Section Eight
Structural Steel

8.1 Design

8.1.1 Design Methods

Structural steel has long been used as a bridge material in New York State. It continues to be commonly used and is the usual choice for spans over 115 feet. Structural steel design should be in accordance with the *NYSDOT LRFD Bridge Design Specifications* for all new and replacement bridges. The *NYSDOT Standard Specifications for Highway Bridges* may be used for rehabilitation of existing bridges.

Load and Resistance Factor Design (LRFD) is the required design method for all new steel structures designed in New York State. It introduces limit states as a design philosophy and uses structural reliability methods to achieve a more uniform level of safety. Factor of Safety is replaced with a new statistically based measure of safety called the Reliability Index "β". LRFD requires a Design Reliability β=3.5, which provides for a notional failure probability of 1 in 10,000.

The LRFD code defines four design limit state categories:

- Strength Limit States - ensure strength and stability, both local and global.
- Service Limit States - impose limits on stress and deformation.
- Fatigue and Fracture Limit States - limit the liveload stress range under regular service conditions.
- Extreme Event Limit States - ensure the structural survival of a bridge during a major event such as a vessel collision, flood, earthquake, etc.

Within each category there are multiple limit states. Steel bridges shall be designed using Strength 1 (for moment and shear), Service 2 (overload, liveload deflection, bolted connections) and fatigue. A Strength 2 limit check of new girders utilizing the NYSDOT Design Permit Vehicle is also required.

LRFD introduces new live load criteria which will provide heavier loads on shorter spans and lighter loads on longer spans than are provided in the LFD specification.

Service Load Design, also known as **Allowable Stress Design (ASD)**, is the older and generally more conservative design method for medium to long bridge spans (over 100 ft). ASD achieves its factor of safety by limiting the stresses on the member to some percentage of the maximum stresses that the member could take before yielding. Since the dead load and live load stresses are considered at the same time, there is no provision for the certainty of the dead loads or the uncertainty of the live loads. As span lengths increase and dead loads become a much higher percentage of the total load, ASD becomes overly conservative and uneconomical.

Strength Design, also known as **Load Factor Design (LFD)**, achieves its factor of safety by applying multipliers, or load factors, to the design loads. These multipliers increase the load effects, or stresses, applied to the member above those induced from the design loads alone. Since the dead loads are known, the load factor applied to them is relatively small. By comparison, live loads are highly variable and, therefore, the applied load factor is relatively large. The factored stresses are then compared to the yield stress, or ultimate capacity, of the loaded member.

The benefit of handling dead loads and live loads separately is that it provides a uniform factor of safety for live load in bridges of any span length. As span length increases and dead load becomes a larger part of the total load, LFD becomes increasingly more economical than ASD because of the smaller load factor applied to the dead load.

LFD must always be checked for deflection and serviceability criteria. Designers are cautioned that at very long span lengths, typically in excess of 400 feet, LFD may not provide adequate reserve strength capacity in the bridge.

8.1.2 Analysis Methods

Straight girders should ordinarily be analyzed by the line element method. Only in very unusual circumstances should it be necessary to analyze a straight girder bridge by a grid, three-dimensional or finite-element analysis. The marginally increased refinement in the analysis offered by these techniques does not usually justify their substantially increased design effort. This conclusion is justified in large part by the fact that design loadings are only an approximation of actual traffic loads.

However, in some instances these more exact methods are justified. They are required for bridges with girders that have enough curvature to meet the requirements for curved girder analysis as defined by AASHTO. Some straight girder bridges that have extremely large skews (in excess of 45°), unfavorable continuous span arrangements, or faying girders (secondary girders framed to main girders for unusual geometric situations) may be candidates for a more exact analysis.

When a bridge is designed using a grid, three-dimensional or finite-element method of analysis; and has diaphragms and/ or bracing members acting as primary members (load paths), the qualifying information and Note 20 from Section 17.3 shall be placed on the contract plans. These conditions have special requirements for fabrication and erection of the bridge. Bridge types where this may apply include:

- Curved girder bridges with radii less than 600 feet
- Multi-span curved girder bridges with skews greater than 45 degrees.
- Curved tub girder bridges
- Skewed truss bridges
- Arch, and Tied Arch bridges
- Rigid frames

8.1.3 Design Considerations

The LRFD specification increases the role and responsibility of the designer to anticipate construction related issues and be aware that stresses during erection or construction are sometimes the controlling conditions of design. Examples of conditions that need to be checked are the erection of the girder and the placement of the concrete deck, both of which occur when there is a long unbraced compression flange. The designer should refer to Article 6.10.3 of the *NYSDOT LRFD Bridge Design Specifications* for requirements for stability checks.

8.2 Steel Types

8.2.1 Unpainted Weathering Steel

The preferred structural steel is unpainted weathering steel. Two grades are available; ASTM A709 Grade 50W and Grade 70 HPS - 70W. This steel eliminates the need for painting because the steel "weathers" to form a protective patina, or thin layer of protective oxide coating, that prevents the steel from further rusting. Its slightly higher cost per pound than nonweathering steels is easily offset by the savings in initial and maintenance painting. This steel should be used in most situations.

However, weathering steel has been known to exhibit problems in certain situations. These have generally been in environments where the steel has been exposed to wet conditions, salt spray or chemical fumes over prolonged periods. In these situations weathering steel may be unable to properly form the protective patina surface. The steel may be prone to delamination during the corrosion process and rapidly lose large amounts of its weathered surface material. Therefore, unpainted weathering steel should not be used under the following circumstances:

- Grade separation structures in "tunnel like" conditions where the steel is highly exposed to salt spray from the under roadway. These conditions can occur when there is minimum vertical clearance and substructures are located relatively close to the travel lanes of the under roadway
- Bridges over low water crossings where the structural steel is less than 8 feet over the ordinary water elevation.
- Marine coastal areas.
- Industrial areas where concentrated chemical fumes may drift directly onto the structure.
- Bridges exposed to spray from adjacent waterfalls or dam spillways, or located in an area of high rainfall, high humidity or persistent fog.
- Areas where debris can collect and primary connections may be exposed to roadway drainage (e.g., bottom chords of thru truss structures).
- Any staining of substructure is unacceptable.
- Color of weathering steel is not appropriate for aesthetic reasons.

It is strongly recommended that all weathering superstructure steel be painted within a distance of 1.5 x depth of the girder from bridge joints. Additionally, if the appearance of a partially painted girder is an aesthetic concern, the exposed area of the fascia girders should be painted for the entire girder length. This would include the entire fascia girder except for the top of the top flange and the interior surfaces of the web and top and bottom flanges. If a timber deck is used, see Section 10 - Timber for additional protective measures.

In locations where the guidelines do not specifically prohibit the use of weathering steel, but conditions such as excessive salt spray may compromise structural performance, the designer should increase flange and web thickness by approximately $\frac{1}{16}$ inch, if weathering steel is used. This will act as sacrificial section in order to achieve the intended service life.

8.2.2 Drip Bars for Unpainted Weathering Steel

The use of unpainted weathering steel for bridge superstructures results in the potential for staining bridge substructures during the period when the superstructure steel is developing a protective oxide coating. Rainwater flowing along the steel carries iron oxide particulates which are deposited on pedestals, abutment stems and pier caps.

While various methods for reducing or eliminating staining of substructures have been tried with varying success, current practice is to attach deflectors, called drip bars, to the bottom flanges of stringers in selected locations.

Drip bars are normally used only on structures having substructure units clearly visible to the public, such as piers or high abutments adjacent to an under roadway. It is not expected they would be used on structures over railroads, water, or at stub abutments of structures over highways.

Use of drip bars is determined at the Preliminary Plan stage of a project. If used, they are attached to the bottom flange of each fascia stringer at the low end of appropriate spans.

8.2.3 Painted Steels

When painted steel is used for aesthetic reasons or in situations where uncoated weathering steel is not desirable, ASTM A709 Grade 50 steel should preferably be used. It is usually the economical choice over Grade 36 steel. In structures that have only a small portion of the steel painted, such as beneath the joint systems of typical plate girder bridges, ASTM A709 Grade 50W steel should be used.

In structures that use painted steel it is possible to design main members using ASTM A709 Grade 50 and use ASTM A709 Grade 36 for secondary members and details. However, the cost differential between ASTM Grade 50 and ASTM Grade 36 is small, and it is therefore recommended for uniformity to use all Grade 50 steel.

In structures that need to have large portions of the steel painted, such as thru trusses, the entire structure should be painted rather than use weathering steel painted only in the splash zone. It is very difficult to paint steel to match the appearance of unpainted weathering steel.

The “Structural Painting Details” note required by Item 572.01, Structural Steel Painting: Shop Applied, shall contain the following information: description of serialized items, estimated structure length, width, vertical clearance, pay items to be used, description and location for pay items 574.02 and 574.03 if necessary, stream classification, and whether or not the structure is over a public water supply.

