometries, which can be found in tie members of basket-handle arches, and octagonal or other geometries, which

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Sections designed according to these provisions shall satisfy the cross-section proportion limits specified in Article 6.12.2.2.2b.

For sections subjected to flexure only, if there are holes in the tension flange at the section under consideration, the tension flange shall satisfy the provisions of Article 6.10.1.8, with f_t taken as the factored stress on the gross area of the tension flange. For sections with holes in a tension flange subjected to flexure combined with axial tension or compression, the provisions of Article 6.8.2.3.3 shall apply. Access holes within a flange of a box-section member shall satisfy the provisions of Article 6.11.1.4.

For noncomposite box-section members subject to torsion, transverse plate bending stresses due to cross-section distortion should be considered for fatigue and at the strength limit state. The factored transverse plate bending stresses should not exceed 20.0 ksi at the strength limit state. Longitudinal warping stresses due to cross-section distortion should be considered for fatigue, but may be ignored at the strength limit state. Transverse plate bending and longitudinal warping stresses due to cross-section distortion shall be determined by rational structural analysis. Transverse stiffeners attached to the webs or flanges should be considered effective with the web or flange in resisting transverse plate bending.

may be used in large steel box-section towers. White et al. (2019b) discuss the extension of these provisions to the design of members with general box cross-section profiles.

AISC Design Guide 25 (White and Jeong, 2019) provides broad guidelines for the design of general nonprismatic I-section members. The concepts employed in these provisions and guidelines also may be employed to extend the provisions of Article 6.12.2.2.2 for calculation of the design resistance of general nonprismatic box-section members.

Any box-section member component plate subjected to stress due to bending about a principal axis of the box parallel to the plate is considered as a flange for bending about that axis. Any component plate subjected to flexural shear orthogonal to the axis of bending, and/or nominally linearly varying normal stresses due to bending about an axis normal to the face of the plate, is considered as a web. Generally, a given box-section component plate may serve as a flange for bending about one principal axis and as a web for bending about the other principal axis.

Where an access hole or perforation is provided in a flange, the hole should be deducted in determining the gross section for checking the requirements of Article 6.10.1.8 or 6.8.2.3.3.

Article 6.11.1.4 provides broad guidelines for the design of access holes, ventilation, and drainage in composite box-section flexural members. These provisions also are generally applicable for noncomposite rectangular box-section members. Additional considerations are necessary for the assessment of the resistance to combined axial tension or compression, flexure and/or torsion at such cross sections.

Straddle beams and integral cap beams should be deep and wide enough and sufficiently uncluttered to allow for convenient maintenance and inspection access. Access holes should be provided at the ends, or if necessary, on the top of these members.

In certain types of box-section members, access holes or other large holes may be placed in one of the webs. AISC Design Guide 2 (Darwin, 1990) provides broad design guidelines that may be adapted for the design of box-section members with large web holes.

Transverse plate bending stresses in flanges and webs of box-section members subject to torsion occur due to changes in direction of the shear-flow vector and are associated with distortion of the box cross-section profile. Most rectangular box sections are capable of resisting torsion with limited distortion of the cross section. Distortion is generally limited by providing sufficient internal bracing in accordance with the provisions of Article 6.7.4.4; as such, torsion is mainly resisted by St. Venant torsional shear flow. In addition, the warping constant is approximately equal to zero for rectangular box-sections. Thus, warping shear and normal stresses due to warping torsion are typically quite small and are usually neglected. For typical welded noncomposite box-

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Internal diaphragms and cross-frames for noncomposite box-section members shall satisfy the provisions of Article 6.7.4.4.

section members subject to large torques, further investigation of the cross-section distorsional stresses may be warranted, particularly for fatigue investigations. Article C6.11.1.1 discusses the use of the beam-on-elastic foundation or BEF analogy (Wright and Abdel-Samad, 1968) for the determination of cross-section distortional stresses and stress ranges.

Bearing stiffeners shall be designed according to the provisions of Article 6.10.11.2. Bearing stiffeners should be attached to diaphragms. At expansion bearings, bearing stiffeners and diaphragms should be designed for eccentricity due to thermal movement.

The provisions of Article 6.12.2.2.2g shall be considered to account for shear lag effects, as applicable.

Thermal movements of the member may cause the diaphragm to be eccentric with respect to the bearing. This eccentricity should be recognized in the design of the diaphragm and bearing stiffeners. The effects of the eccentricity can be recognized by designing the bearing stiffener assembly as a beam-column according to the provisions of Articles 6.10.11.2 and 6.9.2.2.

Flowcharts illustrating the application of the provisions of Article 6.12.2.2.2 for determining the flexural resistance of rectangular noncomposite box-section members, with or without longitudinally stiffened plates, are provided in Article C6.5.2 in Appendix C6.

6.12.2.2.2b—Cross-Section Proportion Limits

Webs without longitudinal stiffeners shall be proportioned such that:

$$\frac{D}{t_w} \leq 150 \qquad (6.12.2.2.2b\text{-}1)$$

where:

D = for welded box sections, the clear distance between the flanges. For HSS, the provisions of Article 6.12.1.2.4 shall apply (in.)

t_w = thickness of the webs (in.). For box sections with different thickness webs, the smaller web thickness. For HSS, the provisions of Article 6.12.1.2.4 shall apply.

C6.12.2.2.2b

Article 6.12.2.2.2b specifies a number of broad not-to-exceed or not-to-be-smaller-than limits on the proportions of box-section members. Various specific design criteria may require dimensions that are more restrictive than these maximum or minimum limits.

Eq. 6.12.2.2.2b-1 is a practical upper limit on the slenderness of webs without longitudinal stiffeners expressed in terms of the web depth, D. By limiting the slenderness of transversely-stiffened webs to this value, the web shear resistance may be increased by providing transverse stiffeners up to a maximum spacing of $3D$. In addition, this is a conservative limit on the slenderness of longitudinally unstiffened webs to avoid potential distortion-induced fatigue considerations.

For mechanically fastened built-up box-section members, b_{fi} and D in these provisions should be taken as the distance between the outer lines of fasteners connecting the component plate elements. It is recommended that the pitch of the fasteners in the compression and tension flanges satisfy the maximum pitch requirements for stitch bolts in compression members and tension members, respectively, specified in Article 6.13.2.6.3.

Webs with longitudinal stiffeners shall be proportioned such that:

$$\frac{D}{t_w} \leq 300 \qquad (6.12.2.2.2b\text{-}2)$$

Web longitudinal stiffeners shall satisfy the provisions of Article 6.10.11.3.

The upper limit given by Eq. 6.12.2.2.2b-2 parallels the upper limit for longitudinally stiffened webs of I-girders specified in Article 6.10.2.1.2.

Longitudinally unstiffened compression and tension flanges should be proportioned such that:

Plates subjected to significant uniform compression stresses at the fatigue and/or service limit states, or during

$$b_{fi}/t_f \leq 90 \qquad (6.12.2.2.2b\text{-}3)$$

where:

b_{fi} = inside width of the box section flanges (in.); for welded box sections, the clear width of the flange under consideration between the webs. For HSS, the provisions of Article 6.12.1.2.4 shall apply.

t_f = thickness of the flange under consideration (in.). For HSS, the provisions of Article 6.12.1.2.4 shall apply.

The thickness of the compression and tension flanges corresponding to the box section principal axis subjected to the larger bending moment should not be less than the thickness of the webs. The thickness of compression and tension flanges shall not be less than 0.5 in., unless otherwise specified by the Owner.

Sections with a longitudinally unstiffened compression flange shall be classified for flexural design as specified in Article 6.12.2.2.2c.

Compression flanges exceeding the limit given by Eq. 6.12.2.2.2b-3 shall include longitudinal stiffeners. Tension flanges with b_{fi}/t_f exceeding 130 shall include longitudinal stiffeners. Longitudinally stiffened flanges should be proportioned such that:

$$w/t_f \leq 90 \qquad (6.12.2.2.2b\text{-}4)$$

where:

w = widths of the flange plate between the centerlines of the individual longitudinal stiffeners and/or between the centerline of a longitudinal stiffener and the inside of the laterally-restrained longitudinal edge of a longitudinally stiffened plate element (in.)

construction, will tend to be limited to b_{fi}/t_{fe} significantly less than 90 or w/t_f less than 90, as applicable, and D/t_w less than 150 or 300, as applicable, by the provisions of Article 6.12.2.2.2f.

The limits of $b_{fi}/t_f \leq 90$, $w/t_f \leq 90$ and $t_f \geq 0.5$ in. are recommended to limit potential local deformation or distortion of box section flanges during fabrication, transportation, erection, and service conditions. Flanges violating these limits may have out-of-plane plate deflections approaching a significant fraction of $b_{fi}/300$ or $w/300$ under their self-weight plus a transverse concentrated load of 300 lb, which is considered by ASCE/SEI 7-16 (ASCE, 2016) as a visible deflection limit. These limits also help alleviate significant buckling distortion due to welding residual stresses resulting in oil canning and waviness of the flange, and the amplification of these distortions by small unintended axial compressive and/or shear stresses under service conditions, such as shifting of true inflection point locations from nominal positions calculated in the design (White et al., 2019a).

For box sections with b_{fi}/t_f or w/t_f of the flange plates greater than 100, some noticeable buckling distortion of the flange plate may occur during fabrication due to placement of typical minimal welding of the flange plate to the webs and/or welding of any stiffeners to the flange plate. In addition, the nominal resistance of compression flanges is relatively small as b_{fi}/t_f or w/t_f exceeds 100. Box flanges with b_{fi}/t_f or w/t_f values larger than about 130 will have difficulty maintaining the $b_{fi}/300$ or $w/300$ out-of-plane deflection under self-weight, or under self-weight with a small concentrated transverse load; therefore, plate out-of-plane sagging due to these nominal loads may be noticeable.

The flange thicknesses corresponding to the box section principal axis direction with the largest bending moment should not be smaller than the corresponding web thicknesses. As such, the plate bending stresses due to distortion of the box section under torsional loads will tend to be larger in the webs than in the flange.

For welded box sections, a minimum thickness of 0.75 in. is recommended for component plates subjected to significant stress due to bending about a cross-section axis parallel to the plates. This suggested minimum thickness is intended to ensure robustness and resiliency of the member response, to facilitate handling, and to minimize distortion and possible cupping of the plates during welding. An absolute minimum thickness of 0.5 in. is specified, unless otherwise permitted by the Owner, to avoid increasing sensitivity to welding distortions and deflections under self-weight and small concentrated applied loads. Smaller thicknesses are common, however, for box-section members employed in long-span bridge construction, where the expense associated with handling and control of distortion in thin stiffened plates is justified by the savings in weight.

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Sections with a longitudinally stiffened compression flange shall be classified for flexural design as specified in Article 6.12.2.2.2d. Flange longitudinal stiffeners shall satisfy the provisions of Article E6.1.4. Transverse stiffeners, when utilized to strengthen or stiffen a longitudinally stiffened flange, shall satisfy the requirements of Article E6.1.5.

In cases where a longitudinally stiffened flange plate acts as a web in one of the directions of bending, the requirement to include transverse stiffeners at a maximum spacing of 2*D* specified in Article 6.10.11.1.1 may be waived unless a transverse stiffener spacing less than 2*D* is employed in the calculation of the shear buckling resistance of the plate, and the requirements of Article 6.10.11.3 for proportioning the longitudinal stiffeners may be waived unless the longitudinal stiffeners are considered in the calculation of R_b in Article 6.10.1.10.2. The longitudinal stiffeners may be included in the calculation of gross cross-section properties regardless of the satisfaction of the maximum spacing requirement of 2*D*.

In cases where a longitudinally stiffened flange plate acts as a web in one direction of bending, and includes transverse stiffeners, the requirements of Article 6.10.11.1 may be waived provided that the transverse stiffeners are not considered in the calculation of the web flexural shear resistance in Article 6.10.9.

The outside width of the box section shall satisfy:

$$b_{fo} \geq D/6 \qquad (6.12.2.2.2b\text{-}5)$$

where:

b_{fo} = outside width of the box section taken as the distance from the outside to the outside of the box-section webs (in.)

D/6 is a minimum limit for the outside width of box-section members. There are no HSS sections with widths smaller than this limit, and smaller widths in welded box sections would necessitate the potential consideration of elastic lateral–torsional buckling in the provisions contained herein.

Flange extensions on compression flanges of box sections shall be proportioned such that:

$$b/t_f \leq 0.38\sqrt{\frac{E}{F_y}} \qquad (6.12.2.2.2b\text{-}6)$$

where:

b = clear projecting width of the compression flange under consideration measured from the outside surface of the web (in.)

t_f = thickness of the flange under consideration (in.)

Eq. 6.12.2.2.2b-6 limits the width-to-thickness ratio of compression flange extensions in box sections such that these components are not subject to any strength reduction associated with local buckling under flexural and/or axial compression. As such, the full gross width of the extensions, measured from the outside surface of the box section webs, may be employed in the calculation of the effective and gross box-section properties.

6.12.2.2.2c—Classification of Sections with a Longitudinally Unstiffened Compression Flange

Sections with a longitudinally unstiffened compression flange that satisfy the following requirement shall qualify as compact web sections:

$$\lambda_w \leq \lambda_{pw} \qquad (6.12.2.2.2c\text{-}1)$$

in which:

C6.12.2.2.2c

Articles 6.12.2.2.2c and 6.12.2.2.2d provide the classification of a comprehensive range of rectangular box-section profiles based on the web and compression–flange slenderness, and define the cross section based parameters R_b, R_{pc}, R_f and M_{yce} employed for calculation of box-section member resistances in Article 6.12.2.2.2e. Depending on the classification of the box section under consideration, the parameters R_b, R_{pc}, and/or R_f may be taken simply equal to 1.0, and

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λ_w = web slenderness

$$= \frac{2D_{ce}}{t_w} \quad (6.12.2.2.2c\text{-}2)$$

λ_{pw} = limiting slenderness ratio for a compact web

$$= 3.1\left(\frac{D_{ce}}{D_{cpe}}\right)\sqrt{\frac{E}{F_{yc}}} \quad (6.12.2.2.2c\text{-}3)$$

For a compact web section,

$$R_{pc} = \frac{M_{pe}}{M_{yce}} \quad (6.12.2.2.2c\text{-}4)$$

where:

D_{ce} = depth of the web in compression in the elastic range, considering early nominal yielding in tension when $S_{xte} < S_{xce}$, and using the effective box cross section based on the effective width of the compression flange, b_e, calculated as specified in Article 6.9.4.2.2b with F_{cr} taken equal to F_{yc}; for welded sections, depth from the inside of the compression–flange; for HSS the provisions of Article 6.12.1.2.4 shall apply (in.)

D_{cpe} = depth of the web in compression at the plastic moment determined using the effective box cross section based on the effective width of the compression flange; for welded sections, depth from the inside of the compression flange; for HSS, the provisions of Article 6.12.1.2.4 shall apply (in.)

F_{yc} = specified minimum yield strength of the compression flange (ksi)

I_{xe} = effective moment of inertia of the cross section about the axis of bending determined using the effective width for the compression flange, b_e, calculated as specified in Article 6.9.4.2.2b with F_{cr} taken equal to F_{yc}, and the gross area for the tension flange (in.^{4}). Except as specified in Article 6.12.2.2.2d, any web and/or tension flange longitudinal stiffeners should be included in the calculation of I_{xe}

M_{pe} = plastic moment using the effective box cross section based on the effective width of the compression flange (kip-in.)

M_{yce} = yield moment with respect to the compression flange taken as $F_{yc}S_{xce}$ (kip-in.) for sections in which $S_{xte} \geq S_{xce}$, and calculated as the moment at nominal first yielding of the compression flange, considering early nominal yielding in tension, for sections in which $S_{xte} < S_{xce}$

R_{pc} = web plastification factor for the compression flange

M_{yce} may be taken equal to the fundamental yield moment of the gross cross section, to the compression–flange, $M_{yc} = S_{xc}F_y$. Typical box sections only require a limited number of the equations provided in Articles 6.12.2.2.2c and 6.12.2.2.2d.

Eq. 6.12.2.2.2c-1 ensures that the section is able to develop the full plastic moment resistance on the effective cross section, M_{pe}, provided that lateral torsional bracing requirements are satisfied. Such cross sections are referred to as compact web sections. The limiting web slenderness ratio, λ_{pw}, is somewhat larger than the value specified for doubly symmetric noncomposite box sections in AISC (2016b), and is slightly larger than the Class 2 limit for noncomposite box sections specified in CEN (2005), which is intended to ensure that the plastic moment of the cross section can be developed, contingent on the satisfaction of other plate slenderness and member unbraced length requirements necessary to develop the plastic moment. For a compact web section, the web plastification factor given by Eq. 6.12.2.2.2c-4 is the shape factor of the effective cross section.

The coefficient 3.1 in Eq. 6.12.2.2.2c-3 can be explained in part by considering the comparable compact web limit defined for general I-section members, Eq. A6.2.1-2, which is illustrated in Figure CA6.2.1-1. The form of Eq. A6.2.1-2 corresponding to $2D_c/t_w$, Eq. A6.2.2-6, appears in Table B4.1 of AISC (2016b). For I-sections with M_p/R_hM_y = 1.12, Eq. A6.2.1-2 gives a coefficient of 3.77, which is essentially the coefficient corresponding to the compact web limit specified in Eq. 6.10.6.2.2-1. This coefficient is a reasonable median value for rolled I-section members and welded doubly symmetric I-section members with comparable proportions. However, for I-sections with M_p/R_hM_y = 1.21, Eq. A6.2.1-2 gives a coefficient of 3.1. The ratio M_p/R_hM_y = 1.21 is more representative of box-section members with webs proportioned near the compact limit. In addition, the coefficient 3.1 in Eq. 6.12.2.2.2c-3 reflects the restraint conditions provided to the webs by the flanges in box-section members. The use of the ratio (D_{ce}/D_{cpe}) in Eq. 6.12.2.2.2c-3 is similar to the use of (D_c/D_{cp}) in Eq. A6.2.2-6. This ratio converts the compact web limit to a value that corresponds to a consistent definition of the web slenderness of $2D_{ce}/t_w$ which is employed with the web compact and noncompact limits in defining the web plastification factor, R_{pc}, in Eq. 6.12.2.2.2c-7.

Eq. 6.12.2.2.2c-1 and the subsequent equations specified herein are expressed generally in terms of the effective cross section, considering the post-buckling response of a slender compression flange. Box-section compression flanges, which are supported by webs along two longitudinal edges, have substantial post-buckling resistance. However, unlike I-section members, where the flange local buckling resistance is limited to incipient theoretical flange local buckling, an interaction exists between the post-buckling response of the compression

S_{xce} = effective elastic section modulus about the axis of bending to the compression flange determined using the effective width for the compression flange, b_e, calculated as specified in Article 6.9.4.2.2b with F_{cr} taken equal to F_{yc}, and the gross area for the tension flange (in.^{3}). Except as specified in Article 6.12.2.2.2d, any web and/or tension flange longitudinal stiffeners should be included in the calculation of S_{xce}. The effective elastic section modulus, S_{xce} shall be determined by dividing the effective moment of inertia, I_{xe}, by the distance to the corresponding extreme fibers of the cross section.

S_{xte} = effective elastic section modulus about the axis of bending to the tension flange determined using the effective width for the compression flange, b_e, calculated as specified in Article 6.9.4.2.2b with F_{cr} taken equal to F_{yc}, and the gross area for the tension flange (in.^{3}). Except as specified in Article 6.12.2.2.2d, any web and/or tension flange longitudinal stiffeners should be included in the calculation of S_{xte}. The effective elastic section modulus, S_{xte}, shall be determined by dividing the effective moment of inertia, I_{xe}, by the distance to the corresponding extreme fibers of the cross section.

t_w = web thickness (in.); for box sections with different thickness webs, the smaller web thickness; for HSS, the provisions of Article 6.12.1.2.4 shall apply.

For compact web sections, the web load-shedding factor, R_b, shall be taken as 1.0.

flange and the other strength limit states of the member. This behavior is captured by the use of the effective cross section, based on the effective width of the compression flange, in the calculation of D_{ce}, D_{cpe}, M_{yce}, and M_{pe}.

The calculations associated with Eqs. 6.12.2.2.2c-1 through 6.12.2.2.2c-4 and the subsequent calculations in Article 6.12.2.2.2c may be simplified as follows:

- For box-sections in which the compression flange is compact, the effective width of the compression flange is equal to its gross width, and thus the calculations in Article 6.12.2.2.2c are all based on the gross box cross section. Otherwise, the calculations in Article 6.12.2.2.2c are based on the effective section, using the effective width of the compression flange, b_e, calculated as specified in Article 6.9.4.2.2b with F_{cr} taken equal to F_{yc}.
- For HSS with slender flange elements, the calculation of the effective section modulus, S_{xce}, the corresponding yield moment, M_{yce}, the plastic depth of web in compression, D_{cpe}, and the effective plastic moment, M_{pe}, may be determined in a manner that parallels the recommended calculation of the effective area for axial compression for these types of sections for Eq. 6.9.4.2.2a-4. For example, the effective section modulus may be calculated as:

$$S_{xce} = S_x - \frac{(b-b_e)t_f^3/12 + (b-b_e)t_f c_c^2}{c} \tag{C6.12.2.2.2c-1}$$

where:

c_c = distance from the neutral axis of the effective cross section to the mid-thickness of the rectangular portion of the compression–flange area, considering the effective width of the slender flange and using the gross cross section for the web (in.)

c = distance from the neutral axis of the effective cross section to the extreme fiber of the compression flange (in.)

This avoids complications in the consideration of the HSS corners. Other variables in Eq. C6.12.2.2.2c-1 are defined in Articles 6.12.2.2.2c and 6.9.4.2.2a.

- For doubly-symmetric box-section members, a conservative estimate of the nominal flexural resistance may be obtained by using the effective width for both the compression and the tension flange in the calculation of the effective section properties. This maintains symmetry of the cross section and thus simplifies the calculations. For singly-symmetric box-section members, the shift in

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the neutral axis should be accounted for in the calculation of the effective section properties.

The limit state of tension flange yielding does not apply to box-section members with a longitudinally unstiffened compression flange. Instead, in cases where $S_{xte} < S_{xce}$, early nominal yielding in tension is to be considered in the calculation of D_{ce} and M_{yce}. Figure C6.12.2.2.2c-1 illustrates the stress state corresponding to these variables for a hybrid cross section with a compression–flange having an effective width less than its actual width, indicated by the black shaded area in the figure. For a homogeneous cross section, F_{yf} is equal to F_{yw}, giving the stress distribution represented by the light grey lines if the flexural strain distribution were the same as in the hybrid case. For a homogeneous cross section, the following closed-form expression for the depth of the web in compression is obtained based on a rigorous strain–compatibility analysis:

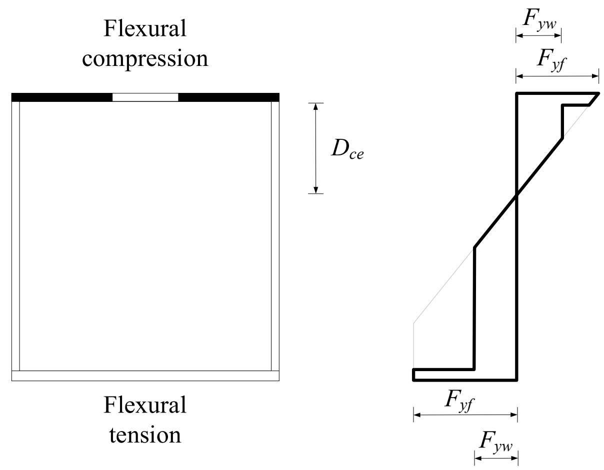

Figure C6.12.2.2.2c-1–Illustration of the Stress State Corresponding to D_{ce} and M_{yce} in a Box Section where $S_{xte} < S_{xce}$, Considering the Early Nominal Yielding that Occurs First at the Tension Flange

$$D_{ce} = \frac{\Delta A + \sqrt{\Delta A^2 + 2A_{fce}A_{wfc} - A_{wfc}^2}}{8t_w} - t_{fc} \quad \text{(C6.12.2.2.2c-2)}$$

in which:

$$\Delta A = A_{ft} + A_w + A_{wfc} - A_{fce} \quad \text{(C6.12.2.2.2c-3)}$$

$$A_{fce} = b_{fce}t_{fc} \quad \text{(C6.12.2.2.2c-4)}$$

$$A_{ft} = b_{ft}t_{ft} \quad \text{(C6.12.2.2.2c-5)}$$

$$A_w = 2Dt_w \quad \text{(C6.12.2.2.2c-6)}$$

$$A_{wfc} = 4t_{fc}t_w \tag{C6.12.2.2.2c-7}$$

where:

b_{fce} = effective width of the compression flange, including corners and flange extensions (in.)
b_{ft} = width of the tension flange, including the corners and flange extensions (in.)
D = depth of the web (in.)
t_{fc} = thickness of the compression flange (in.)
t_{ft} = thickness of the tension flange (in.)
t_w = web thickness (in.)

Furthermore, for $(D/2 - t_{fc}/2) > D_{ce} > 0$, the yield moment to the compression flange is given by

$$M_{yce} = F_{yc}\left[\frac{A_{fce}}{d_{ce}}\left(\frac{D_{ce}t_{fc}}{2} + \frac{t_{fc}^2}{3}\right) + A_{ft}\left(D + \frac{t_{ft}}{2}\right) + t_w\left(D^2 - t_{fc}^2 - \frac{7d_{ce}^2}{3} + 3d_{ce}t_{fc} - \frac{D_{ce}^3}{3d_{ce}}\right)\right] \tag{C6.12.2.2.2c-8}$$

in which:

$$d_{ce} = D_{ce} + t_{fc} \tag{C6.12.2.2.2c-9}$$

where:

F_{yc} = specified minimum yield strength of the compression flange (ksi)
d_{ce} = distance from the extreme fiber of the compression flange to the neutral axis of the effective section (in.)

For cases in which $D_{ce} \leq 0$ from Eq. C6.12.2.2.2c-2, the neutral axis of the section is located within the compression–flange; therefore, the depth of the web in compression, D_{ce}, is to be taken as zero and the web may be taken to be compact. In these cases, M_{yce} need not be calculated and the moment $R_{pc}M_{yce}$ may be taken equal to M_{pe}. For cases in which $D_{ce} > (D/2 - t_{fc}/2)$, the tension flange is not fully yielded at the onset of nominal yielding of the compression–flange. In these cases, D_{ce} may be taken as the elastic depth of the web in compression for the effective cross section, and M_{yce} may be taken correspondingly as $F_{yc}S_{xce}$.

White et al. (2019b) further discuss the calculation of D_{ce} and M_{yce} for hybrid sections with $S_{xte} < S_{xce}$. When M_{yce} is calculated using these procedures, the influence of the hybrid web is included directly in the calculation of M_{yce}. As such, R_h is to be taken equal to 1.0.

For box sections with $S_{xte} < S_{xce}$, simple closed-form equations that include the contribution of web longitudinal stiffeners to D_{ce} and M_{yce} are not available. Therefore, in lieu of determining D_{ce} and M_{yce} from a

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Sections with a longitudinally unstiffened compression flange that satisfy the following requirement shall qualify as noncompact web sections:

$$\lambda_{pw} < \lambda_w \leq \lambda_{rw} \qquad (6.12.2.2.2c\text{-}5)$$

in which:

λ_{rw} = limiting slenderness ratio for a noncompact web

$$= 4.6\sqrt{\frac{E}{F_{yc}}} \qquad (6.12.2.2.2c\text{-}6)$$

For a noncompact web section:

$$R_{pc} = \left[1 - \left(1 - \frac{R_h M_{yce}}{M_{pe}}\right)\left(\frac{\lambda_w - \lambda_{pw}}{\lambda_{rw} - \lambda_{pw}}\right)\right]\frac{M_{pe}}{M_{yce}} \leq \frac{M_{pe}}{M_{yce}}$$

(6.12.2.2.2c-7)

For noncompact web sections, the web load-shedding factor, R_b, shall be taken as 1.0.

Sections with a longitudinally unstiffened compression flange that satisfy the following limit shall qualify as slender web sections:

rigorous strain–compatibility analysis, it is recommended that web longitudinal stiffeners be neglected in the calculation of these variables for these types of sections.

Except as specified above and in Article 6.12.2.2.2d, web longitudinal stiffeners, if present, should be included in the calculation of M_{yce} and S_{xce}; however, any longitudinal stiffeners subjected to compression should be neglected in the computation of M_{pe}. This is due to the limited ability of longitudinal stiffeners to develop larger inelastic strains necessary to develop yielding throughout the depth of the cross section. Any enhancement of the resistance of compact or noncompact web sections due to the placement of web longitudinal stiffeners subjected to compression, other than the increase in M_{yce}, is neglected in these provisions. The section may be classified according to the web slenderness as specified in Articles 6.12.2.2.2c and 6.12.2.2.2d, as applicable, without considering the web longitudinal stiffeners. Web longitudinal stiffeners should be included in the calculation of the elastic gross and effective cross-section properties. In general, these cross-section elements provide a small but measurable contribution to the elastic cross-section properties and to the effective yield moment values. To be included in the calculation of the section properties, the longitudinal stiffener must be structurally continuous at the ends of interior web panels. At end panels or member ends, the longitudinal stiffener should be structurally continuous at one end and positively attached to the diaphragm at the other end.

Eq. 6.12.2.2.2c-6 defines the slenderness limit for a box-section noncompact web. This limit assumes simply-supported boundary conditions at the web-flange juncture, and is the same as the limit given by Eq. 6.10.6.2.3-1 for I-sections in cases where the area of the I-section flanges is small relative to the web area. This limit is slightly larger than the Class 3 limit for noncomposite box sections specified in CEN (2005), which is intended to ensure that the yield moment of the cross section can be developed, contingent on the satisfaction of other necessary plate slenderness and unbraced length requirement. Webs with a slenderness ratio exceeding this limit are termed slender. Sections with slender webs may rely upon significant web post bend-buckling resistance at the strength limit state.

For noncompact web sections, Eq. 6.12.2.2.2c-7 accounts for the influence of the web slenderness on the nominal flexural resistance. As $2D_{ce}/t_w$ approaches the noncompact web slenderness limit, λ_{rw}, the maximum potential value of the nominal flexural resistance—commonly referred to as the plateau strength—approaches $R_f R_h M_{yce}$. Eq. 6.12.2.2.2c-7 defines a web plastification factor that provides a smooth transition in the plateau strength from $R_f R_h M_{yce}$ to $R_f M_{pe}$ as $2D_{ce}/t_w$ varies from λ_{rw} to the compact web slenderness limit.

The plateau strength of a slender web section is $R_b R_f R_h M_{yce}$; the term R_b is discussed below.

$$\lambda_w > \lambda_{rw} \quad (6.12.2.2.2c\text{-}8)$$

For a slender web section, the following shall apply:

- The web plastification factor, R_{pc}, shall be taken as 1.0 for a homogeneous section and as R_h for a hybrid section.
- The web load-shedding factor, R_b, shall be determined using the applicable provisions of Article 6.10.1.10.2. In applying the provisions of Article 6.10.1.10.2, a_{wc} shall be determined with $b_{fc}t_{fc}$ taken as one-half of the flange area for a compact flange section, and as one-half of the effective flange area, $b_e t_{fc}/2$, for a noncompact or slender flange section, including corners and flange extensions; D_c shall be taken as the depth of the web in compression using the effective cross section, D_{ce}, and; for sections having webs of different thickness, t_w shall be taken as the smaller web thickness. The effective width of the flange, b_e, shall be determined as specified in Article 6.9.4.2.2b with F_{cr} taken equal to F_y.

For noncompact web and slender web sections, the following shall apply in the determination of the hybrid factor, R_h:

- For noncompact web and slender web sections with $S_{xte} \geq S_{xce}$, the hybrid factor, R_h, shall be determined using the provisions of Article 6.10.1.10.1.
- For noncompact web and slender web sections with $S_{xte} < S_{xce}$, the hybrid web stress state may be considered directly in the calculation of M_{yce} in lieu

Several simplifications may be applied for the slender web member calculations in many cases:

- The term, R_{pc}, simplifies to R_h for a slender-web hybrid box-section;
- The term, R_{pc}, simplifies to 1.0 for a slender-web homogeneous box-section, and;
- The term, R_f, simplifies to 1.0 and M_{yce} simplifies to M_{yc} if the box section has a compact flange.

Furthermore, the slender web box-section provisions may be applied conservatively, with R_b taken equal to 1.0, for any noncompact- or compact-web box section.

For compact and noncompact web sections, theoretical web bend buckling does not occur for moment levels up to the limit of the flexural resistance. Therefore, the web load-shedding factor, R_b, is simply equal to 1.0 for these sections. For sections containing a compact compression–flange and a compact web, the Article 6.12.2.2.2e plateau resistance $R_bR_{pc}R_fM_{yce}$ is simply equal to M_p. For slender web sections, R_b is to be calculated as specified herein to account for the reduction in the section flexural resistance caused by the shedding of stresses to the compression–flange due to bend buckling of the slender web.

For members with one or more longitudinal web stiffeners placed at d_s/D_{ce} approaching 0.76 or larger, where d_s is the distance from the centerline of the closest longitudinal stiffener to the inner surface of the compression–flange, the value of R_b determined from the provisions of Article 6.10.1.10.2 tends to be relatively small since these provisions neglect the impact of web longitudinal stiffeners on the web bend buckling response for $d_s/D_{ce} \geq 0.76$. These proportions are common in some types of columns and arch ribs. In addition, in narrow box sections, the value of a_{wc} employed in the calculation of R_b can be relatively large. The R_b equation tends to be conservative in these situations. In these cases, a larger plateau strength may be determined by using a strain–compatibility analysis considering an effective width of the longitudinally stiffened webs, as discussed in Lokhande and White (2018).

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of calculating R_h using Article 6.10.1.10.1. In this case R_h shall be taken equal to 1.0.

- In the calculation of R_h using the provisions of Article 6.10.1.10.1, for cross sections in which D_n is the distance from the elastic neutral axis to the compression–flange, A_{fn} shall be taken as one-half of the total compression–flange area for a compact flange section, and as one-half of the total effective compression–flange area for a noncompact or slender flange section, including corners and flange extensions; for cross sections in which D_n is the distance from the elastic neutral axis to the tension flange, A_{fn} shall be taken as one-half the total tension flange area, including corners and flange extensions; for box sections having webs of different thickness, t_w shall be taken as the smaller web thickness.

Sections with a longitudinally unstiffened compression flange that satisfy the following requirement shall qualify as compact flange sections:

$$\lambda_f \leq \lambda_{pf} \qquad (6.12.2.2.2c\text{-}9)$$

in which:

λ_f = compression flange slenderness

$$= \frac{b_{fi}}{t_{fc}} \qquad (6.12.2.2.2c\text{-}10)$$

where:

λ_{pf} = limiting slenderness ratio for a compact flange, taken as the value of λ_r specified in Table 6.9.4.2.1-1 for the type of flange under consideration

b_{fi} = inside width of the box section flanges (in.); forwelded box sections, clear width of the compression flange between the webs. For HSS, the provisions of Article 6.12.1.2.4 shall apply

t_{fc} = thickness of the compression flange (in.). For HSS, the provisions of Article 6.12.1.2.4 shall apply

For compact flange sections:

$$R_f = 1.0 \qquad (6.12.2.2.2c\text{-}11)$$

where:

R_f = compression flange slenderness factor

Sections with a longitudinally unstiffened compression flange that satisfy the following requirement shall qualify as noncompact flange sections:

Box-section flanges that satisfy the nonslender limit in uniform axial compression specified in Article 6.9.4.2.1 are defined herein as compact. For welded built-up box sections, λ_{pf} as specified by these provisions is comparable to the compact flange limit in AISC (2016b), and it is comparable to the Class 1 flange limit in CEN (2005), which is intended to ensure that the section can form a plastic hinge with a rotation capacity sufficient for plastic analysis, contingent on the satisfaction of other necessary plate slenderness and unbraced length requirements. This more restrictive limit is also related to the shift in Winter's classical plate effective width curve for welded built-up box sections discussed in Article C6.9.4.2.2b. For cold-formed HSS, λ_{pf} as specified by these provisions is comparable to the Class 2 flange limit in CEN (2005), which is intended to ensure that the plastic moment of the cross section can be developed, contingent on the satisfaction of other plate slenderness and member unbraced length requirements necessary to develop the plastic moment. For nonwelded built-up box sections and hot-formed HSS, λ_{pf} as specified by these provisions is slightly larger than the Class 2 flange limit in CEN (2005).

Box-section members with longitudinally unstiffened compression flanges traditionally classified in the AISC and AASHTO provisions as noncompact are

$$\lambda_{pf} < \lambda_f \leq \lambda_{rf} \qquad (6.12.2.2.2c\text{-}12)$$

in which:

λ_{rf} = limiting slenderness ratio for a noncompact flange

$$= 1.56\lambda_{pf} \qquad (6.12.2.2.2c\text{-}13)$$

For a noncompact flange section:

$$R_f = \left[1 - 0.15\left(\frac{\lambda_f - \lambda_{pf}}{\lambda_{rf} - \lambda_{pf}}\right)\right] \leq 1.0 \qquad (6.12.2.2.2c\text{-}14)$$

Sections with a longitudinally unstiffened compression flange that satisfy the following limit shall qualify as slender flange sections:

$$\lambda_f > \lambda_{rf} \qquad (6.12.2.2.2c\text{-}15)$$

For a slender flange section:

$$R_f = 0.85 \qquad (6.12.2.2.2c\text{-}16)$$

capable of developing a flexural resistance close to thecross section full plastic moment, M_p, if the webs are compact according to the classifications specified herein and the member is adequately braced (Lokhande and White, 2018). Box sections with a compression flange slenderness larger than the compact flange limit given by Eq. 6.12.2.2.2c-9 have a compression flange effective width smaller than the gross width of the flange, as well a compression flange slenderness factor, R_f, smaller than 1.0.

The compression flange slenderness factor, R_f, accounts for reductions in the plateau strength of noncompact and slender flange box sections due to the limited ability of these types of flanges (1) to develop the inelastic strains necessary to achieve yielding through the depth of the cross section, and (2) to accept the stresses shed to them due to bend buckling of a slender web. These reductions are in addition to the reductions associated with the compression flange effective width as well as the web-based parameters R_b and R_{pc}. Lokhande and White (2018) found that the reductions in the flexural resistance of these types of box-section members is represented with good accuracy by Eqs. 6.12.2.2.2c-12 through 6.12.2.2.2c-16.

For compact-flange box sections, the term R_f is taken simply equal to 1.0.

6.12.2.2.2d—Classification of Sections with a Longitudinally Stiffened Compression Flange

Sections with a longitudinally stiffened compression flange shall be classified as slender web sections.

For the purpose of calculating the yield moment of the effective cross section with respect to the compression flange, the longitudinally stiffened compression flange

C6.12.2.2.2d

Longitudinally stiffened box-section compression flanges are generally unable to withstand inelastic deformations necessary to develop significant yielding throughout the depth of the box section webs without significant reductions in their compressive resistance. Hence, the largest possible flexural resistance of these types of members is limited to the effective yield moment, M_{yce}, which corresponds to the development of the maximum compressive resistance of the longitudinally stiffened compression flange. As such, box sections with longitudinally stiffened compression flanges are in effect handled as slender web sections.

In cases where flange longitudinal stiffeners are provided but are not required for strength, the longitudinal stiffeners may be neglected and the flange may be considered as longitudinally unstiffened for purposes of calculating the member strength. However, all requirements pertaining to the longitudinal stiffeners must be satisfied for such sections.

The limit state of tension flange yielding does not apply to box section members with a longitudinally stiffened compression flange. Instead, in cases where S_{xte}

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shall be represented by the following effective area located at the centroid of the gross area of the entire flange plate and its longitudinal stiffeners:

$$A_{eff} = P_{nsp} / F_{yc} \quad (6.12.2.2.2d\text{-}1)$$

where:

P_{nsp} = nominal compressive resistance of the compression–flange calculated as specified in Article E6.1.3 (kip)

F_{yc} = specified minimum specified yield strength of the longitudinally stiffened compression–flange (ksi)

The effective elastic section moduli, S_{xce} and S_{xte}, shall be determined by dividing the effective moment of inertia, I_{xe}, by the distance to the corresponding extreme fiber of the cross section, where I_{xe} is calculated as specified in Article 6.12.2.2.2c using the effective area, A_{eff}, in lieu of the effective width, b_e, for the compression–flange.

For sections with a longitudinally stiffened compression flange and with $S_{xte} \geq S_{xce}$, D_{ce} shall be determined based on the effective elastic cross section and the nominal yield moment to the compression flange shall be calculated as $M_{yce} = F_{yc}S_{xce}$.

For sections with a longitudinally stiffened compression flange and with $S_{xte} < S_{xce}$, M_{yce} shall be calculated as the moment at nominal first yielding of the effective compression flange, considering early nominal yielding in tension, and the depth of the web in compression for the effective section, D_{ce}, shall be calculated accordingly based on this stress state.

$< S_{xce}$, early nominal yielding in tension is to be considered in the calculation of D_{ce} and M_{yce}. Figure C6.12.2.2.2c-1 illustrates the stress state corresponding to these variables for a hybrid cross section with a longitudinally unstiffened compression flange. For a homogeneous cross section, F_{yf} is equal to F_{yw}. In lieu of a more rigorous strain–compatibility analysis, this idealized model, and the corresponding equations in Article C6.12.2.2.2c, may be employed for calculating the D_{ce} and M_{yce} of a box section with a longitudinally stiffened compression flange by: (1) substituting the effective compression flange area, A_{eff}, determined using Eq. 6.12.2.2.2d-1, for A_{fce}; (2) modeling the effective compression–flange area as a zero-thickness strip located at the centroid of the gross compression flange area including the longitudinal stiffeners; (3) taking t_{fc} as zero inches in the last term of Eq. C6.12.2.2.2c-2; (4) taking the web depth as the depth between the effective compression flange elevation and the elevation of a zero-thickness strip representing the tension flange area and located at the centroid of the tension flange; and (5) neglecting any web longitudinal stiffeners.

When applying Eq. C6.12.2.2.2c-2 to a box section with a longitudinally stiffened flange, if a negative value of D_{ce} is obtained, the effective compression flange is relatively large. In this case, the neutral axis of the cross section corresponding to the nominal first yielding of the idealized zero-thickness strip representing the compression flange is located at the centroid of the effective compression flange. For this extreme case, the strains at the compression flange will be relatively small and essentially the entire cross section below the compression flange will be yielded in tension at the onset of nominal first yielding of the compression flange. Therefore, M_{yce} may be taken equal to the plastic moment of the effective cross section, M_{pe}, for this case. The inelastic deformation of the compression flange needed to develop this flexural resistance is relatively small and can be accommodated by the longitudinally stiffened compression flange. Also, the webs of the box section are loaded entirely in tension at the onset of nominal first yielding of the compression flange, and therefore web local buckling is not a consideration.

For sections with a longitudinally stiffened compression flange, the following shall apply:

- The compression flange slenderness factor, R_f, shall be taken equal to 1.0.
- The web plastification factor, R_{pc}, shall be taken equal to 1.0 for homogeneous sections, and as R_h determined using the provisions of Article 6.10.1.10.1 for hybrid sections with $S_{xte} \geq S_{xce}$.
- In the calculation of R_h using the provisions of Article 6.10.1.10.1, when D_n is the distance from the elastic neutral axis to the compression flange, A_{fn} shall be taken as $A_{eff}/2$, including corners and flange extensions; when D_n is the distance from the elastic

R_f is simply taken equal to 1.0 in sections with longitudinally stiffened compression flanges; the calculation of the effective area of the compression flange quantifies the flexural resistance of these types of box-section members accurately to conservatively without any further reduction (Lokhande and White, 2018).

neutral axis to the tension flange, A_{fn} shall be taken as one-half the total tension flange area including corners, flange extensions, and any longitudinal stiffeners; for sections having webs of different thickness, t_w shall be taken as the smaller web thickness. For hybrid sections in which $S_{xte} < S_{xce}$, the hybrid web stress state may be considered directly in the calculation of M_{yce} in lieu of calculating R_h using Article 6.10.1.10.1. In this case, R_h shall be taken equal to 1.0.

- The web load-shedding factor, R_b, shall be determined using the applicable provisions of Article 6.10.1.10.2. In applying the provisions of Article 6.10.1.10.2, a_{wc} shall be determined with $b_{fc}t_{fc}$ taken as $A_{eff}/2$; D_c shall be taken as the depth of the web in compression using the effective cross section, D_{ce}, and; for sections having webs of different thickness, t_w shall be taken as the smaller web thickness.

Except as specified herein, the areas of all longitudinal stiffeners should be included in the calculation of the elastic gross and effective cross-section properties. In cases in which the webs have different longitudinal stiffening, or in which only one web has longitudinal stiffeners, the longitudinal stiffeners should not be included.

6.12.2.2.2e—General Yielding, Compression Flange Local Buckling and Lateral–Torsional Buckling

The nominal flexural resistance based on the combined influence of general yielding, compression–flange local buckling and lateral–torsional buckling shall be determined as follows:

- If $L_b \le L_p$, then:

$$M_n = R_f R_b R_{pc} M_{yce} \quad (6.12.2.2.2e\text{-}1)$$

- Otherwise:

$$M_n = C_b R_b \left[R_{pc} R_f M_{yce} - \left(R_{pc} R_f M_{yce} - F_{yr} S_{xce} \right) \left(\frac{L_b - L_p}{L_r - L_p} \right) \right] \le R_b R_{pc} R_f M_{yce} \quad (6.12.2.2.2e\text{-}2)$$

in which:

J = St. Venant torsional constant of the gross cross section (in.^{4})

C6.12.2.2.2e

Eqs. 6.12.2.2.2e-1 and 6.12.2.2.2e-2 quantify the nominal flexural resistance considering the combined effects of general yielding, compression–flange local buckling, and lateral–torsional buckling. For longitudinally unstiffened box sections satisfying the cross-section proportion limits of Article 6.12.2.2.2b, the limiting unbraced length L_p for members with $D/b_{fo} < 2.0$ is always larger than $6D$. In addition, the limiting unbraced length L_r for these types of members is commonly larger than $70D$. As such, the reduction in the flexural resistance under uniform bending is never more than approximately 10 percent for longitudinally unstiffened box-section members with $D/b_{fo} < 2.0$ and $L_b/D < 20$ that satisfy the cross-section proportion limits of Article 6.12.2.2.2b. The maximum reduction in the flexural resistance under uniform bending is approximately one-half of this value, i.e., 5 percent, for members with $D/b_{fo} \le 2.0$ and $L_b/D \le 10$. Members that have stocky compression and tension flanges combined with webs proportioned at the maximum limit of Eq. 6.12.2.2.2b-1 exhibit the largest reduction in strength associated with the lateral–torsional buckling limit state. In designs where the moment gradient modifier C_b is greater than 1.10, this reduction is nonexistent.

For longitudinally stiffened box sections satisfying the cross-section proportion limits of Article 6.12.2.2.2b, the limiting unbraced length L_p for members with $D/b_{fo} \le$

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$$= \frac{4A_o^2}{\sum \frac{b_m}{t}} \quad (6.12.2.2.2e\text{-}3)$$

L_p = limiting unbraced length to achieve the nominal flexural resistance $R_f R_b R_{pc} M_{yce}$ under uniform moment (in.)

$$= 0.10 E r_y \frac{\sqrt{JA}}{M_{yce}} \quad (6.12.2.2.2e\text{-}4)$$

L_r = limiting unbraced length for calculation of the lateral–torsional buckling resistance (in.)

$$= 0.60 E r_y \frac{\sqrt{JA}}{F_{yr} S_{xce}} \quad (6.12.2.2.2e\text{-}5)$$

$$b_m = b_{fo} - t_w \quad (6.12.2.2.2e\text{-}6)$$

where:

A = gross cross-sectional area of the box-section, including any longitudinal stiffeners (in.^{2})

A_o = cross-sectional area enclosed by the mid-thickness of the walls of the box-section member (in.^{2})

b_{fo} = outside width of the box section taken as the distance from the outside to the outside of the box-section webs (in.)

b_m = gross width of each plate of the box-section member taken between the mid-thickness of the adjacent plates (in.)

C_b = moment gradient modifier determined as specified in Article A6.3.3

F_{yc} = specified minimum yield strength of the compression flange (ksi)

F_{yr} = compression flange stress at the onset of nominal yielding within the cross section, including residual stress effects, for moment applied about the axis of bending, taken as $0.5F_{yc}$ (ksi)

I_{xe} = effective moment of inertia of the cross section about the axis of bending determined using the effective width for the compression flange, b_e, calculated as specified in Article 6.9.4.2.2b with F_{cr} taken equal to F_{yc}, or using the effective area for the compression flange, A_{eff}, calculated as specified in Article 6.12.2.2.2d, as applicable, and the gross area for the tension flange (in.^{4}). Except as specified in Article 6.12.2.2.2d, any web and/or tension flange longitudinal stiffeners should be included in the calculation of I_{xe}.

L_b = unbraced length (in.)

M_{yce} = yield moment of the effective cross section with respect to the compression flange calculated as

2.0 is always larger than $4D$. In addition, the limiting unbraced length L_r for these types of members is commonly larger than $50D$. As such, the reduction in the flexural resistance under uniform bending is never more than approximately 20 percent for members with $D/b_{fo} \leq 2.0$ and $L_b/D \leq 20$ that satisfy the cross-section proportion limits of Article 6.12.2.2.2b. The maximum reduction in the flexural resistance under uniform bending is approximately one-half of this value, i.e., 10 percent, for members with $D/b_{fo} \leq 2.0$ and $L_b/D \leq 10$. Members that have stocky compression and tension flanges combined with webs proportioned at the maximum limit of Eq. 6.12.2.2.2b-1 exhibit the largest reduction in strength associated with the lateral–torsional buckling limit state. In designs where the moment gradient modifier C_b is greater than 1.20, this reduction is nonexistent.

The limiting unbraced length L_r is commonly larger than the largest practical unbraced length, taken as the smaller of $30D$ and $200r_y$, for all box sections satisfying the cross-section proportion limits of Article 6.12.2.2.2b. Therefore, elastic lateral–torsional buckling need not be considered for all practical noncomposite box section members. Eq. 6.12.2.2.2e-2 may be employed to calculate the resistance for any extreme situations where $L_b > L_r$.

L_p given by Eq. 6.12.2.2.2e-4 is the same as the corresponding limiting length for box sections given in the corresponding AISC (2016b) provisions with the exception that the plastic moment, M_p, is estimated as $1.3M_{yce}$. The bracing requirements to reach the lateral–torsional buckling plateau strength given by Eq. 6.12.2.2.2e-1 are comparable for all types of box-section members, irrespective of whether the cross section is capable of developing the plastic moment M_p or not (Lokhande and White, 2018). For general box-section members, it may be stated that Eq. 6.12.2.2.2e-4 is based on the theoretical length corresponding to an elastic lateral–torsional buckling resistance of $(1.3)(15)M_{yce} = 20M_{yce}$. Therefore, as a general rule, if the elastic lateral–torsional buckling load is greater than $20M_{yce}$, the box-section member may be checked solely for its plateau strength in flexure. The parameters F_{yr} and L_r in this Article differ from the corresponding AISC provisions. These parameters have been determined based on test simulation studies conducted by Lokhande and White (2018), as well as consideration of the lateral–torsional buckling resistance predictions of other standards such as CEN (2006). The limiting length, L_r, is taken as approximately 30 percent of the limiting length corresponding to theoretical elastic lateral–torsional buckling at a compression–flange stress equal to F_{yr}.

Eqs. 6.12.2.2.2e-4 and 6.12.2.2.2e-5 are derived from the fundamental equations for elastic lateral–torsional buckling of doubly symmetric rectangular box-section members. Lokhande and White (2018) show similar equations based on the rigorous theoretical elastic lateral–torsional buckling of singly-symmetric box-

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specified in Article 6.12.2.2.2c or 6.12.2.2.2d, as applicable (kip-in.)

R_f = compression flange slenderness factor determined as specified in Article 6.12.2.2.2c or 6.12.2.2.2d, as applicable

R_b = web load-shedding factor determined as specified in Article 6.12.2.2.2c or 6.12.2.2.2d, as applicable

R_{pc} = web plastification factor for the compression flange determined as specified in Article 6.12.2.2.2c or 6.12.2.2.2d, as applicable

r_y = radius of the gyration of the gross box-section about its minor principal axis, including any longitudinal stiffeners (in.^{4})

S_{xce} = effective elastic section modulus about the axis of bending to the compression flange determined using the effective width for the compression flange, b_e, calculated as specified in Article 6.9.4.2.2b with F_{cr} taken equal to F_{yc}, or using the effective area for the compression flange, A_{eff}, calculated as specified in Article 6.12.2.2.2d, as applicable and the gross area for the tension flange; the effective elastic section modulus, S_{xce} shall be determined by dividing the effective moment of inertia, I_{xe}, by the distance to the corresponding extreme fibers of the actual cross section (in.^{3})

t = thickness of each plate of the box-section member (in.)

section members. The corresponding lateral–torsional buckling resistance predictions using the rigorously derived equations for singly-symmetric box-section members are never more than 1 percent different from the predictions obtained by simply applying the doubly-symmetric section based equations specified herein to singly-symmetric section members. The maximum change in the limiting unbraced length, L_p, obtained by using the equations herein, versus the rigorously derived equations for singly-symmetric box-section members, is approximately 10 percent. The maximum change in the limiting unbraced length, L_r, obtained by using the equations herein versus the rigorously derived equations for singly-symmetric box-section members, is less than 1 percent. Therefore, the much simpler L_p and L_r equations derived assuming doubly symmetric cross sections are specified to address all types of rectangular box-section members in these provisions.

It should be emphasized that the calculations in this Article based on the underlying elastic lateral–torsional buckling equations always use the gross cross-section properties. These are combined with the use of the effective cross-section properties for all terms related to the yield or plastic moment resistance of the cross section.

6.12.2.2.2f—Service and Fatigue Limit States and Constructibility

Box-section webs shall satisfy the provisions of Article 6.10.3.3 for constructibility and the special fatigue requirement specified in Article 6.10.5.3.

C6.12.2.2.2f

The provisions of Article 6.10.3.3 investigate the webs for the shear due to the factored load for constructibility specified in Article 3.4.2.1. The nominal shear resistance for this check is limited to the shear-yielding or shear-buckling resistance, V_{cr}. The use of tension-field action is not permitted under this load during construction.

The provisions of Article 6.10.5.3 are intended to alleviate any significant elastic flexing of the webs due to shearing actions under the unfactored permanent load plus the Fatigue I load combination. The shear is also limited in this case to V_{cr} to ensure that the member is able to sustain an infinite number of loadings without fatigue cracking due to this effect.

For box-section members subject to torsion, web shear yielding or shear buckling should be checked according to the provisions of Articles 6.10.3.3 and 6.10.5.3 for the critical web subjected to additive flexural and torsional shear. Shear yielding or shear buckling of the flange plates in box-section members need not be checked because the flange torsional shears are generally small at these limit states and such a check is unlikely to control. Also, flange shear stresses due to flexure, which are tangent to the wall of the flange plate, are maximum at the connection to the webs, zero at a location within the

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The provisions of Article 6.9.4.5 also shall be satisfied for the applicable plates within the cross section, at the service limit state and for constructibility, when one or more of the following conditions are applicable:

- The section is a slender web section as defined in Article 6.12.2.2.2c;
- The section contains slender longitudinally stiffened plate panels as defined in Article E6.1.2;
- The slenderness, λ_f, of a longitudinally unstiffened compression flange exceeds λ_{pf} as defined in Article 6.12.2.2.2c.

The flanges of noncomposite box-section members also shall satisfy the following requirement at the service limit state:

$$f_f \leq 0.80 R_h F_{yf} \qquad (6.12.2.2.2f\text{-}1)$$

where:

f_f = flange stress due to overall flexure of the box-section member under Service II loads (ksi)
F_{yf} = specified minimum yield strength of the flange (ksi)
R_h = hybrid factor determined as specified in Article 6.12.2.2.2c or 6.12.2.2.2d, as applicable

middle of the plate, and the net shear force in the flanges is zero.

Web longitudinal stiffeners are ignored in the calculation of the web shear yielding or shear buckling resistance, V_{cr}.

Since post-buckling resistance is assumed at the strength limit state in computing the nominal flexural resistance of box-section members with slender webs, box sections containing slender longitudinally stiffened plate panels, sections with a noncompact or slender longitudinally unstiffened compression flange, such members must also satisfy the provisions of Article 6.9.4.5 to ensure that plate local buckling due to flexural stresses does not occur theoretically at the service limit state and for constructibility. Plate local buckling is not checked at the fatigue limit state in Article 6.9.4.5 because the plate buckling check under the Service II loads will tend to control over a similar check under the unfactored permanent load plus the Fatigue I load combination.

For box-section members subjected to combined axial and flexural stresses, Article 6.9.4.5 checks plate local buckling directly for the combined stress state.

6.12.2.2.2g—Flange Effective Width or Area Accounting for Shear Lag Effects

To account for shear lag effects in the calculation of the flexural resistance at the strength limit state, in lieu of a more rigorous analysis, where a box-section member has an effective span, L_{eff}, less than $5b_{fi}$, the following quantities shall be multiplied by:

$$\left(L_{eff}/5\right)/b_{fi} \leq 1.0 \qquad (6.12.2.2.2g\text{-}1)$$

- The effective area, $b_e t_f$, of a longitudinally unstiffened compression flange, with b_e determined as specified in Article 6.9.4.2.2b with F_{cr} taken equal to F_y, or;

C6.12.2.2.2g

This Article provides simplified rules requiring the consideration of shear lag effects only for cases where the effective span, L_{eff}, is less than $5b_{fi}$, with the exception of the calculation of elastic flexural stresses under service and fatigue limit states and for constructibility within cantilevers and within the negative moment regions of continuous-span members. In many practical situations, there is no reduction in the flange effective width due to shear lag. The reductions in flange effectiveness due to shear lag are more significant for cantilevers and negative moment regions at these limit states. Where required, the reductions on the compression flange effective area, $b_e t_f$, the compression flange effective area, A_{eff}, and the tension flange gross area, as applicable, are applied in a simplified manner as a uniform reduction for all the cross sections within the effective span of the member under

- The effective area, A_{eff}, of a longitudinally stiffened compression flange determined as specified in Article 6.12.2.2.2d, and;
- The gross area of the tension flange including any longitudinal stiffeners,

where:

b_{fi} = inside width of the box section flanges (in.); for welded box sections, the clear width of the flange under consideration between the webs. For HSS, the provisions of Article 6.12.1.2.4 shall apply.

L_{eff} = effective span taken as the span length for simple spans (in.); the distance between points of permanent load contraflexure, or between a simple support and a point of permanent load contraflexure, as applicable, for continuous spans; and two times the length from the support to the location of zero moment for cantilever spans.

t_f = thickness of the flange under consideration (in.). For HSS, the provisions of Article 6.12.1.2.4 shall apply.

The centroid of the reduced flange areas, where any reduction applies, shall be taken as the centroidal location of the flange area prior to applying the reduction.

To account for shear lag effects in the calculation of elastic flexural stresses at the service and fatigue limit states and for constructibility, in lieu of a more rigorous analysis, the following shall apply:

- Within the negative moment regions of continuous-span members, and within cantilevers, in cases where L_{eff} is less than $30b_{fi}$, the reduction factor given by Eq. 6.12.2.2.2g-1 shall be replaced by the following:

  - For $L_{eff} / b_{fi} \le 15$:

$$\left[0.376 + 0.0542\frac{L_{eff}}{b_{fi}} - 0.00156\left(\frac{L_{eff}}{b_{fi}}\right)^2\right] \le \frac{\left(L_{eff} / 5\right)}{b_{fi}} \tag{6.12.2.2.2g-2}$$

  - For $30 > L_{eff} / b_{fi} > 15$:

$$\left[0.697 + 0.00940\frac{L_{eff}}{b_{fi}}\right] \le 1.0 \tag{6.12.2.2.2g-3}$$

For longitudinally stiffened compression flanges, an additional factor of 0.9 shall be applied to Eq. 6.12.2.2.2g-2 or 6.12.2.2.2g-3, as applicable.

consideration. These reductions are employed only for the purpose of determining cross-section resistances at the strength limit states, or cross-section stresses at the service and fatigue limit states and for constructibility. They are not intended as reductions to be applied within the bridge structural analysis.

