SECTION 7
STEEL STRUCTURES

TABLE OF CONTENTS

SECTION 7
STEEL STRUCTURES

7.1 MATERIALS AND FABRICATION

7.1.1 Structural Steel Designations

AASHTO M270, Grade 50 shall generally be used for all structural steel. If the structure is to remain uncoated and allowed to weather, AASHTO M270, Grade 50W should be used.

The structural steel designations shall be shown on the plans. The designations shall reference AASHTO material specifications and include the applicable suffix codes. The suffix “T” indicates a Non-Fracture Critical material whereas an “F” indicates a Fracture Critical material. The “T” or “F” is followed by the appropriate AASHTO temperature zone for Connecticut, which is “2.”

Examples:

Non-Fracture Critical Bridge Members	AASHTO M270 Grade 50 T2 AASHTO M270 Grade 50 WT2
Fracture Critical Bridge Members	AASHTO M270 Grade 50 F2 AASHTO M270 Grade 50 WF2

7.1.2 Coated and Uncoated Structural Steel *(Rev. 04/19)*

New structural steel bridges may be either coated or uncoated.

- Uncoated steel shall be weathering steel.
- Coated steel shall be either shop galvanized or metallized and top coated.

In order to reduce future maintenance, the use of coated steel should be minimized. Uncoated weathering steel should be the first choice for structural steel bridges with life-cycle cost as a consideration. The use of galvanizing or metallizing and top coating should also be considered where the look of weathering steel is objectionable.

Weathering steel should be the first consideration for most bridges, especially those in rural areas. The use of weathering steel in urban areas or where the bridge will be highly visible shall be discussed with the Municipal Officials prior to its use. Weathering steel shall be designated for all structural steel bridges over railroads.

Where the use of weathering steel is not appropriate, such as bridges subject to vehicular salt spray, near a salt water environment, or a heavy industrial area, the use of galvanized steel should be considered. Where the length of the structural steel members precludes use of galvanized steel, shop metallizing should be used. Shop metallizing shall include a colored urethane top coat.

A paint only system shall only be used for existing structural steel bridges.

7.1.2.1 Uncoated Weathering Steel

Where weathering steel has been found to be appropriate in accordance with **CTDOT** guidelines, its use should conform to the FHWA Technical Advisory T5140.22, "Uncoated Weathering Steel in Structures," dated Oct. 3, 1989, and amended as follows:

a. The design of weathering steel for bridges subject to vehicular salt spray, near a salt water environment, or a heavy industrial area should incorporate modest increases in flange plate thicknesses to allow for some minor section loss in the future.

b. The interior surfaces of box girders, including all structural steel components within the box girders (such as diaphragms, cross-frames, connection plates, etc.) shall be painted in accordance with the special provision, entitled "Structural Steel (Site No.)." The intermediate coat shall be white (Federal Standard 595 Color No. 27925) in order to facilitate bridge inspection.

c. Whenever possible, unpainted weathering steel bridges must be designed to eliminate deck joints. If deck joints cannot be eliminated, the areas adjacent to the joints shall be protected from leakage. Generally, the ends of the beams directly under joints can be metallized or painted for protection. For bridge decks that extend past the backwall and integral abutments, beam ends need not be painted. The topcoat shall be Brown, Federal Standard 595 Color No. 20062. The steel should be metallized or painted for a distance approximately equal to one and one half times the depth of the girder on either side of the joint. All structural steel components within this distance (such as diaphragms, cross-frames, connection plates, stiffeners, etc.) shall also be painted.

The limits of the structural steel requiring painting shall be delineated on the plans.

d. Proper precautions should also be taken to minimize substructure staining for construction conditions and the service life of the bridge. In general, this will include providing catchments and diversion bars at all bearings and ensuring that the Contractor adequately protects the substructure during construction.

e. Provisions should also be included to control vegetation growth under the structure to reduce the moisture in the air that could have a detrimental effect on the structure.

7.1.2.2 Coated Structural Steel *(Rev. 04/19)*

In general, coated structural steel bridges shall be galvanized or metallized and top coated. For existing bridges, when required, structural steel shall be prepared and coated in accordance with the special provision, entitled "Structural Steel (Site No.)."

With the exception of major structures or architecturally or historically significant structures, the choice of color for shop and field painting the top coat of steel, shall be limited to the following:

a. Green - Federal Standard 595, Color No. 24172
b. Green - Federal Standard 595, Color No. 24277
c. Blue - Federal Standard 595, Color No. 26329

Blue shall be used for bridges that span over waterways. Green shall be used for bridges that span over land or roadways.

The use of galvanized steel or metallizing should be considered in order to reduce future maintenance obligations.

7.1.3 Fasteners *(Rev. 04/19)*

Fasteners shall be high-strength bolts conforming to the requirements of ASTM F3125 Grade A325 or F3125 Grade A490.

On coated structures, the high-strength bolts shall conform to ASTM F3125 Grade A325, Type 1 and be hot-dipped galvanized in accordance with ASTM F2329 or mechanically galvanized in accordance with ASTM B695, Class 55. On uncoated, weathering steel structures, the high-strength bolts shall conform to ASTM F3125 Grade A325, Type 3 or ASTM F3125 Grade A490, Type 3, although ASTM F3125 Grade A325 is preferred.

The high-strength bolt, nut and washer designations shall be shown on the plans. These designations shall reference ASTM *Specifications*, and include types and grades where applicable.