8.2.4 HPS Steel

The use of HPS steel requires approval by the D.C.E.S. HPS steel should be considered only when one of the following conditions exists:

- The layout of the structure can be reorganized to eliminate an entire span. As an example, if a proposed structure designed without using HPS is a five-span simply supported steel superstructure and can be replaced with a three-span continuous structure if HPS is used, HPS steel may be the best solution.
- One or more girders can be eliminated from a bridge cross section.
- The bridge requires a reduced superstructure depth, based on critical vertical clearance issues, which cannot be accomplished without using HPS.

Recent experience has shown that price analyses based on weight savings alone are not truly representative of final erected steel costs. Therefore, designers should include the following parameters in their cost analysis when deciding whether or not to incorporate HPS steel on a project:

- The added cost of splicing the higher strength steel
  - Bolted field splices must develop higher allowable strengths, which necessitate a greater number of bolts and longer length bolts to accommodate the increased pattern size. Consideration should be given to using Grade 50 steel to reduce cost.
  - For shop splices, because of the limits of the rolling stock available, there will be more splices in a specific size flange or web. Also, there will be an increased cost in extra required nondestructive testing.
- Erection cost - Because of extreme flexibility in the structure due to the large span to depth ratio high performance steel allows, there is a concern for lateral flange buckling. Additional falsework may be required to ensure the stability of members during erection.
- Shipping costs will increase because of the greater flexibility of the shipped units.

8.2.5 Other Steels

Various other steel types are used for special situations such as sheet piling and railing tubes. **If any steel other than A709 Grade 36, Grade 50 or Grade 50W is to be used for primary structural members, approval of the D.C.E.S. is required**.

8.2.6 Combination of Steel Types

When more than one type of steel is used in a contract, the types shall be clearly described in the plans. The payment for furnishing and placing these steels shall be made under a single structural steel item. A table titled "Total Weight for Progress Payments" shall be placed on the plans adjacent to the estimate table, indicating the quantity of each type of steel.

8.2.7 Steel Item Numbers

Depending on the type and nature of a project, steel shall be paid for under Item 564.XX or Item 656.0101 as described below. These items include the cost of the steel, shop drilled holes, and bolts.

On steel rehabilitation projects, designers must remember to include item numbers in the contract for steel removal (which includes the cost of bolt and/or rivet removal), field drilling of existing steel, and rivet removal and replacement with high strength bolts where applicable. See Section 19.4.5 for further information regarding rehabilitation of riveted structures.

Item 564.05XX, Structural Steel, L.S.

- New bridges and superstructure replacements.
- Shop drawings reviewed by D.C.E.S.

Item 564.10nnnn, Structural Steel Replacement, lb.

- Minor rehabilitation projects, with variable quantities due to unknown deterioration.
- Secondary member repair/replacement, minor repair to primary members: (e.g., diaphragm replacements and replacement of primary member stiffeners and/or connection angles.)
- Quantities verified by the Engineer-In-Charge.
- Shop Drawings reviewed by the Engineer-In-Charge.
- Stock steel option is allowed.

Item 564.51nnnn, Structural Steel, lb.

- Major rehabilitation contracts, with variable quantities due to unknown deterioration.
- Primary member replacement or strengthening: (e.g., truss rehabilitations, girder web and flange repairs, floor beam and stringer replacements, continuity retrofits and seismic retrofits).
- Quantities verified by the Engineer-In-Charge.
- Shop Drawings reviewed by D.C.E.S.

Item 564.70nnnn, Structural Steel Replacement, Each

- Minor rehabilitation projects with known quantities.
- Secondary member repair/replacement, minor repair to primary member components: (e.g., diaphragm replacements, and replacement of primary member stiffeners and/or connection angles.)
- Shop Drawings reviewed by the Engineer-In-Charge unless otherwise specified in the contract documents. Designer should consult with the Metals Engineering Unit to determine when D.C.E.S. review of shop drawings is required.

- Stock steel option is allowed.

Item 656.01, Miscellaneous Metals, lb.

- Used for extraneous items. (e.g., hand rails, metal floor grating, ladders).
- Shop Drawings reviewed as per *NYSDOT Steel Construction Manual*.

8.3 Redundancy - Fracture Critical Members

8.3.1 Primary and Secondary Members

Primary members are defined as structural elements that are designed to carry live load and act as primary load paths. Examples include: truss chords; girders; floor beams; stringers; arches; towers; bents; rigid frames. Additionally, lateral connection plates welded to the members listed above, and hangers, connection plates, and gusset plates which support the members listed above are primary members. Tub and curved-girder diaphragms are also included.

Secondary members are defined as those structural elements which do not carry primary stress or act as primary load paths.

8.3.2 Redundancy

Redundancy in structures is the ability of a structure to absorb the failure of a main component without the collapse of the structure. Superstructures have three types of redundancy:

- Load path redundancy.
- Structural redundancy.
- Internal redundancy.

With load path redundancy, the loads will be transferred to adjacent members or alternate paths with the failure of a single member. The best example of load path redundancy is a bridge with four or more longitudinal main girders. Structural redundancy is best typified by the middle spans in a continuous span bridge. Indeterminate trusses can also be structurally redundant. Internal redundancy occurs when a girder is composed of a number of components such as angles and plates which are connected by rivets or bolts (not welded). Only the first form of redundancy, load path redundancy, is generally counted on in design

8.3.3 Fracture-Critical Members

Fracture-Critical Members are defined as tension members or tension components of nonredundant members whose failure would result in the collapse of the structure. Tension components include any member that is loaded axially in tension or that portion of a flexural member that is subjected to tensile stress. Any attachment that is welded to a tension area of a fracture critical member or component is considered to be part of that member or component and, therefore, also fracture critical. It is important to realize that members can be nonredundant without being fracture critical (e.g., the compression chord of a truss is nonredundant but it is not fracture critical).

Examples of fracture-critical members or components are the tension flange and web of two- and three-girder systems, tension flange and web of steel pier cap beams, the tension chord and diagonals of trusses, the tie girders of a tied-arch bridge and the floor beams in a truss or thru girder that are spaced more than 12 feet on centers. All single tub and box girder structures shall be considered fracture critical. Some columns are fracture critical as defined by the designing engineer.

Examples of non-fracture-critical members are all components of the girders in any bridge with four or more girders, the compression chord of a truss and the stringers in a floor system of a thru girder or truss. Two- and three-girder pedestrian bridges and truss pedestrian bridges should not be considered fracture critical because they are not subject to high numbers of load cycles.

Bridges containing fracture-critical members should be avoided if possible. However, it is recognized that in many situations there is no good alternative to their use. Vertical clearance restrictions may necessitate the use of thru truss or thru girder structures. When spans become very long it also becomes cost prohibitive to provide a load-path-redundant structure.

Bridges that have fracture critical members have restricted allowable fatigue stress ranges and more stringent fabrication requirements. These issues are covered in the *NYSDOT Standard Specifications for Highway Bridges* and in the *NYSDOT Steel Construction Manual*. The *NYSDOT LRFD Bridge Design Specifications* requirements for fatigue design do not differentiate between redundant and nonredundant members. For this specification, both redundant and nonredundant members are designed for an infinite fatigue life. Fracture-critical members designed with this code are still subject to the fabrication requirements of the *NYSDOT Steel Construction Manual*.

- Designers shall designate and provide a table of all fracture-critical members on the contract plans.
- Designers shall designate tension zones of all fracture-critical members on the contract plans.
- When the Designer has determined that the column or column system is fracture critical, they shall designate all column components as fracture critical on new steel bents where columns experience tension under LRFD Strength III loading.
- When the Designer has determined that the column or column system is fracture critical, they shall designate all column strengthening components as fracture critical on major rehabilitations where a significant portion of the work is associated with the seismic strengthening and/or retrofitting of the structure.

8.4 Economical Design

8.4.1 Girder Spacing

A key element in producing an economical steel bridge design is the selection of girder spacing. While no absolute rule can be stated, the most economical design is usually the one with the least number of girders. There are, however, limitations that must be worked within. There should be a minimum of four girders and their spacing should not ordinarily exceed 12 feet In addition, restrictions on the available clearance requirements may force the use of more girders.

Stage construction requirements may have an impact on girder spacing, but there is no requirement to have more than four girders or an even or odd number of girders. Bridges can generally be stage constructed as easily with four girders as with five. It is good practice to check the economics of two or possibly three alternate girder spacings.

8.4.2 Girder Proportioning for Plate Girders

8.4.2.1 General

It is important to remember when proportioning plate girders that the design resulting in the least weight of structural steel is not necessarily the least costly option. Increased fabrication, construction, transportation and erection costs can easily outweigh a small savings in the quantity of steel used. Economical steel designs use good details and good proportions.

Generally, web and flange plate sizes and lengths for interior and fascia girders should be the same, with differences in deadload deflections between interior and fascia girders accommodated in the camber table.