For the consideration of shear lag effects at the strength limit state, the requirements specified herein are comparable to the Article 4.6.2.6.4 requirements specified for orthotropic steel decks, which recognize the benefits of inelastic redistribution of flange stresses. For the consideration of shear lag effects under service and fatigue limit states and for constructibility, the requirements specified herein also recognize that the reduction in the flange effectiveness in simple-span members and within the positive moment regions of continuous-span members tends to be relatively small, and may be neglected as a coarse approximation, as long as the flange width, b_{fi}, is below the $L_{eff}/5$ limit. However, the reduction in the elastic flange effectiveness reduction for cantilevers and for negative moment regions of continuous spans tends to be more significant. Eqs. 6.12.2.2.2g-2 and 6.12.2.2.2g-3 are a fit to the curve provided by FHWA (1980) and Wolchuk (1997) for longitudinally unstiffened flanges, which is in turn based largely on the work by Moffatt and Dowling (1975, 1976). FHWA (1980) and Wolchuk (1997) recommend a reduction factor for longitudinally stiffened flanges that is up to 10 percent smaller than the value recommended for longitudinally unstiffened flanges. In lieu of a more refined calculation, it is specified that the values given by Eqs. 6.12.2.2.2g-2 and 6.12.2.2.2g-3 be multiplied by 0.9 for longitudinally stiffened flanges. White et al. (2019b) provide further discussion of the rationale for these curves. The requirements specified herein assume that flange extensions are sufficiently narrow such that they are fully effective with respect to shear lag, based on the compactness requirements of Eq. 6.12.2.2.2b-6. Positive and negative bending regions are defined herein based on the distance between the inflection points in positive or negative bending under the component dead load.

For special structures employing box-section members with unusually wide flanges compared to the effective span, the Engineer may wish to conduct refined analyses to obtain a more accurate accounting of shear lag effects. Full nonlinear analyses including the effect of residual stresses are necessary to capture the influence of inelastic stress redistribution on the flange effective widths at the strength limit state, which may be prohibitive for more routine bridge designs.

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- For all other moment regions in cases where L_{eff} is less than $5b_{fi}$, the reduction factor given by Eq. 6.12.2.2.2g-1 shall apply.

6.12.2.2.3—Circular Tubes and Round HSS

For noncomposite circular tubes, including round HSS, the nominal flexural resistance shall be taken as the smaller value based on yielding or local buckling, as applicable. The D/t of circular tubes used as flexural members shall not exceed $0.45E/F_y$.

For yielding, the nominal flexural resistance shall be taken as:

$$M_n = M_p = F_y Z \qquad (6.12.2.2.3\text{-}1)$$

where:

D = outside diameter of tube (in.)
M_p = plastic moment (kip-in.)
t = wall thickness of the tube (in.). For round HSS, the provisions of Article 6.12.1.2.4 shall apply.
Z = plastic section modulus (in.^{3})

For sections where D/t is less than or equal to $0.07E/F_y$, the wall element of the section is defined as compact.

For sections where D/t exceeds $0.07E/F_y$, local buckling shall be checked. For local buckling, the nominal flexural resistance shall be taken as:

- If $\frac{D}{t} \leq \frac{0.31E}{F_y}$, then:

$$M_n = \left(\frac{0.021E}{\frac{D}{t}} + F_y \right) S \qquad (6.12.2.2.3\text{-}2)$$

- If $\frac{D}{t} > \frac{0.31E}{F_y}$, then:

$$M_n = F_{cr} S \qquad (6.12.2.2.3\text{-}3)$$

in which:

F_{cr} = elastic local buckling stress (ksi)

$$= \frac{0.33E}{\frac{D}{t}} \qquad (6.12.2.2.3\text{-}4)$$

where:

S = elastic section modulus (in.^{3})

C6.12.2.2.3

Failure modes and post-buckling behavior of noncomposite circular tubes, including round Hollow Structural Sections (HSS), can be grouped into the following three categories (Sherman, 1992; Ziemian, 2010): 1) for D/t less than about $0.05E/F_y$, a long inelastic plateau occurs in the moment-rotation curve. The cross section gradually ovalizes, then local wave buckles eventually form after which the flexural resistance slowly decays; 2) for $0.05E/F_y \leq D/t \leq 0.10E/F_y$, the plastic moment is nearly achieved but a single local buckle develops and the flexural resistance decays slowly with little or no inelastic plateau; and 3) for $D/t > 0.10E/F_y$, multiple buckles form suddenly with little ovalization and the flexural resistance drops rapidly to a more stable level. The specified flexural resistance equations reflect the above regions of behavior for sections with long constant moment regions and little restraint against ovalization at the failure location. The equations are based on five North American studies involving hot-formed seamless pipe, electric-resistance-welded pipe, and fabricated tubing (Sherman, 1992; Ziemian, 2010).

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6.12.2.2.4—Tees and Double Angles

6.12.2.2.4a—General

For tees and double angles loaded in the plane of symmetry, the nominal flexural resistance shall be taken as the smallest value based on yielding, lateral–torsional buckling, flange local buckling, and local buckling of tee stems and double angle web legs, as applicable.

C6.12.2.2.4a

The provisions for tees and double angles given herein are taken from AISC (2016b). The legs of double angles in continuous contact or with separators may together be assumed treated as the double angle web legs in checking these provisions. The plane of symmetry is assumed to be that formed by their y-axis. For flexure of tees and double angles about the y-axis, which is considered to be a rare case in bridge applications, consult the Commentary to Section F9 of AISC (2016b).

6.12.2.2.4b—Yielding

For tee stems and double angle web legs subject to tension, the nominal flexural resistance based on yielding shall be taken as:

$$M_n = F_y Z_x \leq 1.6 M_y \qquad (6.12.2.2.4b\text{-}1)$$

For tee stems subject to compression, the nominal flexural resistance based on yielding shall be taken as:

$$M_n = M_y \qquad (6.12.2.2.4b\text{-}2)$$

For double angle web legs subject to compression, the nominal flexural resistance based on yielding shall be taken as:

$$M_n = 1.5 M_y \qquad (6.12.2.2.4b\text{-}3)$$

where:

F_y = specified minimum yield strength (ksi)
M_y = yield moment = $F_y S_x$ (kip-in.)
S_x = elastic section modulus about the x-axis with respect to the tip of the tee stem or double angle web legs (in.^{3})
Z_x = plastic section modulus about the x-axis (in.^{3})

C6.12.2.2.4b

The limit on M_n in Eq. 6.12.2.2.4b-1 of $1.6M_y$ for cases where the tee stem or double angle web legs are in tension is intended to indirectly control situations where significant yielding of the stem or web legs may occur at service load levels. Similarly, M_n is limited to M_y in Eq. 6.12.2.2.4b-2 for cases where the tee stem is in compression, and to $1.5M_y$ in Eq. 6.12.2.2.4b-3 for cases where double angle web legs are subject to compression.

6.12.2.2.4c—Lateral–Torsional Buckling

For tee stems and double angle web legs subject to tension, the nominal flexural resistance based on lateral–torsional buckling shall be taken as:

- If $L_b \leq L_p$, then lateral–torsional buckling shall not apply.

- If $L_p < L_b \leq L_r$, then:

$$M_n = M_p - \left(M_p - M_y\right)\left(\frac{L_b - L_p}{L_r - L_p}\right) \qquad (6.12.2.2.4c\text{-}1)$$

C6.12.2.2.4c

Eq. 6.12.2.2.4c-5 is a simplified version of the elastic lateral–torsional buckling equation developed in Kitipornchai and Trahair (1980) and discussed further in Ellifritt et al. (1992). For the case of tee stems and double angle web legs subject to tension, i.e., when the flange is subject to compression, Eq. 6.12.2.2.4c-1 provides a linear transition from the full plastic moment, M_p, to the yield moment, M_y.

The moment gradient modifier C_b specified for I-sections in Article A6.3.3 is not included in the equations of this Article as the application of C_b to cases where the stem is in compression is unconservative. Also, for reverse curvature bending, the portion with the stem in

- If $L_b > L_r$, then:

$$M_n = M_{cr} \qquad (6.12.2.2.4c\text{-}2)$$

in which:

$$L_p = 1.76 r_y \sqrt{\frac{E}{F_y}} \qquad (6.12.2.2.4c\text{-}3)$$

$$L_r = 1.95\left(\frac{E}{F_y}\right)\frac{\sqrt{I_y J}}{S_x}\sqrt{2.36\left(\frac{F_y}{E}\right)\frac{dS_x}{J}+1} \qquad (6.12.2.2.4c\text{-}4)$$

$$M_{cr} = \frac{1.95E}{L_b}\sqrt{I_y J}\left(B+\sqrt{1+B^2}\right) \qquad (6.12.2.2.4c\text{-}5)$$

$$B = 2.3\left(\frac{d}{L_b}\right)\sqrt{\frac{I_y}{J}} \qquad (6.12.2.2.4c\text{-}6)$$

where:

d = depth of the tee or width of the double angle web leg in tension (in.)
I_y = moment of inertia about the y-axis (in.^{4})
J = St. Venant torsional constant (in.^{4})
L_b = unbraced length (in.)
r_y = radius of gyration about the y-axis (in.)
S_x = elastic section modulus about the x-axis with respect to the tip of the tee stem or double angle web legs (in.^{3})

For tee stems and double angle web legs subject to compression anywhere along the unbraced length, the nominal flexural resistance based on lateral–torsional buckling shall be taken as:

- For tee stems:

$$M_n = M_{cr} \le M_y \qquad (6.12.2.2.4c\text{-}7)$$

- For double angle web legs:
  - If $\frac{M_y}{M_{cr}} \le 1.0$, then:

$$M_n = \left(1.92 - 1.17\sqrt{\frac{M_y}{M_{cr}}}\right)M_y \le 1.5M_y \qquad (6.12.2.2.4c\text{-}8)$$

  - If $\frac{M_y}{M_{cr}} > 1.0$, then:

$$M_n = \left(0.92 - \frac{0.17M_{cr}}{M_y}\right)M_{cr} \qquad (6.12.2.2.4c\text{-}9)$$

compression may govern the lateral–torsional buckling resistance even though the corresponding moments may be small in relation to the moments in the other portions of the unbraced length. The lateral–torsional buckling resistance for the case where the stem is in compression is substantially smaller than for the case where the stem is in tension. For cases where the stem is in tension, connection details should be designed to minimize end restraint moments that may cause the stem to be in flexural compression at the ends of the member.

For rolled sections, the St. Venant torsional constant J, including the effect of the web-to-flange fillets, is tabulated in AISC (2017). For fabricated sections, Eq. A6.3.3-9 may be used with one of the flange terms removed.

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in which:

$$B = -2.3\left(\frac{d}{L_b}\right)\sqrt{\frac{I_y}{J}} \qquad (6.12.2.2.4c\text{-}10)$$

where:

d = depth of the tee or width of the double angle web leg in compression (in.)
M_{cr} = elastic lateral–torsional buckling moment determined from Eq. 6.12.2.2.4c-5 (kip-in.)
M_y = yield moment = F_yS_x (kip-in.)
S_x = elastic section modulus about the x-axis with respect to the tip of the tee stem or double angle web legs (in.^{3})

6.12.2.2.4d—Flange Local Buckling

For tee flanges subject to compression, the nominal flexural resistance based on flange local buckling shall be taken as:

- If $\lambda_f \leq \lambda_{pf}$, then flange local buckling shall not apply.

- If $\lambda_{pf} < \lambda_f \leq \lambda_{rf}$, then:

$$M_n = M_p - \left(M_p - 0.7F_yS_{xc}\right)\left(\frac{\lambda_f - \lambda_{pf}}{\lambda_{rf} - \lambda_{pf}}\right) \qquad (6.12.2.2.4d\text{-}1)$$

- If $\lambda_f > \lambda_{rf}$, then:

$$M_n = \frac{0.7ES_{xc}}{\left(\frac{b_f}{2t_f}\right)^2} \qquad (6.12.2.2.4d\text{-}2)$$

For double angles with the flange legs subject to compression, the nominal flexural resistance based on flange local buckling shall be taken as:

- If $\lambda_f \leq \lambda_{pf}$, then flange local buckling shall not apply.

- If $\lambda_{pf} < \lambda_f \leq \lambda_{rf}$, then:

$$M_n = F_yS_{xc}\left(2.43 - 1.72\left(\frac{b_f}{2t_f}\right)\sqrt{\frac{F_y}{E}}\right) \qquad (6.12.2.2.4d\text{-}3)$$

- If $\lambda_f > \lambda_{rf}$, then:

C6.12.2.2.4d

For cases where tee flanges or flange legs of double angles are in compression and λ_f does not exceed λ_{pf}, flange local buckling does not control and need not be checked. Eq. 6.12.2.2.4d-1 represents an inelastic flange local buckling resistance equation provided for tee flanges in AISC (2016b). Eq. 6.12.2.2.4d-3 represents a local buckling resistance equation provided for determining the inelastic local buckling resistance of single-angle legs in Section F10 of AISC (2016b), which is conservatively applied to determine the inelastic local buckling resistance of double angles with the flange legs subject to compression and loaded in the plane of symmetry as recommended in AISC (2016b). Elastic flange local buckling resistance equations for cases with λ_f exceeding λ_{rf}, i.e., for slender flanges, are provided by Eqs. 6.12.2.2.4d-2 and 6.12.2.2.4d-4. The flanges of all rolled T-sections given in AISC (2016b) satisfy Eq. 6.10.2.2-1; therefore, this limit need only be checked for fabricated T-sections.

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$$M_n = \frac{0.7ES_{xc}}{\left(\frac{b_f}{2t_f}\right)^2} \quad (6.12.2.2.4d\text{-}4)$$

in which:

λ_f = flange slenderness = $b_f/2t_f$

M_p = plastic moment (kip-in.) = $F_y Z_x \leq 1.6M_y$ (6.12.2.2.4d-5)

λ_{pf} = limiting slenderness for a compact flange

$$= 0.38\sqrt{\frac{E}{F_y}} \text{ for tees} \quad (6.12.2.2.4d\text{-}6)$$

$$= 0.54\sqrt{\frac{E}{F_y}} \text{ for double angles} \quad (6.12.2.2.4d\text{-}7)$$

λ_{rf} = limiting slenderness for a noncompact flange

$$= 1.0\sqrt{\frac{E}{F_y}} \text{ for tees} \quad (6.12.2.2.4d\text{-}8)$$

$$= 0.91\sqrt{\frac{E}{F_y}} \text{ for double angles} \quad (6.12.2.2.4d\text{-}9)$$

where:

b_f = flange width (in.). For double angles, b_f shall be taken as the sum of the widths of the outstanding legs, not including any gap in-between the angles.

S_{xc} = elastic section modulus about the x-axis with respect to the compression flange (in.^{3})

M_y = yield moment = $F_y S_{xc}$ (kip-in.)

t_f = flange thickness (in.)

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

Flanges of fabricated T-sections in compression or tension shall satisfy Eq. 6.10.2.2-1.

6.12.2.2.4e—Local Buckling of Tee Stems and Double Angle Web Legs

For tee stems subject to compression, the nominal flexural resistance based on local buckling of the stem shall be taken as:

$$M_n = F_{cr} S_x \quad (6.12.2.2.4e\text{-}1)$$

in which:

F_{cr} = critical stress (ksi) taken as:

C6.12.2.2.4e

Separate equations for checking local buckling of tee stems and double angle web legs subject to compression are provided herein. The equations are taken from AISC (2016b). The derivation of the equations is described in the Commentary to Section F9 of AISC (2016b).

- If $\frac{d}{t_w} \leq 0.84\sqrt{\frac{E}{F_y}}$, then

$$F_{cr} = F_y \qquad (6.12.2.2.4e\text{-}2)$$

- If $0.84\sqrt{\frac{E}{F_y}} < \frac{d}{t_w} \leq 1.52\sqrt{\frac{E}{F_y}}$, then

$$F_{cr} = \left(1.43 - 0.515\frac{d}{t_w}\sqrt{\frac{F_y}{E}}\right)F_y \qquad (6.12.2.2.4e\text{-}3)$$

- If $\frac{d}{t_w} > 1.52\sqrt{\frac{E}{F_y}}$, then

$$F_{cr} = \frac{1.52E}{\left(\frac{d}{t_w}\right)^2} \qquad (6.12.2.2.4e\text{-}4)$$

where:

d = total depth of the tee (in.)
S_x = elastic section modulus about the x-axis with respect to the tip of the tee stem (in.^{3})
t_w = thickness of the tee stem (in.)

For double angle web legs subject to compression, the nominal flexural resistance based on local buckling of the web legs shall be taken as:

- If $\frac{d}{t} \leq 0.54\sqrt{\frac{E}{F_y}}$, then local buckling of the web legs shall not apply.

- If $0.54\sqrt{\frac{E}{F_y}} < \frac{d}{t} \leq 0.91\sqrt{\frac{E}{F_y}}$, then:

$$M_n = F_y S_{xc}\left(2.43 - 1.72\left(\frac{d}{t}\right)\sqrt{\frac{F_y}{E}}\right) \qquad (6.12.2.2.4e\text{-}5)$$

- If $\frac{d}{t} > 0.91\sqrt{\frac{E}{F_y}}$, then:

$$M_n = \frac{0.7ES_{xc}}{\left(\frac{d}{t}\right)^2} \qquad (6.12.2.2.4e\text{-}6)$$

where:

d = total depth of the double angle web legs (in.)

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S_{xc} = elastic section modulus about the x-axis with respect to the tip of the double angle web legs (in.^{3})
t = thickness of an individual double angle web leg (in.)

6.12.2.2.5—Channels

For channels in flexure about their strong or x-axis, the nominal flexural resistance shall be taken as the smaller value based on yielding or lateral–torsional buckling, as applicable.

For yielding, the nominal flexural resistance shall be taken as:

$$M_n = M_p = F_y Z_x \quad (6.12.2.2.5\text{-}1)$$

where:

F_y = specified minimum yield strength (ksi)
M_p = plastic moment (kip-in.)
Z_x = plastic section modulus about the x-axis (in.^{3})

Where the unbraced length L_b exceeds L_p, lateral–torsional buckling shall be checked. For lateral–torsional buckling, the nominal flexural resistance shall be taken as:

- If $L_b \le L_r$, then:

$$M_n = C_b\left[M_p - \left(M_p - 0.7F_y S_x\right)\left(\frac{L_b - L_p}{L_r - L_p}\right)\right] \le M_p \quad (6.12.2.2.5\text{-}2)$$

- If $L_b > L_r$, then:

$$M_n = F_{cr} S_x \le M_p \quad (6.12.2.2.5\text{-}3)$$

in which:

F_{cr} = elastic lateral–torsional buckling stress (ksi)

$$= \frac{C_b \pi^2 E}{\left(\frac{L_b}{r_{ts}}\right)^2}\sqrt{1 + 0.078\frac{Jc}{S_x h_o}\left(\frac{L_b}{r_{ts}}\right)^2} \quad (6.12.2.2.5\text{-}4)$$

$$c = \frac{h_o}{2}\sqrt{\frac{I_y}{C_w}} \quad (6.12.2.2.5\text{-}5)$$

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

$$= \frac{t_f b^3 h_o^2}{12}\left(\frac{3bt_f + 2h_o t_w}{6bt_f + h_o t_w}\right) \quad (6.12.2.2.5\text{-}6)$$

C6.12.2.2.5

The provisions for channels in flexure about their strong or x-axis are taken from AISC (2016b). For lateral–torsional buckling, where L_b is less than or equal to the limiting length, L_p, lateral–torsional buckling does not control and need not be checked. The lateral–torsional buckling Eqs. 6.12.2.2.5-2 and 6.12.2.2.5-3 assume that the channel has compact flanges satisfying Eq. 6.12.2.2.5-10 and a compact web satisfying Eq. 6.12.2.2.5-12; hence, flange and web local buckling need not be checked. All rolled channels given in AISC (2016b) have compact flanges and webs for $F_y \le 65$ ksi. Thus, Eqs. 6.12.2.2.5-10 and 6.12.2.2.5-12 need not be checked for rolled channels. To utilize Eqs. 6.12.2.2.5-2 and 6.12.2.2.5-3 for fabricated or bent-plate channels, Eqs. 6.12.2.2.5-10 and 6.12.2.2.5-12 must be satisfied. Eqs. 6.12.2.2.5-2 and 6.12.2.2.5-3 also assume that the channel is restrained at the brace points such that twisting of the member does not occur at those points. For fabricated or bent-plate channels, Eq. 6.12.2.2.5-5 taken from Salmon and Johnson (1996) and Eq. A6.3.3-9 may be used for the computation of C_w and J, respectively. For rolled channels, values of the warping torsional constant, C_w, and the St. Venant torsional constant, J, including the effect of the sloping flanges and web-to-flange fillets, are tabulated in AISC (2017) and may be used in lieu of the values fromthese equations.

For channels in flexure about their weak or y-axis, the limit of $1.6F_yS_y$ on the nominal flexural resistance is intended to indirectly prevent substantial yielding of the member at service load levels.

L_p = limiting unbraced length to achieve the nominal flexural resistance M_p under uniform bending (in.)

$$= 1.76 r_y \sqrt{\frac{E}{F_y}} \quad (6.12.2.2.5\text{-}7)$$

L_r = limiting unbraced length to achieve the nominal onset of yielding under uniform bending with consideration of compression flange residual stress effects (in.)

$$= 1.95 r_{ts} \frac{E}{0.7F_y} \sqrt{\frac{Jc}{S_x h_o}} \sqrt{1 + \sqrt{1 + 6.76\left(\frac{0.7F_y}{E}\frac{S_x h_o}{Jc}\right)^2}} \quad (6.12.2.2.5\text{-}8)$$

$$r_{ts}^2 = \frac{\sqrt{I_y C_w}}{S_x} \quad (6.12.2.2.5\text{-}9)$$

where:

C_b = moment gradient modifier determined as specified in Article A6.3.3
L_b = unbraced length (in.)
b = distance between the toe of the flange and the centerline of the web (in.)
h_o = distance between flange centroids (in.)
I_y = moment of inertia about the y-axis (in.^{4})
J = St. Venant torsional constant (in.^{4})
r_{ts} = radius of gyration used in the determination of L_r (in.)
r_y = radius of gyration about the y-axis (in.)
S_x = elastic section modulus about the x-axis (in.^{3})
t_f = thickness of the flange (in.); for rolled channels, use the average thickness
t_w = thickness of the web (in.)

For channels in flexure about their weak or y-axis, the nominal flexural resistance shall be determined according to the provisions specified in Article 6.12.2.2.1. The nominal flexural resistance shall not exceed $1.6F_yS_y$, where S_y is the elastic section modulus about the y-axis.

The flange slenderness, λ_f, of fabricated or bent-plate channels shall satisfy:

$$\lambda_f \leq \lambda_{pf} \quad (6.12.2.2.5\text{-}10)$$

in which:

λ_f = flange slenderness of the channel = b_f/t_f
λ_{pf} = limiting slenderness for a compact flange

$$= 0.38\sqrt{\frac{E}{F_y}} \quad (6.12.2.2.5\text{-}11)$$

where:

b_f = flange width (in.)
t_f = flange thickness (in.)

The web slenderness of fabricated or bent-plate channels shall satisfy:

$$\frac{D}{t_w} \leq \lambda_{pw} \quad (6.12.2.2.5\text{-}12)$$

in which:

λ_{pw} = limiting slenderness for a compact web

$$= 3.76\sqrt{\frac{E}{F_y}} \quad (6.12.2.2.5\text{-}13)$$

where:

D = web depth (in.)
t_w = web thickness (in.)

6.12.2.2.6—Single Angles

Single angles should not be used as pure flexural members. Single angles subject to combined axial compression and flexure may be designed according to the provisions specified in Article 6.9.4.4.

C6.12.2.2.6

Single angles are not typically intended to serve as pure flexural members in bridge applications. In most practical applications, single angles are subject to flexure about both principal axes due to the eccentricity of applied axial loads. The condition of flexure due to eccentric axial tension is primarily addressed through the use of the shear lag coefficient, U, specified in Article 6.8.2.2. The condition of flexure due to eccentric axial compression may be efficiently handled through the use of an effective slenderness ratio, $(K\ell/r)_{eff}$, as specified in Article 6.9.4.4, which allows single angles satisfying certain specified conditions to be designed as axially loaded compression members for flexural buckling only. Thus, the calculation of the nominal flexural resistance M_n of a single-angle member is typically not required for these common cases. In certain unusual cases discussed in Article C6.9.4.4, single angles subject to combined flexure and axial compression must be evaluated as beam-columns according to the provisions specified in Section H2 of AISC (2016b) in lieu of using the effective slenderness ratio. In such cases, M_n of the single angle member may be determined according to the procedures given in Section F10 of AISC (2016b).

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6.12.2.2.7—Rectangular Bars and Solid Rounds

For rectangular bars and solid rounds in flexure, the nominal flexural resistance shall be taken as the smaller value based on yielding or lateral–torsional buckling, as applicable.

For yielding, the nominal flexural resistance shall be taken as:

- For rectangular bars with $\frac{L_b d}{t^2} \le \frac{0.08E}{F_y}$ in flexure about their major geometric axis, rectangular bars in flexure about their minor geometric axis, and solid rounds:

$$M_n = M_p = F_y Z \le 1.6 M_y \qquad (6.12.2.2.7\text{-}1)$$

where:

d = depth of the rectangular bar (in.)
F_y = specified minimum yield strength (ksi)
L_b = unbraced length for lateral displacement or twist, as applicable (in.)
M_p = plastic moment (kip-in.)
M_y = yield moment (kip-in.)
t = width of the rectangular bar parallel to the axis of bending (in.)
Z = plastic section modulus (in.^{3})

For lateral–torsional buckling, the nominal flexural resistance shall be taken as follows for rectangular bars in flexure about their major geometric axis:

- If $\frac{0.08E}{F_y} < \frac{L_b d}{t^2} \le \frac{1.9E}{F_y}$, then:

$$M_n = C_b \left[1.52 - 0.274\left(\frac{L_b d}{t^2}\right)\frac{F_y}{E}\right] M_y \le M_p \qquad (6.12.2.2.7\text{-}2)$$

- If $\frac{L_b d}{t^2} > \frac{1.9E}{F_y}$, then:

$$M_n = F_{cr} S_x \le M_p \qquad (6.12.2.2.7\text{-}3)$$

in which:

$$F_{cr} = \frac{1.9EC_b}{\frac{L_b d}{t^2}} \text{ (ksi)}$$

where:

C_b = moment gradient modifier determined as specified in Article A6.3.3
S_x = section modulus about the major geometric axis (in.^{3})

C6.12.2.2.7

These provisions apply to solid bars of round or rectangular cross section and are taken from AISC (2016b). The nominal flexural resistance of these sections will typically be controlled by yielding, except for rectangular bars with a depth larger than the width, which may be controlled by lateral–torsional buckling. Since the shape factor for a rectangular cross section is 1.5 and for a round cross section is 1.7, the potential for excessive deflections or permanent deformations under service conditions should be considered.

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For rectangular bars in flexure about their minor geometric axis and for solid rounds, lateral–torsional buckling shall not be considered.

6.12.2.3—Composite Members

6.12.2.3.1—Concrete-Encased Shapes

For concrete-encased shapes that satisfy the provisions of Article 6.9.5.2.3, the nominal resistance of concrete-encased shapes subjected to flexure without compression shall be taken as the lesser of:

$$M_n = M_{ps}\text{, or} \tag{6.12.2.3.1-1}$$

$$M_n = M_{yc} \tag{6.12.2.3.1-2}$$

For the purpose of Article 6.9.2.2, the nominal flexural resistance of concrete-encased shapes subjected to compression and flexure shall be taken as:

- If $\dfrac{P_u}{\phi_c P_n} \geq 0.3$, then:

$$M_n = Z F_y + \frac{(d-2c)\, A_r F_{yr}}{3} + \left(\frac{d}{2} - \frac{A_w F_y}{1.7 f'_c b}\right) A_w F_y \tag{6.12.2.3.1-3}$$

- If $0.0 < \left(\dfrac{P_u}{\phi_c P_n}\right) < 0.3$, then:

  M_n shall be determined by a linear interpolation between the M_n value given by Eq. 6.12.2.3.1-1 or 6.12.2.3.1-2 at $P_u = 0$ and the M_n value given by Eq. 6.12.2.3.1-3 at $(P_u/\phi_c P_n) \geq 0.3$

where:

P_u = axial compressive force due to the factored loading (kip)
P_n = nominal compressive resistance specified in Article 6.9.5.1 (kip)
ϕ_c = resistance factor for axial compression specified in Article 6.5.4.2
M_{ps} = plastic moment of the steel section (kip-in.)
M_{yc} = yield moment of the composite section determined as specified in Article D6.2 (kip-in.)
Z = plastic section modulus of the steel section about the axis of bending (in.^{3})
A_w = web area of the steel section (in.^{2})
f'_c = specified minimum 28-day compressive strength of the concrete (ksi)
A_r = area of the longitudinal reinforcement (in.^{2})

C6.12.2.3.1

The behavior of the concrete-encased shapes and concrete-filled tubes covered in this Article is discussed extensively in Galambos (1998) and AISC (2016b). Such members are most often used as columns or beam columns. The provisions for circular concrete-filled tubes also apply to concrete-filled pipes.

The equation for M_n when $(P_u/\phi_c P_n) \geq 0.3$ is an approximate equation for the plastic moment resistance that combines the flexural strengths of the steel shape, the reinforcing bars, and the reinforced concrete. These resistances are defined in the first, second, and third terms of the equation respectively (Galambos, 1998). The equation has been verified by extensive tests (Galambos and Chapuis, 1980).

No test data are available on the loss of bond in composite beam columns. However, consideration of tensile cracking of concrete suggests $(P_u/\phi_c P_n) = 0.3$ as a conservative limit (AISC, 1999). It is assumed that when $(P_u/\phi_c P_n)$ is less than 0.3, the nominal flexural resistance is reduced below the plastic moment resistance of the composite section given by Eq. 6.12.2.3.1-3.

When there is no axial load, even with full encasement, it is assumed that the bond is only capable of developing the lesser of the plastic moment resistance of the steel section or the yield moment resistance of the composite section.

c = distance from the center of the longitudinal reinforcement to the nearest face of the member in the plane of bending (in.)
d = depth of the member in the plane of flexure (in.)
b = width of the member perpendicular to the plane of flexure (in.)
F_{yr} = specified minimum yield strength of the longitudinal reinforcement (ksi)

6.12.2.3.2—Concrete-Filled Tubes

The nominal flexural resistance of concrete-filled tubes that satisfy the limitations in Articles 6.9.5.2 may be taken as:

- If $\frac{D}{t} < 2.0\sqrt{\frac{E}{F_y}}$, then:

$$M_n = M_{ps} \quad (6.12.2.3.2\text{-}1)$$

- If $2.0\sqrt{\frac{E}{F_y}} < \frac{D}{t} \leq 8.8\sqrt{\frac{E}{F_y}}$, then:

$$M_n = M_{yc} \quad (6.12.2.3.2\text{-}2)$$

C6.12.2.3.2

Eqs. 6.12.2.3.2-1 and 6.12.2.3.2-2 represent a step function for nominal flexural resistance. No accepted transition equation is available at this writing.

6.12.2.3.3—Composite Concrete-Filled Steel Tubes (CFSTs)

The material-based nominal P-M interaction curve of circular CFSTs that satisfy the limitations in Article 6.9.6.2 shall be computed using one of the following methods:

- the plastic stress distribution method (PSDM), or
- the strain compatibility method (SCM).

The crack control provisions of Article 5.8.2.6 and the temperature and shrinkage reinforcement requirements of Article 5.10.6 shall not apply to CFSTs.

C6.12.2.3.3

The PSDM and SCM methods are permitted for evaluating the resistance of CFST members without consideration of buckling. The resistances obtained in experiments performed by a wide range of researchers on CFST members have been compared to the resistances predicted by the PSDM and SCM methods. Both methods provided conservative estimates of the resistance of CFST members. The comparisons were based upon the measured bending moment with a given applied axial load. The experiments showed that the CFST members developed 24 percent larger bending moment than predicted by the PSDM and 65 percent larger bending moment than predicted by the SCM. The standard deviation of the measured bending moment was 18 percent of the mean bending moment for the PSDM method and 114 percent of the mean bending moment for the SCM method. While the SCM has considerably more scatter in its predicted resistance, it provides estimates of curvature and nonlinear deformation; the PSDM does not provide any information on strain or deformation (Marson and Bruneau, 2004; Roeder, Lehman and Bishop, 2010). Research by Kenarangi and Bruneau (2017) has validated the PSDM method for various D/t ratios out to large displacements.

The PSDM is recognized in AISC (2016b). The method is illustrated in Figure C6.12.2.3.3-1. The method uses the full yield strength of the steel in tension and compression. Even under higher axial stresses, the full yield strength of the steel can be achieved because the

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concrete fill restrains local buckling of the steel. The compressive capacity of the concrete is approximated using a uniform concrete stress distribution with a magnitude of stress equal to $0.95f'_c$ over the entire compressive region. The coefficient of 0.95 on the concrete compressive strength is higher than the typical coefficient of 0.85 used for reinforced concrete flexural strength calculations in recognition of the increased confinement of the concrete and the resulting increased deformation capacity provided by the circular steel tube; comparison with test results indicate this method provides an accurate prediction of the flexural resistance. The axial load, P, and bending moment, M, are in equilibrium with the stress state and this neutral axis depth, with the resulting P and M values defining one point on the P-M interaction curve. Other points are defined for other neutral axis locations to fully establish the complete PSDM material-based P-M interaction curve.

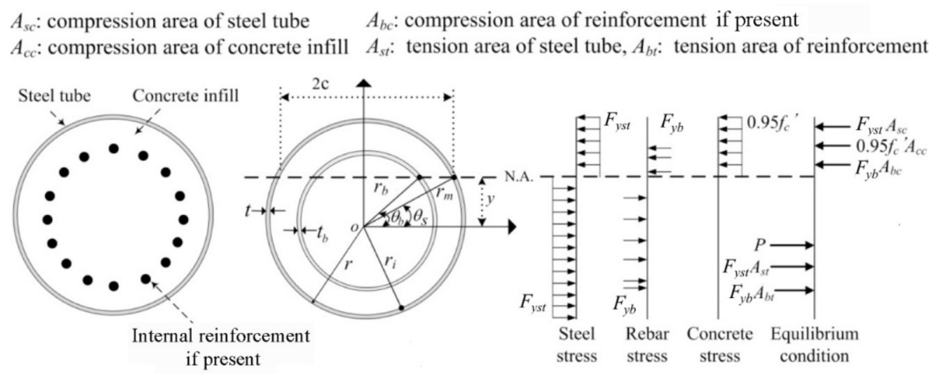

Figure C6.12.2.3.3-1—PSDM Model

For combined axial load and flexure, the PSDM is defined by using multiple assumed locations of the neutral axis to obtain a material-based P-M interaction curve, as illustrated in Figure C6.12.2.3.3-2. This interaction curve defines the material-based resistance of composite CFSTs that is not affected by buckling, secondary moments or P-δ effects.

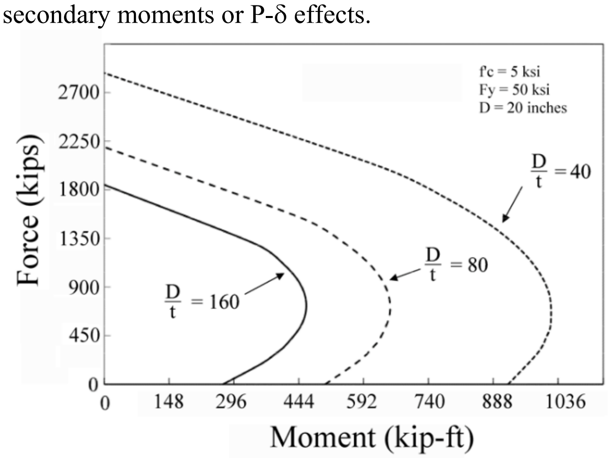

Figure C6.12.2.3.3-2—Material-based P-M Interaction Curves

While the PSDM is recognized in other design specifications, closed-form solutions of the complete material-based interaction curves are provided as follows for circular CFSTs with no internal reinforcement or with one radial row of internal reinforcement.

$$P_n = F_{yst} t r_m \left[(\pi - 2\theta_s) - (\pi + 2\theta_s) \right] + t_b r_b \left[F_{yb} (\pi - 2\theta_b) - (F_{yb} - 0.95 f'_c)(\pi + 2\theta_b) \right] + \frac{0.95 f'_c}{2} \left[(\pi - 2\theta_s) r_i^2 - 2yc \right]$$

(C6.12.2.3.3-1)

$$M_n = 0.95 f'_c c \left[(r_i^2 - y^2) - \frac{c^2}{3} \right] + 4 F_{yst} t c \frac{r_m^2}{r_i} + 4 F_{yb} t_b c_b r_b$$

(C6.12.2.3.3-2)

in which:

$$r_m = r - \frac{t}{2} \quad \text{(C6.12.2.3.3-3)}$$

$$\theta_s = \sin^{-1}\left(\frac{y}{r_m}\right) \quad \text{(C6.12.2.3.3-4)}$$

$$\theta_b = \sin^{-1}\left(\frac{y}{r_b}\right) \quad \text{(C6.12.2.3.3-5)}$$

$$c = r_i \cos\theta_s \quad \text{(C6.12.2.3.3-6)}$$

$$c_b = r_b \cos\theta_b \quad \text{(C6.12.2.3.3-7)}$$

$$t_b = \frac{nA_b}{2\pi r_b} \quad \text{(C6.12.2.3.3-8)}$$

where:

A_b = area of a single reinforcing bar (in.^{2})
c = one half the chord length for a given stress state, as shown in Figure C6.12.2.3.3-1 (in.)
c_b = one half the chord length for a given stress state of a fictional tube modeling the internal reinforcement (in.)
F_{yb} = specified minimum yield strength of the steel bars used for internal reinforcement (ksi)
F_{yst} = specified minimum yield strength of the steel tube (ksi)
f'_c = specified minimum 28-day compressive strength of the concrete (ksi)
M_n = nominal flexural resistance as a function of the nominal axial resistance, P_n (kip-in.)

n = number of uniformly spaced internal reinforcing bars, as shown in Figure C6.12.2.3.3-1

P_n = nominal compressive resistance of the member as function of the nominal flexural resistance, M_n (kips)

r = radius to the outside of the steel tube, as shown in Figure C6.12.2.3.3-1 (in.)

r_b = radius to the center of the internal reinforcing bars, as shown in Figure C6.12.2.3.3-1 (in.)

r_i = radius to the inside of the steel tube, as shown in Figure C6.12.2.3.3-1 (in.)

r_m = radius to the center of the steel tube, as shown in Figure C6.12.2.3.3-1 (in.)

t = thickness of steel tube, as shown in Figure C6.12.2.3.3-1 (in.)

t_b = thickness of a fictional steel tube used to model the contribution of the internal reinforcement, as shown in Figure C6.12.2.3.3-1 (in.)

y = distance from the center of the tube to neutral axis for a given stress state, as shown in Figure C6.12.2.3.3-1 (in.)

θ_b = angle used to define the length c_b for a given stress state (radians)

θ_s = angle used to define the length c for a given stress state (radians)

Smaller D/t values result in larger resistance because the area of steel is larger than a tube with the same diameter and smaller thickness. Larger D/t ratios result in significantly increased normalized flexural resistances for modest compressive loads relative to the flexural resistance corresponding to zero axial load because of the increased contribution of the concrete fill.

In the expressions, a positive value of P implies a compressive force, and y and θ are positive according to the sign convention shown in Figure C6.12.2.3.3-1. The material-based P-M interaction curve is generated by solving the equations for discrete values of y, and connecting those points where y varies between plus and minus r_i. For the case in which internal reinforcement is not present or is not considered, the variables A_b and t_b are equal to zero and several terms do not contribute to the resistance. For the case in which internal reinforcement is considered, θ_b shall be taken as positive $\pi/2$ if the ratio y/r_b is greater than one; θ_b shall be taken as negative $\pi/2$ if y/r_b is less than one. The composite plastic moment resistance without axial load, M_o, corresponds to the point on the material-based P-M interaction curve where P_n from Eq. C6.12.2.3.3-1 equals zero, which can be determined through linear interpolation.

For the cases where a column is integral with a CFST, moment transfer can be either by transferring tension between the reinforcing of the column that extends into the top of the CFST, and the reinforcing inside the CFST, or if the CFST is unreinforced, through the shear head behavior identified by Kenarangi and Bruneau (2017).

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6.12.3—Nominal Shear Resistance of Composite Members

6.12.3.1—Concrete-Encased Shapes

The nominal shear resistance may be taken as:

$$V_n = 0.58F_{yw}Dt_w + \frac{F_{yr}A_v(d-c)}{s} \quad (6.12.3.1\text{-}1)$$

where:

F_{yw} = specified minimum yield strength of the web of the steel shape (ksi)
F_{yr} = specified minimum yield strength of the transverse reinforcement (ksi)
D = web depth of the steel shape (in.)
t_w = thickness of the web or webs of the steel shape (in.)
A_v = cross-sectional area of transverse reinforcement bars that intercept a diagonal shear crack (in.^{2})
s = longitudinal spacing of transverse reinforcement (in.)
d = depth of the member in the plane of shear (in.)
c = distance from the center of the longitudinal reinforcement to the nearest face of the member in the plane of bending (in.)

6.12.3.2—Concrete-Filled Tubes

6.12.3.2.1—Rectangular Tubes

The nominal shear resistance may be taken as:

$$V_n = 1.16Dt_wF_y \quad (6.12.3.2.1\text{-}1)$$

where:

D = web depth of the tube (in.)
t_w = wall thickness of the tube (in.)

6.12.3.2.2—Composite Concrete-Filled Steel Tubes (CFSTs)

The nominal shear resistance of composite concrete filled steel tubes, both with and without internal reinforcement, shall be taken as:

$$V_n = V_s + V_c \quad (6.12.3.2.2\text{-}1)$$

in which:

$$V_s = \frac{2Dt}{\sqrt{3}}F_y \quad (6.12.3.2.2\text{-}2)$$

$$V_c = 0.0316\beta\sqrt{f_c'}A_c \quad (6.12.3.2.2\text{-}3)$$

$$\beta = 6.0 \quad (6.12.3.2.2\text{-}4)$$

C6.12.3.2.2

Previous editions of these specifications limited the shear resistance of composite circular steel tubes to that of the steel tube alone. Research by Roeder et al. (2016) and Kenarangi and Bruneau (2017) indicate the concrete fill adds substantially to the resistance. For very short shear spans, approaching the diameter of the steel tube, the shear behavior is governed by the formation of a strut in the infill concrete.

where:

V_s = nominal shear resistance provided by the steel tube (kips)
V_c = nominal shear resistance provided by the concrete infill (kips)
D = outer diameter of the steel tube (in.)
t = wall thickness of the steel tube (in.)
F_y = specified minimum yield strength of the steel tube (ksi)
f'_c = compressive strength of the concrete fill (ksi)
A_c = area of the concrete fill (in.^{2})

6.13—CONNECTIONS AND SPLICES

6.13.1—General

Except as specified herein, connections and splices for primary members subject only to axial tension or compression shall be designed at the strength limit state for not less than the larger of:

- the average of the factored axial force effect at the point of splice or connection and the factored axial resistance of the member or element at the same point, or
- 75 percent of the factored axial resistance of the member or element.

Connections and splices for primary members subjected to combined force effects, other than splices for flexural members, shall be designed at the strength limit state for the larger of:

- the calculated combined factored force effects, or
- 75 percent of the factored axial resistance of the member determined as specified in Articles 6.8.2 or 6.9.2, as applicable.

Bolted splices for flexural members shall be designed at the strength limit state as specified in Article 6.13.6.1.3. Welded splices for flexural members shall be designed at the strength limit state as specified in Article 6.13.6.2.

Where diaphragms, cross-frames, lateral bracing, stringers, or floorbeams for straight or horizontally-curved flexural members are included in the structural model used to determine force effects, or alternatively, are designed for explicitly calculated force effects from the results of a separate investigation, end connections for these bracing members shall be designed for the calculated factored member force effects. Otherwise, the end connections for these members shall be designed according to the 75 percent resistance provision contained herein.

Insofar as practicable, connections should be made symmetrical about the axis of the members. Connections, except for lacing bars and handrails, shall contain not

C6.13.1

For primary members subjected to force effects acting in multiple directions due to combined loading, such as members in rigid frames, arches, and trusses, a clarification of the design requirements is provided herein related to the determination of the factored resistance of the member. Connections and splices for such members are to be designed for the larger of the calculated combined factored force effects, preferably determined using concurrent forces or conservatively determined using envelope forces if only such forces are available, or 75 percent of the factored axial resistance of the member. The 75 percent resistance requirement is retained to provide a minimum level of stiffness and to be consistent with past practice for the design of connections and splices for axially loaded members.

The exception for bracing members in straight or horizontally curved girder bridges that are included in the structural model using to determine force effects is a result of previous experience with end connection details for these members that were developed invoking the 75 percent and average load provisions specified herein. These details tended to become so large as to be unwieldy resulting in large eccentricities and force concentrations. It has been decided that the negatives associated with these connections justified the exception permitted herein.

less than two bolts. Members, including bracing, should be connected so that their gravity axes will intersect at a point. Eccentric connections should be avoided. Where eccentric connections cannot be avoided, members and connections shall be proportioned for the combined effects of shear and moment due to the eccentricity.

In the case of connections that transfer total member end shear, the gross section shall be taken as the gross section of the connected elements.

The thickness of end connection angles of stringers, floorbeams and girders shall not be less than 0.375 in. End connections for stringers, floorbeams and girders should be made with two angles. Bracket or shelf angles that may be used to furnish support during erection shall not be considered in determining the number of fasteners required to transmit end shear.

Unless otherwise permitted by the contract documents, standard-size bolt holes shall be used in connections in horizontally-curved bridges.

Standard-size bolt holes in connections in horizontally-curved bridges ensure that the steel fits together in the field.

End connections of stringers, floorbeams, and girders should be bolted with high-strength bolts. Welded connections may be permitted when bolting is not practical. Where used, welded end connections shall be designed for vertical loads and the bending moment resulting from the restraint against end rotation.

Where timber stringers frame into steel floorbeams, shelf angles with stiffeners shall be provided to support the total reaction. Shelf angles shall not be less than 0.4375 in. thick.

6.13.2—Bolted Connections

6.13.2.1—General

Bolted steel parts may be coated or uncoated and shall fit solidly together after the bolts are tightened. The contract documents shall specify that all joint surfaces, including surfaces adjacent to the bolt head and nut, shall be specified to be free of scale, except tight mill scale, and free of dirt or other foreign material.

High-strength bolted joints shall be designated as either slip-critical or bearing-type connections. For slip-critical connections, the friction value shall be consistent with the specified condition of the faying surfaces as specified in Article 6.13.2.8. All material within the grip of the bolt shall be steel.

6.13.2.1.1—Slip-Critical Connections

Joints subject to stress reversal, heavy impact loads, severe vibration or located where stress and strain due to joint slippage would be detrimental to the serviceability of the structure shall be designated as slip-critical. They include:

- joints subject to fatigue loading;
- joints in shear with bolts installed in oversized holes;

C6.13.2.1.1

In bolted slip-critical connections subject to shear, the load is transferred between the connected parts by friction up to a certain level of force that is dependent upon the total clamping force on the faying surfaces and the coefficient of friction of the faying surfaces. The connectors are not subject to shear nor is the connected material subject to bearing stress. As loading is increased to a level in excess of the frictional resistance between the

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- joints in shear with bolts installed in short- and long-slotted holes where the force on the joint is in a direction other than perpendicular to the axis of the slot, except where the engineer intends otherwise and so indicates in the contract documents;
- joints subject to significant load reversal;
- joints in which welds and bolts share in transmitting load at a common faying surface;
- joints in axial tension or combined axial tension and shear;
- joints in axial compression only, with standard or slotted holes in only one ply of the connection with the direction of the load perpendicular to the direction of the slot, except for connections designed according to the provisions specified in Article 6.13.6.1.2; and
- joints in which, in the judgment of the Engineer, any slip would be critical to the performance of the joint or the structure and which are so designated in the contract documents.

faying surfaces, slip occurs, but failure in the sense of rupture does not occur. As a result, slip-critical connections are able to resist even greater loads by shear and bearing against the connected material. The strength of the connection is not related to the slip load. These Specifications require that the slip resistance and the shear and bearing resistance be computed separately. Because the combined effect of frictional resistance with shear or bearing has not been systematically studied and is uncertain, any potential greater resistance due to combined effect is ignored.

For slotted holes, perpendicular to the slot is defined as an angle between approximately 80 to 100 degrees to the axis of the slot.

Slip-critical connections shall be proportioned to prevent slip under Load Combination Service II, as specified in Table 3.4.1-1, and to provide bearing, shear, and tensile resistance at the applicable strength limit state load combinations. The provisions of Article 6.13.2.2 apply.

The intent of this provision is to control permanent deformations under overloads caused by slip in joints that could adversely affect the serviceability of the structure. The provisions are intended to apply to the design live load specified in Article 3.6.1.1. If this criterion were to be applied to a permit load situation, a reduction in the load factor for live load should be considered. Slip-critical connections must also be checked for the strength load combinations in Table 3.4.1-1, assuming that the connection has slipped at these high loads and gone into bearing against the connected material.

6.13.2.1.2—Bearing-Type Connections

Bearing-type connections shall be permitted only for joints subjected to axial compression or joints on bracing members and shall satisfy the factored resistance, R_r, at the strength limit state.

C6.13.2.1.2

In bolted bearing-type connections, the load is resisted by shear in the fastener and bearing upon the connected material, plus some uncertain amount of friction between the faying surfaces. The final failure will be by shear failure of the connectors, by tear out of the connected material, or by unacceptable ovalization of the holes. Final failure load is independent of the clamping force provided by the bolts (Kulak et al., 1987).

6.13.2.2—Factored Resistance

For slip-critical connections, the factored resistance, R_r, of a bolt at the Service II Load Combination shall be taken as:

$$R_r = R_n \quad (6.13.2.2\text{-}1)$$

where:

R_n = nominal resistance as specified in Article 6.13.2.8

C6.13.2.2

Eq. 6.13.2.2-1 applies to a service limit state for which the resistance factor is 1.0, and, hence, is not shown in the equation.

The factored resistance, R_r or T_r, of a bolted connection at the strength limit state shall be taken as either:

$$R_r = \phi R_n \qquad (6.13.2.2\text{-}2)$$

$$T_r = \phi T_n \qquad (6.13.2.2\text{-}3)$$

where:

R_n = nominal resistance of the bolt, connection, or connected material as follows:

- For bolts in shear, R_n shall be taken as specified in Article 6.13.2.7
- For the connected material in bearing joints, R_n shall be taken as specified in Article 6.13.2.9
- For connected material in tension or shear, R_n shall be taken as specified in Article 6.13.5

T_n = nominal resistance of a bolt as follows:

- For bolts in axial tension, T_n shall be taken as specified in Article 6.13.2.10
- For bolts in combined axial tension and shear, T_n shall be taken as specified in Article 6.13.2.11

ϕ = resistance factor for bolts specified in Article 6.5.4.2, taken as:

- ϕ_s for bolts in shear,
- ϕ_t for bolts in tension,
- ϕ_{bb} for bolts bearing on material,
- ϕ_y or ϕ_u for connected material in tension, as appropriate, or
- ϕ_v or ϕ_{vu} for connected material in shear.

6.13.2.3—Bolts, Nuts, and Washers

6.13.2.3.1—Bolts and Nuts

The provisions of Article 6.4.3 shall apply.

6.13.2.3.2—Washers

Hardened washers shall be required for connections using ASTM F3125 bolts as specified in Article 11.5.5.4.3 of the *AASHTO LRFD Bridge Construction Specifications*. Hardened washers used with ASTM F3125 bolts shall satisfy the requirements specified in Article 6.4.3.1.3.

Direct tension indicators shall not be installed over oversize or slotted holes in an outer ply, unless a hardened washer or a structural plate washer is also provided.

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6.13.2.4—Holes

6.13.2.4.1—Type

6.13.2.4.1a—General

Unless specified otherwise, standard holes shall be used in high-strength bolted connections.

6.13.2.4.1b—Oversize Holes

Oversize holes may be used in any or all plies of slip-critical connections. They shall not be used in bearing-type connections.

6.13.2.4.1c—Short-Slotted Holes

Short-slotted holes may be used in any or all plies of slip-critical or bearing-type connections. The slots may be used without regard to direction of loading in slip-critical connections, but the length shall be normal to the direction of the load in bearing-type connections.

6.13.2.4.1d—Long-Slotted Holes

Long-slotted holes may be used in only one ply of either a slip-critical or bearing-type connection. Long-slotted holes may be used without regard to direction of loading in slip-critical connections but shall be normal to the direction of load in bearing-type connections.

6.13.2.4.2—Size

The dimension of the holes shall not exceed the values given in Table 6.13.2.4.2-1.

Table 6.13.2.4.2-1—Maximum Hole Sizes

Bolt Dia.	Standard	Oversize	Short Slot	Long Slot
d	Dia.	Dia.	Width × Length	Width × Length
in.	in.	in.	in.	in.
$^5/_8$	$^{11}/_{16}$	$^{13}/_{16}$	$^{11}/_{16} \times {}^7/_8$	$^{11}/_{16} \times 1^9/_{16}$
$^3/_4$	$^{13}/_{16}$	$^{15}/_{16}$	$^{13}/_{16} \times 1$	$^{13}/_{16} \times 1^7/_8$
$^7/_8$	$^{15}/_{16}$	$1^1/_{16}$	$^{15}/_{16} \times 1^1/_8$	$^{15}/_{16} \times 2\ ^3/_{16}$
1	$1^1/_8$	$1^1/_4$	$1^1/_8 \times 1^5/_{16}$	$1^1/_8 \times 2^1/_2$
$\geq 1^1/_8$	$d+^1/_8$	$d+^5/_{16}$	$d+^1/_8 \times d+^3/_8$	$d+^1/_8 \times 2.5d$

6.13.2.5—Size of Bolts

Bolts shall not be less than 0.625 in. in diameter. Bolts 0.625 in. in diameter shall not be used in primary members, except in 2.5-in. legs of angles and in flanges of sections whose dimensions require 0.625-in. fasteners to satisfy other detailing provisions herein. Use of structural shapes that do not allow the use of 0.625-in. fasteners shall be limited to handrails.

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6.13.2.6—Spacing of Bolts

6.13.2.6.1—Minimum Spacing and Clear Distance

The minimum spacing between centers of bolts in standard holes shall be no less than three times the diameter of the bolt. When oversize or slotted holes are used, the minimum clear distance between the edges of adjacent bolt holes in the direction of the force and transverse to the direction of the force shall not be less than twice the diameter of the bolt.

C6.13.2.6.1

In uncoated weathering steel structures, pack-out is not expected to occur in joints where bolts satisfy the maximum spacing requirements specified in Article 6.13.2.6.2 (Brockenbrough, 1983).

6.13.2.6.2—Maximum Spacing for Sealing Bolts

For sealing against the penetration of moisture in joints, the spacing on a single line adjacent to a free edge of an outside plate or shape shall satisfy:

$$s \leq (4.0 + 4.0t) \leq 7.0 \qquad (6.13.2.6.2\text{-}1)$$

If there is a second line of fasteners uniformly staggered with those in the line adjacent to the free edge, at a gauge less than $1.5 + 4.0t$, the staggered spacing, s, in two such lines, considered together, shall satisfy:

$$s \leq 4.0 + 4.0t - \left(\frac{3.0g}{4.0}\right) \leq 7.0 \qquad (6.13.2.6.2\text{-}2)$$

The staggered spacing need not be less than one-half the requirement for a single line.

where:

t = thickness of the thinner outside plate or shape (in.)
g = gauge between bolts (in.)

6.13.2.6.3—Maximum Pitch for Stitch Bolts

Stitch bolts shall be used in mechanically fastened builtup members where two or more plates or shapes are in contact.

The pitch of stitch bolts in compression members shall not exceed $12.0t$. The gauge, g, between adjacent lines of bolts shall not exceed $24.0t$. The staggered pitch between two adjacent lines of staggered holes shall satisfy:

$$p \leq 15.0t - \left(\frac{3.0g}{8.0}\right) \leq 12.0t \qquad (6.13.2.6.3\text{-}1)$$

The pitch for tension members shall not exceed twice that specified herein for compression members. The gauge for tension members shall not exceed $24.0t$. The maximum pitch of fasteners in mechanically fastened builtup members shall not exceed the lesser of the requirements for sealing or stitch.

C6.13.2.6.3

The intent of this provision is to ensure that the parts act as a unit and, in compression members, prevent buckling.

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6.13.2.6.4—Maximum Pitch for Stitch Bolts at the End of Compression Members

The pitch of bolts connecting the component parts of a compression member shall not exceed four times the diameter of the fastener for a length equal to 1.5 times the maximum width of the member. Beyond this length, the pitch may be increased gradually over a length equal to 1.5 times the maximum width of the member until the maximum pitch specified in Article 6.13.2.6.3 is reached.

6.13.2.6.5—End Distance

The end distance for all types of holes measured from the center of the bolt shall not be less than the edge distances specified in Table 6.13.2.6.6-1. When oversize or slotted holes are used, the minimum clear end distance shall not be less than the bolt diameter.

The maximum end distance shall be the maximum edge distance as specified in Article 6.13.2.6.6.

C6.13.2.6.5

The minimum edge and end distance requirements specified in Table 6.13.2.6.6-1 are based on standard fabrication practices and workmanship tolerances, and are equivalent to the requirements specified in AISC (2016b). The provisions of Article 6.13.2.9 related to the bearing resistance of bolt holes must be satisfied for all bolt holes, and are used to ensure that sufficient end distances are provided such that bearing and tear-out limits are not exceeded for holes adjacent to all types of edges. It is recommended that edge and end distances larger than these specified minimum edge distances, but not exceeding the maximum edge and end distances specified herein, be permitted to help ensure that the specified minimum distances are not violated during fabrication after allowing for unavoidable workmanship tolerances.

6.13.2.6.6—Edge Distances

The minimum edge distance shall be as specified in Table 6.13.2.6.6-1.

The maximum edge distance shall not be more than eight times the thickness of the thinnest outside plate or 5.0 in.

Table 6.13.2.6.6-1—Minimum Edge Distances

Bolt Diameter, *d*	Minimum Edge Distance
in.	in.
5/8	7/8
¾	1
7/8	1 ⅛
1	1 ¼
1 ⅛	1 ½
1 ¼	1 5/8
Over 1 ¼	1 ¼ × *d*

6.13.2.7—Shear Resistance

The nominal shear resistance of a high-strength bolt (ASTM F3125) or an ASTM A307 bolt (Grade A or B) at

C6.13.2.7

The nominal resistance in shear is based upon the observation that the shear strength of a single high-the strength limit state in joints whose length between extreme fasteners measured parallel to the line of action of the force is less than or equal to 38.0 in. shall be taken as:

- Where threads are excluded from the shear plane:

$$R_n = 0.56 A_b F_{ub} N_s \quad (6.13.2.7\text{-}1)$$

- Where threads are included in the shear plane:

$$R_n = 0.45 A_b F_{ub} N_s \quad (6.13.2.7\text{-}2)$$

where:

A_b = area of the bolt corresponding to the nominal diameter (in.^{2})

F_{ub} = specified minimum tensile strength of the bolt specified in Article 6.4.3.1.1 (ksi)

N_s = number of shear planes per bolt

The nominal shear resistance of a bolt in lap splice tension connections greater than 38.0 in. in length shall be taken as 0.83 times the value given by Eq. 6.13.2.7-1 or 6.13.2.7-2.

