CASTCONNEX®

HIGH STRENGTH CONNECTOR™ (HSC) MANUAL

For Use With:

ANSI/AISC 341-22
ANSI/AISC 360-22

CAST CONNEX®

High Strength Connector™ Design Manual

for use with

ANSI/AISC 341-22 and ANSI/AISC 360-22

Cast Connex Corporation

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Cover photograph by Yi Yang of Summit Engineering, Santa Rosa, California.

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innovative components for inspired designs

We simplify the design and enhance the performance of structures by enabling Architects and Engineers to use cast steel connections. We take pride in Collaborating in the creation of safer, innovative, and more beautiful built environments.

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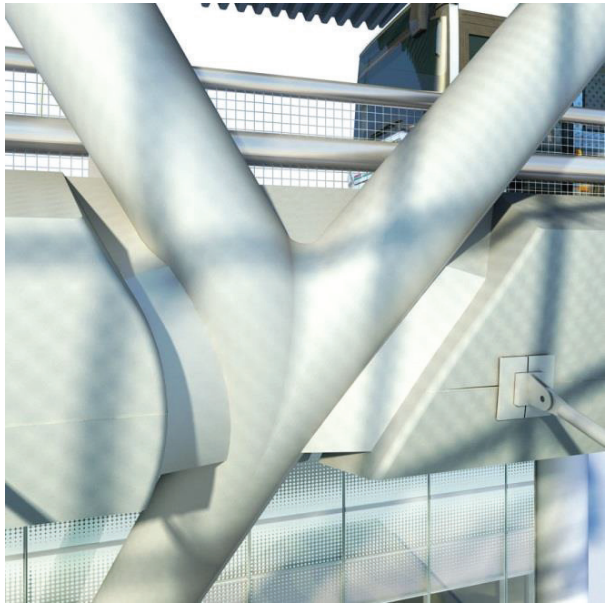


About High Strength Connectors

Concentrically braced frames are amongst the most popular lateral force resisting systems for medium- to low-rise steel structures. In the event of an earthquake, the diagonal brace members of braced frames dissipate seismic energy through yielding in tension and inelastic buckling in compression. This cyclic yielding and buckling imparts arduous loading on the brace's connections. Consequently, steel design codes require that seismic bracing connections be detailed such that they are significantly stronger than the nominal cross-sectional capacity of the brace member. The degree to which the connection strength must surpass the nominal cross-sectional yield capacity of the brace is dependent on the expected overstrength of the brace. Detailing connections to provide this strength can be rather difficult, particularly when dealing with hollow structural section (HSS) members, which are the preferred bracing elements due to their efficiency in carrying compressive loads, their improved aesthetic appearance, and the wide range of sections sizes that are readily available in the United States.

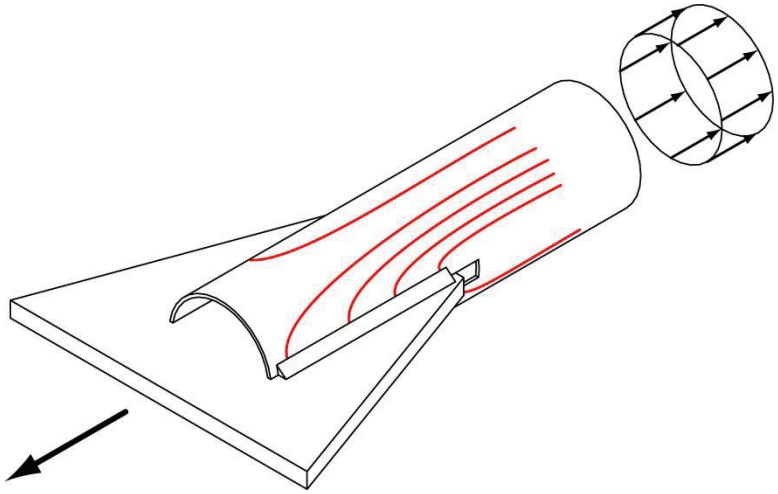


Figure 2 - Shear-lag in conventional slotted HSS-to-gusset connections

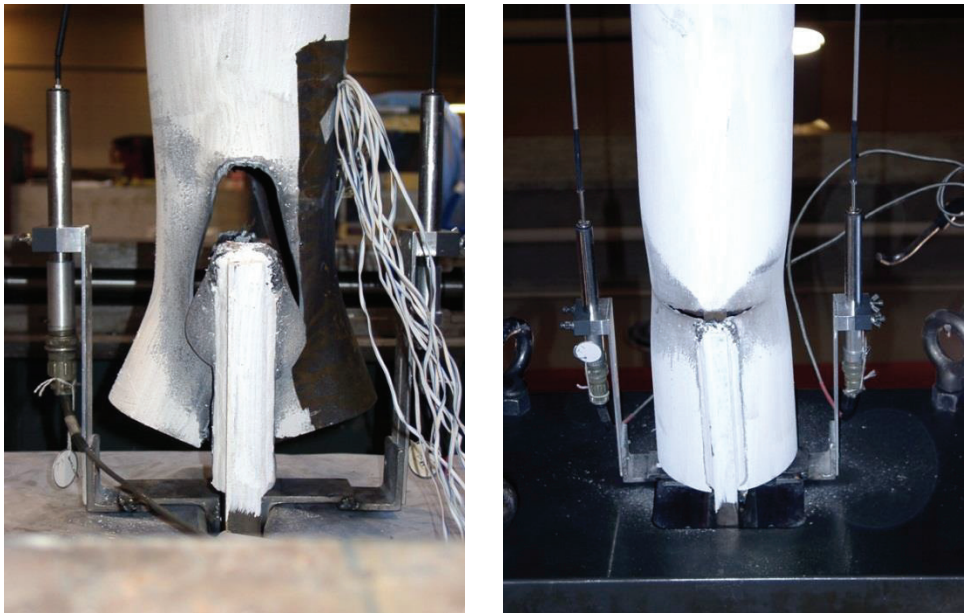


Figure 1 – Typical slotted HSS-to-gusset connection failures: tear-out failure (left); net-section fracture (right) [images courtesy of the University of Toronto]

End connections for HSS brace members are typically achieved through a gusset connection between the brace end and the beam-column intersection. In wind loaded bracing connections, a shear-lag inducing slotted HSS-to-gusset connection can be accommodated since axial loads are typically well below the cross-sectional capacity of the brace (Figure 2). However, both in the laboratory and in the field as witnessed during post-earthquake reconnaissance, conventional slotted HSS-to-gusset connections have been shown to be prone to failure when subjected to inelastic loading (Figure 1).

Recognizing the need for a simple solution to the seismic brace connection dilemma, a research team at the University of Toronto led by Professors Jeffrey A. Packer and Constantin Christopoulos developed standardized cast steel seismic-resistant connectors shaped to eliminate shear-lag in the HSS bracing connections. The geometric freedom that casting manufacturing provides allowed for the design of a connector that accommodates bolted or welded

connection to a gusset plate on one end and complete joint penetration (CJP) welded connection to a round HSS brace member on the other. Thus, in practice, the cast connectors can be welded to round HSS member braces in the shop, with the brace-connector assembly being bolted or welded to the gussets in the field. The resulting seismic-resistant connector technology is patented or patent pending in the US, Canada, and abroad.

Each CAST CONNEX® High Strength Connector™ is standardized to accommodate most round HSS and Pipe members of a given outer diameter. The use of a double-shear bolted connection halves the number of bolts that would otherwise be required in a spliced brace connection; spliced connections are sometimes specified to eliminate the need for field welding in conventional seismic-resistant reinforced brace connections. The specification of a CJP shop weld between the connector and the round HSS eliminates the need for field welding of the welds between the gusset plate and the brace member, if so desired (Figure 3). Alternatively, fillet welds applied in the field can be used to fasten the connector to the gusset.

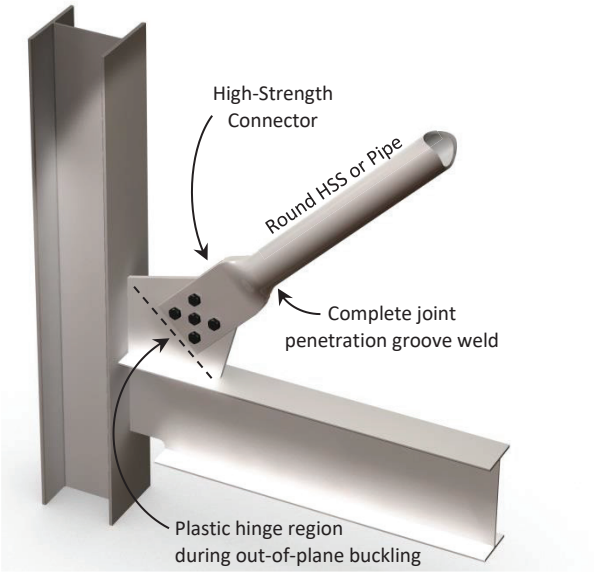


Figure 3 – High-Strength Connector shown in field-bolted configuration

The connectors themselves are manufactured to ASTM A958 Grade SC8620 Class 80/50, and are each subjected to a battery of non-destructive testing to ensure their quality, including:

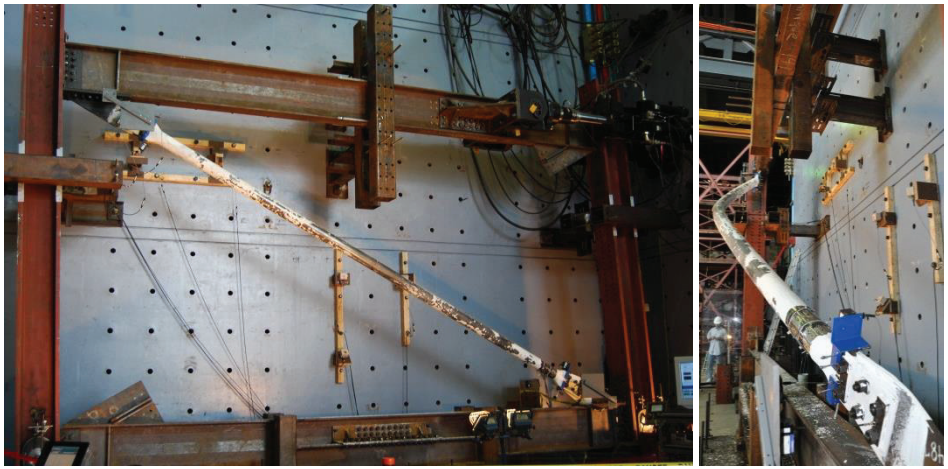
- visual examination,
- magnetic particle inspection,
- and ultrasonic testing.

Some of the many benefits of using High Strength Connectors at brace connections in braced frames are summarized below:

- elimination of field-welding of brace connections dramatically reduces erection cost and time as well as reduces the cost of special inspection; savings to the building owner which often exceed the cost of the connectors,
- use of the connectors results in more compact gusset plate connections which reduces the potential for the formation of undesirable failure modes in the column and beam members of the seismic force resisting system and also helps to ensure the structure behaves as it was modelled (as a braced frame rather than a fixed frame),
- in regions of delegated connection design, specification of High Strength Connectors mitigates risk by ensuring the building's braces can provide the ductility assumed in the design of the seismic force resisting system rather than relying on a third-party designing and detailing the connections correctly,
- though economical enough for industrial applications, the connectors' elegant shaping make them well suited for architecturally exposed applications, and
- High Strength Connectors are **Code Listed under ICC-ES ESR-3031**.

Testing of full-scale brace assemblies equipped with CAST CONNEX® High Strength Connectors™ has been carried out by researchers at the University of Toronto and École Polytechnique in Montreal.

A thorough summary of the development and testing carried out at the University of Toronto is described in the ASCE Journal of Structural Engineering, 134(3), 374-383. Papers on the full-scale testing carried out at École Polytechnique in Montreal were presented at the 14th World Conference of Earthquake Engineering and the 2011 SEAOC Convention. A more detailed report on the testing has also been published jointly by École Polytechnique in Montréal and the University of Toronto. All these publications and more are available for download at **www.castconnex.com**.



(a) (b) (c) (d) (e)

Figure 4 – Full-scale cyclic inelastic testing of a High-Strength Connector-equipped brace: (above) braced frame test; (below) brace test – (a) undeformed; (b) experiencing inelastic buckling; plastic hinge formation in free length of gusset plate; (d) plastic hinge in HSS member; (e) fracture of brace at mid-length after the formation of a local buckle in the tube wall.

Designing with High Strength Connectors

The use of CAST CONNEX® High Strength Connectors™ vastly simplifies the design, detailing, and fabrication of HSS brace member connections that meet the requirements of ANSI/AISC 341-22 in Special Concentrically Braced Frames (SCBF) and Ordinary Concentrically Braced Frames (OCBF).

Special and Ordinary Concentrically Braced Frames

There are several options available to engineers for the lateral force resisting system (LFRS) of steel structures. Concentrically braced frames (CBF) are, in many cases, the most efficient choice of LFRS for medium- to low-rise steel structures for a variety of reasons. First, fabrication cost and erection time are both greatly reduced through the use of simple shear connections throughout the entire structure. Additionally, the nature of the bracing system itself, consisting of several diagonal braces located intermittently throughout the structure, allows for great design versatility. There is additional design flexibility in the variety of brace configurations that are at the designer's disposal (chevron, V, X, single brace, etc). Braced frames are also very stiff in comparison to other LFRS, reducing lateral displacements and thus lessening second-order effects.

Regardless of the CBF configuration, its response in a design-level earthquake is always the same, that is, the brace elements will fully yield in tension and buckle in compression (Figure 5). This yielding and buckling will occur cyclically throughout the duration of the strong ground motion. It is imperative that the brace connections, along with the other elements of the LFRS, are able to resist the forces that will develop during the cyclic tensile yielding and compressive buckling of the brace elements. This is the essence of “Capacity Design”.

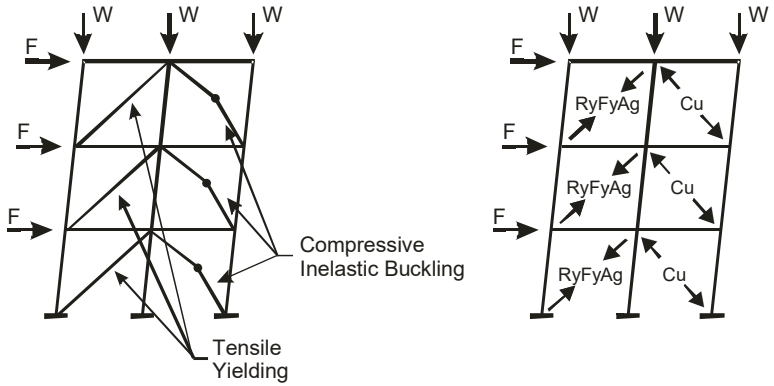


Figure 5 – Illustration of the plastic mechanism formed in a concentrically braced frame during strong-ground motion (left) and forces developed in a ductile concentrically braced frame during an earthquake (right) [adapted from Tremblay, R. (2003). Achieving a stable inelastic seismic response for multi-story concentrically braced steel frames. *AISC Engineering Journal, 40(2), 111-129*.]

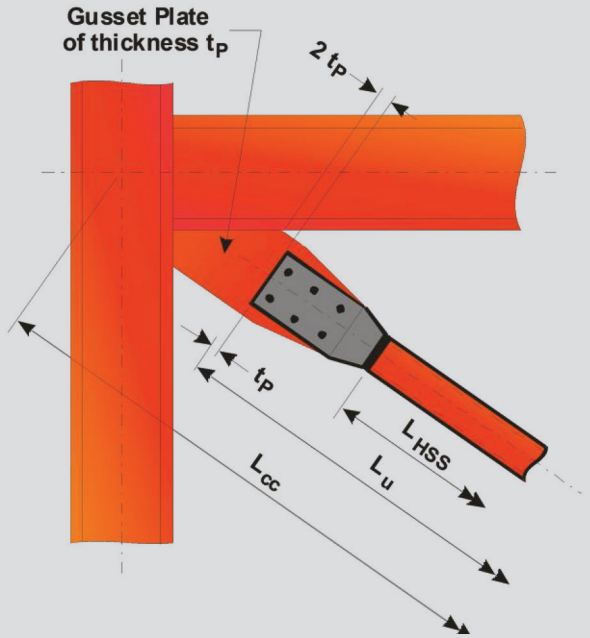
Brace Member Selection

The specification of High Strength Connectors in a structure does not change the way in which the engineer designs the other elements of the seismic-resistant braced frame in any way. For both SCBF and OCBF systems, the engineer should follow the governing building code to determine the appropriate story forces due to the design-level earthquake and subsequently size the elements of the LRFS following the requirements set out in the prevailing building code and ANSI/AISC 341-22. The only additional requirement is that the engineer should specify round HSS or Pipe members having outer diameters corresponding to those of the available line of High Strength Connectors. If the required brace member capacity cannot be achieved using round HSS or Pipe (i.e. all round HSS members having sufficient cross-sectional properties to carry the required load do not meet the D/t and/or KL/r requirements set out for ductile, seismic-resistant brace members), then the engineer should either provide additional braced frames on the particular story in question to reduce the required brace forces in each frame or specify heavier brace elements (i.e. wide-flange sections), for which a conventionally-fabricated connection must be used.

USER NOTE:

On the unbraced length of diagonal bracing elements

A common engineering practice in preliminary sizing of the brace members in braced frames where the engineer can rely on the compressive capacity of the brace is to assume the unbraced length of the brace in compression is its center-to-center length as measured from the centers of the beams and columns to which either end of the brace connects (L_{cc} below). While this is a conservative estimate for the purpose of sizing the brace members themselves, it is unconservative for the estimation of the compressive forces which will develop in the brace during the design-level earthquake. Thus, when determining the compressive force the brace connections and other elements of the LFRS must be capable of transmitting, the unbraced length of the brace element should be taken as the distance measured from the center of each of the "free lengths" just beyond the ends of the HSC components (dimension labeled L_u in the illustration below).



Although the responsibility for designing and detailing structural steel connections varies from region to region, once the elements of the LFRS have been set by the structural engineer of record, the corresponding High Strength Connector for each brace member should be specified on the structural drawings to ensure they are utilized by the fabricator that is contracted for the project.

Brace Connection Design using High Strength Connectors

As per ANSI/AISC 341-22, the required tensile strength of brace connections in SCBF and in some OCBF must be equal to or exceed the expected yield strength of the brace, given by $R_yF_yA_g$ (LRFD) or $R_yF_yA_g/1.5$ (ASD). For SCBF, the required compressive strength of the brace connection must be equal to or exceed the expected brace strength in compression (LRFD) or (1.0/1.5) times the expected brace strength in compression (ASD), where the expected brace strength in compression is the lesser of $R_yF_yA_g$ and $1.14F_{cre}A_g$ where F_{cre} is determined using the equations for F_{cr}, except that the expected yield stress R_yF_y is used in lieu of F_y. For HSS produced to ASTM A500 Gr. B, R_y must be taken as 1.4; for ASTM A500 Gr. C, R_y must be taken as 1.3; and for ASTM A1085 R_y must be taken as 1.25. For Pipe produced to ASTM A53, R_y must be taken as 1.6.

The use of CAST CONNEX® High Strength Connectors™ makes providing the aforementioned connection resistance very simple.

At one end, the connectors are designed with a circular shape and beveled preparation to allow for complete joint penetration shop welding to a range of tubular braces of a given outer diameter for the full development of their expected yield strength. At the other end, the connectors are shaped such that a double shear bolted connection or field-applied fillet welds can be used for connecting the shop-welded brace-connector assembly to conventional gusset plates secured to the beam-column intersection (Figure 6).