Generally, standard-sized holes shall be specified in the component parts of bolted connections. Where design considerations permit, however, connections should be designed to accommodate oversized holes to allow for potential enlargement of holes in the field where necessary to facilitate field erection.

7.1.4 Welding

Welding of fracture critical and non-fracture critical structural steel members or components for highway bridges shall conform to the **AWS D1.5**.

Welding of structural steel members or components, such as sign supports and inspection platforms, shall conform to the **AWS D1.1**.

Welding symbols shall conform to the latest edition of AWS A2.4 - Standard Symbols for Welding, Brazing and Nondestructive Examination.

Fillet weld sizes shall be shown on the plans and shall conform to the sizes shown in **BDM** [Division 3]. Generally, the minimum size fillet weld shall be 5/16 inch. Smaller welds may be required for thin plates. Connections made with fillet welds placed on opposite sides of a common plane of contact shall not be detailed with the weld-all- around symbol. Per the **AWS D1.5**, "fillet welds deposited on opposite sides of a common plane of contact between two parts shall be interrupted at a corner common to both welds."

Weld symbols for complete penetration groove welds shall be specified, without dimensions, by three capital letters, CJP. This allows the weld joint configuration and details to be determined by the fabricator.

Non-destructive testing (NDT) of welds shall be specified with symbols, combined with the welding symbols, for the welds requiring testing. The quantities of non-destructive testing methods required for field welds shall be shown in the "Inspection of Field Welds" block on the General Plan.

Multiple pass welds, inspected by the magnetic particle method, shall have each pass or layer inspected and accepted before proceeding to the next pass or layer.

The welding specifications shall be shown on the plans.

7.1.5 Fabrication

7.1.5.1 General Requirements *(Rev. 04/19)*

Fabrication of structural steel members or components for highway bridges shall conform to the **LRFD** [6].

The structural steel fabricator's plant shall be certified by the AISC Quality Certification Program. The certification requirements depend on the category of structure being fabricated as follows:

For non-fracture critical members:

1. **<u>Bridge Fabricator Simple (SBR) or Bridge Component (CPT)</u>.**
   Typical work includes:
   1. *Bridge cross frames for straight bridges with skew angles less than 30 degrees*
   2. *Highway sign structures*
   3. *Bridge inspection catwalks*
   4. *Grid decks*
   5. *Scuppers*
   6. *Expansion joints*
   7. *Bearings*

2. **<u>Bridge Fabricator Simple (SBR)</u>.**
   Typical work includes:

1. *Straight simple un-spliced rolled beams*

3. **Bridge Fabricator Intermediate (IBR).**
   Typical work includes:
   1. *Rolled beam with field or shop splices, straight or with radius over 500 feet*
   2. *Built up I-shaped plate girder with constant depth except for dapped ends, with or without splices, either straight or with radius over 500 feet*
   3. *Built up I-shaped plate girder with variable depth, either straight or with a radius over 1000 feet*
   4. *Truss with a length 200 feet or less that is entirely pre-assembled at the verified facility and shipped in no more than three sub-assemblies*

4. **Bridge Fabricator Advanced (ABR).**
   Typical work includes:
   1. *Tub or trapezoidal box girders, closed box girder bridges*
   2. *Curved girders with radius under 500 feet*
   3. *Large or non-preassembled trusses, arches*
   4. *Moveable bridges*
   5. *Cable stayed bridges*

If the structure has fracture critical members or components, the fabricator's plant shall also be certified to produce fracture critical members in accordance with a fracture control plan as defined by the **AWS D1.5**. A fabricator with this endorsement will have a suffix "F" added to the above categories (Category IBR,F or Category ABR,F).

The certification requirements for specific components shall be shown on the plans.

7.1.5.2 Special Fabrication Requirements for Box Girders

Vacant

7.2 GENERAL DESIGN REQUIREMENTS

7.2.1 Structure Types

7.2.1.1 Cost Effective Span Lengths

The following are appropriate ranges of cost effective span lengths for various steel bridges types:

TYPE OF BRIDGE	COST EFFECTIVE SPAN LENGTH (ft)
Rolled Beams	50 to 90
Plate Girders	80 to 250
Box Girders	150 to 250

The span lengths shown are for simple span bridges. For continuous bridges, these span lengths can be assumed to be measured from dead load inflection points.

For spans over 250 feet, special design studies must be done. Plate and box girders may still be the structure of choice since they provide redundancy. Other options are arches, trusses or cable stayed bridges, although these structure types should be limited to very long spans.

7.2.1.2 Non-Redundant Systems

Non-redundant systems such as girder and floor beam bridges should be avoided even though they may have an initial lower cost. The reason for this is the lack of redundancy, fatigue problems, and difficulties involved with future widening associated with these types of structures.

The only situations where non-redundant bridges should be considered are in the case of through-girder or through-truss spans where the minimum depth of the superstructure is critical.

7.2.1.3 Box Girders

Generally, box girders should be considered only for very long spans. They should also be investigated for use on curved roadways where torsional rigidity is required. Box girder cross sections shall be a trapezoidal shape with webs sloped equally out from the bottom flange. The webs shall be the same depth. The minimum web depth shall be 78 inches to allow for inspection and maintenance inside the box girders. In general, box girders shall be rotated so that the top and bottom flanges are parallel with the deck cross slope.

7.2.1.4 Short Spans

Rolled beam and girder type bridges may also be used for shorter spans at locations where utilities must be supported between stringer lines.