If the threads of a bolt are included in the shear plane in the joint, the shear resistance of the bolt in all shear planes of the joint shall be the value for threads included in the shear plane.

For ASTM A307 bolts, shear design shall be based on Eq. 6.13.2.7-2. When the grip length of an ASTM A307 bolt exceeds 5.0 diameters, the nominal resistance shall be lowered one percent for each 1/16 in. of grip in excess of 5.0 diameters.

strength bolt is about 0.625 times the tensile strength of that bolt (AISC, 2016b). However, in lap splice tension connections with more than two bolts in the line of force, deformation of the connected material causes nonuniform bolt shear force distribution so that the strength of the connection in terms of the average bolt strength decreases as the joint length increases. Rather than provide a function that reflects this decrease in average fastener strength with joint length, a single reduction factor of 0.90 was applied to the 0.625 multiplier. This accommodates bolts in joints up to 38.0 in. in length without seriously affecting the economy of very short joints (Tide, 2010). For connections longer than 38.0 in., an overall reduction factor of 0.75 was found to be appropriate. Therefore, the nominal shear resistance of bolts in joints longer than 38.0 in. must be further reduced by an additional factor of 0.83 or 0.75/0.90. For flange splices, the 38.0-in. length is to be measured between the extreme bolts on only one side of the connection. The length reduction applies only to lap splice tension connections.

The average value of the nominal resistance for bolts with threads in the shear plane has been determined by a series of tests to be 0.833 ($0.6F_{ub}$), with a standard deviation of 0.03 (Yura et al., 1987). A value of about 0.80 was selected for the specification formula based upon the area corresponding to the nominal body area of the bolt.

The shear strength of bolts is not affected by pretension in the fasteners, provided that the connected material is in contact at the faying surfaces.

The factored resistance equals the nominal shear resistance multiplied by a resistance factor less than that used to determine the factored resistance of a component. This ensures that the maximum strength of the bridge is limited by the strength of the main members rather than by the connections.

The absence of design strength provisions specifically for the case where a bolt in double shear has a nonthreaded shank in one shear plane and a threaded section in the other shear plane is because of the uncertainty of manner of sharing the load between the two shear areas. It also recognizes that knowledge about the bolt placement, which might leave both shear planes in the threaded section, is not ordinarily available to the designer.

The threaded length of an ASTM A307 bolt is not as predictable as that of a high-strength bolt. The requirement to use Eq. 6.13.2.7-2 reflects that uncertainty.

ASTM A307 bolts with a long grip tend to bend, thus reducing their resistance.

6.13.2.8—Slip Resistance

The nominal slip resistance of a bolt in a slip-critical connection shall be taken as:

C6.13.2.8

Extensive data developed through research has been statistically analyzed to provide improved information on slip probability of connections in which the bolts have

$$R_n = K_h K_s N_s P_t \quad (6.13.2.8\text{-}1)$$

where:

N_s = number of slip planes per bolt
P_t = minimum required bolt tension specified in Table 6.13.2.8-1 (kip)
K_h = hole size factor specified in Table 6.13.2.8-2
K_s = surface condition factor specified in Table 6.13.2.8-3

Table 6.13.2.8-1—Minimum Required Bolt Tension, ASTM F3125 Bolts

Bolt Diameter, in.	Required Tension-P_t (kip)	
	Grade A325 or F1852	Grade A490 or F2280
5/8	19	24
¾	28	35
7/8	39	49
1	51	64
1 ⅛	64	80
1 ¼	81	102
1 3/8	97	121
1 ½	118	148

Table 6.13.2.8-2—Values of K_h

For standard holes	1.00
For oversize and short-slotted holes	0.85
For long-slotted holes with the slot perpendicular to the direction of the force	0.70
For long-slotted holes with the slot parallel to the direction of the force	0.60

Table 6.13.2.8-3—Values of K_s

For Class A surface conditions	0.30
For Class B surface conditions	0.50
For Class C surface conditions	0.30
For Class D surface conditions	0.45

The following descriptions of surface condition shall apply to Table 6.13.2.8-3:

- Class A Surface: unpainted clean mill scale, and blast-cleaned surfaces with Class A coatings;
- Class B Surface: unpainted blast-cleaned surfaces to SSPC-SP 6 or better, and blast-cleaned surfaces with

been preloaded to the requirements of Table 6.13.2.8-1. Two principal variables, bolt pretension and coefficient of friction, i.e., the surface condition factor of the faying surfaces, were found to have the greatest effect on the slip resistance of connections.

Hole size factors less than 1.0 are provided for bolts in oversize and slotted holes because of their effects on the induced tension in bolts using any of the specified installation methods. In the case of bolts in long-slotted holes, even though the slip load is the same for bolts loaded transverse or parallel to the axis of the slot, the values for bolts loaded parallel to the axis have been further reduced, based upon judgment, because of the greater consequences of slip.

The criteria for slip resistance are for the case of connections subject to a coaxial load. For cases in which the load tends to rotate the connection in the plane of the faying surface, a modified formula accounting for the placement of bolts relative to the center of rotation should be used (Kulak et al., 1987).

The minimum bolt tension values given in Table 6.13.2.8-1 are equal to 70 percent of the minimum tensile strength of the bolts.

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Class B coatings, or unsealed pure zinc or 85/15 zinc/aluminum thermal-sprayed coatings with a thickness less than or equal to 16 mils;

- Class C Surface: hot-dip galvanized surfaces; and
- Class D Surface: blast-cleaned surfaces with Class D coatings.

The contract documents shall specify that in uncoated joints, paint, including any inadvertent overspray, be excluded from areas closer than one bolt diameter but not less than 1.0 in. from the edge of any hole and all areas within the bolt pattern.

The contract documents shall specify the minimum required value of the surface condition factor.

The effect of ordinary paint coatings on limited portions of the contact area within joints and the effect of overspray over the total contact area have been investigated experimentally (Polyzois and Frank, 1986). The tests demonstrated that the effective area for transfer of shear by friction between contact surfaces was concentrated in an annular ring around and close to the bolts. Paint on the contact surfaces approximately 1.0 in., but not less than the bolt diameter away from the edge of the hole did not reduce the slip resistance. On the other hand, bolt pretension might not be adequate to completely flatten and pull thick material into tight contact around every bolt. Therefore, these Specifications require that all areas between bolts also be free of paint.

On clean mill scale, this research found that even the smallest amount of overspray of ordinary paint, i.e., a coating not qualified as Class A, within the specified paint-free area, reduced the slip resistance significantly. On blast-cleaned surfaces, the presence of a small amount of overspray was not as detrimental. For simplicity, these Specifications prohibit any overspray from areas required to be free of paint in slip-critical joints, regardless of whether the surface is clean mill scale or blast-cleaned.

The mean value of slip coefficients from many tests on clean mill scale, blast-cleaned steel surfaces, unsealed thermal sprayed surfaces, zinc-rich primers, and galvanized and roughened surfaces were taken as the basis for the four classes of surfaces. As a result of research by Frank and Yura (1981), a test method to determine the slip coefficient for coatings used in bolted joints was developed (RCSC, 2014). The method includes long-term creep test requirements to ensure reliable performance for qualified coatings. The method, which requires requalification if an essential variable is changed, is the sole basis for qualification of any coating to be used under these Specifications. In previous specifications, only three categories of surface conditions to be used in slip-critical joints were recognized: Class A for unpainted clean mill scale and for blast-cleaned surfaces with coatings that do not reduce the slip coefficient below that provided by clean mill scale, Class B for unpainted blast-cleaned surfaces and for coatings that do not reduce the slip coefficient below that of blast-cleaned steel surfaces, and Class C for hot-dipped galvanized surfaces. Further research has found that hot-dip galvanized surfaces and clean mill scale demonstrate the same slip coefficients (Donahue et al., 2014); however, to avoid confusion, the separate categories are maintained. Class D has been added to increase the options for organic zinc-rich coatings (Ocel et al., 2014). Unsealed pure zinc and 85/15 zinc/aluminum thermal-sprayed surfaces are classified as Class B surfaces since they comfortably satisfy the Class B requirements (Annan and Chiza, 2014; Ocel, 2014) when the thickness of the coating is less than or equal to 16 mils. Thermal-sprayed coatings can theoretically be applied to any thickness, although little to no slip performance data has been generated on coating thicknesses over 16 mils; hence, the restriction on coating thickness. Slip tests of some sealed thermal-sprayed coatings produced very low slip coefficients, or failed the creep requirement, and therefore are not included in the specification provisions. Metalized surfaces are often sealed with low viscosity epoxy coatings, e.g., penetrating sealers, to fill the pores of the metalizing and enhance their corrosion resistance. Any other faying surface preparation not described in the specification, e.g., sealed thermal-sprayed coatings, thermal-sprayed coatings thicker than 16 mils, zinc-rich primer over hot-dip galvanizing, etc., can be qualified to one of the surface condition classifications in accordance with Appendix A of RCSC (2014) and may be used subject to the approval of the Engineer.

Subject to the approval of the Engineer, coatings providing a surface condition factor less than 0.30 may be used, provided that the mean surface condition factor is established by test. The nominal slip resistance shall be determined as the nominal slip resistance for Class A surface conditions, as appropriate for the hole and bolt type, times the surface condition factor determined by test divided by 0.30.

To cover those cases where a coefficient of friction less than 0.30 might be adequate, the Specification provides that, subject to the approval of the Engineer, and provided that the mean slip coefficient is determined by the specified test procedure, faying surface coatings providing lower slip resistance than a Class A coating may be used. Blast cleaning for the purposes of this Article means the removal of all mill scale. The minimum SSPC blast cleaning standard that will remove all mill scale is SSPC-SP 6. Tightly adhering rust that forms after blast cleaning will not reduce the slip coefficient. Most high-performance coatings require blast cleaning to near-white metal, or SSPC-SP 10.

The contract documents shall specify that:

- Coated joints not be assembled before the coatings have cured for the minimum time used in the qualifying test, and
- Faying surfaces specified to be galvanized shall be hot-dip galvanized in accordance with AASHTO M 111M/M 111 (ASTM A123/A123M). No subsequent treatment of the galvanized surface is required.

If a slip-critical connection is subject to an applied tensile force that reduces the net clamping force, the nominal slip resistance shall be multiplied by the factor specified by Eq. 6.13.2.11-3.

The research cited above also investigated the effect of varying the time from coating the faying surfaces to assembly to ascertain if partially cured paint continued to cure. It was found that all curing ceased at the time the joint was assembled and tightened and that paint coatings that were not fully cured acted as lubricant. Thus, the slip resistance of the joint was severely reduced.

Research by Donahue et al. (2014) found that the slip coefficient of galvanized steel surfaces was sensitive to the silicon content of the steel. Structural steels with low silicon content had lower slip coefficients than steels with higher silicon content. The lower silicon steels produced a pure zinc layer with a lower slip coefficient. Treating the surface of the galvanized specimens did not improve the slip coefficient. The specified slip coefficient of 0.30 is a lower bound based upon tests of bridge steels without any surface treatment and with varying silicon contents after galvanizing.

Where hot-dip galvanized coatings are used, and especially if the joint consists of many plies of thickly coated material, relaxation of bolt tension may be significant and may require retensioning of the bolts subsequent to the initial tightening. This loss may be allowed for in design or pretension may be brought back

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to the prescribed level by a retightening of the bolts after an initial period of "settling-in."

While slip-critical connections with bolts pretensioned to the levels specified in Table 6.13.2.8-1 do not ordinarily slip into bearing when subject to anticipated loads, it is required that they meet the requirements of Article 6.13.2.7 and Article 6.13.2.9 in order to maintain a factor of safety of 2.0, if the bolts slip into bearing as a result of large, unforeseen loads.

6.13.2.9—Bearing Resistance at Bolt Holes

The effective bearing area of a bolt shall be taken as its diameter multiplied by the thickness of the connected material on which it bears. The effective thickness of connected material with countersunk holes shall be taken as the thickness of the connected material, minus one-half the depth of the countersink.

For standard holes, oversize holes, short-slotted holes loaded in any direction, and long-slotted holes parallel to the applied bearing force, the nominal resistance of interior and end bolt holes at the strength limit state, R_n, shall be taken as:

- With bolts spaced at a clear distance between holes not less than 2.0d and with a clear end distance not less than 2.0d:

$$R_n = 2.4dtF_u \quad (6.13.2.9\text{-}1)$$

- If either the clear distance between holes is less than 2.0d, or the clear end distance is less than 2.0d:

$$R_n = 1.2L_c tF_u \quad (6.13.2.9\text{-}2)$$

For long-slotted holes perpendicular to the applied bearing force:

- With bolts spaced at a clear distance between holes not less than 2.0d and with a clear end distance not less than 2.0d:

$$R_n = 2.0dtF_u \quad (6.13.2.9\text{-}3)$$

- If either the clear distance between holes is less than 2.0d, or the clear end distance is less than 2.0d:

$$R_n = L_c tF_u \quad (6.13.2.9\text{-}4)$$

where:

d = nominal diameter of the bolt (in.)
t = thickness of the connected material (in.)
F_u = tensile strength of the connected material specified in Table 6.4.1-1 (ksi)

C6.13.2.9

Bearing stress produced by a high-strength bolt pressing against the side of the hole in a connected part is important only as an index to behavior of the connected part. Thus, the same bearing resistance applies regardless of bolt shear strength or the presence or absence of threads in the bearing area. The critical value can be derived from the case of a single bolt at the end of a tension member.

Using finger-tight bolts, it has been shown that a connected plate will not fail by tearing through the free edge of the material if the distance L, measured parallel to the line of applied force from a single bolt to the free edge of the member toward which the force is directed, is not less than the diameter of the bolt multiplied by the ratio of the bearing stress to the tensile strength of the connected part (Kulak et al., 1987).

The criterion for nominal bearing strength is:

$$\frac{L}{d} \geq \frac{r_n}{F_u} \quad (C6.13.2.9\text{-}1)$$

where:

r_n = nominal bearing pressure (ksi)
F_u = specified minimum tensile strength of the connected part (ksi)

In these Specifications, the nominal bearing resistance of an interior hole is based on the clear distance between the hole and the adjacent hole in the direction of the bearing force. The nominal bearing resistance of an end hole is based on the clear distance between the hole and the end of the member. The nominal bearing resistance of the connected member may be taken as the sum of the smaller of the nominal shear resistance of the individual bolts and the nominal bearing resistance of the individual bolt holes parallel to the line of the force. The clear distance is used to simplify the computations for oversize and slotted holes.

Holes may be spaced at clear distances less than the specified values, as long as the lower value specified by Eq. 6.13.2.9-2 or Eq. 6.13.2.9-4, as applicable, is used for the nominal bearing resistance.

L_c = clear distance between holes or between the hole and the end of the member in the direction of the applied bearing force (in.)

If the nominal bearing resistance of a bolt hole exceeds the nominal shear resistance of the bolt determined as specified in Article 6.13.2.7, the nominal bearing resistance of the bolt hole shall be limited to the nominal shear resistance of the bolt.

6.13.2.10—Tensile Resistance

6.13.2.10.1—General

High-strength bolts subjected to axial tension shall be tensioned to the force specified in Table 6.13.2.8-1. The applied tensile force shall be taken as the force due to the external factored loadings, plus any tension resulting from prying action produced by deformation of the connected parts, as specified in Article 6.13.2.10.4.

6.13.2.10.2—Nominal Tensile Resistance

The nominal tensile resistance of a bolt, T_n, independent of any initial tightening force shall be taken as:

$$T_n = 0.76 A_b F_{ub} \quad (6.13.2.10.2\text{-}1)$$

where:

A_b = area of bolt corresponding to the nominal diameter (in.^{2})

F_{ub} = specified minimum tensile strength of the bolt specified in Article 6.4.3.1.1 (ksi)

C6.13.2.10.2

The recommended design strength is approximately equal to the initial tightening force; thus, when loaded to the service load, high-strength bolts will experience little, if any, actual change in stress. For this reason, bolts in connections, in which the applied loads subject the bolts to axial tension, are required to be fully tensioned.

6.13.2.10.3—Fatigue Resistance

Where high-strength bolts in axial tension are subject to fatigue, the stress range, Δf, in the bolt, due to the fatigue design live load, plus the dynamic load allowance for fatigue loading specified in Article 3.6.1.4, plus the prying force resulting from cyclic application of the fatigue load, shall satisfy Eq. 6.6.1.2.2-1.

The nominal diameter of the bolt shall be used in calculating the bolt stress range. In no case shall the calculated prying force exceed 30 percent of the externally applied load.

Low-carbon ASTM A307 bolts shall not be used in connections subjected to fatigue.

C6.13.2.10.3

Properly tightened ASTM F3125 bolts are not adversely affected by repeated application of the recommended service load tensile stress, provided that the fitting material is sufficiently stiff that the prying force is a relatively small part of the applied tension. The provisions covering bolt tensile fatigue are based upon study of test reports of bolts that were subjected to repeated tensile load to failure (Kulak et al., 1987).

6.13.2.10.4—Prying Action

The tensile force due to prying action shall be taken as:

$$Q_u = \left[\frac{3b}{8a} - \frac{t^3}{20}\right] P_u \quad (6.13.2.10.4\text{-}1)$$

C6.13.2.10.4

Eq. 6.13.2.10.4-1 for estimating the magnitude of the force due to prying is a simplification given in ASCE (1971) of a semiempirical expression (Douty and McGuire, 1965). This simplified formula tends to overestimate the prying force and provides conservative design results (Nair et al., 1974).

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where:

Q_u = prying tension per bolt due to the factored loadings taken as 0 when negative (kip)
P_u = direct tension per bolt due to the factored loadings (kip)
a = distance from center of bolt to edge of plate (in.)
b = distance from center of bolt to the toe of fillet of connected part (in.)
t = thickness of thinnest connected part (in.)

6.13.2.11—Combined Tension and Shear

The nominal tensile resistance of a bolt subjected to combined shear and axial tension, T_n, shall be taken as:

- If $\frac{P_u}{R_n} \leq 0.33$, then:

$$T_n = 0.76\, A_b F_{ub} \tag{6.13.2.11-1}$$

- Otherwise:

$$T_n = 0.76 A_b F_{ub} \sqrt{1 - \left(\frac{P_u}{\phi_s R_n}\right)^2} \tag{6.13.2.11-2}$$

where:

A_b = area of the bolt corresponding to the nominal diameter (in.^{2})
F_{ub} = specified minimum tensile strength of the bolt specified in Article 6.4.3.1.1 (ksi)
P_u = shear force on the bolt due to the factored loads (kip)
R_n = nominal shear resistance of a bolt specified in Article 6.13.2.7 (kip)

The nominal resistance of a bolt in slip-critical connections under Load Combination Service II, specified in Table 3.4.1-1, subjected to combined shear and axial tension, shall not exceed the nominal slip resistance specified in Article 6.13.2.8 multiplied by:

$$1 - \frac{T_u}{P_t} \tag{6.13.2.11-3}$$

where:

T_u = tensile force due to the factored loads under Load Combination Service II (kip)
P_t = minimum required bolt tension specified in Table 6.13.2.8-1 (kip)

C6.13.2.11

The nominal tensile resistance of bolts subject to combined axial tension and shear is provided by elliptical interaction curves, which account for the connection length effect on bolts loaded in shear, the ratio of shear strength to tension strength of threaded bolts, and the ratios of root area to nominal body area and tensile stress area to nominal body area (Chesson et al., 1965). Eqs. 6.13.2.11-1 and 6.13.2.11-2 are conservative simplifications of the set of elliptical curves. The equations representing the set of elliptical curves for various cases may be found in AISC (1988). No reduction in the nominal tensile resistance is required when the applied shear force on the bolt due to the factored loads is less than or equal to 33 percent of the nominal shear resistance of the bolt.

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6.13.2.12—Shear Resistance of Anchor Rods

The nominal shear resistance of an ASTM F1554 anchor rod at the strength limit state shall be taken as:

$$R_n = 0.50 A_b F_{ub} N_s \quad (6.13.2.12\text{-}1)$$

where:

A_b = area of the anchor rod corresponding to the nominal diameter (in.^{2})
F_{ub} = specified minimum tensile strength of the anchor rod (ksi)
N_s = number of shear planes per anchor rod

C6.13.2.12

Eq. 6.13.2.12-1 assumes threads are included in the shear plane since the thread length of anchor rods is not limited by the specification. See Article C6.13.2.7 for further commentary on Eq. 6.13.2.12-1. The joint length effect is not applicable to anchor rods.

For global design of anchorages to concrete, refer to *Building Code Requirements for Structural Concrete* (ACI 318-14).

6.13.3—Welded Connections

6.13.3.1—General

Base metal, weld metal, and welding design details shall conform to the requirements of the AASHTO/AWS D1.5M/D1.5 *Bridge Welding Code*. Welding symbols shall conform to those specified in AWS Publication A2.4.

Matching weld metal shall be used in groove and fillet welds, except that the Engineer may specify electrode classifications with strengths less than the base metal when detailing fillet welds, in which case the welding procedure and weld metal shall be selected to ensure sound welds.

C6.13.3.1

Matching weld metal has a specified minimum tensile strength that is the same as or higher than the lower-strength base metal. Matching strengths for various weld and base metal combinations are specified in the AASHTO/AWS D1.5M/D1.5 *Bridge Welding Code*.

Use of undermatched weld metal is highly encouraged for fillet welds connecting steels with specified minimum yield strength greater than 50 ksi. Research has shown that undermatched welds are much less sensitive to delayed hydrogen cracking and are more likely to produce sound welds on a consistent basis.

6.13.3.2—Factored Resistance

6.13.3.2.1—General

The factored resistance of welded connections, R_r, at the strength limit state shall be taken as specified in Articles 6.13.3.2.2 through 6.13.3.2.4.

The effective area of the weld shall be taken as specified in Article 6.13.3.3. The factored resistance of the connection material shall be taken as specified in Article 6.13.5.

C6.13.3.2.1

The factored resistance of a welded connection is governed by the resistance of the base metal or the deposited weld metal. The nominal resistance of fillet welds is determined from the effective throat area, whereas the nominal strength of the connected parts is governed by their respective thickness.

The classification strength of the weld metal, F_{exx}, in the articles that follow, is taken as the specified minimum tensile strength of the weld metal in ksi, which is reflected in the classification designation of the electrode. For example, the ′70′ in E70XX (SMAW), ER70S (solid-wire GMAW), and E70C (metal-cored GMAW); and the ′7′ in E71XX (FCAW) and F7XX (SAW) indicate a specified minimum tensile strength of 70.0 ksi.

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6.13.3.2.2—Complete Penetration Groove-Welded Connections

6.13.3.2.2a—Tension and Compression

The factored resistance of complete penetration groove-welded connections subjected to tension or compression normal to the effective area or parallel to the axis of the weld shall be taken as the factored resistance of the base metal.

C6.13.3.2.2a

In groove welds, the maximum forces are usually tension or compression. Tests have shown that groove welds of the same thickness as the connected parts are adequate to develop the factored resistance of the connected parts.

6.13.3.2.2b—Shear

The factored resistance of complete penetration groove-welded connections subjected to shear on the effective area shall be taken as the lesser of the value given by Eq. 6.13.3.2.2b-1 or 60 percent of the factored resistance of the base metal in tension:

$$R_r = 0.6\phi_{e1}F_{exx} \quad (6.13.3.2.2b\text{-}1)$$

where:

F_{exx} = classification strength of the weld metal (ksi)
ϕ_{e1} = resistance factor for the weld metal specified in Article 6.5.4.2

6.13.3.2.3—Partial Penetration Groove-Welded Connections

6.13.3.2.3a—Tension or Compression

The factored resistance of partial penetration groove-welded connections subjected to tension or compression parallel to the axis of the weld or compression normal to the effective area shall be taken as the factored resistance of the base metal.

The factored resistance for partial penetration groove-welded connections subjected to tension normal to the effective area shall be taken as the lesser of either the value given by either Eq. 6.13.3.2.3a-1 or the factored resistance of the base metal:

C6.13.3.2.3a

Eq. 6.6.1.2.5-4 should also be considered in the fatigue design of partial penetration groove-welded connections subject to tension.

$$R_r = 0.6\phi_{e1}F_{exx} \quad (6.13.3.2.3a\text{-}1)$$

where:

ϕ_{e1} = resistance factor for the weld metal specified in Article 6.5.4.2

6.13.3.2.3b—Shear

The factored resistance of partial penetration groove-welded connections subjected to shear parallel to the axis of the weld shall be taken as the lesser of either the factored nominal resistance of the connected material

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specified in Article 6.13.5 or the factored resistance of the weld metal taken as:

$$R_r = 0.6\phi_{e2}F_{exx} \tag{6.13.3.2.3b-1}$$

where:

ϕ_{e2} = resistance factor for the weld metal as specified in Article 6.5.4.2

6.13.3.2.4—Fillet-Welded Connections

The resistance of fillet welds which are made with matched or undermatched weld metal and which have typical weld profiles shall be taken as the smaller of the factored shear rupture resistance of the connected material adjacent to the weld leg determined as specified in Article 6.13.5.3, and the product of the effective area specified in Article 6.13.3.3 and the factored shear resistance of the weld metal taken as:

$$R_r = 0.6\phi_{e2}F_{exx} \tag{6.13.3.2.4-1}$$

C6.13.3.2.4

The factored resistance of fillet welds is based on the assumption that failure of such welds is by shear on the effective area whether the shear transfer is parallel or perpendicular to the axis of the line of the weld. In fact, the resistance is greater for shear transfer perpendicular to the weld axis; however, for simplicity the situations are treated the same. Thus, the factored resistance of fillet welds may be controlled by the shear resistance of the weld metal or by the shear rupture resistance of the connected material.

Shear yielding is not critical in welds because the material strain hardens without large overall deformations occurring. Therefore, the factored shear resistance of the weld metal is based on the shear resistance of the weld metal multiplied by a suitable resistance factor to ensure that the connected part will develop its full strength without premature failure of the weldment.

If fillet welds are subjected to eccentric loads that produce a combination of shear and bending stresses, they must be proportioned on the basis of a direct vector addition of the shear forces on the weld.

It is seldom that weld failure will ever occur at the weld leg in the base metal. The applicable effective area for the base metal is the weld leg which is 30 percent greater than the weld throat. If overstrength weld metal is used or the weld throat has excessive convexity, the capacity can be governed by the weld leg and the shear rupture resistance of the base metal.

6.13.3.3—Effective Area

The effective area shall be the effective weld length multiplied by the effective throat. The effective throat shall be the shortest distance from the joint root to the weld face.

C6.13.3.3

Additional requirements can be found in the AASHTO/AWS D1.5M/D1.5 *Bridge Welding Code*, Article 2.3.

6.13.3.4—Size of Fillet Welds

The size of a fillet weld that may be assumed in the design of a connection shall be such that the forces due to the factored loadings do not exceed the factored resistance of the connection specified in Article 6.13.3.2.

C6.13.3.4

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The maximum size of fillet weld that may be used along edges of connected parts shall be taken as:

- For material less than 0.25 in. thick: the thickness of the material, and
- For material 0.25 in. or more in thickness: 0.0625 in. less than the thickness of the material, unless the weld is designated on the contract documents to be built out to obtain full throat thickness.

The minimum size of fillet weld should be taken as specified in Table 6.13.3.4-1. The weld size need not exceed the thickness of the thinner part joined. Smaller fillet welds may be approved by the Engineer based upon applied stress and the use of the appropriate preheat.

Table 6.13.3.4-1—Minimum Size of Fillet Welds

Base Metal Thickness of Thicker Part Joined (T)	Minimum Size of Fillet Weld
in.	in.
$T \leq 3/4$	$1/4$
$3/4 < T$	$5/16$

The requirements for minimum size of fillet welds are based upon the quench effect of thick material on small welds, not on strength considerations. Very rapid cooling of weld metal may result in a loss of ductility. Further, the restraint to weld metal shrinkage provided by thick material may result in weld cracking. A 0.3125-in. fillet weld is the largest that can be deposited in a single pass by manual process, but minimum preheat and interpass temperatures are to be provided.

6.13.3.5—Minimum Effective Length of Fillet Welds

The minimum effective length of a fillet weld shall be four times its size and in no case less than 1.5 in.

6.13.3.6—Fillet Weld End Returns

Fillet welds that resist a tensile force not parallel to the axis of the weld or that are not proportioned to withstand repeated stress shall not terminate at corners of parts or members. Where such returns can be made in the same plane, they shall be returned continuously, full size, around the corner, for a length equal to twice the weld size. End returns shall be indicated in the contract documents.

Fillet welds deposited on the opposite sides of a common plane of contact between two parts shall be interrupted at a corner common to both welds.

C6.13.3.6

Fillet welds satisfying the provisions of Article 6.13.3.7 may be wrapped around the end of a transverse, longitudinal or bearing stiffener, connection plate, or a lap splice connection for sealing. The welds may also be wrapped around the inside of the cope of a stiffener or a connection plate, if desired.

6.13.3.7—Fillet Welds for Sealing

Seal welds should be continuous welds combining the functions of sealing and strength. The portion of a return sealing fillet weld around the ends of a transverse, longitudinal or bearing stiffener, connection plate, or a lap splice connection shall be exempt from the minimum size requirements specified in Article 6.13.3.4, and shall not be considered in determining the resistance of the connection.

C6.13.3.7

Fillet welds wrapping the ends of a transverse, longitudinal or bearing stiffener, connection plate, or a lap splice connection, are simply for sealing and are not to be included in the calculation of the resistance of the connection. The undercutting of the corner of the stiffener or connection plate, even when severe, does not reduce the fatigue performance of the weld, which is controlled by the toe of the transverse fillet weld connecting the stiffener or connection plate to the flange (Spadea and Frank, 2004).

The ends of fillet welded connections in galvanized structures, in particular, should be sealed to prevent the

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acids used to prepare the steel for galvanizing from being trapped in-between the components and then leaching out. Vent holes to allow the trapped air and moisture to escape and prevent destructive pressures from developing between the surfaces may be required if the overlap area exceeds 16.0 in.². Tables 1 and 2 in ASTM A385/A385M provide further guidance and provide the size of the vent holes needed when the overlap area exceeds 16.0 in.².

6.13.4—Block Shear Rupture Resistance

The web connection of coped beams and all tension connections, including connection plates, splice plates and gusset plates, shall be investigated to ensure that adequate connection material is provided to develop the factored resistance of the connection.

The connection shall be investigated by considering all possible failure planes in the member and connection plates. Such planes shall include those parallel and perpendicular to the applied forces. The planes parallel to the applied force shall be considered to resist only shear stresses. The planes perpendicular to the applied force shall be considered to resist only tension stresses.

The factored resistance of the combination of parallel and perpendicular planes shall be taken as:

$$R_r = \phi_{bs} R_p \left(0.58 F_u A_{vn} + U_{bs} F_u A_{tn}\right) \leq \phi_{bs} R_p \left(0.58 F_y A_{vg} + U_{bs} F_u A_{tn}\right) \tag{6.13.4-1}$$

where:

- R_p = reduction factor for holes taken equal to 0.90 for bolt holes punched full size and 1.0 for bolt holes drilled full size or subpunched and reamed to size
- A_{vg} = gross area along the plane resisting shear stress (in.^{2})
- A_{vn} = net area along the plane resisting shear stress (in.^{2})
- U_{bs} = reduction factor for block shear rupture resistance taken equal to 0.50 when the tension stress is non-uniform and 1.0 when the tension stress is uniform
- A_{tn} = net area along the plane resisting tension stress (in.^{2})
- F_y = specified minimum yield strength of the connected material (ksi)
- F_u = specified minimum tensile strength of the connected material specified in Table 6.4.1-1 (ksi)
- ϕ_{bs} = resistance factor for block shear specified in Article 6.5.4.2

The gross area shall be determined as the length of the plane multiplied by the thickness of the component. The net area shall be the gross area, minus the number of whole or fractional holes in the plane, multiplied by the

C6.13.4

Block shear rupture is one of several possible failure modes for splices, connections, and gusset plates. Investigation of other failure modes and critical sections is still required, e.g., a net section extending across the full plate width, and, therefore, having no parallel planes, may be a more severe requirement for a girder flange or splice plate than the block shear rupture mode. The provisions of Articles 6.13.5, 6.13.6, and 6.14.2.8 should be consulted.

Tests on coped beams have indicated that a tearing failure mode can occur along the perimeter of the bolt holes (Birkemoe and Gilmour, 1978). This block shear failure mode is one in which the resistance is determined by the sum of the nominal shear resistance on a failure path(s) and the nominal tensile resistance on a perpendicular segment. The failure path is defined by the centerlines of the bolt holes. The block shear rupture mode is not limited to the coped ends of beams. Tension member connections are also susceptible. The block shear rupture mode should also be checked around the periphery of welded connections.

A conservative model has been adopted to predict the block shear rupture resistance in which the resistance to rupture along the shear plane is added to the resistance to rupture on the tensile plane. Block shear is a rupture or tearing phenomenon and not a yielding phenomenon. However, gross yielding along the shear plane can occur when tearing on the tensile plane commences if $0.58F_uA_{vn}$ exceeds $0.58F_yA_{vg}$. Therefore, Eq. 6.13.4-1 limits the term $0.58F_uA_{vn}$ to not exceed $0.58F_yA_{vg}$. Eq. 6.13.4-1 is consistent with the philosophy for tension members where the gross area is used for yielding and the net area is used for rupture.

In certain cases, e.g., coped beam connections with multiple rows of bolts, the tensile stress on the end plane is nonuniform because the rows of bolts nearest the beam end pick up most of the shear (Ricles and Yura, 1983; Kulak and Grondin, 2001). Therefore, a reduction factor, U_{bs}, has been included in Eq. 6.13.4-1 to approximate the effect of the nonuniform stress distribution on the tensile plane in such cases. For the majority of connections encountered in steel bridges, U_{bs} will equal 1.0. The reduction factor, R_p, conservatively accounts for the reduced rupture resistance in the vicinity of bolt holes that are punched full size (Brown et al. 2007), as discussed further in Article C6.8.2.1.

nominal hole diameter specified in Table 6.13.2.4.2-1 times the thickness of the component.

In determining the net section of cuts carrying tension stress, the effect of staggered holes adjacent to the cuts shall be determined in accordance with Article 6.8.3. For net sections carrying shear stress, the full effective diameter of holes centered within two diameters of the cut shall be deducted. Holes further removed may be disregarded.

6.13.5—Connection Elements

6.13.5.1—General

This Article shall be applied to the design of connection elements such as splice plates, gusset plates, corner angles, brackets, and lateral connection plates in tension or shear, as applicable.

6.13.5.2—Tension

The factored resistance, R_r, in tension shall be taken as the least of the values given by either Eqs. 6.8.2.1-1 and 6.8.2.1-2 for yielding and fracture, respectively, or the block shear rupture resistance specified in Article 6.13.4.

In determining P_{nu}, as specified in Eq. 6.8.2.1-2, for lateral connection plates, splice plates, and gusset plates, the reduction factor, U, specified in Article 6.8.2.2, shall be taken to be equal to 1.0, and the net area of the plate, A_n, used in Eq. 6.8.2.1-2, shall not be taken as greater than 85 percent of the gross area of the plate.

C6.13.5.2

Because the length of the lateral connection plate, splice plate, or gusset plate is small compared to the member length, inelastic deformation of the gross section is limited. Hence, the net area of the connecting element is limited to $0.85A_g$ in recognition of the limited inelastic deformation and to provide a reserve capacity (Kulak et al., 1987).

6.13.5.3—Shear

The factored shear resistance, R_r, of the connection element shall be taken as the smaller value based on shear yielding or shear rupture.

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

$$R_r = \phi_v 0.58 F_y A_{vg} \qquad (6.13.5.3\text{-}1)$$

where:

A_{vg} = gross area of the connection element subject to shear (in.^{2})
F_y = specified minimum yield strength of the connection element (ksi)
ϕ_v = resistance factor for shear as specified in Article 6.5.4.2

For shear rupture, the factored shear resistance, R_r, of the connection element shall be taken as:

$$R_r = \phi_{vu} 0.58 R_p F_u A_{vn} \qquad (6.13.5.3\text{-}2)$$

where:

A_{vn} = net area of the connection element subject to shear (in.^{2})
F_u = tensile strength of the connection element (ksi)
R_p = reduction factor for holes taken equal to 0.90 for bolt holes punched full size and 1.0 for bolt holes drilled full size or subpunched and reamed to size
ϕ_{vu} = resistance factor for shear rupture of connection elements as specified in Article 6.5.4.2

6.13.6—Splices

6.13.6.1—Bolted Splices

6.13.6.1.1—Tension Members

Splices for tension members shall be designed at the strength limit state to satisfy the requirements specified in Article 6.13.1. Where a section changes at a splice, the smaller of the two connected sections shall be used in the design. Splices for tension members shall also satisfy the requirements specified in Article 6.13.5.2. Splices for tension members shall be designed using slip-critical connections as specified in Article 6.13.2.1.1.

6.13.6.1.2—Compression Members

Except as specified herein, splices for compression members shall be designed at the strength limit state to satisfy the requirements specified in Article 6.13.1. Where a section changes at a splice, the smaller of the two connected sections shall be used in the design. Splices for compression members detailed with milled ends in full contact bearing at the splices and for which the contract documents specify inspection during fabrication and erection, may be proportioned for not less than 50 percent of the lower factored resistance of the sections spliced.

Splices in truss chords, arch members, and columns should be located as near to the panel points as practicable and usually on that side where the smaller force effect occurs. The arrangement of plates, angles, or other splice elements shall be such as to make proper provision for all force effects in the component parts of the members spliced.

C6.13.6.1.2

This is consistent with the provisions of past editions of the AASHTO *Standard Specifications* (2002) which permitted up to 50 percent of the force in a compression member to be carried through a splice by bearing on milled ends of components.

6.13.6.1.3—Flexural Members

6.13.6.1.3a—General

In continuous spans, splices should be made at or near points of dead load contraflexure. Web and flange splices in areas of stress reversal shall be investigated for both positive and negative flexure.

Flange splices shall be designed as specified in Article 6.13.6.1.3b and web splices shall be designed as specified in Article 6.13.6.1.3c. In both web and flange

C6.13.6.1.3a

The method for the design of bolted splices for flexural members specified herein is based in principle on an alternative method first suggested by Sheikh-Ibrahim and Frank (1998, 2001) whereby all the factored moment is assumed to be resisted by the flange splices, provided the flange splices are capable of resisting the moment. Should the factored moment splices, there shall not be less than two rows of bolts on each side of the joint. Oversize or slotted holes shall not be used in either the member or the splice plates at bolted splices.

Bolted splices for flexural members shall be designed using slip-critical connections as specified in Article 6.13.2.1.1. The connections shall also be proportioned to prevent slip during the casting of the concrete deck.

The factored flexural resistance of the flanges at the point of splice at the strength limit state shall satisfy the applicable provisions of Articles 6.10.6.2 or 6.11.6.2.

exceed the moment resistance provided by the flange splices, the web splice is assumed to resist the additional moment in addition to its design shear. The primary difference in this approach versus previous approaches is that the moment due to the eccentricity of the shear is ignored in the method specified herein.

Bolted splices located in regions of stress reversal near points of dead load contraflexure must be checked for both positive and negative flexure to determine the governing condition.

To ensure proper alignment and stability of the girder during construction, web and flange splices are not to have less than two rows of bolts on each side of the joint. Also, oversize or slotted holes are not permitted in either the member or the splice plates at bolted splices of flexural members for improved geometry control during erection and because a strength reduction may occur when oversize or slotted holes are used in eccentrically loaded bolted web connections.

Also, for improved geometry control, bolted connections for both web and flange splices are to be proportioned to prevent slip under the maximum actions induced during the casting of the concrete deck.

Eq. 6.10.1.8-1 provides a limit on the maximum factored major-axis bending stress permitted on the gross section of the girder, neglecting the loss of area due to holes in the tension flange at the bolted splice. Eq. 6.10.1.8-1 will prevent a bolted splice from being located at a section where the factored flexural resistance of the section at the strength limit state exceeds the moment at first yield, M_y, unless the factored stress in the tension flange at that section is limited to the value given by the equation.

Since the combined areas of the flange and web splice plates will typically equal or exceed the areas of the smaller web and flange to which they are attached, and the flange and web are checked separately for either equivalent or more critical fatigue category details, fatigue of the base metal adjacent to the slip-critical connections in the splice plates will not control the design of the splice plates.

6.13.6.1.3b—Flange Splices

Flange splice plates and their connections shall be designed to develop the smaller design yield resistance of the flanges on either side of the splice. The design yield resistance of each flange, P_{fy}, at the point of splice shall be taken as:

$$P_{fy} = F_{yf} A_e \quad (6.13.6.1.3b\text{-}1)$$

in which:

A_e = effective area of the flange under consideration (in.^{2}). A_e shall be taken as:

C6.13.6.1.3b

In determining the factored shear resistance of the bolts, if the flange splice plate thickness closest to the nut is greater than or equal to 0.5-in. thick, the shear resistance of the bolts should be determined assuming the threads are excluded from the shear planes for bolts less than 1.0 in. in diameter. For bolts greater than or equal to 1.0 in. in diameter, the nominal shear resistance of the bolts should be determined assuming the threads are excluded from the shear planes if the flange splice plate thickness closest to the nut is greater than 0.75 in. in thickness. Otherwise, the threads should be assumed included in the shear planes. The preceding assumes there is one washer under the nut, and that there is no stick-out

$$A_e = \left(\frac{\phi_u F_u}{\phi_y F_{yf}} \right) A_n \leq A_g \qquad (6.13.6.1.3b\text{-}2)$$

where:

ϕ_u = resistance factor for fracture of tension members as specified in Article 6.5.4.2
ϕ_y = resistance factor for yielding of tension members as specified in Article 6.5.4.2
A_n = net area of the flange under consideration determined as specified in Article 6.8.3 (in.^{2})
A_g = gross area of the flange under consideration (in.^{2})
F_u = specified minimum tensile strength of the flange under consideration determined as specified in Table 6.4.1-1 (ksi)
F_{yf} = specified minimum yield strength of the flange under consideration (ksi)

For each flange, the smaller design yield resistance at the point of splice, P_{fy}, shall be divided by the factored shear resistance of the bolts, determined as specified in Article 6.13.2.2, to determine the total number of flange splice bolts required on one side of the splice at the strength limit state. Where filler plates are required, the provisions of Article 6.13.6.1.4 shall apply. The bearing resistance of the flange splice bolt holes shall also be checked at the strength limit state as specified in Article 6.13.2.9.

The moment resistance provided by the flanges at the point of splice shall be checked against the factored moment at the strength limit state. Should the factored moment exceed the moment resistance provided by the flanges, the additional moment shall be resisted by the web as specified in Article 6.13.6.1.3c. For composite sections subject to positive flexure, the moment resistance provided by the flanges at the strength limit state shall be computed as P_{fy} for the bottom flange times the moment arm taken as the vertical distance from the mid-thickness of the bottom flange to the mid-thickness of the concrete deck including the concrete haunch. For composite sections subject to negative flexure and noncomposite sections subject to positive or negative flexure, the moment resistance provided by the flanges shall be computed as P_{fy} for the top or bottom flange, whichever is smaller, times the moment arm taken as the vertical distance between the mid-thickness of the top and bottom flanges.

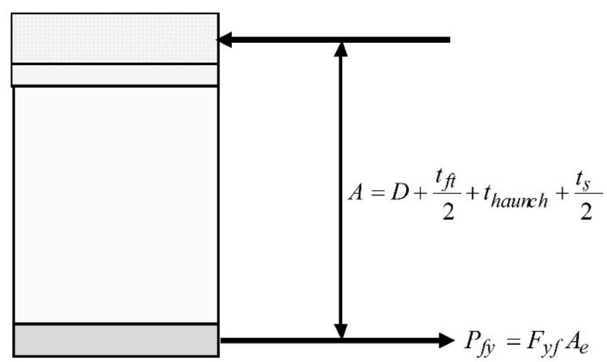
At the strength limit state, the design force in the splice plates shall not exceed the factored resistance in tension specified in Article 6.13.5.2.

The moment resistance provided by the nominal slip resistance of the flange splice bolts that are required to satisfy the strength limit state shall be checked against the factored moment for checking slip. The nominal slip resistance of the flange splice bolts shall be determined as specified in Article 6.13.2.8. Should the factored moment

beyond the nut, which represents the worst case for this determination.

The use of the effective flange area in the computation of P_{fy} accounts for the loss in section causing a reduction in the fracture resistance of the net section at the connection for loading conditions in which the flange is subject to tension. The effective flange area is conservatively used for both tension and compression flanges. The load-shedding factor, R_b, and the hybrid factor, R_h, are not included in the equation for P_{fy} since the effect of the load shedding due to local web buckling and due to early local yielding of the lower-strength web is considered in the design of the girder flanges at the point of splice thorough the application of R_b and R_h to reduce the nominal flexural resistance of the flanges below F_{yf}.

Figure C6.13.6.1.3b-1 illustrates the computation of the moment resistance provided by the flanges at the strength limit state, neglecting the contribution from the web and considering only the flange force, for composite sections subject to positive flexure at the strength limit state. The moment resistance is taken equal to P_{fy} for the bottom flange times the moment arm, A, where the thickness of the concrete haunch is assumed measured from the top of the web to the bottom of the concrete deck:



Moment resistance is equal to P_{fy} for the bottom flange times the moment arm, A.

Figure C6.13.6.1.3b-1—Calculation of the Moment Resistance Provided by the Flanges for Composite Sections Subject to Positive Flexure

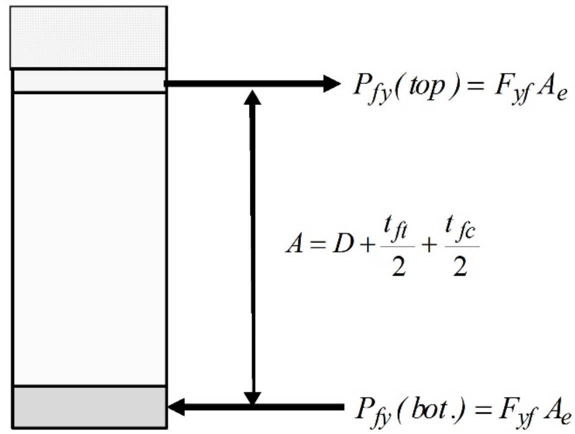
Figure C6.13.6.1.3b-2 illustrates the computation of the moment resistance provided by the flanges at the strength limit state for composite sections subject to negative flexure and noncomposite sections at the strength limit state taken equal to P_{fy} for the top or bottom flange, whichever is smaller, times the moment arm, A:

exceed the moment resistance provided by the nominal slip resistance of the flange splice bolts, the additional moment shall be resisted by the web as specified in Article 6.13.6.1.3c. The factored moments for checking slip shall be taken as the moment at the point of splice under Load Combination Service II, as specified in Table 3.4.1-1, and also the factored moment at the point of splice due to the deck casting sequence as specified in Article 3.4.2.1.

For the following box sections:

- single box sections in straight bridges;
- multiple box sections in straight bridges not satisfying the requirements of Article 6.11.2.3;
- single or multiple box sections in horizontally curved bridges; or
- single or multiple box sections with box flanges that are not fully effective according to the provisions of Article 6.11.1.1,

the vector sum of the St. Venant torsional shear in the bottom flange and P_{fy} shall be considered in the design of the bottom flange splice at the strength limit state. For checking slip, the St. Venant torsional shear shall be subtracted from the nominal slip resistance of the bottom flange splice bolts prior to computing the moment resistance.



Moment resistance is equal to $P_{fy(top)}$ or $P_{fy(bot.)}$, whichever is smaller, times the moment arm, A.

Figure C6.13.6.1.3b-2—Calculation of the Moment Resistance Provided by the Flanges for Composite Sections Subject to Negative Flexure and Noncomposite Sections

The moment resistance provided by the flanges can potentially be increased by staggering the flange bolts.

When checking for slip, the moment resistance provided by the nominal slip resistance of the flange splice bolts is calculated as shown in Figures C6.13.6.1.3b-1 and C6.13.6.1.3b-2, with the appropriate nominal slip resistance of the flange splice bolts substituted for P_{fy}. For checking slip due to the factored deck casting moment, the moment resistance of the noncomposite section is used.

Flange splice plates subjected to tension are to be checked for yielding on the gross section, fracture on the net section, and block shear rupture at the strength limit state according to the provisions of Article 6.13.5.2. Block shear rupture will usually not govern the design of splice plates of typical proportion. Flange splice plates subjected to compression at the strength limit state are to be checked only for yielding on the gross section of the plates. The factored yield resistance of splice plates in compression is the same as the factored yield resistance of splice plates in tension, and therefore, need not be checked. Buckling of splice plates in compression is not a concern since the unsupported length of the plates is limited by the maximum bolt spacing and end distance requirements.

For a flange splice with inner and outer splice plates, P_{fy} at the strength limit state may be assumed divided equally to the inner and outer plates and their connections when the areas of the inner and outer plates do not differ by more than ten percent. For this case, the connections are proportioned assuming double shear. Should the areas of the inner and outer plates differ by more than ten percent, the design force in each splice plate and its connection at the strength limit state should instead be determined by multiplying P_{fy} by the ratio of the area of the splice plate under consideration to the total area of the

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inner and outer splice plates. For this case, the connections are proportioned for the maximum calculated splice-plate force acting on a single shear plane. When checking for slip of the connection for a flange splice with inner and outer splice plates, the flange slip force is assumed divided equally to the two slip planes regardless of the ratio of the splice plate areas. Slip of the connection cannot occur unless slip occurs on both planes.

For the box sections cited in this Article, the vector sum of the St. Venant torsional shear in the bottom flange and P_{fy} is to be considered in the design of the bottom flange splice at the strength limit state. When checking for slip, the St. Venant torsional shear is conservatively subtracted from the nominal slip resistance of the bottom flange splice bolts prior to computing the moment resistance, rather than using the vector sum. St. Venant torsional shears and longitudinal warping stresses due to cross-section distortion are typically neglected in top flanges of tub-girder sections once the flanges are continuously braced. Longitudinal warping stresses due to cross-section distortion do not need to be considered in the design of the bottom flange splices at the strength limit state since the flange splices are designed to develop the full design yield resistance of the flanges. These stresses are typically relatively small in the bottom flange at the service limit state and for constructibility and may be neglected when checking the bottom flange splices for slip.

For flanges with one web in straight girders and in horizontally curved girders, the effects of flange lateral bending need not be considered in the design of the bolted flange splices since the combined areas of the flange splice plates will typically equal or exceed the area of the smaller flange to which they are attached. The flange is designed so that the yield stress of the flange is not exceeded at the flange tips under combined major-axis and lateral bending for constructibility and at the strength limit state. Flange lateral bending is also less critical at locations in-between the cross-frames or diaphragms where bolted splices are located. The rows of bolts provided in the flange splice on each side of the web provide the necessary couple to resist the lateral bending. Flange lateral bending will increase the flange slip force on one side of the splice and decrease the slip force on the other side of the splice; slip cannot occur unless it occurs on both sides of the splice.

6.13.6.1.3c—Web Splices

As a minimum, web splice plates and their connections shall be designed at the strength limit state for a design web force taken equal to the smaller factored shear resistance of the web, $V_r = \phi_v V_n$, on either side of the splice determined according to the provisions of Article 6.10.9 or 6.11.9, as applicable. The factored shear resistance of the web splice plates, V_r, at the strength limit

C6.13.6.1.3c

The factored shear resistance of the bolts should be based on threads included in the shear planes, unless the web splice-plate thickness exceeds 0.5 in. As a minimum, two vertical rows of bolts spaced at the maximum spacing for sealing bolts specified in Article 6.13.2.6.2 should be provided, with a closer spacing and/or additional rows provided only as needed.

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state shall not exceed the lesser of the factored shear resistances of the web splice plates determined as specified in Articles 6.13.4 and 6.13.5.3.

Should the moment resistance provided by the flanges at the point of splice, determined as specified in Article 6.13.6.1.3b, not be sufficient to resist the factored moment at the strength limit state, the web splice plates and their connections shall instead be designed for a design web force taken equal to the vector sum of the smaller factored shear resistance and a horizontal force in the web that provides the necessary moment resistance in conjunction with the flanges.

The horizontal force in the web shall be computed as the portion of the factored moment at the strength limit state at the point of splice that exceeds the moment resistance provided by the flanges divided by the appropriate moment arm. For composite sections subject to positive flexure, the moment arm shall be taken as the vertical distance from the mid-depth of the web to the mid-thickness of the concrete deck including the concrete haunch. For composite sections subject to negative flexure and noncomposite sections subject to positive or negative flexure, the moment arm shall be taken as one-quarter of the web depth.

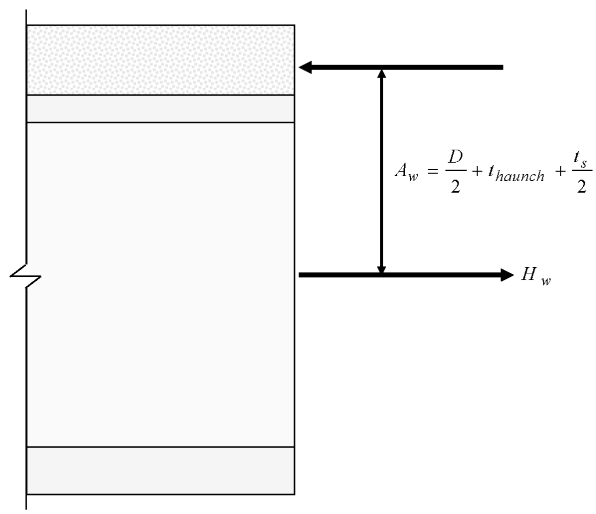
The computed design web force shall be divided by the factored shear resistance of the bolts, determined as specified in Article 6.13.2.2, to determine the total number of web splice bolts required on one side of the splice at the strength limit state. The bearing resistance of the web at bolt holes shall also be checked at the strength limit state as specified in Article 6.13.2.9.

As a minimum, bolted connections for web splices shall be checked for slip under a web slip force taken equal to the factored shear in the web at the point of splice. Should the moment resistance provided by the nominal slip resistance of the flange splice bolts, determined as specified in Article 6.13.6.1.3b, not be sufficient to resist the factored moment for checking slip at the point of splice, the web splice bolts shall instead be checked for slip under a web slip force taken equal to the vector sum of the factored shear and a horizontal force in the web that provides the necessary slip resistance in conjunction with the flange splices. The horizontal force in the web shall be computed as the portion of the factored moment for checking slip at the point of splice that exceeds the moment resistance provided by the nominal slip resistance of the flange splice bolts divided by the appropriate moment arm determined as specified herein. The factored shear for checking slip shall be taken as the shear in the web at the point of splice under Load Combination Service II, as specified in Table 3.4.1-1, or the factored shear in the web at the point of splice due to the deck casting sequence as specified in Article 3.4.2.1, whichever governs.

For the box sections specified in Article 6.13.6.1.3b, the shear for checking slip shall be taken as the sum of the factored flexural and St. Venant torsional shears in the web subjected to additive shears. For boxes with inclined

Since the web splice is being designed to develop the full factored shear resistance of the web as a minimum at the strength limit state and the eccentricity of the shear is small relative to the depth of the connection, the effect of the small moment introduced by the eccentricity of the web connection may be ignored at all limit states. Also, for the box sections specified in Article 6.13.6.1.3b, the effect of the additional St. Venant torsional shear in the web may be ignored at the strength limit state since the web splice is being designed as a minimum for the full factored shear resistance of the web.

Figure C6.13.6.1.3c-1 illustrates the computation of the horizontal force in the web, H_w, where necessary for composite sections subject to positive flexure. The web moment is taken as the portion of the factored moment that exceeds the moment resistance provided by the flanges. H_w is then taken as the web moment divided by the moment arm, A_w, taken from the mid-depth of the web to the mid-thickness of the concrete deck including the concrete haunch.



$$Web\ Moment = H_w A_w$$

$$H_w = \frac{Web\ Moment}{A_w}$$

A_w is measured from the mid-depth of the web to the mid-thickness of the deck.

Figure C6.13.6.1.3c-1—Calculation of the Horizontal Force in the Web, H_w, for Composite Sections Subject to Positive Flexure

Figure C6.13.6.1.3c-2 illustrates the computation of the horizontal force in the web, H_w, where necessary for composite sections subject to negative flexure and noncomposite sections. The web moment is again taken as the portion of the factored moment that exceeds the moment resistance provided by the flanges. In this case, however, H_w is taken as the web moment divided by $D/4$, as shown in Figure C6.13.6.1.3c-2.

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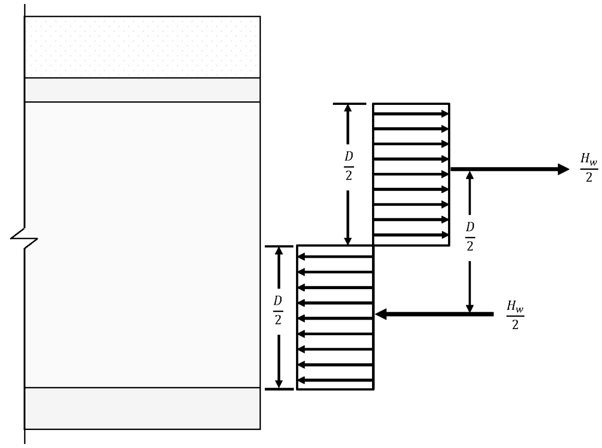
webs, the factored shear shall be taken as the component of the factored vertical shear in the plane of the web.

The computed web slip force shall be divided by the nominal slip resistance of the bolts, determined as specified in Article 6.13.2.8, to determine the total number of web splice bolts required on one side of the splice to resist slip.

Webs shall be spliced symmetrically by plates on each side. The splice plates shall extend as near as practical for the full depth between flanges without impinging on bolt assembly clearances or fillet areas on rolled beams. For bolted web splices with thickness differences of 0.0625 in. or less, filler plates should not be provided.



$$Web\ Moment = \frac{H_w}{2}\left(\frac{D}{2}\right)$$

$$H_w = \frac{Web\ Moment}{D/4}$$

Figure C6.13.6.1.3c-2—Calculation of the Horizontal Force in the Web, H_w, for Composite Sections Subject to Negative Flexure and Noncomposite Sections

The required moment resistance in the web for the case shown in Figure C6.13.6.1.3c-1 is provided by a horizontal tensile force, H_w, assumed acting at the mid-depth of the web that is equilibrated by an equal and opposite horizontal compressive force in the concrete deck. The required moment resistance in the web for the case shown in Figure C6.13.6.1.3c-2 is provided by two equal and opposite horizontal tensile and compressive forces, $H_w/2$, assumed acting at a distance $D/4$ above and below the mid-height of the web. In each case, there is no net horizontal force acting on the section.

Because the resultant web force is assumed divided equally to all of the bolts, the traditional vector analysis is not applied.

Since slip is a serviceability requirement, the effect of the additional St. Venant torsional shear in the web is to be considered for the box sections specified in Article 6.13.6.1.3b when checking for slip.

When a moment contribution from the web is required, the resultant forces causing bearing on the web bolt holes are inclined. The bearing resistance of each bolt hole in the web can conservatively be calculated in this case using the clear edge distance, as shown on the left of Figure C6.13.6.1.3c-3. This calculation is conservative since the resultant forces act in the direction of inclined distances that are larger than the clear edge distance. This calculation is also likely to be a conservative calculation for the bolt holes in the adjacent rows. Should the bearing resistance be exceeded, it is recommended that the edge distance be increased slightly in lieu of increasing the number of bolts or thickening the web. Other options would be to calculate the bearing resistance based on the inclined distance or to resolve the resultant force in the direction parallel to the edge distance, or to refine the calculation for the bolt holes in the adjacent rows. In cases where the bearing resistance of the web splice plates controls, the smaller of the clear edge or end distance on the splice plates can be used to compute the bearing resistance of each hole as shown on the right of Figure C6.13.6.1.3c-3.

