SECTION 7 - STRUCTURAL STEEL

7.1 - STRUCTURAL STEEL TYPES

Table 7.1 on the following page lists the current steel types allowed by the Thruway Authority for structural applications on bridges. Prices for these different steel types are generally similar but increase slightly with strength. Therefore, choosing a steel type for a particular structure should be based on durability, ease to maintain, and availability of material. While Grade 50 and 50 **Weathering (W)** steels are available in rolled shapes and plates, Grade 36, **High Performance Steel (HPS)** Grades 70**W** and 100**W** steels are only available in plates.

7.1.1 - COMBINATIONS OF DIFFERENT TYPES OF STRUCTURAL STEEL

In general, when more than one type of steel is used in one contract, the types used shall be clearly described in the plans. The payment for furnishing and placing these steels shall be made under the most appropriate and current structural steel items. When lump-sum item numbers are used, 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. When per-pound item numbers are used, a weight table is not required. Per-pound item numbers should be used under most circumstances in order to facilitate the tracking of steel prices.

STEEL SPECIFICATION TABLE

ASTM A709 & AASHTO M270 GRADE	**MIN. YIELD F_y (ksi)**	**MIN. TENSILE F_u (ksi)**
36	36	58
50	50	65
50**W**	50	70
HPS70**W***	70	85
HPS100**W****	100	110

TABLE 7.1

* **Thermo-Mechanical Control Process (TMCP)** Steel is a newer version of **HPS**70**W** that is not quenched and tempered.
It is preferred over **HPS**70**W** steel when available.

** This steel shall only be used with approval from the **DSD**.

7.2 - MINIMUM THICKNESS OF METAL

Structural steel (including lateral bracing, cross frames, diaphragms and all types of gusset plates), except for webs of certain rolled shapes, fillers and in railings, shall be not less than 3/8 inch thick. The web thickness of rolled beams, channels, or structural tees shall not be less than ¼ inch. Thicker dimensions should be considered to accommodate in service corrosion related section loss in areas below bridge joints, low overhead clearance, or where snow and ice are likely to accumulate. This pertains to both weathering and non-weathering steel types.

It should be noted that there are other provisions in this section pertaining to thickness for fillers, segments of compression members, gusset plates, etc.

For web plates, flanges, stiffeners and other plates, refer to "Plate Girders" in the **AASHTO 17th**

Edition and the **AASHTO LRFD 4th Edition**. For compression members, refer to "Trusses" in the **AASHTO 17th Edition and** the **AASHTO LRFD 4th Edition**. The Authority requires minimum plate thicknesses as follows:

- Girder Webs ≥ 1/2 inch thick
- Intermediate Stiffeners and Connection Plates ≥ 1/2 inch thick
- Bearing Stiffeners ≥ 1 inch thick
- Girder Flanges ≥ 1 inch thick
- Gusset Plates ≥ 3/8 inch thick

7.3 - CAMBER

The Contract Plans shall show the design cambers for steel girders, diaphragms/crossframes & formwork, concrete dead loads, superimposed dead load, and vertical curve, each separately, and the total of the above. Offsets from a straight line (end-to-end of member) shall be given at intervals of 22 feet, or one-tenth of the span length, whichever is less. With curved girders, offsets shall be given at diaphragm lines (see Subsection 7.7). The designer shall note that the camber required in individual girders may vary due to loading, particularly between sections of a stage construction project. Refer to Subsection 3.4.1 for guidelines on cambering structural steel on stage construction projects. Differing camber requirements can be expected between stages due to variances in the dead loads. These differences need to be accounted for to facilitate the connection of diaphragms or cross-frames between stages. **Camber shall be checked in the fabrication shop in the <u>vertical position under girder dead load only.</u> This requirement shall not be waived and shall be clearly noted**

on the plans in the Camber Notes on the Camber Table Sheet and in the Erection/Camber Monitoring Procedure Notes on the Framing Plan Sheet. Refer to Appendix "B" for these notes. There are two reasons for this **mandatory vertical camber check**. First, allowable tolerances in girder material sizes and overall girder depth dimensions as allowed by the **NYSSCM** may reduce or increase the amount of the theoretical girder dead load deflection. The no-load horizontal camber check typically done in the fabrication shop does not account for these variations that will ultimately increase or decrease the Moment of Inertia of the girder and reduce or increase the girder dead load deflection. Second, it is extremely important that the girders have the correct camber once erected. Since correcting camber requires that the girders be completely supported as per Section 15 of the **NYSSCM**, they cannot be corrected in the field without removal from the structure. This is a very expensive operation due to remobilization of cranes & traffic control and project delays.

7.3.1 - SAG CAMBERS

A. General

By definition, a girder is said to have sag (or negative) camber if any portion of 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. 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 superstructure transition length occurs or any horizontally curved superstructure supported on straight girders. The Authority's policy is to avoid sag camber on new bridge structures whenever possible. This policy is based on the fact that these girders are aesthetically objectionable to the public because of their unstable appearance. 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.

B. Avoiding Sag Camber

Designers may find that the approved geometrics have not considered the Authority's policy regarding sag cambers. If this condition exists, the designer shall use the following guidelines to minimize the effect or eliminate designing a sag cambered superstructure.

a. 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.

b. If a revision in 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 8 inches. In those cases where a deeper haunch is required, the 8 inch haunch shall be used in conjunction with a sag camber unless otherwise approved by the **DSD**.

7.4 - BOTTOM OF SLAB ELEVATIONS

Bottom of slab elevations shall be shown over each stringer at centerlines of bearings and at intervals of 22 feet or one-tenth of the span length, whichever is less. With curved girders, show at diaphragm lines (10 feet < L < 22 feet, where "L" is the maximum distance between haunch measurement locations).

7.5 - FLANGE THICKNESS AND WIDTH CHANGES

7.5.1 – FLANGE THICKNESS CHANGES

When designing a bridge plate girder, the stresses in the flanges vary greatly depending on the location within the spans. On simple span structures, the stress at the center of the span is at its maximum due to bending stresses. On continuous structures, the bending stresses in the flanges over intermediate pier supports are typically higher than between these supports. Figure 7.5 on Page 7-8 illustrates when it is economical to vary the thickness of flanges. These guidelines are based on the cost per pound of fabricated steel versus the cost of a full penetration groove weld butt splice that would be required to join two flange plates of different thickness. The longer distance a thinner plate can be used, the more economical it is to introduce a butt splice because of the increased weight savings. On simple spans, splices should be located at the first and/or third quarter points. On continuous structures, splices should be located at the deal load points of contraflexure. During girder design, the designer should look at the maximum stresses at these points, determine the required smaller plate size, and use Figure 7.5 to determine if transitioning the plate thickness is economical. When doing so, the designer shall remember that the maximum thickness transition at any joint between two flange plates shall not exceed a ratio of 1 to 2 and that the minimum flange plate thickness is 1 inch. The requirements of these welded butt splices are as follows: There shall be a smooth transitional slope between the offset thicknesses of welded butt splices of flanges. This slope shall not exceed 1 on 2.5. Refer to Detail C5-2 in Appendix C for a thickness transition detail.

7.5.2 – FLANGE WIDTH CHANGES

Flange width changes shall not be used to reduce weight as are thickness changes as described above. Width changes for this purpose complicates typical fabrication procedures and produces significant wasted material. Width transitions shall only be used on the bottom flange at the abutment bearings where required to facilitate the size of the bearing device used. In most cases, reducing the width of the bottom flange will be required at these locations. Width transition slopes shall not exceed 1 on 4. Refer to Detail C5-2 in Appendix C for a width transition at bearing detail.

7.6 - DESIGNATION OF TENSION ZONES

For other than simple spans, the Contract Plans shall clearly indicate the limits on the flanges of all stringers that are subject to tensile stresses. Tensile stress zones may be calculated from either combined stresses or moments (at 10th points or diaphragm locations, whichever govern, see Section 2 – Loads and Ratings). Linear interpolation may be used to locate boundaries of tension zones. The actual distance computed shall be rounded up to the next 6 inches. This shall be done to facilitate radiographic inspection and the control of welding during fabrication, erection and biennial inspections. This requirement shall apply to reconstruction projects which require new deck slabs, as well as to new structures.