7.2.2 Span Layout

7.2.2.1 Member Spacing

Member spacing should be maximized in order to reduce the number of members required thereby reducing the costs for fabrication, shipping, erection and future maintenance. However, in order to provide redundancy, a minimum of four stringer lines should be used in a bridge cross section.

Generally, the most economical spacing for rolled beams is between 8 feet and 9.5 feet. It is recommended that the minimum spacing for I-shaped plate girders and top flanges of box girders be kept to 9 feet.

7.2.2.2 Deck Overhang

The concrete deck overhangs, measured from the centerline of the fascia member, should be limited to 4 feet or to the depth of the member, whichever is less. For deck overhangs greater than 4 feet, the designer shall include requirements in the contract documents for special forming requirements needed to prevent torsional rotation of the fascia member during concrete placement. This rotation is caused by the effect of the typical forming brackets used in construction.

7.2.2.3 Framing Geometry

Members should be laid out parallel and uniformly spaced as much as practical. If this is unavoidable, the live load distribution factors, as outlined in the **LRFD,** shall not be used. The designer should carefully investigate these situations to account for the variation in live load and member stiffness.

7.2.3 Continuity

Continuous spans shall be used for all multiple span bridges. Provisions for thermal movement of the bridge shall generally be made at or behind the abutments. For bridges on a grade, provisions for thermal movement of the bridge shall generally be made at the high end of the bridge.

7.2.4 Fatigue

7.2.4.1 General Requirements

The provisions in the **LRFD** shall be followed for the design of bridges. Fatigue requirements shall apply to elements of steel members where the summation of the calculated maximum live load tension stress and the dead load stresses results in net tension.

7.2.4.2 Rolled Beams

7.2.4.2.1 Simple Spans with Cover Plates

For simple span rolled beams with cover plates, the cover plates shall be extended approximately full length. The cover plates shall be fillet welded across the ends.

7.2.4.2.2 Continuous Spans with Cover Plates

For continuous span rolled beams with cover plates, the cover plates shall be terminated with end welds in non-fatigue regions. For the bottom flange, the regions are near the interior supports and, for the top flange, the regions are near the middle of the spans.

7.2.4.3 Diaphragm Connection Plates

For all types of steel bridges, the design of the flanges should be based on Category C in order to allow the welding of diaphragm connection plates to the flanges. If a preliminary design does not satisfy the requirements of Category C, then one of the following options should be followed:

a. The flange can be increased in size to reduce the live load stress range.

b. The location of flange splices can be changed to reduce the live load stress range.

c. The connection can be bolted to reduce it to a Category B detail.

Note: For option c, the weld of the connection plate to the web adjacent to the flange is also a Category C detail, which is subject to virtually the same stress range, and may also need to be bolted. For this reason, this option should be avoided.

7.2.5 Fracture Critical Bridge Members

7.2.5.1 Definitions

Fracture Control Plan (FCP) - The Fracture Control Plan is the materials testing and fabrication provisions for Fracture Critical Members as outlined in the **AWS D1.5**.

Fracture Critical Member (FCM) - Fracture Critical members or member components are tension members or tension components of bending members (including those subject to reversal of stress), the failure of which would be expected to result in collapse of the bridge. The designation "FCM" shall mean fracture critical member or member component. Members and components that are not subject to tensile stress under any condition of live load are not fracture critical.

Attachments - Any attachments welded to a tensile zone of a FCM member shall be considered a FCM when any dimension of the attachment exceeds 4 inches in the direction parallel to the calculated tensile stress in the FCM. Attachments shall meet all requirements of the Fracture Control Plan.

Welds - All welds to FCM's shall be considered fracture critical and shall conform to the requirements of the Fracture Control Plan. Welds in compression members or compression areas of bending members are not fracture critical.

7.2.5.2 General Provisions

All fracture critical members shall be identified on the plans. Each FCM shall be individually designated on the plans by three capital letters, FCM, enclosed in a diamond. Each portion of a bending member that is fracture critical including welds shall be clearly described giving the limits of the FCM.

Based on the definitions above, the following guidelines shall be followed for designation of FCM's on plans:

I-Shaped Girder Bridges - For longitudinal girder bridges, FCM components of the beams shall be considered fracture critical if there are three or less girders in the bridge cross section. This requirement does not apply to temporary stages in construction.

Box Girder Bridges - For longitudinal box girder bridges, FCM components of the beams shall be considered fracture critical if there are two or less box girders in the bridge cross section. For the case of a two-box girder cross section, the top flanges and the welds of the webs to the top flanges shall not be considered fracture critical. This requirement does not apply to temporary stages in construction.

7.2.6 Diaphragms and Cross-Sections

7.2.6.1 General Provisions

Intermediate and end bearing diaphragms and cross frames (cross members) shall be provided for rolled beams, plate girders and box girders. They shall be designed and located, unless otherwise noted, in accordance with the **LRFD**.

Intermediate cross members for rolled beams and plate girders shall preferably be placed at the 0.4 point of end spans of continuous bridges and at the center of interior spans. If practical, they should also be placed adjacent to a field splice. Cross members shall be spaced as far apart as possible to limit the overall number but still satisfying the AASHTO criteria. The need for cross members shall be investigated for all stages of construction.

7.2.6.2 Skewed Bridges

On bridges skewed less than 20 degrees, the intermediate diaphragms shall be placed in line along the skew. On bridges skewed more than 20 degrees, intermediate cross members shall be placed normal to the main members and staggered, not placed in a line, across the width of the bridge.