8.4.2.2 Depth

There is an optimum depth to plate girder design. If there is flexibility in the allowable girder depth then a number of options should be explored to develop an economical design. Weight and cost of a girder will usually decrease as girder depth increases but only to a point. Beyond this point the weight and cost will increase as the girder depth is further increased. Very deep girders with small flanges may prove to be unstable and difficult to transport and erect.

8.4.2.3 Flanges

Minimum flange thickness shall be ¾” and minimum plate girder flange width shall be 12”.

When designing flanges, it is important to keep in mind that, in general, the most economical way for steel fabricators to make up flanges is to butt weld together several wide plates of varying thickness and then strip the flanges from the wide plate. Plate is usually purchased in widths starting at 4 feet. For the ordinary bridge, this usually makes it more economical to vary flange thickness rather than width. In large bridges, where there are significant changes in girder section needed and the quantities of each plate size are large, this guideline may be impractical or irrelevant.

Flanges should not be excessively wide compared to girder depth nor should they be excessively thick compared to the girder web thickness. A good rule of thumb is that the flange thickness should be no more than six times the web thickness.

As moment and shear change along the length of the girder, the required section of the girder also changes. It is frequently economical to introduce flange splices to utilize a lighter flange plate where possible. The savings in material achieved by making the splice must be balanced against the increased fabrication cost to make the butt weld. If the mass of material saved by making the splice is more than the amount computed by the following guidelines, then it is economical to make the splice.

Grade 36 steel:

lbs saved ≥ 300 + (25 x cross sectional area of smaller flange (in^2))

Grade 50 and 50W steel:

lbs saved ≥ 1.33 x (300 + (25 x cross sectional area of smaller flange (in^2)))

When making flange plate size changes, the thicker plate shall not be greater than twice the thickness of the thinner plate. It is good practice not to change the sectional area of the flange plates by more than a factor of 2 or the width by more than 8 inches. Flange transitions shall be tapered 1 on 4 for width transitions and 1 on 2.5 for thickness transitions. It is usually preferred to transition thickness rather than width.

8.4.2.4 Webs

It is recommended that webs of plate girders be at least ½” in thickness.

Web thickness is varied only in unusual circumstances. It is the standard practice to keep web thickness constant throughout the length of the girder. This is done for uniformity and in keeping splice and connection details simpler.

The main issue in economic web designs is whether or not to use stiffeners. It is usually the best choice to thicken webs sufficiently so that transverse stiffeners are not needed on girders under 48 in. depth. For girder webs above that depth, a good economic choice is usually to thicken the web sufficiently so that only a few transverse stiffeners are required in areas of high shear. Longitudinal stiffeners are rarely used and they become an option only with very large web depths. Designers should always check to see whether a stiffened or unstiffened web is more economical. Web thickness should be determined for both cases. The following guide can be used to help make the choice. It is economical to use the thicker web if the necessary thickness increase of the web does not exceed the amounts shown:

Grade 36 steel:

Increase in $t_w \leq (N(36 + W_{ST}) / 41L)$

Grade 50 and 50W steel:

Increase in $t_w \leq (N(28 + W_{ST}) / 41L)$

where:

t_w = web thickness in inches

N = number of stiffeners to be removed

W_{st} = weight in lb/linear ft. of one stiffener

L = length of web in feet to be increased

8.4.2.5 Stability During Erection

Stability of structural steel during transportation and erection is the Contractor's responsibility. However, designers must ensure that the structural steel can be erected without requiring extraordinary means of support. If the structure is designed using the *NYSDOT Standard Specifications for Highway Bridges*, the designer must check the local buckling stress of the compression flange due to steel dead load only during erection procedures. The designer must assume the location of field splices, determine segment lengths, and analyze each segment using the buckling stress and factor of safety requirements given in "Blue Page" Article 10.34.7 of the *NYSDOT Standard Specifications for Highway Bridges*. The stability of the spliced girder is the responsibility of the Contractor. If the calculated Factor of Safety against local compression buckling is less than 1.1, the designer shall increase the area of the compression flange or specify other means of temporary bracing.

If the structure is designed using the *NYSDOT LRFD Bridge Design Specifications*, splices are done by the designer and detailed in the Contract Plans. The girder segments must be checked according to the provisions of "Blue Page" Article 6.10.3.1.a (place Blue Page Note 1 or Note 2 on the plans). Detailed information on splice locations and maximum shipping lengths is provided in section 8.11.

8.4.3 Rolled Beams

Designers should check the economics of using rolled beams versus plate girders on short spans (under 100 ft.). Four alternatives in order of increasing fabrication cost should be considered.

- Rolled section
- Rolled section with cover plate on bottom flange
- Rolled section with cover plates on both top and bottom flanges
- Fabricated plate girder

Either of the first two alternatives may be more economical than a plate girder that uses less steel weight. Only in rare situations would the third alternative be cost effective because the total amount of time required to fabricate the beam would be comparable to that of a plate girder. Designers should not compare alternatives based on material weight savings alone. Rather, they should include potential savings achieved through the elimination of an operation during fabrication or through the elimination of field operations.

When specifying heavy W-shapes of any length (section weight greater than 370 lb/ft or flange thickness greater than 2 ¼") or any rolled beam longer than 65 feet in length the designer should check with the Metals Engineering Unit for availability of the shape.

Generally, cover plates should be used only on simple span structures. Two options are available:

- Full-length cover plates.
- Partial length cover plates using the end bolted detail shown in Fig. 10.3.1C in the *NYSDOT Standard Specifications for Highway Bridges* or Fig. 6.6.1.2.3-1 in the *NYSDOT LRFD Bridge Design Specifications*.

When full-length cover plates are used, they shall be extended so that the end of the plate is a maximum distance of 12 inches from the centerline of bearings. The purpose of the limitations is to move the undesirable Category E fatigue detail to a region of low stress range. Full length cover plates shall be welded to the flanges as shown in Figure 8.1.

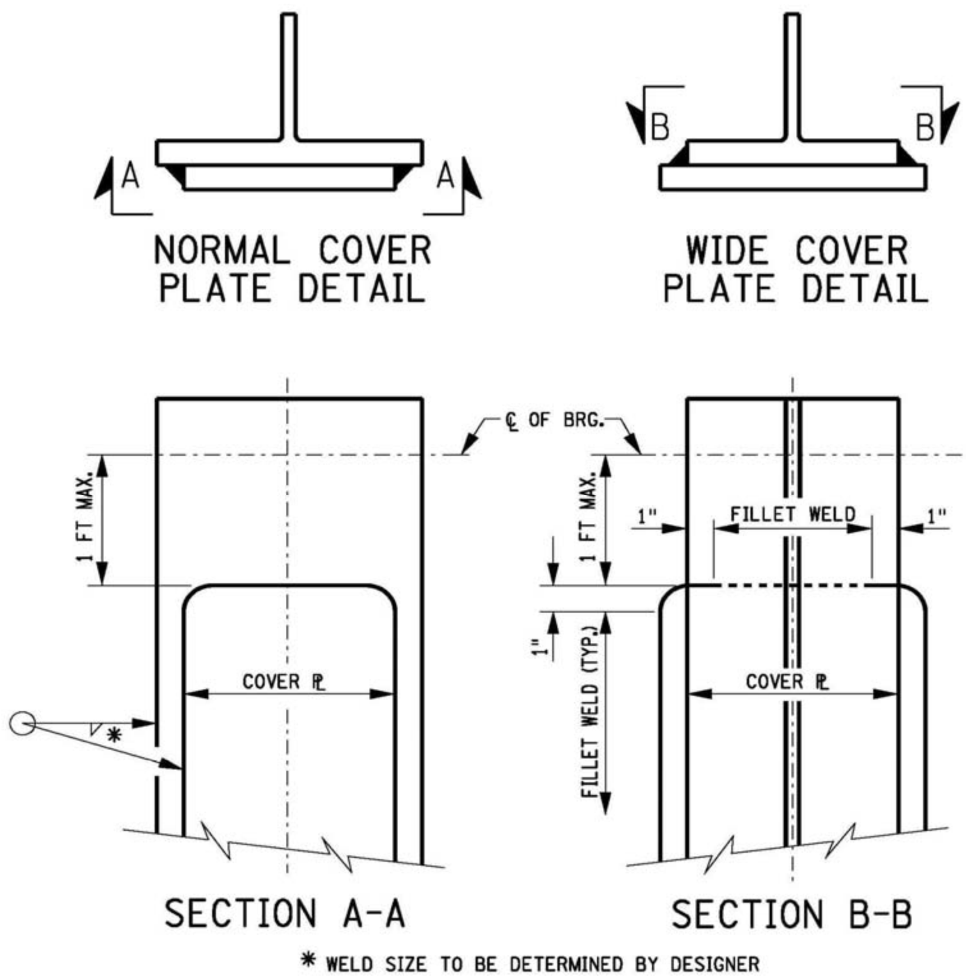


Figure 8.1
Cover Plate Connections

8.5 Metal Thicknesses

An effort should be made to design and detail steel plate in the following thicknesses:

$^3/_{16}$"	$^9/_{16}$	$^{15}/_{16}$	1 ⅝
¼	⅝	1	1 ¾
$^5/_{16}$	$^{11}/_{16}$	1 ⅛	1 ⅞
$^3/_8$	¾	1 ¼	2
$^7/_{16}$	$^{13}/_{16}$	1 ⅜	2 ⅛
½	⅞	1 ½	2 ¼

Over 2 ¼" use ¼" increments.