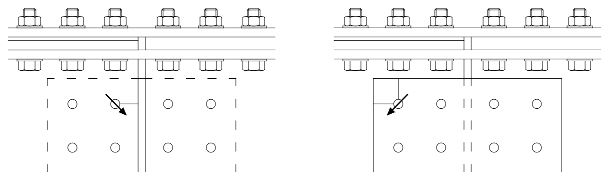


Figure C6.13.6.1.3c-3—Computing the Bearing Resistance of the Web Splice Bolt Holes for an Inclined Resultant Design Web Force

Required bolt assembly clearances are given in AISC (2017).

6.13.6.1.4—Fillers

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

- The fillers shall be extended beyond the gusset or splice material, and the filler extension shall be secured by enough additional bolts to distribute the total stress in the member uniformly over the combined section of the member and the filler or
- As an alternative, the fillers need not be extended and developed provided that the factored resistance of the bolts in shear at the strength limit state, specified in Article 6.13.2.2, is reduced by the following factor:

$$R = \left[\frac{(1+\gamma)}{(1+2\gamma)} \right] \qquad (6.13.6.1.4\text{-}1)$$

where:

γ = A_f/A_p

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

A_p = smaller of either the connected plate area on the side of the connection with the filler or the sum of the splice plate areas on both sides of the connected plate (in.^{2}); for truss gusset plate chord splices, when considering the gusset plate(s), only the portion of the gusset plate(s) that overlaps the connected plate shall be considered in the calculation of the splice plate areas

C6.13.6.1.4

Fillers are to be secured by means of additional fasteners so that the fillers are, in effect, an integral part of a shear-connected component at the strength limit state. The integral connection results in well-defined shear planes and no reduction in the factored shear resistance of the bolts.

In lieu of extending and developing the fillers, the reduction factor given by Eq. 6.13.6.1.4-1 may instead be applied to the factored resistance of the bolts in shear. This factor compensates for the reduction in the nominal shear resistance of a bolt caused by bending in the bolt and will typically result in the need to provide additional bolts in the connection. The reduction factor is only to be applied on the side of the connection with the fillers. The factor in Eq. 6.13.6.1.4-1 was developed mathematically (Sheikh-Ibrahim, 2002), and verified by comparison to the results from an experimental program on axially loaded bolted splice connections with undeveloped fillers (Yura, et al., 1982). The factor is more general than a similar factor given in AISC (2016b) in that it takes into account the areas of the main connected plate, splice plates and fillers and can be applied to fillers of any thickness. Unlike the empirical AISC factor, the factor given by Eq. 6.13.6.1.4-1 will typically be less than 1.0 for connections utilizing 0.25-in. thick fillers in order to ensure both adequate shear resistance and limited deformation of the connection.

For slip-critical connections, the nominal slip resistance of a bolt need not be adjusted for the effect of the fillers. The resistance to slip between filler and either connected part is comparable to that which would exist between the connected parts if fillers were not present.

A tolerance of up to 0.25 in. on the total filler thickness is permitted should fabrication or rolling

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For slip-critical connections, the nominal slip resistance of a bolt, specified in Article 6.13.2.8, shall not be adjusted for the effect of the fillers.

Fillers 0.25 in. or more in thickness shall consist of not more than two plates, unless approved by the Engineer. The actual total filler thickness may exceed the total filler thickness shown in the contract documents by up to a maximum of 0.25 in.

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

tolerances require the use of an additional filler not shown in the contract documents in order to adequately mate the fillers with the outer surface of the flange on the other side of the splice. A reduction in the specified filler thickness is permitted without restriction. Test results (Frank and Yura, 1981; Dusicka and Lewis, 2010 have shown that a 0.25-in-thick filler does not significantly reduce the resistance of the connection.

For fillers 0.25 in. or greater in thickness in axially loaded bolted connections, the specified minimum yield strength of the fillers should theoretically be greater than or equal to the specified minimum yield strength of the connected plate times the factor $[1/(1+\gamma)]$ in order to provide fully developed fillers that act integrally with the connected plate. However, such a requirement may not be practical or convenient due to material availability issues. As a result, premature yielding of the fillers, bolt bending and increased deformation of the connection may occur in some cases at the strength limit state. To control excessive deformation of the connection, a lower limit on the specified minimum yield strength of the filler plate material is recommended for fillers 0.25 in. or greater in thickness. Connections where the fillers are appropriately extended and developed or where additional bolts are provided according to Eq. 6.13.6.1.4-1 in lieu of extending the fillers, but that do not satisfy the recommended yield strength limit, will still have adequate reserve shear resistance in the connection bolts. However, such connections will have an increased probability of larger deformations at the strength limit state. For fillers less than 0.25 in. in thickness, the effects of yielding of the fillers and deformation of the connection are considered inconsequential. For applications involving the use of weathering steels, a weathering grade product should be specified for the filler plate material.

6.13.6.2—Welded Splices

Welded splice design and details shall conform to the requirements of the latest edition of AASHTO/AWS D1.5M/D1.5 *Bridge Welding Code* and the following provisions specified herein.

Welded splices for tension and compression members shall be designed to resist the design axial force specified in Article 6.13.1. Tension and compression members may be spliced by means of full penetration butt welds. Flexural members shall be spliced by means of full penetration butt welds. The use of splice plates should be avoided.

Welded field splices should be arranged to minimize overhead welding.

Material of different widths spliced by butt welds shall have symmetric transitions conforming to Figure 6.13.6.2-1. The type of transition selected shall be consistent with the detail categories of Table 6.6.1.2.3-1 for the groove-welded splice connection used in the design of the member. The contract documents shall specify that butt weld splices joining

C6.13.6.2

Flange width transition details typically show the transition starting at the butt splice. Figure 6.13.6.2-1 shows a preferred detail where the splice is located a minimum of 3.0 in. from the transition for ease in fitting runoff tabs. Where possible, constant width flanges are preferred in a shipping piece.

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material of different thicknesses be ground to a uniform slope between the offset surfaces, including the weld, of not more than one in 2.5.

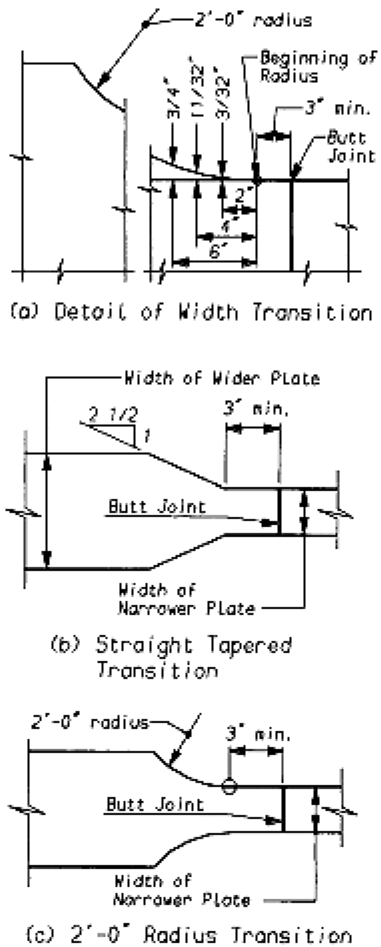

Figure 6.13.6.2-1—Splice Details

6.13.7—Rigid Frame Connections

6.13.7.1—General

All rigid frame connections shall be designed to resist the moments, shear, and axial forces due to the factored loading at the strength limit state.

C6.13.7.1

The provisions for rigid frame connections are well documented in Chapter 8 of ASCE (1971).

The rigidity is essential to the continuity assumed as the basis for design.

6.13.7.2—Webs

The thickness of an unstiffened beam web shall satisfy:

$$t_w \geq \sqrt{3}\left(\frac{M_c}{\phi_v F_y d_b d_c}\right) \qquad (6.13.7.2\text{-}1)$$

where:

C6.13.7.2

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F_y = specified minimum yield strength of the web (ksi)
M_c = column moment due to the factored loadings (kip-in.)
d_b = beam depth (in.)
d_c = column depth (in.)
ϕ_v = resistance factor for shear as specified in Article 6.5.4.2

When the thickness of the connection web is less than that given in Eq. 6.13.7.2-1, the web shall be strengthened by diagonal stiffeners or by a reinforcing plate in contact with the web over the connection area.

At knee joints where the flanges of one member are rigidly framed into the flange of another member, stiffeners shall be provided on the web of the second member opposite the compression flange of the first member where:

$$t_w < \frac{A_f}{t_b + 5k} \qquad (6.13.7.2\text{-}2)$$

and opposite the tension flange of the first member where:

$$t_c < 0.4\sqrt{A_f} \qquad (6.13.7.2\text{-}3)$$

where:

t_w = thickness of web to be stiffened (in.)
k = distance from outer face of flange to toe of web fillet of member to be stiffened (in.)
t_b = thickness of flange transmitting concentrated force (in.)
t_c = thickness of flange of member to be stiffened (in.)
A_f = area of flange transmitting concentrated load (in.^{2})

The provision for checking the beam or connection web ensures adequate strength and stiffness of the steel frame connection.

In bridge structures, diagonal stiffeners of minimum thickness will provide sufficient stiffness. Alternately, web thickness may be increased in the connection region.

The provisions for investigating a member subjected to concentrated forces applied to its flange by the flanges of another member framing into it are intended to prevent crippling of the web and distortions of the flange. It is conservative to provide stiffeners of a thickness equal to that of the flanges of the other member.

6.14—PROVISIONS FOR STRUCTURE TYPES

6.14.1—Through-Girder Spans

Where beams or girders comprise the main members of through-spans, such members shall be stiffened against lateral deformation by means of gusset plates or knee braces with solid webs connected to the stiffeners on the main members and the floorbeams. Design of gusset plates shall satisfy the requirements of Article 6.14.2.8.

C6.14.1

This requirement may be combined with other plate stiffening requirements.

6.14.2—Trusses

6.14.2.1—General

Trusses should have inclined end posts. Laterally unsupported hip joints shall be avoided.

Main trusses shall be spaced a sufficient distance apart, center-to-center, to prevent overturning.

Effective depths of the truss shall be assumed as follows:

- The distance between centers of gravity of bolted chords and
- The distance between centers of pins.

6.14.2.2—Truss Members

Members shall be symmetrical about the central plane of the truss.

If the shape of the truss permits, compression chords shall be continuous.

If web members are subject to reversal of stress, their end connections shall not be pinned.

Counters should be avoided.

C6.14.2.2

Chord and web truss members should usually be made of H-shaped, channel-shaped, or box-shaped members. The member or component thereof may be a rolled shape or a fabricated shape using welding or mechanical fasteners. Side plates or components should be solid. Cover plates or web plates may be solid or perforated.

In chords composed of angles in channel-shaped members, the vertical legs of the angles preferably should extend downward.

Counters are sometimes used as web members of light trusses.

Counters should be rigid. If used, adjustable counters should have open turnbuckles, and in the design of these members an allowance of 10.0 ksi shall be made for initial stress. Only one set of diagonals in any panel should be adjustable. Sleeve nuts and loop bars should not be used. The load factor for initial stress should be taken as 1.0.

6.14.2.3—Secondary Stresses

The design and details shall be such that secondary stresses will be as small as practicable. Stresses due to the dead load moment of the member shall be considered, as shall those caused by eccentricity of joints or working lines. Secondary stresses due to truss distortion or floorbeam deflection need not be considered in any member whose width measured parallel to the plane of distortion is less than one-tenth of its length.

6.14.2.4—Diaphragms

Diaphragms in truss members shall be provided according to the requirements specified in Article 6.7.4.5.

6.14.2.5—Camber

The length of the truss members shall be adjusted such that the camber will be equal to or greater than the deflection produced by the dead load.

The gross area of each truss member shall be used in computing deflections of trusses. If perforated plates are used, the effective area of the perforated plate shall be the net volume between centers of perforations divided by the length from center-to-center of perforations.

Design requirements for perforated plates shall satisfy the requirements specified in Articles 6.8.5.2 and 6.9.4.3.2.

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6.14.2.6—Working Lines and Gravity Axes

Main members shall be proportioned so that their gravity axes will be as nearly as practicable in the center of the section.

In compression members of an unsymmetrical section, such as chord sections formed of side segments and a cover plate, the gravity axis of the section shall coincide as nearly as practicable with the working line, except that eccentricity may be introduced to counteract dead load flexure. In two-angle bottom chord or diagonal members, the working line may be taken as the gauge line nearest the back of the angle or at the center of gravity for welded trusses.

6.14.2.7—Portal and Sway Bracing

6.14.2.7.1—General

The need for vertical cross-frames used as sway bracing in trusses shall be investigated. Any consistent structural analysis with or without intermediate sway bracing shall be acceptable as long as equilibrium, compatibility, and stability are satisfied for all applicable limit states.

6.14.2.7.2—Through-Truss Spans

Through-truss spans shall have portal bracing or the strength and stiffness of the truss system shall be shown to be adequate without a braced portal. If portal bracing is used, it should be of the two-plane or box-type, rigidly connected to the end post and the top chord flanges, and be as deep as the clearance will allow. If a single-plane portal is used, it should be located in the central transverse plane of the end posts, with diaphragms between the webs of the posts to provide for a distribution of the portal stresses.

The portal, with or without bracing, shall be designed to take the full reaction of the top chord lateral system, and the end posts shall be designed to transfer this reaction to the truss bearings.

6.14.2.7.3—Deck Truss Spans

Deck truss spans shall have sway bracing in the plane of the end posts, or the strength and stiffeners of the truss system shall be shown to be adequate. Where sway bracing is used, it shall extend the full depth of the trusses below the floor system, and the end sway bracing shall be proportioned to carry the entire upper lateral load to the supports through the end posts of the truss.

C6.14.2.7.3

Generally, full depth sway bracing is easily accommodated in deck trusses, and its use is encouraged.

6.14.2.8—Gusset Plates

6.14.2.8.1—General

Gusset or connection plates should be used for connecting truss members, except where the members are

C6.14.2.8.1

The provisions provided in this Article are intended for the design of double gusset-plate connections used pin-connected. The fasteners connecting each member shall be symmetrical with the axis of the member, so far as practicable, and the connection of all the elements of the member should be given consideration to facilitate the load transfer.

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

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

Bolted gusset plate connections shall satisfy the applicable requirements of Articles 6.13.1 and 6.13.2. Where filler plates are required, the provisions of Article 6.13.6.1.4 shall apply.

in trusses. The validity of the requirements for application to single gusset-plate connections has not been verified.

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

6.14.2.8.2—Multilayered Gusset and Splice Plates

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

C6.14.2.8.2

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

6.14.2.8.3—Shear Resistance

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

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

$$V_r = \phi_{vy} 0.58 F_y A_{vg} \Omega \tag{6.14.2.8.3-1}$$

where:

- ϕ_{vy} = resistance factor for truss gusset plate shear yielding specified in Article 6.5.4.2
- Ω = shear reduction factor for gusset plates taken as 0.88
- A_{vg} = gross area of the shear plane (in.^{2})
- F_y = specified minimum yield strength of the gusset plate (ksi)

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

C6.14.2.8.3

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

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

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

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

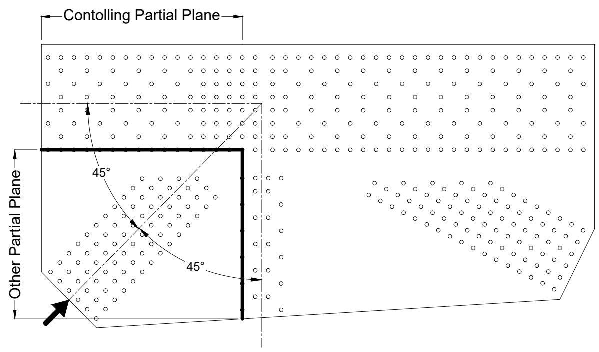

Figure 6.14.2.8.3-1—Example of a Controlling Partial Shear Plane that Parallels the Chamfered End of the Compression Member Since that Member Frames In at an Angle of 45 Degrees to Both the Chord and the Vertical

Research has shown that the buckling of connections with tightly spaced members is correlated with shear yielding around the compression members (Ocel, 2013). This is important because the buckling criteria used in Article 6.14.2.8.4 would overestimate the compressive buckling resistance of these types of connections. Once a plane yields in shear, the reduction in the plate modulus reduces the out-of-plane stiffness such that the stability of the plate is affected. Generally, truss verticals and chord members are not subject to the partial plane shear yielding check because there is no adjoining member fastener line that can yield in shear and cause the compression member to become unstable. For example, the two compression members shown in Figure C6.14.2.8.3-1 would not be subject to a partial plane shear check.

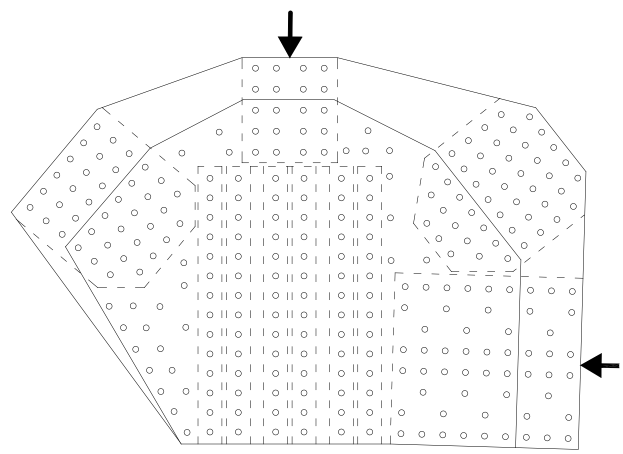

Figure C6.14.2.8.3-1—Example Showing Truss Vertical and Chord Members in Compression that Do Not Require a Partial Shear Plane Check

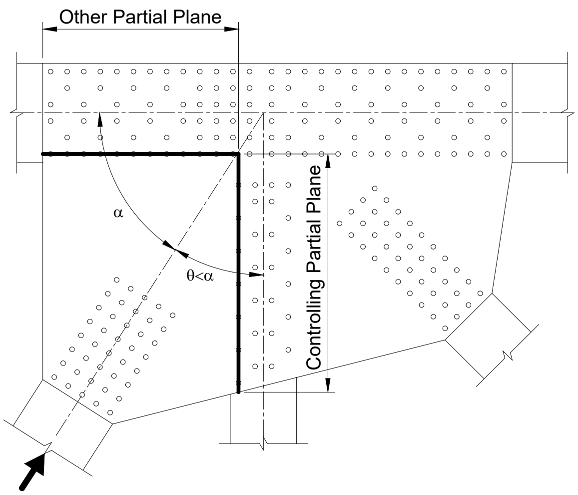

Figure 6.14.2.8.3-2—Example of a Controlling Partial Shear Plane on the Side of a Compression Member Without a Chamfered End that has the Smaller Framing Angle between that Member and the Other Adjoining Members (i.e., θ < α)

6.14.2.8.4—Compressive Resistance

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

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

where:

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

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

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

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

where:

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

L_{mid} = distance from the middle of the Whitmore section to the nearest member fastener line in the

C6.14.2.8.4

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

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

direction of the member, as shown in Figure 6.14.2.8.4-1 (in.)

t_g = gusset-plate thickness (in.)

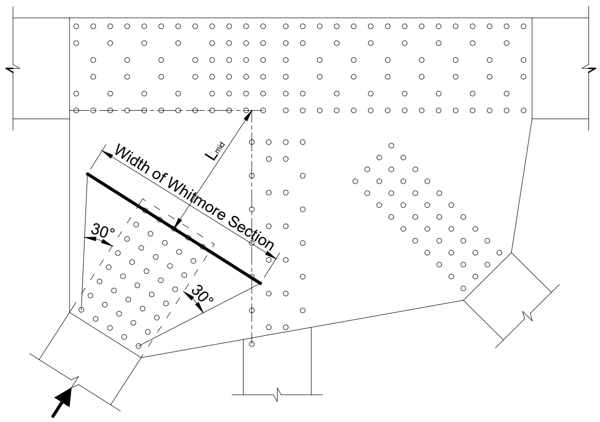

Figure 6.14.2.8.4-1—Example Connection Showing the Whitmore Section for a Compression Member Derived from 30 Degree Dispersion Angles and the Distance L_{mid}

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

6.14.2.8.5—Tensile Resistance

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

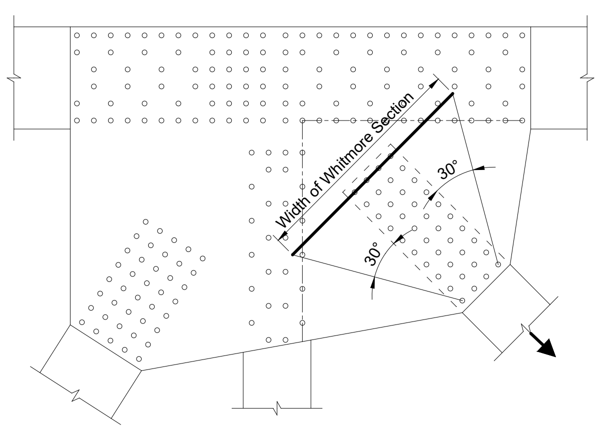

Figure 6.14.2.8.5-1—Example Connection Showing the Whitmore Section for a Tension Member Derived from 30 Degree Dispersion Angles

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

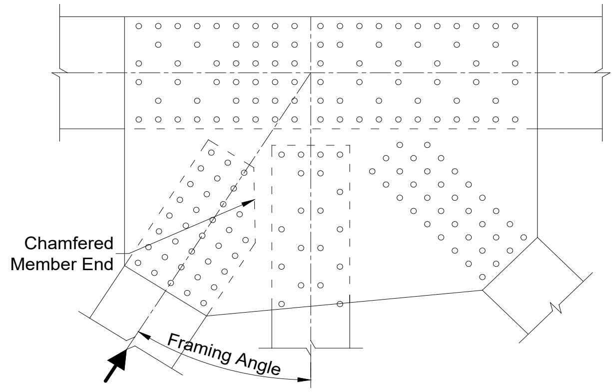

Figure C6.14.2.8.4-1—Example Connection Showing a Typical Chamfered Member End and Member Framing Angle

6.14.2.8.6—Chord Splices

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

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

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

in which:

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

$$\frac{KL_{splice}\sqrt{12}}{t_g} < 25 \quad (6.14.2.8.6\text{-}2)$$

where:

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

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

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

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

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

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

t_g = gusset plate thickness (in.)

C6.14.2.8.6

This Article is not intended to cover the design of chord splices that occur outside of the gusset plates; this situation is covered by the provisions of Article 6.13.6.1.1 or 6.13.6.1.2, as applicable. For gusset plates also serving the role of a chord splice, the forces from all members framing into the connection must be considered. The chord splice forces are the resolved axial forces acting on each side of the spliced section, as illustrated in Figure C6.14.2.8.6-1. Generally, a difference in the resolved forces on the two sides of a chord splice arises due to the use of envelope forces, i.e., forces due to noncurrent loads. Where envelope forces are used, the resolved forces should include consideration of the concurrence or nonconcurrence of forces to avoid potentially unconservative reductions in the connection force. The chord splice should be investigated for the larger of the two resolved forces on either side of the splice.

For chord splices that are in full compression under the loads being considered and that are detailed with milled ends in full contact bearing at the splice, resolved forces may be adjusted to account for the transfer of a portion of the compressive load through end bearing as long as the capacity in bearing is verified. The portion of the compressive load that may be transferred in bearing in such cases is specified in Article 6.13.6.1.2.

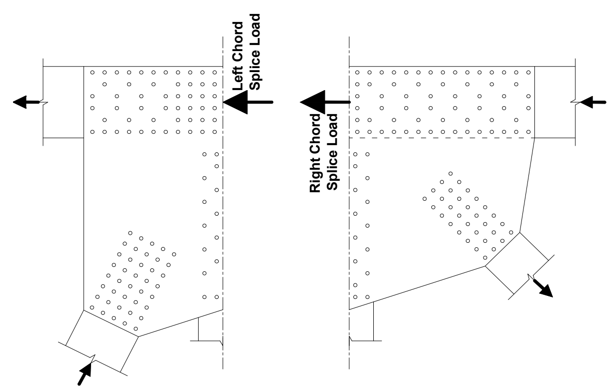

Figure C6.14.2.8.6-1—Example Connection Showing the Resolution of the Member Forces into Forces Acting on Each Side of a Chord Splice

The resistance equations in this Article assume the gusset and splice plates behave as one combined spliced section to resist the applied axial load and eccentric bending that occur due to the fact that the resultant forces on the section are offset from the centroid of the combined section, as illustrated in Figure C6.14.2.8.6-2. The combined spliced section is treated as a beam and the factored resistance at the strength limit state is determined assuming the stress in the combined section at the limit of usable resistance is equal to the specified minimum yield strength of the gusset plate if the slenderness limit for the spliced section given by Eq. 6.14.2.8.6-2 is met, which

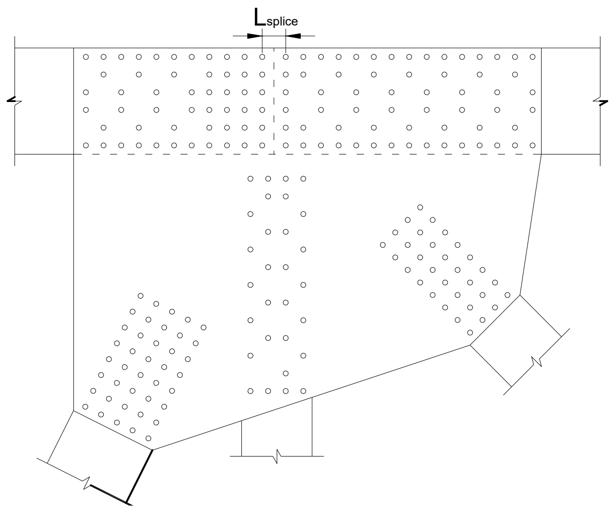

Figure 6.14.2.8.6-1—Example Connection Showing Chord Splice Parameter, L_{splice}

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

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

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

where:

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

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

A_n = net area of all plates in the cross section intersecting the spliced plane (in.^{2})

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

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

F_u = specified minimum tensile strength of the gusset plate (ksi)

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

S_n = net section modulus of all plates in the cross section intersecting the spliced plane (in.^{3})

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

will typically be the case. If not, the Engineer will need to derive a reduced value of F_{cr} to account for possible elastic buckling of the gusset plate within the splice.

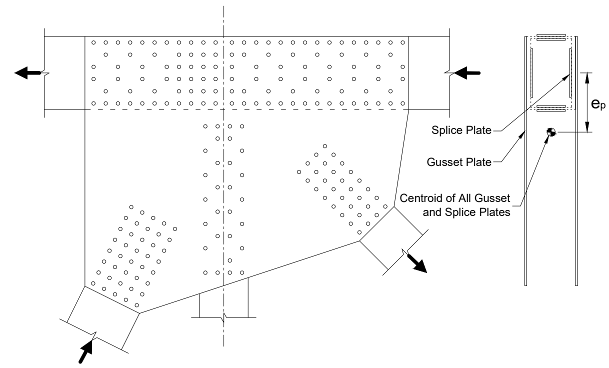

Figure C6.14.2.8.6-2—Illustration of the Combined Spliced Section at a Chord Splice

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

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

6.14.2.8.7—Edge Slenderness

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

C6.14.2.8.7

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

6.14.2.9—Half Through-Trusses

The vertical truss members and the floorbeams and their connections in half through-truss spans shall be proportioned to resist a lateral force of not less than 0.30 klf applied at the top chord panel points of each truss, considered as a permanent load for the Strength I Load Combination and factored accordingly.

The top chord shall be considered as a column with elastic lateral supports at the panel points.

C6.14.2.9

A discussion of the buckling analysis of columns with elastic lateral supports is contained in Timoshenko and Gere (1961) and in Ziemian (2010).

6.14.2.10—Factored Resistance

The factored resistance of tension members shall satisfy the requirements specified in Article 6.8.2.

The factored resistance of compression members shall satisfy the requirements specified in Article 6.9.2.

The nominal bending resistance of the members whose factored resistance is governed by interaction equations, specified in Articles 6.8.2.3 or 6.9.2.2, shall be evaluated as specified in Article 6.12.

6.14.3—Orthotropic Deck Superstructures

6.14.3.1—General

The provisions of this Article shall apply to the design of steel bridges that utilize a stiffened steel plate as a deck. An orthotropic deck shall be considered an integral part of the bridge superstructure and shall participate in resisting global force effects on the bridge. Connections between the deck and the main structural members shall be designed for interaction effects specified in Article 9.4.1.

The combined effects of global and local forces shall be considered when analyzing the orthotropic deck. The effect of torsional distortions of the cross-sectional shape shall be accounted for in analyzing the deck plate and girders of orthotropic box girder bridges.

C6.14.3.1

Orthotropic deck roadways may be used as upper or lower flanges of trusses, plate girder or box girder bridges, stiffening members of suspension or cable-stayed bridges, tension ties of arch bridges, etc.

Detailed provisions for the design of orthotropic decks are given in Article 9.8.3.

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6.14.3.2—Decks in Global Compression

6.14.3.2.1—General

The following potential stability-related behaviors shall be evaluated in the orthotropic plate: local buckling of the deck plate between ribs, local buckling of the rib wall, and buckling of the orthotropic panel between floorbeams.

6.14.3.2.2—Local Buckling

For local buckling, the slenderness of each component shall be considered: the rib spacing to deck thickness ratio, the closure spacing to deck thickness ratio (for closed ribs), and the rib height to rib thickness ratio for the ribs. The effective width for each component shall be determined in accordance with Article 6.9.4.2.

C6.14.3.2.2

Article 6.9.4 applies the recommended method for quantification of strength reduction resulting from local buckling, as given by *Specification for Structural Steel Buildings* for slender element cross sections (AISC, 2016b). This unified effective width method is based on Peköz (1986) and is also the basis of *North American Specification for the Design of Cold-Formed Steel Structural Members* (AISI, 2016).

For local buckling considerations, the deck plate and the rib walls can be considered to be plates supported along two longitudinal edges.

6.14.3.2.3—Panel Buckling

Each panel between floorbeams may be simplified for analysis as an isolated strut comprised of the rib and effective deck width specified in Figure 6.14.3.2.3-1.

C6.14.3.2.3

In-plane compressive resistance is controlled by stability. Buckling behavior of stiffened plate panels is a complicated problem due to the two-way orthogonal stiffening behavior and partially restrained boundary supports on four sides of the panel. A summary of relevant historical research on this subject is provided in Troitsky (1977) and Ziemian (2010). Similar to stiffened plate elements, reserve post-buckling strength in the panel exists beyond the point of initial buckling and can be quantified by use of the local effective width approach.

A simplified approach to estimating the buckling strength of the stiffened panel is to analyze the panel as a series of isolated column struts comprised of a stiffener and the associated effective width of plating (Horne and Narayanan, 1977). Then, basic column theory can be employed. This approach conservatively neglects the bending and membrane stiffness of the panel in the transverse direction and the torsional stiffness of the closed rib sections. Alternately, refined analysis may be used for a more accurate assessment of panel buckling strength with full consideration of the orthogonal stiffening behavior.

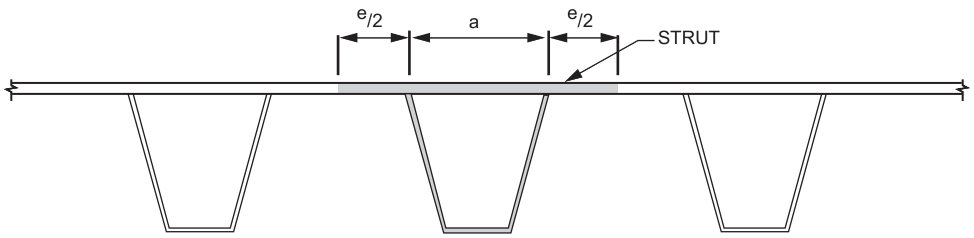

Figure 6.14.3.2.3-1—Idealized Strut for Evaluation of Compressive Resistance

where:

a = the width of the closed rib at the deck plate, centerline rib plate to centerline rib plate (in.)
e = the clear spacing between adjacent ribs, centerline rib plate to centerline rib plate (in.)

The critical buckling stress of the strut shall be determined in accordance with Article 6.9.4 or by methods of refined analysis as defined in Article 4.6.3.2.3.

6.14.3.3—Effective Width of Deck

The provisions of Article 4.6.2.6.4 shall apply.

6.14.3.4—Superposition of Global and Local Effects

In calculating extreme force effects in the deck, global and local effects shall be superimposed. Such combined force effects shall be computed for the same configuration and position of live load.

6.14.4—Solid Web Arches

6.14.4.1—General

These provisions are applicable for arch ribs satisfying the following limits:

- For flange extensions of box sections, flanges of I-sections, and/or web longitudinal stiffeners:

$$\frac{b}{t} \leq 0.12\frac{R}{b} \quad (6.14.4.1\text{-}1)$$

- For flanges of box sections:

$$\frac{b_{fi}}{t} \leq 0.47\frac{R}{b_{fi}} \quad (6.14.4.1\text{-}2)$$

where:

R = radius of curvature of the arch rib at the mid-depth of the web for the section under consideration (in.)

C6.14.4.1

The restrictions on arch rib proportions specified in this Article eliminate the need for the consideration of any reduction on the strength of arch rib plate elements due to the influence of the vertical curvature of the arch rib. The geometry of most arch ribs is such that no reduction is required for the influence of the vertical curvature on the strength of the flange plates and the web stiffener plates. White et al. (2019b) provide equations for a reduction in the effective yield strength of the arch rib component plates that extends the provisions of Article 6.14.4 to a broader range of geometries.

b_{fi} = clear width of the flange under consideration between the insides of the webs (in.)

b = unsupported width of the cross-section plate component under consideration (in.) taken as follows:

- For flange extensions of box sections:

  = clear projecting width of the flange under consideration measured from the outside surface of the web (in.)

- For I-section flanges:

  = one-half the total width of the flange (in.)

- For web longitudinal stiffeners:

  = projecting width of the longitudinal stiffener relative to the surface of the web (in.)

t = thickness of the cross-section plate component under consideration (in.)

Where longitudinal stiffeners are employed on flanges of arch ribs, transfer of the radial load from the axial force in the longitudinal stiffeners acting through the vertical curve to the webs of the arch rib shall be considered.

The use of longitudinal stiffeners on arch rib flanges is discouraged. The action of the axial force in flange longitudinal stiffeners acting through the vertical curvature of the arch rib induces significant radial forces from the longitudinal stiffeners, which must be transferred to the web or webs of the arch rib. In addition, for other than potentially free-standing arches, it is unlikely that the flanges of arch ribs would need to be wide enough to benefit from longitudinal stiffening of the flanges.

For unusual cases where the flanges in box-section arch ribs have longitudinal stiffeners, the radial load from the axial force in the longitudinal stiffeners acting through the vertical curve must be transferred from the longitudinal stiffeners to the webs of the arch rib, or longitudinally to transverse stiffening elements and then to the webs of the arch rib. For flanges with only one longitudinal stiffener and a wide spacing of transverse stiffeners and diaphragms, the likely predominant load path for this force transfer is via transverse bending of the flange plate. White et al. (2019b) provide further discussion of this consideration.

Web longitudinal stiffeners on arch ribs should be flat plates and shall satisfy the requirements of Article E6.1.3.

Tee or angle-section stiffeners on the webs of arch ribs also tend to exhibit significant bending in the direction normal to the stem or leg attached to the web, due to the axial force in the stiffener acting through the vertical curvature of the arch rib. This tends to cause twisting of the stiffener about the location of its connection to the stiffened plate, which exacerbates the behavior associated with the tripping limit state of the stiffener discussed in Article CE6.1.3.

If the requirements of Article 6.10.11.3 are not satisfied for longitudinally stiffened webs, the member flexural resistance should be calculated neglecting the longitudinal stiffeners when determining R_b in Article 6.10.1.10.2.

6.14.4.2—Web Slenderness

The web or webs of arch ribs shall satisfy the following in addition to the applicable web slenderness requirements from Articles 6.10.2.1 or 6.12.2.2.2b:

$$\frac{d_{fs}}{t_w} \leq \frac{0.40\dfrac{E}{F_{yf}}}{\sqrt{1+\dfrac{0.18}{(R/D)}\dfrac{E}{F_{yf}}}} \quad (6.14.4.2\text{-}1)$$

where:

d_{fs} = web depth for ribs with webs that are longitudinally unstiffened; maximum distance between the compression or the tension flange and the adjacent longitudinal stiffener for ribs with webs that are longitudinally stiffened (in.)
D = web depth (in.)
F_{yf} = specified minimum yield strength of the flange under consideration (ksi)
R = radius of curvature of the arch rib at the mid-depth of the web for the section under consideration (in.)
t_w = web thickness (in.)

C6.14.4.2

Eq. 6.14.4.2-1 is a not-to-exceed limit that ensures the web or webs of arch ribs are sufficiently stout such that they are capable of resisting transverse compression forces developed by the flanges acting through the vertical curve of the arch rib. This equation is adapted from the Eurocode Part 1-5 (CEN 2006) and AISC (2016b) equations for the limit state of "flange induced buckling," traditionally referred to in American structural engineering practice as flange vertical buckling. Eq. 6.14.4.2-1 reduces to the corresponding equation in AISC (2016b) in the limit that the radius of curvature of the arch rib, R, approaches infinity; this equation addresses the impact of the initial vertical curvature of the rib, in addition to the curvature due to bending, via the ratio R/D.

6.14.4.3—Moment Amplification

For moment amplification, the provisions specified in Article 4.5.3.2.2c shall be satisfied.

6.14.4.4—Nominal Compressive Resistance

The nominal compressive resistance of noncomposite arch ribs shall be determined using the provisions specified in Article 6.9.4.1.

In lieu of a more rigorous buckling analysis, the in-plane elastic critical flexural buckling resistance of arch ribs shall be calculated using the K values specified in Table 4.5.3.2.2c-1.

In lieu of a more rigorous buckling analysis, the out-of-plane elastic critical flexural buckling resistance shall be calculated as specified in Article 4.6.2.5. The characteristics of the framing in the out-of-plane direction shall be considered in determining the out-of-plane elastic critical buckling resistance using either approach.

6.14.4.5—Nominal Flexural Resistance

The nominal flexural resistance of noncomposite arch ribs shall be determined using the provisions of Article 6.10 or 6.12, as applicable. The developed unbraced length along the vertical curve between the brace points, L_{db}, shall be used for the unbraced length L_b.

C6.14.4.5

For unbraced lengths in vertically curved members such as arch ribs, the lateral–torsional buckling resistance of the member is reduced by the influence of the vertical curvature when the unbraced length is subjected to moments causing compression on the flange farthest from

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The reduction of the lateral–torsional buckling resistance due to the vertical curvature of the arch rib shall be considered. For box-section arch ribs with L_{db}/R greater than 0.20, subjected to bending moments causing compression on the flange farthest from the center of curvature of the rib, where R is the minimum radius of curvature of the arch rib measured to the mid-depth of the web, the moment gradient modifier, C_b, may be multiplied by 0.90 in lieu of a more refined buckling analysis.

the center of curvature; that is, moments that tend to "straighten" the arch. The lateral–torsional buckling resistance is increased by moments causing compression on the flange closest to the center of curvature; that is, moments that tend to increase the curvature of the arch. Increases in the lateral–torsional buckling resistance due to these effects should be neglected. For typical unbraced lengths of box-section arch ribs with L_{db}/R greater than 0.20, subjected to moments that would reduce the lateral–torsional buckling resistance, 0.90 is a reasonable lower-bound on the reduction in the elastic lateral–torsional buckling resistance. This reduction may be applied conservatively to the C_b modifier. The adjustment to the C_b modifier for arch ribs composed of doubly-symmetric open or closed sections may be determined more rigorously by a set of closed-form equations provided by Dowswell (2018). For arch ribs composed of singly-symmetric open or closed sections, the adjustment to the C_b modifier may be determined by solving equations provided by Trahair and Papangelis (1987).

6.14.4.6—Combined Axial Compression or Tension with Flexure and Torsion

The interaction between axial compression or tension resistances, flexural resistances, and flexural shear and/or torsion shall be considered as specified in Articles 6.9.2.2 and 6.8.2.3, as applicable.

6.15—PILES

6.15.1—General

Piles shall be designed as structural members capable of safely supporting all imposed loads.

For a pile group composed of only vertical piles which is subjected to lateral load, the pile structural analysis shall include explicit consideration of soil–structure interaction effects as specified in Article 10.7.3.9.

C6.15.1

Typically, due to the lack of a detailed soil–structure interaction analysis of pile groups containing both vertical and battered piles, evaluation of combined axial and flexural loading will only be applied to pile groups containing all vertical piles.

6.15.2—Structural Resistance

Resistance factors, ϕ_c and ϕ_f, for the strength limit state shall be taken as specified in Article 6.5.4.2. The resistance factors for axial resistance of piles in compression which are subject to damage due to driving shall be applied only to that section of the pile likely to experience damage. Therefore, the specified ϕ_c factors for axial resistance of 0.50 to 0.70 for piles in compression without bending shall be applied only to the axial capacity of the pile. The ϕ_c factors of 0.70 and 0.80 and the ϕ_f factor of 1.00 shall be applied to the combined axial and flexural resistance of the pile in the interaction equation for the compression and flexure terms, respectively.

C6.15.2

Due to the nature of pile driving, additional factors must be considered in selection of resistance factors that are not normally accounted for in steel members. The factors considered in development of the specified resistance factors include:

- Unintended eccentricity of applied load about pile axis,
- Variations in material properties of pile, and
- Pile damage due to driving.

These factors are discussed by Davisson et al. (1983). While the resistance factors specified herein generally conform to the recommendations given by Davisson et al.

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(1983), they have been modified to reflect current design philosophy.

The factored compressive resistance, P_r, includes reduction factors for unintended load eccentricity and material property variations as well as a reduction for potential damage to piles due to driving, which is most likely to occur near the tip of the pile. The resistance factors for computation of the factored axial pile capacity near the tip of the pile are 0.50 to 0.60 and 0.60 to 0.70 for severe and good driving conditions, respectively. These factors include a base axial compression resistance factor ϕ_c equal to 0.90, modified by reduction multipliers of 0.78 and 0.87 for eccentric loading of H-piles and pipe piles, respectively, and reduction multipliers of 0.75 and 0.875 for difficult and moderately difficult driving conditions.

For steel piles, flexure occurs primarily toward the head of the pile. This upper zone of the pile is less likely to experience damage due to driving. Therefore, relative to combined axial compression and flexure, the resistance factor for axial resistance range of ϕ_c = 0.70 to 0.80 accounts for both unintended load eccentricity and pile material property variations, whereas the resistance factor for flexural resistance of ϕ_f = 1.00 accounts only for base flexural resistance. This design approach is illustrated on Figure C6.15.2-1 which illustrates the depth to fixity as determined by P-Δ analysis.

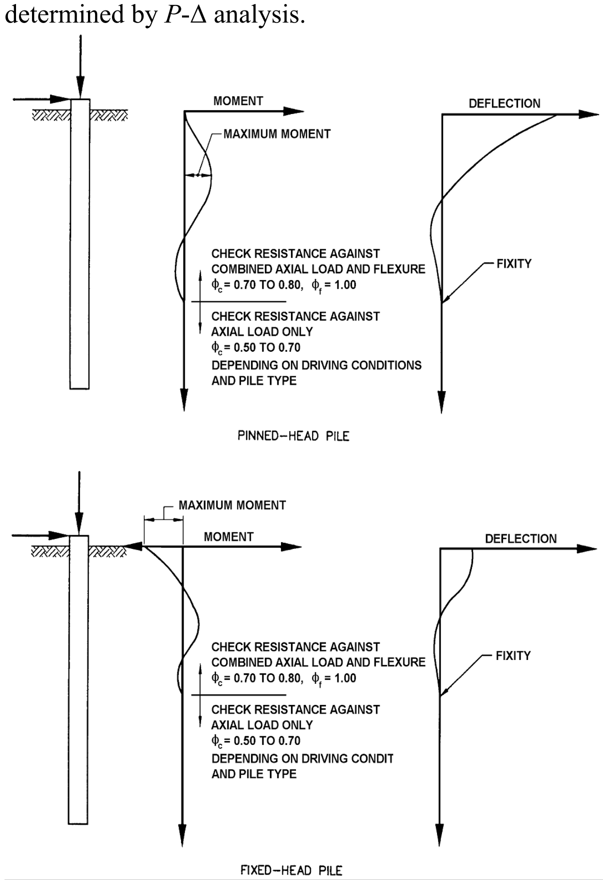

Figure C6.15.2-1—Distribution of Moment and Deflection in Vertical Piles Subjected to Lateral Load

If an unusual situation resulted in significant bending at the pile tip, possible pile damage should be considered in evaluating resistance to combined flexure and axial load.

6.15.3—Compressive Resistance

6.15.3.1—Axial Compression

For piles under axial load, the factored resistance of piles in compression, P_r, shall be taken as specified in Article 6.9.2.1 using the resistance factor, ϕ_c, specified in Article 6.5.4.2.

6.15.3.2—Combined Axial Compression and Flexure

Piles subjected to axial load and flexure shall be designed in accordance with Article 6.9.2.2 using the resistance factors, ϕ_c and ϕ_f, specified in Article 6.5.4.2.

6.15.3.3—Buckling

Instability of piles which extend through water or air shall be accounted for as specified in Article 6.9. Piles which extend through water or air shall be assumed to be fixed at some depth below the ground. Stability shall be determined in accordance with provisions in Article 6.9 for compression members using an equivalent length of the pile equal to the laterally unsupported length, plus an embedded depth to fixity. The depth to fixity shall be determined in accordance with Article 10.7.3.13.4 for battered piles or P-Δ analysis for vertical piles.

C6.15.3.3

An approximate method acceptable to the Engineer may be used in lieu of a P-Δ analysis.

6.15.4—Maximum Permissible Driving Stresses

Maximum permissible driving stresses for top driven steel piles shall be taken as specified in Article 10.7.8.

6.16—PROVISIONS FOR SEISMIC DESIGN

6.16.1—General

The provisions of Article 6.16 shall apply only to the design of slab-on-steel-girder bridge superstructures at the extreme event limit state.

In addition to the requirements specified herein, minimum support length requirements specified in Article 4.7.4.4 shall also apply.

A clear seismic load path shall be established within the superstructure to transmit the inertia forces to the substructure based on the stiffness characteristics of the concrete deck, cross-frames or diaphragms, and bearings. The flow of the seismic forces shall be accommodated through all affected components and connections of the steel superstructure within the prescribed load path,

C6.16.1

These Specifications are based on the work published by Itani et al. (2010, NCHRP (2002, 2006, MCEER/ATC (2003), Caltrans (2006), AASHTO (2011), and AISC (2016a and 2016b). The Loma Prieta earthquake of 1989, Petrolia earthquakes of 1992, Northridge earthquake of 1994, and the Hyogoken-Nanbu (Kobe earthquake of 1995 provided new insights into the behavior of steel details under seismic loads. The Federal Highway Administration, Caltrans, and the American Iron and Steel Institute initiated a number of research projects that have produced information that is useful for both the design of new steel-girder structures and the retrofitting of existing steel-girder structures.

including, but not limited to, the longitudinal girders, cross-frames or diaphragms, steel-to-steel connections, deck-to-steel interface, bearings, and anchor rods.

This new information relates to all facets of seismic engineering, including design spectra, analytical techniques, and design details. Bridge designers working in Seismic Zones 2, 3, or 4 are encouraged to avail themselves of current research reports and other literature to augment these Specifications.

Steel-girder bridges are generally considered to perform well in earthquakes. However, the aforementioned earthquakes showed the vulnerability of steel-girder bridges to damage if they are not designed and detailed to resist the seismic motions (Roberts, 1992; Astaneh-Asl et al., 1994; Itani and Reno, 1995; Bruneau et al., 1996; and Carden et al., 2005a). Typical damage included unseated longitudinal girders and failure of cross-frames and their connections, expansion joints, and bearings. In a few cases, most notably during the Kobe earthquake, major gravity load-carrying members failed, triggered in some instances by the failure of components elsewhere in the superstructure. These earthquakes confirmed the vulnerability of steel-girder bridges during seismic events. New areas of concern that emerged included:

- Lack of understanding of the seismic load paths in steel-girder bridges;
- Damage to steel superstructure components, e.g., girders, shear connectors, end cross-frames, bearing stiffeners, bearings, and anchor rods; and
- Failure of steel substructures.

Seismic design specifications in the U.S. currently do not require the explicit design of bridge superstructures for seismic loads. The assumption is made that a superstructure that is designed for out-of-plane gravity load has sufficient strength, by default, to resist in-plane seismic loads. However, recent earthquakes have shown the fallacy of this assumption and have shown that a load path should be clearly defined, analyzed, and designed for seismic loads.

Research on the seismic behavior of steel-girder bridge superstructures (Astaneh-Asl and Donikian, 1995; Itani, 1995; Dicleli and Bruneau, 1995a and 1995b; Itani and Rimal, 1996; Carden et al., 2005a and 2005b, and Bahrami et al., 2010) further confirmed that seismically induced damage is likely in superstructures subjected to large earthquakes and that appropriate measures should be taken to ensure satisfactory seismic performance.

The Kobe earthquake has demonstrated the potential vulnerability of non-ductile steel substructures. However, given that steel substructures are less commonly used in North America and are not of a standardized type when used, they are not addressed herein. The Engineer may find information on this topic in AASHTO (2011) and MCEER/ATC (2003) to complement information available elsewhere in the literature.

These specifications concentrate on the seismic design and detailing of steel-girder bridge

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superstructures. These types of superstructures have experienced moderate earthquakes and have been investigated analytically and experimentally in the aforementioned research. The common thread among these investigations was that these types of superstructures are vulnerable during earthquakes if they are not designed and detailed to resist the resulting seismic forces. A continuous and clearly defined load path is necessary for the transmission of the superstructure inertia forces to the substructure.

6.16.2—Materials

Structural steels used within the seismic load path shall meet the requirements of Article 6.4.1, except as modified herein.

When a member or connection is protected by capacity design, the required nominal resistance of the member or connection shall be determined based on the expected yield strength, R_yF_y, of the adjoining member(s), where F_y is the specified minimum yield strength of the steel used in the adjoining member(s) and R_y is the ratio of the expected yield strength to the specified minimum yield strength. For AASHTO M 270M/M 270 (ASTM A709/A709M) Grade 36, R_y shall be taken equal to 1.5 for hot-rolled structural shapes and 1.3 for plates. For AASHTO M 270M/M 270 (ASTM A709/A709M) Grades 50, 50S and 50W, R_y shall be taken equal to 1.1.

C6.16.2

Previous earthquakes have shown that cross-frames at support locations transfer the inertia forces of the superstructure to the substructure. Therefore, the connections of the adjoining cross-frame members must be protected during seismic events. This is achieved by utilizing a capacity-design methodology in which the cross-frame connections are designed based on the expected nominal resistance of the adjoining members. This methodology serves to confine the ductility demand to the members that have the available excess resistance to ensure ductile behavior. In the capacity-design methodology, all the components surrounding the nonlinear element are designed based on the maximum expected nominal resistance of that element. The capacity-design methodology requires a realistic estimate of the expected nominal resistance of the designated yielded members. To this end, the expected yield strength of various steel materials has been established through a survey of mill test reports and ratios of the expected to nominal yield strength, R_y, have been provided elsewhere (AISC, 2016a) and they are adopted herein. The expected resistance of the designated member is therefore to be determined based on the expected yield strength, R_yF_y, which amplifies the nominal resistance to account for the effect of strain-hardening if the member is expected to undergo nonlinear response.

6.16.3—Design Requirements for Seismic Zone 1

For steel-girder bridges located in Seismic Zone 1, defined as specified in Article 3.10.6, the design of all support cross-frame or diaphragm members and their connections and the connections of the superstructure to the substructure shall satisfy the minimum requirements specified in Articles 3.10.9 and 4.7.4.4.

C6.16.3

These requirements for Zone 1 are to ensure a clear load path for seismic forces.

6.16.4—Design Requirements for Seismic Zones 2, 3, or 4

6.16.4.1—General

Components of slab-on-steel girder bridges located in Seismic Zones 3 or 4, defined as specified in Article 3.10.6, shall be designed using one of the two types of response strategies

C6.16.4.1

The conventional seismic design strategy for slab-on-steel-girder bridges, denoted herein as Type A, is to provide an elastic superstructure in combination with a ductile substructure. In such cases, the support cross-specified in this Article. One of the two types of response strategies should be considered for bridges located in Seismic Zone 2:

- *Type A*—Design an elastic superstructure with a ductile substructure according to these Specifications.
- *Type B*—With the approval of the Owner including the design methodology, design an elastic superstructure and substructure with a fusing mechanism at the interface between the superstructure and substructure.

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

Support cross-frame members on bridges located in Seismic Zones 3 or 4 shall be considered primary members for seismic design. Structural analysis for seismic loads shall consider the relative stiffness of the concrete deck, girders, support cross-frames or diaphragms, and the substructure.

frames are designed to transfer the seismic forces elastically and the inelasticity is limited to the concrete substructure, which is typically designed according to the provisions of Article 5.11. As used in this Article, an elastic component is one in which the demand-to-nominal resistance ratio is less than 1.0.

Providing an essentially elastic superstructure and substructure by utilizing response modification devices such as seismic isolation as a fusing mechanism is a viable alternative strategy to Type A for designing steel-girder bridges to resist earthquake loading. When seismic isolation is used, the Engineer is referred to the AASHTO *Guide Specifications for Seismic Isolation Design* (2014).

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

The recommended design provisions for bridges located in Seismic Zone 2 have been included in this Article with the requirements for bridges located in Seismic Zones 3 and 4. Bridges located in Seismic Zone 2 have a reasonable probability of being subjected to significant seismic forces because the upper boundary for this zone in the current edition of the specifications is significantly higher than in previous editions due to the increase in the return period for the design earthquake from 500 to 1,000 years.

In horizontally-curved or skewed steel bridges, or both, cross-frame forces due to gravity loads may govern over seismic loads depending on boundary conditions at abutments; pier flexibility; and degree of curvature or skew, or both.

6.16.4.2—Deck

Reinforced concrete decks attached by shear connectors satisfying the requirements of Article 6.16.4.3 shall be designed to provide horizontal diaphragm action to transfer seismic forces to the supports as specified in this Article.

Where the deck has a span-to-width ratio of 3.0 or less and the net mid-span lateral seismic displacement of the superstructure is less than twice the average of the adjacent lateral seismic support displacements, the deck within that span may be assumed to act as a rigid horizontal diaphragm designed to resist only the shear resulting from the seismic forces. Otherwise, the deck shall be assumed to act as a flexible horizontal diaphragm designed to resist shear and bending, as applicable, resulting from the seismic forces.

C6.16.4.2

In general, reinforced concrete decks on steel-girder bridges with adequate stud connectors have sufficient rigidity in their horizontal plane that their response approaches rigid-body motion. Therefore, the deck can provide a horizontal diaphragm action to transfer seismic forces to support cross-frames or diaphragms. The seismic forces are collected at the support cross-frames or diaphragms and transferred to the substructure through the bearings and anchor rods. Thus, the support cross-frames or diaphragms must be designed for the resulting seismic forces. The lateral loading of the intermediate cross-frames in between the support locations for straight bridges is minimal in this case, consisting primarily of the local tributary inertia forces from the girders. Adequate stud connectors are required to ensure the necessary diaphragm action as previous earthquake reconnaissance showed that for some bridges in California in which the

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The transverse seismic shear force on the deck, F_{px}, within the span under consideration shall be determined as:

$$F_{px} = \frac{W_{px}}{W} F \quad (6.16.4.2\text{-}1)$$

where:

F = force (kip) determined as follows:

- For structures in Seismic Zone 2 designed using Strategy Type A, the elastic transverse base shears at the support under consideration divided by a response modification factor, *R*, equal to 1.0.
- For structures in Seismic Zones 3 or 4 designed using Strategy Type A, the lesser of:
  - The elastic transverse base shears at the support under consideration divided by a response modification factor, *R*, equal to 1.0, and
  - The inelastic hinging force determined as specified in Article 3.10.9.4.3.
- For structures in Seismic Zones 2, 3 or 4 designed using Strategy Type B, the expected lateral resistance of the fusing mechanism multiplied by an appropriate overstrength factor.

W = total weight of the deck, steel girders, and, where applicable, cap beam, plus one-half the column weight within the span under consideration (kip)

W_{px} = weight of the deck plus one-half of the weight of the steel girders in the span under consideration (kip)

shear connectors at support locations were damaged during a seismic event, the deck in fact slid on the top of the steel girders (Roberts, 1992 and Carden et al., 2005a).

During a seismic event, inertia forces generated by the mass of the deck must be transferred to the support cross-frames or diaphragms. The seismic forces are transferred through longitudinal and transverse shear forces and axial forces.

In cases where the deck may be idealized as a rigid horizontal diaphragm, F_{px} is distributed to the supports based on their relative stiffnesses. In cases where the deck must be idealized as a flexible horizontal diaphragm, F_{px} is distributed to the supports based on their respective tributary areas. Decks idealized as rigid diaphragms need only be designed for shear. Decks idealized as flexible diaphragms must be designed for both shear and bending as maximum in-plane deflections of the deck under lateral loads in this case are more than twice the average of the lateral deflections at adjacent support locations. Concrete decks may be designed for shear and bending moments based on strut and tie models (STM), as defined in Article 5.8.2.

In cases where the deck cannot provide horizontal diaphragm action, the Engineer should consider providing lateral bracing to serve as a horizontal diaphragm to transfer the seismic forces.

F_{px} in Eq. 6.16.4.2-1 represents the total transverse seismic shear force that the deck is subjected to within a particular span. At skewed supports in structures designed using Strategy Type A, *F* should be taken as the sum of the absolute values of the components of the transverse and longitudinal base shears parallel to the skew combined as specified in Article 3.10.8, and as shown in Figure C6.16.4.2-1.

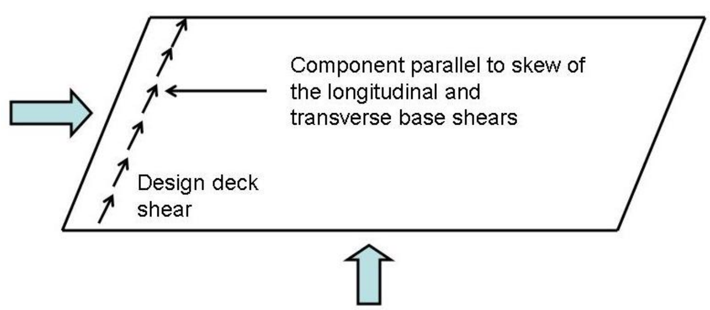

Figure C6.16.4.2-1—Design Deck Shear, F_{px}, at Skewed Supports

Shear keys are typically designed to fuse during the design event earthquake. In lieu of experimental test data, the overstrength ratio for shear key resistance may be obtained from the *Guide Specifications for LRFD Seismic Bridge Design* (2011). Thus, where reinforced concrete shear keys are used as a fusing mechanism, the expected lateral resistance of the shear keys, including an overstrength factor, should be taken equal to $1.5V_{ni}$, where V_{ni} is equal to the nominal interface shear resistance determined as specified in Article 5.7.4.

6.16.4.3—Shear Connectors

Stud shear connectors shall be provided along the interface between the deck and the steel girders, or along the interface between the deck and the top of the support cross-frames or diaphragms, or both, as necessary to transfer the seismic forces. If reinforced concrete diaphragms that are connected integrally with the bridge deck are used at support locations, then the shear connectors on the steel girders at those locations need not be designed according to the provisions of this Article.

The shear connectors on the girders assumed effective at the support under consideration shall be taken as those spaced no further than $9t_w$ on each side of the outer projecting element of the bearing stiffeners at that support.

The diameter of the shear connectors within this region shall not be greater than 2.5 times the thickness of the top chord of the cross-frame or the top flange of the diaphragm.

At support locations, shear connectors on the girders or on the support cross-frames or diaphragms, or both, as necessary, shall be designed to resist the combination of shear and axial forces corresponding to the transverse seismic shear force, F_{px}, determined as specified in Article 6.16.4.2.