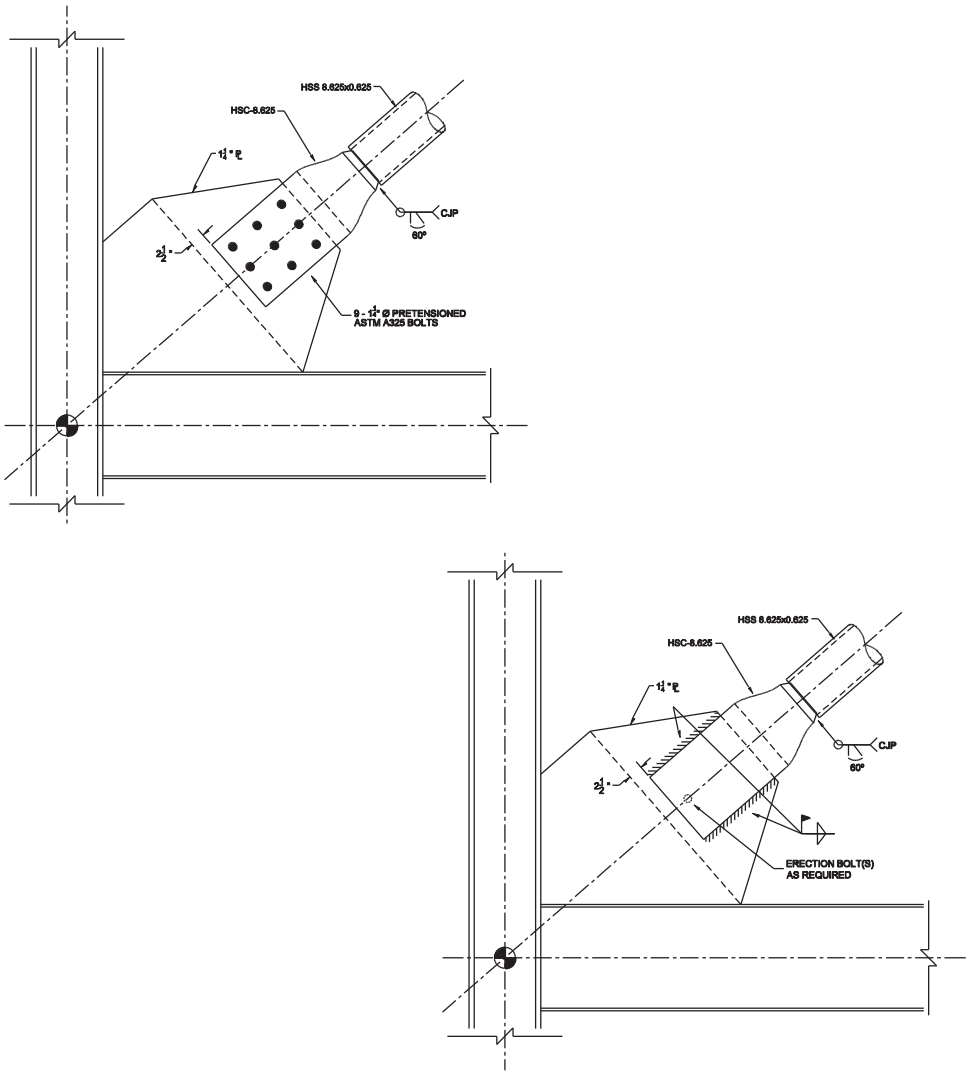


Figure 6 – Field bolted (top) and field welded (bottom) brace connection configurations using High-Strength Connectors

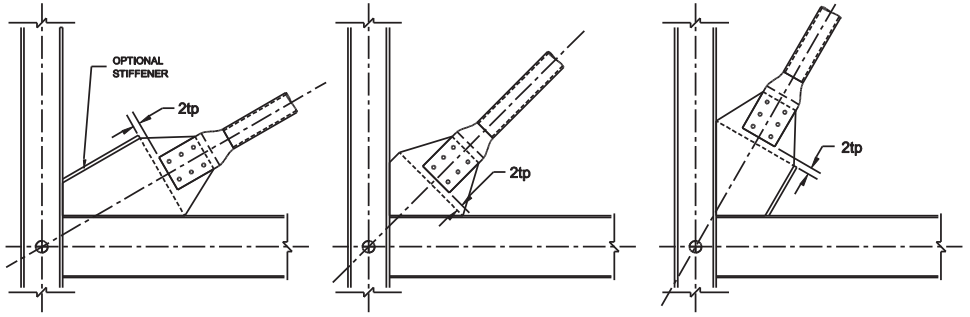


Figure 7 – Gusset plate detailing to accommodate out-of-plane inelastic deformation of a brace for various bracing angles [adapted from Astaneh-Asl, A., Goel, S.C., and Hanson, R.D., (1985). Cyclic out-of plane buckling of double-angle bracing. [*ASCE Journal of Structural Engineering, 111(5): 1135-1153*.]

An additional requirement for the use of High Strength Connectors in SCBF is that a free length of gusset plate should be provided beyond the ends of each connector to accommodate the inelastic end rotations that will be induced during out-of-plane buckling of the brace. The ANSI/AISC 341-22 commentary suggests that the width of this hinge-region should be at least twice the thickness of the gusset plate. This detail is illustrated above for various brace angles (Figure 7).

Although the use of High Strength Connectors makes the design of seismic-resistant brace end connections straightforward, detailing the gusset plate to which the brace-assembly connects remains a complex issue. Detailing of the gusset must be carried out with a clear understanding of the loads which must be transmitted and with an appreciation for the stability issues which may arise.

A good practical resource that discusses the design of gusset plates in seismic-resistant braced frames is the December 2006 issue of Steel Tips, entitled "*Seismic Detailing of Gusset Plates for Special Concentrically Braced Frames*" by Abolhassan Astaneh-Asl, Michael L. Cochran, and Rafael Sabelli. Steel Tips is published by the Structural Steel Educational Council (SSEC).

Specification Language – Division 05 12 00

Besides calling for the connectors in the appropriate brace frame elevations of the structural steel drawings, the appropriate specification language should be included in the Division 05 12 00 specification for structural steel, typically under a products subsection. Suggested specification language can be downloaded from **www.castconnex.com**.

Design Table and Detailing Assumptions

The design tables provided in this User Manual present suggested bolted connection details for the connection between a given HSC and gusset plate for a variety of HSS or Pipe brace members, each having unique expected yield strength [$R_yF_yA_g$ (LRFD) or $R_yF_yA_g/1.5$ (ASD)]. For every unique HSS or Pipe element, a suggested detail for a bearing-type bolted connection is indexed by: number of bolts required, bolt diameter (3/4", 7/8", 1", 1 1/8", or 1 1/4"), and bolt grade (Group 120: eg. A325, or Group 150: eg. A490). **ANSI/AISC 341-22 requires that all seismic-resistant bolted connections have pretensioned high-strength bolts**. Because pretensioning of bolts is labor intensive, the number of bolts in each of the suggested connection details has been minimized.

It is the responsibility of the Engineer of Record to confirm the connection resistance for each detail prior to use. As the connection resistance is often governed by "block shear rupture", changes to the gauge and pitch of bolts indicated in the details can adversely affect the resistance of the connection.

For the details provided, the following material properties were assumed unless otherwise noted:

HSC	F_y = 50 ksi, F_u = 80 ksi
Gusset	F_y = 36 ksi, F_u = 58 ksi

Material of equal or higher strength than that which is listed above (or noted on the connection detail) for the gusset plate must be provided if a detail provided in this user manual is to be followed.

Bolt capacities for bearing-type connections are calculated according to ANSI/AISC 360-22. High-Strength Connectors have been designed such that bolt threads are excluded from the shear planes. Although not covered in this design guide, slip-critical bolted connections can be designed by the Engineer of Record. Slip-critical bolted connections could be advantageous, as ANSI/AISC 341-22 permits the use of oversized holes in slip-critical connections provided the holes are oversized in one ply only.

Sample Connection Design

Assume that an engineer has sized all of the main structural members in a SCBF according to the governing building code and ANSI/AISC 341-22, and that these members are as shown below.

Detail a bearing-type brace end connection for the HSS 6.625x0.500 brace element at Joint J-1 assuming the HSS member is produced according to ASTM A500, Grade C and that pretensioned Group 150, ASTM F3125 Grade A490 bolts of 1-inch diameter are to be used.

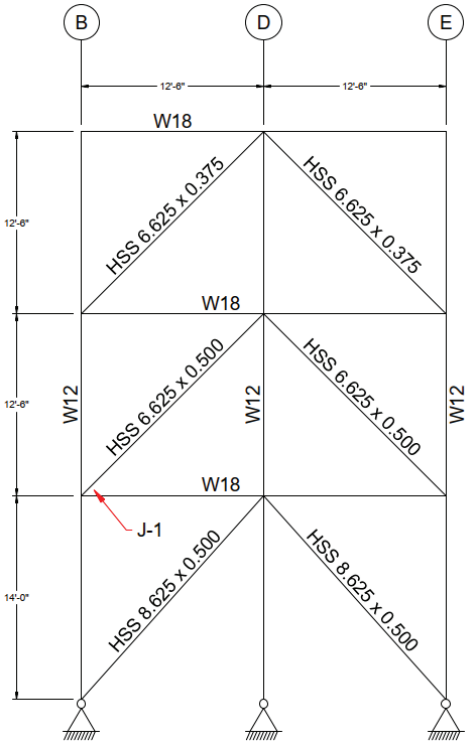


Figure 8 – Frame for sample connection design

Using High-Strength Connectors and this User Manual, the design procedure is simple:

As the HSS brace member has an outer diameter of 6.625-inches, we must specify HSC-6.625 connectors. We begin by opening the User Manual to the HSC-6.625 design table and find the ASTM A500 Grade C, HSS 6.625x0.500 section on the table. Reading across the row, we can see that for 1-inch, Group 150, ASTM F3125 Grade A490 bolts in a bearing-type connection, the required connection detail index is **8-1”-490**. Flipping to detail 8-1”-490 in the HSC-6.625 section of the manual, we find the detail that is shown below (Figure 9).

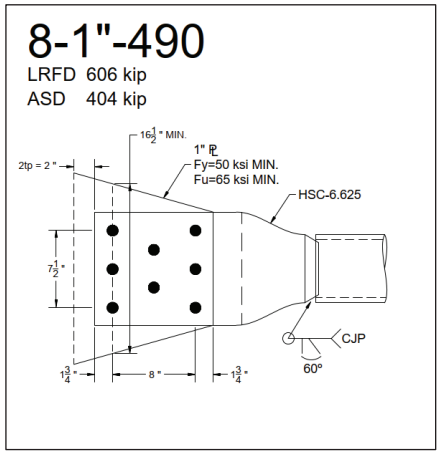


Figure 9 – Sample connection detail

We know that, according to ANSI/AISC 341-22, an HSS 6.625x0.500 brace member produced to ASTM A500, Grade C has an expected yield strength of $R_yF_yA_g$ = (1.3)(50 ksi)(9.00 in^2) = 585 kip (LRFD) (note that this information is also provided in the design table). As the connection detail shows a resistance of 606 kip (LRFD) (which should be confirmed by the connection designer prior to using the detail), we know that the detail selected is suitable. We then simply insert the detail into the drawing using the minimum net-section dimension given and the rules for the 2tp free length.

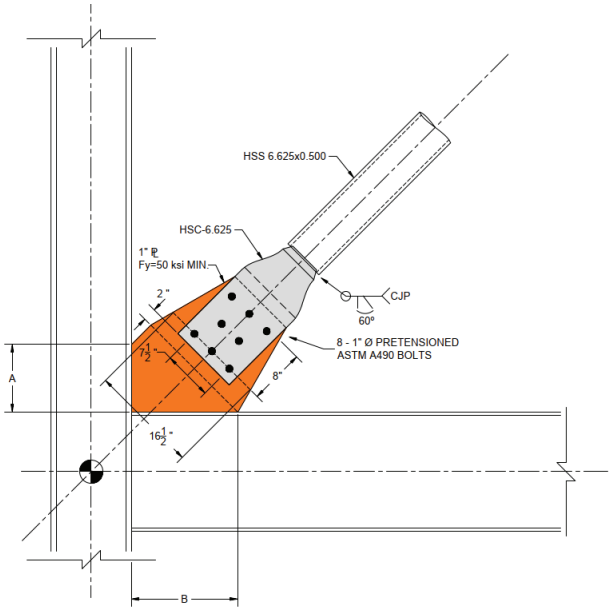


Figure 10 – Sample connection detail inserted into beam-column intersection

The design of the brace end connection is now complete.

The next step involves checking the adequacy of the gusset and detailing the connection between the gusset and beam, column, or beam and column, which should be done in accordance to ANSI/AISC 341-22 and ANSI/AISC 360-22. When detailing these connections, it is best to consider the preferences of local fabricators and erectors with respect to field bolting versus field welding, erection practices, etc.

Adequacy checking and detailing of the gusset connection is outside of the scope of this User Manual. An excellent practical resource that discusses the design of gusset plates in seismic-resistant braced frames is the December 2006 issue of Steel Tips, entitled “Seismic Detailing of Gusset Plates for Special Concentrically Braced Frames” by Abolhassan Astaneh-Asl, Michael L. Cochran, and Rafael Sabelli. Steel Tips is published by the Structural Steel Educational Council (SSEC).

In all likelihood, the resulting connections between the gusset and the beam, column, or beam and column will require larger dimensions “A” and “B” than were provided based on using the optimized connection detail provided in this User Manual. When increasing the size of the gusset, be sure to respect the 2tp requirements, as illustrated below.

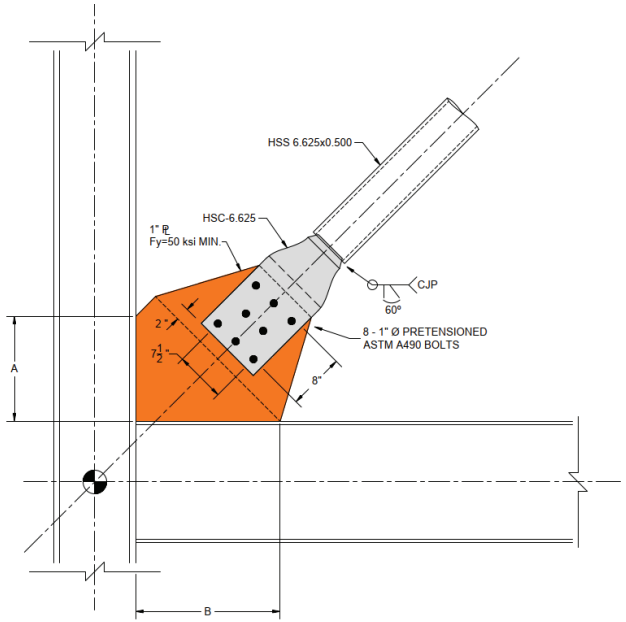


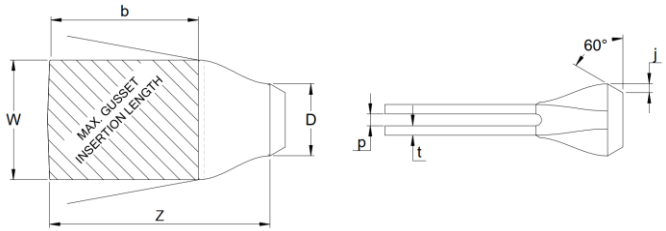
Figure 11 – 2tp requirements should be maintained when increasing dimensions “A” and “B”

The final check that must be carried out by the designer is to **confirm that the brace can be installed in the field**. Refer to the Site Erection section of this User Manual for more information on field installation of braces equipped with HSC.

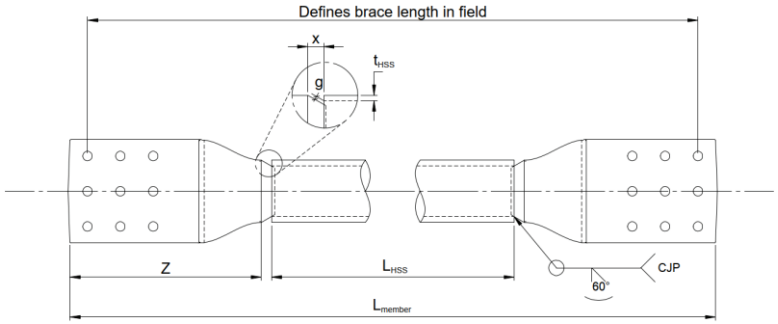
Fabrication with High-Strength Connectors

Detailing

The following figures, table, and equations are meant to assist in detailing HSC brace assemblies. Electronic versions of HSC geometry are available upon request at **info@castconnex.com**.



	Z [in.]	D [in.]	b [in.]	w [in.]	t [in.]	p_{min} [in.]	p_{max} [in.]	j [in.]
HSC-4.000	14 ¼	4	10	7	½	9/16	5/8	19/32
HSC-5.563	19 1/16	5 9/16	13	9	5/8	13/16	7/8	19/32
HSC-6.625	20 3/8	6 5/8	13	11	7/8	1 1/16	1 1/8	25/32
HSC-8.625	27 1/8	8 5/8	18	14	1	1 5/16	1 3/8	7/8
HSC-10.75	25 5/8	10 ¾	16	16	1 ¼	1 9/16	1 5/8	7/8
HSC-12.75	27 ¾	12 ¾	17	19	1 ¼	1 13/16	1 7/8	7/8
HSC-14.00	29 ¾	14	17	19	1 ½	1 13/16	1 7/8	7/8



$$L_{HSS} = L - 2[Z + X]$$

$$X = 2g + t_{HSS} \cdot \sqrt{3}$$

When using these equations to estimate the length of HSS required for a given brace, note that the actual t_{HSS} can be significantly thinner than the nominal value. Refer to the relevant HSS or Pipe specification

Fitting

When fitting a bolted brace assembly, it is important to note that the actual length of the brace is set by the distance between the bolt pattern at each end of the brace and that the **HSC should be carefully aligned in all directions (including roll) prior to welding**. For brace assemblies which are to be field bolted, some users have found it helpful to drill the bolt patterns into the HSC after having welded both connectors to the hollow section as this has allowed for improved control of brace length. However, other methods for fitting have also been successful.

Figure 12 – Fitting of HSC brace assembly. Fitter simultaneously ensures:
1) the HSC connectors are level and in-line
2) the appropriate weld root gap is provided at the joints
3) the overall length of the brace assembly is correct

Drilling

HSC are produced with steel that is very tough. As a result, drilling should be carried out using a high quality carbide-tipped tool operated at the correct drill

speed. When drilling, the tool should past through both HSC flanges. Failure to clear the slug produced during the drilling of the first flange before starting the second core may result in tool fracture. Consider tack-welding a tie bar between the connector's tabs to reduce chatter during drilling.

Welding

HSC are manufactured using material produced to ASTM A958 Grade SC8620 Class 80/50. **Heat treatment for this material may include quenching and tempering (QT).** As a result, it is important to follow good welding practices for QT materials for welds applied to the HSC. In all seismic applications, **the welded joints must meet all the requirements stipulated in ANSI/AISC 341-22, AWS D1.1, and AWS D1.8 for seismic-resistant demand critical welded connections.**

Although ASTM A958 Grade SC8620 Class 80/50 is a weldable base metal with both mechanical and chemical properties similar to those of a standard wrought steel grades, it is not a pre-approved base metal according to the American Welding Society (AWS). Because of this, and because of the nature of the weld that must be applied between the HSC and the HSS or Pipe member, **the Engineer of Record must approve a Welding Procedure Specification (WPS) and a Procedure Qualification Record (PRQ) must be produced.**

<u>Demand Critical Complete Joint Penetration Groove Weld</u>

The demand critical weld between the HSC and the HSS or Pipe member **must provide complete joint penetration (CJP).** Note that the significant mass of the HSC in the region of the CJP necessitates the application of pre-heating prior to welding.