Table 8-1
Steel Plate Thicknesses

Structural steel, (including lateral bracing, cross frames, diaphragms and all types of gusset plates), except for the webs of certain rolled shapes, shall have a minimum thickness of $^3/_8$". The web thicknesses of rolled beams, channels and structural tees shall be a minimum of ¼". However, webs less than $^3/_8$" may require special welding procedures. These minimum thicknesses are specified to insure adequate protection against potential loss of section from corrosion. In areas where the metal is exposed to marked corrosive influences, it should be increased in thickness or specially protected.

Fill plates necessary to make connections are not subject to the $^3/_8$" minimum thickness requirements.

When plates are called out on the plans, their dimensions are called out in the following order: width x thickness x length.

8.6 Connections

8.6.1 General

Connections are a very important part of any structural steel design. Good details are important for strength, serviceability and maintenance of the structure as well as for economical construction.

Shop connections are usually designed as welded connections. Bolted connections are preferred in the field because automatic shop welding processes are often impractical in the field.

8.6.2 Bolts

All bolted connections on bridge projects shall be designed as slip critical, with Class A surface conditions, unless otherwise approved by the D.C.E.S. Bolt lengths shall be such that threads are excluded from the shear planes in the connection. When individual bolts are shown in horizontal joints on the plans, they should be shown with the bolt head up.

8.6.2.1 Bolt Types

ASTM A325 high strength bolts are preferred. A490 bolts should be used only when necessary and require D.C.E.S. approval.

Designers shall provide the following information on the contract plans for all structural steel connections: the design surface condition (Class A or B), the number of bolts, the bolt type, and the bolt diameter.

Bolt types are as follows:

- Non-Weathering steel applications (Shop applied zinc-rich primer)
  Mxx high-strength ASTM A325 (Type 1) or
  Mxx high-strength ASTM A325 (Type 1, hot dipped galvanized)

  Designers shall show both types of bolts on the contract plans. Choice is at Contractors discretion with only one type of bolt used per bridge.

- Weathering steel applications (Painted or Unpainted)
  Mxx high-strength ASTM A325 (Type 3)

- Galvanized steel applications
  Mxx high-strength ASTM A325 (Type 1, hot dipped galvanized)

8.6.2.2 Bolt Sizes

The normal size of high-strength bolts is ⅞ inch. An effort should be made to keep field bolts all the same size to avoid confusion.

⅝ inch bolts shall not be used in members carrying calculated stress except in 2.5 inch legs of angles and in flanges of sections requiring ⅝ inch fasteners. Structural shapes which do not permit the use of ⅝ inch fasteners shall not be used except in handrails.

The diameter of fasteners in angles carrying calculated stress shall not exceed ¼ the width of the angle leg in which they are placed. In angles whose size is not determined by calculated stress, ⅝ inch fasteners may be used in 2 inch legs.

8.6.2.3 Bolt Spacing

Bolt spacing is not ordinarily shown on the contract plans. This detail is best left to the fabricator. The contract plans should show the number of bolts and be checked to assure that the connection can be fabricated. However, bolt spacing is required on all splice design drawings.

The pitch of fasteners is the distance along the line of principal stress, in inches, between centers of adjacent fasteners, measured along one or more fastener lines. The gage of fasteners is the distance in inches between adjacent lines of fasteners or the distance from the back of angle or other shape to the first line of fasteners. The pitch of fasteners shall be governed by the requirements for sealing.

See the *NYSDOT Steel Construction Manual* for minimum bolt spacing and edge distances.

Stitch bolts shall be used in mechanically fastened built up members where two or more plates or shapes are in contact. The pitch of these fasteners shall be as per *NYSDOT* LRFD Article 6.13.2.6.4 through 6.13.2.6.6.

8.6.3 Welding

8.6.3.1 Weld Sizes

Intermediate stiffener and connection plate welds to flanges and webs shall not exceed $\frac{5}{16}$ in, unless required by design. Longitudinal stiffener to web welds shall not exceed $\frac{5}{16}$ in, unless required by design.

The minimum flange to web fillet weld sizes shall be as per *NYSDOT* LRFD Article 6.13.3.4.

8.6.3.2 Weld Detailing

When complete joint penetration groove (CJP) welds are called for, the only information that should ordinarily be shown on the plans is “CJP” in the tail of the welding callout. The joint configuration should not be called out. This is the responsibility of the fabricator to select and show on the shop drawings. Special finishing and contour can be shown if required.

For T and corner joints designers shall show UT testing requirements on the contract plans.

Partial joint penetration groove (PJP) welds are used only in special circumstances. They should be used only after consultation with the Metals Engineering Unit. Transversely loaded partial penetration groove welds shall not be used except as permitted in LRFD Article 9.8.3.7.2.

Designers and detailers are referred to the American Institute of Steel Construction (AISC) *Steel Construction Manual,* the American Welding Society publication D1.5, and the NYSDOT *Steel Construction Manual* for information on the proper method of detailing welded joints.

8.6.4 Copes

Simple shear coped beam connections have a history of being vulnerable to fatigue cracking initiating at the cope, and should be avoided whenever possible. This is especially pertinent in floor beams and stringers of truss and thru girder spans. There are design situations, however, where coped connections cannot be avoided because of framing considerations. Two cases shall be considered for main/primary members:

Case 1 Cope depths < 6 inches:
The minimum radius of the cope shall be 2 inches.

Case 2 Cope depths ≥ 6 inches:

The minimum radius of the cope shall be 6 inches to reduce the stress concentration that may be present at a notch or tight radius cope.

Cope depths greater than 6 inches shall be reinforced using a horizontal reinforcement plate welded on each side of the web within the limits of the cope. (See Figure 8.2)

Designers may contact the Metals Engineering Unit for specific guidance when this situation arises.

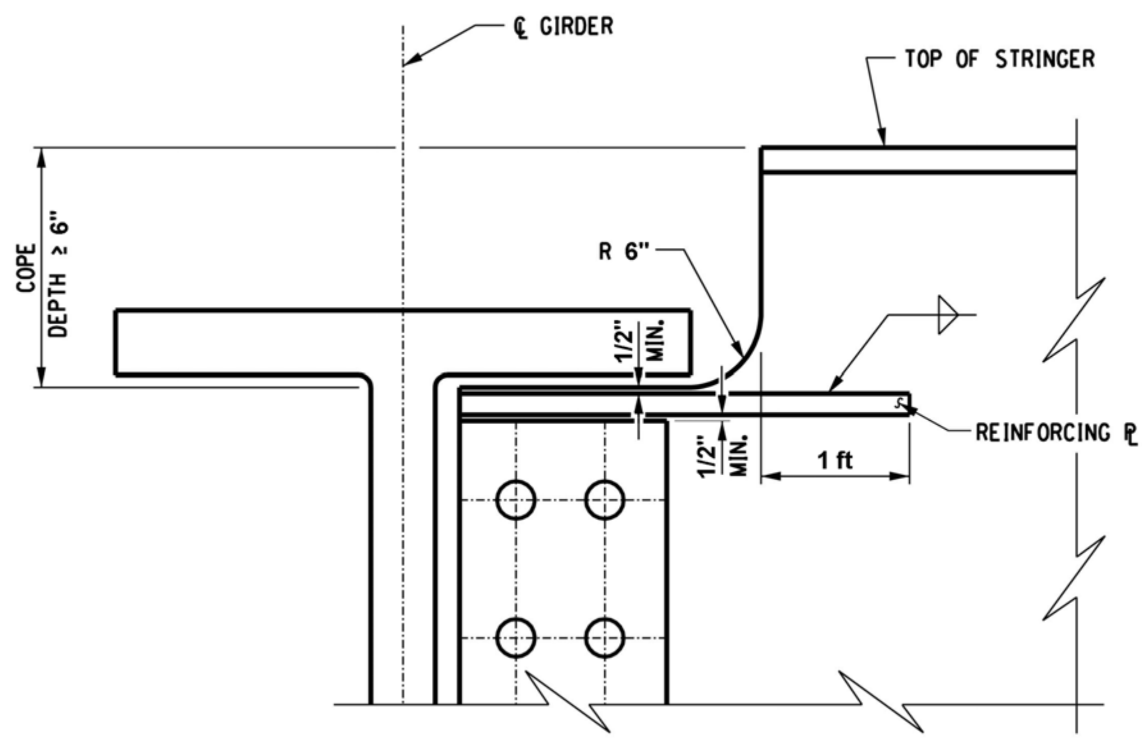


Figure 8.2 – Reinforced Cope Detail

8.6.5 Connection Design

Connections shall be designed as slip-critical connections. Slip-critical connections are required in primary members because they carry live load. Diaphragms and laterals in curved-girder bridges carry live load and are primary members. Diaphragms in straight-girder bridges are secondary members. The *NYSDOT Steel Construction Manual* allows the use of oversize holes for secondary members. Oversize holes cannot be used in bearing-type connections, therefore the connections must be designed as slip critical. Where floor beams are connected directly to stiffeners, knee braces or connection plates, the floor beams shall not be coped. The flanges shall be cut and chipped to provide a smooth faying surface as shown in Figure 8.3.