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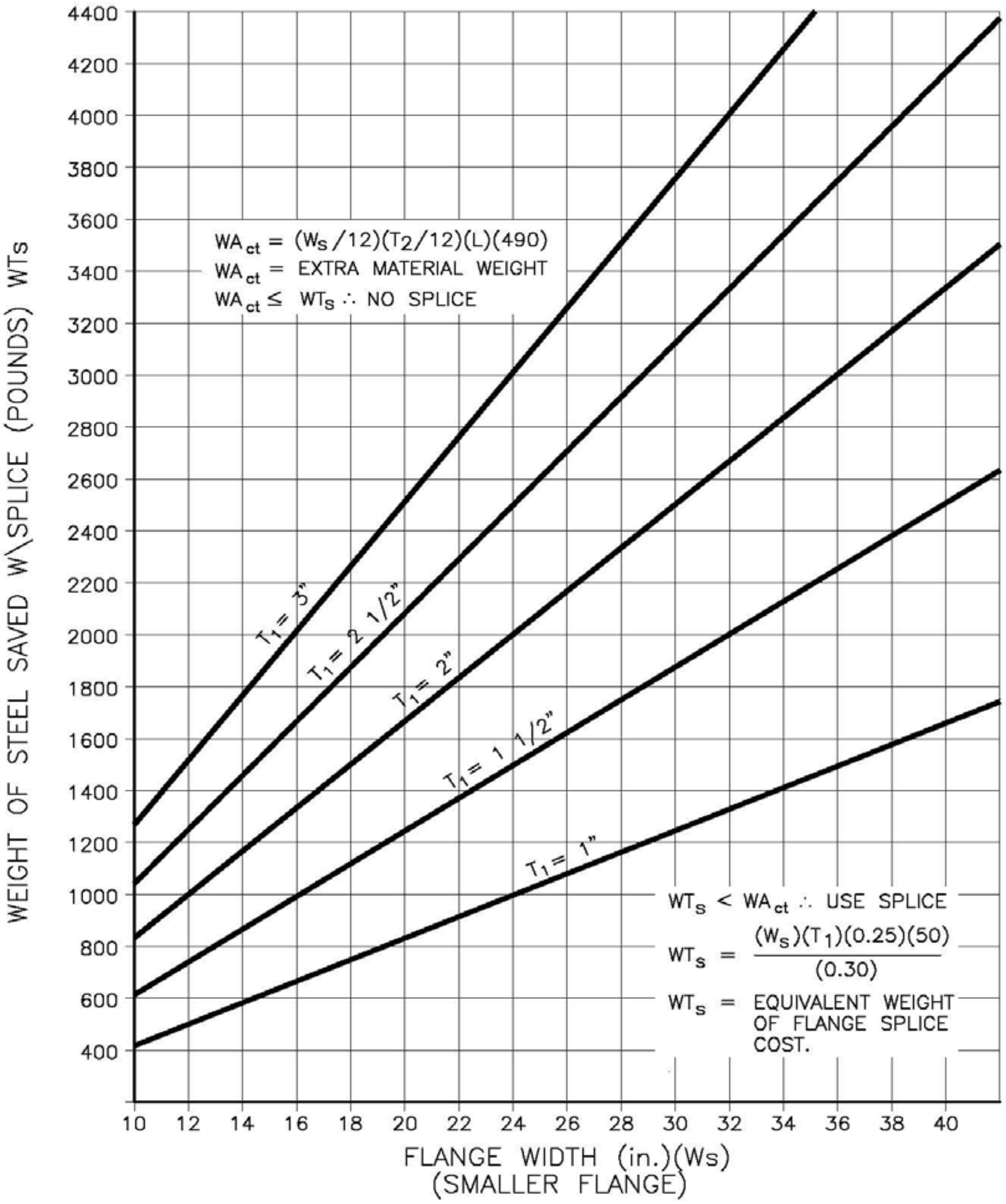


W_S = WIDTH OF SMALLER FLANGE (in.)

L = LENGTH OF PROPOSED FLANGE EXTENSION (FT.)

T_1 = THICKNESS OF SMALLER FLANGE (in.)

T_2 = THICKNESS DIFFERENCE BETWEEN FLANGES (IN).

ECONOMICAL USE OF BUTT WELDED SPLICES IN PLATE GIRDER FLANGES

FIGURE 7.5

7.7 – INTERMEDIATE TRANSVERSE STIFFENERS AND CONNECTION PLATES

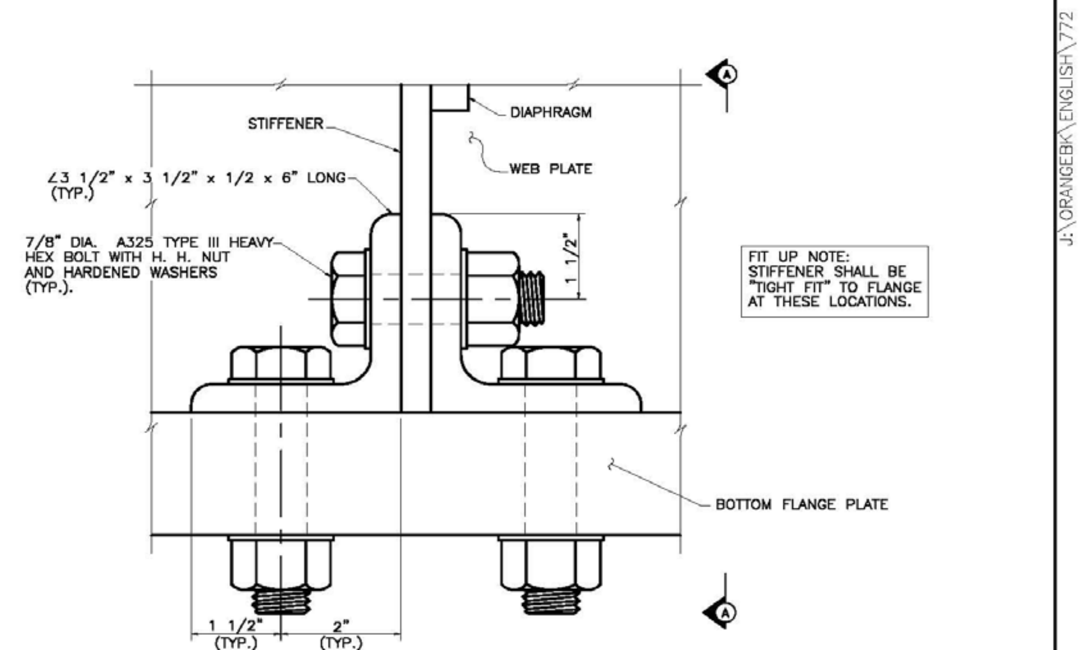
7.7.1 - GENERAL

On fascia beams and girders, intermediate transverse stiffeners shall be placed on the side of the web which is not exposed to view. On interior beams and girders, intermediate transverse stiffeners shall be located on both sides of the web, except where they are used in conjunction with a longitudinal stiffener on the other side of the web (Longitudinal stiffeners are not generally allowed. See Subsection 7.8). Transverse stiffeners shall be a minimum of 1/2 inch thick and 5 inches wide and shall be welded as described in Subsections 7.7.2 & 7.7.3 below with 5/16 inch fillet welds, regardless of base metal thickness. Intermediate transverse stiffeners not used as connection plates shall be placed perpendicular to the web. All intermediate transverse stiffeners and connection plates shall extend full height from the bottom flange to the top flange. Other fabrication details shall be in accordance with the **NYSSCM** except as modified herein. Intermediate transverse stiffeners used as connection plates shall be a minimum of 1/2 inch thick and 7 inches wide (for 2 lines of bolts) and shall be welded 100% as described in Subsections 7.7.2 & 7.7.3 below with 5/16 inch fillet welds, regardless of base metal thickness. Intermediate transverse stiffeners used as connection plates shall be placed on the skew up to 30°. For skews beyond 30°, Intermediate transverse stiffeners used as connection plates shall be placed perpendicular to the web. Connection plates shall be placed on the skew up to 30°. For skews beyond 30°, connection plates shall be placed perpendicular to the web. Between stages of stage construction bridges, it is recommended that the connection plate on one side remain blank (no holes drilled) from the fabricator. In most cases it is difficult to ensure that the two stages will line up perfectly for bolt installation. By leaving one side blank, the contractor can use the holes predrilled in the diaphragm/crossframe as a template to drill the holes in the connection plate at the time of installation.

7.7.2 - SIMPLE SPANS

The intermediate transverse stiffeners, intermediate transverse stiffeners used as connection plates, and connection plates on simply supported beams and girders shall consist of plates fillet welded to the web and to the flange which is in compression at that point under dead loading and superimposed dead loading. Intermediate transverse stiffeners used as connection plates and connection plates shall be rigidly connected to the flange in tension. The way in which the connection plate is attached to the flange in tension depends on the limiting factor in the design of the structure; strength, deflection, or fatigue life. If strength or deflection is the limiting factor, the connection plate may be attached to the flange with fillet welds. In the case where the fatigue life of the plate/flange weld is the limiting factor, the connection plates shall be bolted with angles to the tension flange. See Detail 7.7.2.

Intermediate stiffeners that are not used as connection plates need not be rigidly connected to the tension flange. In this case, the stiffener may be placed paint tight against the tension flange. Intermediate stiffeners and connection plates shall be placed either vertical, perpendicular to the flange or to a tangent to the flange at each location.