HSC are supplied with a 60° weld preparation, thus HSS or Pipe members need only be square-cut to the appropriate length (refer to the Fitting section of this User Manual) and tack-welded to the HSC prior to CJP welding. Past users have welded successfully using a motorized turning roll.

A sample Welding Procedure Specification can be downloaded from our website **www.castconnex.com**

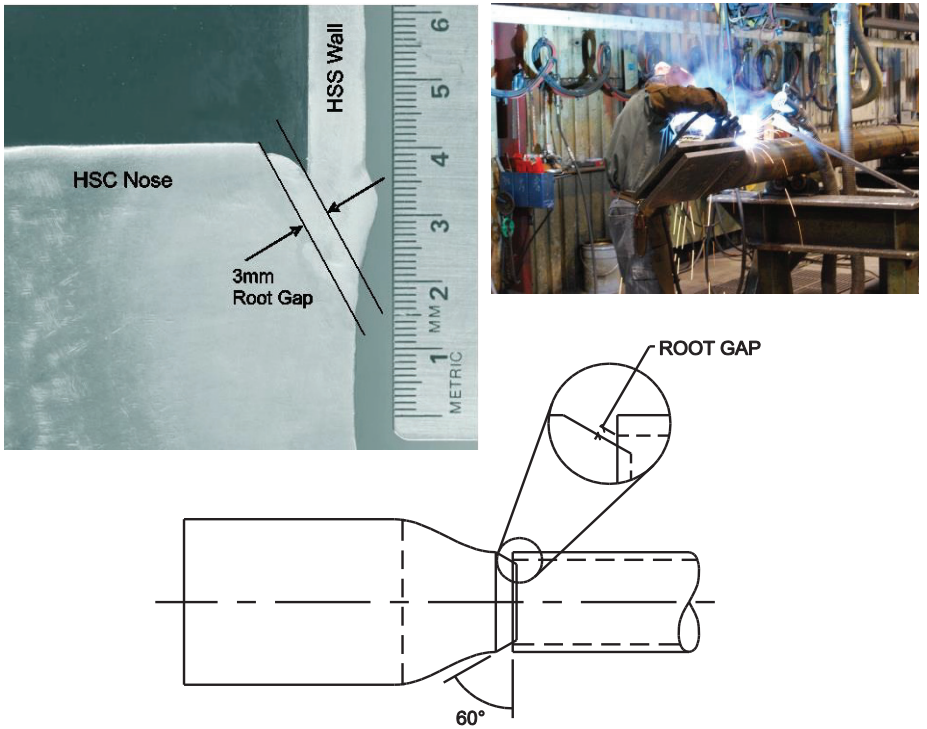


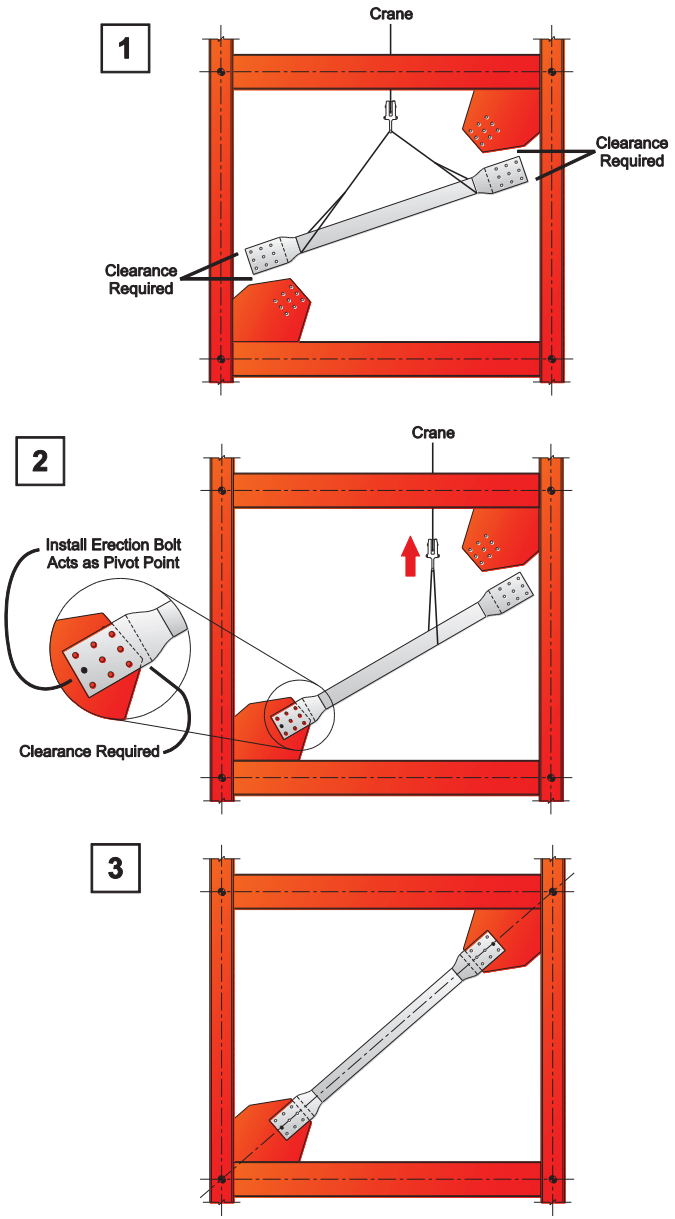
Figure 13 – Welded joint configuration

WARNING: If the Complete Joint Penetration Weld between the HSC and the hollow section brace element is not applied correctly, or if fillet welds that may be applied in the field between the HSC and gusset plate are not applied correctly, fracture of these welds or of the base metal may occur during a seismic event or as a result of fatigue. CAST CONNEX CORPORATION, ITS AFFILIATES, SUBSIDIARIES OR RELATED COMPANIES, ASSUME NO LIABILITY WHATSOEVER WITH RESPECT TO THE QUALITY OF ANY WELDS APPLIED TO HSC BY ANY END USER OF THE HSC PRODUCT.

Site Erection

It is quite common for the beams and columns in a steel braced frame to be erected prior to the installation of the brace member. In these cases, the designer should ensure that the brace can be installed in the field given the specific geometry of the frame, gusset plates, and brace assembly. The diagrams below illustrate the most common sequence for brace installation in this circumstance and should help the designer understand some of the constraints that arise in this situation. **When gusset plate to connector connections are too compact to allow for erection as illustrated here, a note should be added to the brace elevation calling for the brace member to be installed prior to the installation of the beam above or for the brace to be installed in tandem with the beam above**.

Depending on the specifics of a given project, the erection schemes for primary structural steel elements can vary widely, particularly with respect to braced frames. Whenever possible, the design team should consult with the contractors involved in the project to gain an understanding of their preferred practices and any specific erection constraints. The images below are only meant to make users of this manual aware of some of the issues that may arise and do not present the only possible erection scheme for braced frames.



Design Tables and Connection Details

Please visit **www.castconnex.com** or contact **info@castconnex.com** to access the latest version of the design tables and connection details.

HSC-4.000

ANSI/AISC 341-22

$$\frac{D}{t} \leq \frac{0.053 \cdot E}{R_y F_y}$$

ASTM A500
Grade B

Fy = 46 ksi thus D/t ≤ 23.9

Ry·Fy = 64 ksi

Shape	Wall Thickness, t		D/t	Area, A	Ry·Fy·A	BEARING-TYPE CONNECTIONS[1] Detail Number					
						Group 120 (eg. A325)			Group 150 (eg. A490)		
	Nominal	Design[2]				Bolt Size			Bolt Size		
	in.	in.		in.^{2}	kip	3/4	7/8	1	3/4	7/8	1
HSS 4.000	0.337	0.313	12.8	3.63	234	6-3/4"-325	**X**	**X**	6-3/4"-490	**X**	**X**
	0.313	0.291	13.7	3.39	218	6-3/4"-325	5-7/8"-325	**X**	6-3/4"-490	5-7/8"-490	**X**
	0.250	0.233	17.2	2.75	177	5-3/4"-325	5-7/8"-325	4-1"-325	5-3/4"-490	5-7/8"-490	4-1"-490
	0.237	0.220	18.1	2.62	169	5-3/4"-325	4-7/8"-325	4-1"-325	5-3/4"-490	4-7/8"-490	4-1"-490
	0.226	0.210	19.0	2.50	161	5-3/4"-325	4-7/8"-325	3-1"-325	5-3/4"-490	4-7/8"-490	3-1"-490
	0.220	0.205	19.6	2.44	157	5-3/4"-325	4-7/8"-325	3-1"-325	5-3/4"-490	4-7/8"-490	3-1"-490
	0.188	0.175	22.9	2.10	135	4-3/4"-325	4-7/8"-325	3-1"-325	4-3/4"-490	4-7/8"-490	3-1"-490

1. Connections must have pretensioned high-strength bolts. The following are suggested bolt lengths:
   2¾" Long bolt for 3/4" and 7/8" Group 120 (eg. A325) or Group 150 (eg. A490)
   3" Long bolt for 1" Group 120 (eg. A325) or Group 150 (eg. A490)
2. Design wall thickness for ASTM A500 HSS sections taken as $0.93 \cdot t_{nominal}$.

X Connector tabs not large enough to accommodate the number of bolts required

HSC-4.000

ANSI/AISC 341-22

$$\frac{D}{t} \leq \frac{0.053 \cdot E}{R_y F_y}$$

ASTM A500
Grade C

Fy = 50 ksi thus D/t ≤ 23.6

Ry·Fy = 65 ksi

Shape	Wall Thickness, t		D/t	Area, A	Ry·Fy·A	BEARING-TYPE CONNECTIONS[1] Detail Number					
						Group 120 (eg. A325)			Group 150 (eg. A490)		
	Nominal	Design[2]				Bolt Size			Bolt Size		
	in.	in.		in.^{2}	kip	3/4	7/8	1	3/4	7/8	1
HSS 4.000	0.337	0.313	12.8	3.63	236	6-3/4"-325	**X**	**X**	6-3/4"-490	**X**	**X**
	0.313	0.291	13.7	3.39	220	6-3/4"-325	5-7/8"-325	**X**	6-3/4"-490	5-7/8"-490	**X**
	0.250	0.233	17.2	2.75	179	5-3/4"-325	5-7/8"-325	4-1"-325	5-3/4"-490	5-7/8"-490	4-1"-490
	0.237	0.220	18.1	2.62	170	5-3/4"-325	5-7/8"-325	4-1"-325	5-3/4"-490	5-7/8"-490	4-1"-490
	0.226	0.210	19.0	2.50	163	5-3/4"-325	4-7/8"-325	3-1"-325	5-3/4"-490	4-7/8"-490	3-1"-490
	0.220	0.205	19.6	2.44	159	5-3/4"-325	4-7/8"-325	3-1"-325	5-3/4"-490	4-7/8"-490	3-1"-490
	0.188	0.175	22.9	2.10	137	4-3/4"-325	4-7/8"-325	3-1"-325	4-3/4"-490	4-7/8"-490	3-1"-490

1. Connections must have pretensioned high-strength bolts. The following are suggested bolt lengths:
   2¾" Long bolt for 3/4" and 7/8" Group 120 (eg. A325) or Group 150 (eg. A490)
   3" Long bolt for 1" Group 120 (eg. A325) or Group 150 (eg. A490)
2. Design wall thickness for ASTM A500 HSS sections taken as $0.93 \cdot t_{nominal}$.

X Connector tabs not large enough to accommodate the number of bolts required

HSC-4.000

ANSI/AISC 341-22

$$\frac{D}{t} \leq \frac{0.053 \cdot E}{R_y F_y}$$

ASTM A53
Grade B

Fy = 35 ksi thus D/t ≤ 27.4

Ry·Fy = 56 ksi

Shape	Wall Thickness, t		D/t	Area, A	Ry·Fy·A	BEARING-TYPE CONNECTIONS[1] Detail Number					
						Group 120 (eg. A325)			Group 150 (eg. A490)		
	Nominal	Design[2]				Bolt Size			Bolt Size		
	in.	in.		in.^{2}	kip	3/4	7/8	1	3/4	7/8	1
Pipe 3½											
XS	0.318	0.296	13.5	3.44	193	6-3/4"-325	5-7/8"-325	**X**	6-3/4"-490	5-7/8"-490	**X**
STD	0.226	0.210	19.0	2.50	140	4-3/4"-325	4-7/8"-325	3-1"-325	4-3/4"-490	4-7/8"-490	3-1"-490

1. Connections must have pretensioned high-strength bolts. The following are suggested bolt lengths:
   2¾" Long bolt for 3/4" and 7/8" Group 120 (eg. A325) or Group 150 (eg. A490)
   3" Long bolt for 1" Group 120 (eg. A325) or Group 150 (eg. A490)
2. Design wall thickness for ASTM A53 Pipe sections taken as $0.93 \cdot t_{nominal}$.

X Connector tabs not large enough to accommodate the number of bolts required

HSC-4.000

ANSI/AISC 341-22

$$\frac{D}{t} \leq \frac{0.053 \cdot E}{R_y F_y}$$

ASTM A1085
Grade A

Fy = 50 ksi

Ry·Fy = 63 ksi

thus D/t ≤ 24.6

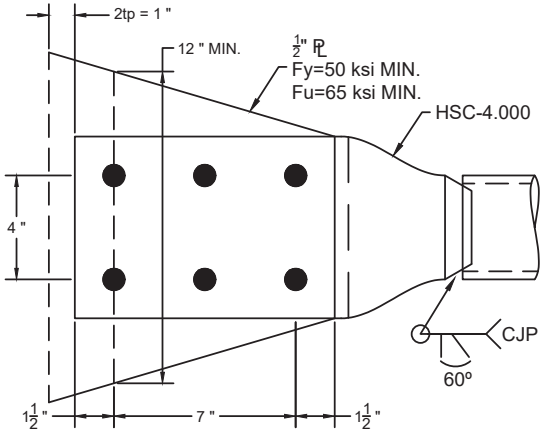
Shape	Wall Thickness, t		D/t	Area, A	Ry·Fy·A	BEARING-TYPE CONNECTIONS[1] Detail Number					
						Group 120 (eg. A325)			Group 150 (eg. A490)		
	Nominal	Design[2]				Bolt Size			Bolt Size		
	in.	in.		in.^{2}	kip	3/4	7/8	1	3/4	7/8	1
HSS 4.000	0.313	0.313	12.8	3.63	227	6-3/4"-325	**X**	**X**	6-3/4"-490	**X**	**X**
	0.250	0.250	16.0	2.95	184	5-3/4"-325	5-7/8"-325	4-1"-325	5-3/4"-490	5-7/8"-490	4-1"-490
	0.237	0.237	16.9	2.80	175	5-3/4"-325	5-7/8"-325	4-1"-325	5-3/4"-490	5-7/8"-490	4-1"-490
	0.226	0.226	17.7	2.68	167	5-3/4"-325	4-7/8"-325	3-1"-325	5-3/4"-490	4-7/8"-490	3-1"-490
	0.220	0.220	18.2	2.61	163	5-3/4"-325	4-7/8"-325	3-1"-325	5-3/4"-490	4-7/8"-490	3-1"-490
	0.188	0.188	21.3	2.25	141	4-3/4"-325	4-7/8"-325	3-1"-325	4-3/4"-490	4-7/8"-490	3-1"-490

1. Connections must have pretensioned high-strength bolts. The following are suggested bolt lengths:
   2¾" Long bolt for 3/4" and 7/8" Group 120 (eg. A325) or Group 150 (eg. A490)
   3" Long bolt for 1" Group 120 (eg. A325) or Group 150 (eg. A490)
2. Design wall thickness for ASTM A1085 HSS sections taken as $1.0 \cdot t_{nominal}$.