7.2.6.3 I-Shaped Beams and Girders with Web Depth ≤ 4 feet

7.2.6.3.1 Intermediate Diaphragms

Channel members shall be typically used for end and intermediate diaphragms. For intermediate diaphragms, the channel size shall be dependent upon the main member's depth. See the following table:

Member DEPTH (in)*	CHANNEL SIZE
21 - 24	C 12 x 20.7
27 - 30	C 15 x 33.9
33 - 36	MC 18 x 42.7

* Member depth is normal beam depth for rolled beams, actual web depth for plate girders.

7.2.6.3.2 End Bearing Diaphragms

End bearing diaphragms are required at all abutments and at intermediate locations where slab continuity is broken. These diaphragms shall preferably be channel sections and should be designed as simple span members with vertical dead loads, and live loads plus impact applied. The preferred channel size shall be C 15 x 33.9 and MC 18 x 42.7. For bridges with severe skew angles or wide girder spacings, wide flange sections or K-frames may be used in lieu of channels.

End diaphragms typically shall be placed along the centerline of bearings and be set on a sloped line. A minimum clear distance of 12 inches shall be provided between end diaphragms and front face of backwall.

The bridge skew angle shall be considered in determining the length of the end diaphragm. Consideration shall be given to composite action in the design of all end diaphragms. For both non-composite and composite end diaphragms, shear connectors, 7/8 inch diameter, with a maximum spacing of 12 inches, shall be welded to the top flange of the end diaphragms.

7.2.6.4 I-Shaped Beams and Girders with Web Depths > 4 feet

7.2.6.4.1 Intermediate Diaphragms

Cross frames shall be used for intermediate diaphragms. Intermediate cross frames shall be designed to satisfy the lateral wind load stresses and slenderness ratio, KL/r, requirements outlined in the **LRFD**. When computing the AASHTO allowable compressive stress, a value of 0.75 shall be used for the effective length factor, K.

The most economical intermediate cross frame considered for use shall be the X- type. When additional bracing is required, K-type frames should also be considered. All members shall be fabricated from equal leg angles or WT sections. Cross frames shall be as deep as practical. The cross frame depth shall be constant to facilitate fabrication.

7.2.6.4.2 End Bearing Diaphragms

End bearing cross frames shall have a K-type configuration with a channel member typically used at the top. All other members shall be equal leg angles or WT sections.

The design of the top member shall follow that outlined for the end bearing diaphragms in **BDM** [7.6.3.2]. The size of the end diaphragm's bottom chord may be increased to provide for future jacking of the girder ends. For both non-composite and composite end bearing diaphragms, shear connectors, 7/8 inch diameter with a maximum spacing at 12 inches shall be welded to top flange of top chord member.

7.2.6.5 Box-Girders

7.2.6.5.1 Intermediate Diaphragms

Intermediate cross frames, not required for the completed bridge, may be required for construction purposes and shall be located and spaced as a matter of engineering judgment. They may be installed as temporary members or left- in-place as permanent members. Consideration shall be given to locate, at a minimum, intermediate cross frames at the lifting points of each shipping piece, on each side of a field splice, and at maximum positive moment sections. These cross frames shall be designed to satisfy the construction load stresses and slenderness ratio requirements. Typical cross frame configurations shall be the X and K types. All members shall be fabricated from the equal leg angles.

7.2.6.5.2 End Bearing Diaphragms

For the design of simple and continuous bridges of moderate length supported by two or more single cell boxes, **LRFD** requires internal diaphragms at each support to resist transverse rotation, displacement and distortion. Intermediate cross members for these types of bridges are not required. If plate diaphragms are used, they shall be connected to the webs and flanges of the section. Access holes shall be provided.

7.2.7 Lateral Bracing

7.2.7.1 Requirements for I-Shaped Members

7.2.7.1.1 Design Requirements

For I-Shaped members, the need to laterally brace the bottom flanges shall be investigated as per **LRFD**. Lateral bracing should be avoided whenever possible. Reducing the cross frame spacing or modifying flange plate dimensions shall be considered when attempting to eliminate the bracing.

Bracing members, if required, shall be designed to satisfy lateral wind load stresses and slenderness ratio, KL/ r, requirements. The allowable fatigue stress ranges shall not be exceeded at the connections. Warren type patterns with single members is recommended.

7.2.7.1.2 Detailing Requirements

Bracing members shall typically consist of equal leg angles or WT sections attached to the flange or web via gusset plates, clip angles or WT sections. Gusset plates shall be bent to accommodate the difference in elevation between girders. If it is not practical to make connections to the flange, then connections shall be made to the web. Flange connections shall not interfere with the web to flange welds.

The minimum thickness of gusset plates shall be 9/16 inches. The minimum size angle used as a connecting or bracing member shall be L 4 x 4 x 5/16. Angles with unequal legs should not be used.

The need to temporarily brace the compression flange for stability during erection shall be investigated. This can be accomplished with intermediate diaphragms.

7.2.7.2 Requirements for Box Girders

7.2.7.2.1 Design Requirements

For box girders, generally no external lateral bracing should be required between the box sections. To increase the torsional stiffness of an individual box section during fabrication, erection and placement of the slab, permanent, internal lateral bracing either full or partial length shall be placed at or near the plane of the top flanges.