The resistance of stud shear connectors subject to combined shear and axial forces shall be evaluated according to the tension–shear interaction equation given as follows:

$$\left(\frac{N_u}{N_r}\right)^{5/3} + \left(\frac{Q_u}{Q_r}\right)^{5/3} \leq 1.0 \qquad (6.16.4.3\text{-}1)$$

in which:

$$h_h = h_{eff} - d_h \geq \frac{w_h}{3} \qquad (6.16.4.3\text{-}2)$$

N_r = factored tensile resistance of a single stud shear connector (kip)

$$= \phi_{st} N_n \qquad (6.16.4.3\text{-}3)$$

N_n = nominal tensile resistance of a single stud shear connector (kip)

$$= \psi_g \psi_{ed} \frac{A_{nc}}{A_{nco}} N_b \leq A_{sc} F_u \qquad (6.16.4.3\text{-}4)$$

ψ_g = group effect modification factor taken as follows:

- For transverse spacing:

  ψ_g = 0.95 for two studs
  ψ_g = 0.90 for three studs

C6.16.4.3

Stud shear connectors play a significant role in transferring the seismic forces from the deck to the support cross-frames or diaphragms. These seismic forces are transferred to the substructure at support locations. Thus, the shear connectors at support locations are subjected to the largest seismic forces, unless reinforced concrete diaphragms that are connected integrally with the bridge deck are used. Failure of these shear connectors will cause the deck to slip on the top flange of the girder and, thus, alter the seismic load path (Caltrans, 2001; Carden et al., 2005a; and Bahrami et al., 2010).

The shear center of composite steel-girder superstructures is located above the deck (Zahrai and Bruneau, 1998 and Bahrami et al., 2010). Therefore, during a seismic event, the superstructure will be subjected to torsional moments along the longitudinal axis of the bridge that produce axial forces on the shear connectors in addition to the longitudinal and transverse shears. Lateral deformations during a seismic event produce double curvature in the top chord of the cross-frame, creating axial forces in the shear connectors on that member that must be considered. Experimental and analytical investigations (Carden et al., 2005a and Bahrami et al., 2010) showed that the seismic demand on shear connectors that are placed only on the girders at support locations may cause significant damage to the connectors and the deck.

ACI (2014) provides equations for anchorage to concrete of pre- and post-installed anchors subject to shear and axial forces. However, these equations are not used in this Article for the design of shear connectors on slab-on-steel-girder bridges subject to combined shear and axial forces. Mouras et al. (2008) investigated the behavior of shear connectors placed on a steel girder under static and dynamic axial loads. The effects of haunches in reinforced concrete decks, stud length, the number of studs, and the arrangement of the studs in the transverse and longitudinal directions of the bridge were investigated. Based on this investigation, several modifications were recommended to the ACI equations that are reflected in the equations given in this Article. These modifications ensure a ductile response of the shear connectors that is beneficial in seismic applications. The modifications are as follows:

- provision for adequate embedment of the shear connectors to engage the reinforcement in the deck slab,
- use of an effective haunch height instead of the effective height given in the ACI equations, and
- consideration of a group modification factor for longitudinal and transverse spacing. This factor accounts for the overlapping of the cones when studs are closely spaced.

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- For longitudinal spacing:

  $\psi_g = 0.95$ for spacing $\leq 3h_{eff}$
  $\psi_g = 1.0$ for spacing $> 3h_{eff}$

ψ_{ed} = edge modification factor taken as follows:

$$\psi_{ed} = 0.7 + 0.3\frac{C_a}{1.5h_h} \leq 1.0 \qquad (6.16.4.3\text{-}5)$$

A_{nco} = projected area of concrete failure for a single stud shear connector based on the concrete breakout resistance in tension (in.^{2})

= $9h_h^2$ (6.16.4.3-6)

N_b = concrete breakout resistance in tension of a single stud shear connector in cracked concrete (kip)

= $0.76\sqrt{f'_c}h_h^{1.5}$ (6.16.4.3-7)

where:

ϕ_{st} = resistance factor for shear connectors in tension specified in Article 6.5.4.2

A_{nc} = projected area of concrete for a single stud shear connector or group of connectors approximated from the base of a rectilinear geometric figure that results from projecting the failure surface outward $1.5h_h$ from the centerline of the single connector or, in the case of a group of connectors, from a line through a row of adjacent connectors (in.^{2})

A_{sc} = cross-sectional area of a stud shear connector (in.^{2})

C_a = smallest distance from center of stud to the edge of the concrete (in.)

d_h = depth of haunch (in.)

F_u = specified minimum tensile strength of a stud shear connector determined as specified in Article 6.4.4 (ksi)

h_{eff} = effective embedment depth of a stud shear connector (in.)

h_h = effective height of the stud above the top of the haunch to the underside of the head (in.)

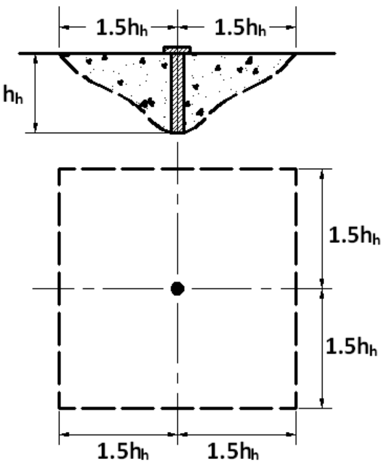
N_u = seismic axial force demand per stud at the support cross-frame or diaphragm location under consideration (kip)

Q_u = seismic shear demand per stud at the support cross-frame or diaphragm location under consideration due to the governing orthogonal combination of seismic shears (kip)

Q_r = factored shear resistance of a single stud shear connector determined as specified in Article 6.10.10.4.1 (kip)

w_h = width of haunch perpendicular to bridge span axis (in.)

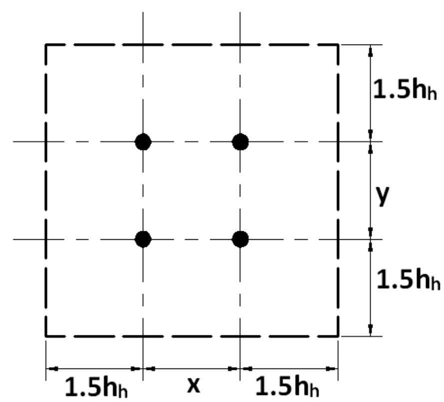
The calculation of the projected area of concrete failure for a single stud shear connector based in the concrete breakout resistance in tension, A_{nco}, is illustrated in Figure C6.16.4.3-1.



$$A_{nco} = (1.5h_h + 1.5h_h)(1.5h_h + 1.5h_h) = 9h_h^2 \qquad (C6.16.4.3\text{-}1)$$

Figure C6.16.4.3-1–Calculation of A_{nco}

The calculation of A_{nc} for a group of four studs consisting of two rows with two studs per row is illustrated in Figure C6.16.4.3-2.



$$A_{nc} = (3h_h + x)(3h_h + y) \qquad (C6.16.4.3\text{-}2)$$

Figure C6.16.4.3-2–Calculation of A_{nc} for a Group of Four Studs (two rows with two studs per row)

Experimental investigation by Bahrami et al. (2010) showed that these equations for the shear and axial resistance and their interaction may be used to satisfactorily determine the resistance of stud shear connectors under the combined loading effects.

For the seismic design of continuous composite spans, shear connectors should be provided throughout the length of the bridge. The requirements of this Article notwithstanding, should shear connectors be omitted in regions of negative flexure, positive attachment of the deck to the support cross-frames or diaphragms located at interior piers must still be provided. Analytical investigation (Carden et al., 2005a) showed that a lack of shear connectors in regions of negative flexure caused the seismic forces to be transferred into the steel girders at points of permanent load contraflexure, causing large weak-axis bending stresses in the girders. In addition, the intermediate cross-frames were subjected to large seismic forces, while the support cross-frames were subjected to smaller forces. This indicated that the seismic load path had been significantly altered.

6.16.4.4—Elastic Superstructures

For an elastic superstructure, support cross-frame members or support diaphragms shall be designed according to the applicable provisions of Articles 6.7, 6.8, or 6.9, or some combination thereof, to remain elastic during a seismic event.

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

C6.16.4.4

To achieve an elastic superstructure, the various components of the support cross-frames or the support diaphragms, as applicable, must be designed to remain elastic under the forces that are generated during the design earthquake according to the applicable provisions of Articles 6.7, 6.8, or 6.9, or some combination thereof. No other special seismic requirements are specified for these members in this case.

The elastic superstructure can have steel cross-frames of various configurations, steel diaphragms, or reinforced concrete diaphragms. The Tennessee DOT has as an alternative used reinforced concrete diaphragms over bent locations. The details of these diaphragms and others are discussed in Bahrami et al. (2010) and Itani and Reno (1995).

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APPENDIX A6—FLEXURAL RESISTANCE OF COMPOSITE I-SECTIONS IN NEGATIVE FLEXURE AND NONCOMPOSITE I-SECTIONS WITH COMPACT OR NONCOMPACT WEBS IN STRAIGHT BRIDGES

A6.1—GENERAL

These provisions shall apply only to sections in straight bridges whose supports are normal or skewed not more than 20 degrees from normal, and with intermediate diaphragms or cross-frames placed in contiguous lines parallel to the supports, that satisfy the following requirements:

- the specified minimum yield strengths of the flanges and web do not exceed 70.0 ksi,
- the web satisfies the noncompact slenderness limit:

$$\frac{2D_c}{t_w} \leq \lambda_{rw} \qquad \text{(A6.1-1)}$$

and:

- the flanges satisfy the following ratio:

$$\frac{I_{yc}}{I_{yt}} \geq 0.3 \qquad \text{(A6.1-2)}$$

in which:

λ_{rw} = limiting slenderness ratio for a noncompact web

$$= 4.6\sqrt{\frac{E}{F_{yc}}} \leq \lambda_{rw} = \left(3.1+\frac{5.0}{a_{wc}}\right)\sqrt{\frac{E}{F_{yc}}} \leq 5.7\sqrt{\frac{E}{F_{yc}}} \qquad \text{(A6.1-3)}$$

a_{wc} = ratio of two times the web area in compression to the area of the compression flange

$$= \frac{2D_c t_w}{b_{fc} t_{fc}} \qquad \text{(A6.1-4)}$$

where:

D_c = depth of the web in compression in the elastic range (in.). For composite sections, D_c shall be determined as specified in Article D6.3.1.

I_{yc} = moment of inertia of the compression flange of the steel section about the vertical axis in the plane of the web (in.^{4})

I_{yt} = moment of inertia of the tension flange of the steel section about the vertical axis in the plane of the web (in.^{4})

Otherwise, the section shall be proportioned according to the provisions specified in Article 6.10.8.

CA6.1

The optional provisions of Appendix A6 account for the ability of compact and noncompact web I-sections to develop flexural resistances significantly greater than M_y when the web slenderness, $2D_c/t_w$, is well below the noncompact limit of Eq. A6.1-1, which is a restatement of Eq. 6.10.6.2.3-1, and when sufficient requirements are satisfied with respect to the flange specified minimum yield strengths, the compression flange slenderness, $b_{fc}/2t_{fc}$, and the lateral brace spacing. These provisions also account for the beneficial contribution of the St. Venant torsional constant, J. This may be useful, particularly under construction situations, for sections with compact or noncompact webs having larger unbraced lengths for which additional lateral–torsional buckling resistance may be required. Also, for heavy column shapes with $D/b_f < 1.7$, which may be used as beam-columns in steel frames, both the inelastic and elastic buckling resistances are heavily influenced by J.

The potential benefits of the Appendix A6 provisions tend to be small for I-sections with webs that approach the noncompact web slenderness limit of Eq. A6.1-1. For these cases, the simpler and more streamlined provisions of Article 6.10.8 are recommended. The potential gains in economy by using Appendix A6 increase with decreasing web slenderness. The Engineer should give strong consideration to utilizing Appendix A6 for sections in which the web is compact or nearly compact. In particular, the provisions of Appendix A6 are recommended for sections with compact webs, as defined in Article A6.2.1.

The provisions of Appendix A6 are fully consistent with and are a direct extension of the main procedures in Article 6.10.8 in concept and in implementation. The calculation of potential flexural resistances greater than M_y is accomplished through the use of the web plastification parameters R_{pc} and R_{pt} of Article A6.2, corresponding to flexural compression and tension, respectively. These parameters are applied much like the web bend-buckling and hybrid girder parameters R_b and R_h in the main specification provisions.

I-section members with a specified minimum yield strength of the flanges greater than 70.0 ksi are more likely to be limited by Eq. A6.1-1 and are likely to be controlled by design considerations other than the Strength Load Combinations in ordinary bridge construction. In cases where Eq. A6.1-1 is satisfied with F_{yc} > 70.0 ksi, the implications of designing such members in general using a nominal flexural resistance greater than M_y have not been sufficiently studied to merit the use of Appendix A6.

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Sections designed according to these provisions shall qualify as either compact web sections or noncompact web sections determined as specified in Article A6.2.

Eq. A6.1-2 is specified to guard against extremely monosymmetric noncomposite I-sections, in which analytical studies indicate a significant loss in the influence of the St. Venant torsional rigidity, GJ, on the –lateral–torsional buckling resistance due to cross-section distortion. The influence of web distortion on the –lateral–torsional buckling resistance is larger for such members. If the flanges are of equal thickness, this limit is equivalent to $b_{fc} \geq 0.67b_{ft}$.

A6.1.1—Sections with Discretely Braced Compression Flanges

At the strength limit state, the following requirement shall be satisfied:

$$M_u + \frac{1}{3} f_\ell S_{xc} \leq \phi_f M_{nc} \qquad \text{(A6.1.1-1)}$$

where:

ϕ_f = resistance factor for flexure specified in Article 6.5.4.2

f_ℓ = flange lateral bending stress determined as specified in Article 6.10.1.6 (ksi)

M_{nc} = nominal flexural resistance based on the compression flange determined as specified in Article A6.3 (kip-in.)

M_u = bending moment about the major-axis of the cross section determined as specified in Article 6.10.1.6 (kip-in.)

M_{yc} = yield moment with respect to the compression flange determined as specified in Article D6.2 (kip-in.)

S_{xc} = elastic section modulus about the major axis of the section to the compression flange taken as M_{yc}/F_{yc} (in.^{3})

CA6.1.1

Eq. A6.1.1-1 addresses the effect of combined major-axis bending and compression flange lateral bending using an interaction equation approach. This equation expresses the flexural resistance in terms of the section major-axis bending moment, M_u, and the flange lateral bending stress, f_ℓ, computed from an elastic analysis, applicable within the limits on f_ℓ specified in Article 6.10.1.6 (White and Grubb, 2005).

For adequately braced sections with a compact web and compression flange, Eqs. A6.1.1-1 and A6.1.2-1 are generally a conservative representation of the resistance obtained by procedures that address the effect of flange wind moments given in Article 6.10.3.5.1 of AASHTO (1998). In the theoretical limit that the web area becomes negligible relative to the flange area, these equations closely approximate the results of an elastic-plastic section analysis in which a fraction of the width from the tips of the flanges is deducted to accommodate the flange lateral bending. The conservatism of these equations relative to the theoretical solution increases with increasing $D_{cp}t_w/b_{fc}t_{fc}$, f_ℓ, and/or $\left|D_{cp} - D_c\right|$. The conservatism at the limit on f_ℓ specified by Eq. 6.10.1.6-1 ranges from about three to ten percent for practical flexural I-sections.

The multiplication of f_ℓ by S_{xc} in Eq. A6.1.1-1 and by S_{xt} in Eq. A6.1.2-1 stems from the derivation of these equations, and is explained further in White and Grubb (2005). These equations may be expressed in a stress format by dividing both sides by the corresponding elastic section modulus, in which case, Eq. A6.1.1-1 reduces effectively to Eqs. 6.10.3.2.1-2 and 6.10.8.1.1-1 in the limit that the web approaches its noncompact slenderness limit. Correspondingly, Eq. A6.1.2-1 reduces effectively to Eqs. 6.10.7.2.1-2 and 6.10.8.1.2-1 in this limit.

The elastic section moduli, S_{xc} in this Article and S_{xt} in Article A6.1.2, are defined as M_{yc}/F_{yc} and M_{yt}/F_{yt}, respectively, where M_{yc} and M_{yt} are calculated as specified in Article D6.2. This definition is necessary so that for a composite section with a web proportioned precisely at the noncompact limit given by Eq. A6.1-1, the flexural resistance predicted by Appendix A6 is approximately the same as that predicted by Article

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6.10.8. Differences between these predictions are due to the simplifying assumptions of $J = 0$ versus $J \neq 0$ in determining the elastic lateral–torsional buckling resistance and the limiting unbraced length L_r, the use of $k_c = 0.35$ versus the use of k_c from Eq. A6.3.2-6 in determining the limiting slenderness for a noncompact flange, and the use of a slightly different definition for F_{yr}. The maximum potential flexural resistance, shown as F_{max} in Figure C6.10.8.2.1-1, is defined in terms of the flange stresses as R_hF_{yf} for a section with a web proportioned precisely at the noncompact web limit and designed according to the provisions of Article 6.10.8, where R_h is the hybrid factor defined in Article 6.10.1.10.1. As discussed in Article 6.10.1.1.1a, for composite sections, the elastically computed flange stress to be compared to this limit is to be taken as the sum of the stresses caused by the loads applied separately to the steel, short-term composite and long-term composite sections. The resulting provisions of Article 6.10.8 are a reasonable strength prediction for slender-web sections in which the web is proportioned precisely at the noncompact limit. By calculating S_{xc} and S_{xt} in the stated manner, elastic section moduli are obtained that, when multiplied by the corresponding flexural resistances predicted from Article 6.10.8 for the case of a composite slender-web section proportioned precisely at the noncompact web limit, produce approximately the same flexural resistances as predicted in Appendix A6.

For composite sections with web slenderness values that approach the compact web limit of Eq. A6.2.1-2, the effects of the loadings being applied to the different steel, short-term and long-term sections are nullified by the yielding within the section associated with the development of the stated flexural resistance. Therefore, for compact web sections, these Specifications define the maximum potential flexural resistance, shown as M_{max} in Figure C6.10.8.2.1-1, as the plastic moment M_p, which is independent of the effects of the different loadings.

A6.1.2—Sections with Discretely Braced Tension Flanges

At the strength limit state, the following requirement shall be satisfied:

$$M_u + \frac{1}{3} f_\ell S_{xt} \leq \phi_f M_{nt} \quad \text{(A6.1.2-1)}$$

where:

M_{nt} = nominal flexural resistance based on tension yielding determined as specified in Article A6.4 (kip-in.)

M_{yt} = yield moment with respect to the tension flange determined as specified in Article D6.2 (kip-in.)

CA6.1.2

Eq. A6.1.2-1 parallels Eq. A6.1.1-1 for discretely braced compression flanges, but applies to the case of discretely braced flanges in flexural tension due to the major-axis bending moment.

When f_ℓ is equal to zero and M_{yc} is less than or equal to M_{yt}, the flexural resistance based on the tension flange does not control and Eq. A6.1.2-1 need not be checked. The web plastification factor for tension flange yielding, R_{pt}, from Article A6.2 also need not be computed for this case.

S_{xt} = elastic section modulus about the major axis of the section to the tension flange taken as M_{yt}/F_{yt} (in.^{3})

A6.1.3—Sections with Continuously Braced Compression Flanges

At the strength limit state, the following requirement shall be satisfied:

$$M_u \leq \phi_f R_{pc} M_{yc} \tag{A6.1.3-1}$$

where:

M_{yc} = yield moment with respect to the compression flange determined as specified in Article D6.2 (kip-in.)
R_{pc} = web plastification factor for the compression flange determined as specified in Article A6.2.1 or Article A6.2.2, as applicable

CA6.1.3

Flange lateral bending need not be considered in continuously braced flanges, as discussed further in Article C6.10.1.6.

A6.1.4—Sections with Continuously Braced Tension Flanges

At the strength limit state, the following requirement shall be satisfied:

$$M_u \leq \phi_f R_{pt} M_{yt} \tag{A6.1.4-1}$$

where:

M_{yt} = yield moment with respect to the tension flange determined as specified in Article D6.2 (kip-in.)
R_{pt} = web plastification factor for the tension flange determined as specified in Article A6.2.1 or Article A6.2.2, as applicable

A6.2—WEB PLASTIFICATION FACTORS

A6.2.1—Compact Web Sections

Sections that satisfy the following requirement shall qualify as compact web sections:

$$\frac{2D_{cp}}{t_w} \leq \lambda_{pw(D_{cp})} \tag{A6.2.1-1}$$

in which:

$\lambda_{pw(D_{cp})}$ = limiting slenderness ratio for a compact web corresponding to $2D_{cp}/t_w$

CA6.2.1

Eq. A6.2.1-1 ensures that the section is able to develop the full plastic moment capacity M_p provided that other flange slenderness and lateral torsional bracing requirements are satisfied. This limit is significantly less than the noncompact web limit shown in Table C6.10.1.10.2-2. It is generally satisfied by rolled I-shapes, but typically not by the most efficient built-up sections.

Eq. A6.2.1-2 is a web compactness limit that accounts for the higher demands on the web in noncomposite monosymmetric I-sections and in composite I-sections in negative bending with larger shape factors, M_p/M_y (White and Barth, 1998) (Barth et al., 2005). This updated web compactness limit eliminates the need for providing an interaction equation between the web and flange compactness requirements (AASHTO, 1998) (AASHTO, 2002).

$$= \frac{\sqrt{\dfrac{E}{F_{yc}}}}{\left(0.54\dfrac{M_p}{R_h M_y} - 0.09\right)^2} \le \lambda_{rw}\left(\frac{D_{cp}}{D_c}\right) \quad \text{(A6.2.1-2)}$$

λ_{rw} = limiting slenderness ratio for a noncompact web

$$= 4.6\sqrt{\frac{E}{F_{yc}}} \le \lambda_{rw} = \left(3.1 + \frac{5.0}{a_{wc}}\right)\sqrt{\frac{E}{F_{yc}}} \le 5.7\sqrt{\frac{E}{F_{yc}}} \quad \text{(A6.2.1-3)}$$

in which:

a_{wc} = ratio of two times the web area in compression to the area of the compression flange

$$= \frac{2D_c t_w}{b_{fc} t_{fc}} \quad \text{(A6.2.1-4)}$$

where:

D_c = depth of the web in compression in the elastic range (in.). For composite sections, D_c shall be determined as specified in Article D6.3.1.
D_{cp} = depth of the web in compression at the plastic moment determined as specified in Article D6.3.2 (in.)
M_y = yield moment taken as the smaller of M_{yc} and M_{yt} determined as specified in Article D6.2 (kip-in.)
R_h = hybrid factor determined as specified in Article 6.10.1.10.1

The web plastification factors shall be taken as:

$$R_{pc} = \frac{M_p}{M_{yc}} \quad \text{(A6.2.1-5)}$$

$$R_{pt} = \frac{M_p}{M_{yt}} \quad \text{(A6.2.1-6)}$$

where:

M_p = plastic moment determined as specified in Article D6.1 (kip-in.)
M_{yc} = yield moment with respect to the compression flange determined as specified in Article D6.2 (kip-in.)
M_{yt} = yield moment with respect to the tension flange determined as specified in Article D6.2 (kip-in.)
R_{pc} = web plastification factor for the compression flange
R_{pt} = web plastification factor for tension flange yielding

Eq. A6.2.1-2 reduces to the previous web compactness limit given by Equation 6.10.4.1.2-1 in AASHTO (1998) when $M_p/M_y = 1.12$, which is representative of the shape factor for doubly-symmetric noncomposite I-sections. The previous web compactness limit is retained in Eq. 6.10.6.2.2-1 for composite sections in positive flexure since research does not exist to quantify the web compactness requirements for these types of sections with any greater precision, and also since most composite sections in positive flexure easily satisfy this requirement.

The compactness restrictions on the web imposed by Eq. A6.2.1-2 are approximately the same as the requirements implicitly required for development of the plastic moment resistance, M_p, by the Q formula in AASHTO (1998). Both of these requirements are plotted as a function of M_p/M_y for F_{yc} = 50.0 ksi in Figure CA6.2.1-1.

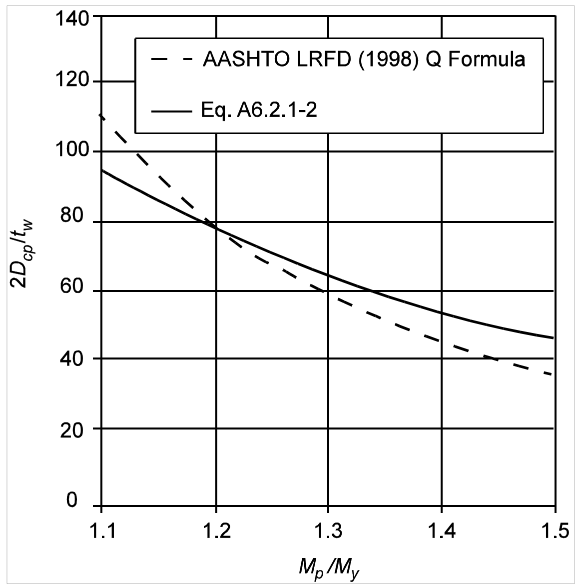

Figure CA6.2.1-1—Web Compactness Limits as a Function of M_p/M_y from the AASHTO (1998) Q formula and from Eq. A6.2.1-2 for F_{yc} = 50.0 ksi

For a compact web section, the maximum potential moment resistance, represented by M_{max} in Figure C6.10.8.2.1-1, is simply equal to M_p. Eqs. A6.2.1-5 and A6.2.1-6 capture this attribute and eliminate the need to repeat the subsequent flexural resistance equations in a nearly identical fashion for compact and noncompact web sections. For a compact web section, the web plastification factors are equivalent to the cross-section shape factors.

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A6.2.2—Noncompact Web Sections

Sections that do not satisfy the requirement of Eq. A6.2.1-1, but for which the web slenderness satisfies the following requirement:

$$\lambda_w < \lambda_{rw} \quad \text{(A6.2.2-1)}$$

shall qualify as noncompact web sections, where:

λ_w = slenderness ratio for the web based on the elastic moment

$$= \frac{2D_c}{t_w} \quad \text{(A6.2.2-2)}$$

λ_{rw} = limiting slenderness ratio for a noncompact web

$$= 4.6\sqrt{\frac{E}{F_{yc}}} \leq \lambda_{rw} = \left(3.1 + \frac{5.0}{a_{wc}}\right)\sqrt{\frac{E}{F_{yc}}} \leq 5.7\sqrt{\frac{E}{F_{yc}}} \quad \text{(A6.2.2-3)}$$

D_c = depth of the web in compression in the elastic range (in.). For composite sections, D_c shall be determined as specified in Article D6.3.1.

The web plastification factors shall be taken as:

$$R_{pc} = \left[1 - \left(1 - \frac{R_h M_{yc}}{M_p}\right)\left(\frac{\lambda_w - \lambda_{pw(D_c)}}{\lambda_{rw} - \lambda_{pw(D_c)}}\right)\right]\frac{M_p}{M_{yc}} \leq \frac{M_p}{M_{yc}} \quad \text{(A6.2.2-4)}$$

$$R_{pt} = \left[1 - \left(1 - \frac{R_h M_{yt}}{M_p}\right)\left(\frac{\lambda_w - \lambda_{pw(D_c)}}{\lambda_{rw} - \lambda_{pw(D_c)}}\right)\right]\frac{M_p}{M_{yt}} \leq \frac{M_p}{M_{yt}} \quad \text{(A6.2.2-5)}$$

where:

$\lambda_{pw(D_c)}$ = limiting slenderness ratio for a compact web corresponding to $2D_c/t_w$

$$= \lambda_{pw(D_{cp})}\left(\frac{D_c}{D_{cp}}\right) \leq \lambda_{rw} \quad \text{(A6.2.2-6)}$$

CA6.2.2

Eqs. A6.2.2-4 and A6.2.2-5 account for the influence of the web slenderness on the maximum potential flexural resistance, M_{max} in Figure C6.10.8.2.1-1, for noncompact web sections. As $2D_c/t_w$ approaches the noncompact web limit λ_{rw}, R_{pc} and R_{pt} approach values equal to R_h and the maximum potential flexural resistance expressed within the subsequent limit state equations approaches a limiting value of R_hM_y. As $2D_{cp}/t_w$ approaches the compact web limit $\lambda_{pw(D_{cp})}$, Eqs. A6.2.2-4 and A6.2.2-5 define a smooth transition in the maximum potential flexural resistance, expressed by the subsequent limit state equations, from M_y to the plastic moment resistance M_p. For a compact web section, the web plastification factors R_{pc} and R_{pt} are simply the section shape factors corresponding to the compression and tension flanges, M_p/M_{yc} and M_p/M_{yt}. The subsequent flexural resistance equations are written using R_{pc} and R_{pt} for these types of sections, rather than expressing the maximum resistance simply as M_p, to avoid repetition of strength equations that are otherwise identical.

In Eqs. A6.2.2-4 and A6.2.2-5, explicit maximum limits of M_p/M_{yc} and M_p/M_{yt} are placed on R_{pc} and R_{pt}, respectively. As a result, the larger of the base resistances, $R_{pc}M_{yc}$ or $R_{pt}M_{yt}$, is limited to M_p for a highly monosymmetric section in which M_{yc} or M_{yt} can be greater than M_p. The limits on I_{yc}/I_{yt} given in Article 6.10.2.2 will tend to prevent the use of extremely monosymmetric sections that have M_{yc} or M_{yt} values greater than M_p. The upper limits on R_{pc} and R_{pt} have been provided to make Eqs. A6.2.2-4 and A6.2.2-5 theoretically correct in these extreme cases, even though the types of monosymmetric sections where these limits control will not likely occur.

Eq. A6.2.2-6 converts the web compactness limit given by Eq. A6.2.1-2, which is defined in terms of D_{cp}, to a value that can be used consistently in terms of D_c in Eqs. A6.2.2-4 and A6.2.2-5. In cases where $D_c/D > 0.5$, D_{cp}/D is typically larger than D_c/D; therefore, $\lambda_{pw(D_c)}$ is smaller than $\lambda_{pw(D_{cp})}$. However, when $D_c/D < 0.5$, D_{cp}/D is typically smaller than D_c/D and $\lambda_{pw(D_c)}$ is larger than $\lambda_{pw(D_{cp})}$. In extreme cases where D_c/D is significantly less than 0.5, the web slenderness associated with the elastic cross section, $2D_c/t_w$, can be larger than λ_{rw} while that associated with the plastic cross section, $2D_{cp}/t_w$, can be smaller than $\lambda_{pw(D_{cp})}$ without the upper limit of $\lambda_{rw}(D_{cp}/D_c)$ that is placed on this value. That is, the elastic web is classified as slender while the plastic web is classified as compact. In these cases, the compact web limit is defined as $\lambda_{pw(D_{cp})} = \lambda_{rw}(D_{cp}/D_c)$. This is a conservative approximation aimed at protecting against the occurrence of bend-buckling in the web prior to reaching the section plastic resistance.

The ratio D_c/D is generally greater than 0.5 for noncomposite sections with a smaller flange in compression, such as typical composite I-girders in positive bending before they are made composite.

A6.3—FLEXURAL RESISTANCE BASED ON THE COMPRESSION FLANGE

A6.3.1—General

Eq. A6.1.1-1 shall be satisfied for both local buckling and lateral–torsional buckling using the appropriate value of M_{nc} determined for each case as specified in Articles A6.3.2 and A6.3.3, respectively.

CA6.3.1

All of the I-section compression flange flexural resistance equations of these Specifications are based consistently on the logic of identifying the two anchor points shown in Figure C6.10.8.2.1-1 for the case of uniform major-axis bending. Anchor point 1 is located at the length $L_b = L_p$ for lateral–torsional buckling (LTB) or the flange slenderness $b_{fc}/2t_{fc} = \lambda_{pf}$ for flange–local buckling (FLB) corresponding to development of the maximum potential flexural resistance, labeled as F_{max} or M_{max} in the figure, as applicable. Anchor point 2 is located at the length L_r or flange slenderness λ_{rf} for which the inelastic and elastic lateral–torsional buckling or flange–flange local buckling resistances are the same.

In Article A6.3, this resistance is taken as $R_bF_{yr}S_{xc}$, where F_{yr} is taken as the smaller of $0.7F_{yc}$, F_{yw}, or $R_hF_{yt}S_{xt}/S_{xc}$, but not smaller than $0.5F_{yc}$. The first two of these resistances are the same as in Article 6.10.8. The third resistance expression, $R_hF_{yt}S_{xt}/S_{xc}$, which is simply the elastic compression flange stress at the cross-section moment $R_hF_{yt}S_{xt} = R_hM_{yt}$, is specific to Article A6.3 and captures the effects of significant early tension-flange yielding in sections with a small depth of web in compression. In sections that have this characteristic, the early tension-flange yielding invalidates the elastic lateral–torsional buckling equation on which the noncompact bracing limit L_r is based, and also makes the corresponding elastic flange local buckling equation suspect due to potential significant inelastic redistribution of stresses to the compression flange. The limit $R_hF_{yt}S_{xt}/S_{xc}$ rarely controls for bridge I-girders, but it may control in some instances of pier negative moment sections in composite continuous spans, prior to the section becoming composite, in which the top flange is significantly smaller than the bottom flange. For $L_b > L_r$ or $b_{fc}/2t_{fc} > \lambda_{rf}$, the lateral–torsional buckling and flange–flange local buckling resistances are governed by elastic buckling. However, the elastic flange–flange local buckling resistance equations are not specified explicitly in these provisions since the limits of Article 6.10.2.2 preclude elastic flange–flange local buckling for specified minimum yield strengths up to and including $F_{yc} = 70.0$ ksi, which is the limiting yield strength for the application of the provisions of Appendix A6.

For unbraced lengths subjected to moment gradient, the lateral–torsional buckling resistances for the case of

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uniform major-axis bending are simply scaled by the moment gradient modifier C_b, with the exception that the lateral–torsional buckling resistance is capped at F_{max} or M_{max}, as illustrated by the dashed line in Figure C6.10.8.2.1-1. The maximum unbraced length at which the lateral–torsional buckling resistance is equal to F_{max} or M_{max} under a moment gradient may be determined from Article D6.4.1 or D6.4.2, as applicable. The flange-local buckling resistance for moment gradient cases is the same as that for the case of uniform major-axis bending, neglecting the relatively minor influence of moment gradient effects.

A6.3.2—Local Buckling Resistance

The flexural resistance based on compression flange local buckling shall be taken as:

- If $\lambda_f \leq \lambda_{pf}$, then:

$$M_{nc} = R_{pc}M_{yc} \tag{A6.3.2-1}$$

- Otherwise:

$$M_{nc} = \left[1 - \left(1 - \frac{F_{yr}S_{xc}}{R_{pc}M_{yc}}\right)\left(\frac{\lambda_f - \lambda_{pf}}{\lambda_{rf} - \lambda_{pf}}\right)\right] R_{pc}M_{yc} \tag{A6.3.2-2}$$

in which:

λ_f = slenderness ratio for the compression flange

$$= \frac{b_{fc}}{2t_{fc}} \tag{A6.3.2-3}$$

λ_{pf} = limiting slenderness ratio for a compact flange

$$= 0.38\sqrt{\frac{E}{F_{yc}}} \tag{A6.3.2-4}$$

λ_{rf} = limiting slenderness ratio for a noncompact flange

$$= 0.95\sqrt{\frac{Ek_c}{F_{yr}}} \tag{A6.3.2-5}$$

k_c = flange local buckling coefficient

- For built-up sections:

$$= \frac{4}{\sqrt{\dfrac{D}{t_w}}} \tag{A6.3.2-6}$$

CA6.3.2

Eq. A6.3.2-4 defines the slenderness limit for a compact flange, whereas Eq. A6.3.2-5 gives the slenderness limit for a noncompact flange. The nominal flexural resistance of a section with a compact flange is independent of the flange slenderness, whereas the flexural resistance of a section with a noncompact flange is expressed as a linear function of the flange slenderness as illustrated in Figure C6.10.8.2.1-1. The compact flange slenderness limit is the same as specified in AISC (2016b), AASHTO (1998), and Article 6.10.8.2.2. For different grades of steel, this slenderness limit is specified in Table C6.10.8.2.2-1. All current ASTM W shapes except W21×48, W14×99, W14×90, W12×65, W10×12, W8×31, W8×10, W6×15, W6×9 and W6×8.5 have compact flanges at $F_y \leq 50.0$ ksi.

Eq. A6.3.2-6 for the flange local buckling coefficient comes from the implementation of Johnson′s (1985) research in AISC (2016b). The value $k_c = 0.35$ is a lower bound to values back-calculated by equating the resistances from these provisions, or those of Article 6.10.8.2.2 where this Article is not applicable, to the measured resistances from Johnson′s and other tests such as those conducted by Basler et al. (1960). Tests ranging from $D/t_w = 72$ to 245 were considered. One of the tests from Basler et al. (1960) with $D/t_w = 185$, in which the compression flange was damaged in a previous test and then subsequently straightened and cut-back to a narrower width prior to retesting, exhibited a back-calculated k_c of 0.28. This test was not considered in selecting the lower bound. Other tests by Johnson (1985) that had higher D/t_w values exhibited back-calculated k_c values greater than 0.4. A value of $k_c = 0.43$ is obtained for ideally simply-supported boundary conditions at the web-flange juncture (Timoshenko and Gere, 1961). Smaller values of k_c correspond to the fact that web local buckling in more slender webs tends to destabilize the compression flange. The value of $k_c = 0.76$ for rolled shapes is taken from AISC (2016b).

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$0.35 \le k_c \le 0.76$

- For rolled shapes:

$$= 0.76$$

where:

F_{yr} = compression flange stress at the onset of nominal yielding within the cross section, including residual stress effects, but not including compression flange lateral bending, taken as the smaller of $0.7F_{yc}$, $R_hF_{yt}\ S_{xt}/S_{xc}$ and F_{yw}, but not less than $0.5F_{yc}$

M_{yc} = yield moment with respect to the compression flange determined as specified in Article D6.2 (kip-in.)

M_{yt} = yield moment with respect to the tension flange determined as specified in Article D6.2 (kip-in.)

R_h = hybrid factor determined as specified in Article 6.10.1.10.1

R_{pc} = web plastification factor for the compression flange determined as specified in Article A6.2.1 or Article A6.2.2, as applicable

S_{xc} = elastic section modulus about the major axis of the section to the compression flange taken as M_{yc}/F_{yc} (in.^{3})

S_{xt} = elastic section modulus about the major axis of the section to the tension flange taken as M_{yt}/F_{yt} (in.^{3})

A6.3.3—Lateral–Torsional Buckling Resistance

For unbraced lengths in which the member is prismatic, the flexural resistance based on lateral–torsional buckling shall be taken as:

- If $L_b \le L_p$, then:

$$M_{nc} = R_{pc}M_{yc} \tag{A6.3.3-1}$$

- If $L_p < L_b \le L_r$, then:

$$M_{nc} = C_b\left[1-\left(1-\frac{F_{yr}S_{xc}}{R_{pc}M_{yc}}\right)\left(\frac{L_b-L_p}{L_r-L_p}\right)\right]R_{pc}M_{yc} \le R_{pc}M_{yc} \tag{A6.3.3-2}$$

- If $L_b > L_r$, then:

$$M_{nc} = F_{cr}S_{xc} \le R_{pc}M_{yc} \tag{A6.3.3-3}$$

in which:

L_b = unbraced length (in.).

CA6.3.3

Eq. A6.3.3-4 defines the compact unbraced length limit for a member subjected to uniform major-axis bending, whereas Eq. A6.3.3-5 gives the corresponding noncompact unbraced length limit. The nominal flexural resistance of a member braced at or below the compact limit is independent of the unbraced length, whereas the flexural resistance of a member braced at or below the noncompact limit is expressed as a linear function of the unbraced length as illustrated in Figure C6.10.8.2.1-1. The compact bracing limit of Eq. A6.3.3-4 is similar to the bracing requirement for use of the general compact-section flexural resistance equations and/or the Q formula equations in AASHTO (1998). The limit given by Eq. A6.3.3-4 is generally somewhat more restrictive than the limit given by the corresponding L_p equation in AASHTO (1998) and AISC (2016b). The limit given by Eq. A6.3.3-4 is based on linear regression analysis within the region corresponding to the inelastic lateral torsional buckling equation, shown qualitatively in Figure C6.10.8.2.1-1, for a wide range of data from experimental flexural tests involving uniform major-axis bending and in which the physical effective length for lateral torsional buckling is effectively 1.0.

Note that the most economical solution is not necessarily achieved by limiting the unbraced length to L_p in

L_p = limiting unbraced length to achieve the nominal flexural resistance $R_{pc}M_{yc}$ under uniform bending (in.)

$$=1.0r_t\sqrt{\frac{E}{F_{yc}}} \qquad \text{(A6.3.3-4)}$$

L_r = limiting unbraced length to achieve the nominal onset of yielding in either flange under uniform bending with consideration of compression flange residual stress effects (in.)

$$= 1.95r_t\frac{E}{F_{yr}}\sqrt{\frac{J}{S_{xc}h}}\sqrt{1+\sqrt{1+6.76\left(\frac{F_{yr}}{E}\frac{S_{xc}h}{J}\right)^2}} \qquad \text{(A6.3.3-5)}$$

C_b = moment gradient modifier. In lieu of an alternative rational analysis, C_b may be calculated as follows:

- For unbraced cantilevers and for members where $M_{mid}/M_2 > 1$ or $M_2 = 0$

$$C_b = 1.0 \qquad \text{(A6.3.3-6)}$$

- For all other cases,

$$C_b = 1.75 - 1.05\left(\frac{M_1}{M_2}\right) + 0.3\left(\frac{M_1}{M_2}\right)^2 \leq 2.3 \qquad \text{(A6.3.3-7)}$$

F_{cr} = elastic lateral–torsional buckling stress (ksi)

$$=\frac{C_b\pi^2 E}{\left(L_b/r_t\right)^2}\sqrt{1+0.078\frac{J}{S_{xc}h}\left(L_b/r_t\right)^2} \qquad \text{(A6.3.3-8)}$$

J = St. Venant torsional constant (in.^{4})

$$=\frac{Dt^3{}_w}{3}+\frac{b_{fc}t^3{}_{fc}}{3}\left(1-0.63\frac{t_{fc}}{b_{fc}}\right)+\frac{b_{ft}t^3{}_{ft}}{3}\left(1-0.63\frac{t_{ft}}{b_{ft}}\right) \qquad \text{(A6.3.3-9)}$$

r_t = effective radius of gyration for lateral torsional buckling (in.)

order to reach the maximum flexural resistance, M_{max}, particularly if the moment gradient modifier, C_b, is taken equal to 1.0.

Eq. A6.3.3-8 gives the exact beam-theory based solution for the elastic lateral–torsional buckling of a doubly-symmetric I-section (Timoshenko and Gere, 1961) for the case of uniform major-axis bending when C_b is equal to 1.0 and when r_t is defined as specified by Eq. C6.10.8.2.3-1. Eq. A6.3.3-10 is a simplification of this r_t equation obtained by assuming $D = h = d$. For sections with thick flanges, Eq. A6.3.3-10 gives an r_t value that can be as much as three to four percent conservative relative to the exact equation. Use of Eq. C6.10.8.2.3-1 is permitted for software calculations or if the Engineer requires a more precise calculation of the elastic lateral–torsional buckling resistance. The format of Eq. A6.3.3-8 and the corresponding L_r limit of Eq. A6.3.3-5 are particularly convenient for design usage since the terms L_b, r_t, J, S_{xc}, and h are familiar and are easily calculated or can be readily obtained from design tables. Also, by simply setting J equal to zero, Eq. A6.3.3-8 reduces to the elastic lateral–torsional buckling resistance used in Article 6.10.8.2.3.

Eq. A6.3.3-8 also gives an accurate approximation of the exact beam-theory based solution for elastic lateral–torsional buckling of monosymmetric I-section members (White and Jung, 2003). For the case of $J > 0$ and uniform bending, and considering I-sections with $D/b_f > 2$, $b_{fc}/2t_{fc} > 5$ and $L_b = L_r$, the error in Eq. A6.3.3-8 relative to the exact beam-theory solution ranges from 12 percent conservative to two percent unconservative (White and Jung, 2003). A comparable I_{yc}-based equation in AASHTO (1998) gives maximum unconservative errors of approximately 14 percent for the same set of parameters studied. For the unusual case of a noncomposite compact or noncompact web section with $I_{yc}/I_{yt} > 1.5$ and $D/b_{fc} < 2$, $D/b_{ft} < 2$ or $b_{ft}/t_{ft} < 10$, consideration should be given to using the exact beam-theory equations (White and Jung, 2003) in order to obtain a more accurate solution, or else J from Eq. A6.3.3-9 may be factored by 0.8 to account for the tendency of Eq. A6.3.3-8 to overestimate the lateral–torsional buckling resistance in such cases. For highly monosymmetric I-sections with a smaller compression flange or for composite I-sections in negative flexure, both Eq. A6.3.3-8 and the prior I_{yc}-based equation in AASHTO (1998) are somewhat conservative compared to rigorous beam-theory based solutions. This is due to the fact that these equations do not account for the restraint against lateral buckling of the compression flange provided by the larger tension flange or the deck. However, the distorsional flexibility of the web significantly reduces this beneficial effect in many practical situations.

Eq. A6.3.3-9 is taken from El Darwish and Johnston (1965) and provides an accurate approximation of the St. Venant torsional constant, J, neglecting the effect of the web-to-flange fillets. For a compression or tension

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$$= \frac{b_{fc}}{\sqrt{12\left(1+\frac{1}{3}\frac{D_c t_w}{b_{fc} t_{fc}}\right)}} \qquad \text{(A6.3.3-10)}$$

where:

F_{yr} = compression flange stress at the onset of nominal yielding within the cross section, including residual stress effects, but not including compression flange lateral bending, taken as the smaller of $0.7F_{yc}$, $R_hF_{yt}\ S_{xt}/S_{xc}$ and F_{yw}, but not less than $0.5F_{yc}$

D_c = depth of the web in compression in the elastic range (in.). For composite sections, D_c shall be determined as specified in Article D6.3.1.

h = depth between the centerline of the flanges (in.)

M_{mid} = major-axis bending moment at the middle of the unbraced length, calculated from the moment envelope value that produces the largest compression at this point in the flange under consideration, or the smallest tension if this point is never in compression (kip-in.). M_{mid} shall be due to the factored loads and shall be taken as positive when it causes compression and negative when it causes tension in the flange under consideration.

M_0 = moment at the brace point opposite to the one corresponding to M_2, calculated from the moment envelope value that produces the largest compression at this point in the flange under consideration, or the smallest tension if this point is never in compression (kip-in.). M_0 shall be due to the factored loads and shall be taken as positive when it causes compression and negative when it causes tension in the flange under consideration.

M_1 = moment at the brace point opposite to the one corresponding to M_2, calculated as the intercept of the most critical assumed linear moment variation passing through M_2 and either M_{mid} or M_0, whichever produces the smaller value of C_b (kip-in.). M_1 may be calculated as follows:

flange with a ratio, $b_f/2t_f$, greater than 7.5, the term in parentheses given in Eq. A6.3.3-9 for that flange may be taken equal to one. Equations from El Darwish and Johnston (1965) that are employed in the calculation of AISC (2011) manual values for J and include the effect of the web-to-flange fillets are included in Seaburg and Carter (1997).

The Engineer should note the importance of the web term D_ct_w within Eq. A6.3.3-10. Prior Specifications have often used the radius of gyration of only the compression flange, $r_{yc} = b_{fc}/\sqrt{12}$, within design equations for lateral–torsional buckling. This approximation can lead to significant unconservative predictions relative to experimental and refined finite-element results. The web term in Eq. A6.3.3-10 accounts for the destabilizing effects of the flexural compression within the web.

The effect of the variation in the moment along the length between brace points is accounted for by using the moment gradient modifier, C_b. Article C6.10.8.2.3 discusses the C_b parameter in detail. Article 6.10.8.2.3 addresses unbraced lengths in which the member is nonprismatic. Article A6.3.3 extends the provisions for such unbraced lengths to members with compact and noncompact webs.

Where C_b is greater than 1.0, indicating the presence of a moment gradient, the lateral–torsional buckling resistances may alternatively be calculated by the equivalent procedures specified in Article D6.4.2. Both the equations in this Article and in Article D6.4.2 permit M_{max} in Figure C6.10.8.2.1-1 to be reached at larger unbraced lengths when C_b is greater than 1.0. The procedures in Article D6.4.2 allow the Engineer to focus directly on the maximum unbraced length at which the flexural resistance is equal to M_{max}. The use of these equivalent procedures is strongly recommended when C_b values greater than 1.0 are utilized in the design.

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- When the variation in the moment along the entire length between the brace points is concave in shape:

$$M_1 = M_0 \tag{A6.3.3-11}$$

- Otherwise:

$$M_1 = 2M_{mid} - M_2 \geq M_0 \tag{A6.3.3-12}$$

M_2 = except as noted below, largest major-axis bending moment at either end of the unbraced length causing compression in the flange under consideration, calculated from the critical moment envelope value (kip-in.). M_2 shall be due to the factored loads and shall be taken as positive. If the moment is zero or causes tension in the flange under consideration at both ends of the unbraced length, M_2 shall be taken as zero.
M_{yc} = yield moment with respect to the compression flange determined as specified in Article D6.2 (kip-in.)
M_{yt} = yield moment with respect to the tension flange determined as specified in Article D6.2 (kip-in.)
R_h = hybrid factor determined as specified in Article 6.10.1.10.1
R_{pc} = web plastification factor for the compression flange determined as specified in Article A6.2.1 or Article A6.2.2, as applicable
S_{xc} = elastic section modulus about the major axis of the section to the compression flange taken as M_{yc}/F_{yc} (in.^{3})
S_{xt} = elastic section modulus about the major axis of the section to the tension flange taken as M_{yt}/F_{yt} (in.^{3})

For unbraced lengths where the member consists of noncomposite monosymmetric sections and is subject to reverse curvature bending, the lateral–torsional buckling resistance shall be checked for both flanges, unless the top flange is considered to be continuously braced.

For unbraced lengths in which the member is nonprismatic, the flexural resistance based on lateral–torsional buckling may be taken as the smallest resistance within the unbraced length under consideration determined from Eq. A6.3.3-1, A6.3.3-2, or A6.3.3-3, as applicable, assuming the unbraced length is prismatic. The flexural resistance M_{nc} at each section within the unbraced length shall be taken equal to this resistance multiplied by the ratio of S_{xc} at the section under consideration to S_{xc} at the section governing the lateral–torsional buckling resistance. The moment gradient modifier, C_b, shall be taken equal to 1.0 in this case and L_b shall not be modified by an effective length factor.

For unbraced lengths containing a transition to a smaller section at a distance less than or equal to 20 percent of the unbraced length from the brace point with the smaller moment, the flexural resistance based on lateral–torsional buckling may be determined assuming the transition to the smaller section does not exist,

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provided the lateral moment of inertia of the flange or flanges of the smaller section is equal to or larger than one-half the corresponding value in the larger section.

A6.4—FLEXURAL RESISTANCE BASED ON TENSION FLANGE YIELDING

The nominal flexural resistance based on tension flange yielding shall be taken as:

$$M_{nt} = R_{pt} M_{yt} \tag{A6.4-1}$$

where:

M_{yt} = yield moment with respect to the tension flange determined as specified in Article D6.2 (kip-in.)

R_{pt} = web plastification factor for tension flange yielding determined as specified in Article A6.2.1 or Article A6.2.2, as applicable

CA6.4

Eq. A6.4-1 implements a linear transition in the flexural resistance between M_p and M_{yt} as a function of $2D_c/t_w$ for monosymmetric sections with a larger tension flange and for composite sections in negative flexure where first yielding occurs in the top flange or in the longitudinal reinforcing steel. In the limit that $2D_c/t_w$ approaches the noncompact web limit given by Eq. A6.2.2-3, Eq. A6.4-1 reduces to the tension flange yielding limit specified in Article 6.10.8.3.

For sections in which $M_{yt} > M_{yc}$, Eq. A6.4-1 does not control and need not be checked.

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APPENDIX B6—MOMENT REDISTRIBUTION FROM INTERIOR-PIER I-SECTIONS IN STRAIGHT CONTINUOUS-SPAN BRIDGES

B6.1—GENERAL

This Article shall apply for the calculation of redistribution moments from the interior-pier sections of continuous span I-section flexural members at the service and/or strength limit states. These provisions shall apply only for I-section members that satisfy the requirements of Article B6.2.

CB6.1

These optional provisions provide a simple rational approach for calculating the moment redistribution from interior-pier sections due to the effects of yielding. This approach utilizes elastic moment envelopes and does not require the direct use of any inelastic analysis methods. The restrictions of Article B6.2 ensure significant ductility and robustness at the interior-pier sections.

In conventional elastic analysis and design, moment and shear envelopes are typically determined by elastic analysis with no redistribution due to the effects of yielding considered. The sections are dimensioned for a resistance equal to or greater than that required by the envelopes. Designs to meet these requirements often involve the addition of cover plates to rolled beams, which introduces details that often have low fatigue resistance, or the introduction of multiple flange transitions in welded beams, which can result in additional fabrication costs. Where appropriate, the use of these provisions to account for the redistribution of moments makes it possible to eliminate such details by using prismatic sections along the entire length of the bridge or between field splices. This practice can improve overall fatigue resistance and provide significant fabrication economies.

Development of these provisions is documented in a number of comprehensive reports (Barker et al., 1997) (Schilling et al., 1997) (White et al., 1997) and in a summary paper by Barth et al. (2004), which gives extensive references to other supporting research. These provisions account for the fact that the compression flange slenderness, $b_{fc}/2t_{fc}$, and the cross-section aspect ratio, D/b_{fc}, are the predominant factors that influence the moment-rotation behavior at adequately braced interior-pier sections. The provisions apply to sections with compact, noncompact, or slender webs.

B6.2—SCOPE

Moment redistribution shall be applied only in straight continuous span I-section members whose support lines are not skewed more than 10 degrees from normal and along which there are no discontinuous cross-frames. Sections may be either composite or noncomposite in positive or negative flexure.

Cross sections throughout the unbraced lengths immediately adjacent to interior-pier sections from which moments are redistributed shall have a specified minimum yield strength not exceeding 70.0 ksi. Holes shall not be placed within the tension flange over a distance of two times the web depth on either side of the interior-pier sections from which moments are redistributed. All other sections having tension flange holes shall satisfy the requirements of Article 6.10.1.8 after the moments are redistributed.

CB6.2

The subject procedures have been developed predominantly in the context of straight nonskewed bridge superstructures without discontinuous cross-frames. Therefore, their use is restricted to bridges that do not deviate significantly from these idealized conditions.

The development of these provisions focused on nonhybrid and hybrid girders with specified minimum yield strengths up to and including 70.0 ksi. Therefore, use of these provisions with larger yield strengths is not permitted. The influence of tension-flange holes on potential net section fracture at cross sections experiencing significant inelastic strains is not well-known. Therefore, tension flange holes are not allowed over a distance of two times the web depth, D, on either side of the interior-pier sections from which moments are

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Moments shall be redistributed only at interior-pier sections for which the cross sections throughout the unbraced lengths immediately adjacent to those sections satisfy the requirements of Articles B6.2.1 through B6.2.6. If the refined method of Article B6.6 is used for calculation of the redistribution moments, all interior-pier sections are not required to satisfy these requirements; however, moments shall not be redistributed from sections that do not satisfy these requirements. Such sections instead shall satisfy the provisions of Articles 6.10.4.2, 6.10.8.1 or Article A6.1, as applicable, after redistribution. If the provisions of Articles B6.3 or B6.4 are utilized to calculate interior-pier redistribution moments, the unbraced lengths immediately adjacent to all interior-pier sections shall satisfy the requirements of Articles B6.2.1 through B6.2.6.

redistributed. The distance $2D$ is an approximate upper bound for the length of the zone of primary inelastic response at these pier sections.

Unless a direct analysis is conducted by the Refined Method outlined in Article B6.6, all the interior-pier sections of a continuous-span member are required to satisfy the requirements of Articles B6.2.1 through B6.2.6 in order to redistribute the pier moments. This is because of the approximations involved in the simplified provisions of Articles B6.3 and B6.4 and the fact that inelastic redistribution moments from one interior support generally produce some nonzero redistribution moments at all of the interior supports.

B6.2.1—Web Proportions

The web within the unbraced length under consideration shall be proportioned such that:

$$\frac{D}{t_w} \leq 150 \qquad \text{(B6.2.1-1)}$$

$$\frac{2D_c}{t_w} \leq 6.8\sqrt{\frac{E}{F_{yc}}} \qquad \text{(B6.2.1-2)}$$

and:

$$D_{cp} \leq 0.75D \qquad \text{(B6.2.1-3)}$$

where:

D_c = depth of the web in compression in the elastic range (in.). For composite sections, D_c shall be determined as specified in Article D6.3.1.

D_{cp} = depth of the web in compression at the plastic moment determined as specified in Article D6.3.2 (in.)

CB6.2.1

Eq. B6.2.1-1 simply parallels Eq. 6.10.2.1.1-1 and is intended to eliminate the use of any benefits from longitudinal stiffening of the web at the pier section. The moment-rotation characteristics of sections with longitudinal web stiffeners have not been studied. Eqs. B6.2.1-2 and B6.2.1-3 are limits of the web slenderness and the depth of the web in compression considered in the development of these procedures.

B6.2.2—Compression Flange Proportions

The compression flange within the unbraced length under consideration shall be proportioned such that:

$$\frac{b_{fc}}{2t_{fc}} \leq 0.38\sqrt{\frac{E}{F_{yc}}} \qquad \text{(B6.2.2-1)}$$

and:

$$b_{fc} \geq \frac{D}{4.25} \qquad \text{(B6.2.2-2)}$$

CB6.2.2

The compression flange is required to satisfy the compactness limit within the unbraced lengths adjacent to the pier section. This limit is restated in Eq. B6.2.2-1. Slightly larger $b_{fc}/2t_{fc}$ values than this limit have been considered within the supporting research for these provisions. The compactness limit from Articles A6.3.2 and 6.10.8.2 is used for simplicity.

Eq. B6.2.2-2 represents the largest aspect ratio $D/b_{fc} = 4.25$ considered in the supporting research. As noted in Articles C6.10.2.2 and CB6.1, increasing values of this ratio have a negative influence on the strength and moment-rotation characteristics of I-section members.

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B6.2.3—Section Transitions

The steel I-section member shall be prismatic within the unbraced length under consideration.

CB6.2.3

Only members that are prismatic within the unbraced lengths adjacent to interior piers have been considered in the supporting research. Therefore, section transitions are prohibited in these regions.

B6.2.4—Compression Flange Bracing

The unbraced length under consideration shall satisfy:

$$L_b \leq \left[0.1 - 0.06\left(\frac{M_1}{M_2}\right)\right]\frac{r_t E}{F_{yc}} \quad \text{(B6.2.4-1)}$$

where:

L_b = unbraced length (in.)

M_1 = bending moment about the major-axis of the cross section at the brace point with the lower moment due to the factored loads, taken as either the maximum or minimum moment envelope value, whichever produces the smallest permissible unbraced length (kip-in.)

M_2 = bending moment about the major-axis of the cross section at the brace point with the higher moment due to the factored loads, taken as the critical moment envelope value (kip-in.)

r_t = effective radius of gyration for lateral–torsional buckling within the unbraced length under consideration determined from Eq. A6.3.3-10 (in.)

(M_1/M_2) shall be taken as negative when the moments cause reverse curvature.

CB6.2.4

Eq. B6.2.4-1 gives approximately the same results as the compact-section compression flange bracing requirements in Article 6.10.4.1.7 of AASHTO (1998), but is written in terms of r_t rather than r_y. The use of r_y in the prior equation leads to an ambiguity in the application of this bracing limit to composite sections in negative flexure. Furthermore, since r_t focuses strictly on the compression region of the cross section and does not involve the top flange or the deck for a composite section in negative flexure, it is believed to address the bracing requirements for such a section in a more correct fashion.