STIFFENER BOLTING DETAIL
FASCIA GIRDER ONLY

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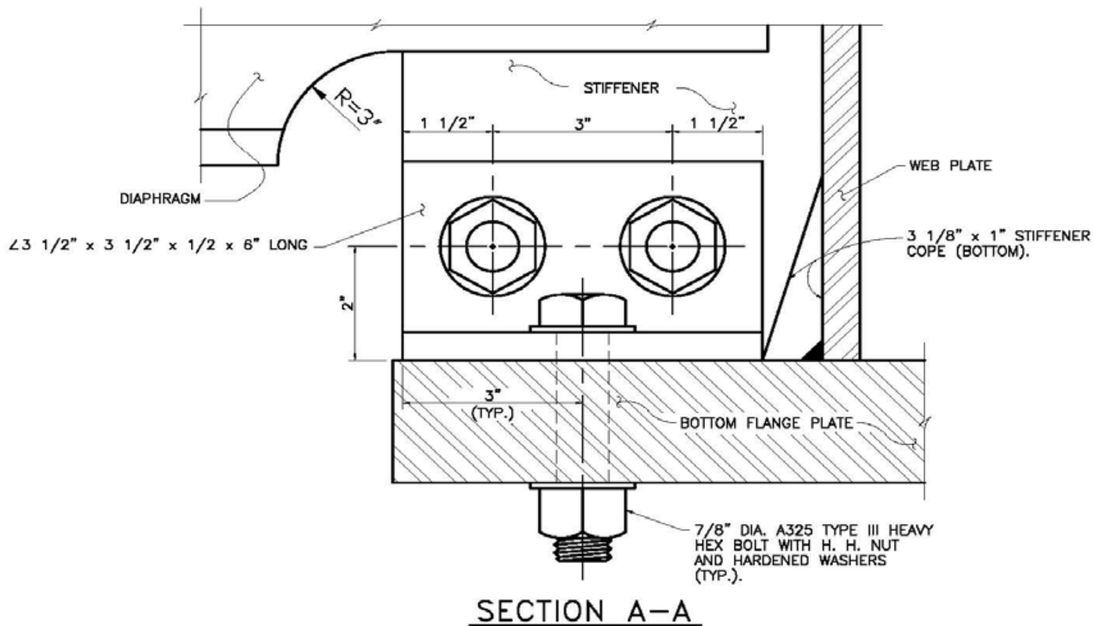

SECTION A–A

N.T.S.

DETAIL 7.7.2

7.7.3 - CONTINUOUS SPANS

The intermediate transverse stiffeners, intermediate transverse stiffeners used as connection plates, and connection plates on continuous spans shall be attached to the beam or girder web and compression flange as described for simply-supported beams and girders above. The intermediate transverse stiffeners used as connection plates and connection plates shall be attached to the flanges as described above, except in the live load stress reversal zones, where the top and bottom flanges shall both be treated as tension flanges.

7.8 - LONGITUDINAL STIFFENERS

Longitudinal stiffeners shall not be used on new structural steel plate girders unless approved by the **DSD**. When approved, longitudinal stiffeners shall be designed as per **AASHTO** requirements. On fascia girders, longitudinal stiffeners shall be placed on the exterior side of the web and be continuous for the full length required. On interior girders, longitudinal stiffeners shall be placed on one side of the web between intermediate transverse stiffeners, intermediate transverse stiffeners used as connection plates, and connection plates. Longitudinal stiffeners shall be continuous between these plates. Longitudinal stiffeners shall be a minimum of 1/2 inch thick and 5 inches wide and shall be fillet welded 100% on both sides with 5/16" minimum welds, regardless of base metal thickness.

7.9 - BEARING STIFFENERS

All bearing stiffeners on straight beams and girders shall be 100% fillet welded to the web and either welded to the bottom flange with a complete penetration groove weld or milled to bear against the bottom flange. On horizontally curved beams and girders, bearing stiffeners must be complete penetration groove welded to the bottom flange. Fascia bearing stiffeners and interior bearing stiffeners not used as connection plates shall be placed perpendicular to the web. Bearing stiffeners used as connection plates shall be placed on the skew up to 30°. For skews beyond 30°, the bearing stiffeners shall be placed perpendicular to the web.

All bearing stiffeners may be either fillet welded or placed paint tight against the top flange, except that where the top flange is in tension, the bearing stiffener must be placed paint tight against the top flange and not welded to the flange. When the bearing stiffener thickness is greater than 1 inch, partial penetration groove welds or fillet welds are recommended to weld the bearing stiffener to the flange so that flange distortion will be reduced. In this case, the stiffener must be "milled to bear" at the flange prior to welding. The ends of all beams and girders and all bearing stiffeners shall be vertical after dead load and superimposed dead load deflection. Alternate fabrication details may be submitted by the Contractor for approval by the Project Designer. Conformance to Authority requirements must be verified.

7.10 - DIAPHRAGMS & CROSS FRAMES

Diaphragms and cross frames shall be designed in accordance with Subsection 10.20 of the **AASHTO 17th Edition** and Subsection 6.7.4 of the **AASHTO LRFD 4th Edition**. The steel used should be the same grade as that used for the main members. The exception to this is when Grade

70**W** or greater steel is used for the main members. Rolled shapes are currently not available in these higher strength steels. In these cases, Grade 50**W** steel should be used for the diaphragms and cross frames. At bearing locations, diaphragms and cross frames shall be placed along the centerline of bearing, regardless of the skew angle. For skew angles above 30°, diaphragms and cross frames shall be connected to the bearing stiffener with a cold bent gusset plate to account for the skew angle. See Detail 7.10.a. At intermediate locations, diaphragms and cross frames shall be placed along the skew, regardless of the skew angle. For skew angles above 30°, diaphragms and cross frames shall be connected to the intermediate stiffener or connection plate with a cold bent gusset plate to account for the skew angle. See Detail 7.10.b. The use of diaphragms versus cross frames is governed by economics as well as geometrics. Diaphragms are heavier per linear foot but require much less fabrication than cross frames. Because of the high cost of fabrication, diaphragms, albeit heavier, are more economical than cross frames per linear foot. For this reason, diaphragms should always be used whenever possible. See the use restrictions that follow.

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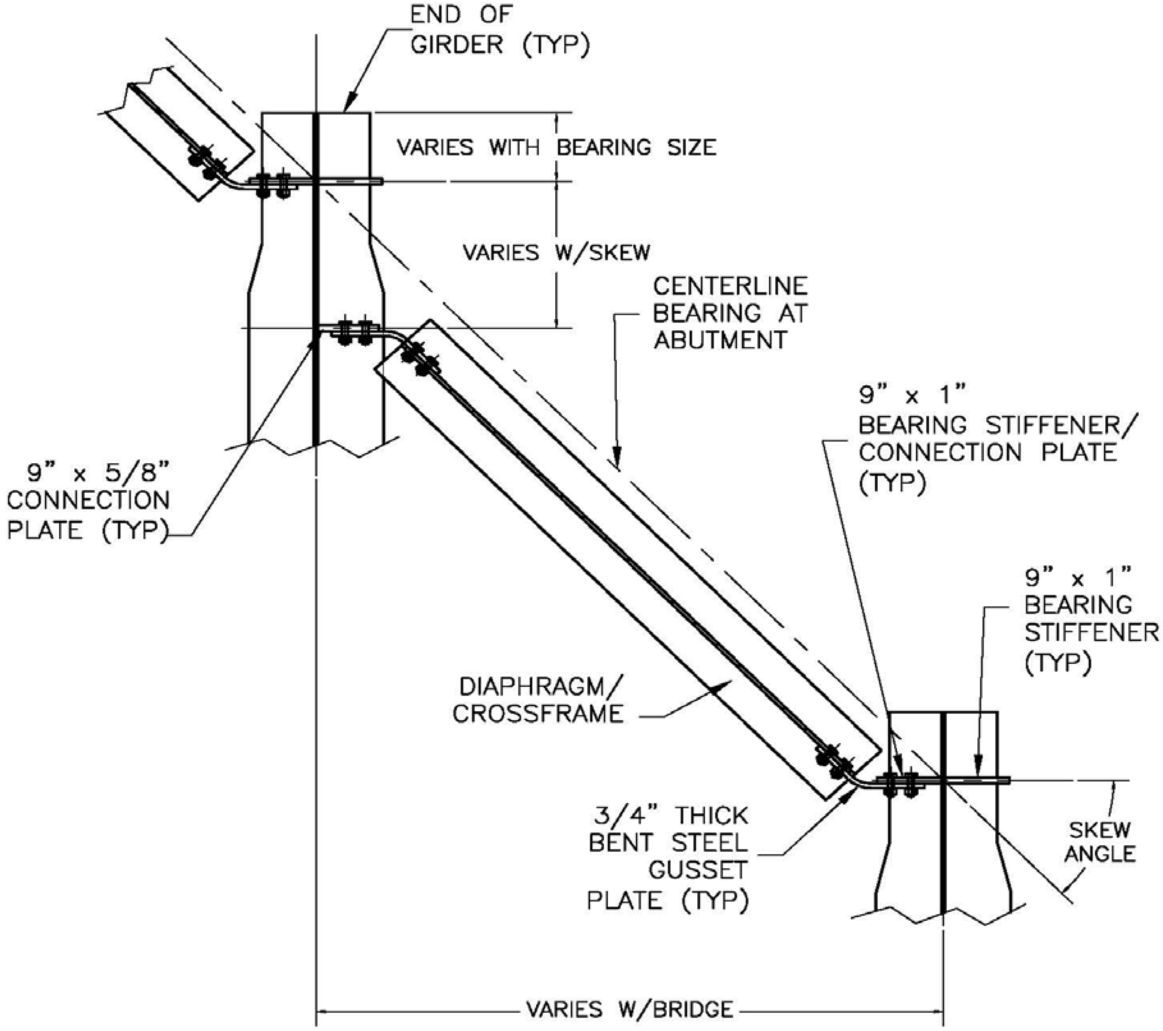
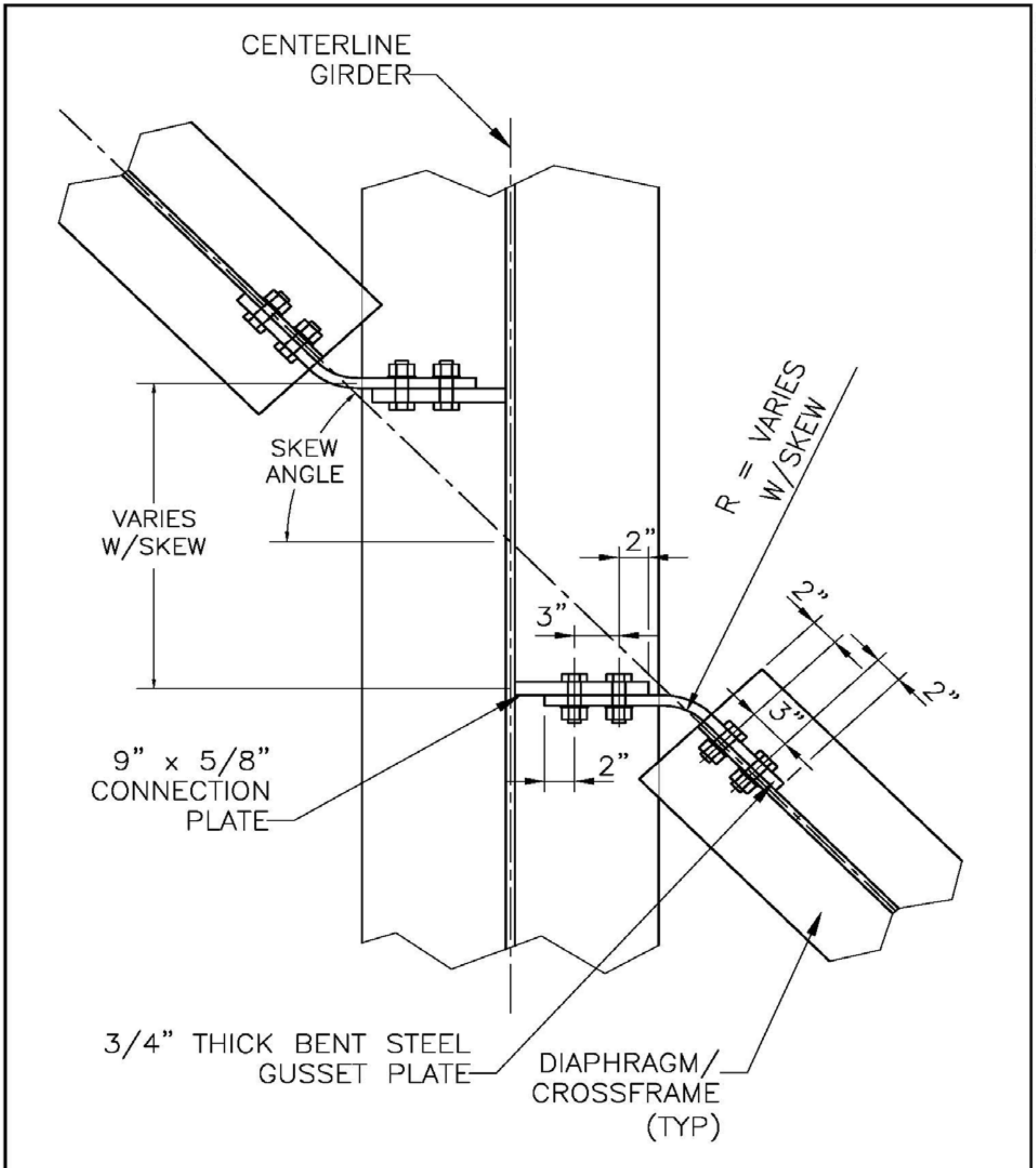