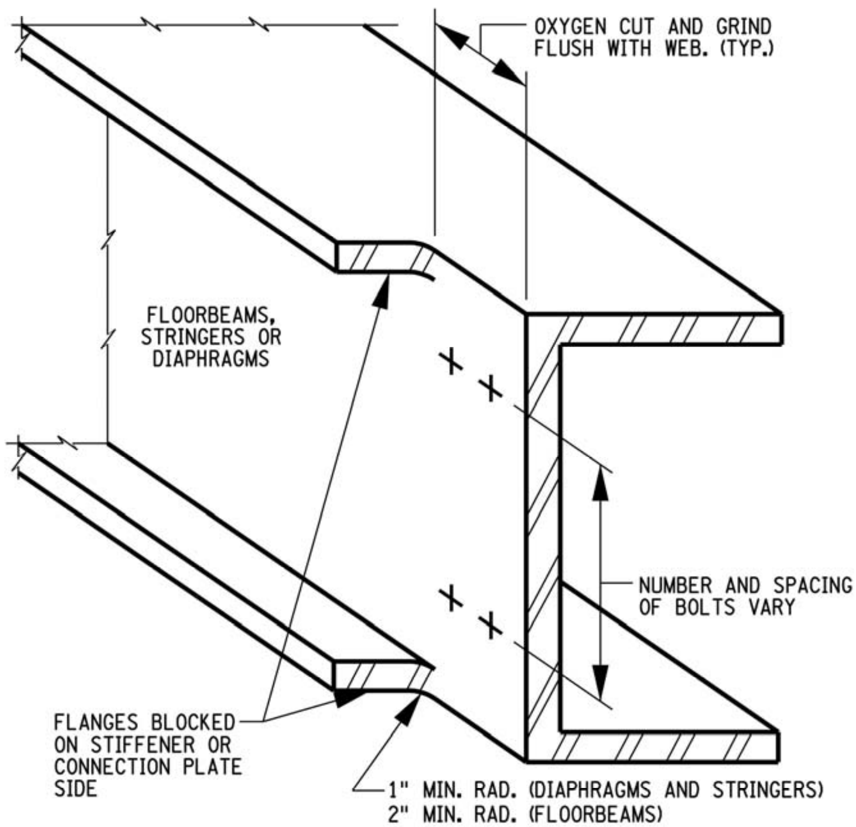


Figure 8.3
Blocked Flange Detail

Article 6.13.1 of the *NYSDOT LRFD Bridge Design Specifications* states that the end connections of diaphragms and cross frames shall be designed for the calculated member loads. It is not necessary to design the end connections of diaphragms and cross frames for 75% of their shear or axial capacity.

8.7 Stiffeners

8.7.1 Bearing Stiffeners

Bearing stiffeners shall be a minimum of ¾" thick and a minimum of 7" wide. Bearing stiffeners shall be placed parallel to the skew for skews ≤ 20 degrees, and normal to the web for skews >20 degrees.

Bearing stiffener welds shall be as shown on the current BD SG sheets.

The ends of all beams and girders and all bearing stiffeners shall be vertical after dead load deflection.

When two pairs of bearing stiffeners are used for very large reactions, the stiffeners must be placed a sufficient distance apart to permit access to weld the stiffeners to the web. The spacing between stiffeners should be at least equal to their width.

8.7.2 Intermediate Stiffeners and Connection Plates

Intermediate stiffeners shall be a minimum of ⅜" thick and 4" wide. Connection plates for straight girder cross frames and diaphragms shall be a minimum of ½" thick and 7" wide. Connection plates for curved girder cross frames and diaphragms shall be a minimum of $^{9}/_{16}$" thick and 7" wide. Connection plates also serve as intermediate stiffeners.

Connection plates shall be placed parallel to the skew for skews ≤20 degrees, and normal to the web for skews >20 degrees. Transverse intermediate stiffeners that are not connection plates shall be placed normal to the web.

On fascia girders, intermediate stiffeners shall be placed on the side of the web which is not exposed to view. On interior girders, they shall be located on alternate sides of the web, except where they are used in conjunction with a longitudinal stiffener on the other side.

Intermediate stiffener welds shall be as shown on the current BD SG sheets.

Connection plate welds shall be as shown on the current BD SG sheets.

- When welding directly to the tension flange, designers shall limit the fatigue stress range to category C'.

8.7.3 Longitudinal Stiffeners

Use of longitudinal stiffeners should be avoided whenever possible.

Generally, longitudinal stiffeners shall be continuous for their entire length, with intermediate transverse stiffeners and connection plates cut short to avoid intersecting welds. Exceptions are when the longitudinal stiffener is interrupted by a field splice in the girder, or when the stiffener is no longer required by design. In these circumstances, the designer shall be responsible for providing the appropriate termination details that comply with the *NYSDOT Steel Construction Manual* on the contract plans

When longitudinal stiffeners are required, show them placed on one side of the web only. On fascia girders they shall be placed on the web surface exposed to view. The intermediate transverse stiffeners, if necessary, shall be placed on the opposite side of the web. The longitudinal stiffeners shall be attached to the web plate with full-length, continuous, $^5\!/_{16}$" fillet welds. Fabrication details including transverse connection plate and longitudinal stiffener-intersection details shall be in accordance with the *NYSDOT Steel Construction Manual*.

8.8 Designation of Tension Zones

The Contract Plans shall clearly indicate the limits of tensile stress on each flange of all continuous steel girders. This will facilitate control of materials and welding inspection during fabrication and erection, as specified in the NYSDOT *Steel Construction Manual*. This requirement shall apply to reconstruction projects which require new deck slabs, as well as to new structures.

On continuous steel girder bridges a sufficiently accurate approximation of the point of combined load contraflexure may be obtained from moment diagrams alone. Using the moment tables shown on the plans, the designer can total dead load moment, superimposed dead load moment, and the appropriate live load moment at incremental points along the girder. The point where zero combined moment occurs can be found by interpolation. This point will reasonably represent the end of a tension zone and shall be shown as such on the plans.

If stress calculations are available, stresses may be used instead of moments. Designers need not calculate stresses for this purpose alone. The moment diagram method produces a conservative estimate of the tension zone limits. Stress calculations improve on this estimate by factoring in the effect of differing section moduli. However, actual loadings and section moduli may vary from the assumed values.

Where tension zones terminate less than 10 feet beyond the dead load point of contraflexure, the distance of 10 feet± shall be shown. The actual distance computed shall be shown for distances greater than 10 feet.

8.9 Camber

Design cambers include: structural steel dead load, concrete dead load, superimposed dead load, vertical curve, and total of the above. The dead load from a future wearing surface shall be included in the determination of camber. When cambers vary between girders due to differing concrete slab loads, concrete placement sequence, or stage construction issues, they shall be shown separately in the table.

A camber table and camber diagram shall be shown on the plans. See the current structural steel Bridge Detail (BD) Sheets for details.

If a steel member is designed with no camber, a note shall be placed on the plans instructing the fabricator to place the mill camber up.

8.9.1 Sag Camber

By definition, a girder is said to have sag (or negative) camber if any portion of the curve formed by the top of web in the completed structure falls below a working line constructed through the top of web points at the girder ends.

Note that all intermediate support points are ignored when applying the above definition. The designer's attention is directed to the fact that sag camber can be introduced into a girder from superstructure geometry other than from a sag vertical curve. These other conditions include any superstructure (straight or curved) in which a superelevation transition length occurs within the span, or a horizontally curved superstructure supported on straight girders.

Girders with sag cambers are to be avoided because their unstable appearance is aesthetically objectionable. An exception to this policy may be made when the under feature of the structure is a waterway. This exception recognizes a reduced concern for aesthetics.

Designers may find that approved geometrics for a bridge project have not considered the Office of Structures' policy regarding sag cambers. If this condition exists, the Designer shall use the following guidelines to minimize the effect or eliminate, when possible, designing a sag cambered superstructure.

1. Investigate the possibility of revising the geometrics (i.e., modifying or relocating the sag vertical curve and/or modifying or relocating the superelevation transition off the superstructure). In those cases where a deeper haunch is required, the 8" reinforced haunch should be used in conjunction with a sag camber.