X Connector tabs not large enough to accommodate the number of bolts required

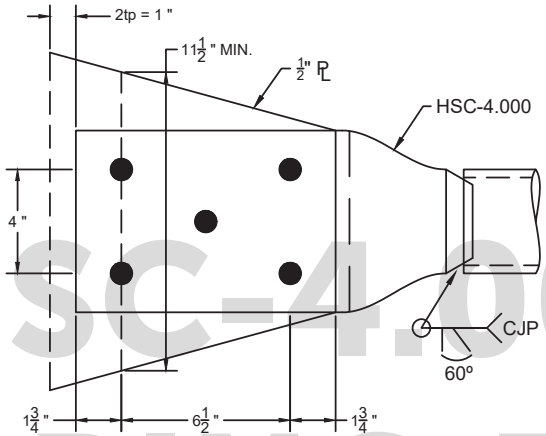
6-3/4"-325

LRFD 238 kip
ASD 158.8 kip



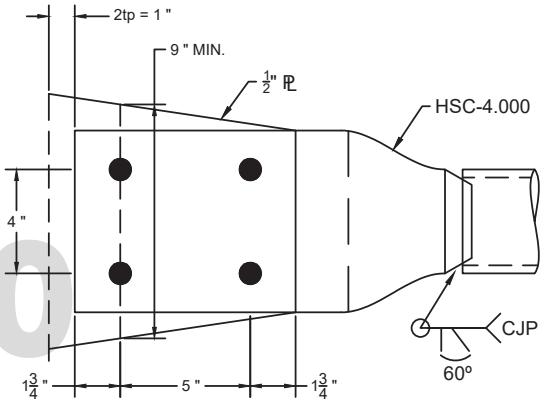
5-3/4"-325

LRFD 186.3 kip
ASD 124.0 kip



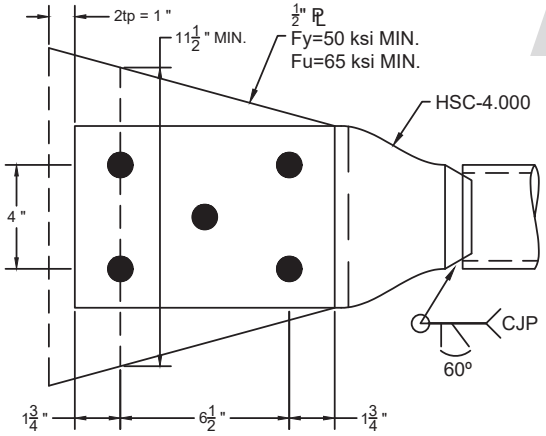
4-3/4"-325

LRFD 145.8 kip
ASD 97.0 kip



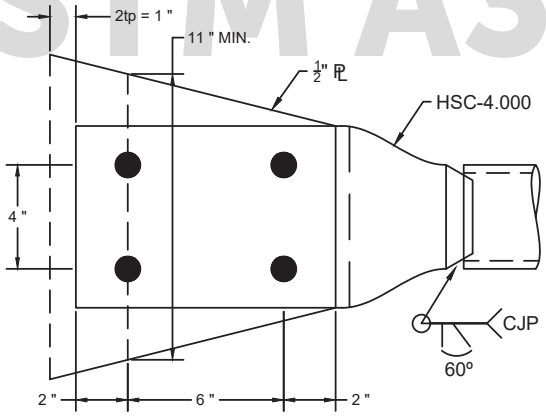
5-7/8"-325

LRFD 222 kip
ASD 148.0 kip



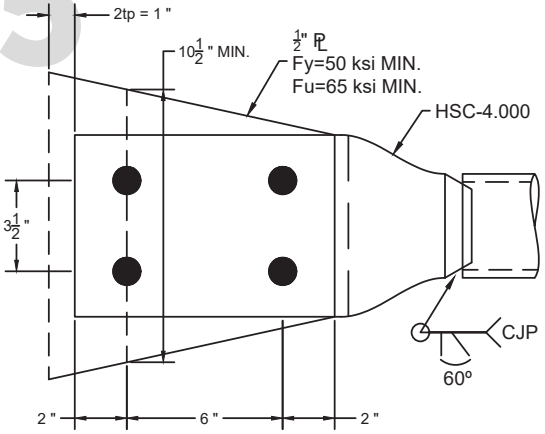
4-7/8"-325

LRFD 171.3 kip
ASD 114.2 kip



4-1"-325

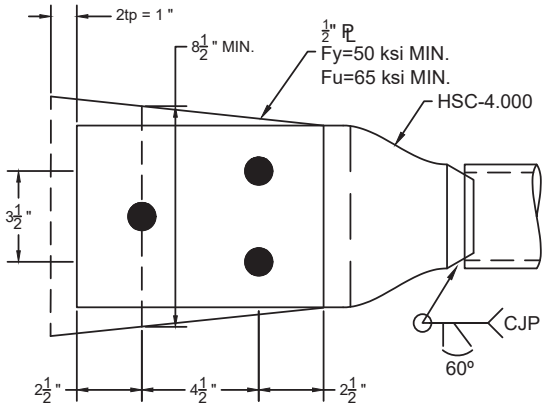
LRFD 190.7 kip
ASD 127.2 kip



3-1"-325

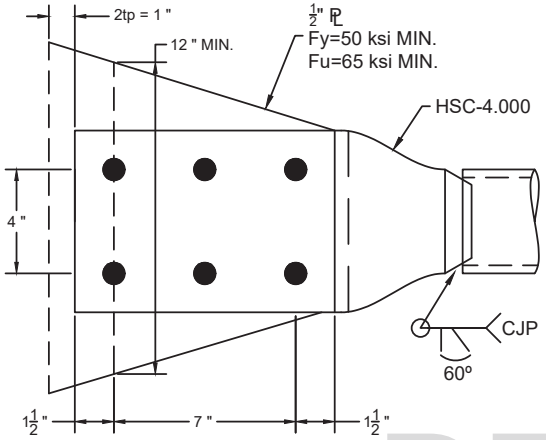
LRFD 171.8 kip

ASD 114.6 kip



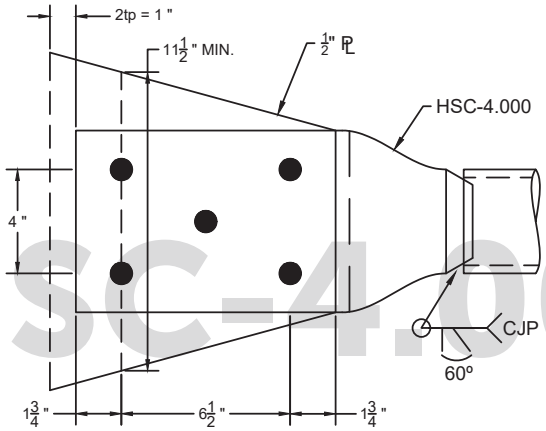
6-3/4"-490

LRFD 238 kip
ASD 158.8 kip



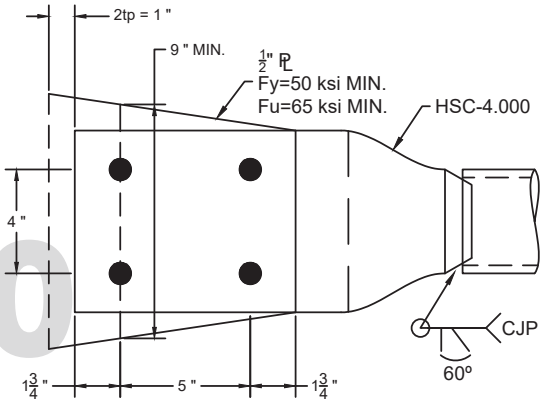
5-3/4"-490

LRFD 186.3 kip
ASD 124.0 kip



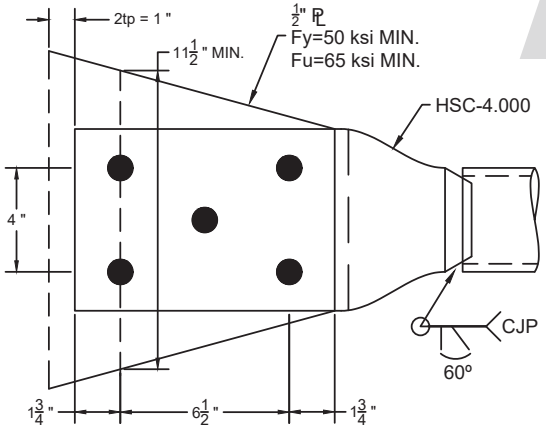
4-3/4"-490

LRFD 166.4 kip
ASD 110.9 kip



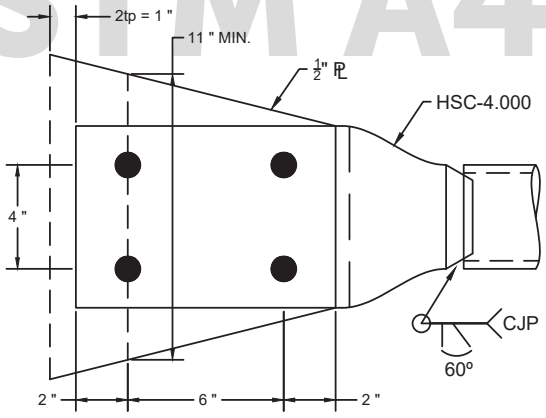
5-7/8"-490

LRFD 222 kip
ASD 148.0 kip



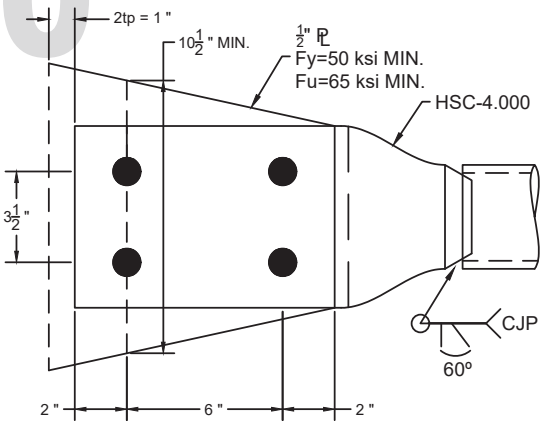
4-7/8"-490

LRFD 171.3 kip
ASD 114.2 kip



4-1"-490

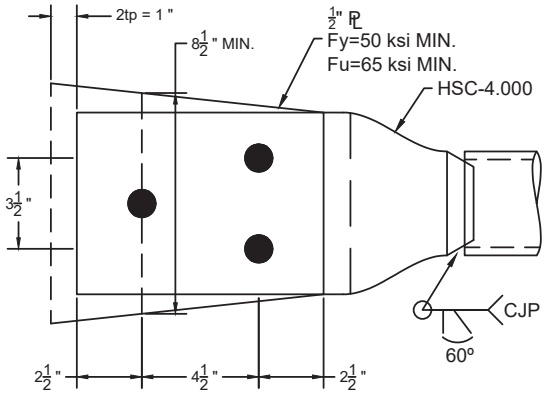
LRFD 190.7 kip
ASD 127.2 kip



3-1"-490

LRFD 171.8 kip

ASD 114.6 kip



HSC-5.563

ANSI/AISC 341-22

$$\frac{D}{t} \leq \frac{0.053 \cdot E}{R_y F_y}$$

ASTM A500
Grade B

Fy = 46 ksi

Ry·Fy = 64 ksi

thus D/t ≤ 23.9

Shape	Wall Thickness, t		D/t	Area, A	Ry·Fy·A	BEARING-TYPE CONNECTIONS[1] Detail Number					
						Group 120 (eg. A325)			Group 150 (eg. A490)		
	Nominal	Design[2]				Bolt Size			Bolt Size		
	in.	in.		in.^{2}	kip	3/4	7/8	1	3/4	7/8	1
HSS 5.563	0.375	0.349	15.9	5.71	368	9-3/4"-325	8-7/8"-325	6-1"-325	8-3/4"-490	6-7/8"-490	5-1"-490
	0.258	0.240	23.2	4.01	258	6-3/4"-325	5-7/8"-325	4-1"-325	6-3/4"-490	4-7/8"-490	4-1"-490

1. Connections must have pretensioned high-strength bolts. The following are suggested bolt lengths:
   3¼" Long bolt for 3/4" and 7/8" Group 120 (eg. A325) or Group 150 (eg. A490)
   3½" Long bolt for 1" Group 120 (eg. A325) or Group 150 (eg. A490)
2. Design wall thickness for ASTM A500 HSS sections taken as $0.93 \cdot t_{nominal}$.

X Connector tabs not large enough to accommodate the number of bolts required

HSC-5.563

ANSI/AISC 341-22

$$\frac{D}{t} \leq \frac{0.053 \cdot E}{R_y F_y}$$

ASTM A500
Grade C

Fy = 50 ksi

Ry·Fy = 65 ksi

thus D/t ≤ 23.6

Shape	Wall Thickness, t		D/t	Area, A	Ry·Fy·A	BEARING-TYPE CONNECTIONS[1] Detail Number					
						Group 120 (eg. A325)			Group 150 (eg. A490)		
	Nominal	Design[2]				Bolt Size			Bolt Size		
	in.	in.		in.^{2}	kip	3/4	7/8	1	3/4	7/8	1
HSS 5.563	0.375	0.349	15.9	5.71	371	10-3/4"-325	8-7/8"-325	6-1"-325	8-3/4"-490	6-7/8"-490	5-1"-490
	0.258	0.240	23.2	4.01	261	7-3/4"-325	5-7/8"-325	4-1"-325	6-3/4"-490	4-7/8"-490	4-1"-490

1. Connections must have pretensioned high-strength bolts. The following are suggested bolt lengths:
   3¼" Long bolt for 3/4" and 7/8" Group 120 (eg. A325) or Group 150 (eg. A490)
   3½" Long bolt for 1" Group 120 (eg. A325) or Group 150 (eg. A490)
2. Design wall thickness for ASTM A500 HSS sections taken as $0.93 \cdot t_{nominal}$.

X Connector tabs not large enough to accommodate the number of bolts required

HSC-5.563

ANSI/AISC 341-22

$$\frac{D}{t} \leq \frac{0.053 \cdot E}{R_y F_y}$$

ASTM A53
Grade B

Fy = 35 ksi thus D/t ≤ 27.4

Ry·Fy = 56 ksi

Shape	Wall Thickness, t		D/t	Area, A	Ry·Fy·A	BEARING-TYPE CONNECTIONS[1] Detail Number					
						Group 120 (eg. A325)			Group 150 (eg. A490)		
	Nominal	Design[2]				Bolt Size			Bolt Size		
	in.	in.		in.^{2}	kip	3/4	7/8	1	3/4	7/8	1
Pipe 5											
XS	0.375	0.349	15.9	5.71	320	9-3/4"-325	6-7/8"-325	5-1"-325	7-3/4"-490	5-7/8"-490	5-1"-490
STD	0.258	0.240	23.2	4.01	225	6-3/4"-325	4-7/8"-325	4-1"-325	5-3/4"-490	4-7/8"-490	4-1"-490

1. Connections must have pretensioned high-strength bolts. The following are suggested bolt lengths:
   3¼" Long bolt for 3/4" and 7/8" Group 120 (eg. A325) or Group 150 (eg. A490)
   3½" Long bolt for 1" Group 120 (eg. A325) or Group 150 (eg. A490)
2. Design wall thickness for ASTM A53 Pipe sections taken as $0.93 \cdot t_{nominal}$.

X Connector tabs not large enough to accommodate the number of bolts required

HSC-5.563

ANSI/AISC 341-22

$$\frac{D}{t} \leq \frac{0.053 \cdot E}{R_y F_y}$$

ASTM A1085
Grade A

Fy = 50 ksi

Ry·Fy = 63 ksi

thus D/t ≤ 24.6

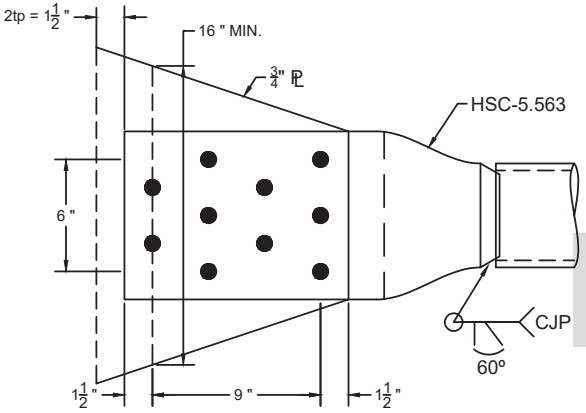
Shape	Wall Thickness, t		D/t	Area, A	Ry·Fy·A	BEARING-TYPE CONNECTIONS[1] Detail Number					
						Group 120 (eg. A325)			Group 150 (eg. A490)		
	Nominal	Design[2]				Bolt Size			Bolt Size		
	in.	in.		in.^{2}	kip	3/4	7/8	1	3/4	7/8	1
HSS 5.563	0.375 0.258	0.375 0.258	14.8 21.6	6.11 4.30	382 269	10-3/4"-325 7-3/4"-325	8-7/8"-325 5-7/8"-325	6-1"-325 5-1"-325	8-3/4"-490 6-3/4"-490	6-7/8"-490 4-7/8"-490	5-1"-490 4-1"-490

1. Connections must have pretensioned high-strength bolts. The following are suggested bolt lengths:
   3¼" Long bolt for 3/4" and 7/8" Group 120 (eg. A325) or Group 150 (eg. A490)
   3½" Long bolt for 1" Group 120 (eg. A325) or Group 150 (eg. A490)
2. Design wall thickness for ASTM A1085 HSS sections taken as $1.0 \cdot t_{nominal}$.