Since the negative moment envelope always tends to be concave in shape in the vicinity of interior-pier sections, the consideration of the moment values at the middle of the unbraced length, as required in general for the calculation of C_b in Articles 6.10.8.2.3 and A6.3.3, is not necessary. Consideration of the moment gradient effects based on the ratio of the end values, M_1/M_2, is sufficient and conservative.

If $D_c t_w / b_{fc} t_{fc}$ in Eq. 6.10.8.2.3-9 or A6.3.3-10 is taken as a representative value of 2.0 and F_{yc} is taken as 50 ksi, Eq. B6.2.4-1 is satisfied when $L_b < 13b_{fc}$ for $M_1/M_2 = 0$ and $L_b < 9b_{fc}$ for $M_1/M_2 = 0.5$.

B6.2.5—Shear

Webs with or without transverse stiffeners within the unbraced length under consideration shall satisfy the following requirement at the strength limit state:

$$V_u \leq \phi_v V_{cr} \quad \text{(B6.2.5-1)}$$

where:

ϕ_v = resistance factor for shear specified in Article 6.5.4.2

V_u = shear in the web due to the factored loads (kip)

V_{cr} = shear-buckling resistance determined from Eq. 6.10.9.2-1 for unstiffened webs and from Eq. 6.10.9.3.3-1 for stiffened webs (kip)

CB6.2.5

Use of web shear post-buckling resistance or tension-field action is not permitted within the vicinity of the pier sections designed for redistribution of the negative bending moments.

B6.2.6—Bearing Stiffeners

Bearing stiffeners designed by the provisions of Article 6.10.11.2 shall be placed at the interior-pier section under consideration.

B6.3—SERVICE LIMIT STATE

B6.3.1—General

Load combination Service II in Table 3.4.1-1 shall apply.

B6.3.2—Flexure

B6.3.2.1—Adjacent to Interior-Pier Sections

With the exception that the requirement of Eq. 6.10.4.2.2-4 shall be satisfied, the provisions of Article 6.10.4.2 shall not be checked within the regions extending in each adjacent span from interior-pier sections satisfying the requirements of Article B6.2 to the nearest flange transition or point of dead-load contraflexure, whichever is closest.

CB6.3.2.1

In checking permanent deflections under Load Combination Service II, local yielding is permitted at interior supports satisfying the requirements of Article B6.2. This results in redistribution. The permanent deflections are controlled by imposing the appropriate flange stress limits of Article 6.10.4.2 in each adjacent span at sections outside the nearest flange transition location or point of permanent-load contraflexure, whichever is closest to the interior support under consideration, after redistribution. The appropriate redistribution moments are to be added to the elastic moments due to the Service II loads prior to making these checks. The influence of the strength and ductility at the interior-pier sections is considered within the calculation of the redistribution moments. Therefore, the flange stress limits of Article 6.10.4.2 need not be checked within the regions extending into each adjacent span from the interior-pier section under consideration to the closest point cited above. The provisions of Appendix B6 are not intended to relax the requirement of Eq. 6.10.4.2.2-4. This requirement should be satisfied based on the elastic moments before redistribution.

Additional cambering to account for the small residual deformations associated with redistribution of interior-pier section moments is not recommended. A full-scale bridge designed to permit redistribution of negative moments sustained only very small permanent deflections when tested under the overload condition (Roeder and Eltvik, 1985).

B6.3.2.2—At All Other Locations

Sections at all other locations shall satisfy the provisions of Article 6.10.4.2, as applicable, after redistribution. For composite sections in positive flexure, the redistribution moments shall be applied to the long-term composite section when computing flexural stresses in the steel section. For computing longitudinal flexural stresses in the concrete deck due to the redistribution

CB6.3.2.2

The redistribution moments are in effect permanent moments that remain in the structure. The corresponding locked-in redistribution stresses in composite sections tend to decrease with time as a result of creep in the concrete. However, these redistribution stresses may be continually renewed by subsequent passages of similar loadings. Therefore, the flexural stresses moments, the provisions of Article 6.10.1.1.1d shall apply.

The redistribution moments shall be calculated according to the provisions specified in Article B6.3.3 and shall be added to the elastic moments due to the Service II loads.

in the steel section due to these moments are to be conservatively calculated based on the long-term composite section.

B6.3.3—Redistribution Moments

B6.3.3.1—At Interior-Pier Sections

At each interior-pier section where the flexural stresses are not checked as permitted in Article B6.3.2.1, the redistribution moment for the Service II loads shall be taken as:

$$M_{rd} = \left|M_e\right| - M_{pe} \qquad \text{(B6.3.3.1-1)}$$

in which:

$$0 \le M_{rd} \le 0.2\left|M_e\right| \qquad \text{(B6.3.3.1-2)}$$

where:

M_{pe} = negative-flexure effective plastic moment for the service limit state determined as specified in Article B6.5 (kip-in.)

M_e = critical elastic moment envelope value at the interior-pier section due to the Service II loads (kip-in.)

CB6.3.3.1

Eqs. B6.3.3.1-1, B6.4.2.1-1 and B6.4.2.1-2 are based on concepts from shakedown analysis of continuous-span girders under repeated application of moving loads (ASCE, 1971) (Schilling et al., 1997) using an effective plastic moment that accounts for the interior-pier section moment-rotation characteristics. Shakedown is the appropriate limit state related to moment redistribution in bridges (Galambos et al., 1993).

At the service limit state, the effective plastic moment in Eq. B6.3.3.1-1 is based on an estimated upper-bound plastic rotation of 0.009 radians at the pier sections, determined by direct inelastic analysis of various trial designs (Schilling, 1986). Flange lateral bending effects are not considered in Eq. B6.3.3.1-1 since due to the restrictions of Article B6.2, the flange lateral bending effects at the interior supports under the Service II Load Combination are taken to be negligible. The refinement of these calculations by consideration of flange lateral bending effects is considered unjustified.

Eq. B6.3.3.1-2 is intended to prevent the use of an interior-pier section that is so small that it could potentially violate the assumed upper-bound inelastic rotation of 0.009 radians under Service II conditions. Note that if the upper limit of Eq. B6.3.3.1-2 is violated, a new interior-pier section must be selected that will ensure that this limit is satisfied.

B6.3.3.2—At All Other Locations

The redistribution-moment diagram for the Service II load combination shall be determined by connecting with straight lines the redistribution moments at adjacent interior-pier sections. The lines shall be extended to any points of zero redistribution moment at adjacent supports, including at the abutments.

CB6.3.3.2

Figure CB6.3.3.2-1 illustrates a typical redistribution moment diagram for a three-span continuous member for which the redistribution moments are greater than zero at both interior-pier sections. After the live loads are removed, the redistribution moments are held in equilibrium by the support reactions. Therefore, the redistribution moments must vary linearly between the supports.

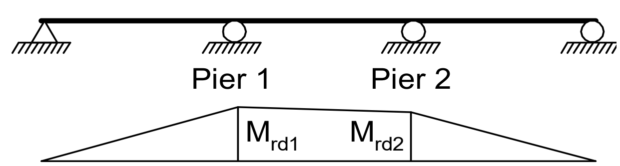

Figure CB6.3.3.2-1—Typical Redistribution Moment Diagram

B6.4—STRENGTH LIMIT STATE

B6.4.1—Flexural Resistance

B6.4.1.1—Adjacent to Interior-Pier Sections

The flexural resistances of sections within the unbraced lengths immediately adjacent to interior-pier sections satisfying the requirements of Article B6.2 shall not be checked.

CB6.4.1.1

Yielding is permitted at interior supports at the strength limit state, and results in redistribution of moments. The influence of the strength and ductility at the interior-pier sections is considered within the calculation of the redistribution moments. Therefore, the flexural resistances of sections within the unbraced lengths immediately adjacent to interior-pier sections from which moments are redistributed need not be checked.

B6.4.1.2—At All Other Locations

Sections at all other locations shall satisfy the provisions of Articles 6.10.7, 6.10.8.1, or A6.1, as applicable, after redistribution. For composite sections in positive flexure, the redistribution moments shall be applied to the long-term composite section when computing flexural stresses in the steel section. For computing longitudinal flexural stresses in the concrete deck due to the redistribution moments, the provisions of Article 6.10.1.1.1d shall apply.

The redistribution moments shall be calculated using the provisions of Article B6.4.2 and shall be added to the elastic moments due to the factored loads at the strength limit state.

CB6.4.1.2

Regions outside of unbraced lengths immediately adjacent to interior-pier sections from which moments are redistributed are designed in the same fashion as when the procedures of this Article are not applied, with the exception that the appropriate redistribution moments are to be added to the elastic moments due to the factored loads at the strength limit state prior to making the design checks.

B6.4.2—Redistribution Moments

B6.4.2.1—At Interior-Pier Sections

At each interior-pier section where the flexural resistances are not checked as permitted in Article B6.4.1.1, the redistribution moment at the strength limit state shall be taken as the larger of:

$$M_{rd} = \left| M_e \right| + \frac{1}{3} f_\ell S_{xc} - \phi_f M_{pe} \quad \text{(B6.4.2.1-1)}$$

or:

$$M_{rd} = \left| M_e \right| + \frac{1}{3} f_\ell S_{xt} - \phi_f M_{pe} \quad \text{(B6.4.2.1-2)}$$

in which:

$$0 \le M_{rd} \le 0.2 \left| M_e \right| \quad \text{(B6.4.2.1-3)}$$

where:

f_ℓ = lateral bending stress in the flange under consideration at the interior-pier section (ksi).

CB6.4.2.1

At the strength limit state, the effective plastic moment in Eqs. B6.4.2.1-1 and B6.4.2.1-2 is based on an estimated upper bound plastic rotation of 0.03 radians at the pier sections, determined by direct inelastic analysis of various trial designs (Schilling, 1986).

Flange lateral bending effects are conservatively included in Eqs. B6.4.2.1-1 and B6.4.2.1-2 to account for the reduction in the flexural resistance of the interior-pier section at the strength limit state due to these effects. The inclusion of f_ℓ in these equations is intended primarily to address the design for wind loads. Eq. B6.4.2.1-3 is intended to prevent the use of an interior-pier section that is so small that it could potentially violate the assumed upper-bound inelastic rotation of 0.03 radians at the strength limit state. Note that if the upper limit of Eq. B6.4.2.1-3 is violated, a new interior-pier section must be selected that will ensure that this limit is satisfied.

A form of Eqs. B6.4.2.1-1 and B6.4.2.1-2 was proposed in the original research by Barker et al. (1997) that included a resistance factor for shakedown of ϕ_{sd} = 1.1. The resistance factor of ϕ_{sd} = 1.1 is justified for this limit state because the shakedown loading is

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For continuously braced tension or compression flanges, f_ℓ shall be taken as zero.

ϕ_f = resistance factor for flexure specified in Article 6.5.4.2

M_{pe} = negative-flexure effective plastic moment for the strength limit state determined as specified in Article B6.5 (kip-in.)

M_e = critical elastic moment envelope value at the interior-pier section due to the factored loads (kip-in.)

M_{yc} = yield moment with respect to the compression flange determined as specified in Article D6.2 (kip-in.)

M_{yt} = yield moment with respect to the tension flange determined as specified in Article D6.2 (kip-in.)

S_{xc} = elastic section modulus about the major axis of the section to the compression flange taken as M_{yc}/F_{yc} (in.^{3})

S_{xt} = elastic section modulus about the major axis of the section to the tension flange taken as M_{yt}/F_{yt} (in.^{3})

generally less than the maximum plastic resistance and because progressively increasing permanent deflections give ample warning of pending failure. The resistance factor for flexure ϕ_f of Article 6.5.4.2 is selected in these provisions to account for the fact that yielding within regions of positive flexure and the corresponding redistribution of positive bending moments to the interior-pier sections is not considered. Also, as discussed in Article C6.10.7.1.2, additional requirements are specified in continuous spans where significant yielding may occur prior to reaching the section resistances of compact sections in positive flexure.

B6.4.2.2—At All Other Sections

The redistribution-moment diagram for the strength limit state shall be determined using the same procedure specified for the Service II load combination in Article B6.3.3.2.

CB6.4.2.2

Figure CB6.3.3.2-1 illustrates a typical redistribution moment diagram.

B6.5—EFFECTIVE PLASTIC MOMENT

B6.5.1—Interior-Pier Sections with Enhanced Moment-Rotation Characteristics

For interior-pier sections satisfying the requirements of Article B6.2 and which contain:

- Transverse stiffeners spaced at $D/2$ or less over a minimum distance of $D/2$ on each side of the interior-pier section

or:

- Ultracompact webs that satisfy:

$$\frac{2D_{cp}}{t_w} \leq 2.3\sqrt{\frac{E}{F_{yc}}} \qquad \text{(B6.5.1-1)}$$

where:

D_{cp} = depth of the web in compression at the plastic moment determined as specified in Article D6.3.2 (in.)

the effective plastic moment shall be taken as:

- For the Service Limit State:

CB6.5.1

Tests have shown that members with interior-pier sections that satisfy either of the requirements of this Article, in addition to the requirements of Article B6.2, exhibit enhanced moment-rotation characteristics relative to members that satisfy only the requirements of Article B6.2 (White et al., 1997) (Barth et al., 2004). These additional requirements involve the use of:

- Transverse stiffeners close to the interior-pier section to help restrain the local buckling distortions of the web and compression flange within this region, or
- A web that is sufficiently stocky such that its distortions are reduced and the flange local buckling distortions are highly restrained, termed an ultracompact web.

For noncompact web and slender web sections, the influence of the web slenderness on the effective plastic moment is captured through the maximum flexural resistance term M_n in Eqs. B6.5.1-2 and B6.5.1-3, and in Eqs. B6.5.2-1 and B6.5.2-2.

$$M_{pe} = M_n \quad \text{(B6.5.1-2)}$$

- For the Strength Limit State:

$$M_{pe} = \left(\begin{array}{l} 2.78 - 2.3 \dfrac{b_{fc}}{t_{fc}} \sqrt{\dfrac{F_{yc}}{E}} \\ -0.35 \dfrac{D}{b_{fc}} + 0.39 \dfrac{b_{fc}}{t_{fc}} \sqrt{\dfrac{F_{yc}}{E}} \dfrac{D}{b_{fc}} \end{array} \right) M_n \le M_n \quad \text{(B6.5.1-3)}$$

where:

M_n = nominal flexural resistance of the interior-pier section taken as the smaller of $F_{nc}S_{xc}$ and $F_{nt}S_{xt}$, with F_{nc} and F_{nt} determined as specified in Article 6.10.8. For sections with compact or noncompact webs, M_n may be taken as the smaller of M_{nc} and M_{nt} determined as specified in Appendix A6 (kip-in.).

B6.5.2—All Other Interior-Pier Sections

For interior-pier sections satisfying the requirements of Article B6.2, but not satisfying the requirements of Article B6.5.1, the effective plastic moment shall be taken as:

- For the Service Limit State:

$$M_{pe} = \left(\begin{array}{l} 2.90 - 2.3 \dfrac{b_{fc}}{t_{fc}} \sqrt{\dfrac{F_{yc}}{E}} \\ -0.35 \dfrac{D}{b_{fc}} + 0.39 \dfrac{b_{fc}}{t_{fc}} \sqrt{\dfrac{F_{yc}}{E}} \dfrac{D}{b_{fc}} \end{array} \right) M_n \le M_n \quad \text{(B6.5.2-1)}$$

- For the Strength Limit State:

$$M_{pe} = \left(\begin{array}{l} 2.63 - 2.3 \dfrac{b_{fc}}{t_{fc}} \sqrt{\dfrac{F_{yc}}{E}} \\ -0.35 \dfrac{D}{b_{fc}} + 0.39 \dfrac{b_{fc}}{t_{fc}} \sqrt{\dfrac{F_{yc}}{E}} \dfrac{D}{b_{fc}} \end{array} \right) M_n \le M_n \quad \text{(B6.5.2-2)}$$

CB6.5.2

Eqs. B6.5.2-1 and B6.5.2-2 are based on a lower-bound estimate of the moment-rotation characteristics of interior-pier sections that satisfy the limits of Article B6.2 (Barth et al., 2004). Cases with unbraced lengths smaller than the limit given by Eq. B6.2.4-1, significant torsional restraint from a composite deck, and/or compression flange slenderness values significantly smaller than the compact flange limit often exhibit significantly enhanced moment-rotation characteristics and corresponding larger effective plastic moments than the values obtained from these equations.

The web slenderness, $2D_c/t_w$ or $2D_{cp}/t_w$, does not appear directly in Eqs. B6.5.2-1 and B6.5.2-2. For noncompact and slender web sections, the influence of the web slenderness on the effective plastic moment is captured through the maximum flexural resistance term M_n.

B6.6—REFINED METHOD

B6.6.1—General

Continuous span I-section flexural members satisfying the requirements of Article B6.2 also may be proportioned based on a direct analysis. In this approach, the

CB6.6.1

The Engineer is also provided the option to use a refined method in which a direct shakedown analysis is conducted at the service and/or strength limit states. This redistribution moments shall be determined by satisfying rotational continuity and specified inelastic moment-rotation relationships at selected interior-pier sections. Direct analysis may be employed at the service and/or strength limit states. The elastic moment envelope due to the factored loads shall be used in this analysis.

For the direct analysis, the redistribution moments shall be determined using the elastic stiffness properties of the short-term composite section assuming the concrete deck to be effective over the entire span length. For composite sections in positive flexure, the redistribution moments shall be applied to the long-term composite section when computing elastic flexural stresses in the steel section. For computing elastic longitudinal flexural stresses in the concrete deck due to the redistribution moments, the provisions of Article 6.10.1.1.1d shall apply.

Sections adjacent to interior piers from which moments are redistributed shall satisfy the requirements of Article B6.3.2.1 at the service limit state and Article B6.4.1.1 at the strength limit state. All other sections shall satisfy all applicable provisions of Articles 6.10.4.2, 6.10.7, 6.10.8.1, or A6.1 after a solution is found.

In applying direct analysis at the strength limit state, the ordinates of the nominal moment-rotation curves shall be multiplied by the resistance factor for flexure specified in Article 6.5.4.2. In applying direct analysis at the Service II limit state, the nominal moment-rotation curves shall be used.

analysis requires the simultaneous satisfaction of continuity and moment-rotation relationships at all interior-pier sections from which moments are redistributed. If software that handles this type of calculation along with the determination of the elastic moment envelopes does not exist, significant manual work is required in conducting the analysis calculations. The Engineer can gain some additional benefit when using direct analysis in that the restriction that all interior-pier sections within the member satisfy the requirements of Article B6.2.1 is relaxed. Also, the directly calculated inelastic rotations at the interior pier sections will tend to be smaller than the upper-bound values that the equations in Articles B6.3 through B6.5 are based upon.

The redistribution moments are to be computed using the stiffness properties of the short-term composite section because the redistribution moments are formed by short-term loads.

Although direct analysis methods can be formulated that account for the redistribution of moments from regions of positive flexure, there is typically no significant economic benefit associated with redistribution of positive bending moments. This is because, in most practical cases, the interior-pier sections have the highest elastic stresses. Also, the development of some inelastic rotations at the pier sections simply allows a continuous-span member to respond in a fashion involving only slightly less rotational restraint from the adjacent spans than if these sections remain elastic.

With the exception of the additional requirements of Article 6.10.7.1.2 for composite sections subjected to positive flexure within continuous spans in which the adjacent interior-pier sections do not satisfy Article B6.2, these Specifications generally neglect the influence of partial yielding prior to and associated with the development of member maximum flexural resistances. Therefore, the influence of partial yielding within regions of positive flexure on the redistribution of moments to the interior piers and on the calculated inelastic pier rotations is also to be neglected within the direct analysis approach. The unconservative attributes associated with neglecting positive-moment yielding prior to reaching the maximum flexural resistance within regions of positive flexure are offset by:

- the use of $\phi_f = 1.0$ rather than a shakedown resistance factor of $\phi_{sd} = 1.1$ as originally formulated by Barker et al. (1997) and discussed in Article CB6.4.2.1, and
- the lower-bound nature of the moment-rotation relationships utilized for the interior-pier sections.

Moment-rotation relationships have been proposed that account for yielding in positive flexure, such as in Barker et al. (1997). However, these relationships account in only a very simplistic fashion for the distributed yielding effects that tend to occur over a significant length due to the small moment gradients that typically exist within regions

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of positive flexure. Significantly greater accuracy can be achieved in the analysis for these effects by the use of distributed plasticity analysis models rather than plastic-hinge type models. However, these types of analysis models are not readily accessible to the Engineer at the present time.

B6.6.2—Nominal Moment-Rotation Curves

At interior-pier sections that satisfy the requirements of Article B6.2, the nominal moment-rotation curve given in Figure B6.6.2-1 may be used.

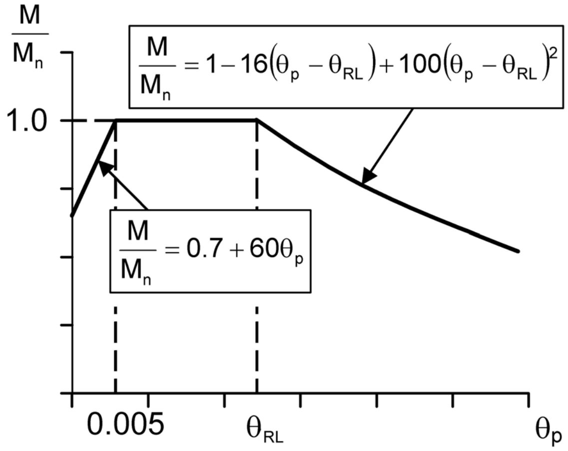

Figure B6.6.2-1—Nominal Moment-Rotation Curve for Interior-Pier Sections Satisfying Article B6.2

in which:

θ_{RL} = plastic rotation at which the interior-pier section moment nominally begins to decrease with increasing θ_p (radians)

$$= 0.137 - 0.143\frac{b_{fc}}{t_{fc}}\sqrt{\frac{F_{yc}}{E}} - 0.0216\frac{D}{b_{fc}} + 0.0241\frac{D}{b_{fc}}\frac{b_{fc}}{t_{fc}}\sqrt{\frac{F_{yc}}{E}} \quad \text{(B6.6.2-1)}$$

for sections that satisfy the additional requirements specified in Article B6.5.1, and:

$$= 0.128 - 0.143\frac{b_{fc}}{t_{fc}}\sqrt{\frac{F_{yc}}{E}} - 0.0216\frac{D}{b_{fc}} + 0.0241\frac{D}{b_{fc}}\frac{b_{fc}}{t_{fc}}\sqrt{\frac{F_{yc}}{E}} \quad \text{(B6.6.2-2)}$$

for all other sections.

where:

θ_p = plastic rotation at the interior-pier section (rad.)
M = bending moment about the major-axis of the cross section due to the factored loads (kip-in.)
M_n = nominal flexural resistance of the interior-pier section taken as the smaller of $F_{nc}S_{xc}$ and $F_{nt}S_{xt}$, with F_{nc} and F_{nt} determined as specified in

CB6.6.2

The moment-rotation relationships in this Article are developed in White et al. (1997) and Barth et al. (2004). The moment-rotation relationships for interior-pier sections with enhanced moment-rotation characteristics that satisfy the additional limits of Article B6.5.1 are given by Eq. B6.6.2-1, which is obtained by replacing the coefficient 0.128 in Eq. B6.6.2-2 by 0.137 (Barth et al., 2004). It is expected that exceeding the limits of Article B6.2 may result in substantial degradation of the interior-pier moment-rotation characteristics. Therefore, the restrictions of Article B6.2 may not be relaxed by use of alternative moment-rotation relationships.

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Article 6.10.8 (kip-in.). For sections with compact or noncompact webs, M_n may be taken as the smaller of M_{nc} and M_{nt} determined as specified in Appendix A6. For load combinations that induce significant flange lateral bending stresses, the influence of flange lateral bending shall be considered by deducting the larger of $\frac{1}{3}f_\ell S_{xc}$ or $\frac{1}{3}f_\ell S_{xt}$ from the above values.

f_ℓ = lateral bending stress in the flange under consideration at the interior-pier section (ksi). For continuously braced tension or compression flanges, f_ℓ shall be taken as zero.

Other nominal moment-rotation relationships may be employed for interior-pier sections that satisfy the requirements of Article B6.2 provided that the relationships are developed considering all potential factors that influence the moment-rotation characteristics within the restrictions of those requirements.

Interior-pier sections not satisfying the requirements of Article B6.2 shall be assumed to remain elastic in the analysis, and shall satisfy the provisions of Articles 6.10.4.2, 6.10.8.1, or Article A6.1, as applicable, after a solution is found.

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APPENDIX C6—BASIC STEPS FOR STEEL BRIDGE SUPERSTRUCTURES

C6.1—GENERAL

This outline is intended to be a generic overview of the design process. It should not be regarded as fully complete, nor should it be used as a substitute for a working knowledge of the provisions of this section.

C6.2—GENERAL CONSIDERATIONS

A. Design Philosophy (1.3.1)
B. Limit States (1.3.2)
C. Design and Location Features (2.3) (2.5)

C6.3—SUPERSTRUCTURE DESIGN

A. Develop General Section
    1. Roadway Width (Highway Specified)
    2. Span Arrangements (2.3.2) (2.5.4) (2.5.5) (2.6)
    3. Select Bridge Type—assumed to be I- or Box Girder
B. Develop Typical Section
    1. I-girder
        a. Composite (6.10.1.1) or Noncomposite (6.10.1.2)
        b. Hybrid or Nonhybrid (6.10.1.3)
        c. Variable Web Depth (6.10.1.4)
        d. Cross-section Proportion Limits (6.10.2)
    2. Box Girder
        a. Multiple Boxes or Single Box (6.11.1.1) (6.11.2.3)
        b. Hybrid or Nonhybrid (6.10.1.3)
        c. Variable Web Depth (6.10.1.4)
        d. Cross-section Proportion Limits (6.11.2)
        e. Bearings (6.11.1.2)
        f. Orthotropic Deck (6.14.3)
C. Design Conventionally Reinforced Concrete Deck
    1. Deck Slabs (4.6.2.1)
    2. Minimum Depth (9.7.1.1)
    3. Empirical Design (9.7.2)
    4. Traditional Design (9.7.3)
    5. Strip Method (4.6.2.1)
    6. Live Load Application (3.6.1.3.3) (4.6.2.1.4) (4.6.2.1.5)
    7. Distribution Reinforcement (9.7.3.2)
    8. Overhang Design (A13.4) (3.6.1.3.4)
    9. Minimum Negative Flexure Concrete Deck Reinforcement (6.10.1.7)
D. Select Resistance Factors
    1. Strength Limit State (6.5.4.2)
E. Select Load Modifiers
    1. Ductility (1.3.3)
    2. Redundancy (1.3.4)
    3. Operational Importance (1.3.5)
F. Select Load Combinations and Load Factors (3.4.1)
    1. Strength Limit State (6.5.4.1) (6.10.6.1) (6.11.6.1)
    2. Service Limit State (6.10.4.2.1)
    3. Fatigue and Fracture Limit State (6.5.3)
G. Calculate Live Load Force Effects
    1. Select Live Loads (3.6.1) and Number of Lanes (3.6.1.1.1)
    2. Multiple Presence (3.6.1.1.2)
    3. Dynamic Load Allowance (3.6.2)
    4. Distribution Factor for Moment (4.6.2.2.2)
        a. Interior Beams with Concrete Decks (4.6.2.2.2b)

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b. Exterior Beams (4.6.2.2.2d)
c. Skewed Bridges (4.6.2.2.2e)
5. Distribution Factor for Shear (4.6.2.2.3)
a. Interior Beams (4.6.2.2.3a)
b. Exterior Beams (4.6.2.2.3b)
c. Skewed Bridges (4.6.2.2.3c)
6. Stiffness (6.10.1.5)
7. Wind Effects (4.6.2.7)
8. Reactions to Substructure (3.6)
H. Calculate Force Effects From Other Loads Identified in Step C6.3.F
I. Design Required Sections—Illustrated for Design of I-Girder
1. Flexural Design
a. Composite Section Stresses (6.10.1.1.1)
b. Flange Stresses and Member Bending Moments (6.10.1.6)
c. Fundamental Section Properties (D6.1) (D6.2) (D6.3)
d. Constructibility (6.10.3)
(1) General (2.5.3) (6.10.3.1)
(2) Flexure (6.10.3.2) (6.10.1.8) (6.10.1.9) (6.10.1.10.1) (6.10.8.2) (A6.3.3—optional)
(3) Shear (6.10.3.3)
(4) Deck Placement (6.10.3.4)
(5) Dead Load Deflections (6.10.3.5)
e. Service Limit State (6.5.2) (6.10.4)
(1) Elastic Deformations (6.10.4.1)
(a) Optional Live-load Deflection Control (2.5.2.6.2)
(b) Optional Criteria for Span-to-Depth Ratios (2.5.2.6.3)
(2) Permanent Deformations (6.10.4.2)
(a) General (6.10.4.2.1)
(b) Flexure (6.10.4.2.2) (Appendix B6—optional) (6.10.1.9) (6.10.1.10.1)
f. Fatigue and Fracture Limit State (6.5.3) (6.10.5)
(1) Fatigue (6.10.5.1) (6.6.1)
(2) Fracture (6.10.5.2) (6.6.2)
(3) Special Fatigue Requirement for Webs (6.10.5.3)
g. Strength Limit State (6.5.4) (6.10.6)
(1) Composite Sections in Positive Flexure (6.10.6.2.2) (6.10.7)
(2) Composite Sections in Negative Flexure and Noncomposite Sections (6.10.6.2.3) (6.10.8) (Appendix A6—optional) (Appendix B6—optional) (D6.4—optional)
(3) Net Section Fracture (6.10.1.8)
(4) Flange-strength Reduction Factors (6.10.1.10)
2. Shear Design
a. General (6.10.9.1)
b. Unstiffened Web (6.10.9.2)
c. Stiffened Web (6.10.9.3)
(1) General (6.10.9.3.1)
(2) Interior Panels (6.10.9.3.2)
(3) End Panels (6.10.9.3.3)
d. Stiffener Design (6.10.11)
(1) Transverse Intermediate Stiffeners (6.10.11.1)
(2) Bearing Stiffeners (6.10.11.2) (D6.5)
(3) Longitudinal Stiffeners (6.10.11.3)
3. Shear Connectors (6.10.10)
a. General (6.10.10.1)
b. Fatigue Resistance (6.10.10.2)
c. Special Requirements for Points of Permanent Load Contraflexure (6.10.10.3)
d. Strength Limit State (6.10.10.4)
J. Dimension and Detail Requirements
1. Material Thickness (6.7.3)
2. Bolted Connections (6.13.2)
a. Minimum Design Capacity (6.13.1)

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   b. Net Sections (6.8.3)
   c. Bolt Spacing Limits (6.13.2.6)
   d. Slip Critical Bolt Resistance (6.13.2.2) (6.13.2.8)
   e. Shear Resistance (6.13.2.7)
   f. Bearing Resistance (6.13.2.9)
   g. Tensile Resistance (6.13.2.10)
3. Welded Connections (6.13.3)
4. Block Shear Rupture Resistance (6.13.4)
5. Connection Elements (6.13.5)
6. Splices (6.13.6)
   a. Bolted Splices (6.13.6.1)
   b. Welded Splices (6.13.6.2)
7. Cover Plates (6.10.12)
8. Diaphragms and Cross-frames (6.7.4)
9. Lateral Bracing (6.7.5)

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C6.4—FLOWCHARTS FOR FLEXURAL DESIGN OF I-SECTION MEMBERS

C6.4.1—Flowchart for LRFD Article 6.10.3

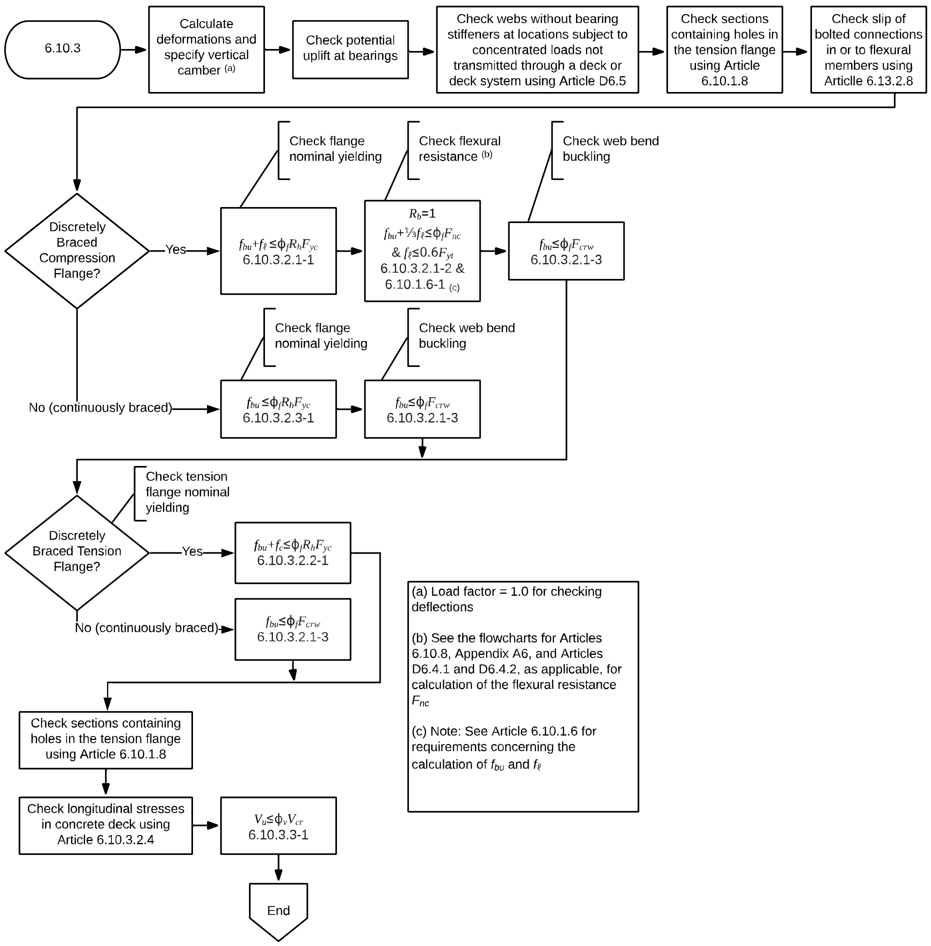


Figure C6.4.1-1—Flowchart for LRFD Article 6.10.3—Constructibility

C6.4.2—Flowchart for LRFD Article 6.10.4

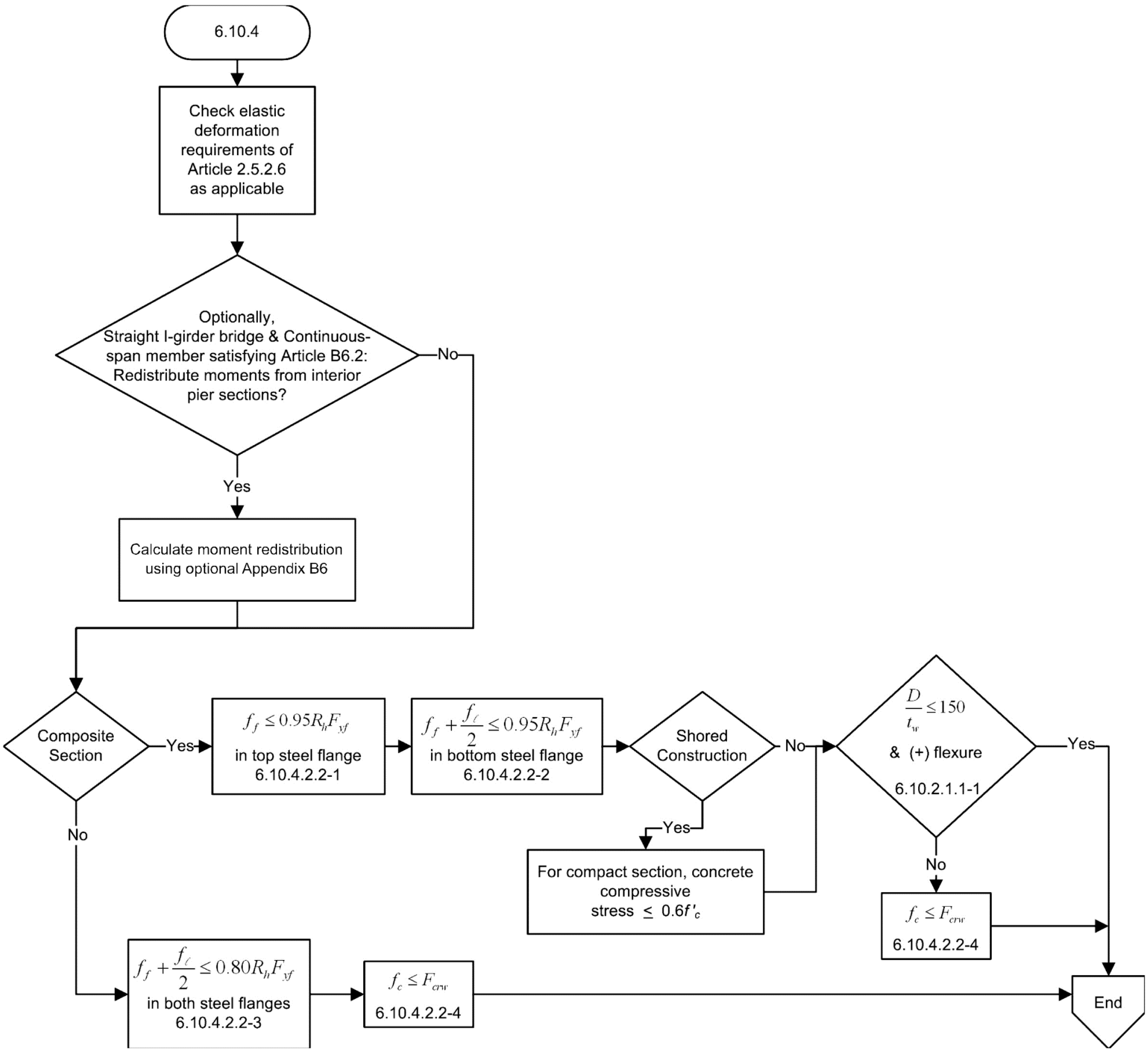


Figure C6.4.2-1—Flowchart for LRFD Article 6.10.4—Service Limit State

C6.4.3—Flowchart for LRFD Article 6.10.5

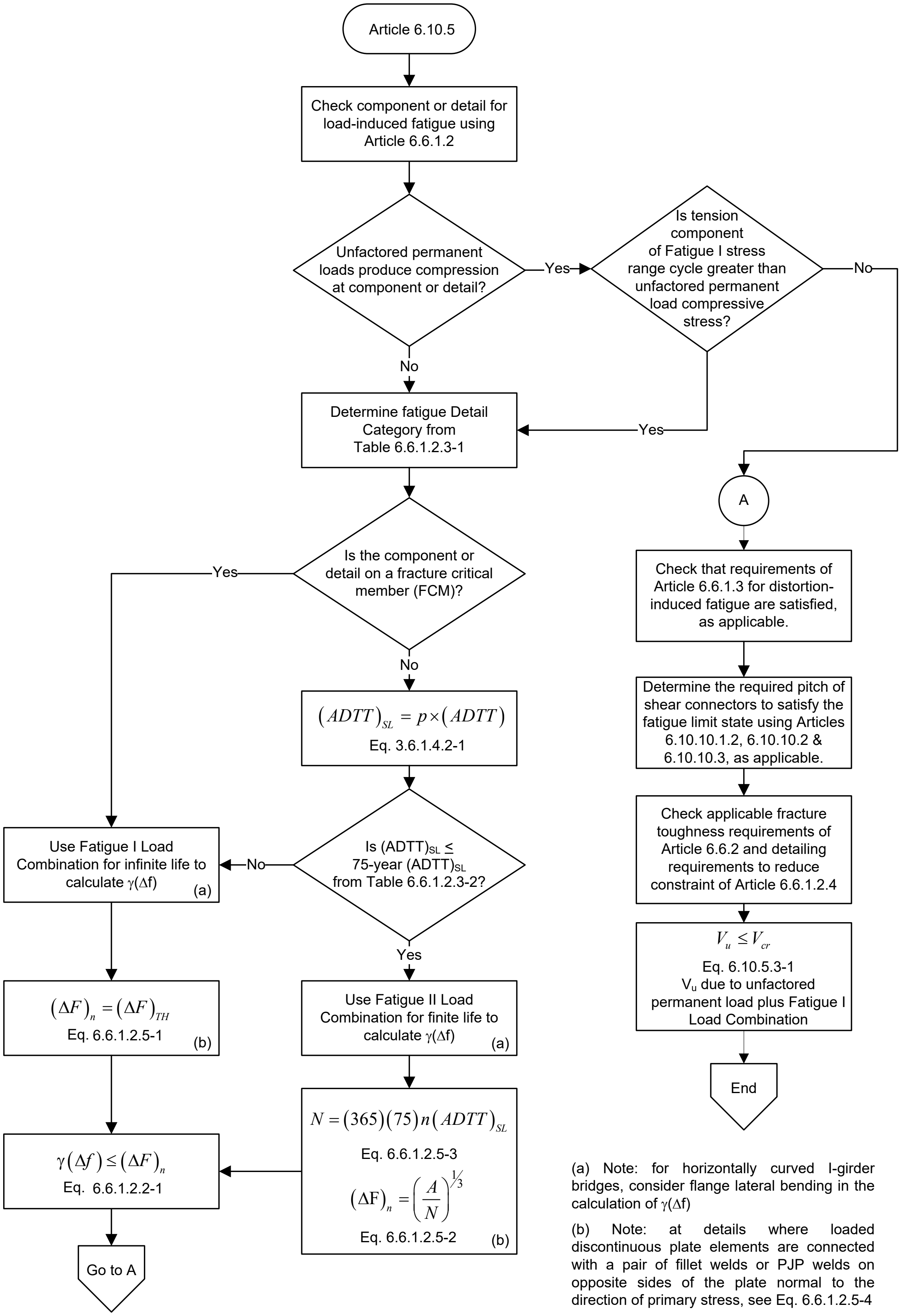
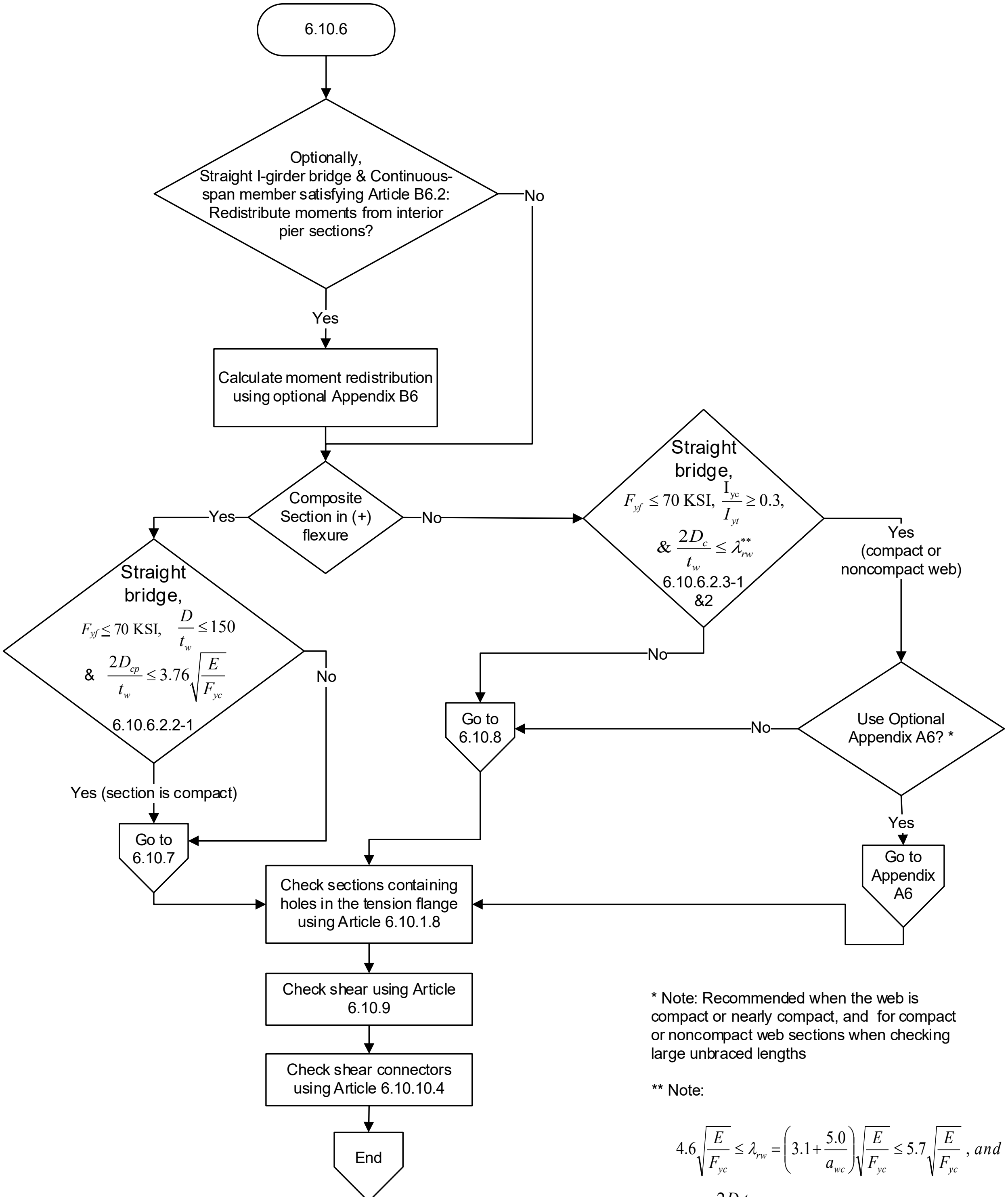


Figure C6.4.3-1—Flowchart for LRFD Article 6.10.5—Fatigue and Fracture Limit State

C6.4.4—Flowchart for LRFD Article 6.10.6



* Note: Recommended when the web is compact or nearly compact, and for compact or noncompact web sections when checking large unbraced lengths

** Note:

$$4.6\sqrt{\frac{E}{F_{yc}}} \le \lambda_{rw} = \left(3.1+\frac{5.0}{a_{wc}}\right)\sqrt{\frac{E}{F_{yc}}} \le 5.7\sqrt{\frac{E}{F_{yc}}}, \textit{ and}$$

$$a_{wc} = \frac{2D_c t_w}{b_{fc} t_{fc}}$$

Figure C6.4.4-1—Flowchart for LRFD Article 6.10.6—Strength Limit State

C6.4.5—Flowchart for LRFD Article 6.10.7

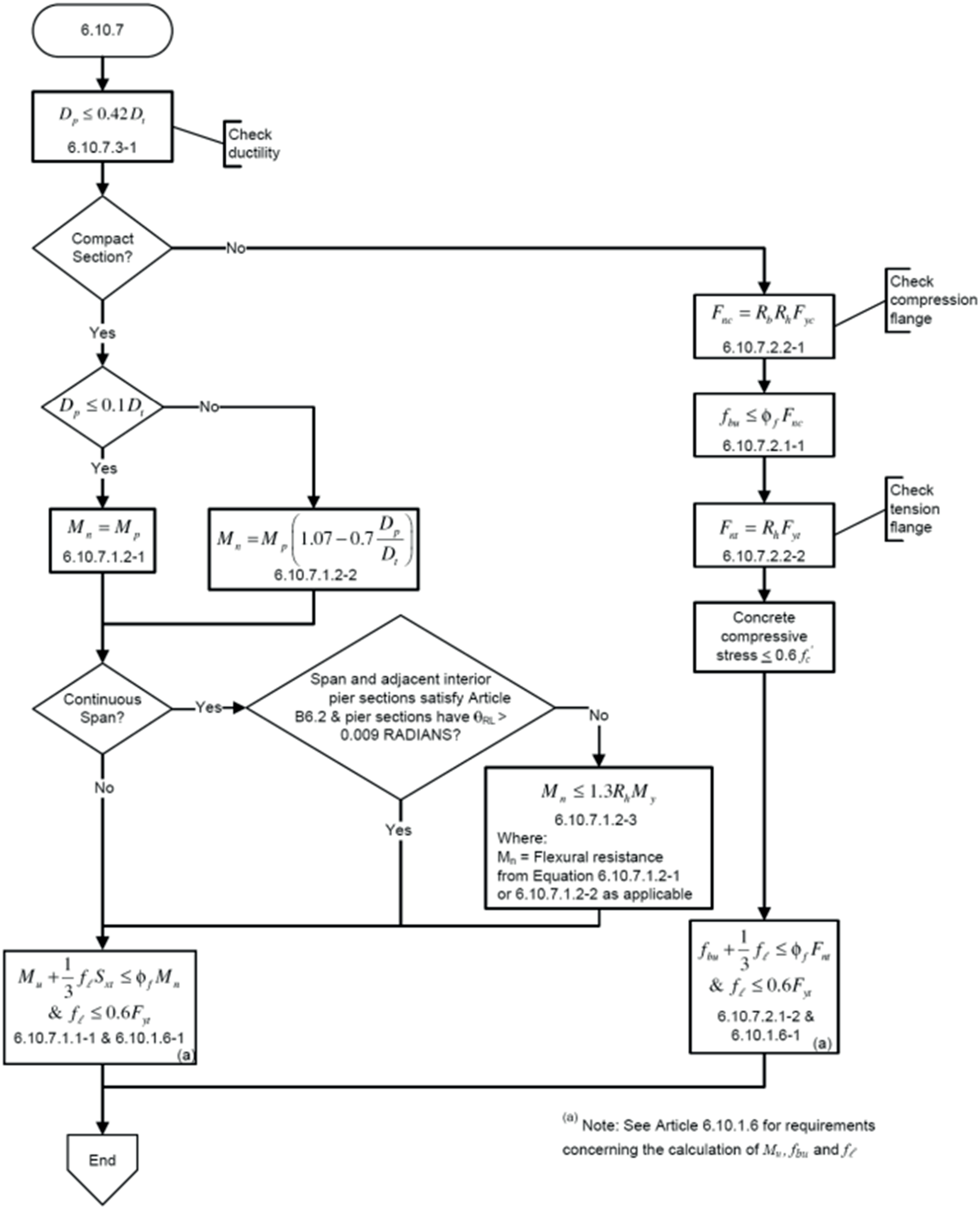


Figure C6.4.5-1—Flowchart for LRFD Article 6.10.7—Composite Sections in Positive Flexure

C6.4.6—Flowchart for LRFD Article 6.10.8

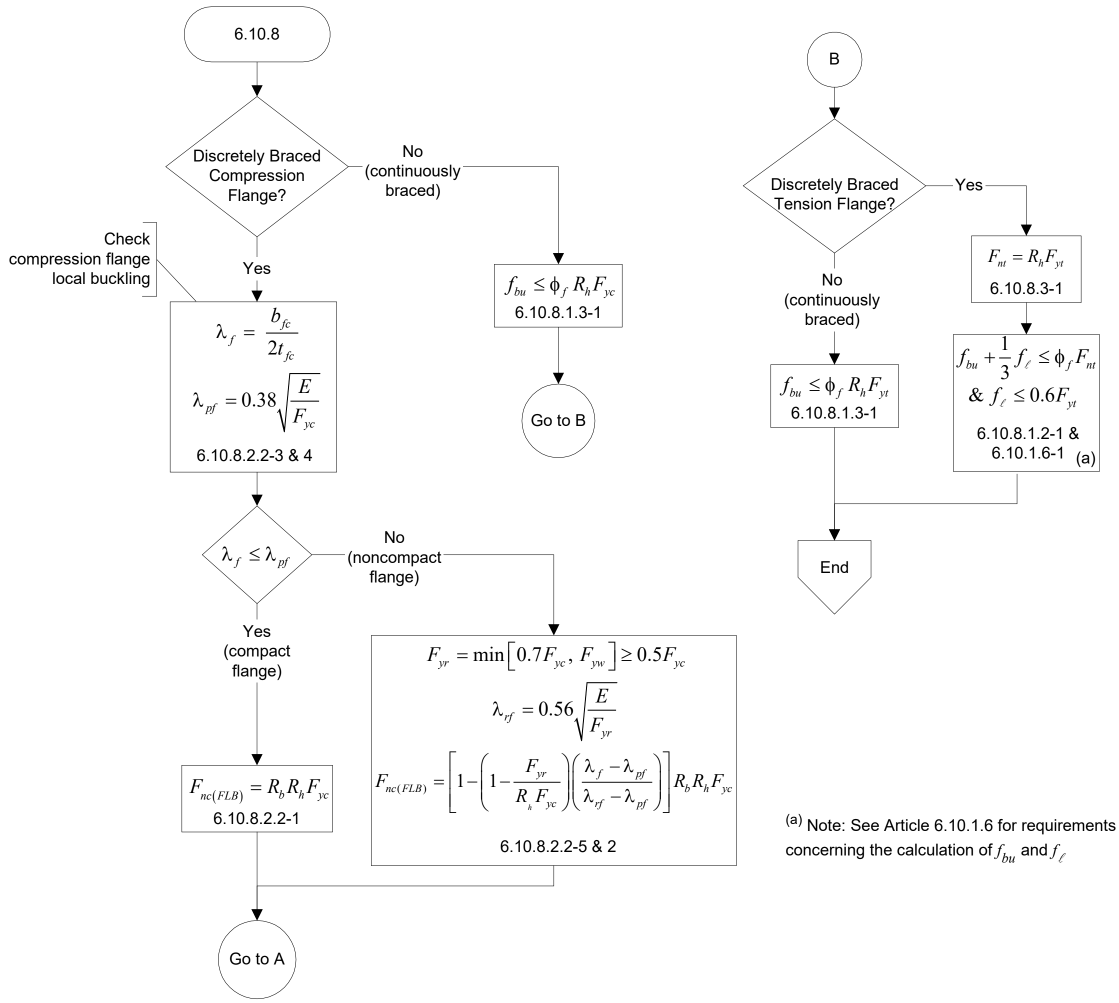


Figure C6.4.6-1—Flowchart for LRFD Article 6.10.8—Composite Sections in Negative Flexure and Noncomposite Sections

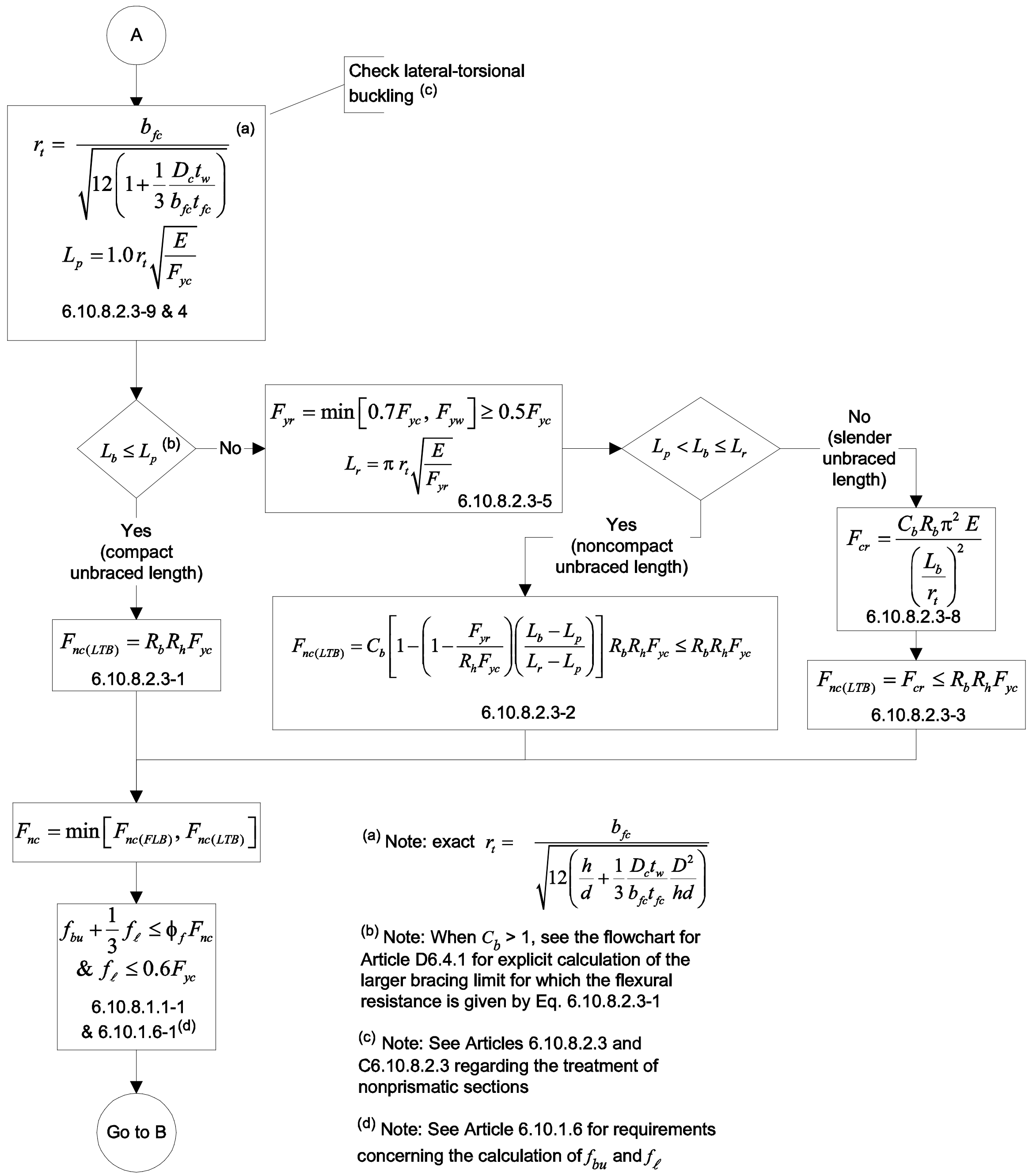


Figure C6.4.6-1 (continued)—Flowchart for LRFD Article 6.10.8—Composite Sections in Negative Flexure and Noncomposite Sections