DETAIL 7.10.a
TYPICAL SKEWED DIAPHRAGM CONNECTION
AT ABUTMENT (PIER SIMILAR)
N.T.S.



DETAIL 7.10.b

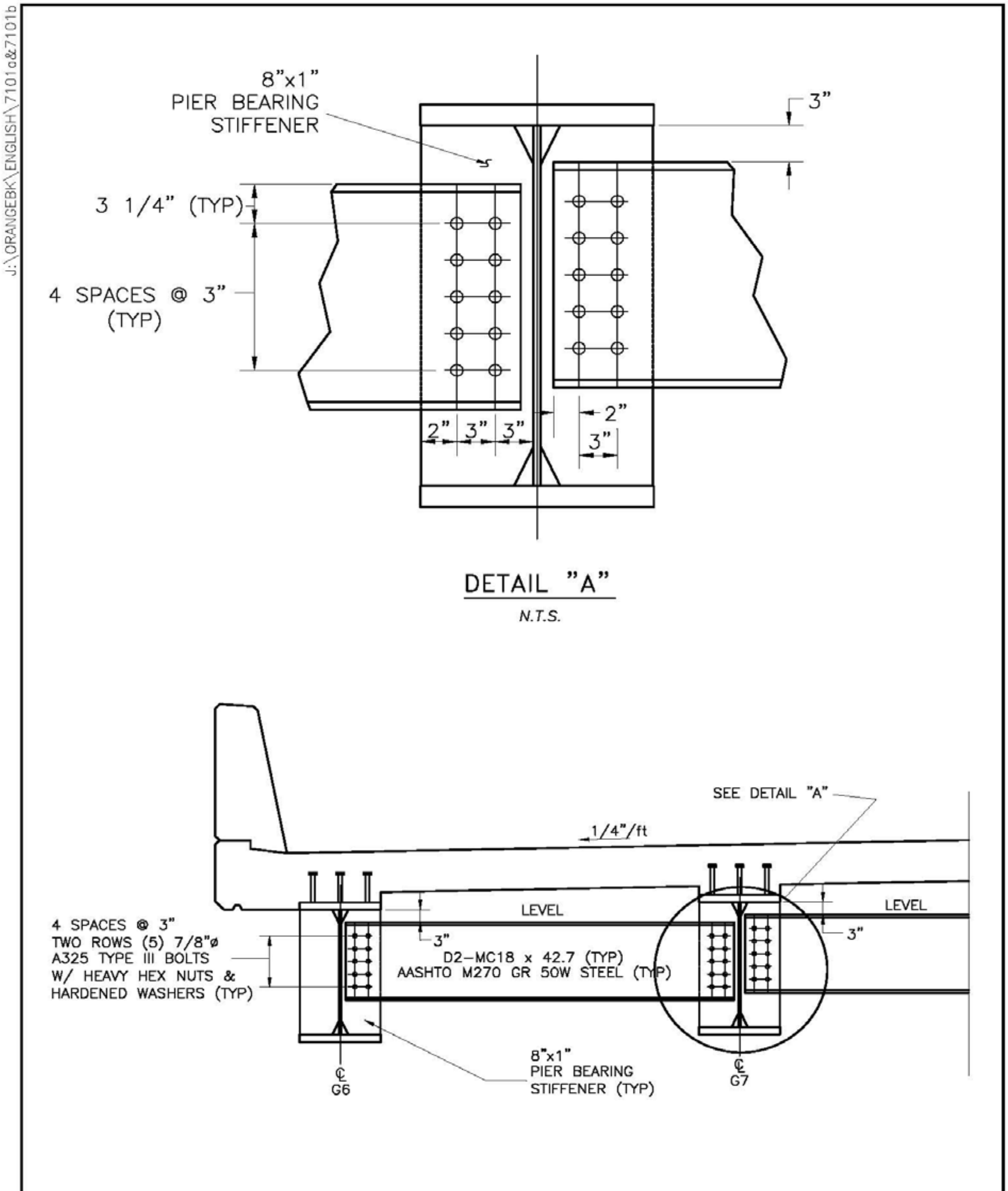
TYPICAL SKEWED DIAPHRAGM CONNECTION

INTERMEDIATE LOCATIONS

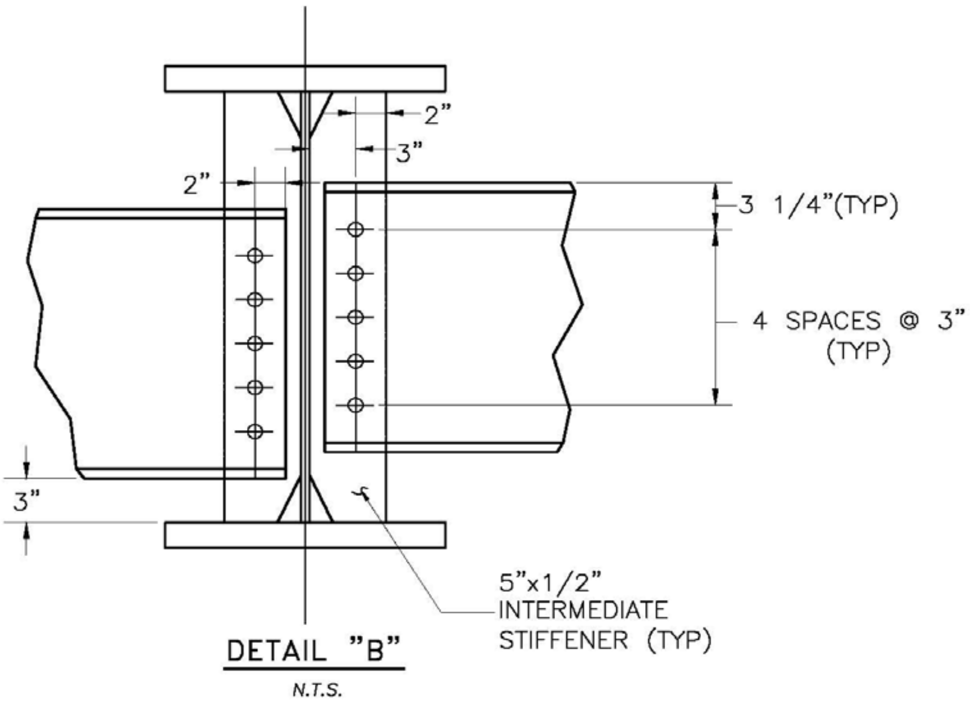
N.T.S.