2. If a revision of the geometrics is not possible, a variable haunch shall be introduced to eliminate the need for the sag camber. The depth of haunch for this purpose shall be limited to a nominal 8".

8.10 Moment, Shear and Design Load Tables

A table showing moment, shear, and design loads shall be provided on the plans. See the current structural steel BD sheets for details. Moments and shears shall be given at the same intervals as the camber table. Moments and shears for AASHTO HL 93 and the NYSDOT Design Permit Vehicle need to be shown separately.

8.11 Splices

8.11.1 Girder Splices

Girder details for all LRFD projects with spans of more than 140 feet or where a splice is otherwise required shall be prepared with field splice locations and splice design details shown on the plans. Details and location access constraints control the erection procedure. However, designers must always assure themselves that girders can be field spliced following the criteria shown in this section.

In the design of long stringers and girders, simple or continuous, straight or curved, consideration should be given to the need for field splices. Bolted field splices are preferred over welded field splices, because of substantial savings in time and money. Fill plates are not allowed.

Except for those cases where it is obvious that no field splice will be required (span lengths less than 130 feet. for straight or large radius curved members), the flanges should have sufficient excess area at points where splicing is anticipated to permit a bolted splice to be made.

Splice locations are generally selected near points of dead load contraflexure and where there is sufficient flange area to permit hole drilling while still maintaining the required net area.

DESIGN

General Practice

For simple spans or continuous spans where the total girder is less than 140 feet in length, the girder may be assumed to be erected as a single segment and no splice design will be necessary.

For simple spans greater than 140 feet. in length, the preferred location for the splice, based on load considerations only, is at the one-third point.

For continuous spans greater than 140 feet. in length, the preferred location for the splice, based on load considerations only, is near the dead load contraflexure point. Note that on longer structures the points of dead load contraflexure can be greater than 140 feet apart, in which case the preferred locations would be where the size of the splice and number of bolts is minimized.

Additional constraints on splice location include the following:

- The minimum distance from a flange plate transition groove weld to the nearest flange splice bolt hole or lateral gusset plate bolt hole is 12 inches
- The centerline of field splice shall be located >5 feet from a flange plate transition groove weld.
- The minimum distance from a lateral gusset plate to the end of a flange splice plate is 6 inches.
- The minimum distance from a stiffener or connection plate to the end of a flange splice plate is 12 inches.
- The minimum distance from a stiffener or connection plate to a groove welded splice in either the flange or web is 6 inches.

As is current practice, the compression flange must be designed considering the steel dead load acting on the unbraced length (before diaphragms are attached). Refer to Section 8.4.2.5 for requirements for stability of the structural steel during transportation and erection.

It is preferable to group the design of the splices at any splice location by designing all splices using the heaviest section or greatest moment rather than vary the splice designs across the structure. This avoids confusion and possible construction problems, and should provide the most economical solution. In addition, it is preferable to have one design for all splice locations rather than having a different design at each splice point.

Vertical Clearance

When locating the splice, the designer shall consider the effect of the splice on vertical clearance. Vertical clearance at the splice location will be reduced by the bottom flange splice plate, washer, nut and free end of bolt (see AISC table titled “Entering and Tightening Clearances”). If the splice affects minimum or critical vertical clearance, the designer shall show the revised minimum or critical vertical clearance on the plans. Vertical clearance issues may control the location of splices.

Erection

Erected and spliced segments must be statically stable. Depending on the span arrangement, this may require the use of falsework or splicing of the girder on the ground. Note that when a girder is spliced on the ground the unbraced compression flange length may increase. The girder must be stable during all phases of erection and construction.

Structures which are difficult to erect (e.g., tub girders, long simple spans) should show a suggested method of steel erection in the Contract Plans. This is required because the Contractor is responsible only for additional stresses caused by their erection scheme, and the Contractor may assume the simplest erection method possible if none is shown on the plans.

Falsework

A generalized falsework schematic should be shown on the plans when it is required for stability of the compression flange or stability of the structure. When falsework is required, the designer must get approvals from the appropriate agencies. The Rail Unit, Real Estate or Highway Design (for Temporary Traffic Control) may typically need to be contacted. Railroads will not allow falsework within the track zone and also may not allow any splices above the tracks. Maintenance and Protection of Traffic issues may also control the location or use of falsework. Design of the falsework is the responsibility of the Contractor, subject to the approval of the D.C.E.S.

Shipping

The maximum shipping length is 140 feet based on permitting and geometric limitations. The maximum girder depth is typically 14.0 feet, although depths up to 16 feet may be used in special circumstances with the approval of the Metals Engineering Unit. The issue of special hauling permits is typically handled by the fabricator and is controlled by weight of the girder segment and the configuration of the truck and trailer used. The maximum shipping weight of a segment is 100 tons.

Cranes

For typical structures, the designer may assume the maximum single crane pick is 100 tons. Nearly all structures constructed for the Department are erected by a single crane of this type. For structures which require larger or multiple cranes to erect, contact the Metals Engineering Unit for assistance. When splicing needs to be done before erection it should be noted on the plans so the Contractor is aware of the possible need for a larger (or multiple) crane(s) at bidding.

Additional Items

A High-Performance-Steel simple span may be long enough to require the use of two field splices.

Falsework up to 16 feet in height may be assumed to cost $5,000 per location for typical 40 to 50 foot wide structures. It is preferable to avoid the cost of these temporary structures and strengthen the compression flanges if the cost is similar.

Fracture-Critical Members shall have splice plates constructed from Fracture-Critical material.

Design Calculations

Bolted designs shall use ASTM A325 bolts only. Bolts should be designed as per the *NYSDOT LRFD Bridge Design Specifications* and the *NYSDOT Steel Construction Manual (SCM)*. Bolts must be designed both for strength and for slip-critical loading using Class A surface conditions unless otherwise approved by the D.C.E.S. Bolt lengths shall be such that threads are excluded from the shear planes in the connection. Designers should reference *NYSDOT SCM*-Section 2 on bolting and splices (including fill plates, as appropriate). Use 7/8 inch bolts for typical girder splices. Unusual structures may require a larger bolt size.

Refer to the American Institute of Steel Construction Table titled “Entering and Tightening Clearances” and to Section 8.6.2 of this manual for a discussion of bolted connections.

Computer Programs

AISIsplice is the recommended program for splice design. For questions involving this program contact the Structures IT Systems Unit or Metals Engineering Unit.

AISIsplice has the following limitations:

- It is limited to straight steel I-girders.
- It will not design hybrid splices, girders must be homogeneous.
- Flanges should be parallel at the location of the splice (do not locate the splice at a location the web depth is varying).
- Bolt patterns are limited to constant pitch, nonstaggered patterns.
- The program designs only symmetric splices, which may not be the most cost effective.
- The program may calculate section properties of the concrete deck slab incorrectly when the top flange of the girder is embedded into the slab.

Currently the department has no software which can design curved girder, tub girder, hybrid or box section splices. Contact the D.C.E.S. when designing splices for these types of girders.

Estimate

The splices should be paid for under the appropriate items. No additional weight calculations are necessary for typical structures, as the typical 3% accounts for the splice plates and bolts.

8.11.2 Rolled Beam Splices

When rolled beams are used for continuous structures, the field splices should be located in areas where no cover plates are required and consideration should be given to the fact that the fatigue strength of the section adjacent to the bolted connection (Category B*) is less than the fatigue strength of the base metal in areas where there is no splice (Category A*).

* See Article 6.6.1.2 of the *NYSDOT LRFD Bridge Design Specifications* or Article10.3.1 of the *NYSDOT Standard Specifications for Highway Bridges*.

8.12 Framing Plans

Typical framing plans for steel structures are shown in the current structural steel BD sheets. Diaphragms shall be placed parallel to the skew angle for skews 20° and less. Diaphragms shall be placed perpendicular to the girders for skews over 20°.

8.13 Curved Girders

On curved girder projects with radii less than 600 feet it is important to coordinate with the Metals Engineering Unit early in the design phase to assure that special fabrication and erection concerns are addressed.

Diaphragms in curved girder structures are primary members and designed to carry dead and live load. Except for end diaphragms they should be placed radial to the girder in a single line across the bridge. A diaphragm should not be placed along the line of support at an interior skewed support. Curved girders have special diaphragm and lateral details that are shown on the current structural steel BD sheets.

Curved girders that are designed as straight girders because their curvature does not exceed the limitation contained in the *NYSDOT LRFD Bridge Design Specifications* still need special provisions for design and detailing. These girders must also use the diaphragm and lateral connections details for curved-girder bridges.

8.14 Trusses

It is important to coordinate with the Metals Engineering Unit of the Office of Structures early in the design phase of a truss project to assure that fabrication concerns are addressed.