C6.4.7—Flowchart for Appendix A6

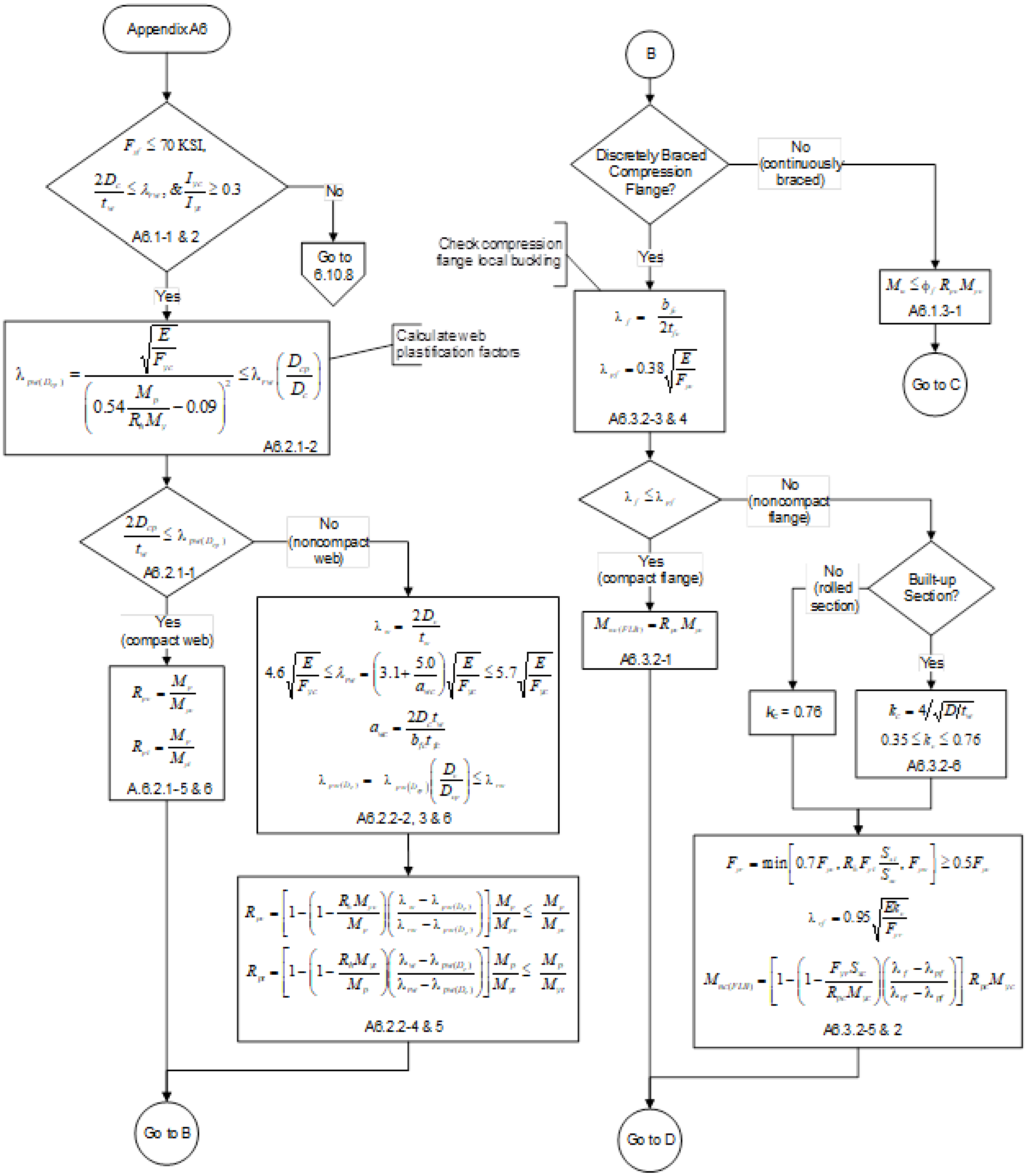


Figure C6.4.7-1—Flexural Resistance of Composite I-sections in Negative Flexure and Noncomposite I-sections with Compact or Noncompact Webs in Straight Bridges

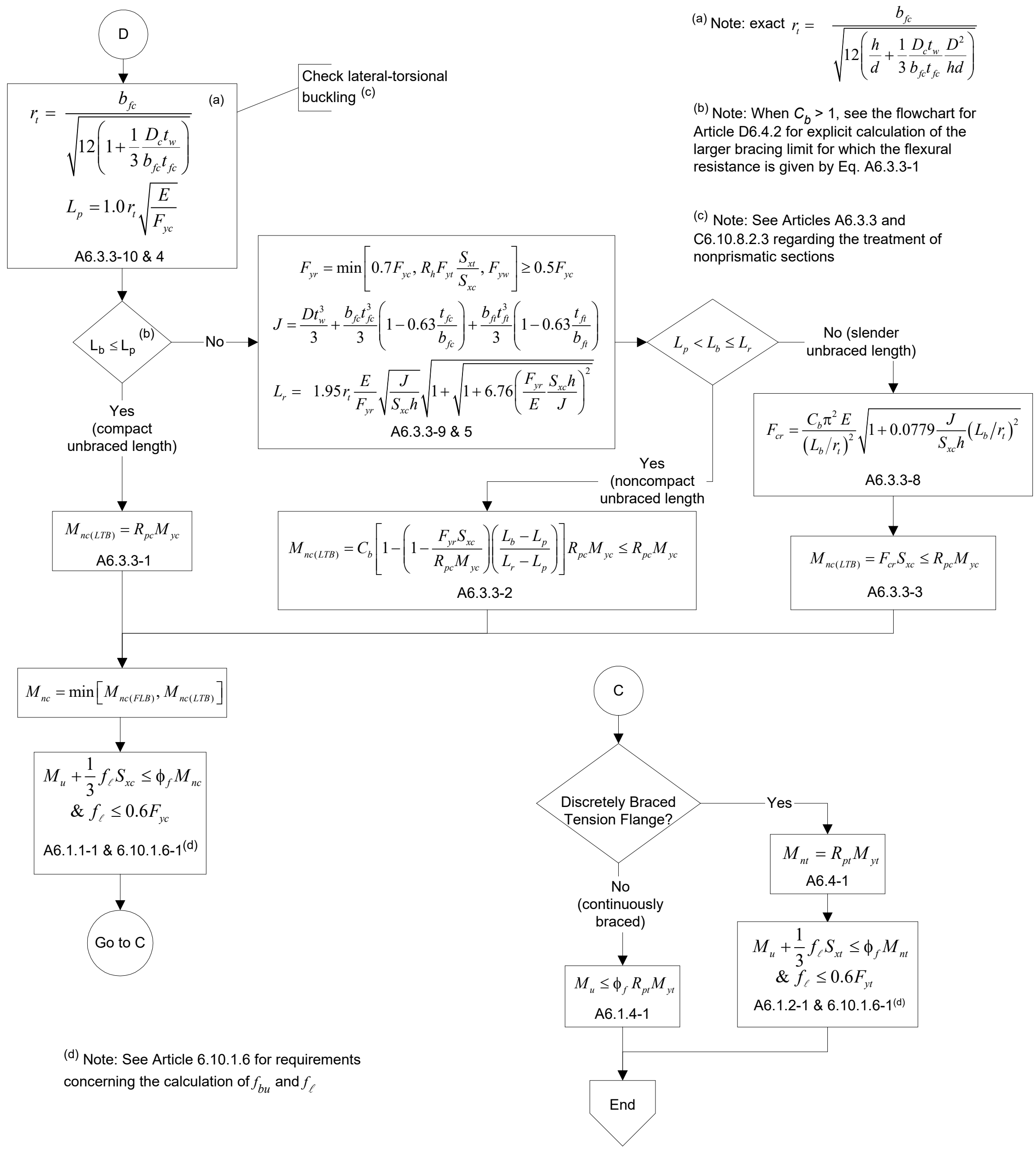


Figure C6.4.7-1 (continued)—Flexural Resistance of Composite I-sections in Negative Flexure and Noncomposite I-sections with Compact or Noncompact Webs in Straight Bridges

C6.4.8—Flowchart for Article D6.4.1

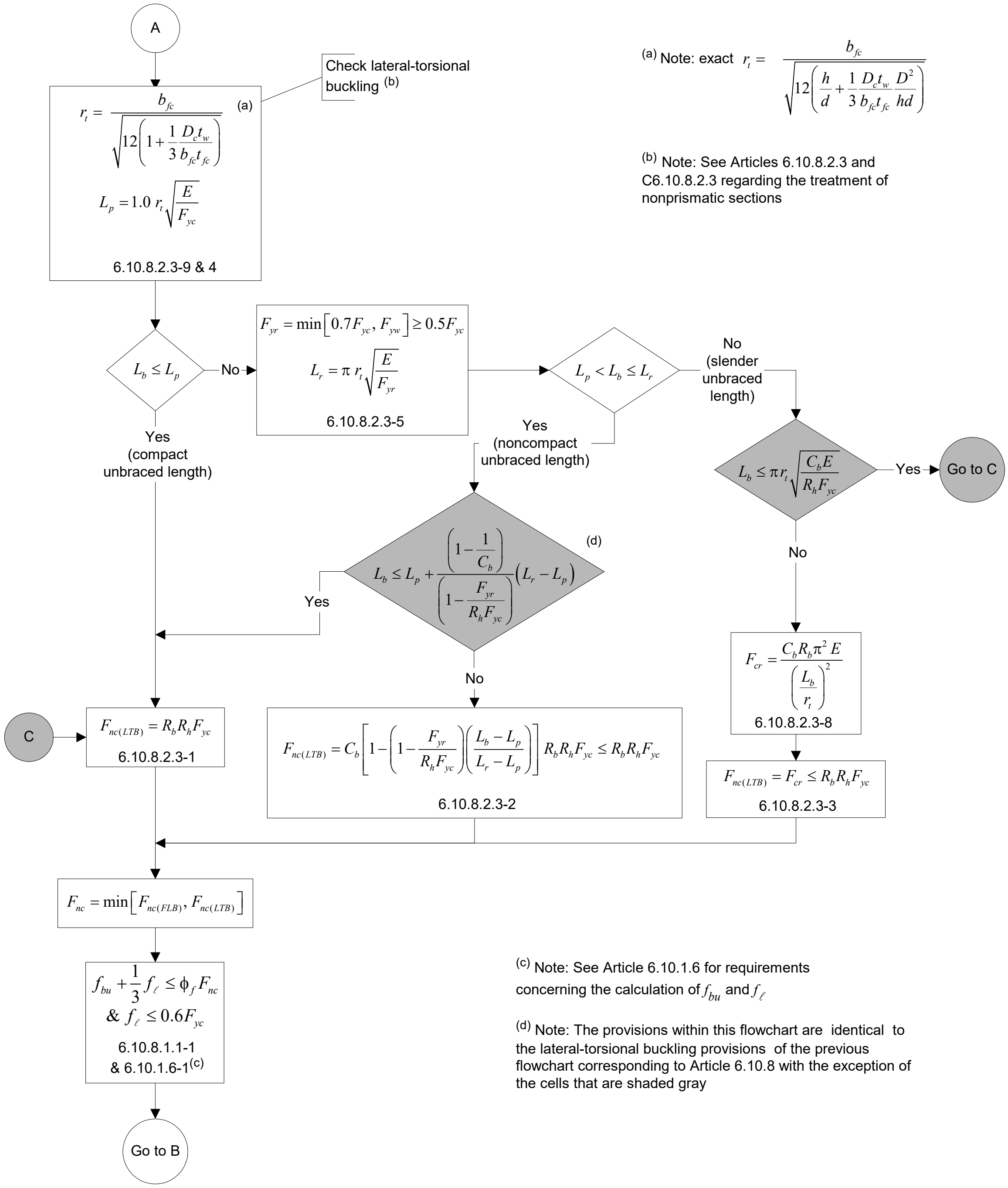


Figure C6.4.8-1—Flowchart for Article D6.4.1—LTB Provisions of Article 6.10.8.2.3 with Emphasis on Unbraced Length Requirements for Development of the Maximum Flexural Resistance

C6.4.9—Flowchart for Article D6.4.2

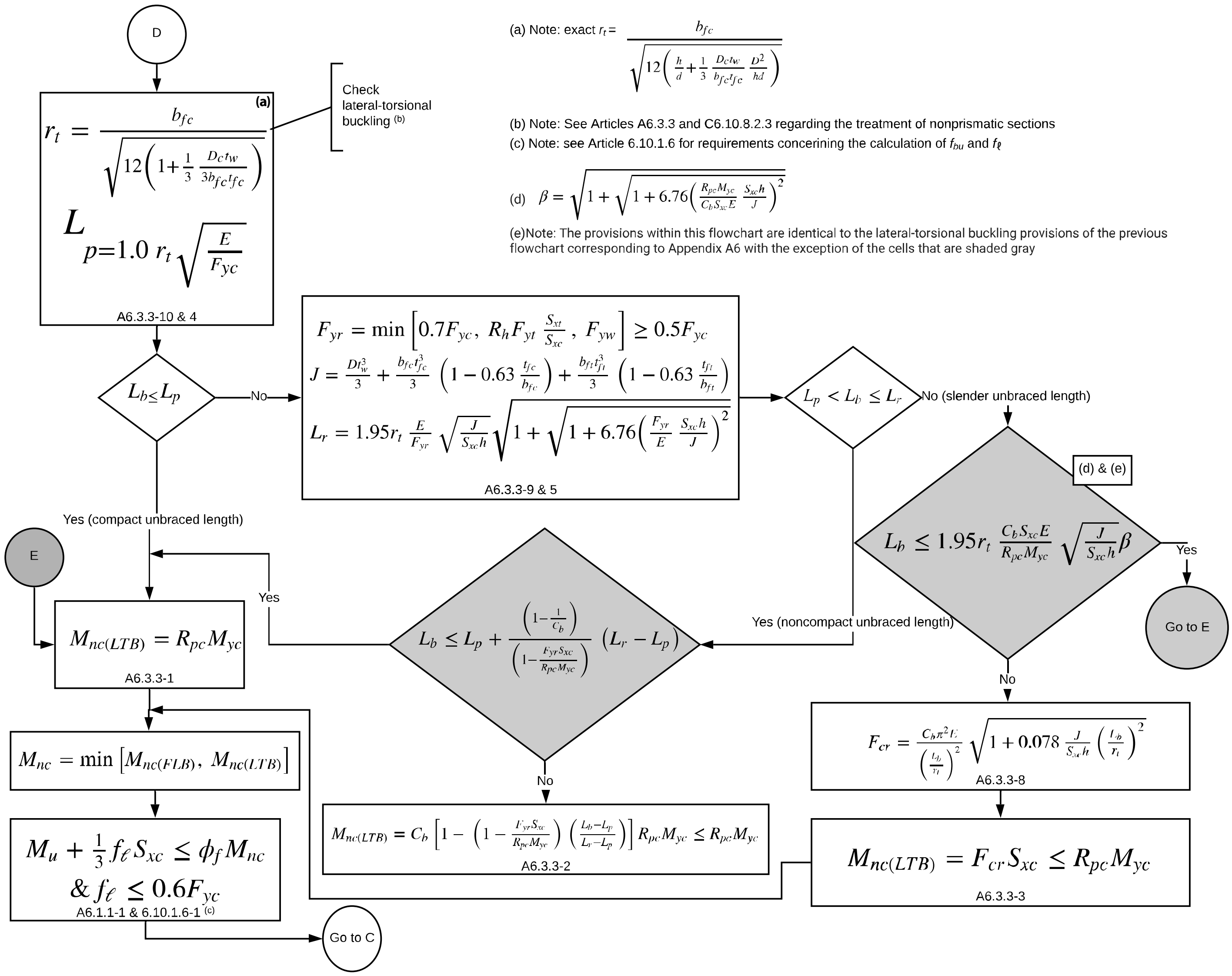


Figure C6.4.9-1—Flowchart for Article D6.4.2—LTB Provisions of Article A6.3.3 with Emphasis on Unbraced Length Requirements for Development of the Maximum Flexural Resistance

C6.4.10—Moment Gradient Modifier, C_b (Sample Cases)

Unbraced cantilevers and members where $f_{mid}/f_2 > 1$ or $f_2 = 0$: $C_b = 1$

Otherwise: $C_b = 1.75 - 1.05(f_1/f_2) + 0.3(f_1/f_2)^2 \leq 2.3$

If variation of moment is concave between brace points: $f_1 = f_0$

Otherwise: $f_1 = 2f_{mid} - f_2 \geq f_0$

Examples:

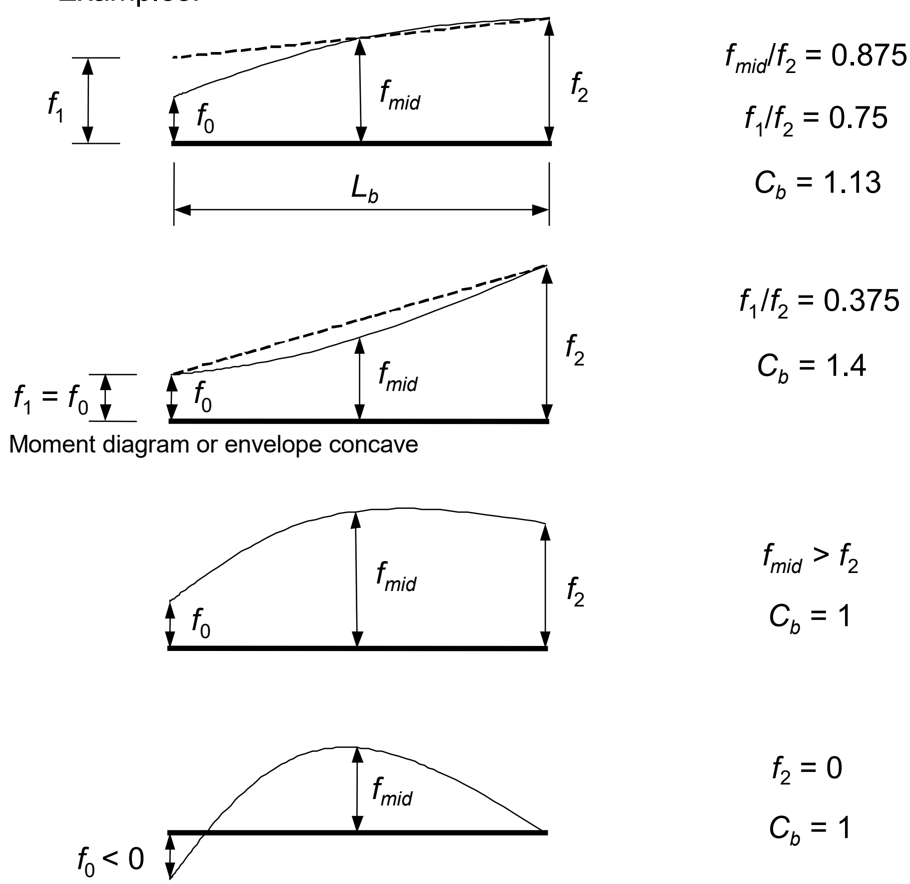
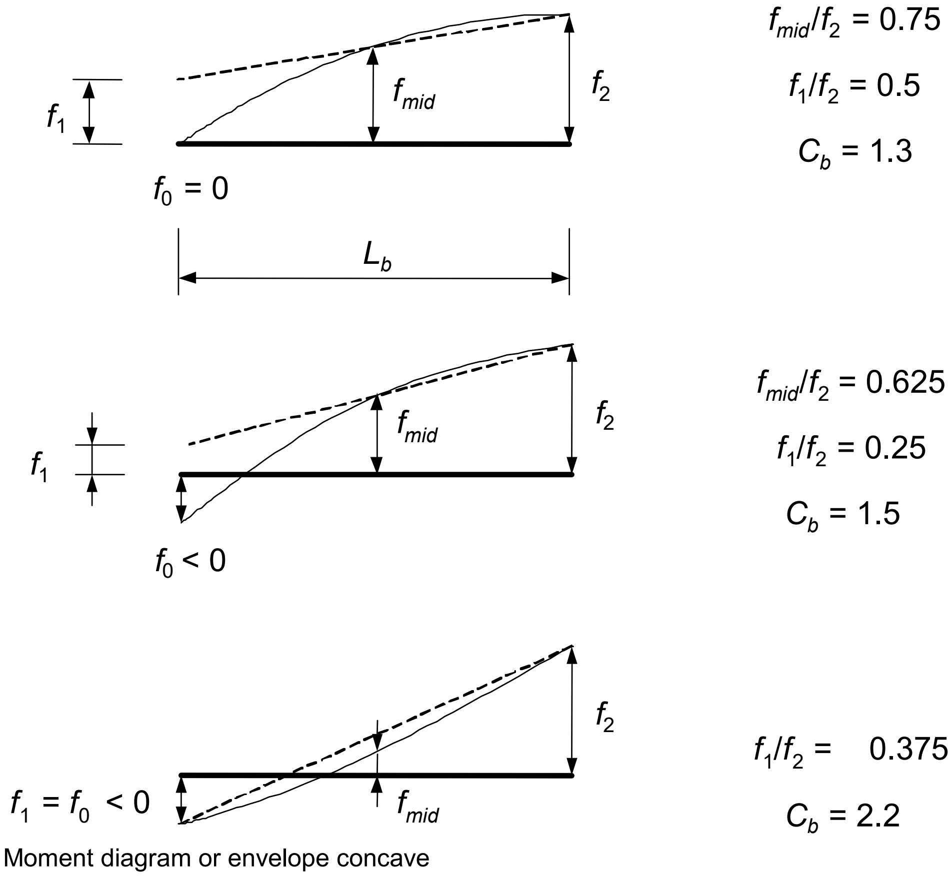


Figure C6.4.10-1—Moment Gradient Modifier, C_b (Sample Cases)

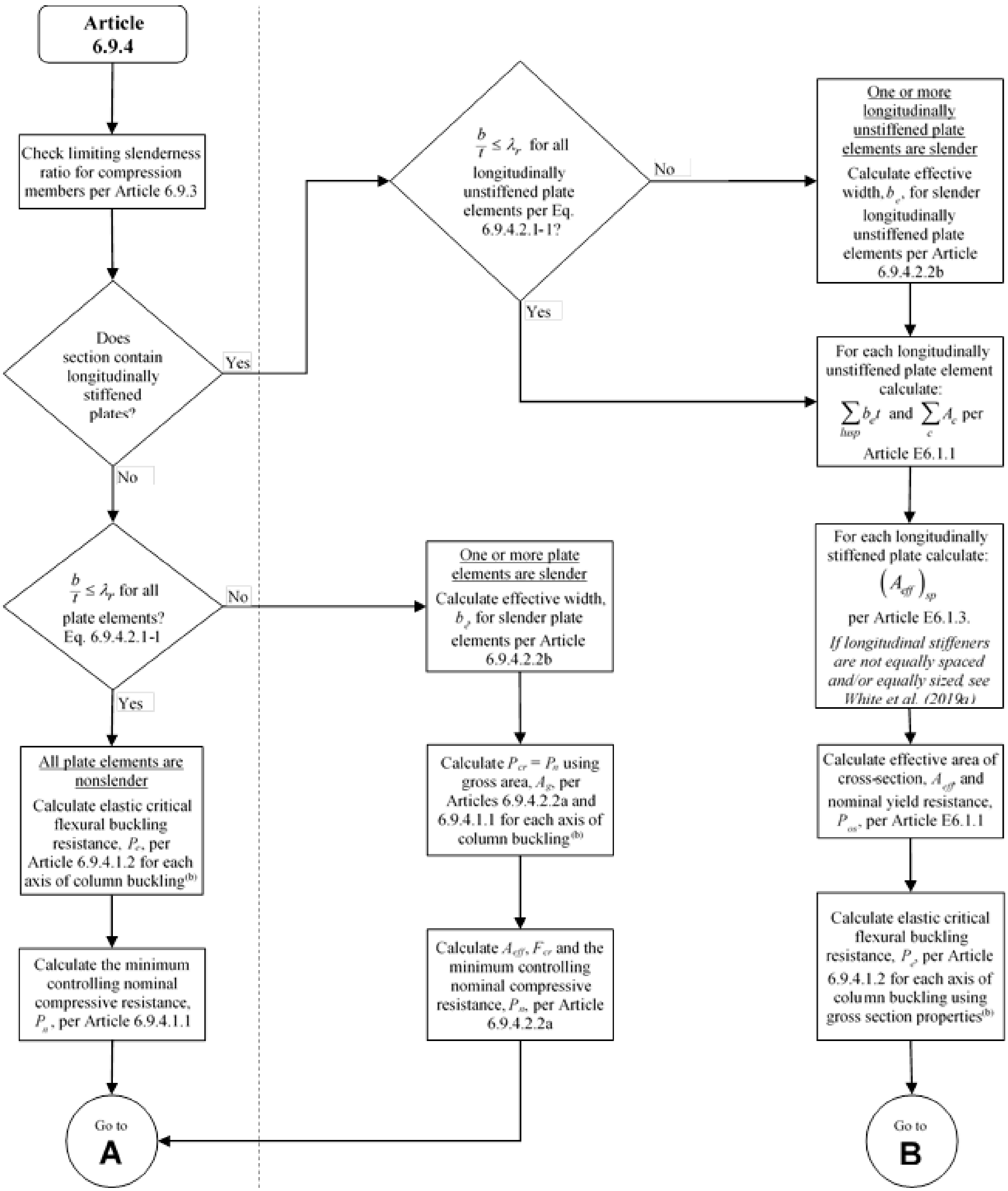


Note: The above examples assume that the member is prismatic within the unbraced length, or the transition to a smaller section is within $0.2L_b$ from the braced point with the lower moment. Otherwise, use $C_b = 1$.

Figure C6.4.10-1—(continued) Moment Gradient Modifier, C_b (Sample Cases)

C6.5—FLOWCHARTS FOR LRFD ARTICLES 6.9.4 AND 6.12.2.2.2

C6.5.1—Flowchart for LRFD Article 6.9.4[(a)]



(a) Note: Process varies for HSS and mechanically fastened built-up sections.

(b) Note: For closed sections, only flexural buckling applies per Article 6.9.4.1.1.

Figure C6.5.1-1—Flowchart for LRFD Article 6.9.4—Compressive Resistance of Noncomposite I- or Box-Section Members

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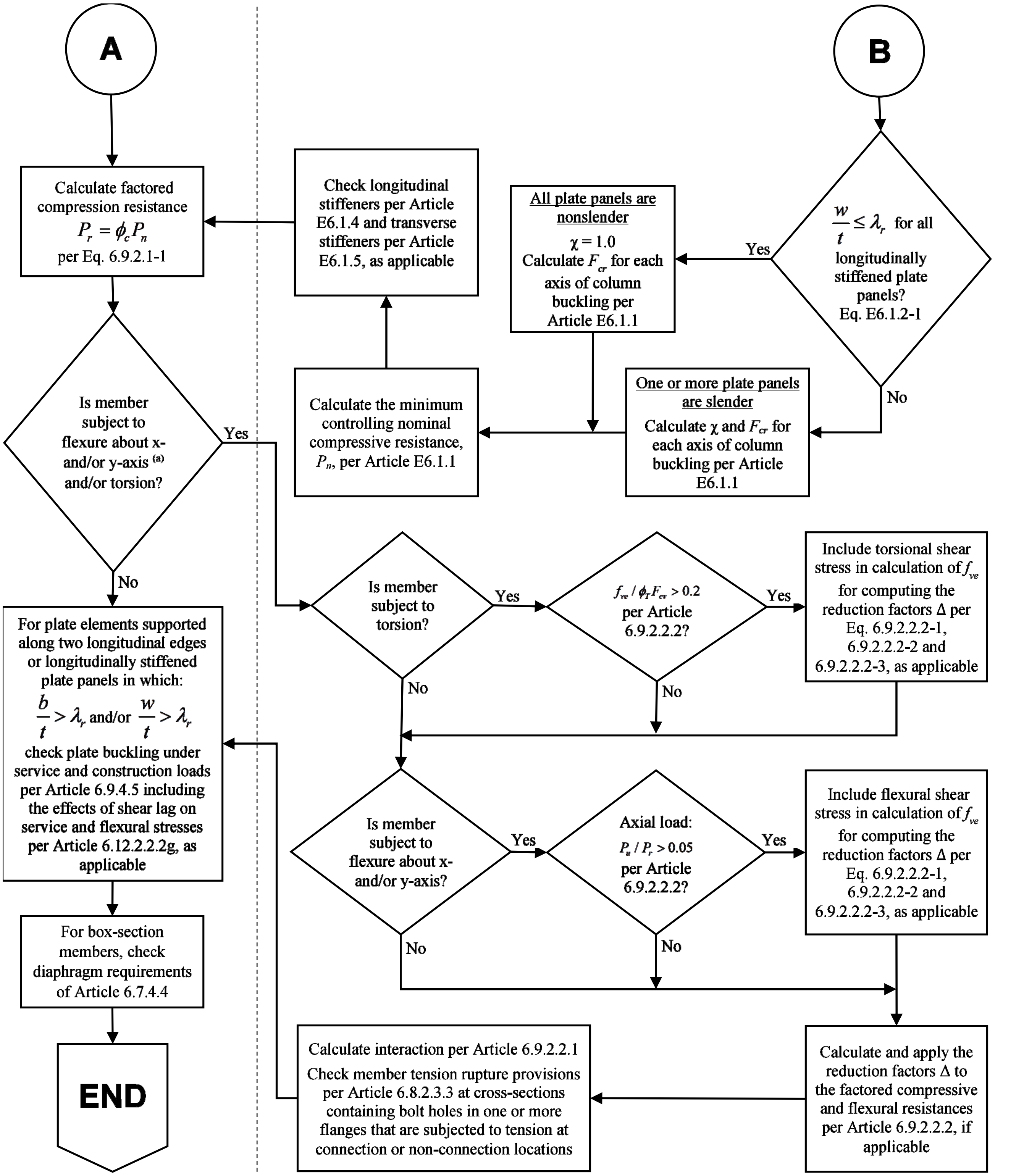
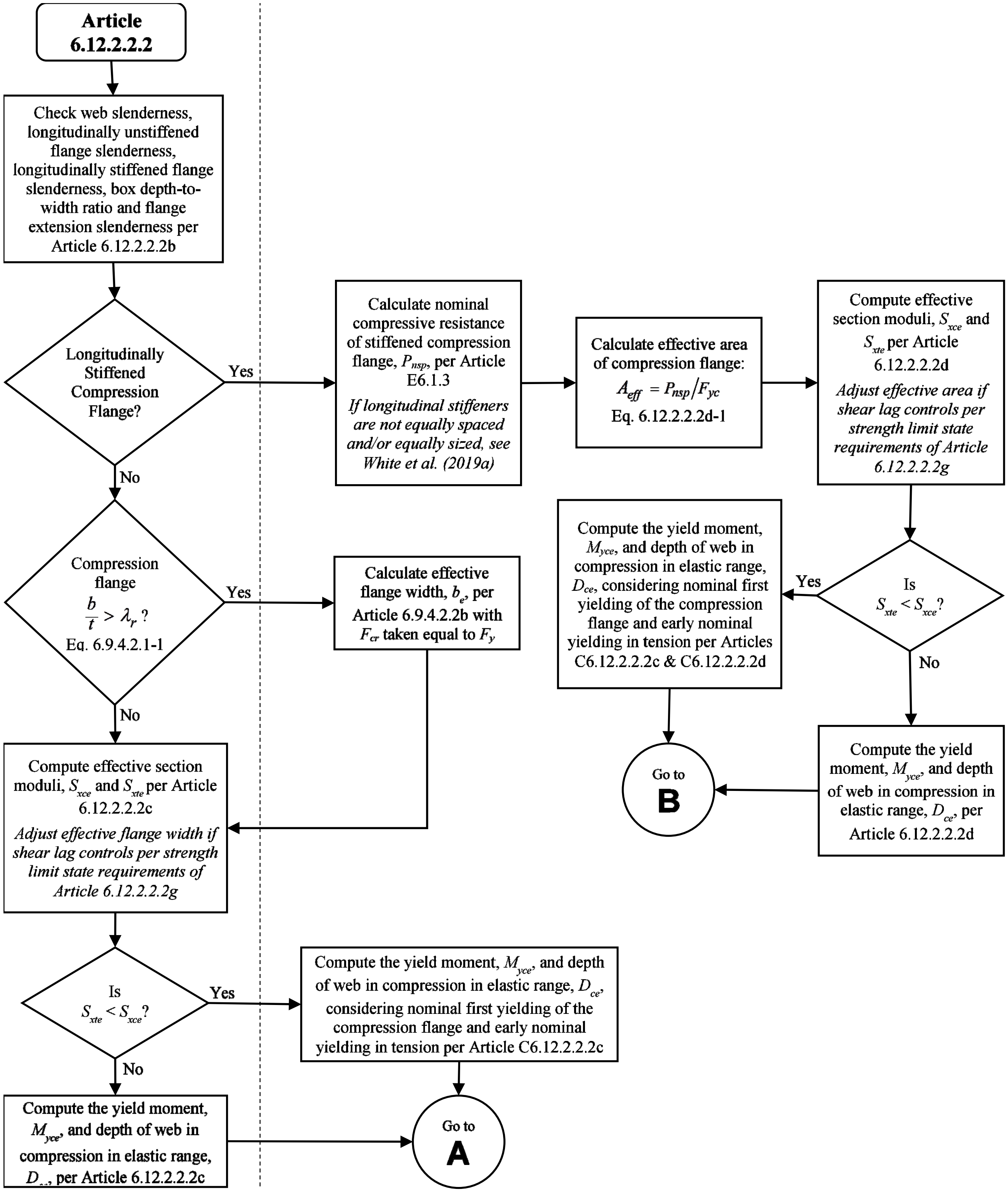


(a) Note: If member is subject to uniaxial flexure only and P_u/P_r is less than or equal to 0.05, follow the 'No' path.

Figure C6.5.1-1 (continued)—Flowchart for LRFD Article 6.9.4—Compressive Resistance of Noncomposite I- or Box-Section Members

C6.5.2—Flowchart for LRFD Article 6.12.2.2.2[a, b]



(a) Note: Process varies for HSS and mechanically fastened built-up sections.
(b) Note: Process is shown for flexure of a single compression flange about one axis. Repeat the process as necessary for other flanges subject to compression based on symmetry and axes of loading.

Figure C6.5.2-1—Flowchart for LRFD Article 6.12.2.2.2—Flexural Resistance of Rectangular Noncomposite Box-Section Members

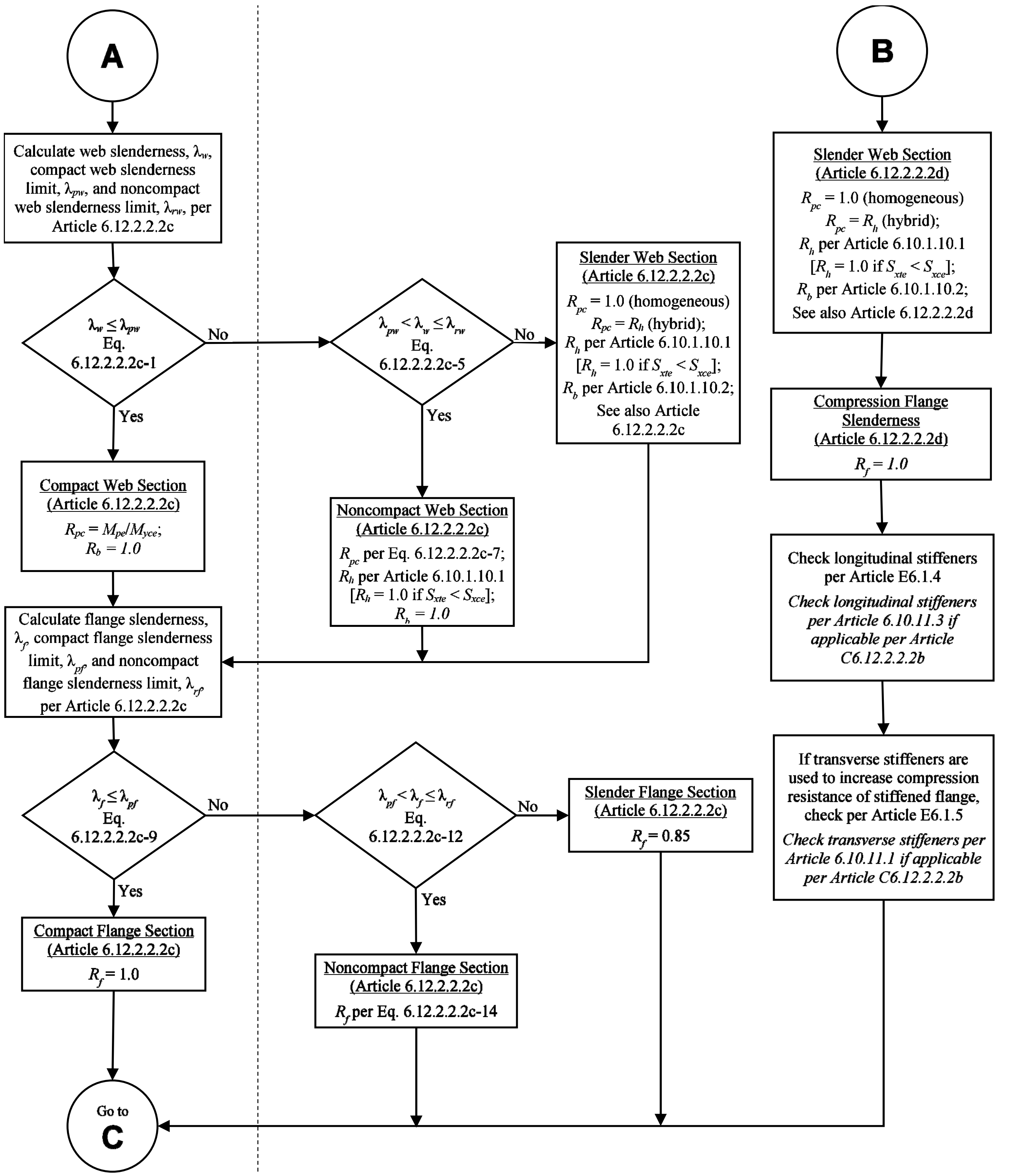
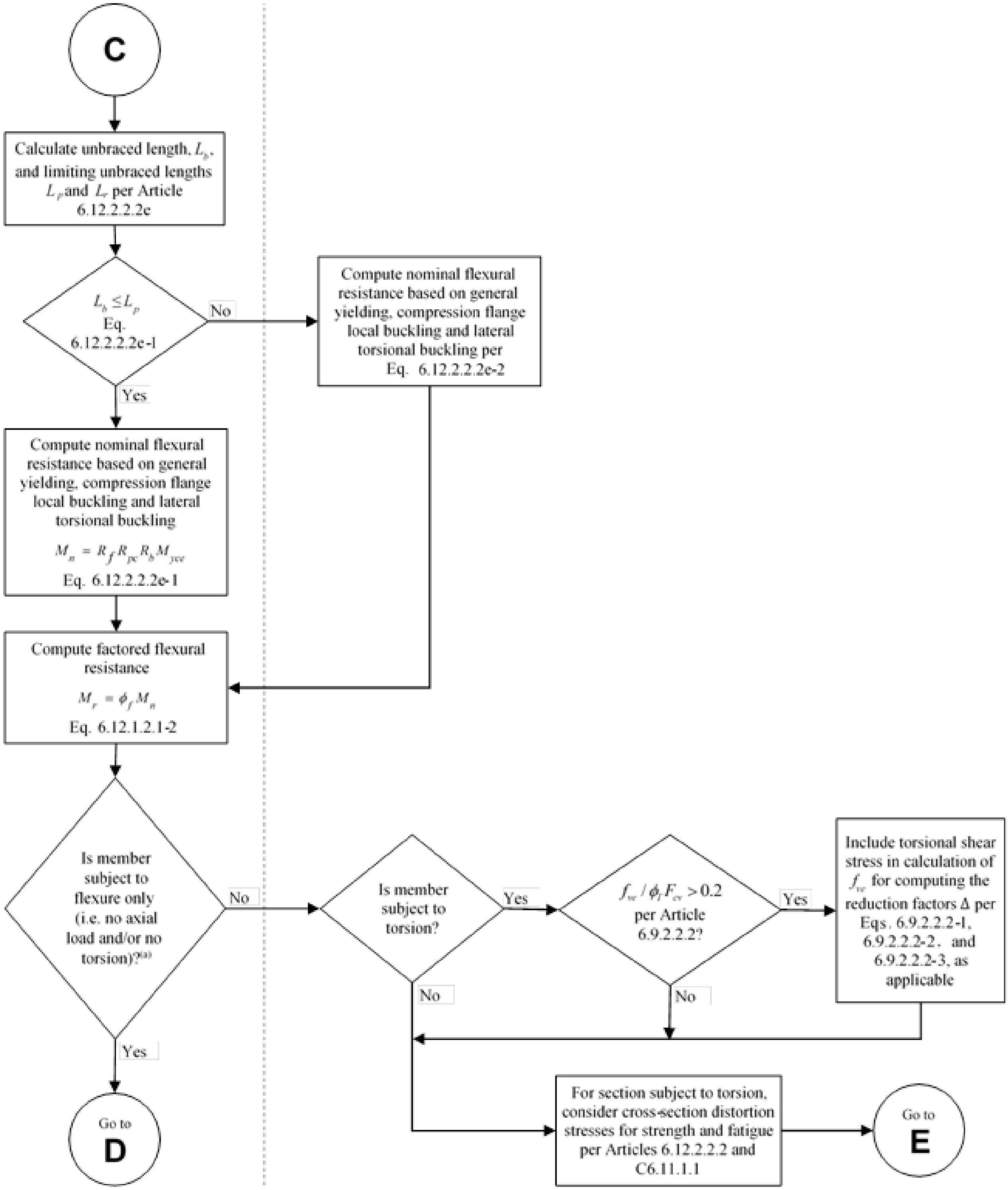


Figure C6.5.2-1 (continued)—Flowchart for LRFD Article 6.12.2.2.2—Flexural Resistance of Rectangular Noncomposite Box-Section Members



(a) Note: If member is subject to uniaxial flexure only and P_u/P_r is less than or equal to 0.05, follow the 'Yes' path.

Figure C6.5.2-1 (continued)—Flowchart for LRFD Article 6.12.2.2.2—Flexural Resistance of Rectangular Noncomposite Box-Section Members

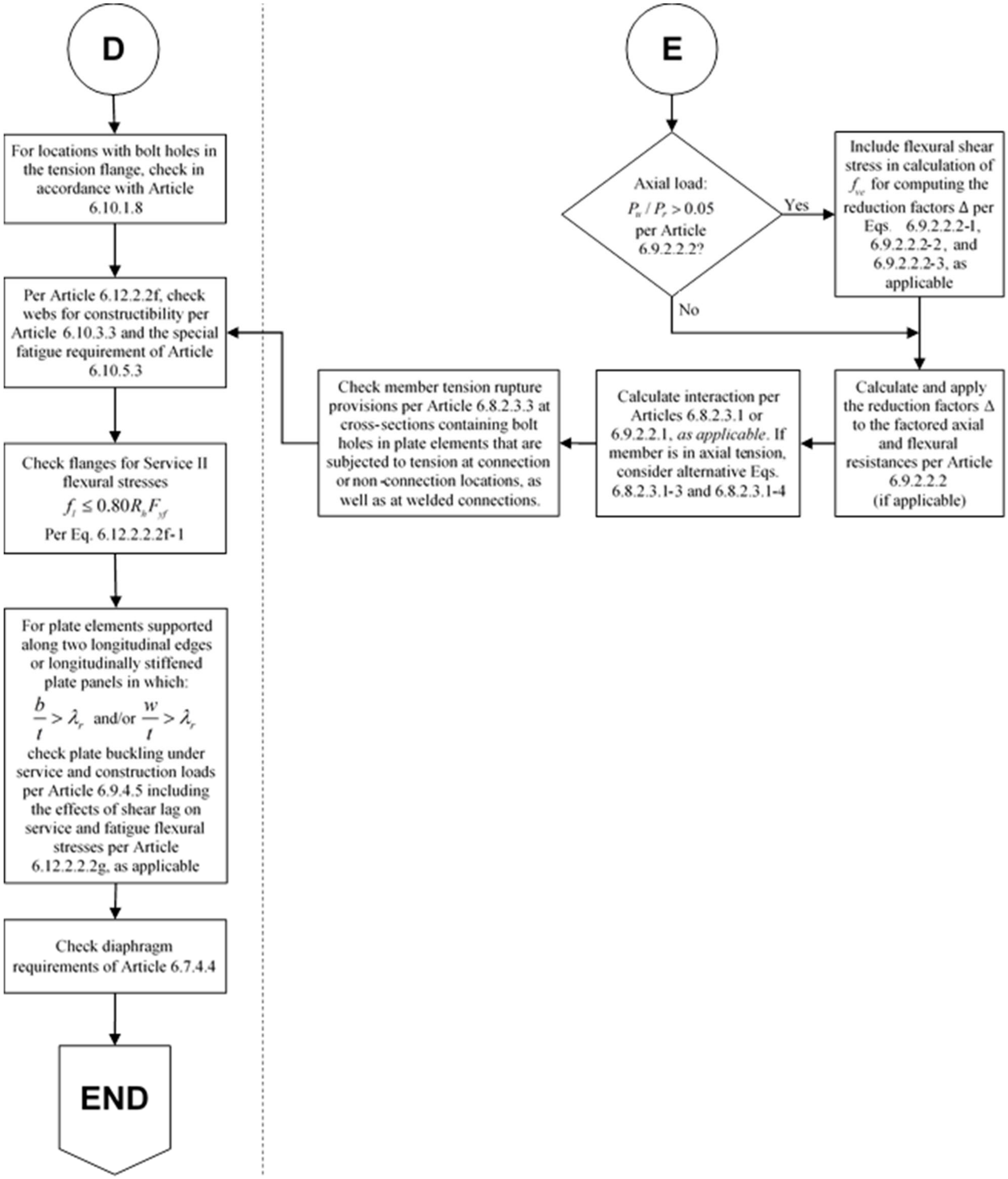


Figure C6.5.2-1 (continued)—Flowchart for LRFD Article 6.12.2.2.2—Flexural Resistance of Rectangular Noncomposite Box-Section Members

APPENDIX D6—FUNDAMENTAL CALCULATIONS FOR FLEXURAL MEMBERS

D6.1—PLASTIC MOMENT

The plastic moment, M_p, shall be calculated as the moment of the plastic forces about the plastic neutral axis. Plastic forces in steel portions of a cross section shall be calculated using the yield strengths of the flanges, the web, and reinforcing steel, as appropriate. Plastic forces in concrete portions of the cross section that are in compression may be based on a rectangular stress block with the magnitude of the compressive stress equal to $0.85f'_c$. Concrete in tension shall be neglected.

The position of the plastic neutral axis shall be determined by the equilibrium condition that there is no net axial force.

The plastic moment of a composite section in positive flexure can be determined by:

- calculating the element forces and using them to determine whether the plastic neutral axis is in the web, top flange or concrete deck;
- calculating the location of the plastic neutral axis within the element determined in the first step; and
- calculating M_p. Equations for the various potential locations of the plastic neutral axis (PNA) are given in Table D6.1-1.

The forces in the longitudinal reinforcement may be conservatively neglected. To do this, set P_{rb} and P_{rt} equal to zero in the equations in Table D6.1-1.

The plastic moment of a composite section in negative flexure can be calculated by an analogous procedure. Equations for the two cases most likely to occur in practice are given in Table D6.1-2.

The plastic moment of a noncomposite section may be calculated by eliminating the terms pertaining to the concrete deck and longitudinal reinforcement from the equations in Tables D6.1-1 and D6.1-2 for composite sections.

In the equations for M_p given in Tables D6.1-1 and D6.1-2, d is the distance from an element force to the plastic neutral axis. Element forces act at (a) mid-thickness for the flanges and the concrete deck, (b) mid-depth of the web, and (c) center of reinforcement. All element forces, dimensions, and distances should be taken as positive. The condition should be checked in the order listed in Tables D6.1-1 and D6.1-2.

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Table D6.1-1—Calculation of $\bar{Y}$ and M_p for Sections in Positive Flexure

Case	PNA	Condition	$\bar{Y}$ and M_p
I	In Web	$P_t + P_w \geq P_c + P_s + P_{rb} + P_{rt}$	$\bar{Y} = \left(\frac{D}{2}\right)\left[\frac{P_t - P_c - P_s - P_{rt} - P_{rb}}{P_w} + 1\right]$ $M_p = \frac{P_w}{2D}\left[\bar{Y}^2 + \left(D - \bar{Y}\right)^2\right] + \left[P_s d_s + P_{rt} d_{rt} + P_{rb} d_{rb} + P_c d_c + P_t d_t\right]$
II	In Top Flange	$P_t + P_w + P_c \geq P_s + P_{rb} + P_{rt}$	$\bar{Y} = \left(\frac{t_c}{2}\right)\left[\frac{P_w + P_t - P_s - P_{rt} - P_{rb}}{P_c} + 1\right]$ $M_P = \frac{P_c}{2t_c}\left[\bar{Y}^2 + \left(t_c - \bar{Y}\right)^2\right] + \left[P_s d_s + P_{rt} d_{rt} + P_{rb} d_{rb} + P_w d_w + P_t d_t\right]$
III	Concrete Deck, Below P_{rb}	$P_t + P_w + P_c \geq \left(\frac{c_{rb}}{t_s}\right) P_s + P_{rb} + P_{rt}$	$\bar{Y} = (t_s)\left[\frac{P_c + P_w + P_t - P_{rt} - P_{rb}}{P_s}\right]$ $M_p = \left(\frac{\bar{Y}^2 P_s}{2t_s}\right) + \left[P_{rt} d_{rt} + P_{rb} d_{rb} + P_c d_c + P_w d_w + P_t d_t\right]$
IV	Concrete Deck, at P_{rb}	$P_t + P_w + P_c + P_{rb} \geq \left(\frac{c_{rb}}{t_s}\right) P_s + P_{rt}$	$\bar{Y} = c_{rb}$ $M_p = \left(\frac{\bar{Y}^2 P_s}{2t_s}\right) + \left[P_{rt} d_{rt} + P_c d_c + P_w d_w + P_t d_t\right]$
V	Concrete Deck, Above P_{rb} Below P_{rt}	$P_t + P_w + P_c + P_{rb} \geq \left(\frac{c_{rt}}{t_s}\right) P_s + P_{rt}$	$\bar{Y} = (t_s)\left[\frac{P_{rb} + P_c + P_w + P_t - P_{rt}}{P_s}\right]$ $M_p = \left(\frac{\bar{Y}^2 P_s}{2t_s}\right) + \left[P_{rt} d_{rt} + P_{rb} d_{rb} + P_c d_c + P_w d_w + P_t d_t\right]$
VI	Concrete Deck, at P_{rt}	$P_t + P_w + P_c + P_{rb} + P_{rt} \geq \left(\frac{c_{rt}}{t_s}\right) P_s$	$\bar{Y} = c_{rt}$ $M_p = \left(\frac{\bar{Y}^2 P_s}{2t_s}\right) + \left[P_{rb} d_{rb} + P_c d_c + P_w d_w + P_t d_t\right]$
VII	Concrete Deck, Above P_{rt}	$P_t + P_w + P_c + P_{rb} + P_{rt} < \left(\frac{c_{rt}}{t_s}\right) P_s$	$\bar{Y} = (t_s)\left[\frac{P_{rb} + P_c + P_w + P_t + P_{rt}}{P_s}\right]$ $M_p = \left(\frac{\bar{Y}^2 P_s}{2t_s}\right) + \left[P_{rt} d_{rt} + P_{rb} d_{rb} + P_c d_c + P_w d_w + P_t d_t\right]$

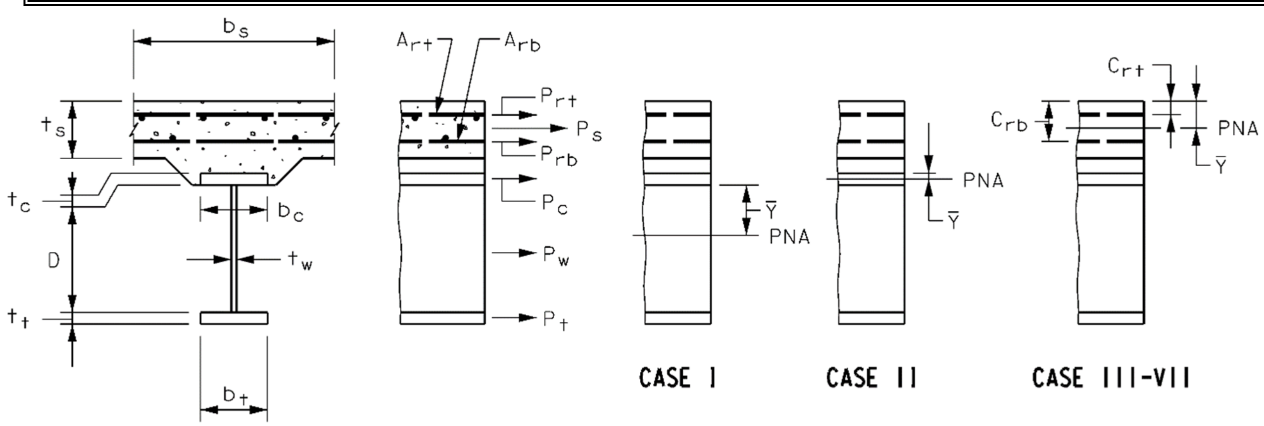

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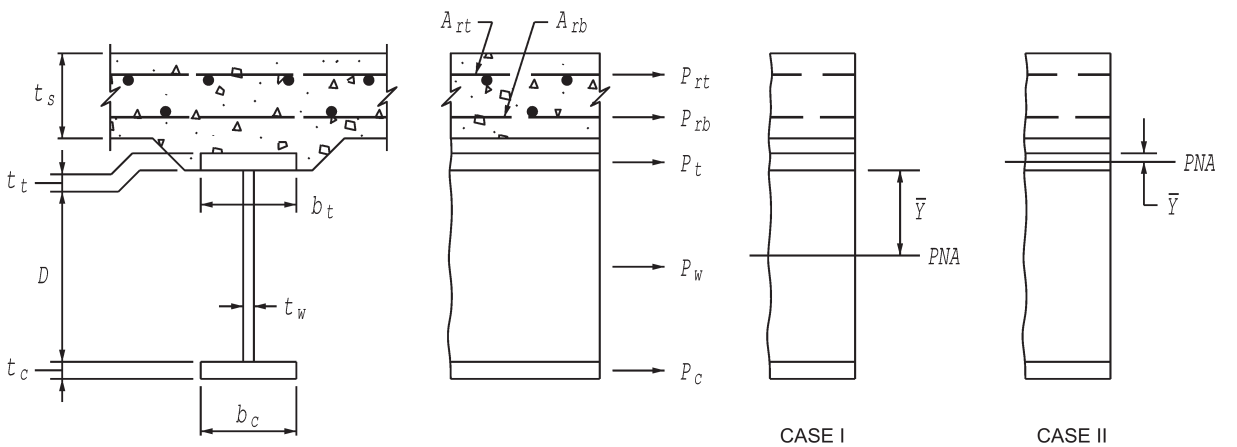
Table D6.1-2—Calculation of $\bar{Y}$ and M_p for Sections in Negative Flexure

Case	PNA	Condition	$\bar{Y}$ and M_p
I	In Web	$P_c + P_w \geq P_t + P_{rb} + P_{rt}$	$\bar{Y} = \left(\frac{D}{2}\right)\left[\frac{P_c - P_t - P_{rt} - P_{rb}}{P_w} + 1\right]$ $M_p = \frac{P_w}{2D}\left[\bar{Y}^2 + \left(D - \bar{Y}\right)^2\right] + [P_{rt}d_{rt} + P_{rb}d_{rb} + P_t d_t + P_c d_c]$
II	In Top Flange	$P_c + P_w + P_t \geq P_{rb} + P_{rt}$	$\bar{Y} = \left(\frac{t_t}{2}\right)\left[\frac{P_w + P_c - P_{rt} - P_{rb}}{P_t} + 1\right]$ $M_p = \frac{P_t}{2t_t}\left[\bar{Y}^2 + \left(t_t - \bar{Y}\right)^2\right] + [P_{rt}d_{rt} + P_{rb}d_{rb} + P_w d_w + P_c d_c]$



in which:

$$P_{rt} = F_{yrt}A_{rt}$$
$$P_s = 0.85 f'_c b_s t_s$$
$$P_{rb} = F_{yrb}A_{rb}$$
$$P_c = F_{yc}b_c t_c$$
$$P_w = F_{yw}Dt_w$$
$$P_t = F_{yt}b_t t_t$$

where:

M_p = plastic moment (kip-in.)
P_t = minimum required bolt tension (kip); plastic force in the tension flange used to compute the plastic moment (kip)
P_w = plastic force in the web used to compute the plastic moment (kip)
P_c = plastic force in the compression flange used to compute the plastic moment (kip)

P_s = plastic compressive force in the concrete deck used to compute the plastic moment (kip)
P_{rb} = plastic force in the bottom layer of longitudinal deck reinforcement used to compute the plastic moment (kip)
P_{rt} = plastic force in the top layer of longitudinal deck reinforcement used to compute the plastic moment (kip)
$\bar{y}$ = distance from the plastic neutral axis to the top of the element where the plastic neutral axis is located (in.)
d_s = distance from the centerline of the closest plate longitudinal stiffener or from the gauge line of the closest angle longitudinal stiffener to the inner surface or leg of the compression flange element (in.); distance from the plastic neutral axis to the midthickness of the concrete deck used to compute the plastic moment (in.)
d_{rt} = distance from the plastic neutral axis to the centerline of the top layer of longitudinal concrete deck reinforcement used to compute the plastic moment (in.)
d_{rb} = distance from the plastic neutral axis to the centerline of the bottom layer of longitudinal concrete deck reinforcement used to compute the plastic moment (in.)
d_c = depth of a column in a rigid frame (in.); distance from the plastic neutral axis to the midthickness of the compression flange used to compute the plastic moment (in.)
d_t = distance from the plastic neutral axis to the midthickness of the tension flange used to compute the plastic moment (in.)
d_w = distance from the plastic neutral axis to the middepth of the web used to compute the plastic moment (in.)
c_{rb} = distance from the top of the concrete deck to the centerline of the bottom layer of longitudinal concrete deck reinforcement (in.)
c_{rt} = distance from the top of the concrete deck to the centerline of the top layer of longitudinal concrete deck reinforcement (in.)
b_c = full width of the compression flange (in.)
b_t = full width of the tension flange (in.)
A_{rt} = area of the top layer of longitudinal reinforcement within the effective concrete deck width (in.^{2})
A_{rb} = area of the bottom layer of longitudinal reinforcement within the effective concrete deck width (in.^{2})
F_{yrt} = specified minimum yield strength of the top layer of longitudinal concrete deck reinforcement (ksi)
F_{yrb} = specified minimum yield strength of the bottom layer of longitudinal concrete deck reinforcement (ksi)

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D6.2—YIELD MOMENT

D6.2.1—Noncomposite Sections

The yield moment, M_y, of a noncomposite section shall be taken as the smaller of the moment required to cause nominal first yielding in the compression flange, M_{yc}, and the moment required to cause nominal first yielding in the tension flange, M_{yt}, at the strength limit state. Flange lateral bending in all types of sections and web yielding in hybrid sections shall be disregarded in this calculation. Longitudinal stiffeners should be included in calculating the section properties of the member gross cross section when determining the yield moment.

D6.2.2—Composite Sections in Positive Flexure

The yield moment of a composite section in positive flexure shall be taken as the sum of the moments applied separately to the steel and the short-term and long-term composite sections to cause nominal first yielding in either steel flange at the strength limit state. Flange lateral bending in all types of sections and web yielding in hybrid sections shall be disregarded in this calculation.

The yield moment of a composite section in positive flexure may be determined as follows:

- Calculate the moment M_{D1} caused by the factored permanent load applied before the concrete deck has hardened or is made composite. Apply this moment to the steel section.
- Calculate the moment M_{D2} caused by the remainder of the factored permanent load. Apply this moment to the long-term composite section.
- Calculate the additional moment M_{AD} that must be applied to the short-term composite section to cause nominal yielding in either steel flange.
- The yield moment is the sum of the total permanent load moment and the additional moment.

Symbolically, the procedure is:

1) Solve for M_{AD} from the equation:

$$F_{yf} = \frac{M_{D1}}{S_{NC}} + \frac{M_{D2}}{S_{LT}} + \frac{M_{AD}}{S_{ST}} \qquad \text{(D6.2.2-1)}$$

2) Then calculate:

$$M_y = M_{D1} + M_{D2} + M_{AD} \qquad \text{(D6.2.2-2)}$$

where:

S_{NC} = noncomposite section modulus (in.^{3})
S_{ST} = short-term composite section modulus (in.^{3})
S_{LT} = long-term composite section modulus (in.^{3})

M_{D1}, M_{D2},
& M_{AD} = moments due to the factored loads applied to the appropriate sections (kip-in.)

M_y shall be taken as the lesser value calculated for the compression flange, M_{yc}, or the tension flange, M_{yt}.

D6.2.3—Composite Sections in Negative Flexure

For composite sections in negative flexure, the procedure specified in Article D6.2.2 is followed, except that the composite section for both short-term and long-term moments shall consist of the steel section and the longitudinal reinforcement within the effective width of the concrete deck. Thus, S_{ST} and S_{LT} are the same value. Also, M_{yt} shall be taken with respect to either the tension flange or the longitudinal reinforcement, whichever yields first. For the calculation of M_{yt} with respect to the longitudinal reinforcement, M_{D1} shall be taken equal to zero in Eqs. D6.2.2-1 and D6.2.2-2, and F_{yf} in Eq. D6.2.2-1 shall be taken equal to the specified minimum yield strength of the longitudinal reinforcement.