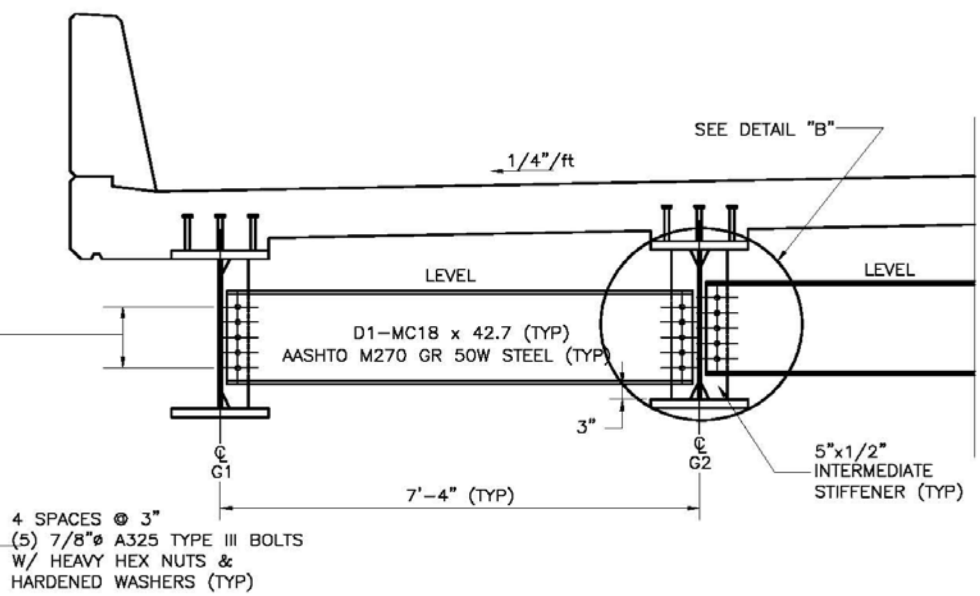
7.10.1 - DIAPHRAGMS

Diaphragms shall be made from standard **American Institute of Steel Construction (AISC)** rolled beam or channel sections. Beam sections must be coped at the ends to allow connection to the main members while channel sections do not. Diaphragms shall be used on all bridges with rolled beam main members. They should also be used on all plate girder bridges with a maximum web depth of 53 inches. Diaphragm depth shall be ¾ of the girder web depth where possible. At the bearings the diaphragms should be placed high enough on the bearing stiffener (3 inches± from the top flange) to allow for the installation and removal of the bearing anchor bolts. At intermediate locations, the diaphragms should be placed as low as possible on the stiffener/connection plate (3 inches± from the bottom flange). The placement of the diaphragm at this location on the connection plate will enable the main member to resist twisting caused by the torsional loading of fascia overhangs and differential girder loading during and after construction. Connection of the diaphragm to the connection plate should be with no less than one line of ¾ inch diameter **ASTM** A325 bolts and checked for anticipated loading. See Details 7.10.1a and 7.10.1b on the following pages.



DETAIL 7.10.1a
TYPICAL DIAPHRAGM CONNECTION
AT BEARINGS
N.T.S.





DETAIL 7.10.1b

TYPICAL DIAPHRAGM CONNECTION

INTERMEDIATE LOCATIONS

N.T.S.

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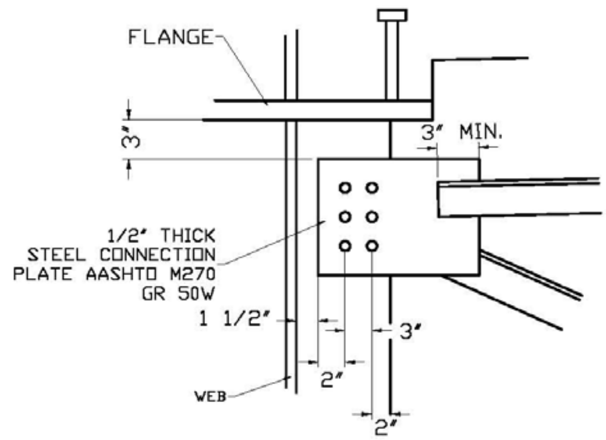
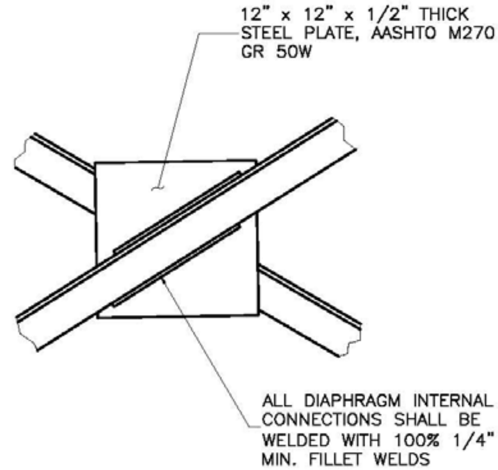
7.10.2 - CROSS FRAMES

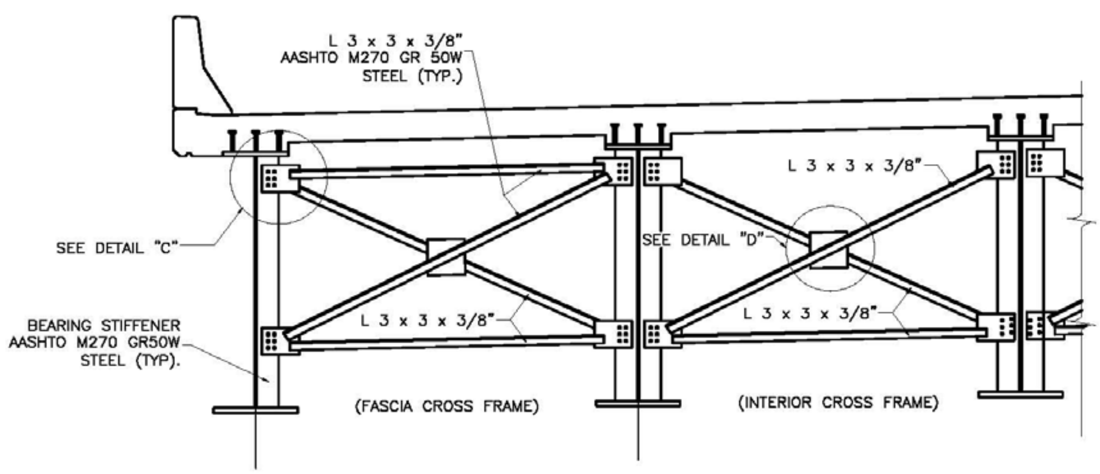
Cross frames shall be designed for the anticipated loading and made from a combination of standard **AISC** rolled channel and/or angle shapes and plates welded together in an “X” or “K” (“V” in **AASHTO**) configuration. The use of “X” versus “K” frames is dependent on height to width ratios as well as loading. Consult the appropriate **BD** sheet for recommendations. Cross frames shall only be used on bridges with plate girder main members with a minimum web depth over 53 inches. At bearing stiffener, the cross frame depth should be ¾ of the girder web depth and place 3 inches± from top flange. This will allow for the installation and removal of the bearing anchor bolts. At intermediate locations, the cross frames should be full height excluding 3 inches± from top & bottom flange. The full depth cross frames will enable the main member to resist the twisting mentioned above. Connection of the cross frames to the connection plate should be with no less than two lines of ¾ inch diameter **ASTM** A325 bolts at each of the four corners and checked for anticipated loading. See Details 7.10.2a and 7.10.2b on the following pages.



DETAIL C
CONNECTION DETAIL
N.T.S.



DETAIL D
CONNECTION DETAIL
N.T.S.