X Connector tabs not large enough to accommodate the number of bolts required

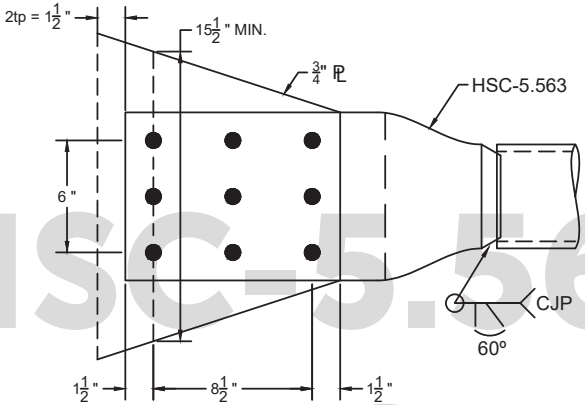
10-3/4"-325

LRFD 389 kip
ASD 259 kip



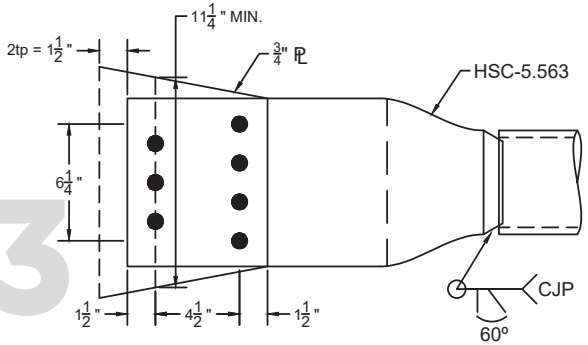
9-3/4"-325

LRFD 377 kip
ASD 251 kip



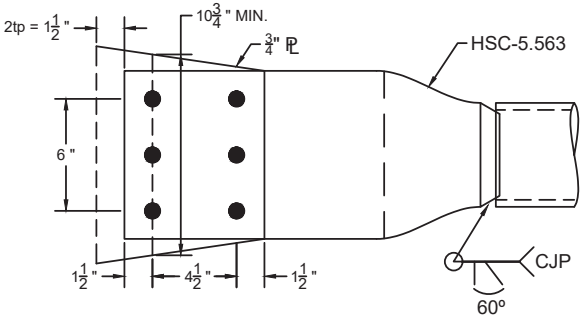
7-3/4"-325

LRFD 273 kip
ASD 181.9 kip



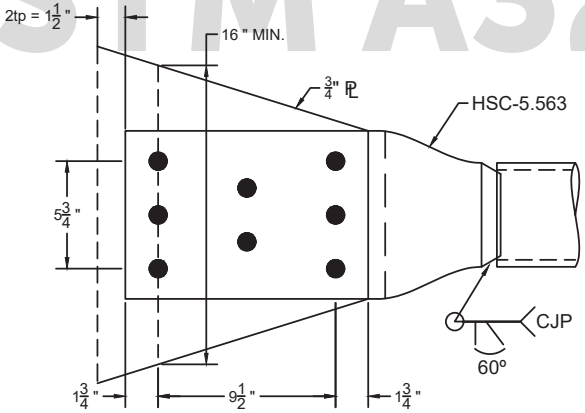
6-3/4"-325

LRFD 261 kip
ASD 173.8 kip



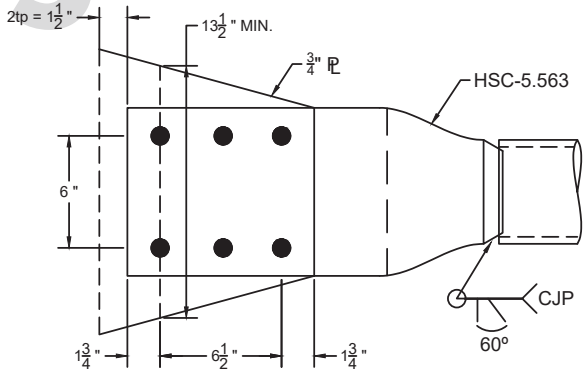
8-7/8"-325

LRFD 389 kip
ASD 259 kip



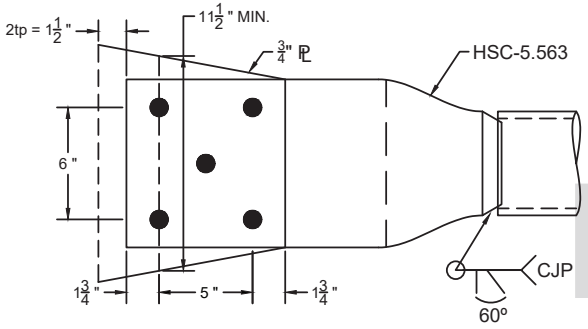
6-7/8"-325

LRFD 328 kip
ASD 218 kip



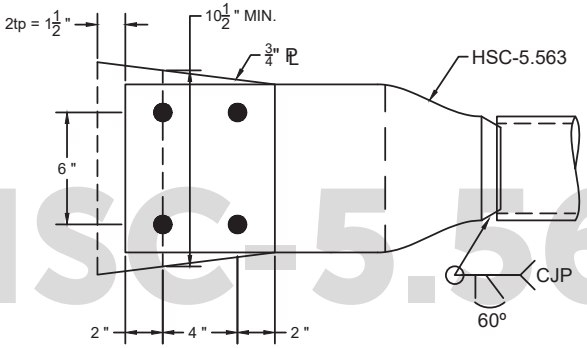
5-7/8"-325

LRFD 279 kip
ASD 185.9 kip



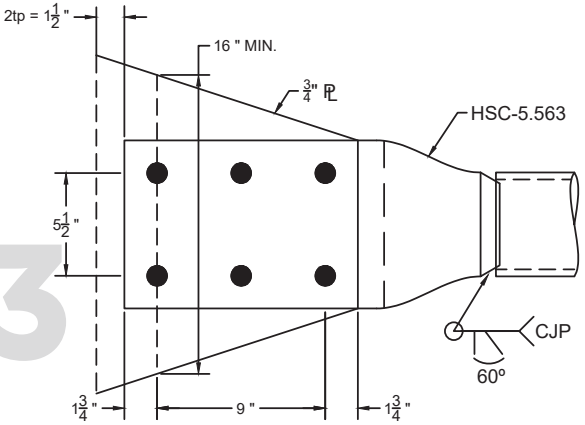
4-7/8"-325

LRFD 243 kip
ASD 161.7 kip



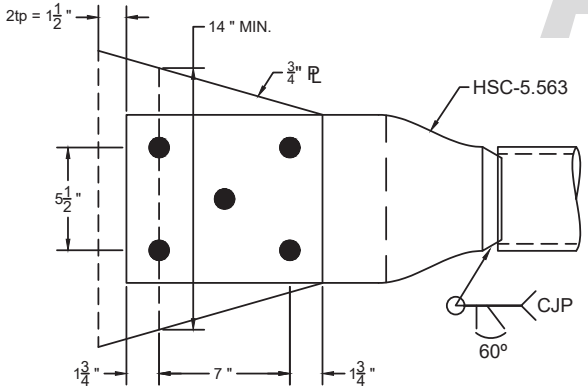
6-1"-325

LRFD 386 kip
ASD 257 kip



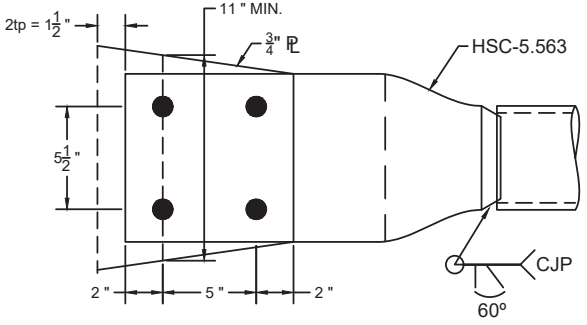
5-1"-325

LRFD 328 kip
ASD 219 kip



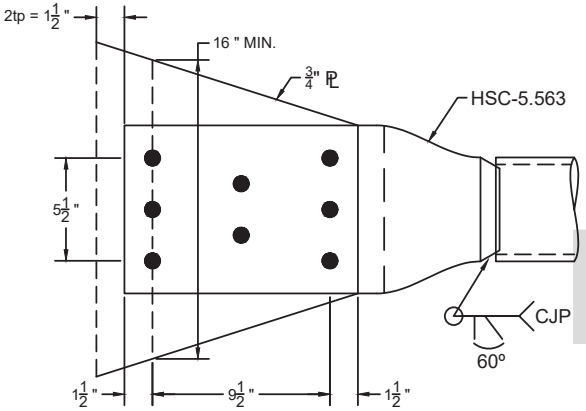
4-1"-325

LRFD 267 kip
ASD 177.8 kip



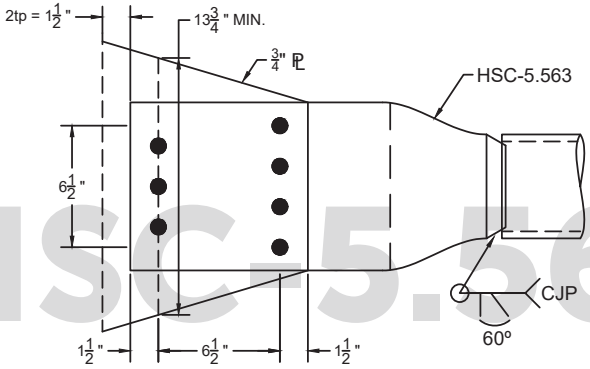
8-3/4"-490

LRFD 389 kip
ASD 259 kip



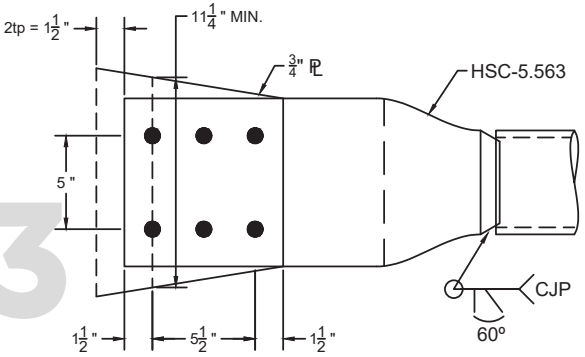
7-3/4"-490

LRFD 334 kip
ASD 222 kip



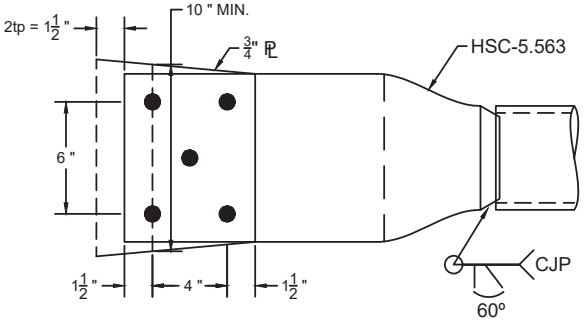
6-3/4"-490

LRFD 273 kip
ASD 181.9 kip



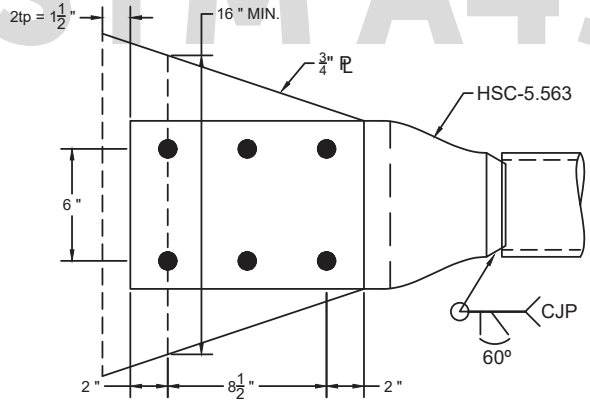
5-3/4"-490

LRFD 243 kip
ASD 161.7 kip



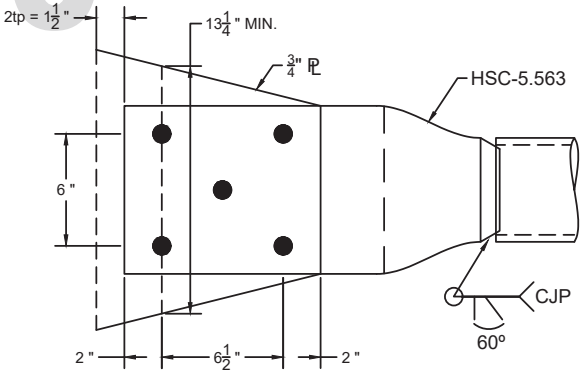
6-7/8"-490

LRFD 384 kip
ASD 256 kip



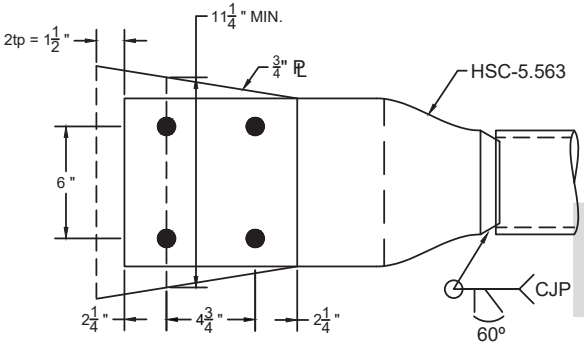
5-7/8"-490

LRFD 322 kip
ASD 214 kip



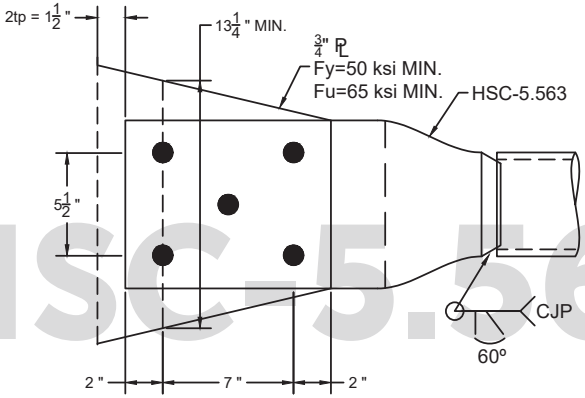
4-7/8"-490

LRFD 273 kip
ASD 181.9 kip



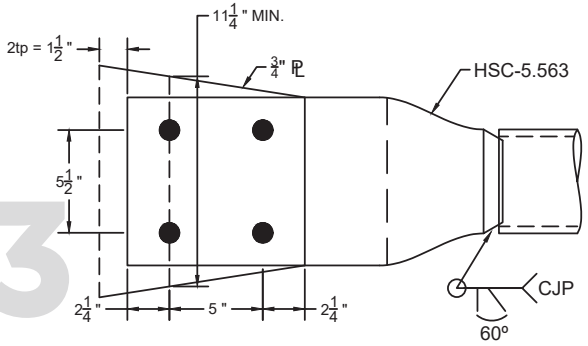
5-1"-490

LRFD 386 kip
ASD 258 kip



4-1"-490

LRFD 273 kip
ASD 181.9 kip



HSC-6.625

ANSI/AISC 341-22

$$\frac{D}{t} \leq \frac{0.053 \cdot E}{R_y F_y}$$

ASTM A500
Grade B

Fy = 46 ksi

Ry·Fy = 64 ksi

thus D/t ≤ 23.9

Shape	Wall Thickness, t		D/t	Area, A	Ry·Fy·A	BEARING-TYPE CONNECTIONS[1] Detail Number					
						Group 120 (eg. A325)			Group 150 (eg. A490)		
	Nominal	Design[2]				Bolt Size			Bolt Size		
	in.	in.		in.^{2}	kip	3/4	7/8	1	3/4	7/8	1
HSS 6.625	0.500	0.465	14.2	9.00	580	14-3/4"-325	10-7/8"-325	8-1"-325	12-3/4"-490	9-7/8"-490	8-1"-490
	0.432	0.402	16.5	7.85	506	12-3/4"-325	9-7/8"-325	8-1"-325	10-3/4"-490	7-7/8"-490	6-1"-490
	0.375	0.349	19.0	6.88	443	11-3/4"-325	8-7/8"-325	6-1"-325	9-3/4"-490	7-7/8"-490	5-1"-490
	0.312	0.290	22.8	5.77	372	9-3/4"-325	7-7/8"-325	5-1"-325	7-3/4"-490	6-7/8"-490	5-1"-490

1. Connections must have pretensioned high-strength bolts. The following are suggested bolt lengths:
   4" Long bolt for 3/4" and 7/8" Group 120 (eg. A325) or Group 150 (eg. A490)
   4¼" Long bolt for 1" Group 120 (eg. A325) or Group 150 (eg. A490)
2. Design wall thickness for ASTM A500 HSS sections taken as $0.93 \cdot t_{nominal}$.

X Connector tabs not large enough to accommodate the number of bolts required

HSC-6.625

ANSI/AISC 341-22

$$\frac{D}{t} \leq \frac{0.053 \cdot E}{R_y F_y}$$

ASTM A500
Grade C

Fy = 50 ksi

Ry·Fy = 65 ksi

thus D/t ≤ 23.6

Shape	Wall Thickness, t		D/t	Area, A	Ry·Fy·A	BEARING-TYPE CONNECTIONS[1] Detail Number					
						Group 120 (eg. A325)			Group 150 (eg. A490)		
	Nominal	Design[2]				Bolt Size			Bolt Size		
	in.	in.		in.^{2}	kip	3/4	7/8	1	3/4	7/8	1
HSS 6.625	0.500	0.465	14.2	9.00	585	14-3/4"-325	10-7/8"-325	9-1"-325	12-3/4"-490	9-7/8"-490	8-1"-490
	0.432	0.402	16.5	7.85	511	12-3/4"-325	9-7/8"-325	8-1"-325	10-3/4"-490	7-7/8"-490	6-1"-490
	0.375	0.349	19.0	6.88	447	11-3/4"-325	8-7/8"-325	6-1"-325	9-3/4"-490	7-7/8"-490	5-1"-490
	0.312	0.290	22.8	5.77	375	9-3/4"-325	7-7/8"-325	5-1"-325	7-3/4"-490	6-7/8"-490	5-1"-490

1. Connections must have pretensioned high-strength bolts. The following are suggested bolt lengths:
   4" Long bolt for 3/4" and 7/8" Group 120 (eg. A325) or Group 150 (eg. A490)
   4¼" Long bolt for 1" Group 120 (eg. A325) or Group 150 (eg. A490)
2. Design wall thickness for ASTM A500 HSS sections taken as $0.93 \cdot t_{nominal}$.

X Connector tabs not large enough to accommodate the number of bolts required

HSC-6.625

ANSI/AISC 341-22

$$\frac{D}{t} \leq \frac{0.053 \cdot E}{R_y F_y}$$

ASTM A53
Grade B

Fy = 35 ksi

Ry·Fy = 56 ksi

thus D/t ≤ 27.4

Shape	Wall Thickness, t		D/t	Area, A	Ry·Fy·A	BEARING-TYPE CONNECTIONS[1] Detail Number					
						Group 120 (eg. A325)			Group 150 (eg. A490)		
	Nominal	Design[2]				Bolt Size			Bolt Size		
	in.	in.		in.^{2}	kip	3/4	7/8	1	3/4	7/8	1
Pipe 6											
XS	0.432	0.402	16.5	7.85	440	12-3/4"-325	9-7/8"-325	8-1"-325	9-3/4"-490	7-7/8"-490	5-1"-490
STD	0.280	0.260	25.4	5.21	292	7-3/4"-325	5-7/8"-325	4-1"-325	6-3/4"-490	4-7/8"-490	4-1"-490

1. Connections must have pretensioned high-strength bolts. The following are suggested bolt lengths:
   4" Long bolt for 3/4" and 7/8" Group 120 (eg. A325) or Group 150 (eg. A490)
   4¼" Long bolt for 1" Group 120 (eg. A325) or Group 150 (eg. A490)
2. Design wall thickness for ASTM A53 Pipe sections taken as $0.93 \cdot t_{nominal}$.

X Connector tabs not large enough to accommodate the number of bolts required

HSC-6.625

ANSI/AISC 341-22

$$\frac{D}{t} \leq \frac{0.053 \cdot E}{R_y F_y}$$

ASTM A1085
Grade A

Fy = 50 ksi

Ry·Fy = 63 ksi

thus D/t ≤ 24.6

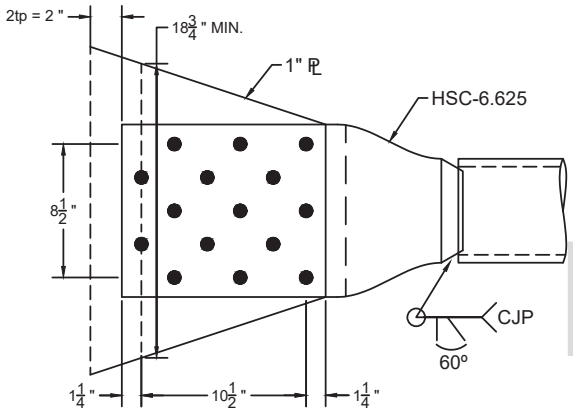
Shape	Wall Thickness, t		D/t	Area, A	Ry·Fy·A	BEARING-TYPE CONNECTIONS[1] Detail Number					
						Group 120 (eg. A325)			Group 150 (eg. A490)		
	Nominal	Design[2]				Bolt Size			Bolt Size		
	in.	in.		in.^{2}	kip	3/4	7/8	1	3/4	7/8	1
HSS 6.625	0.500	0.500	13.3	9.62	601	15-3/4"-325	11-7/8"-325	9-1"-325	12-3/4"-490	9-7/8"-490	8-1"-490
	0.432	0.432	15.3	8.40	525	14-3/4"-325	10-7/8"-325	8-1"-325	11-3/4"-490	8-7/8"-490	6-1"-490
	0.375	0.375	17.7	7.36	460	12-3/4"-325	9-7/8"-325	8-1"-325	9-3/4"-490	7-7/8"-490	6-1"-490
	0.312	0.312	21.2	6.19	387	11-3/4"-325	7-7/8"-325	6-1"-325	8-3/4"-490	6-7/8"-490	5-1"-490
	0.280	0.280	23.7	5.58	349	9-3/4"-325	6-7/8"-325	5-1"-325	7-3/4"-490	5-7/8"-490	4-1"-490

1. Connections must have pretensioned high-strength bolts. The following are suggested bolt lengths:
   4" Long bolt for 3/4" and 7/8" Group 120 (eg. A325) or Group 150 (eg. A490)
   4¼" Long bolt for 1" Group 120 (eg. A325) or Group 150 (eg. A490)
2. Design wall thickness for ASTM A1085 HSS sections taken as $1.0 \cdot t_{nominal}$.