8.14.1 General Considerations

Trusses are a viable structural form when there are clearance restrictions on beam depth that would preclude the use of girder spans. Trusses also become an economic option when span lengths are long enough to make plate girders impractical. Trusses are a very efficient structural form in the use of material, however their complex fabrication tends to make them costly. They are also usually nonredundant structures which leads to special design considerations.

A modified Warren truss (incorporates verticals) is usually appropriate for most highway bridge applications, although other truss forms can be considered.

Skewed trusses should be avoided if possible. The skew makes fabrication difficult and costly and introduces out of plane bending problems to the structure. Small skew angles can often be eliminated by a small increase in the span length.

End portals and sway bracing should be placed a minimum of 16’-6” clear above the roadway surface (includes usable shoulder), regardless of minimum vertical clearance requirements for that highway classification.

It is desirable to keep sidewalks inside the trusses rather than placing them on outside cantilevers. A vertical faced concrete parapet should be used between a sidewalk and the truss. This provides more lateral stability to the structure and keeps traffic and road salts away from critical members. Adequate clearance should be maintained between the concrete barrier or parapet and the truss to accommodate formwork.

When a metal railing system is used on bridge rehabilitation projects with concrete decks, it is preferred that the system be anchored in the deck and not attached to the truss elements. Consideration should be given to providing a clear zone to accommodate lateral deflection of the railing system.

Weathering steel is recommended for trusses because of its superior toughness. See Section 8.2.3 for painting guidelines. Galvanized steel may also be an option for trusses.

8.14.2 Truss Design Guidelines

Geometry:

Truss and member proportions should follow the guidelines provided in the NYSDOT LRFD Bridge Design Specifications or the NYSDOT Standard Specifications for Highway Bridges.

Sections:

Designers should keep variations in member shapes and sizes to a minimum. To achieve this objective, it is often desirable to establish a constant out-to-out dimension for all chord members. Based on past experience, it is frequently more cost effective to use fabricated members than rolled sections because of their tighter tolerances. Rolled sections may vary for "tilt" and "in-out" by more than $^3/_{16}$". and sometimes require further work to bring them into the necessary tolerances.

Designers should use closed box sections for bottom chords whenever possible. Although closed box sections are more expensive to fabricate, they eliminate the long term maintenance and durability concerns associated with H-shaped sections. H-shape sections tend to trap debris and moisture.

Framing:

The floor system framing of trusses should be designed as simply supported although it is recognized that some negative end moments can and probably will develop. This should be considered when designing fatigue resistant details.

Stringers should be framed from floorbeam to floorbeam. Stringers that run continuously over the tops of floorbeams have led to uplift and fatigue problems. Additionally, consideration should be given to framing stringers below the plane of the floorbeam top flange to eliminate the cope at the top of the stringer.

Internal Diaphragms:

Designers shall include internal diaphragms within fabricated closed box chord sections. These diaphragms are to be located at panel points, and elsewhere where required by design.

Camber:

Because the steel fabrication industry prefers assembling trusses in a fully cambered position, (i.e.: member lengths adjusted for deadload and vertical curve cambers), designers are advised to evaluate the secondary force effects which will arise when the truss is fabricated in this fashion. It should be noted however, that these secondary force effects are generally minor when the truss proportions follow the guidelines provided in the *NYSDOT LRFD Bridge Design Specifications* or the *NYSDOT Standard Specifications for Highway Bridges*.

Gusset Plates

Design guidance for gusset plates can be found in Structure Design Advisory 08-001 (LRFD) and load rating guidance can be found in Technical Advisory 09-001 (LFD)

8.14.3 Truss Detailing Guidelines

Floor beam to truss connections should be blocked and never coped.

Details should be used that allow accessibility to make field bolted connections. Hand holes in the bottoms of closed box sections will be needed for erection purposes. These holes shall be protected with screening to prevent roosting birds from entering.

Details that allow accessibility for cleaning and high pressure washing are desirable.

Fill plates in bolted connections are sometimes necessary. Fillers greater than or equal to ¼" thick shall be designed in accordance with Section 6.13.6.1.5 of the NYSDOT *LRFD Bridge Design Specifications*.

Use Category "C" or better welded fatigue details on all fracture critical members.

Internal diaphragms on closed box sections should be detailed as being fillet welded to three sides, and tight fit to the fourth.

Designers shall include the following information on the contract plans, to facilitate the quality assurance review of the steel fabrication drawings:

- Table of Fracture Critical Members
- Table of LRFD Member Forces: DC1, DC2, DW, LL + Impact (AASHTO HL-93 and NYSDOT Permit Vehicle)
- Truss Camber Diagram: Provide the lengths members must be lengthened or shortened to compensate for dead load and vertical camber. Dimensions provided should include total unfactored deadload (DC1 + DC2 + DW) and vertical curve camber.
- Truss Working Lines Diagram: Provide member lengths (with horizontal components adjusted for grades greater than 3%), and offsets to datum for grade.

8.15 Miscellaneous Details

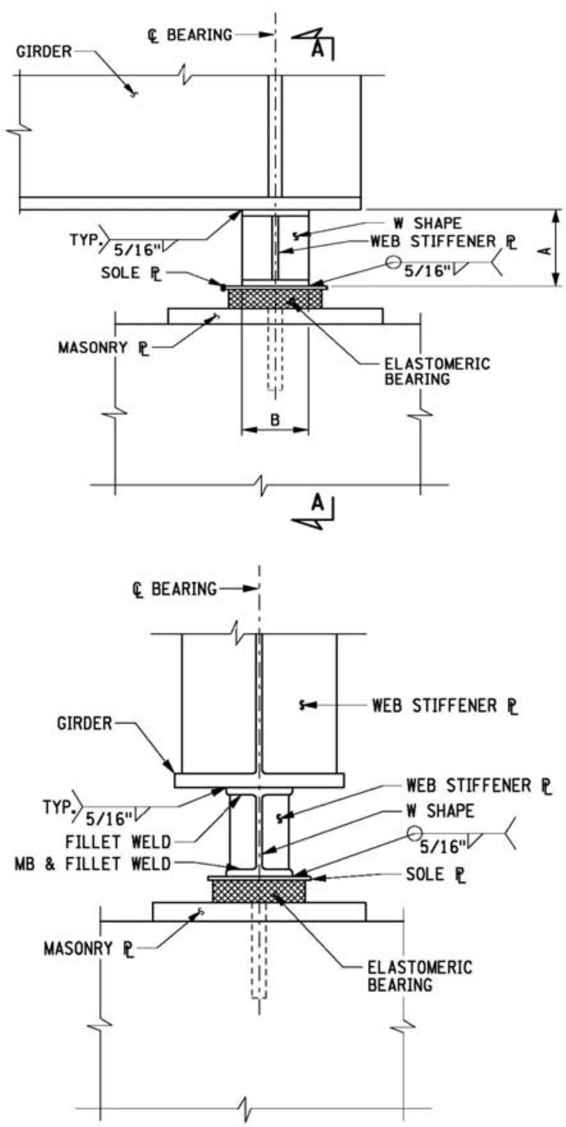
8.15.1 Bolsters

Bolsters are steel supports placed beneath the girder and above the bearing. They are typically used at piers when two spans have different depths. In new construction it is almost always preferable to step the concrete of the cap beam or pedestal instead of using bolsters. For aesthetic reasons it may be appropriate to investigate alternative designs that would not have adjacent spans with different girder depths. (See Section 23.)

When bolsters are used, they must be carefully designed and detailed. Two types of bolsters are available, based on their aspect ratios.