DETAIL 7.10.2a
CROSS FRAME DETAILS
AT BEARINGS
N.T.S

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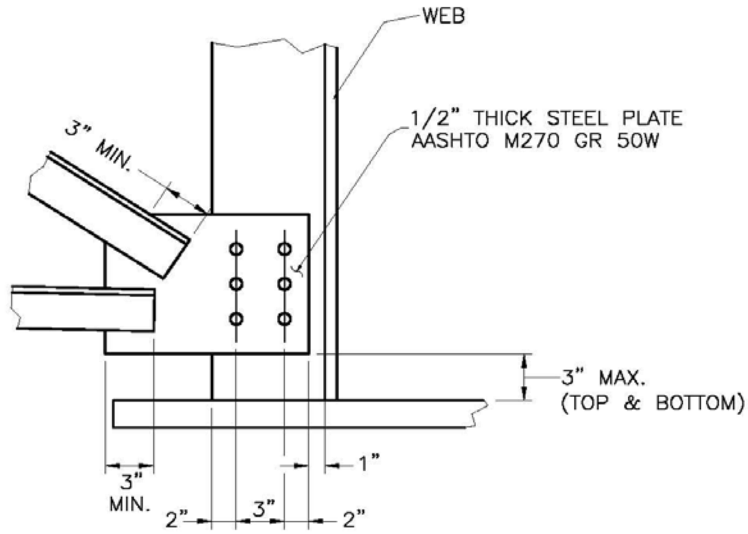
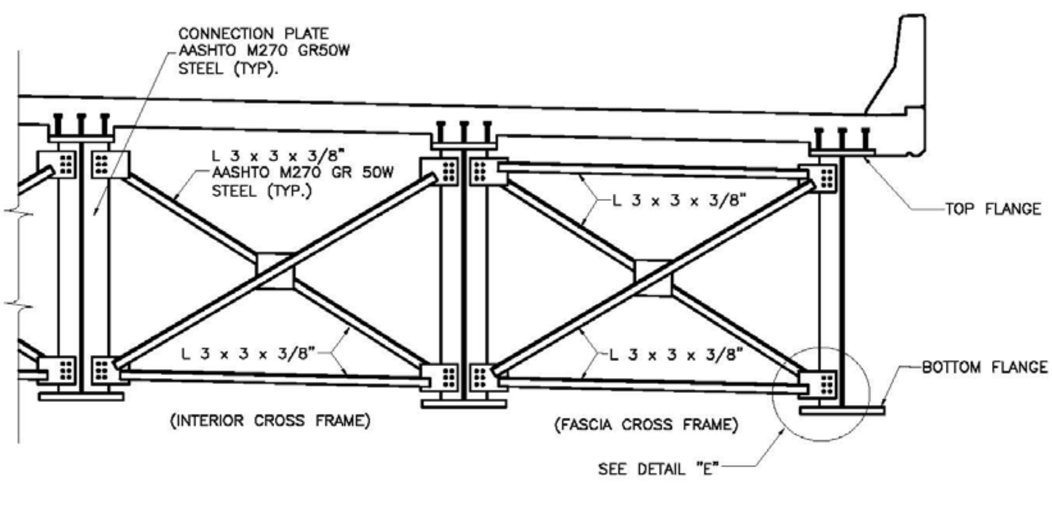


DETAIL E
CONNECTION DETAIL
N.T.S.



DETAIL 7.10.2b
CROSS FRAME DETAILS
AT INTERMEDIATE LOCATIONS
N.T.S.

7.11 - LATERAL BRACING

Lateral bracing, when required, shall be designed in accordance with Subsection 10.21 of the **AASHTO 17th Edition** and Subsection 6.7.5 of the **AASHTO LRFD 4th Edition**. Lateral bracing is required when the intermediate diaphragms or cross bracing is insufficient in supporting the bottom flange from resisting transverse loading (i.e. wind or temporary overhang brackets used to form the deck overhang concrete). Lateral bracing shall be connected to the bottom flange with plates bolted through the flange. Lateral connection plates shall be located such that they will not interfere with connections at bearing and intermediate stiffeners/connection plates. Other connection plate details shall be as shown on the appropriate **BD** Sheets.

7.12 - COVER PLATES

The use of cover plates welded to rolled beams on bridges is discouraged. The increased fabrication costs plus future maintenance, inspection, and fatigue life problems have rendered cover plate attachment to a rolled section uneconomical as an alternative to welded plate girders. If a standard rolled beam section cannot carry the design loads, the designer shall design a welded plate girder section in its place.

In the case of existing beams that have cover plates, the ends of the cover plate shall be retrofitted via end bolting as detailed on the Thruway Standard Sheet. As shown on the sheet, the existing transverse weld(s) connecting the ends of the cover plate to the flange shall be ground off smooth and flush with the existing steel surfaces.

7.13 - SAFETY HANDRAILS

Safety handrails for use during bridge inspection shall be installed on plate girders having any portion of its depth equal to 4 feet or more. In the case of tapered or fish-bellied girders with only a portion of its depth at 4 feet or greater, the entire length of the girder string shall be fitted with safety handrails. Handrails shall be placed on both sides of interior girders and on the interior side only of fascia girders. Details of field-erected and shop-erected handrails are available on the appropriate **BD** Sheet. The cost of handrails shall be included under the structural steel item.

7.14 - DRIP BARS

Drip bars shall be placed on the top and bottom of the bottom flange of all girders on the low end preceding a substructure pedestal or integral abutment face (minimum clearance of 3 feet to face of substructure). Drip bars shall be used to prevent water and salt damage to bridge bearings and substructure concrete. See Standard Detail drawing in Appendix C.

7.14.1 - DRIP BAR FASTENERS

Drip bars shall be bolted to the bottom flange of plate girders and rolled beams with ½ inch minimum diameter **ASTM** A325 bolts with heavy hex nuts and hardened washers (see Subsection 7.22.4).

7.14.2 - DRIP BAR SEALING

Drip bars shall be caulked to prevent water penetration between bars and girder surfaces.

7.15 - WIDTHS OF OUTSTANDING LEGS OF ANGLES

The widths of outstanding legs of angles in compression (except where reinforced by plates) shall not exceed the following:

1. In main members carrying axial stress, 12 times the thickness.
2. In bracing and other secondary members, 16 times the thickness.

For other limitations, see "Truss Compression Members", in the **AASHTO 17th Edition** and the **AASHTO LRFD 4th Edition**.

7.16 - COPES FOR FRAMED CONNECTIONS

When the top or bottom of a beam is coped for a depth of 6 inches or more, a flange plate of the same width and thickness as the adjacent flanges shall be welded to the coped portion of the web and the flanges. The minimum radius of the cope shall be 6 inches.

7.17 - MINIMUM SIZE OF FILLET WELDS

The minimum fillet weld size shall be as shown in the following table:

MINIMUM FILLET WELD SIZES	
BASE METAL THICKNESS OF THICKER PART JOINED (T)	**MINIMUM SIZE FILLET WELD ****
	BRIDGES
T ≤ ¾ inch	¼ inch*
T > ¾ inch	5/16 inch*

TABLE 7.17

* Single pass welds must be used. The minimum seal weld shall be a ¼ inch fillet weld.

** Weld size is minimum. Designers must perform the necessary analysis to determine the actual weld size required to satisfy the design criteria.

Notes:
1. The weld size need not exceed the thickness of the thinner part joined.
2. The weld size need not exceed 5/16 inch for the transverse stiffener to compression flange weld.

7.18 - FASTENERS

Fasteners shall be designed and detailed in accordance with the following sections:

7.18.1 - SIZE OF FASTENERS (HIGH STRENGTH BOLTS)

Fasteners shall be of the size required by design, but generally shall be 3/4 inch or 7/8 inch in diameter. Fasteners 5/8 inch in diameter shall not be used in members carrying calculated stress except in 2 ½ inch legs of angles and flanges of sections whose dimensions require 5/8 inch diameter fasteners to satisfy other detailing provisions below. 1/2 inch diameter fasteners shall be used to connect drip bars as stated in Subsection 7.14.1. All bolts shall be **ASTM** A325 Type I (**ASTM** A490 Type I bolts may only be used with approval of the **DSD**). All nuts shall be **ASTM** A563 or

A194. All washers shall be **ASTM** F436. The diameter of fasteners in angles carrying calculated stress (primary member) shall not exceed 25% the width of the leg in which they are placed. In angles whose size is not determined by calculated stress (secondary member), 5/8 inch diameter fasteners may be used in 2 inch legs, 3/4 inch diameter in 2 ½ inch legs, 7/8 inch diameter fasteners in 3 inch legs, and 1 inch diameter fasteners in 3 ½ inch legs. Structural shapes which do not permit the use of 5/8 inch diameter fasteners shall not be used except in handrails.

7.18.2 - SPACING OF FASTENERS

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 maximum pitch of fasteners shall be governed by the **AASHTO** requirements for sealing. The minimum distance between centers of fasteners shall not be less than the values shown in the following table:

FASTENER DIAMETER	MINIMUM SPACING
1 1/8 inch	4 inches
1 inch	3 1/2 inches
7/8 inch	3 inches
3/4 inch	2 1/2 inches
5/8 inch	2 1/4 inches

TABLE 7.18.2

7.18.3 - MAXIMUM SPACING OF FASTENERS

For sealing, the maximum spacing of fasteners along the free edge of a plate shall be 4 inches plus four times the thickness of the thinner plate, but not more than 7 inches.