X Connector tabs not large enough to accommodate the number of bolts required

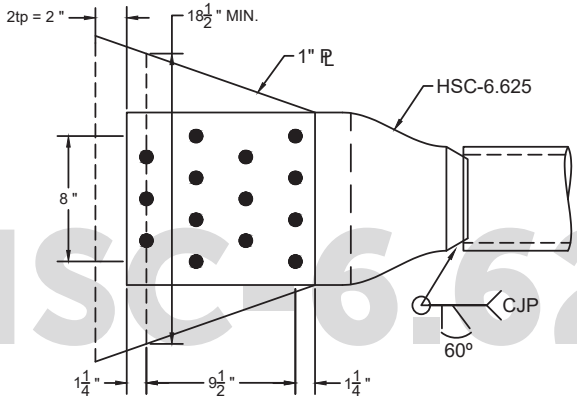
15-3/4"-325

LRFD 608 kip
ASD 404 kip



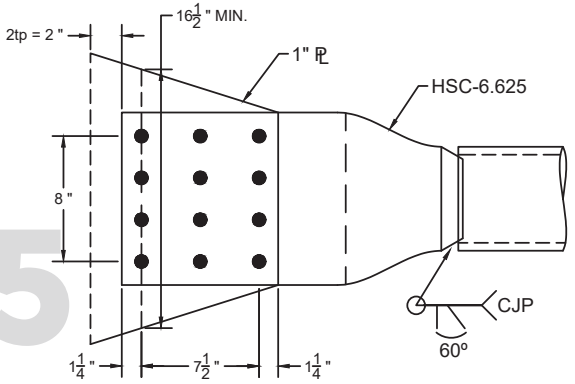
14-3/4"-325

LRFD 599 kip
ASD 399 kip



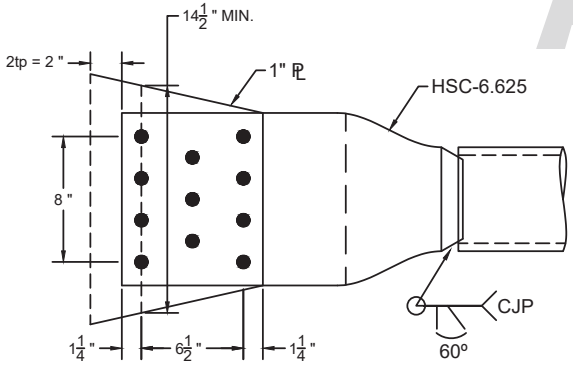
12-3/4"-325

LRFD 517 kip
ASD 345 kip



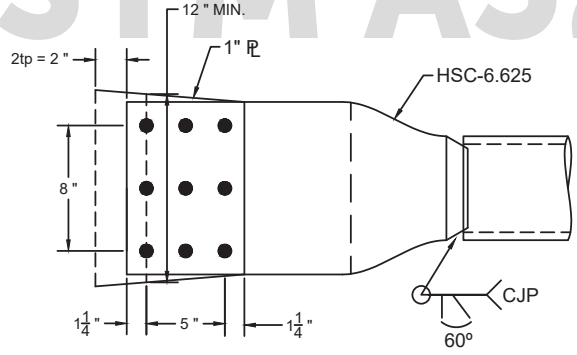
11-3/4"-325

LRFD 468 kip
ASD 312 kip



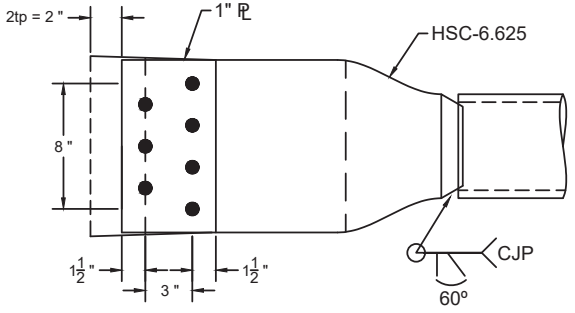
9-3/4"-325

LRFD 389 kip
ASD 259 kip



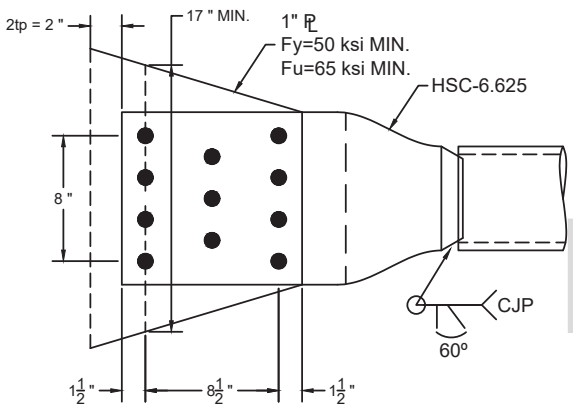
7-3/4"-325

LRFD 315 kip
ASD 210 kip



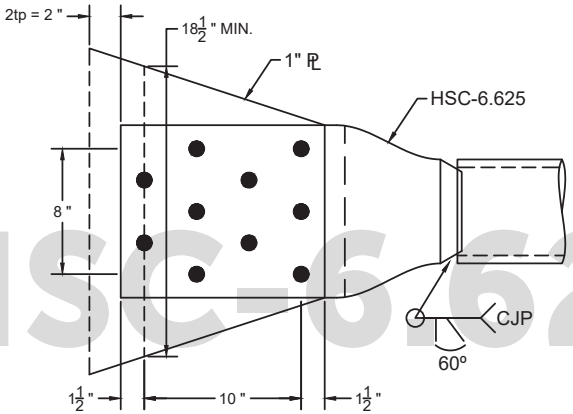
11-7/8"-325

LRFD 607 kip
ASD 404 kip



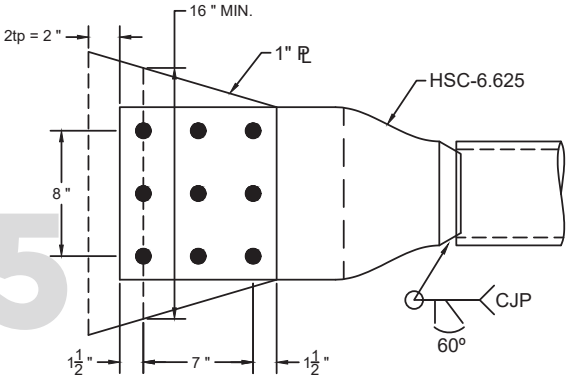
10-7/8"-325

LRFD 591 kip
ASD 394 kip



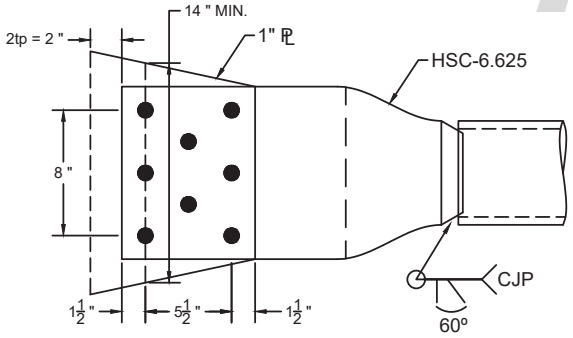
9-7/8"-325

LRFD 518 kip
ASD 345 kip



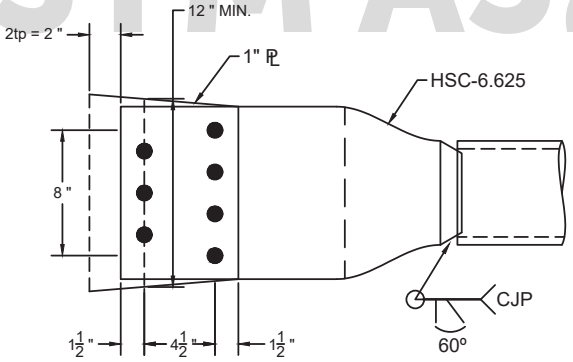
8-7/8"-325

LRFD 454 kip
ASD 302 kip



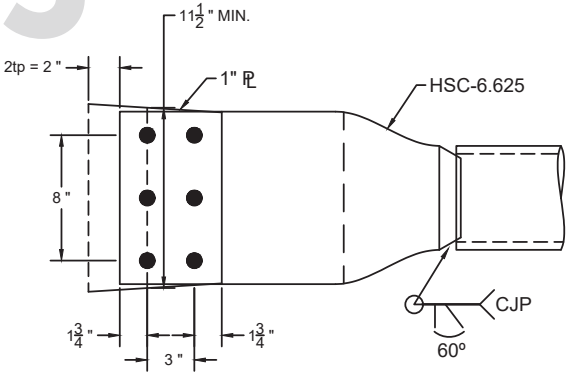
7-7/8"-325

LRFD 389 kip
ASD 259 kip



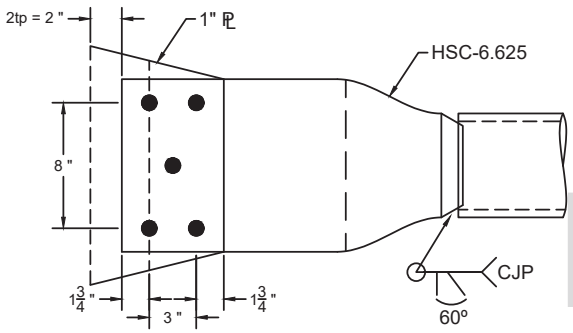
6-7/8"-325

LRFD 368 kip
ASD 245 kip



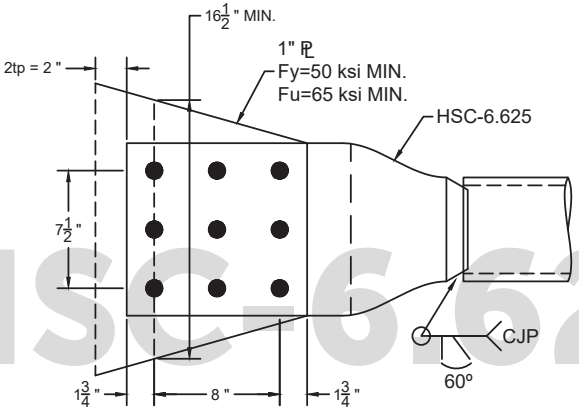
5-7/8"-325

LRFD 307 kip
ASD 204 kip



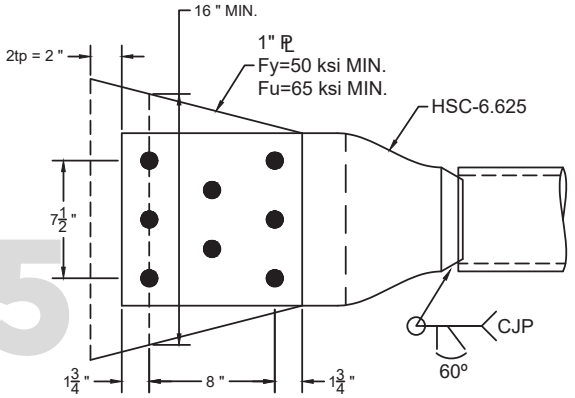
9-1"-325

LRFD 606 kip
ASD 404 kip



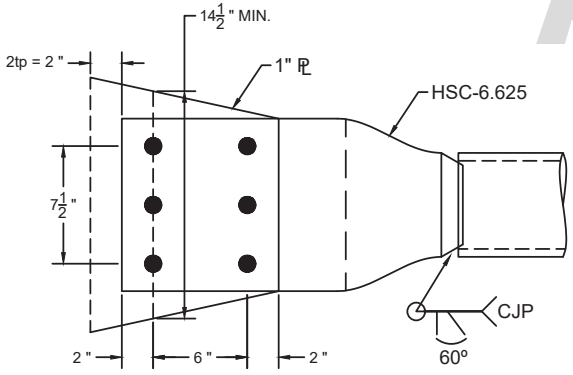
8-1"-325

LRFD 587 kip
ASD 391 kip



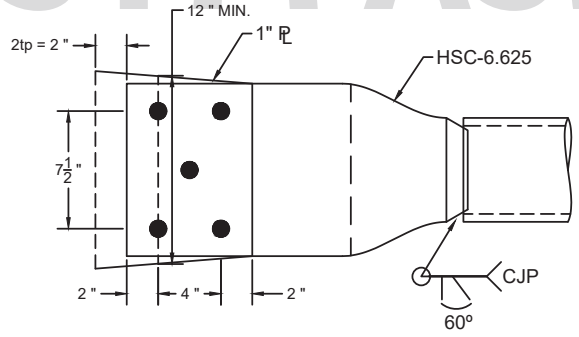
6-1"-325

LRFD 463 kip
ASD 308 kip



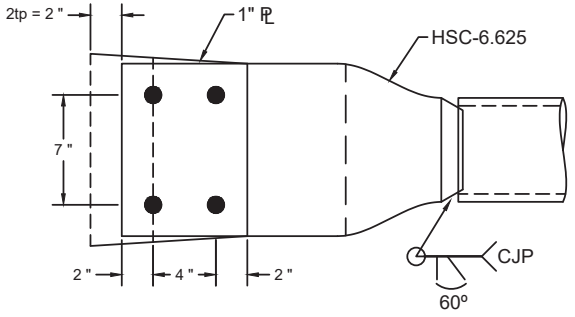
5-1"-325

LRFD 389kip
ASD 259 kip



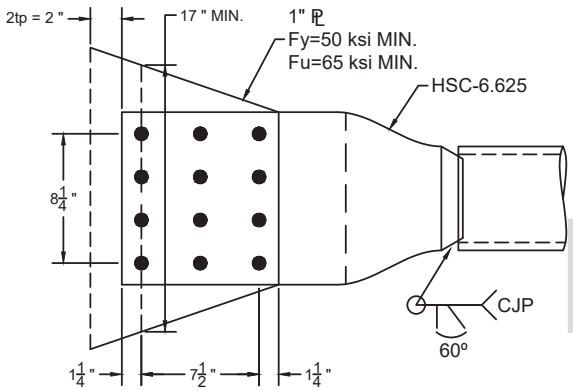
4-1"-325

LRFD 310 kip
ASD 207 kip



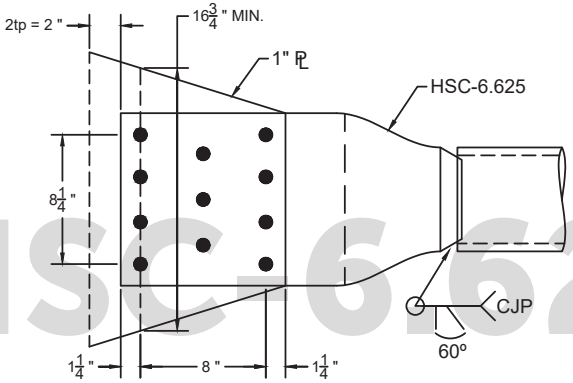
12-3/4"-490

LRFD 621 kip
ASD 414 kip



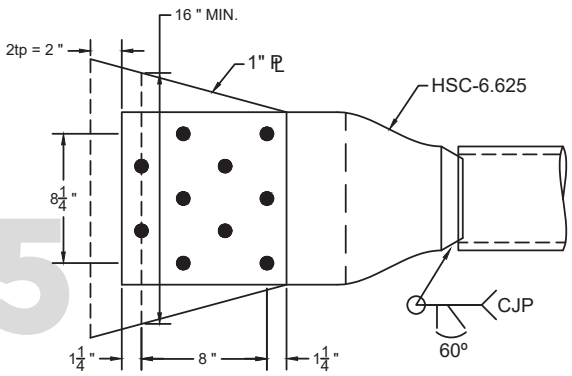
11-3/4"-490

LRFD 543 kip
ASD 361 kip



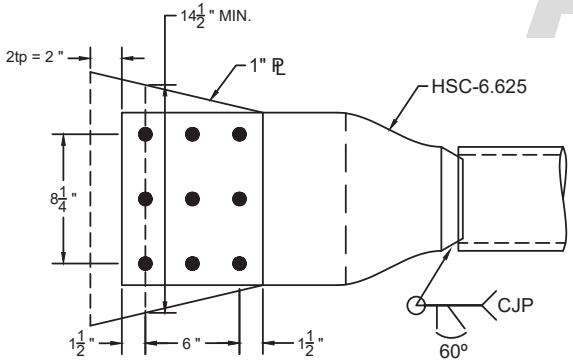
10-3/4"-490

LRFD 518 kip
ASD 345 kip



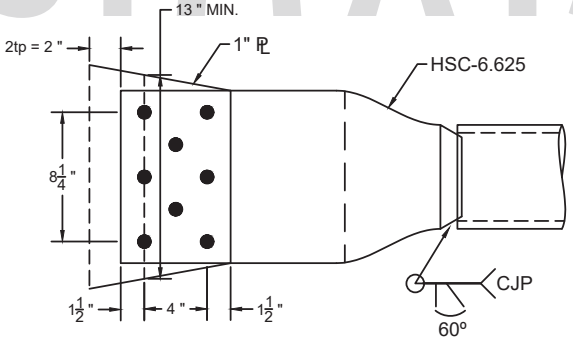
9-3/4"-490

LRFD 470 kip
ASD 313 kip



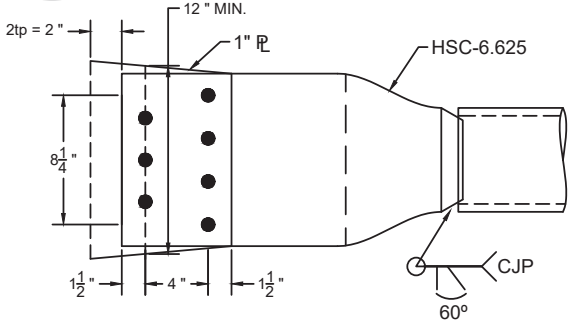
8-3/4"-490

LRFD 417 kip
ASD 277 kip



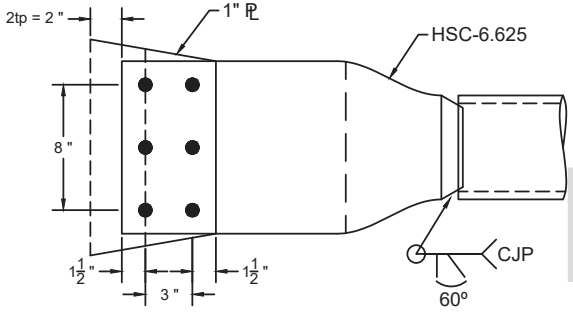
7-3/4"-490

LRFD 389 kip
ASD 259 kip



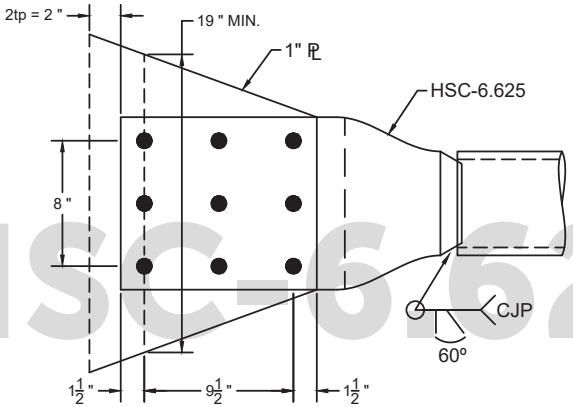
6-3/4"-490

LRFD 334 kip
ASD 223 kip



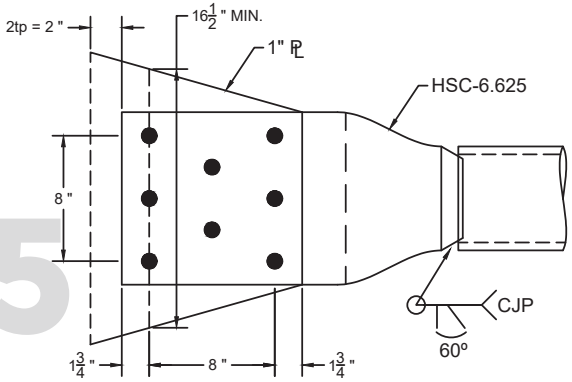
9-7/8"-490

LRFD 615 kip
ASD 409 kip



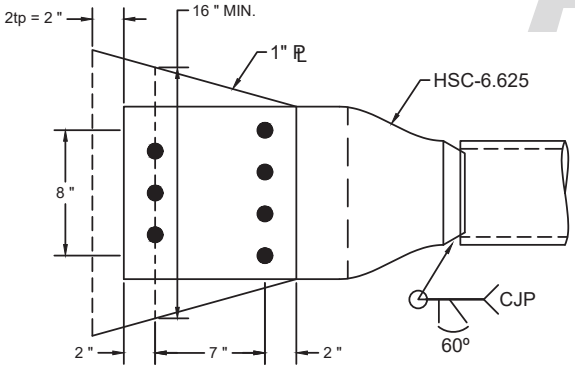
8-7/8"-490

LRFD 535 kip
ASD 356 kip



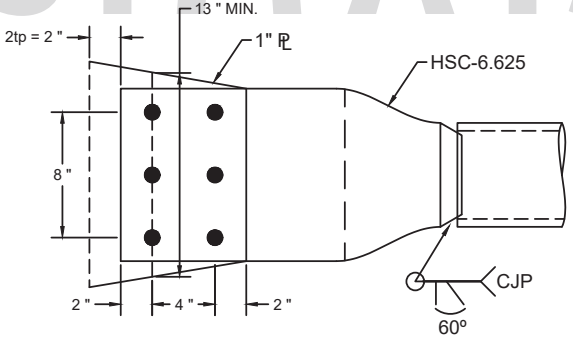
7-7/8"-490

LRFD 518 kip
ASD 345 kip



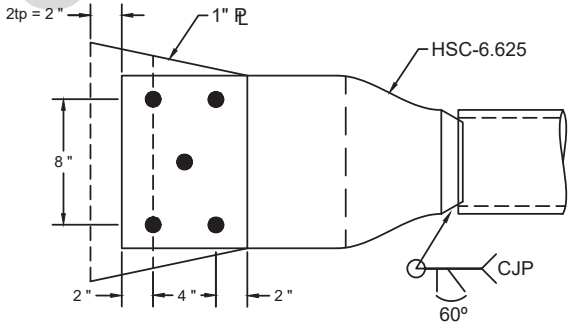
6-7/8"-490

LRFD 409 kip
ASD 272 kip



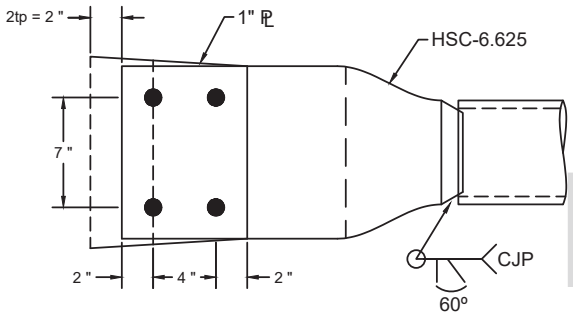
5-7/8"-490

LRFD 356 kip
ASD 237 kip



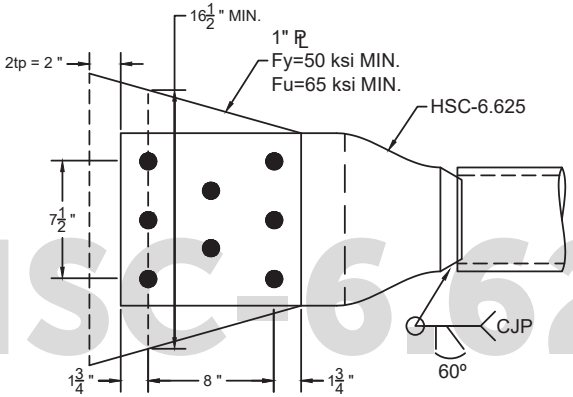
4-7/8"-490

LRFD 303 kip
ASD 202 kip



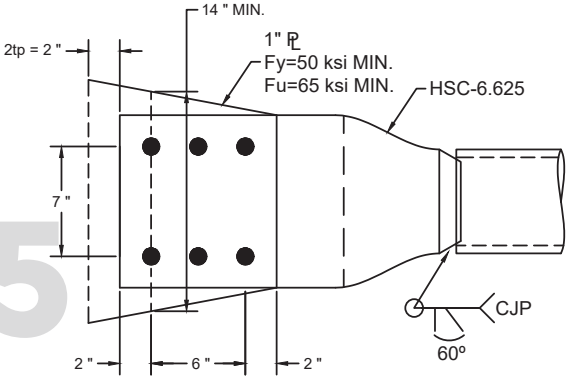
8-1"-490

LRFD 606 kip
ASD 404 kip



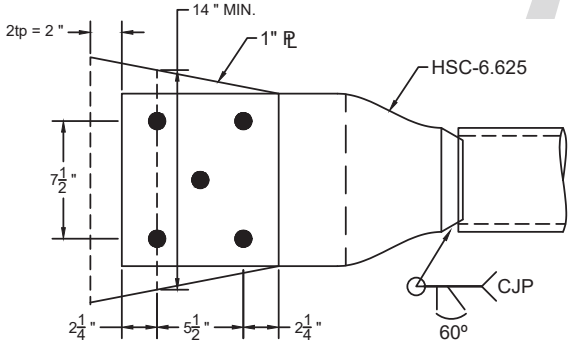
6-1"-490

LRFD 555 kip
ASD 370 kip



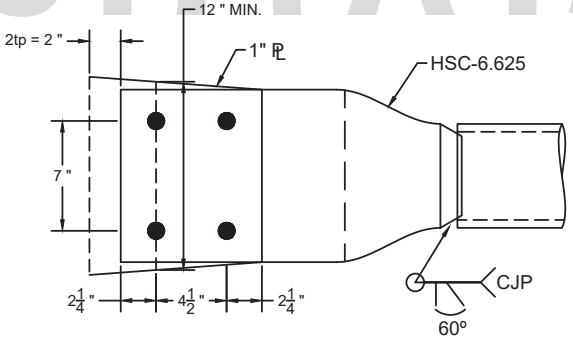
5-1"-490

LRFD 449 kip
ASD 299 kip



4-1"-490

LRFD 374 kip
ASD 249 kip



HSC-8.625

ANSI/AISC 341-22

$$\frac{D}{t} \leq \frac{0.053 \cdot E}{R_y F_y}$$

ASTM A500
Grade B

Fy = 46 ksi

Ry·Fy = 64 ksi

thus D/t ≤ 23.9

Shape	Wall Thickness, t		D/t	Area, A	Ry·Fy·A	BEARING-TYPE CONNECTIONS[1] Detail Number					
						Group 120 (eg. A325)			Group 150 (eg. A490)		
	Nominal	Design[2]				Bolt Size			Bolt Size		
	in.	in.		in.^{2}	kip	1	1⅛	1¼	1	1⅛	1¼
HSS 8.625	0.625 0.500	0.581 0.465	14.8 18.5	14.69 11.92	946 768	13-1"-325 10-1"-325	10-1 1/8"-325 8-1 1/8"-325	8-1 1/4"-325 7-1 1/4"-325	10-1"-490 8-1"-490	8-1 1/8"-490 7-1 1/8"-490	7-1 1/4"-490 6-1 1/4"-490