Bracing members and their connections shall be similar to those for I-Shaped members. The bracing shall be designed to resist the shear flow across the top of the section, satisfying stress and slenderness ratio requirement. Warren type bracing without transverse members should be considered because of efficiency. X-bracing patterns should be avoided for economy.

Allowable fatigue stress ranges shall not be exceeded where the gusset plate attaches to the flange or web.

7.2.8 Stiffeners

7.2.8.1 Bearing Stiffeners

7.2.8.1.1 Design Requirements

Rolled Beams - Bearing stiffeners are generally not required, but shall be provided when the web shear stress at the reaction exceeds that permitted by AASHTO. The full beam depth times the web thickness shall be used to compute the shear stress. When bearing stiffeners are required, their design shall follow the provisions for plate girders.

Plate Girders - Bearing stiffeners shall be placed at all bearing locations and other locations supporting concentrated loads.

When computing the AASHTO allowable compressive stress for the stiffener plates, a value of 0.75 shall be used for the effective length factor, K.

The stiffener plate to web fillet welds shall be designed to satisfy the total reaction and the **AWS D1.5**. The weld size shall not be less than 5/16 inches. For skewed plates, the **AWS D1.5** design requirements for skewed joints should be considered when sizing the welds.

7.2.8.1.2 Detailing Requirements

To simplify fabrication, the stiffener plates on any one structure should have the same width and thickness. The minimum thickness of a stiffener plate shall be 9/16 inches.

Stiffener plates which act as connection plates shall be fillet welded to the top flange and milled to bear and fillet welded to the bottom flange. To avoid possible warping of the bottom flange, complete penetration groove welds should not be used to attach the plate to the flange. When the plates are welded to the tension flange at interior supports of continuous bridges, the allowable fatigue stress range shall not be exceeded (see **BDM** [7.2.4]).

When the bearing stiffeners consist of two pairs of plates, they shall be offset sufficiently to permit proper welding.

The stiffener plates shall be placed symmetrically over the bearings and be vertical after the application of full dead loads.

7.2.8.2 Intermediate Transverse Stiffeners

7.2.8.2.1 Design Requirements

The design of intermediate transverse stiffeners shall be according to the **LRFD**.

If intermediate stiffeners are used, they shall be designed for one side of the web only, for reasons of economy.

7.2.8.2.2 Detailing Requirements

To simplify fabrication, the stiffener plates on any one structure should have the same width and thickness. The minimum thickness of a stiffener plate shall be 9/16 inches.

Intermediate stiffeners not supporting concentrated loads shall be detailed with a tight fit against the compression flange and cut short at the tension flange. This will greatly reduce the amount of labor for the installation of the stiffener. For stiffeners that are also used as diaphragm connection plates, the requirements for diaphragm connection plates shall be followed.

The intermediate stiffeners shall be detailed on one side of the web. The use of intermediate stiffeners on the outside face of exterior girders is not acceptable.

For details, see **BDM** [Division 3].

7.2.8.3 Longitudinal Stiffeners

7.2.8.3.1 Design Requirements

The design of longitudinal stiffeners shall be according to the **LRFD**. Generally, the use of longitudinal stiffeners is discouraged. Longitudinal stiffeners are generally not economical for spans less than 300 feet. If longitudinal stiffeners are used, for reasons of economy, they shall be designed for one side of the web only. Butt splices in longitudinal stiffeners shall be made before attachment to the web, and tested by the ultrasonic method.

7.2.8.3.2 Detailing Requirements

Longitudinal stiffeners shall be welded to the web plates and cut back 3/4 inches when interrupted by connection plates and bolted splices. The longitudinal stiffener need not be made continuous across bolted splices.

7.2.9 Connections and Splices

7.2.9.1 General

Shop connections may be made by either bolting or welding. Generally, all field connections should be made with high strength bolts. The use of field welding is discouraged due to difficulties with achieving proper coatings in the field. Welded field splices are not allowed.

7.2.9.1.1 Design Requirements

All bolted connections shall be designed as slip critical connections in accordance with **AASHTO LRFD**. Connections on uncoated bridges and coated bridges shall be designed with Class B surface conditions. Connections on Metallized bridges shall be designed with Class B surface conditions and shall be unsealed only at the connection. Connections on galvanized bridges shall be designed with Class C surface conditions.

In general, connections shall be designed with 7/8 inch diameter ASTM F3125 Grade A325 high strength bolts.

7.2.9.1.2 Detailing Requirements

The bolt diameter, hole size, bolt spacing and edge distances shall be shown on the plans. The type of connection (slip critical) and the class of faying surfaces (Class B or C) shall be specified on the plans.

To facilitate steel erection, only one type and diameter of bolt should be specified on any one bridge. Splices should be designed as though oversized holes were to be used to allow reaming in the field to facilitate fit-up. However, standard sized holes shall be specified on the plans.

7.2.9.2 Bolted Splices

7.2.9.2.1 Detailing Requirements

The bolt diameter, hole size, bolt spacing and edge distances shall be shown on the plans. The minimum thickness of web and flange splice plate shall be 9/16 inches. Splice plates shall be detailed with a minimum edge distance of 2 inches. The maximum distance between the ends of the members being spliced shall be 1 inch.

Location of shop and field splices is dependent upon such factors as design criteria, available length of plates and members, transportation of members, erection and site limitations, etc. Refer to **BDM** [1.3] on the transportation of members for additional information and guidance.