7.18.4 - EDGE DISTANCE OF FASTENERS

The minimum distance from the center of any fastener to the edge of a sheared or flame-cut plate and in the flanges or legs of rolled sections shall not be less than those shown in the following table:

FASTENER DIAMETER	EDGE DISTANCE SHEARED/FLAME-CUT PLATE	EDGE DISTANCE FLANGES OR LEGS OF ROLLED SECTIONS
1 1/8 inches	2 inches	1 1/2 inches
1 inch	1 3/4 inches	1 1/4 inches
7/8 inch	1 1/2 inches	1 1/8 inch
3/4 inch	1 1/4 inches	1 inch
5/8 inch	1 1/8 inches	7/8 inch

TABLE 7.18.4

The maximum distance from any edge shall be eight times the thickness of the thinnest outside plate or section but shall not exceed 5 inches. In bearing-type connections having no more than two lines of fasteners in a line parallel to the direction of the stress, the distance between the center of the nearest fastener and that end of the connected member towards which the pressure from the fastener is directed, shall be as specified in Subsection 10.24.6.2 of the **AASHTO 17th Edition** and Subsection 6.13.2.6 of the **AASHTO LRFD 4th Edition**.

7.19 - FIELD SPLICES

Field splices are sometimes required on long simple spans or on continuous spans. When required, they shall be designed and detailed in accordance with the following sections.

7.19.1 - WELDED FIELD SPLICES

No welded field splices of rolled beams or plate girders shall be permitted.

7.19.2 - BOLTED FIELD SPLICES

Bolted field splices shall be used when plate girder lengths exceed 160 feet or when delivery limitations exist. Rolled beam lengths shall be subject to availability from the individual mills. Bolted splices shall be located at or near the point of dead load contraflexure on continuous spans whenever possible. Bolted splices shall be located at or near the first and/or third quarter points on simply supported spans whenever possible.

7.19.2.1 - DESIGN CRITERIA

Bolted field splices shall be designed as "slip critical" as defined in **AASHTO 17th Edition**, Subsection 10.24.1.4 with allowable stresses as shown in **AASHTO 17th Edition** Table 10.32.3C and **AASHTO LRFD 4th Edition**, Subsection 6.13. Bolted splice fasteners shall also be checked in bearing with allowable stresses as shown in **AASHTO 17th Edition**, Table 10.32.3B. The surface classification of new steel (blast cleaned and primed) shall be Class "B". The surface classification of existing steel (mill scale or paint) shall be Class "A". Where connections are subject to linear loads (flange splices), the formula given in **AASHTO 17th Edition**, Subsection 10.32.3.2.1. shall be checked. Include a note on the contract drawings stating whether a Class "A" or Class "B" surface preparation is required by design.

7.19.2.2 - SPLICE DETAILS

Bolted splice details shall be included in the proposed contract drawings and in conformance with **AASHTO 17th Edition**, Subsection 10.18 and **AASHTO LRFD 4th Edition**, Subsection 6.13.

7.19.2.3 - FILLER PLATES

Filler plates in bolted splices shall be allowed where the splice is at a section property change only if provisions of applicable subsections of **AASHTO 17th Edition**, Subsection 10.18 and **AASHTO LRFD 4th Edition**, Subsection 6.13 have been satisfied and a Class "B" surface treatment on all contact surfaces is specified (the Class "B" surface is standard on blast cleaned and/or zinc primed surfaces of shop painted structural steel). No minimum thickness for filler plates exists. However, it is recommended that for new construction, filler plates should be a minimum of 1/8 inch thick. Filler plates may be used on existing beams with a "Class A" surface if the splice is located at an intermediate support.

7.20 - BEARINGS WELDED TO BEAMS OR GIRDERS

A note shall be placed on the Plans indicating that the bearings are to be field welded to beams or girders and that the cost of field welding shall be included in the unit prices bid for the structural steel or bearing items. Weld size shall be as shown on the appropriate Thruway Structures Standard Sheet or as required by Design (whichever is greater).

7.21 - SHEAR CONNECTORS

In order to account for the new two-course and monolithic bridge decks with the associated new reinforcement depths, the following guidelines shall be used when sizing shear connectors on new and rehabilitated bridge decks.