1. Connections must have pretensioned high-strength bolts. The following are suggested bolt lengths:
   4¾" Long bolt for 1" Group 120 (eg. A325) or Group 150 (eg. A490)
   5" Long bolt for 1¼" and 1⅛ Group 120 (eg. A325) or Group 150 (eg. A490)
2. Design wall thickness for ASTM A500 HSS sections taken as $0.93 \cdot t_{nominal}$.

X Connector tabs not large enough to accommodate the number of bolts required

HSC-8.625

ANSI/AISC 341-22

$$\frac{D}{t} \leq \frac{0.053 \cdot E}{R_y F_y}$$

ASTM A500
Grade C

Fy = 50 ksi

Ry·Fy = 65 ksi

thus D/t ≤ 23.6

Shape	Wall Thickness, t		D/t	Area, A	Ry·Fy·A	BEARING-TYPE CONNECTIONS[1] Detail Number					
						Group 120 (eg. A325)			Group 150 (eg. A490)		
	Nominal	Design[2]				Bolt Size			Bolt Size		
	in.	in.		in.^{2}	kip	1	1⅛	1¼	1	1⅛	1¼
HSS 8.625	0.625 0.500	0.581 0.465	14.8 18.5	14.69 11.92	955 775	14-1"-325 11-1"-325	10-1 1/8"-325 9-1 1/8"-325	8-1 1/4"-325 7-1 1/4"-325	10-1"-490 9-1"-490	8-1 1/8"-490 7-1 1/8"-490	7-1 1/4"-490 6-1 1/4"-490

1. Connections must have pretensioned high-strength bolts. The following are suggested bolt lengths:
   4¾" Long bolt for 1" Group 120 (eg. A325) or Group 150 (eg. A490)
   5" Long bolt for 1¼" and 1⅛ Group 120 (eg. A325) or Group 150 (eg. A490)
2. Design wall thickness for ASTM A500 HSS sections taken as $0.93 \cdot t_{nominal}$.

X Connector tabs not large enough to accommodate the number of bolts required

HSC-8.625

ANSI/AISC 341-22

$$\frac{D}{t} \leq \frac{0.053 \cdot E}{R_y F_y}$$

ASTM A53
Grade B

Fy = 35 ksi

Ry·Fy = 56 ksi

thus D/t ≤ 27.4

Shape	Wall Thickness, t		D/t	Area, A	Ry·Fy·A	BEARING-TYPE CONNECTIONS[1] Detail Number					
						Group 120 (eg. A325)			Group 150 (eg. A490)		
	Nominal	Design[2]				Bolt Size			Bolt Size		
	in.	in.		in.^{2}	kip	1	1⅛	1¼	1	1⅛	1¼
Pipe 8 XS	0.500	0.465	18.5	11.92	668	10-1"-325	8-1 1/8"-325	7-1 1/4"-325	8-1"-490	6-1 1/8"-490	5-1 1/4"-490

1. Connections must have pretensioned high-strength bolts. The following are suggested bolt lengths:
   4¾" Long bolt for 1" Group 120 (eg. A325) or Group 150 (eg. A490)
   5" Long bolt for 1¼" and 1⅛ Group 120 (eg. A325) or Group 150 (eg. A490)
2. Design wall thickness for ASTM A53 Pipe sections taken as $0.93 \cdot t_{nominal}$.

X Connector tabs not large enough to accommodate the number of bolts required

HSC-8.625

ANSI/AISC 341-22

$$\frac{D}{t} \leq \frac{0.053 \cdot E}{R_y F_y}$$

ASTM A1085
Grade A

Fy = 50 ksi

Ry·Fy = 63 ksi

thus D/t ≤ 24.6

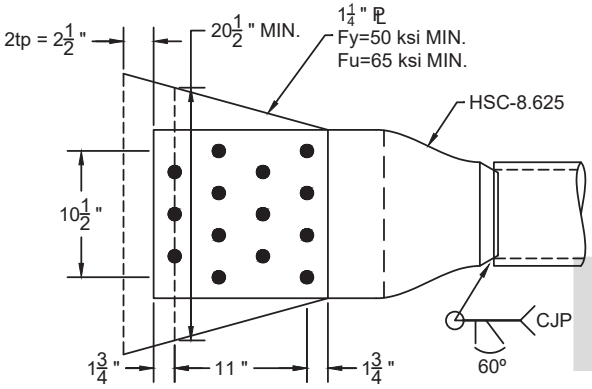
Shape	Wall Thickness, t		D/t	Area, A	Ry·Fy·A	BEARING-TYPE CONNECTIONS[1] Detail Number					
						Group 120 (eg. A325)			Group 150 (eg. A490)		
	Nominal	Design[2]				Bolt Size			Bolt Size		
	in.	in.		in.^{2}	kip	1	1⅛	1¼	1	1⅛	1¼
HSS 8.625	0.625	0.625	13.8	15.71	982	14-1"-325	11-1 1/8"-325	9-1 1/4"-325	11-1"-490	9-1 1/8"-490	7-1 1/4"-490
	0.500	0.500	17.3	12.76	798	11-1"-325	9-1 1/8"-325	7-1 1/4"-325	9-1"-490	7-1 1/8"-490	6-1 1/4"-490
	0.375	0.375	23.0	9.72	607	8-1"-325	7-1 1/8"-325	6-1 1/4"-325	7-1"-490	5-1 1/8"-490	4-1 1/4"-490

1. Connections must have pretensioned high-strength bolts. The following are suggested bolt lengths:
   4¾" Long bolt for 1" Group 120 (eg. A325) or Group 150 (eg. A490)
   5" Long bolt for 1¼" and 1⅛ Group 120 (eg. A325) or Group 150 (eg. A490)
2. Design wall thickness for ASTM A1085 HSS sections taken as $1.0 \cdot t_{nominal}$.