7.2.9.3 Cross Member Connections

7.2.9.3.1 Design Requirements

The design of the connection of cross members shall be consistent with the design of the members being attached. The connections for the end bearing cross members shall be designed for the shear due to dead and live loads plus impact.

For intermediate diaphragm connections, the number of bolts should be kept to 4 on each side of the diaphragm. In all cases, the number of bolts should be kept to a minimum.

7.2.9.3.2 Detailing Requirements

In general, when detailing bolted connections, the size, number and general layout of the bolts should be shown. Bolt hole spacing and edge distances should be left to the fabricator.

Holes for end diaphragm connections shall be located parallel to the main member’s web. Standard sized holes shall be used in the cross members while oversized holes, unless otherwise noted, shall be used in the stiffener or connection plates. At one side of a cross member, standard sized holes field drilled through the stiffener or connection plate may be used as an alternate method for erection.

Long slotted holes in the stiffener or connection plates shall be considered for erection of intermediate cross members for girders adjacent to a stage construction line.

For bridges with skews more than 20 degrees, when the differential dead load deflection of adjacent girders at any intermediate cross member connection is 3/4 inches or more, long slotted holes shall be detailed in the stiffener or connection plates attached to the girder with the larger deflection. The following note should appear on the plans when long slotted holes are used:

Bolts in long slotted holes shall only be finger-tightened prior to pouring the deck slab and then fully-tightened immediately after completing the pour.

Gusset plates shall be made rectangular to simplify fabrication.

Shop welds shall be made on one side, as much as practical, to avoid having to turn over the cross member assemblies in the fabricating shop.

7.2.10 Composite Construction

7.2.10.1 Design Requirements

All structural members in contact with and supporting a concrete deck shall be designed for composite action.

In general, 7/8 inch diameter stud type shear connectors shall be used for composite construction. Spirals, angles or channel shear connectors are not permitted.

7.2.10.2 Detailing Requirements

The minimum height shear connector is 4 inches. The maximum height of unstacked shear connectors is 8 inches. Stacked shear connectors shall be used at the locations where the haunch depth exceeds 6 inches.

Shear connectors are typically welded to the members in the field. Field welding through a mist coat of up to 2 mils of zinc primer is permissible.

Only the diameter of the shear connectors shall be shown on the plans. Shear connector heights shall not be shown on the plans. The heights shall be determined after the erected members have been surveyed and the haunch depths calculated.

On flange splice plates, one row of shear connectors shall be placed along the centerline of the splice plates.

7.2.11 Dead Load Deflection and Cambers

7.2.11.1 Simple Span Bridges

Dead load deflection and camber diagrams are not required for simple span bridges. Dead load deflections and cambers shall be calculated at the mid-span of the structure for the following listed items for each member and tabulated on the plans:

1. Structural Steel Deflections. Deflections due to the weight of the beams or girders, including the diaphragms and bracing and calculated using the moment of inertia of the steel section.

2. Additional Dead Load Deflections. Deflections due to the uncured concrete slab and haunches, utilities, and any other loads supported by the steel section alone. These deflections shall be calculated using the moment of inertia of the steel section.

3. Composite Dead Load Deflections. Deflections due to the parapets, curbs, sidewalks, railings, bituminous concrete overlay and any other loads that are placed after the slab has cured. This deflection shall be calculated using the moment of inertia of the composite section with a modular ratio equal to 3 times that of the final section as outlined in the **LRFD**.

4. Total Dead Load Camber. Camber required to compensate for the summation of the structural steel, slab dead load and the composite dead load deflections listed above.

5. Vertical Curve Ordinate Camber. Camber required when the member falls within the limits of a summit vertical curve. When the member falls within the limits of a sag vertical curve, provisions for sag ordinates must be made within the concrete haunch and shall not be specified in the camber table.

6. Extra Camber. Extra camber shall be provided when the grade of the roadway is on a tangent grade or on a sag vertical curve and is computed as follows:

    - Extra Camber (inch) = L / 100, where: L = Span Length (feet)

When the roadway is on a crest vertical curve, the extra camber is to be specified only when it exceeds the vertical curve ordinate. In this case, the amount of extra camber to be tabulated shall be only that portion in excess of the vertical curve ordinate.

7. Total Camber. The Total Camber is equal to the summation of all calculated cambers and is that dimension to which the member is to be fabricated.

For a table for the dead load deflections and cambers, see **BDM** [Division 3].

7.2.11.2 Continuous Span Bridges

Dead load deflections and cambers shall be tabulated for the following listed items for each member and shown on the plans. The locations tabulated shall be the member bearing points and points at equal spaces along the member at approximately 10 feet on center:

- Structural Steel Deflections: Same as for simple span bridges.
- Additional Dead Load Deflections: Same as for simple span bridges.
- Composite Dead Load Deflections: Same as for simple span bridges except that composite section properties should be used for both positive and negative moment regions.
- Total Dead Load Camber: Same as for simple span bridges but measured to a reference line, which is a theoretical straight line in each span connecting the points located at the top of the web at the centerlines of bearing.
- Vertical Curve Ordinate Camber: Same as for simple span bridges.
- Extra Camber: Extra camber shall not be provided for continuous bridges.
- Total Camber: The Total Camber is equal to the summation of all calculated cambers and is that dimension to which the member is to be fabricated.

For a table for the dead load deflections and cambers, see **BDM** [Division 3]. A diagram for dead load deflection shall not be shown. A total camber diagram shall be shown on the plans. For details of a total camber diagram, see **BDM** [Division 3].