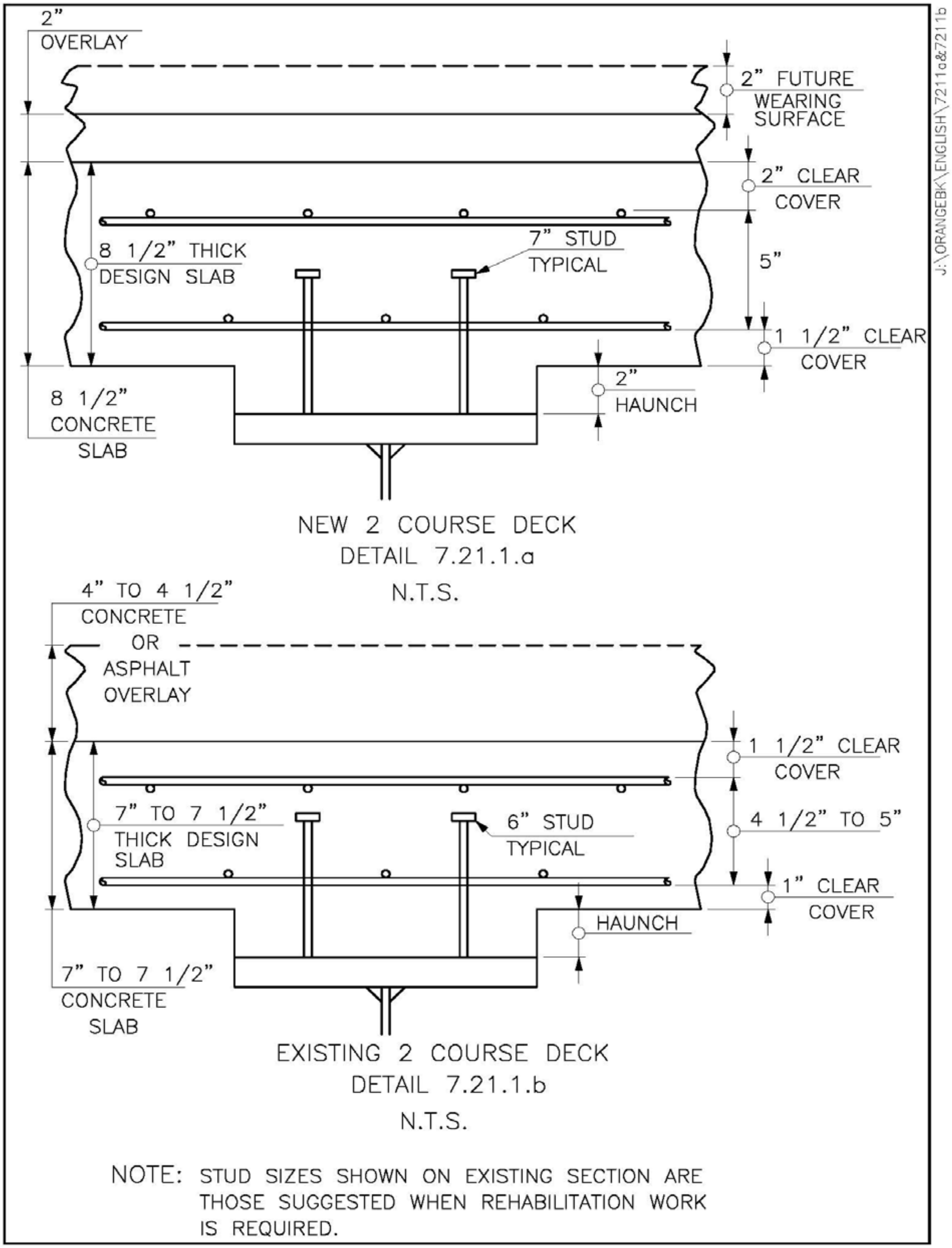
7.21.1 - SHEAR CONNECTOR LENGTH

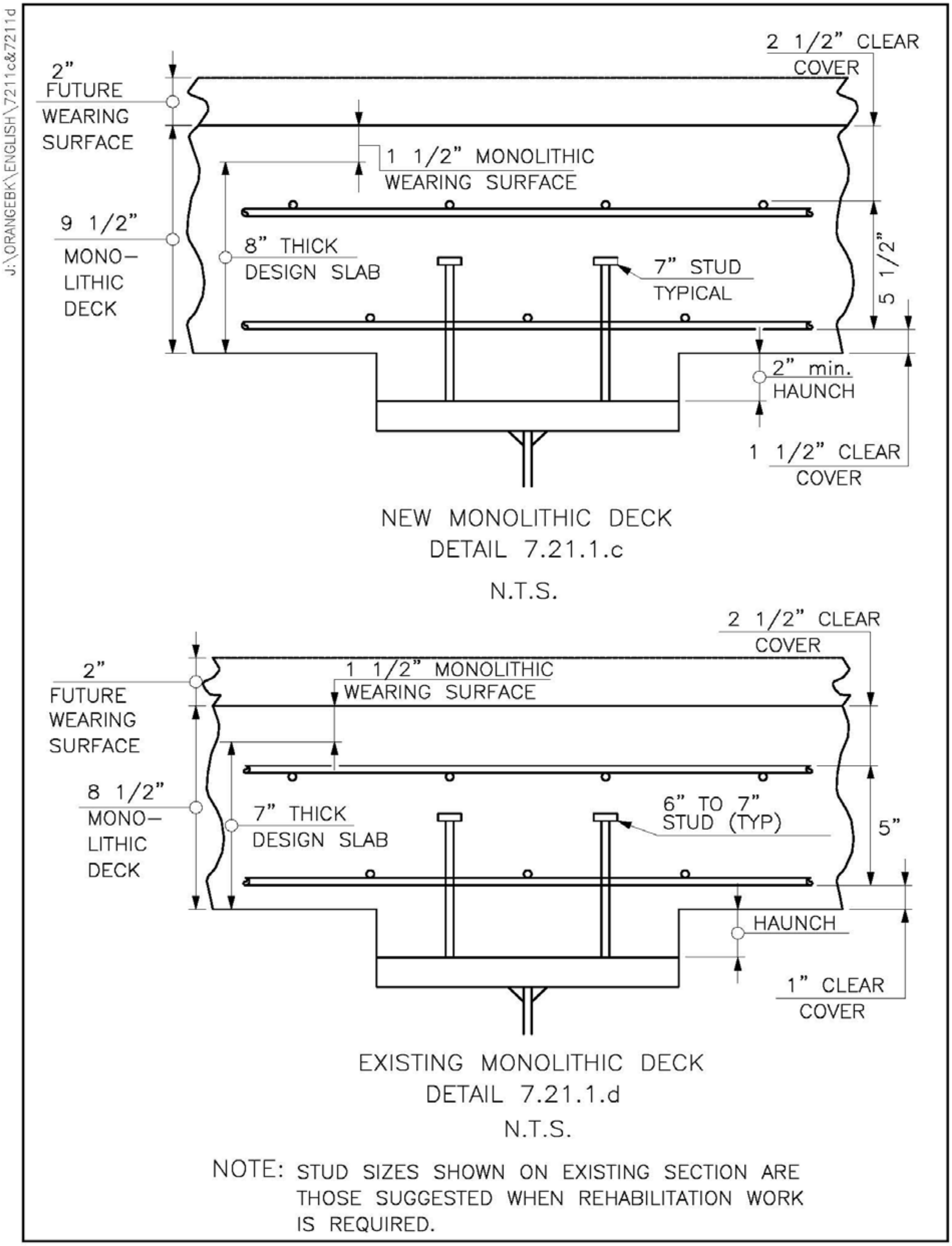
Stud shear connector length is illustrated in Details 7.21.1.a through 7.21.1.d and shall be governed by the following table:

STUD SHEAR CONNECTOR LENGTH		
DECK THICKNESS	**HAUNCH DEPTH**	**SHEAR STUD LENGTH**
8 1/2 to 9 1/2 inches	1 inch and less *	6 inches
	>1 to 4 inches	7 inches
	>4 inches	4 inches w/haunch reinf.
Less than 8 1/2 inches	1 inch and less *	4 inches
	>1 to 4 inches	6 inches
	>4 inches	4 inches w/haunch reinf.

TABLE 7.21.1

* Haunches less than or equal to 1 inch shall only be used when required by partial replacements and repairs of existing bridge decks.


2"
OVERLAY
2" FUTURE
WEARING
SURFACE
2" CLEAR
COVER
8 1/2" THICK
DESIGN SLAB
7" STUD
TYPICAL
5"
1 1/2" CLEAR
COVER
2"
HAUNCH
8 1/2"
CONCRETE
SLAB
NEW 2 COURSE DECK
DETAIL 7.21.1.a
N.T.S.
4" TO 4 1/2"
CONCRETE
OR
ASPHALT
OVERLAY
1 1/2" CLEAR
COVER
7" TO 7 1/2"
THICK DESIGN
SLAB
6" STUD
TYPICAL
4 1/2" TO 5"
1" CLEAR
COVER
HAUNCH
7" TO 7 1/2"
CONCRETE
SLAB
EXISTING 2 COURSE DECK
DETAIL 7.21.1.b
N.T.S.
NOTE: STUD SIZES SHOWN ON EXISTING SECTION ARE
THOSE SUGGESTED WHEN REHABILITATION WORK
IS REQUIRED.
J:\ORANGEBK\ENGLISH\7211a&7211b


J:\ORANGEBK\ENGLISH\7211c&7211d
2 1/2" CLEAR COVER
2" FUTURE WEARING SURFACE
1 1/2" MONOLITHIC WEARING SURFACE
9 1/2" MONO-LITHIC DECK
8" THICK DESIGN SLAB
7" STUD TYPICAL
5 1/2"
2" min. HAUNCH
1 1/2" CLEAR COVER
NEW MONOLITHIC DECK
DETAIL 7.21.1.c
N.T.S.
2 1/2" CLEAR COVER
2" FUTURE WEARING SURFACE
1 1/2" MONOLITHIC WEARING SURFACE
8 1/2" MONO-LITHIC DECK
7" THICK DESIGN SLAB
6" TO 7" STUD (TYP)
5"
HAUNCH
1" CLEAR COVER
EXISTING MONOLITHIC DECK
DETAIL 7.21.1.d
N.T.S.
NOTE: STUD SIZES SHOWN ON EXISTING SECTION ARE THOSE SUGGESTED WHEN REHABILITATION WORK IS REQUIRED.

7.21.2 - MINIMUM COVER

The minimum clear depth of cover over shear connectors shall be the same as required for the top mat reinforcing. Shear connectors shall penetrate at least to the top of the bottom mat of reinforcement in the deck unless haunch reinforcement is used.

7.21.3 - SHEAR CONNECTOR DIAMETER

The diameter of the studs is dependent on the required strength for design per connector. Generally two to three rows of studs will be either 3/4 inch or 7/8 inch diameter.

7.21.4 - SHEAR CONNECTOR PLACEMENT

Shear connectors are placed in the positive moment areas of simply supported, continuous, and rigid frame structures as required to insure composite action with the compressive deck concrete. Shear connectors shall also be required in the negative moment regions of all continuous and rigid frame structures. Longitudinal reinforcement in the slab shall be designed as composite with the girder in the tension zone.

7.21.5 - DESIGN METHODS

Shear connectors shall be designed for fatigue and checked for ultimate strength requirements as described in the **AASHTO 17th Edition**, Subsection 10.38 and the **AASHTO LRFD 4th Edition**, Subsection 6.10.10.

7.21.6 - SIZING LIMITATION

Only one size (diameter and length) shear connector shall be used on a bridge if at all possible.

7.21.7 - SHEAR CONNECTOR SPACING AND EDGE DISTANCE

Shear studs shall be placed as required by design. The longitudinal spacing between studs shall be constant within each group of studs placed. This constant spacing is determined by that required at the location of maximum shear (i.e. at abutment or live load stress reversal zone). Varied spacing may be considered only when a significant cost saving or an unusually small spacing (less than 4 inches) would make bar reinforcement placement extremely difficult. The number of rows used shall be controlled by the following spacing requirements:

1. Minimum spacing between studs must be greater than or equal to 4 stud diameters center-to-center.
2. Maximum spacing between studs must be less than or equal to 2 feet center-to-center.
3. The minimum edge distance from the edge of flange to the edge of the stud shall be 2 inches.

7.22 - UNPAINTED WEATHERING STEEL

Unpainted weathering steel (**ASTM** A709 50**W**, 70**W**) may be used for the superstructure of new bridges and superstructure replacement projects dependant on conformance with the criteria set forth in the following sections.

7.22.1 - EXCLUSIONS TO USE OF WEATHERING STEEL

Any one of the following situations shall prohibit the use of weathering steel:

A. Marine (salt water) environments

B. Low clearance over water (normal water elevation)

   a. less than 8 feet over standing water

   b. less than 4 feet over running water

C. Corrosive environment (i.e.: densely urban manufacturing area, highly humid areas)

D. Depressed highway in confined environment.

E. Replacement of individual components of original non-weathering steel structures unless they will be painted to match the existing bridge.

7.22.2 - USE OF WEATHERING STEEL

The following conditions shall both be satisfied if weathering steel is to be used:

A. No portion of the girders shall be buried or otherwise subjected to long periods of wetness.

B. All surfaces shall be kept clean of any oil, dirt, grease, mill scale, or any other substance that would create non-uniform corrosion of the surface of the steel.

7.22.3 - FASTENERS FOR WEATHERING STEEL

All bolts used with weathering steel shall be **ASTM** A325 Type III weathering bolts (A490 Type III bolts may only be used when approved by the **DSD**). All nuts used with weathering steel shall be **ASTM** A563 or A194, Grade C3 or DH3. All washers used with weathering steel shall be F436.

7.22.4 - DRIP BAR FASTENERS - WEATHERING STEEL APPLICATION

Drip bars shall be bolted to the bottom flange with ½ inch diameter **ASTM** A325 Type III bolts. Nuts and washers shall meet the specifications of Subsection 7.22.3 (Refer to Subsection 7.14 for additional information on drip bars).

7.22.5 - ADDITIONAL STEEL THICKNESSES

1/16 inch shall be added to the design thickness of all plates for the flanges and webs of plate girders to account for the initial corrosion of the weathering steel. This additional 1/16 inch is not required on rolled shapes (i.e. rolled beams, angles and channels).

7.22.6 - STRUCTURAL STEEL PAINTING AT EXPANSION JOINTS

Expansion joint use on weathering steel bridges shall be avoided. When expansion joints are required, the structural steel shall be painted out 5 feet± from the end of steel (at a joint) each side of the joint. A durable paint system that blends well in color to that of the weathered steel shall be used.

7.22.7 - SPECIAL NOTES – See Appendix B.