D6.2.4—Sections with Cover Plates

For sections containing flange cover plates, M_{yc} or M_{yt} shall be taken as the smallest value of moment associated with nominal first yielding based on the stress in either the flange under consideration or in any of the cover plates attached to that flange, whichever yields first. Flange lateral bending in all types of sections and web yielding in hybrid sections shall be disregarded in this calculation.

D6.3—DEPTH OF THE WEB IN COMPRESSION

D6.3.1—In the Elastic Range (D_c)

For composite sections in positive flexure, the depth of the web in compression in the elastic range, D_c, shall be the depth over which the algebraic sum of the factored stresses in the steel, long-term composite and short-term composite sections from the dead and live loads, plus impact, is compressive.

In lieu of computing D_c at sections in positive flexure from stress diagrams, the following equation may be used:

$$D_c = \left(\frac{-f_c}{|f_c| + f_t} \right) d - t_{fc} \geq 0 \qquad \text{(D6.3.1-1)}$$

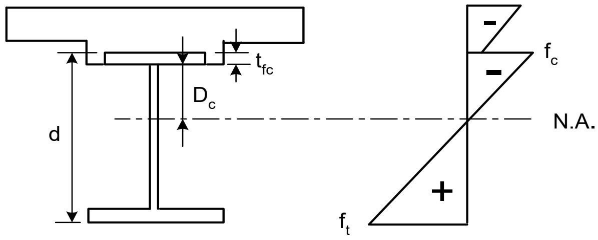

Figure D6.3.1-1—Computation of D_c at Sections in Positive Flexure

CD6.3.1

At sections in positive flexure, D_c of the composite section will increase with increasing span length because of the increasing dead-to-live load ratio. Therefore, in general it is important to recognize the effect of the dead-load stress on the location of the neutral axis of the composite section in regions of positive flexure.

According to these Specifications, for composite sections in positive flexure, Eq. D6.3.1-1 only need be employed for checking web bend-buckling at the service limit state and for computing the R_b factor at the strength limit state for sections in which web longitudinal stiffeners are required based on Article 6.10.2.1.1. Eq. D6.3.1-1 is never needed for composite sections in positive flexure when the web satisfies the requirement of Article 6.10.2.1.1 such that longitudinal stiffeners are not required. Articles C6.10.1.9.2, C6.10.1.10.2, and C6.10.4.2.2 discuss the rationale for these calculations, which introduce a dependency of the flexural resistance on the applied load whenever R_b < 1, and therefore, potentially complicate subsequent rating calculations for these section types. Article C6.10.1.9.1 explains why the calculation of D_c is not required for composite sections in positive flexure when the web satisfies Article 6.10.2.1.1.

where:

d = depth of the steel section (in.)

f_c = sum of the factored compression flange stresses caused by the different loads, i.e., *DC1*, the permanent load acting on the noncomposite section; *DC2*, the permanent load acting on the long-term composite section; *DW*, the wearing surface load; and *LL+IM*; acting on their respective sections (ksi). f_c shall be taken as negative when the stress is in compression. Flange lateral bending shall be disregarded in this calculation.

f_t = the sum of the factored tension-flange stresses caused by the different loads (ksi). Flange lateral bending shall be disregarded in this calculation.

For composite sections in negative flexure, D_c shall be computed for the section consisting of the steel girder plus the longitudinal reinforcement with the exception of the following. For composite sections in negative flexure at the service limit state where the concrete deck is considered effective in tension for computing flexural stresses on the composite section due to Load Combination Service II, D_c shall be computed from Eq. D6.3.1-1.

For composite sections in negative flexure, the concrete deck is typically not considered to be effective in tension. Therefore, the distance between the neutral axis locations for the steel and composite sections is small in this case and the location of the neutral axis for the composite section is largely unaffected by the dead-load stress. Therefore, for the majority of situations, these Specifications specify the use of D_c computed simply for the section consisting of the steel girder plus the longitudinal reinforcement, without considering the algebraic sum of the stresses acting on the noncomposite and composite sections. This eliminates potential difficulties in subsequent load rating since the resulting D_c is independent of the applied loading, and therefore the flexural resistance in negative bending, which depends on D_c, does not depend on the applied load. The single exception is that if the concrete deck is assumed effective in tension in regions of negative flexure at the service limit state, as permitted for composite sections satisfying the requirements specified in Article 6.10.4.2.1, Eq. D6.3.1-1 must be used to compute D_c. For this case, in Figure D6.3.1-1, the stresses f_c and f_t should be switched, the signs shown in the stress diagram should be reversed, t_{fc} should be the thickness of the bottom flange, and D_c should instead extend from the neutral axis down to the top of the bottom flange.

D6.3.2—At Plastic Moment (D_{cp})

For composite sections in positive flexure, the depth of the web in compression at the plastic moment, D_{cp}, shall be taken as follows for cases from Table D6.1-1 where the plastic neutral axis is in the web:

$$D_{cp} = \frac{D}{2}\left(\frac{F_{yt}A_t - F_{yc}A_c - 0.85f'_cA_s - F_{yrs}A_{rs}}{F_{yw}A_w} + 1\right) \quad \text{(D6.3.2-1)}$$

where:

A_c = area of the compression flange (in.^{2})

A_{rs} = total area of the longitudinal reinforcement within the effective concrete deck width (in.^{2})

A_s = area of the concrete deck (in.^{2})

A_t = area of the tension flange (in.^{2})

A_w = area of the web (in.^{2})

D_{cp} = depth of the web in compression at the plastic moment (in.)

F_{yrs} = specified minimum yield strength of the longitudinal reinforcement (ksi)

For all other composite sections in positive flexure, D_{cp} shall be taken equal to zero.

For composite sections in negative flexure, D_{cp} shall be taken as follows for cases from Table D6.1-2 where the plastic neutral axis is in the web:

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$$D_{cp} = \frac{D}{2A_w F_{yw}}\left[F_{yt}A_t + F_{yw}A_w + F_{yrs}A_{rs} - F_{yc}A_c\right] \tag{D6.3.2-2}$$

For all other composite sections in negative flexure, D_{cp} shall be taken equal to D.

For noncomposite sections where:

$$F_{yw}A_w \geq \left|F_{yc}A_c - F_{yt}A_t\right| \tag{D6.3.2-3}$$

D_{cp} shall be taken as:

$$D_{cp} = \frac{D}{2A_w F_{yw}}\left[F_{yt}A_t + F_{yw}A_w - F_{yc}A_c\right] \tag{D6.3.2-4}$$

For all other noncomposite sections, D_{cp} shall be taken equal to D.

D6.4—LATERAL–TORSIONAL BUCKLING EQUATIONS FOR C_B > 1.0, WITH EMPHASIS ON UNBRACED LENGTH REQUIREMENTS FOR DEVELOPMENT OF THE MAXIMUM FLEXURAL RESISTANCE

D6.4.1—By the Provisions of Article 6.10.8.2.3

For unbraced lengths in which the member is prismatic, the lateral–torsional buckling resistance of the compression flange shall be taken as:

- If $L_b \leq L_p$, then:

$$F_{nc} = R_b R_h F_{yc} \tag{D6.4.1-1}$$

- If $L_p < L_b \leq L_r$, then:

  - If $L_b \leq L_p + \dfrac{\left(1-\dfrac{1}{C_b}\right)}{\left(1-\dfrac{F_{yr}}{R_h F_{yc}}\right)}\left(L_r - L_p\right)$, then:

$$F_{nc} = R_b R_h F_{yc} \tag{D6.4.1-2}$$

Otherwise:

$$F_{nc} = C_b\left[1-\left(1-\frac{F_{yr}}{R_h F_{yc}}\right)\left(\frac{L_b - L_p}{L_r - L_p}\right)\right]R_b R_h F_{yc} \leq R_b R_h F_{yc} \tag{D6.4.1-3}$$

- If $L_b > L_r$, then:

CD6.4.1

For values of the moment gradient modifier C_b greater than 1.0, the maximum lateral–torsional buckling resistance F_{max} shown in Figure C6.10.8.2.1-1 may be reached at larger unbraced lengths. The provisions in this Article are equivalent to those in Articles 6.10.8.2.3, but allow the Engineer to focus on the conditions for which the lateral–torsional buckling resistance is equal to $F_{max} = R_bR_hF_{yc}$ when the effects of moment gradient are included in determining the limits on L_b.

The largest unbraced length for which the lateral–torsional buckling resistance of Article 6.10.8.2.3 is equal to the flange local buckling or flange–flange local buckling resistance of Article 6.10.8.2.2 may be determined by substituting $F_{nc(FLB)}/R_b$ for R_hF_{yc} in checking the L_b requirement for the use of Eq. D6.4.1-2 or D6.4.1-4 as applicable, where $F_{nc(FLB)}$ is the flange–flange local buckling resistance obtained from Article 6.10.8.2.2.

If $D_ct_w/b_{fc}t_{fc}$ in Eq. 6.10.8.2.3-9 is taken as a representative value of 2.0, F_{yc} is taken as 50 ksi, and $F_{yw} > 0.7F_{yc}$, the lateral–torsional buckling resistance of Article 6.10.8.2.3 is equal to F_{max} for $L_b < 22b_{fc}$ when C_b > 1.75 and $L_b < 17b_{fc}$ for C_b > 1.3. The Engineer should note that, even with relatively small values of C_b, the unbraced length requirements to achieve a flexural resistance of F_{max} are significantly larger than those associated with uniform major-axis bending and C_b = 1. Article C6.10.8.2.3 discusses the appropriate calculation of C_b > 1 for bridge design.

- If $L_b \le \pi r_t \sqrt{\dfrac{C_b E}{R_h F_{yc}}}$, then:

$$F_{nc} = R_b R_h F_{yc} \quad \text{(D6.4.1-4)}$$

- Otherwise:

$$F_{nc} = F_{cr} \le R_b R_h F_{yc} \quad \text{(D6.4.1-5)}$$

All terms in the above equations shall be taken as defined in Article 6.10.8.2.3.

D6.4.2—By the Provisions of Article A6.3.3

For unbraced lengths in which the member is prismatic, the flexural resistance based on lateral–torsional buckling shall be taken as:

- If $L_b \le L_p$, then:

$$M_{nc} = R_{pc} M_{yc} \quad \text{(D6.4.2-1)}$$

- If $L_p < L_b \le L_r$, then:

  - If $L_b \le L_p + \dfrac{\left(1-\dfrac{1}{C_b}\right)}{\left(1-\dfrac{F_{yr}S_{xc}}{R_{pc}M_{yc}}\right)}\left(L_r - L_p\right)$, then:

$$M_{nc} = R_{pc} M_{yc} \quad \text{(D6.4.2-2)}$$

  - Otherwise:

$$M_{nc} = C_b \left[1-\left(1-\frac{F_{yr}S_{xc}}{R_{pc}M_{yc}}\right)\left(\frac{L_b - L_p}{L_r - L_p}\right)\right] R_{pc}M_{yc} \le R_{pc}M_{yc} \quad \text{(D6.4.2-3)}$$

- If $L_b > L_r$, then:

  - If :

$$L_b \le 1.95 r_t \frac{C_b S_{xc} E}{R_{pc} M_{yc}} \sqrt{\frac{J}{S_{xc} h}} \sqrt{1+\sqrt{1+6.76\left(\frac{R_{pc}M_{yc}}{C_b S_{xc} E}\frac{S_{xc}h}{J}\right)^2}}$$

then:

$$M_{nc} = R_{pc} M_{yc} \quad \text{(D6.4.2-4)}$$

  - Otherwise:

$$M_{nc} = F_{cr} S_{xc} \le R_{pc} M_{yc} \quad \text{(D6.4.2-5)}$$

All terms in the above equations shall be taken as defined in Article A6.3.3.

CD6.4.2

For values of the moment gradient modifier C_b greater than 1.0, the maximum lateral–torsional buckling resistance M_{max} shown in Figure C6.10.8.2.1-1 may be reached at larger unbraced lengths. The provisions in this Article are equivalent to those in Article A6.3.3, but allow the Engineer to focus on the conditions for which the lateral–torsional buckling resistance is equal to $M_{max} = R_{pc}M_{yc}$ when the effects of moment gradient are included in determining the limits on L_b.

The largest unbraced length for which the lateral–torsional buckling resistance of Article A6.3.3 is equal to the flange local buckling or flange–local buckling resistance of Article A6.3.2 may be determined by substituting $M_{nc(FLB)}$ for $R_{pc}M_{yc}$ in checking the L_b requirement for the use of Eq. D6.4.2-2 or D6.4.2-4 as applicable, where $M_{nc(FLB)}$ is the flange–local buckling resistance obtained from Article A6.3.2.

Article A6.3.3 typically requires similar to somewhat smaller values than Article 6.10.8.2.3 for the limits on L_b required to reach the member resistance of $M_{max} > R_hM_{yc}$, depending on the magnitude of R_{pc}. If $D_ct_w/b_{fc}t_{fc}$ in Eq. A6.3.3-10 is taken as a representative value of 2.0, F_{yc} is taken as 50 ksi, and $F_{yw} > 0.7F_{yc}$, then for $R_{pc} = 1.12$, the lateral–torsional buckling resistance of this Article is typically equal to M_{max} when $L_b < 22b_{fc}$ for $C_b > 1.75$ and when $L_b < 15b_{fc}$ for $C_b > 1.30$. For $R_{pc} = 1.30$ and using the above assumptions, M_{max} is achieved by the lateral–torsional buckling equations when $L_b < 20b_{fc}$ for $C_b > 1.75$ and when $L_b < 13b_{fc}$ for $C_b > 1.30$. The Engineer should note that, even with relatively small values of C_b, the unbraced length requirements to achieve a flexural resistance of M_{max} are significantly larger than those associated with uniform major-axis bending and $C_b = 1$. Article C6.10.8.2.3 discusses the appropriate calculation of $C_b > 1$ for bridge design.

D6.5—CONCENTRATED LOADS APPLIED TO WEBS WITHOUT BEARING STIFFENERS

D6.5.1—General

At bearing locations and at other locations subjected to concentrated loads, where the loads are not transmitted through a deck or deck system, webs without bearing stiffeners shall be investigated for the limit states of web local yielding and web crippling according to the provisions of Articles D6.5.2 and D6.5.3.

CD6.5.1

The equations of this Article are essentially identical to the equations given in AISC (2016b). The limit state of sidesway web buckling given in AISC (2016b) is not included because it governs only for members subjected to concentrated loads directly applied to the steel section, and for members for which the compression flange is braced at the load point, the tension flange is unbraced at this point, and the ratio of D/t_w to L_b/b_{ft} is less than or equal to 1.7. These conditions typically do not occur in bridge construction.

Built-up sections and rolled shapes without bearing stiffeners at the indicated locations should either be modified to comply with these requirements, or else bearing stiffeners designed according to the provisions of Article 6.10.11.2 should be placed on the web at the location under consideration.

For unusual situations in which diametrically opposed concentrated loads are directly applied to the web of the steel section at the level of each of the flanges, such as if a concentrated force were applied directly over a reaction point at an unstiffened location along the length of a girder, the AISC (2016b) provisions pertaining to additional stiffener requirements for concentrated forces should be considered.

D6.5.2—Web Local Yielding

Webs subject to compressive or tensile concentrated loads shall satisfy:

$$R_u \leq \phi_b R_n \quad \text{(D6.5.2-1)}$$

in which:

R_n = nominal resistance to the concentrated loading (kip)

- For interior-pier reactions and for concentrated loads applied at a distance from the end of the member that is greater than d:

$$R_n = (5k + N) F_{yw} t_w \quad \text{(D6.5.2-2)}$$

- Otherwise:

$$R_n = (2.5k + N) F_{yw} t_w \quad \text{(D6.5.2-3)}$$

where:

ϕ_b = resistance factor for bearing specified in Article 6.5.4.2

CD6.5.2

This limit state is intended to prevent localized yielding of the web resulting from either high compressive or tensile stress due to a concentrated load or bearing reaction.

A concentrated load acting on a rolled shape or a built-up section is assumed critical at the toe of the fillet located a distance k from the outer face of the flange resisting the concentrated load or bearing reaction, as applicable. For a rolled shape, k is published in the available tables giving dimensions for the shapes. For a built-up section, k may be taken as the distance from the outer face of the flange to the web toe of the web-to-flange fillet weld.

In Eq. D6.5.2-2 for interior loads or interior-pier reactions, the load is assumed to distribute along the web at a slope of 2.5 to 1 and over a distance of $(5k + N)$. An interior concentrated load is defined as a load applied at a distance from the end of the member that is greater than the depth of the steel section, d. In Eq. D6.5.2-3 for end loads or end reactions, the load is assumed to distribute along the web at the same slope over a distance of $(2.5k + N)$. These criteria are largely based on the work of Johnston and Kubo (1941) and Graham et al. (1959).

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d = depth of the steel section (in.)
k = distance from the outer face of the flange resisting the concentrated load or bearing reaction to the web toe of the fillet (in.)
N = length of bearing (in.). N shall be greater than or equal to k at end bearing locations.
R_u = factored concentrated load or bearing reaction (kip)

D6.5.3—Web Crippling

Webs subject to compressive concentrated loads shall satisfy:

$$R_u \leq \phi_w R_n \quad \text{(D6.5.3-1)}$$

in which:

R_n = nominal resistance to the concentrated loading (kip)

- For interior-pier reactions and for concentrated loads applied at a distance from the end of the member that is greater than or equal to d/2:

$$R_n = 0.8t_w^2\left[1+3\left(\frac{N}{d}\right)\left(\frac{t_w}{t_f}\right)^{1.5}\right]\sqrt{\frac{EF_{yw}t_f}{t_w}} \quad \text{(D6.5.3-2)}$$

- Otherwise:
  - If $N/d \leq 0.2$, then:

$$R_n = 0.4t_w^2\left[1+3\left(\frac{N}{d}\right)\left(\frac{t_w}{t_f}\right)^{1.5}\right]\sqrt{\frac{EF_{yw}t_f}{t_w}} \quad \text{(D6.5.3-3)}$$

  - If $N/d > 0.2$, then:

$$R_n = 0.4t_w^2\left[1+\left(\frac{4N}{d}-0.2\right)\left(\frac{t_w}{t_f}\right)^{1.5}\right]\sqrt{\frac{EF_{yw}t_f}{t_w}} \quad \text{(D6.5.3-4)}$$

where:

ϕ_w = resistance factor for web crippling specified in Article 6.5.4.2
t_f = thickness of the flange resisting the concentrated load or bearing reaction (in.)

CD6.5.3

This limit state is intended to prevent local instability or crippling of the web resulting from a high compressive stress due to a concentrated load or bearing reaction.

Eqs. D6.5.3-2 and D6.5.3-3 are based on research by Roberts (1981). Eq. D6.5.3-4 for $N/d > 0.2$ was developed after additional testing by Elgaaly and Salkar (1991) to better represent the effect of longer bearing lengths at the ends of members.

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APPENDIX E6—NOMINAL COMPRESSIVE RESISTANCE OF NONCOMPOSITE MEMBERS CONTAINING LONGITUDINALLY STIFFENED PLATES

E6.1—NOMINAL COMPRESSIVE RESISTANCE

E6.1.1—General

For noncomposite compression member cross sections containing any longitudinally stiffened plates, the nominal compressive resistance, P_n, shall be taken as the smallest value based on the applicable modes of flexural buckling, torsional buckling, and flexural–torsional buckling, and shall be computed as follows:

$$P_n = \chi F_{cr} A_{eff} \quad \text{(E6.1.1-1)}$$

in which:

- For noncomposite rectangular box cross sections containing one or more longitudinally stiffened flange plates in the direction associated with column flexural buckling, and where $\lambda_{max} > \lambda_r$,

$$\chi = 1 - r_1 r_2 \quad \text{(E6.1.1-2)}$$

$$r_1 = 0.5\left(K\ell / r_s - 50\right)/90 \geq 0 \quad \text{(E6.1.1-3)}$$

$$r_2 = \frac{\lambda_{\max} - \lambda_r}{90 - \lambda_r} \geq 0 \quad \text{(E6.1.1-4)}$$

- Otherwise:

$$\chi = 1.0 \quad \text{(E6.1.1-5)}$$

- If $\frac{P_{os}}{P_e} \leq 2.25$:

$$F_{cr} = \left[0.658^{\left(\frac{P_{os}}{P_e}\right)}\right] F_y \quad \text{(E6.1.1-6)}$$

- Otherwise:

$$F_{cr} = 0.877 \frac{P_e}{P_{os} / F_y} \quad \text{(E6.1.1-7)}$$

A_{eff} = effective area of the cross section (in.^{2})

$$= \sum_{lusp} b_e\, t + \sum_{c} A_c + \sum_{lsp} \left(A_{eff}\right)_{sp} \quad \text{(E6.1.1-8)}$$

P_{os} = nominal yield resistance (kip)

$$= F_y \left(\sum_{lusp} bt + \sum_{c} A_c + \sum_{lsp} \left(A_{eff}\right)_{sp}\right) \quad \text{(E6.1.1-9)}$$

CE6.1.1

This Article implements an extension of the unified effective width/unified effective area method, described further in Article C6.9.4.2.2a, incorporating the consideration of longitudinally stiffened component plates. Eqs. E6.1.1-1 through E6.1.1-9 capture the influence of buckling of individual longitudinally unstiffened and longitudinally stiffened plates on the overall member axial compressive resistance in a simple yet accurate to conservative manner (White et al., 2019b).

Longitudinally stiffened plates are addressed by adding their effective cross-sectional area, $(A_{eff})_{sp}$, calculated based on the yield strength of the stiffened plate, with the gross areas of the other components of the cross section to determine the nominal yield resistance of the cross section, P_{os}, given by Eq. E6.1.1-9. In addition, $(A_{eff})_{sp}$ is combined with the effective areas of any longitudinally unstiffened plates and the gross area of the corners of the box-section in Eq. E6.1.1-8 to determine the cross-section effective area. White et al. (2019b) explain the rationale for this approach.

Eqs. E6.1.1-2 through E6.1.1-4 define a strength reduction factor, χ, accounting for local-global buckling interaction effects in noncomposite rectangular box-section members having longitudinally stiffened flange plates with slender panels between the stiffeners in the direction of column flexural buckling. The flange plates are defined as the plates parallel to the axis of buckling, i.e., they are subjected to uniform flexural compression from the bending associated with column flexural buckling.

If λ_{max} is less than or equal to λ_r for the panels of the stiffened flange plates under consideration, χ is equal to 1.0.

The χ factor given by Eqs. E6.1.1-2 to E6.1.1-4 is applicable for specified minimum yield strengths up to F_y = 70 ksi, and for $\lambda_{max} \leq 90$ and $K\ell/r_s \leq 140$, which are limits specified in Articles 6.12.2.2.2b and 6.9.3.

For nonhomogeneous members, this Article specifies that F_y may conservatively be taken as the smallest specified yield strength of all the cross-section elements for the calculation of F_{cr}. White et al. (2019b) discuss a more rigorous approach for the determination of the axial compressive resistance of nonhomogeneous members with doubly-symmetric I- and box-section profiles.

The terms in Eq. E6.1.1-8 and in Eq. E6.1.1-9 for the effective area of the cross-section, A_{eff}, and nominal yield resistance, P_{os}, are taken as zero when the cross section does not contain the corresponding plate element type; for example, the first term of Eqs. E6.1.1-8 and E6.1.1-9 is

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where:

λ_{max} = maximum w/t of the panels within the longitudinally stiffened flange plate under consideration

λ_r = nonslender limit for longitudinally stiffened plate panels defined in Article E6.1.2

$\sum_{lusp}$ = summation over all longitudinally unstiffened cross-section plates

$\sum_{c}$ = summation over all the corner areas of a noncomposite box-section member

$\sum_{lsp}$ = summation over all the longitudinally stiffened cross-section plates

A_c = gross cross-sectional area of the corner pieces of a noncomposite box-section member (in.^{2})

$\left(A_{eff}\right)_{sp}$ = effective area of the longitudinally stiffened plate under consideration determined as specified in Article E6.1.3 (in.^{2})

b = width of the longitudinally unstiffened plate under consideration determined as specified in Table 6.9.4.2.1-1 (in.)

b_e = effective width of the longitudinally unstiffened plate under consideration determined as specified in Article 6.9.4.2.2b for slender plate elements, and taken equal to b for nonslender plate elements (in.)

F_y = specified minimum yield strength (ksi); for nonhomogeneous cross-section members, F_y may be taken as the smallest specified minimum yield strength of all the cross-section elements in lieu of a more refined calculation

$K\ell$ = effective length in the plane of buckling (in.)

P_e = elastic critical buckling resistance determined as specified in Article 6.9.4.1.2 for flexural buckling, and as specified in Article 6.9.4.1.3 for torsional buckling or flexural–torsional buckling, as applicable, using the gross section properties (kip)

r_s = radius of gyration about the axis normal to the plane of buckling calculated using the gross section properties (in.)

t = thickness of the longitudinally unstiffened plate under consideration (in.)

w = width of the flange plate between the centerlines of the individual longitudinal stiffeners or between the centerline of the longitudinal stiffener and the inside of the laterally-restrained longitudinal edge of the longitudinally stiffened plate under consideration, as applicable, equal to the

taken as zero if the cross section contains only longitudinally stiffened plates. The second term of Eqs. E6.1.1-8 and E6.1.1-9 is the total gross cross-sectional area contributed by the four corner pieces of a noncomposite box-section member not included in the clear width of the component plates; for all other members, A_c is taken as zero. Flange extensions on box-section members, if present, should be evaluated to determine if they are nonslender or slender plate elements and included accordingly in Eqs. E6.1.1-8 and E6.1.1-9.

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tributary width in the case of equally-spaced longitudinal stiffeners (in.)

For all cross-section plates that are supported along two longitudinal edges and are slender as defined in Article 6.9.4.2.2a, and for slender panels of longitudinally stiffened plates as defined in Article E6.1.2, the provisions of Article 6.9.4.5 also shall be satisfied.

E6.1.2—Classification of Longitudinally Stiffened Plate Panels

Longitudinally stiffened plate panels satisfying the following limit shall be defined as nonslender under uniform axial compression:

$$\frac{w}{t} \le \lambda_r \qquad \text{(E6.1.2-1)}$$

where:

λ_r = nonslender limit for longitudinally stiffened plate panels, equal to $1.09\sqrt{E/F_{ysp}}$ if the plate has one longitudinal stiffener, or $1.49\sqrt{E/F_{ysp}}$ if the plate has two or more longitudinal stiffeners

w = width of the plate between the centerlines of the individual longitudinal stiffeners or between the centerline of the longitudinal stiffener and the inside of the laterally-restrained longitudinal edge of the longitudinally stiffened plate under consideration, as applicable, equal to the tributary width in the case of equally-spaced longitudinal stiffeners (in.)

t = thickness of the longitudinally stiffened plate under consideration (in.)

F_{ysp} = specified minimum yield strength of the longitudinally stiffened plate under consideration (ksi)

Local buckling effects shall be neglected for nonslender longitudinally stiffened plate panels. Otherwise, a panel of a longitudinally stiffened plate shall be defined as slender under uniform axial compression and its local buckling effects shall be considered using the provisions of Articles E6.1.1 and E6.1.3.

CE6.1.2

This Article defines the nonslender limit, λ_r, for panels of longitudinally stiffened plates. For panels in plates containing only one longitudinal stiffener, the general λ_r limit from Table 6.9.4.2.1-1 for plate elements supported along two longitudinal edges is employed. For panels in plates containing two or more longitudinal stiffeners, the larger value $1.49\sqrt{E/F_{ysp}}$ is adopted recognizing the larger net buckling and postbuckling resistance due to the edge restraint conditions from adjacent panels (Lokhande and White, 2018).

E6.1.3—Nominal Compressive Resistance and Effective Area of Plates with Equally-Spaced Equal-Size Longitudinal Stiffeners

The nominal compressive resistance of plates with equally-spaced equal-size longitudinal stiffeners, P_{nsp}, shall be determined as follows:

CE6.1.3

Longitudinal stiffening of web and/or flange plates for compressive resistance can be important to the overall design economy of large box girders, arch ribs and tie girders, and steel towers in longer-span steel bridges.

$$P_{nsp} = nP_{ns} + 2P_{nR} \tag{E6.1.3-1}$$

in which:

P_{ns} = nominal compressive resistance of an individual stiffener strut composed of the stiffener plus the tributary width of the longitudinally stiffened plate under consideration (kip)

$$= P_{nsF} + 0.15P_{esT} \leq P_{yes} \tag{E6.1.3-2}$$

and

P_{nR} = nominal compressive resistance provided by an individual laterally-restrained longitudinal edge of the longitudinally stiffened plate under consideration (kip)

$$= \left(1 - \frac{P_{ns}}{P_{yes}}\right)\left[0.45\left(F_{ysp} + \frac{P_{ns}}{A_{es}}\right)A_{gR}\right] + \left(\frac{P_{ns}}{P_{yes}}\right)P_{yeR} \leq P_{yeR} \tag{E6.1.3-3}$$

where:

n = number of longitudinal stiffeners

The following terms apply to the calculation of P_{ns}:

P_{nsF} = nominal flexural buckling resistance of an individual stiffener strut (kip) determined as follows:

- If $\frac{P_{ys}}{P_{esF}} \leq 2.25$, then:

$$P_{nsF} = 0.658^{\frac{P_{ys}}{P_{esF}}} P_{yes} \tag{E6.1.3-4}$$

- Otherwise:

$$P_{nsF} = 0.877\frac{P_{esF}}{A_{gs}}A_{es} \tag{E6.1.3-5}$$

P_{esF} = elastic flexural buckling resistance of an individual stiffener strut (kip)

$$= \frac{\pi^2 EI_s}{\ell^2} + \frac{\pi^2 EwI_p}{b_{sp}^4}\ell^2 \tag{E6.1.3-6}$$

I_p = lateral moment of inertia of a unit width of the longitudinally stiffened plate under consideration (in.^{3})

Longitudinal stiffening can be beneficial when reduction of structural weight is a premium, and where the design stresses developed by a corresponding longitudinally unstiffened plate satisfying the strength requirements are relatively low due to local buckling effects. In addition, longitudinal stiffening can be beneficial for large plate widths where the required thickness needed to satisfy the strength demands is not available using a longitudinally unstiffened plate.

Typically, longitudinal stiffening should not be considered for total plate widths less than about 60.0 in. Longitudinally unstiffened plates are usually more economical in these cases; thickening the plate rather than adding longitudinal stiffeners may also be more economical for plate widths larger than 60.0 in.

Articles E6.1.3 through E6.1.5 provide a streamlined intuitive approach for the design of longitudinally stiffened plates. Article E6.1.3 addresses the compressive resistance of plates designed using equally-spaced, equal-size longitudinal stiffeners. White et al. (2019b) provide an extension of these provisions for calculation of the compressive resistance of stiffened plates using unequally-spaced and/or unequal-size longitudinal stiffeners. These types of plates can be addressed conservatively by neglecting the presence of the longitudinal stiffener or stiffeners and calculating the resistance of the hypothetical longitudinally unstiffened plate.

Eq. E6.1.3-1 implements a unified method for the design of longitudinally stiffened plates with or without transverse stiffeners developed by King (2017) and Lokhande and White (2018). The method considers explicitly the influence of plate bending, plate torsion and longitudinal stiffener flexure, and is derivable both from column on elastic foundation and orthotropic plate buckling idealizations. These three contributions to the buckling resistance are combined in a manner that characterizes the longer strength plateau associated with plate buckling. Explicit combination of the three contributions to the stiffened plate compressive resistance facilitates design optimization since the relative importance of each effect is clear.

The characteristic buckling length of the longitudinal stiffener struts, which is the theoretical length between the inflection points within the buckling mode of the struts for an infinitely long plate, is calculated directly, such that the impact of transverse stiffener or diaphragm spacing can be directly ascertained. The method recognizes the postbuckling resistance of the plate panels between the longitudinal stiffeners, and/or between the longitudinal stiffeners and the laterally-restrained longitudinal edge of the stiffened plate. The method also recognizes that the edge stress is larger than the ultimate stress of the stiffener strut, and it takes into account the observation that the edge stress is typically less than yield stress under the ultimate strength condition (Lokhande and White, 2018).

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$$= \frac{t_{sp}^3}{12(1-\nu^2)} \qquad \text{(E6.1.3-7)}$$

ℓ = buckling length of the individual stiffener struts, taken equal to the smaller of a and ℓ_c (in.), where:

- a = longitudinal spacing between locations of transverse stiffeners or diaphragms that provide transverse lateral restraint to the longitudinally stiffened plate under consideration (in.)
- ℓ_c = characteristic buckling length of the stiffener struts of the longitudinally stiffened plate under consideration (in.)

$$= \left(\frac{I_s}{wI_p}\right)^{1/4} b_{sp} \qquad \text{(E6.1.3-8)}$$

A_{es} = effective area of an individual stiffener strut (in.^{2})

$$= A_s + w_e t_{sp} \qquad \text{(E6.1.3-9)}$$

A_{gs} = gross area of an individual stiffener strut (in.^{2})

$$= A_s + wt_{sp} \qquad \text{(E6.1.3-10)}$$

P_{esT} = plate torsional stiffness contribution to the elastic buckling resistance of an individual stiffener strut (kip)

$$= \frac{\pi^2}{(1-\nu)b_{sp}^2}\frac{Gwt_{sp}^3}{3} \qquad \text{(E6.1.3-11)}$$

P_{yes} = effective yield load of an individual stiffener strut (kip)

$$= F_{ysp}A_{es} \qquad \text{(E6.1.3-12)}$$

P_{ys} = yield load of an individual stiffener strut (kip)

$$= F_{ysp}A_{gs} \qquad \text{(E6.1.3-13)}$$

where:

ν = Poisson's ratio = 0.3
A_s = gross area of an individual longitudinal stiffener, excluding the tributary width of the longitudinally stiffened plate under consideration (in.^{2})
b_{sp} = total width of the longitudinally stiffened plate, taken as the inside distance between the plates providing lateral restraint to its longitudinal edges (in.)
F_{ysp} = specified minimum yield strength of the longitudinally stiffened plate under consideration (ksi)
G = shear modulus of elasticity for steel = 0.385E (ksi)

This procedure provides a more explicit, general, accurate and transparent evaluation of the influence of longitudinal and transverse stiffening than prior procedures. Furthermore, it avoids anomalies that occur for certain geometries in the Eurocode Part 1-5 procedures (CEN, 2006), which employ an interpolation between a strength curve for unstiffened plates and a buckling curve for compression members.

The term P_{nsF} in Eq. E6.1.3-2 addresses the contribution from the flexural buckling resistance of the longitudinal stiffener struts, including the assistance from the transverse bending stiffness of the plate, via the elastic buckling load term P_{esF}. For cases where the spacing of the transverse stiffening elements, a, is greater than the characteristic buckling length, ℓ_c, given by Eq. E6.1.3-8, and therefore $\ell = \ell_c$, the two contributions to P_{esF} in Eq. E6.1.3-6 are equal and the elastic buckling load may be calculated simply as

$$P_{esF} = \frac{2\pi^2 E}{b_{sp}^2}\sqrt{wI_pI_s} \qquad \text{(CE6.1.3-1)}$$

Alternatively, when $\ell = \ell_c$, P_{esF} can simply be taken as two times the result from the first term of Eq. E6.1.3-6.

Given the stiffener strut elastic flexural buckling resistance, the strut nominal flexural buckling resistance is quantified by the familiar AISC/AASHTO column strength equations, Eqs. E6.1.3-4 and E6.1.3-5.

Eq. E6.1.3-2 also addresses the contribution from the torsional stiffness of the plate, via the term 0.15P_{esT}. The maximum value of P_{ns} is limited to the yield load of the stiffener strut including the effective width of the plate panels tributary to the longitudinal stiffener. The term 0.15P_{esT} in Eq. E6.1.3-2 captures the plate elastic torsional stiffness contribution to the resistance of the stiffener struts. This torsional stiffness contribution can be significant for narrow plates with a single longitudinal stiffener, and leads to an increase in strength up to about 7 percent and a lengthening of the strength plateau for these types of plates.

The calculations of this Article may be simplified in certain cases:

- Due to the influence of the 0.15P_{esT} term, P_{ns} may be taken equal to the full yield strength of the stiffener struts, P_{ys}, when $P_{ys}/P_{esF} \le 0.2$ in designs with a single longitudinal stiffener, $\ell = \ell_c$, and when the plate panels are nonslender as defined in Article E6.1.2. Otherwise, when $P_{ys}/P_{esF} \le 0.1$, the effects of the longitudinal stiffener slenderness are small and P_{ns} in Eq. E6.1.3-2 may be taken equal to P_{yes}.
- For plates with two or more longitudinal stiffeners, the contribution of 0.15P_{esT} to the resistance in Eq. E6.1.3-2 is relatively small and may be neglected.

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I_s = moment of inertia of an individual stiffener strut composed of the stiffener plus the gross tributary width, w, of the longitudinally stiffened plate under consideration, taken about an axis parallel to the face of the longitudinally stiffened plate and passing through the centroid of the combined area of the longitudinal stiffener and its gross tributary plate width (in.^{4})

t_{sp} = thickness of the longitudinally stiffened plate under consideration (in.)

w = width of the plate between the centerlines of the individual longitudinal stiffeners or between the centerline of the longitudinal stiffener and the inside of the laterally-restrained longitudinal edge of the longitudinally stiffened plate under consideration, as applicable, equal to the gross tributary width in the case of equally-spaced longitudinal stiffeners (in.)

w_e = effective width of the plate tributary to each stiffener strut, taken as the corresponding value of b_e calculated as specified in Article 6.9.4.2.2b with w substituted for b, with F_{cr} taken as F_{ysp} and with λ_r taken as specified in Article E6.1.2 (in.)

The following additional terms apply to the calculation of P_{nR}:

A_{gR} = gross tributary area of the laterally-restrained longitudinal edge of the longitudinally stiffened plate under consideration (in.^{2})

$$=\frac{w}{2}t_{sp} \quad \text{(E6.1.3-14)}$$

P_{yeR} = effective yield load of an individual laterally-restrained longitudinal edge of the longitudinally stiffened plate under consideration (kip)

$$= F_{ysp}A_{eR} \quad \text{(E6.1.3-15)}$$

A_{eR} = effective tributary area of the laterally-restrained longitudinal edge of the longitudinally stiffened plate under consideration (in.^{2})

$$=\frac{w_e}{2}t_{sp} \quad \text{(E6.1.3-16)}$$

The longitudinal stiffeners shall satisfy the requirements of Article E6.1.4. Transverse stiffeners, when utilized to strengthen or stiffen a longitudinally stiffened plate, shall satisfy the requirements of Article E6.1.5.

The effective area of plates with equally-spaced equal-size longitudinal stiffeners shall be calculated as follows:

- For plates with nonslender plate panels as defined in Article E6.1.2, the plate panels do not experience any reduction in strength due to local buckling effects. Therefore, $w_e = w$, $A_{es} = A_{gs}$, and $P_{yes} = P_{ys}$ for these cases.

The term P_{nR} from Eq. E6.1.3-3 gives the contribution from the laterally-restrained longitudinal edges of a longitudinally stiffened plate. This equation specifies a simple linear interpolation between the yield load of the edge, P_{yeR}, based on the plate effective width tributary to the edge, in the limit that P_{ns} is equal to P_{yes}, and the compression force given by $0.45(F_{ysp} + P_{ns}/A_{es})$, acting on A_{gR}, in the limit that P_{ns} becomes small.

Figure CE6.1.3-1 illustrates the definition of a number of the variables in this Article.

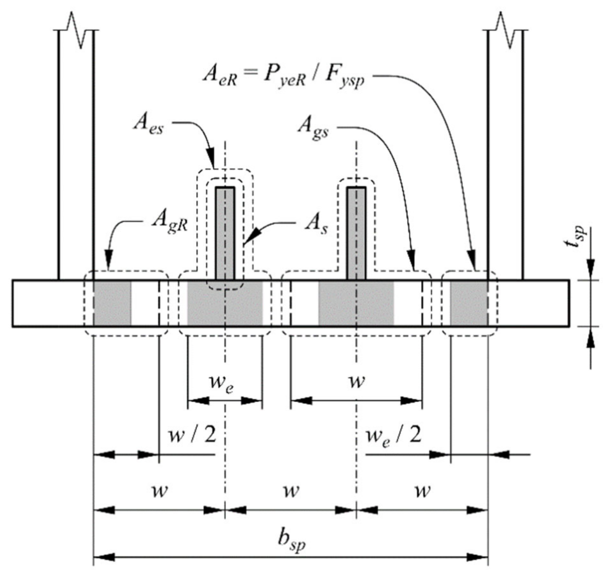

Figure CE6.1.3-1—Illustration of Variables for a Longitudinally Stiffened Plate

When the spacing between transverse stiffeners and/or diaphragms is smaller than the characteristic buckling length, ℓ_c, the buckling length of the stiffener struts, ℓ, is taken as the corresponding spacing, a. In this situation, the strength of the stiffened plate is increased due to the transverse stiffening. Otherwise, the spacing of the transverse stiffeners does not have any significant impact on the strength of the stiffened plate. The characteristic buckling length, ℓ_c, is the theoretical length between the inflection points within the buckling mode of the stiffener struts for an infinitely long plate.

The effective area of longitudinally stiffened plates, $(A_{eff})_{sp}$, is employed in Article E6.1.1 in the calculation of the axial compressive resistance of members containing these types of plate elements.

$$\left(A_{eff}\right)_{sp} = \frac{P_{nsp}}{F_{ysp}} \qquad \text{(E6.1.3-17)}$$

E6.1.4—Longitudinal Stiffeners

Longitudinal stiffeners should consist of a flat rectangular plate, a rib, an angle, or a T-section welded to one side of the plate. The specified minimum yield strength of the stiffeners should not be less than the specified minimum yield strength of the plate to which they are attached.

Longitudinal stiffeners shall be structurally continuous over their specified length and should be continuously welded to the plate.

CE6.1.4

The provisions contained herein apply generally for the design of the longitudinal stiffeners on all longitudinally stiffened plates.

Early yielding of lower strength stiffeners would result in a significant reduction in their effectiveness; therefore, the specified minimum yield strength of the stiffeners should not be less than the specified minimum yield strength of the plate to which they are attached. Otherwise, the strength of the stiffened plate may be calculated by taking F_{ysp} equal to the specified minimum yield strength of the stiffeners in Articles E6.1.1 and E6.1.3. T-sections may not be available in higher grades of steel. In these cases, a T-section can be fabricated from plates or bars cut from plate.

Longitudinal stiffeners must be structurally continuous along their length to develop the resistance of the corresponding stiffened plates. Longitudinal stiffeners should either be continuous through any intermediate internal diaphragms or transverse stiffeners, or discontinued and positively attached to each side of the diaphragms or transverse stiffeners such that they act as continuous elements. Cutouts may be used in diaphragms or transverse stiffeners to accommodate continuous longitudinal stiffeners. Where cutouts are used, the longitudinal stiffeners should be attached to the internal diaphragms or transverse stiffeners. T-section longitudinal stiffeners may be conveniently attached to the diaphragms or transverse stiffeners by welds or bolts with a pair of clip angles. A welded tab plate may also be used to make the attachment. Similar attachment of the longitudinal stiffeners should also be considered at end diaphragms.

Should it be necessary to discontinue a longitudinal stiffener at a bolted field splice, consideration should be given to extending the stiffener to the free edge of the plate element, where the normal stress is zero. If the plate element on the other side of the splice is longitudinally unstiffened, its resistance should be checked accordingly to determine if the flange is satisfactory without a stiffener or if a slight increase in the flange thickness will suffice without providing a stiffener. Where necessary to extend the longitudinal stiffener beyond a field splice, splicing the stiffener across the field splice is recommended. The continuity of the longitudinal stiffener and the integrity of the stiffened plate must be maintained across the splice.

If the stiffener is terminated outside the splice and the termination is subject to a net tensile stress, determined as specified in Article 6.6.1.2.1, the termination of the stiffener weld to the plate must be checked for fatigue according to the terminus detail. Terminating the longitudinal stiffener by positively attaching it to a

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The cross-section elements of longitudinal stiffeners should satisfy:

$$\frac{b}{t} \leq \lambda_r \tag{E6.1.4-1}$$

where:

λ_r = corresponding width-to-thickness ratio limit for the longitudinal stiffener plate element under consideration as specified in Table 6.9.4.2.1-1

b = longitudinal stiffener plate element width as specified in Table 6.9.4.2.1-1 (in.)

t = longitudinal stiffener plate element thickness (in.)

In addition, tee and angle section longitudinal stiffeners should satisfy:

$$\frac{J_s}{I_{ps}} \geq 5.0\frac{F_y}{E} \tag{E6.1.4-2}$$

where:

F_y = specified minimum yield strength of the longitudinally stiffened plate element under consideration (ksi)

J_s = St. Venant torsional constant of the longitudinal stiffener alone, not including the contribution from the stiffened plate (in.^{4})

I_{ps} = polar moment of inertia of the longitudinal stiffener alone about the attached edge (in.^{4})

transverse stiffener is also another possible alternative, which may lead to a larger fatigue resistance.

Eq. E6.1.4-1 ensures that the resistance of longitudinal stiffeners will not be impacted by local buckling of the stiffener cross-section elements. Eq. E6.1.4-2 ensures against torsional buckling, or tripping, of tee and angle section stiffeners about the edge of the stiffener attached to the plate. For flat plate longitudinal stiffeners, these two equations give the same requirement; therefore, only Eq. E6.1.4-1 needs to be checked. Eq. E6.1.4-2 neglects the warping contribution to the torsional resistance, which tends to be small in many practical cases considering the length between the locations of torsional restraint at transverse stiffeners and/or diaphragms.

Satisfaction of Eq. E6.1.4-1 is considered most appropriate for new designs. In cases where Eq. E6.1.4-1 is violated in any cross-section component of longitudinal stiffeners in existing structures, it is recommended that the resistance of the longitudinally stiffened plate specified in Article E6.1.3 should be calculated by using an effective width equal to $\lambda_r t$ within the corresponding longitudinal stiffener cross-section components. For plates of the longitudinal stiffener cross-section supported only on one longitudinal edge, such as flat plate stiffeners and flanges of T-section stiffeners, the effective width $\lambda_r t$ is placed adjacent to that edge. For plates of the longitudinal stiffener cross section supported on two longitudinal edges, such as the stem of T-section stiffeners, half of the above effective width is placed adjacent to each edge. This approach gives an appropriate estimate of the influence of slender cross-section elements in longitudinal stiffeners. The use of the traditional approach of substituting a factored longitudinal stress in the stiffener, f_a, due to axial compression and flexure is not recommended. The true stresses in the longitudinal stiffener can easily approach F_y in certain portions of the stiffener length, due to the axial compression plus second-order bending in the stiffener, including the influence of stiffener initial out-of-straightness. The wavelength associated with local buckling of the stiffener cross-section components is typically relatively short, and this can lead to a local failure along the stiffener length that is not captured by simply substituting f_a for F_y within Eq. E6.1.4-1.

Eq. E6.1.4-2 tends to require relatively thick plates for tee and angle stiffeners to ensure that the stiffener does not fail in a mode involving torsional buckling about the connection to the stiffened plate, commonly referred to as tripping. This equation is considered as being most appropriate for new designs. Eq. E6.1.4-2 neglects the potential contribution from the stiffened plate to twisting of the stiffener about its connection to the plate. Quantification of this torsional restraint is a complex problem that has not yet been fully studied (White et al., 2019b).

As discussed above, traditional approaches to conservatively estimate plate local buckling resistances

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have sometimes replaced F_y in equations for the nonslender plate limit, such as in Eq. E6.1.4-1, by the factored longitudinal compressive stress in the component due to axial force plus bending within the structural member, f_a. The tripping limit state for an open section longitudinal stiffener is a different problem than local buckling of a flat plate. As a coarse relaxed estimate of the requirement in Eq. E6.1.4-2, to avoid tripping, and until further research can be conducted to better ascertain an improved estimate of the tripping resistance, it is recommended that F_y in this equation may be replaced by the average between the stiffener specified minimum yield strength, F_y, and the maximum factored longitudinal stress in the stiffener, f_a. The use of the factored longitudinal stress in this way as a modified stiffener tripping check is considered sufficient because of the long buckling length associated with the tripping limit state. White et al. (2019b) discuss a number of potential alternative methods for verifying the torsional stability of open-section longitudinal stiffeners.

Longitudinal stiffeners on flanges with $b_{sp}/t_{sp} > 90$ generally shall satisfy:

$$a/r_s \leq 120 \tag{E6.1.4-3}$$

in which:

r_s = radius of gyration of the stiffener strut about an axis parallel to the plane of the stiffened plate (in.)

$$= \sqrt{I_s/A_{gs}} \tag{E6.1.4-4}$$

where:

a = longitudinal spacing between locations of transverse stiffeners or diaphragms that provide transverse lateral restraint to the longitudinally stiffened plate under consideration (in.)
b_{sp} = total width of the longitudinally stiffened plate, taken as the inside distance between the plates providing lateral restraint to its longitudinal edges (in.)
A_{gs} = gross area of an individual stiffener strut as defined in Article E6.1.3 (in.^{2})
I_s = moment of inertia of an individual stiffener strut as defined in Article E6.1.3 (in.^{4})
t_{sp} = thickness of the longitudinally stiffened plate under consideration (in.)

The limit $a/r_s \leq 120$ on longitudinally stiffened flange plates ensures against excessive out-of-plane deflection of a stiffened plate under its self-weight plus a small transverse concentrated load (White et al., 2019a). This limit is applied regardless of whether or not the member is in a horizontal configuration in the final constructed geometry, to limit such deflections with the member oriented horizontally during construction operations. This limit need not be satisfied for flanges with $b_{sp}/t_{sp} \leq 90$ since the out-of-plane deformations due to the above load effects tend to be small in these cases without consideration of the longitudinal stiffening.

E6.1.5—Transverse Stiffeners

E6.1.5.1—General

Transverse stiffeners provided to enhance the resistance of a longitudinally stiffened plate should

CE6.1.5.1

The provisions contained herein apply for the design of transverse stiffeners that are provided to enhance the consist of a flat rectangular plate or a T-section continuously welded to one side of the stiffened plate, or a top or bottom strut of an internal cross-frame or a wide-flange section placed across the outstanding end of the longitudinal stiffeners.

With the exception of longitudinally stiffened plates containing transverse stiffeners in which:

- the characteristic length, ℓ_c, from Eq. E6.1.3-8 is less than the spacing, a, defined in Article E6.1.3, and;
- the transverse stiffeners are not subjected to any directly applied bending or axial compression,

transverse stiffeners used to increase the compressive resistance of a longitudinally stiffened plate shall satisfy the moment of inertia requirements specified in Article E6.1.5.2.

Transverse stiffeners generally should have a moment of inertia, I_t, defined in Article E6.1.5.2 greater than or equal to the moment of inertia of the longitudinal stiffeners, I_s, defined in Article E6.1.3.

The cross-section elements of transverse stiffeners also should satisfy the requirements of Eqs. E6.1.4-1 and E6.1.4-2.

resistance of a longitudinally stiffened plate subjected to a net axial compression within the plate, with or without flexure within the plane of the plate.

For longitudinally stiffened plates containing transverse stiffeners that satisfy both situations specified herein, the transverse stiffeners do not serve any purpose to enhance the axial compressive resistance of the longitudinally stiffened plate, and, any potential destabilization of the transverse stiffeners from the longitudinal compression in the plate is not a consideration. Therefore, in these cases, the moment of inertia requirements of Article E6.1.5.2 may be waived.

The provisions contained herein do not apply for cases where the transverse stiffener is subjected to a concentrically applied axial compressive force, and/or directly applied loads causing bending of the stiffener, whether or not the characteristic length, ℓ_c, is less than the spacing, a. White et al. (2019b) discuss example cases where this may potentially occur, including top or bottom struts of internal cross-frames that serve as transverse stiffeners to enhance the resistance of the longitudinally stiffened plate, and provide an alternative moment of inertia requirement to handle such cases.

The minimum requirement that the transverse stiffener moment of inertia, I_t, be greater than or equal to the moment of inertia, I_s, of the longitudinal stiffeners, combined with the requirement from Eq. E6.1.4-3, helps ensure against excessive out-of-plane deflection of a stiffened plate under its self-weight plus a small transverse concentrated load in cases where the calculated axial compression in the plate is small, and therefore the requirements from Eqs. E6.1.5.2-1 and E6.1.5.2-2 are small (White et al., 2019a). One example of this situation is a case where transverse stiffeners are installed to serve only as points of termination of longitudinal stiffeners in a tension zone. In designs where b_{sp}/a_{max} is close to or greater than 1.0, where b_{sp} is the total width of the stiffened plate as defined in Article E6.1.4 and a_{max} is the largest of the longitudinal spacings to the adjacent transverse stiffeners or diaphragms providing lateral restraint to the plate, I_t may need to be larger than I_s to satisfy out-of-plane deflection criteria under service loads (White et al., 2019a).

Transverse stiffeners should also satisfy Eqs. E6.1.4-1 and E6.1.4-2 to avoid potential local buckling of the elements of the stiffener, as well as torsional buckling, or tripping, of T-section stiffeners about the edge of the stiffener attached to the plate. These requirements are easily satisfied by flat plate transverse stiffeners, and as noted in Article CE6.1.4, Eqs. E6.1.4-1 and E6.1.4-2 are equivalent for these stiffener types. However, Eq. E6.1.4-2 can be prohibitive for larger transverse stiffeners containing flange elements, such as T-sections, in large box-section members. In these cases, the transverse stiffeners should be designed less conservatively by

Longitudinal stiffeners shall be structurally continuous at transverse stiffeners. Transverse stiffeners also shall be structurally continuous and attached at their ends to the plates providing lateral restraint to the edge of the longitudinally stiffened plate under consideration.

considering them explicitly as beam-column or beam members subjected to the axial force and/or bending moment induced in the stiffener, as discussed further in White et al. (2019b).

Longitudinal stiffeners must be structurally continuous at any transverse stiffeners. Cutouts may be placed in the transverse stiffeners to accommodate the continuous longitudinal stiffeners. Alternatively, a top or bottom strut of an internal cross-frame, or a wide-flange section, satisfying the requirements specified herein may serve as a transverse stiffener and be placed across the outstanding end of the longitudinal stiffeners. In either case, the longitudinal stiffeners should be attached to the transverse stiffeners. T-section longitudinal stiffeners may be conveniently attached to the transverse stiffeners by welds or bolts with a pair of clip angles. A welded tab plate may also be used to attach the stiffeners.

Alternatively, transverse stiffeners that are welded to the stiffened plate may be discontinued and welded to the continuous longitudinal stiffeners such that they act as continuous elements across the longitudinal stiffeners as shown in Table 6.6.1.2.4-1.

In all cases, the attachment of a longitudinal stiffener to the transverse stiffener should be designed for the following force perpendicular to the plane of the stiffened plate:

$$V_{u\ell} = 0.01P_{ups} + V_{u\ell 1} \qquad \text{(CE6.1.5.1-1)}$$

where:

P_{ups} = factored axial force within the longitudinal stiffener strut under consideration, composed of the longitudinal stiffener and the tributary plate width, determined from a structural analysis considering the gross cross section (kip)

$V_{u\ell 1}$ = first-order force transferred by the attachment due to any directly applied factored loads on the longitudinal stiffener perpendicular to the plane of the stiffened plate, as applicable (kip)

The attachment at the ends of a transverse stiffener should be designed to transmit the following force perpendicular to the plane of the stiffened plate:

$$V_{ut} = 0.005P_{up}\left(\frac{b_{sp}}{a}\right) + 0.01P_{ut} + V_{ut1} \qquad \text{(CE6.1.5.1-2)}$$

where a, b_{sp}, and P_{up} are defined in Article E6.1.5.2, P_{ut} is the direct factored axial compression force in the transverse stiffener, as applicable, and:

V_{ut1} = first-order force in the direction perpendicular to the plane of the stiffened plate at the attachment due to any directly applied factored loads, as applicable (kip)

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E6.1.5.2—Moment of Inertia

Transverse stiffeners that are:

- used to strengthen a longitudinally stiffened plate, and
- are not subjected to a concentrically applied axial compressive force and/or directly applied loads causing bending of the stiffener

shall satisfy the following moment of inertia requirements:

$$I_t \geq 0.05 \frac{P_{up}}{a_{\min}} \frac{b_{sp}^3}{E} \quad \text{(E6.1.5.2-1)}$$

and

$$I_t \geq \left(0.0009 \frac{E}{F_y} \frac{c}{b_{sp}} + 0.02\right) \frac{P_{up}}{a_{\min}} \frac{b_{sp}^3}{E} \quad \text{(E6.1.5.2-2)}$$

where:

a_{min} = smallest of the longitudinal spacings to the adjacent transverse stiffeners or diaphragms providing lateral restraint to the plate (in.)

c = largest distance from the neutral axis to the extreme fiber of the transverse stiffener considered in the calculation of I_t (in.)

b_{sp} = total inside width between the plate elements providing lateral restraint to the longitudinal edges of the plate under consideration (in.)

F_y = smallest specified minimum yield strength of the stiffened plate and the transverse stiffener under consideration (ksi)

I_t = moment of inertia of the transverse stiffener, including a width of the stiffened plate equal to $9t_{sp}$, but not more than the actual dimension available, on each side of the stiffener avoiding any overlap with contributing parts to adjacent stiffeners or diaphragms, taken about the centroidal axis of the combined section. The reduced cross section at cutouts to accommodate longitudinal stiffeners shall be considered; the smallest moment of inertia at such cutouts shall be used for I_t (in.^{4})

P_{up} = total factored longitudinal compression force in the plate under consideration, determined from a structural analysis considering the gross cross section, including the longitudinal stiffeners, and including all sources of factored longitudinal normal compressive stresses from axial loading and from flexure; in cases where the plate is subjected to longitudinal normal stresses in tension over a portion of its width, the tensile stresses shall be neglected in determining this force (kip)

t_{sp} = thickness of the stiffened plate (in.)

CE6.1.5.2

Eq. E6.1.5.2-1 is a generalized and simplified form of the moment of inertia requirement given by Eq. C6.11.11.2-4 for transverse stiffeners in composite box-section compression flanges. This equation ensures adequate lateral stiffness to resist the destabilizing load effects from the factored longitudinal compression force in the stiffened plate, including the effects from the longitudinal stiffeners. In many situations, the factored longitudinal compression force in the stiffened plate, quantified by the term P_{up}, provides the only significant demand on the transverse stiffeners.

Eq. E6.1.5.2-2 is an indirect moment of inertia requirement necessary to ensure that the transverse stiffeners have adequate lateral strength to resist the destabilizing load effects from the factored longitudinal compression force in the plate, including the effects from the longitudinal stiffeners. Alternatively, Eq. E6.1.5.2-2 may be expressed as the following general requirement on c, given a provided moment of inertia, I_t, to avoid early yielding of the stiffener:

$$c \leq 1{,}200 \frac{F_y}{E} \left(\frac{I_t}{\dfrac{P_{up}}{a_{\min}} \dfrac{b_{sp}^3}{E}} - \frac{2}{\pi^4} \right) b_{sp} \quad \text{(CE6.1.5.2-1)}$$

The moment of inertia of the transverse stiffener is commonly calculated assuming a width of the stiffened plate equal to $9t_{sp}$ on each side of the stiffener as a flange contribution from the plate, as stated in the definition for I_t. However, a width of $b_{sp}/10$, or a total width of $b_{sp}/5$, may be more appropriate and may be used. This width is based on the consideration of shear lag for the plate acting as a flange of the transverse stiffener. The width $9t_{sp}$ is recommended within these provisions as a conservative value for one-half of this flange width, recognizing that there may be some reduction in the effectiveness of the plate acting as a flange of the transverse stiffener due to high normal or shear stresses from other actions in the plate. Either of the above values for the width of the plate contributing to the moment of inertia of the transverse stiffener is limited by the actual dimension available on each side of the stiffener, avoiding any overlap with contributing parts to adjacent stiffeners or diaphragms.

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