7.2.12 Bearings

For bearing requirements, see **BDM** [9].

7.2.13 Superstructure Jacking Requirements

7.2.13.1 Design Requirements

Provisions for jacking of the superstructure shall be provided at all locations that have bearings that will require future maintenance. These bearings include all types other than fixed bearings.

7.2.13.1.1 New Construction

At abutments, preference shall be given to widening of the bridge seat and providing auxiliary jacking stiffeners so that jacks may be placed in front of the bearing to jack under the beam. Provision for massive diaphragms, which restrict access to the ends of the beam and backwalls should be avoided.

At piers with continuous caps, preference shall be given to designing diaphragms for jacking forces and providing auxiliary pads on pier caps.

Other unusual situations (i.e., piers consisting of individual columns under each girder) will require special study and may require provisions for jacking from ground level.

7.2.13.1.2 Rehabilitation Projects

For superstructure replacements, jacking provisions shall be provided only if economically viable. Jacking requirements should not be allowed to justify major substructure modifications where the substructures are otherwise adequate.

Lift points shall be located adjacent to the bearings and may be on main or secondary members. Preferably, lift points shall be over the bridge seats of abutments and the tops of piers so that jacks may be founded on these components minimizing the need for extensive temporary structures.

The jacking lift points shall be designed for the total dead load and the live load plus impact. If there are more than 5 lines of girders, the jacking lift points shall be designed for 150% of these values in order to jack individual girders in the future.

Superstructure and substructure members and components shall be strengthened as required to support the jacking loads.

7.2.13.2 Detailing Requirements

Lift points shall be clearly identified on the plans. The dead and live loads required to jack the bearing shall also be shown on the plans. If there are more than five lines of girders, two sets of loads shall be shown. The loads shall be for simultaneous jacking of all girders, and for jacking of individual girders. Additional stiffeners or brackets, if required, shall be shown on the plans.

7.2.14 Inspection Hand Rails

When girders are 5 feet or more in depth, a safety hand bar shall be provided 42 inches above the bottom flange for inspection access on both sides of all girders except the outside face of fascia girders. The bar shall have a minimum diameter of 1 inch and shall be designed for a minimum point load of 270 pounds.

7.3 STRUCTURE TYPE SPECIFIC REQUIREMENTS

7.3.1 Rolled Beams

The use of rolled beams should be investigated for appropriate span lengths since the cost of fabrication is significantly lower than equivalent I-shaped plate girders.

If cover plates are used on rolled beams, the width of the cover plate shall be narrower than the flange. The minimum thickness of a cover plate shall be 9/16 inches. The ends of the cover plates shall be rectangular in shape with rounded corners. Tapered end cover plates are not permitted. The attachment of cover plates to rolled beams shall be made with fillet welds. Rolled beams with cover plates, if used, shall be designed for fatigue. See **BDM** [7.2.4].

All fillet welds connecting the cover plate to the beam shall be non-destructively tested by the magnetic particle method.

The plans shall clearly state that, if the cover plate is fabricated by butt welding two or more plates together, the butt welds shall be non-destructively tested by the ultrasonic tested prior to attaching the cover plate to the beam.

7.3.2 I-Shaped Plate Girders

7.3.2.1 Hybrid Girders

The design of hybrid I-shaped plate girders should be avoided.

7.3.2.2 Web Plates

The minimum thickness of web plates shall be 3/8 inches. Web plate depths shall be specified in 2 inch increments.

In general, for plate girders with web depths less than 50 inches, unstiffened webs are more economical. For web depths greater than 50 inches, the following alternates shall be investigated for the web design to determine which is the most cost effective:

a. Fully stiffened web with minimum web plate thickness.
b. Unstiffened web.
c. Partially stiffened web with only a few stiffeners near supports.

In order to determine which of these alternates is most cost effective, the 1 to 4 rule should be used. That is, if the web and flange material costs $1 per pound, then the connection plate material costs $4 per pound.

7.3.2.3 Flange Plates

The minimum thickness of flange plates shall be 3/4 inches to eliminate warping of the plates when they are welded to the web. The flange plate widths shall be specified in 2 inch increments.

To minimize potential stability problems during various phases of construction, the minimum width of flange plates shall be determined based on the maximum of the following:

a. the length of the unsupported field piece divided by 85, or
b. 12 inches.

Flange width transitions shall generally be avoided. Flange plate thickness should be varied instead. At flange plate transitions, the thickness of the thinner plate shall not be less than ½ the thickness of the thicker flange.

The number and spacing of flange plate thickness transitions should be based on the total cost of the finished girder. While numerous flange transitions will produce the lightest girder, the fabrication costs for the splices may result in a higher total cost. The designer should investigate eliminating flange transitions, especially where they are closely spaced. As a rule, the approximate weight of flange material that should be saved in order to justify the introduction of a flange transition is as follows:

M = 255 + 21A
M = Weight of steel, pounds
A = Cross sectional area of thinner flange plate, square inches

In order to eliminate shop welded butt splices, field splices should be located at flange plate transitions.

7.3.2.4 Shop Splices

Shop flange splices shall be located a minimum of 6 inches from web splices.

Both web and flange splices shall be located a minimum of 6 inches from stiffeners and connection plates.

This information on web and flange plate shop splices shall be shown on the plans.

7.3.3 Box Girders

7.3.3.1 Hybrid Girders

The design of hybrid box girders should be avoided.