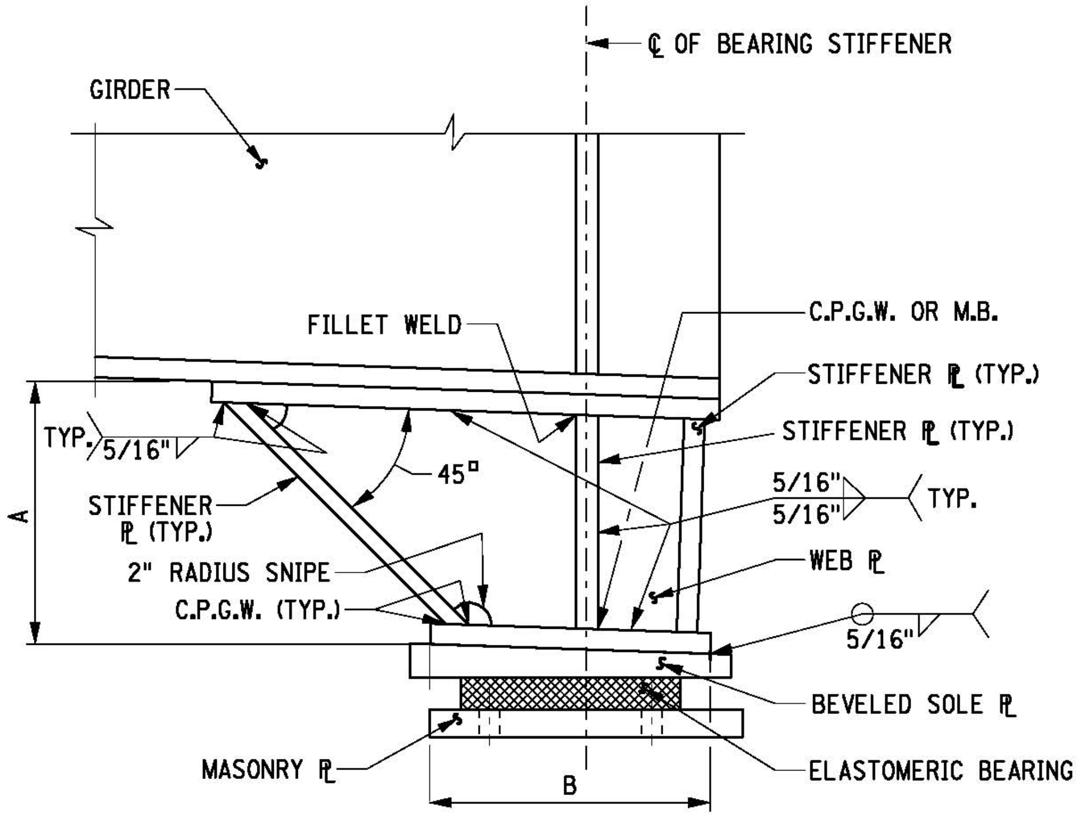
- Low bolster, A/B <1   Use rolled section, See Figure 8.4
- High bolster, A/B ≥1   Use fabricated section, See figure 8.5

Bearing stiffeners on bolsters should meet the same design, detailing and fabrication requirements as bearing stiffeners on girders.



Note: weld sizes shown are minimums

Figure 8.4
Low Bolster Detail and Section A-A



Note: weld sizes shown are minimums

Figure 8.5
High Bolster Detail

Bolsters shall be paid for separately under Item 564.70, Structural Steel Replacement. They are not included in the bearing item in order to assure that the steel fabrication is performed in the proper manner.

8.15.2 Safety Handrail

Safety handrails for use during bridge inspections shall be used on girders having a web depth of 5 feet or greater. They should be used on both sides of interior girders and on the inside of fascia girders. Details of field-erected and shop-erected handrails are available on current BD sheets. Cost of handrails shall be included in the unit prices bid for the structural steel.

8.16 Railroad Structures

8.16.1 General Considerations

Railroad structures are commonly 2 or 3 girder structures that contain fracture critical elements.

Contract plans shall include:

- A listing of all primary/main members.
- Tension zones defined for floor-beams and girders
- A table of all fracture critical members.

8.16.2 Design

Design of railroad structures shall be in accordance with current A.R.E.M.A. specifications.

8.16.3 Details

The purpose of knee brackets is to brace the compression flange of through girders and support the ballast curb plate. The flanges of the knee brackets should not be interrupted by notching to accommodate the curb plates. Although this will cause the cover plates to be installed in multiple segments, the integrity of the knee bracket outweighs the ease of installation issue.

Curb plates should be notched to fit around stiffeners and girder web attachments as needed. The curb and cover plate needs to be contiguous to protect the membrane system. Curb plates shall be bolted to knee braces and the girder web using clip angles. Welding should only be considered where access is a problem. Unless alternatives are impractical, curb plates should not be welded to the intermediate stiffeners.

The deck plate may be welded to the curb plate. The knee bracket must be cut short to allow for the attachment of the curb plate to the deck plate. The deck plate needs to be installed under the knee bracket during construction. This leaves a gap underneath the knee bracket to allow the deck plate to be installed. The curb plate is configured to have a v-groove joint at the junction of the curb and deck that can be welded with a partial penetration grove weld in the field.

8.17 Movable Bridges

Design of projects of this complexity requires special consideration. Early involvement with the Metals Engineering Unit is highly recommended.

A very different set of criterion must be followed on moveable structures, such as bascule or post-lift bridges. Specifically the nondestructive testing requirements for the machine parts, etc., for the electrical and mechanical portions of the bridge must be clearly defined on the contract plans. Additionally there may be stair wells, hatches, and other appurtenances that should be detailed and shown with the proper steel payment item on the contract plans.

Contract plans should also include:

- Identification of main members and/ or tension components
- Identification of the tension or reversal zones.
- Listing of fracture critical members or members that must meet minimum toughness (CVN) requirements i.e. bascule lateral bracing or edge beams.
- Special Non-destructive testing requirements.

Designers should consult the AASHTO LRFD Movable Highway Bridge Design Specifications.

8.18 Pedestrian Bridges

8.18.1 General

Pedestrian bridges may be detailed as I-beams, box girders or a prefabricated truss. I-beam or box girder pedestrian bridges should be completely designed and detailed in the contract plans. Prefabricated truss pedestrian bridges require a different approach because they are designed by the manufacturer after the contract has been awarded. See Section 2.6.4 for loading requirements.

The contract documents must provide sufficient details so the manufacturer can supply the intended type of structure. Discussions with the owner should include any project specific aesthetic or architectural treatments required. The Regional Landscape Group can provide guidance on aesthetic or architectural treatments choices and any special requirements from the Americans with Disability Act (ADA).

While the designer must provide the manufacturer with enough information so that an accurate bid can be prepared, the designer must also recognize that unnecessary restrictions may result in excessively high bids. Only specify those specific requirements absolutely necessary for the bridge to meet the project's safety, aesthetic and structural requirements.

8.18.2 Design Guidelines

The designer should follow the considerations and guidelines for trusses in Section 8.14. While Section 8.14 is intended for highway trusses, designer should review it for application to pedestrian trusses.

Skewed supports should be avoided for prefabricated trusses. Grades greater than 5% should be avoided on pedestrian bridges because it involves additional ADA requirements.

8.18.3 Detailing Guidelines

If a particular type of truss is required it must be clearly indicated in the contract plans. The truss types typically used are Warren, Pratt, Bowstring and Howe. Designers should indicate all acceptable truss types in the contract documents. For additional information see the following section on truss member styles.

Deck joint and accessory details should be indicated but full details should not be shown, as they are included as part of the proprietary bridge superstructure.

Bearings need to be shown on the plans but not detailed because they are designed by the contractor's engineer in accordance with the pedestrian bridge specification.

Truss Member Style:

Top chords: The top chords may be sloped or horizontal. Trusses with sloping top chords or lenticular configurations are often preferred based on aesthetic considerations.

Verticals: The locations of vertical members must be indicated on the contract plans, if required.

Overhead/portal bracing: Overhead/portal bracing details must be shown if allowed or required. If overhead/portal bracing details are not allowed that must also be clearly indicated on the plans. If overhead bracing is allowed vertical clearance requirements shall be shown. Standard vertical clearances can be found in Section 2.

Camber:

The required camber shall be indicated in the contract plans for non-prefabricated bridges.

For prefabricated bridges the camber is the responsibility of the fabricator. However, the designer should indicate desired final appearance of bottom chord (e.g., flat or follow profile) for aesthetic reasons.

Finish:

The required finish of the steel shall be indicated on the contract plans. Finish options include painted, weathered, or galvanized steel. When steel is to be painted, the required color must be indicated in the contract documents

Weathering steel should be specified in accordance with the guidelines provided in Section 8.2.1. Weathering steel tubes shall not be specified when the bridge is expected to remain open during the winter months and will be salted.

Deck Type:

The type and specific details of decking are required on the contract plans. The designer may choose from many types of decking including concrete, timber (glulam), steel grating, fiber reinforced polymer (FRP), or plastic (pvc or composite). Timber shall not be placed directly on weathering steel.

The recommended minimum depth for a concrete deck is 6”.

Width Requirements/Guidelines:

Horizontal clearance requirements shall be indicated on the contract plans, i.e. clear width between trusses/railing/curbing etc.

Deck widths should generally be greater than span/22 when overhead bracing is not allowed and span/30 when overhead bracing is allowed.

A minimum clear width of 10 feet is recommended for passage of emergency vehicles.

Deck widths for prefabricated structures greater than 14 feet should be avoided, as they require a longitudinal splice in the bridge for shipment.

Railing/Protective System:

Indicate railing height, material type (steel tube or wood) and type of finish.

Provide details for Horizontal safety rails, vertical pickets and protective fencing if required. When fencing is required provide: height, type (galvanized or epoxy-coated), color and maximum opening.

Indicate minimum 5” toe rails located no more than 2” above the deck.

Provide ADA compliant handrail when grade exceeds 5%.

Indicate end treatment and approach railing types.

Truss Accessories:

Bollards, ramps, stairs, lighting, signing, and utility hangers are examples of items that can be provided on a pedestrian bridge. Specific details that are required must be shown on the contract plans and the item(s) under which they are to be paid must be indicated.

Bollards should be used to limit vehicular traffic.