7.3.3.2 Web Plates

The minimum thickness of web plates shall be 3/8 inches. Web plate depths shall be specified 6 inch increments.

7.3.3.3 Flange Plates

The minimum thickness of flange plates shall be 3/4 inches to eliminate warping of the plates when they are welded to the web. A maximum flange plate thickness of 3 inches shall be used for box girders.

The minimum is 2 inch increments.

Flange width transitions shall generally be avoided. Flange plate thickness may be varied instead. At flange plate transitions, the thickness of the thinner plate shall not be less than ½ the thickness of the thicker flange.

The number and spacing of flange plate thickness transitions should be based on the total cost of the finished girder. While numerous flange transitions will produce the lightest girder, the fabrication costs for the splices may result in a higher total cost. The designer should investigate eliminating flange transitions, especially where they are closely spaced. As a rule, the approximate weight of flange material that should be saved in order to justify the introduction of a flange transitions is as follows:

$M = 255 + 21A$
M = Weight of steel, pound
A = Cross sectional area of thinner flange plate, square inches

In order to eliminate a shop welded butt splice, field splices should be located at flange plate transitions.

7.3.3.4 Shop Splices

Shop flange splices shall be located a minimum of 6 inches from web splices.

Both web and flange splices shall be located a minimum of 6 inches from stiffeners and connection plates.

This information on web and flange plate shop splices shall be shown on the plans.

7.3.3.5 Bolted Field Splices

Where bolted field splices are called for, the splice shall be detailed to provide adequate clearance for bolting the connections at the acute corners between the top flange and the web for both bolts and splice plates.

7.3.3.6 Fabrication Requirements

A minimum distance of 1 inch shall be provided between the outside face of the web and the edge of the bottom flange as a holding shelf for the flux deposited by the welding machine. At web stiffeners, provide a 1/2 inch clearance above a line 60 degrees from the bottom flange to accommodate a traveling welding machine.

7.3.3.7 Access Manholes

Access manholes shall be provided in the end or bottom flange of box girders. These manholes shall be located and detailed such that bridge inspectors can gain access without the need for special equipment. The distance between the end diaphragm and the backwall should be increased to a minimum of 2 feet when access is provided in the end diaphragms of box girders. For access through the bottom flange, ladder supports shall be incorporated. The preferred location for access is through the ends of the boxes.

The manholes shall have rounded corners fitted with a hinged cover and provided with an appropriate locking system and all access doors shall open inward. When access is provided through the end diaphragms, the access door should be covered with a steel wire mesh to allow ventilation. If access manholes are provided through the bottom flange, the access doors should be designed to be lightweight. Access holes shall be provided through all solid diaphragms.

Stresses resulting from the introduction of access holes in steel members shall be investigated and kept within all allowable limits.

7.3.3.8 Stay-In-Place Forms

Box girders shall be designed for the additional weight of remain-in-place forms placed within the boxes to form the deck slab.

7.3.3.9 Drainage

In order to provide drainage of the inside of the box girder, 2 inch minimum diameter drains shall be provided at the low end of the girder. The corners of all plates should be clipped so as not to trap moisture inside the girder. Bridge deck drainage may extend vertically through the girder but shall not be carried longitudinally within it.

7.3.3.10 Utilities

Gas, water and sewer lines are prohibited from being located within box girders. Electric, telephone and cable companies should be discouraged from locating their lines within the boxes. All utilities can generally be accommodated outside of and between the girders.

7.3.4 Curved Girders (Includes rolled beams, plate girders and box girders)

When designing curved girder structures, designers must investigate all temporary and permanent loading conditions, including loading from wet concrete in the deck pour, for all stages of construction. Future re-decking must also be considered as a separate loading condition. Diaphragms must be designed as full load carrying members. A three-dimensional analysis representing the structure as a whole and as it will exist during all intermediate stages and under all construction loading conditions is essential to accurately predict stresses and deflections in all girders and diaphragms and must be performed by the designer.

The designer is responsible for assuring that the structure is constructable and that it will be stable during all stages and under all loading conditions. To achieve this end, the designer must supply basic erection data on the contract plans. This information must include, but is not limited to, the following:

- Pick points and reactions at pick points for all girder sections.

- Temporary support points to be used during all stages and loading conditions, and reactions for which support towers should be designed at all of these points.

- Deflections to be expected in all girders under all conditions of temporary support and under all anticipated loading conditions.

- Direction pertaining to the connection of diaphragms to assure stability during all temporary conditions.

Specifications prepared for this work must require the Contractor to submit full erection plans, prepared and stamped by a Professional Engineer registered in the State of Connecticut, for review by the **CTDOT**. These plans will be reviewed by the designer as a working drawing and comments forwarded from the Office of Engineering to the District Engineering Manager having jurisdiction over the project for transmittal to the Contractor. The designer's review must ensure that all information given on the Contract plans has been accurately accounted for in the Contractor's erection plans.

The designer shall provide any such additional information, up to and including full erection plans in the Contract documents as directed by the **CTDOT**.

Further design information for curved structures is contained in the AASHTO *Guide Specifications for Horizontally Curved Steel Girder Highway Bridges*.

7.3.5 Through-Girders

Vacant

7.3.6 Trusses

Vacant

7.3.7 Rigid Frames

Vacant

7.3.8 Pin and Hanger Structures

The design of pin and hanger structures is not allowed.

7.3.9 Steel Piers and Pier Caps

Vacant

7.3.10 Railway Bridges

Vacant