X Connector tabs not large enough to accommodate the number of bolts required

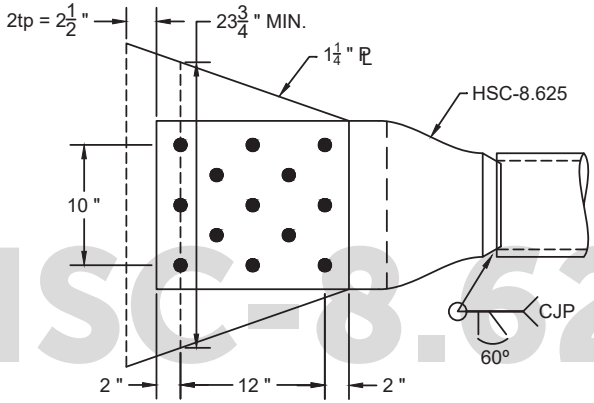
14-1"-325

LRFD 985 kip
ASD 657 kip



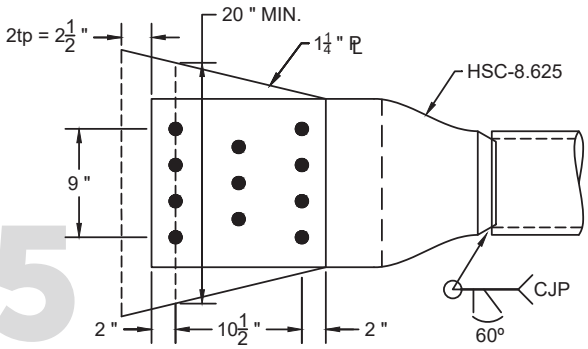
13-1"-325

LRFD 962 kip
ASD 640 kip



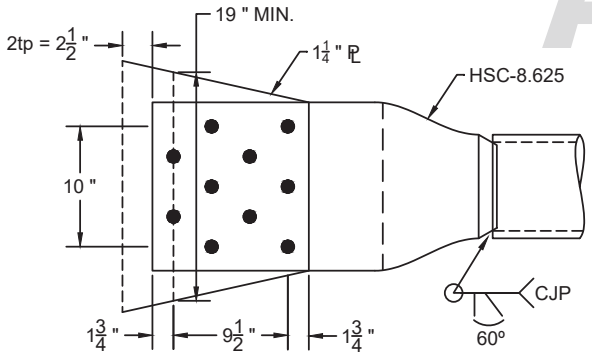
11-1"-325

LRFD 802 kip
ASD 535 kip



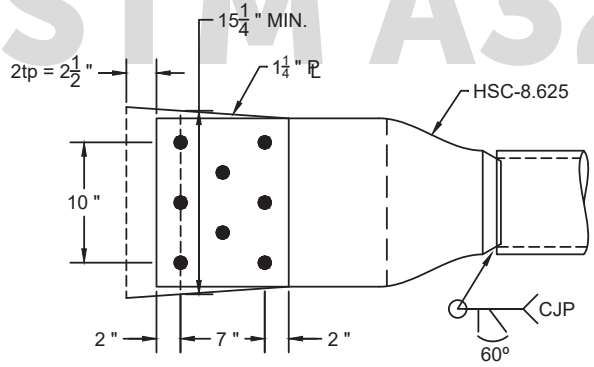
10-1"-325

LRFD 770 kip
ASD 512 kip



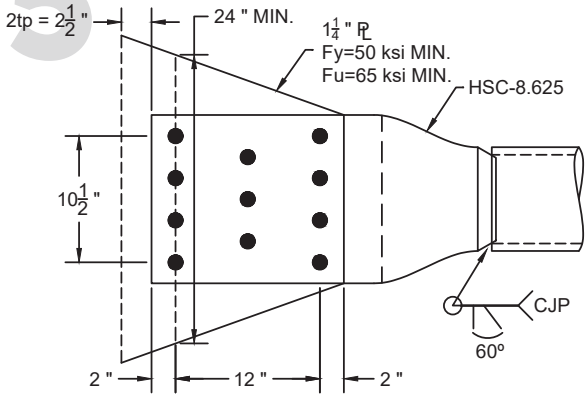
8-1"-325

LRFD 618 kip
ASD 411 kip



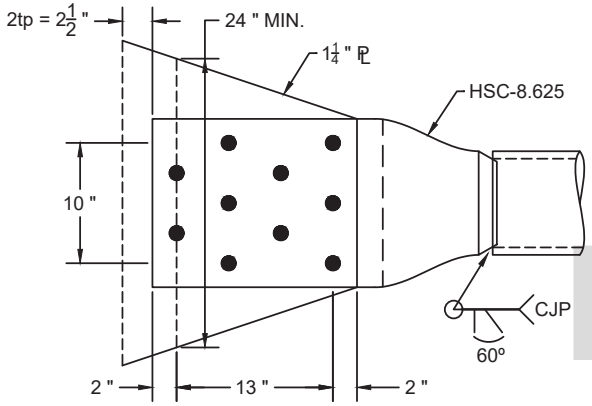
11-1 1/8"-325

LRFD 1073 kip
ASD 715 kip



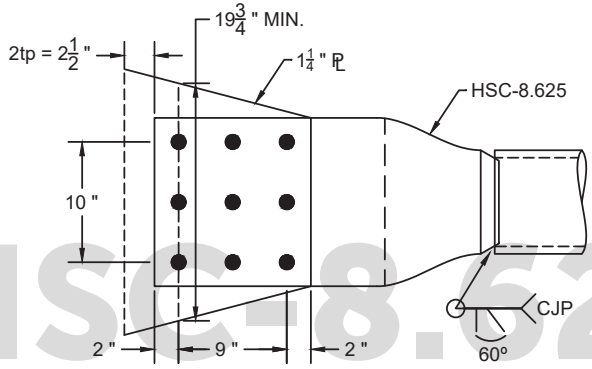
10-1 1/8"-325

LRFD 996 kip
ASD 644 kip



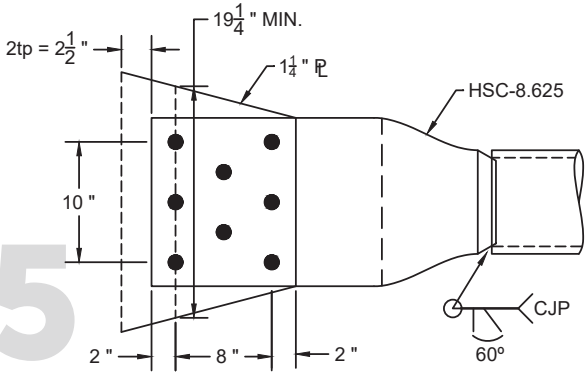
9-1 1/8"-325

LRFD 800 kip
ASD 532 kip



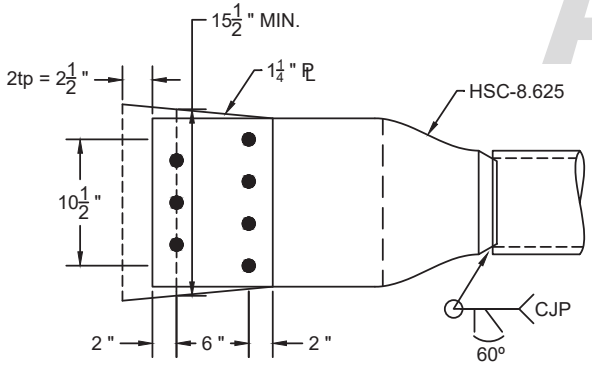
8-1 1/8"-325

LRFD 776 kip
ASD 517 kip



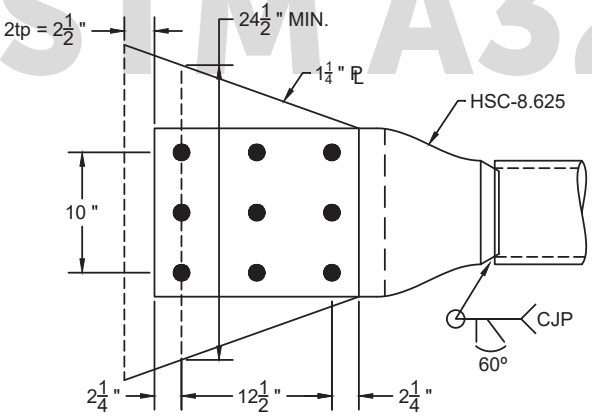
7-1 1/8"-325

LRFD 626 kip
ASD 417 kip



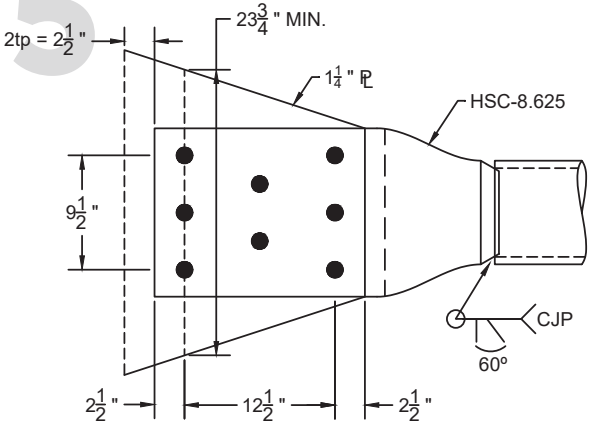
9-1 1/4"-325

LRFD 985 kip
ASD 657 kip



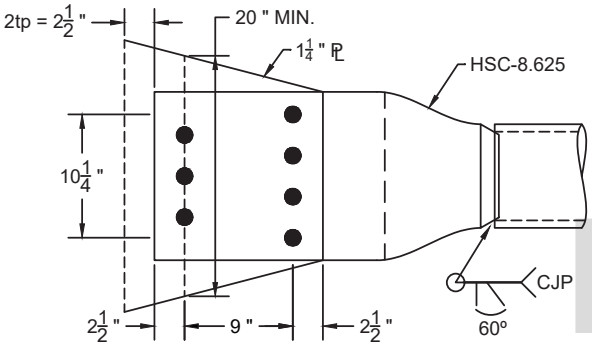
8-1 1/4"-325

LRFD 962 kip
ASD 640 kip



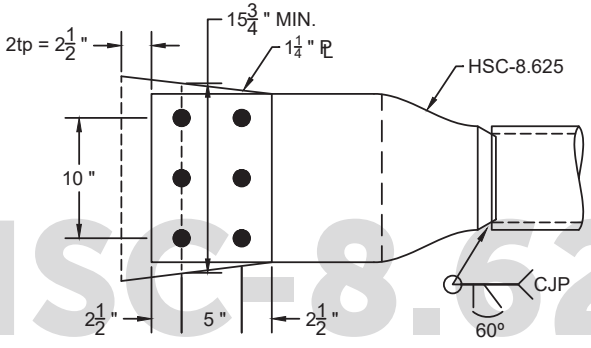
7-1 1/4"-325

LRFD 810 kip
ASD 539 kip



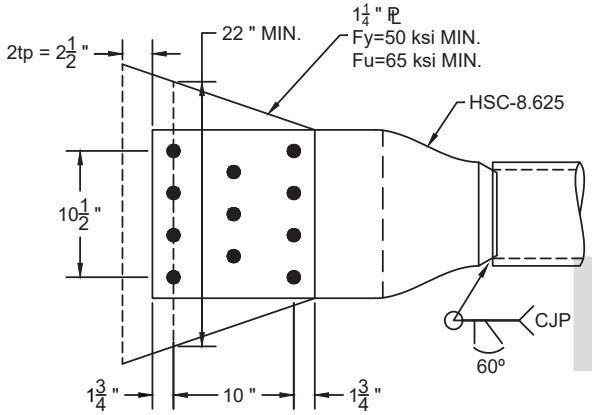
6-1 1/4"-325

LRFD 619 kip
ASD 413 kip



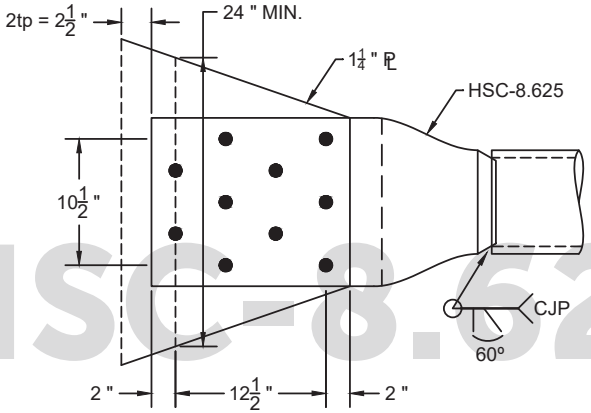
11-1"-490

LRFD 997 kip
ASD 664 kip



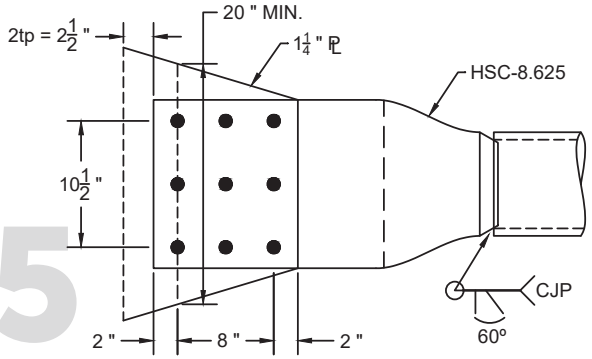
10-1"-490

LRFD 972 kip
ASD 647 kip



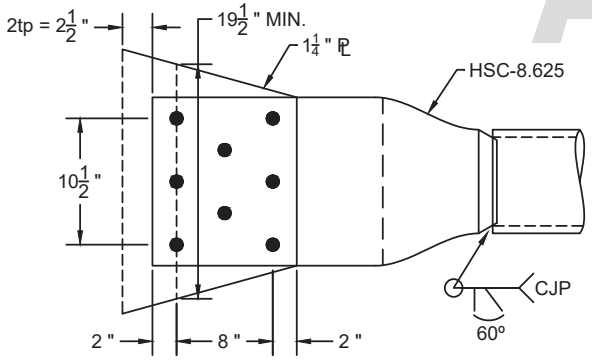
9-1"-490

LRFD 799 kip
ASD 532 kip



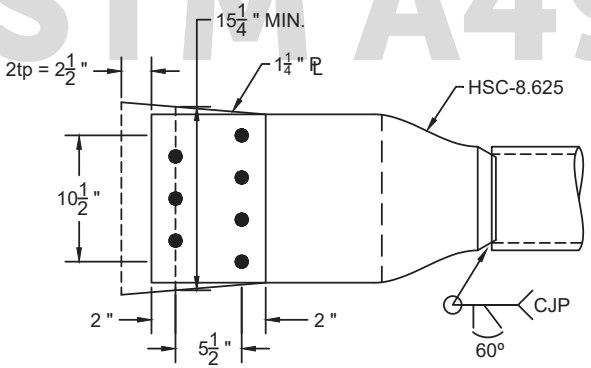
8-1"-490

LRFD 776 kip
ASD 517 kip



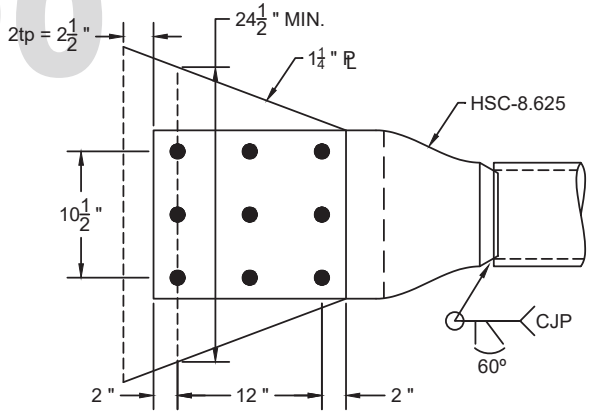
7-1"-490

LRFD 618 kip
ASD 411 kip



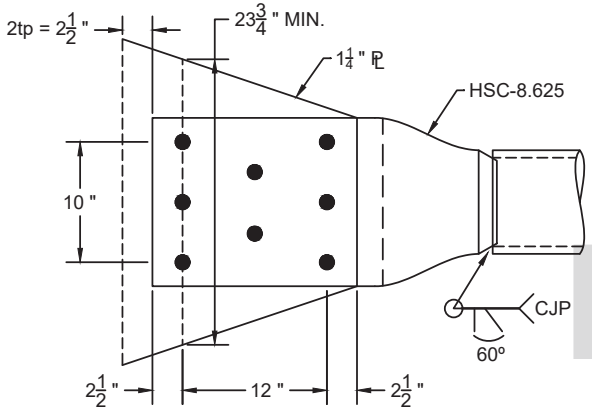
9-1 1/8"-490

LRFD 986 kip
ASD 656kip



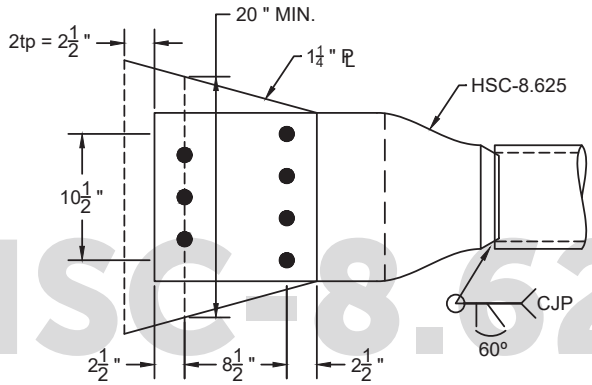
8-1 1/8"-490

LRFD 962 kip
ASD 640 kip



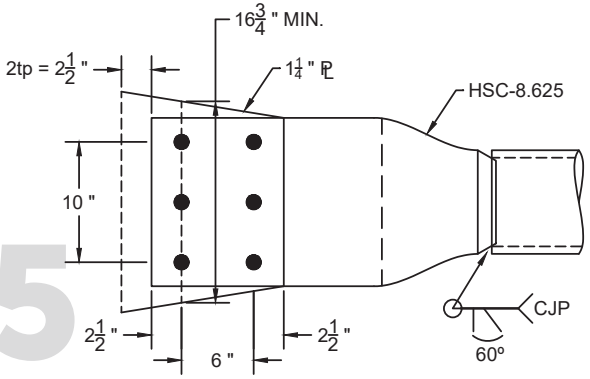
7-1 1/8"-490

LRFD 810 kip
ASD 539 kip



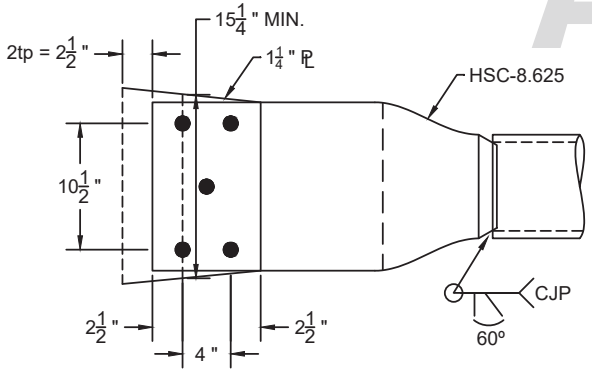
6-1 1/8"-490

LRFD 678 kip
ASD 451 kip



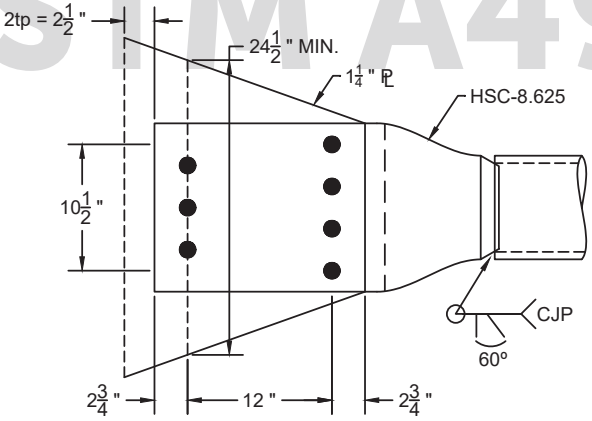
5-1 1/8"-490

LRFD 612 kip
ASD 407 kip



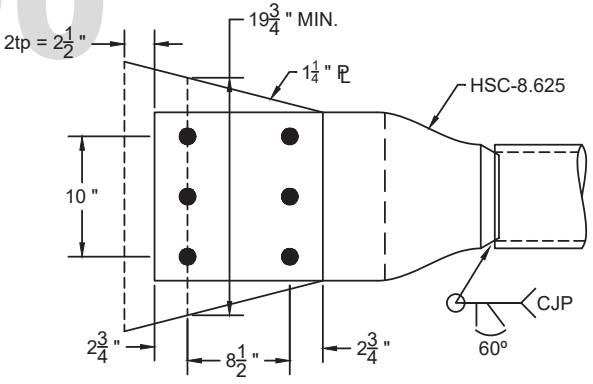
7-1 1/4"-490

LRFD 986 kip
ASD 656 kip



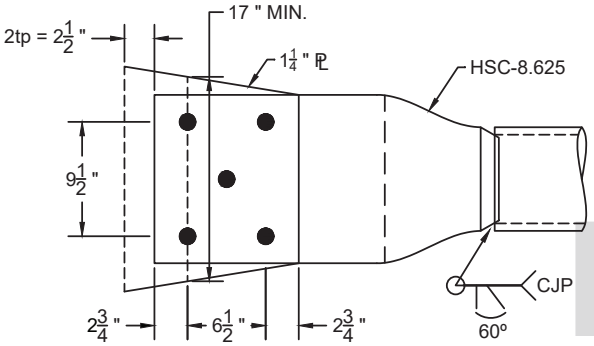
6-1 1/4"-490

LRFD 800 kip
ASD 532 kip



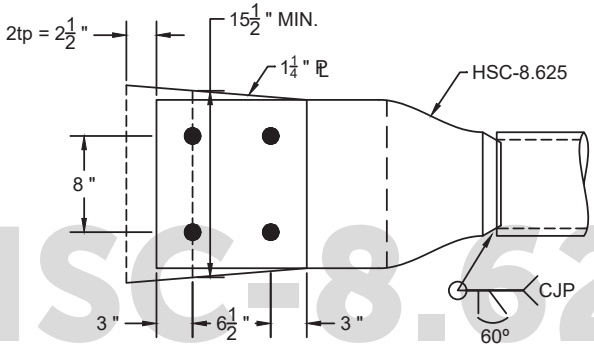
5-1 1/4"-490

LRFD 689 kip
ASD 458 kip



4-1 1/4"-490

LRFD 611 kip
ASD 407 kip



HSC-10.75

ANSI/AISC 341-22

$$\frac{D}{t} \leq \frac{0.053 \cdot E}{R_y F_y}$$

ASTM A500
Grade B

Fy = 46 ksi

Ry·Fy = 64 ksi

thus D/t ≤ 23.9

Shape	Wall Thickness, t		D/t	Area, A	Ry·Fy·A	BEARING-TYPE CONNECTIONS[1] Detail Number					
						Group 120 (eg. A325)			Group 150 (eg. A490)		
	Nominal	Design[2]				Bolt Size			Bolt Size		
	in.	in.		in.^{2}	kip	1	1⅛	1¼	1	1⅛	1¼
HSS 10.75	0.625 0.500	0.581 0.465	18.5 23.1	18.57 15.02	1,196 968	16-1"-325 13-1"-325	12-1 1/8"-325 10-1 1/8"-325	10-1 1/4"-325 8-1 1/4"-325	13-1"-490 10-1"-490	10-1 1/8"-490 8-1 1/8"-490	8-1 1/4"-490 7-1 1/4"-490

1. Connections must have pretensioned high-strength bolts. The following are suggested bolt lengths:
   4¾" Long bolt for 1" Group 120 (eg. A325) or Group 150 (eg. A490)
   5" Long bolt for 1¼" and 1⅛ Group 120 (eg. A325) or Group 150 (eg. A490)
2. Design wall thickness for ASTM A500 HSS sections taken as $0.93 \cdot t_{nominal}$.

X Connector tabs not large enough to accommodate the number of bolts required

HSC-10.75

ANSI/AISC 341-22

$$\frac{D}{t} \leq \frac{0.053 \cdot E}{R_y F_y}$$

ASTM A500
Grade C

Fy = 50 ksi

Ry·Fy = 65 ksi

thus D/t ≤ 23.6

Shape	Wall Thickness, t		D/t	Area, A	Ry·Fy·A	BEARING-TYPE CONNECTIONS[1] Detail Number					
						Group 120 (eg. A325)			Group 150 (eg. A490)		
	Nominal	Design[2]				Bolt Size			Bolt Size		
	in.	in.		in.^{2}	kip	1	1⅛	1¼	1	1⅛	1¼
HSS 10.75	0.625 0.500	0.581 0.465	18.5 23.1	18.57 15.02	1,207 977	16-1"-325 13-1"-325	12-1 1/8"-325 10-1 1/8"-325	10-1 1/4"-325 8-1 1/4"-325	13-1"-490 10-1"-490	10-1 1/8"-490 8-1 1/8"-490	8-1 1/4"-490 7-1 1/4"-490

1. Connections must have pretensioned high-strength bolts. The following are suggested bolt lengths:
   4¾" Long bolt for 1" Group 120 (eg. A325) or Group 150 (eg. A490)
   5" Long bolt for 1¼" and 1⅛ Group 120 (eg. A325) or Group 150 (eg. A490)
2. Design wall thickness for ASTM A500 HSS sections taken as $0.93 \cdot t_{nominal}$.

X Connector tabs not large enough to accommodate the number of bolts required

HSC-10.75

ANSI/AISC 341-22

$$\frac{D}{t} \leq \frac{0.053 \cdot E}{R_y F_y}$$

ASTM A53
Grade B

Fy = 35 ksi

Ry·Fy = 56 ksi

thus D/t ≤ 27.4

Shape	Wall Thickness, t		D/t	Area, A	Ry·Fy·A	BEARING-TYPE CONNECTIONS[1] Detail Number					
						Group 120 (eg. A325)			Group 150 (eg. A490)		
	Nominal	Design[2]				Bolt Size			Bolt Size		
	in.	in.		in.^{2}	kip	1	1⅛	1¼	1	1⅛	1¼
Pipe 10 XS	0.500	0.465	23.1	15.02	841	11-1"-325	9-1 1/8"-325	7-1 1/4"-325	9-1"-490	7-1 1/8"-490	6-1 1/4"-490

1. Connections must have pretensioned high-strength bolts. The following are suggested bolt lengths:
   4¾" Long bolt for 1" Group 120 (eg. A325) or Group 150 (eg. A490)
   5" Long bolt for 1¼" and 1⅛ Group 120 (eg. A325) or Group 150 (eg. A490)
2. Design wall thickness for ASTM A53 Pipe sections taken as $0.93 \cdot t_{nominal}$.

X Connector tabs not large enough to accommodate the number of bolts required

HSC-10.75

ANSI/AISC 341-22

$$\frac{D}{t} \leq \frac{0.053 \cdot E}{R_y F_y}$$

ASTM A1085
Grade A

Fy = 50 ksi

Ry·Fy = 63 ksi

thus D/t ≤ 24.6

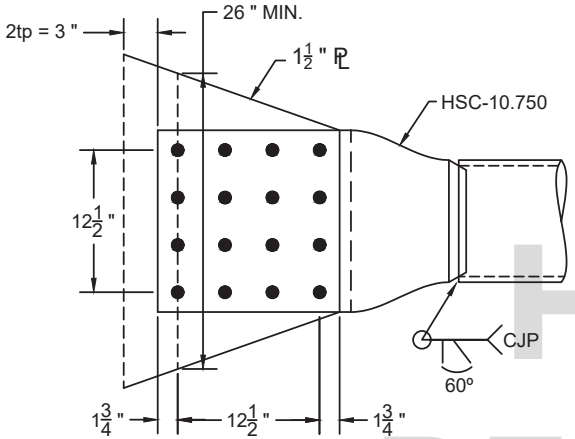
Shape	Wall Thickness, t		D/t	Area, A	Ry·Fy·A	BEARING-TYPE CONNECTIONS[1] Detail Number					
						Group 120 (eg. A325)			Group 150 (eg. A490)		
	Nominal	Design[2]				Bolt Size			Bolt Size		
	in.	in.		in.^{2}	kip	1	1⅛	1¼	1	1⅛	1¼
HSS 10.75	0.625 0.500	0.625 0.500	17.2 21.5	19.88 16.10	1,243 1,006	16-1"-325 13-1"-325	14-1 1/8"-325 10-1 1/8"-325	10-1 1/4"-325 9-1 1/4"-325	13-1"-490 11-1"-490	10-1 1/8"-490 9-1 1/8"-490	9-1 1/4"-490 7-1 1/4"-490

1. Connections must have pretensioned high-strength bolts. The following are suggested bolt lengths:
   4¾" Long bolt for 1" Group 120 (eg. A325) or Group 150 (eg. A490)
   5" Long bolt for 1¼" and 1⅛ Group 120 (eg. A325) or Group 150 (eg. A490)
2. Design wall thickness for ASTM A1085 HSS sections taken as $1.0 \cdot t_{nominal}$.

X Connector tabs not large enough to accommodate the number of bolts required

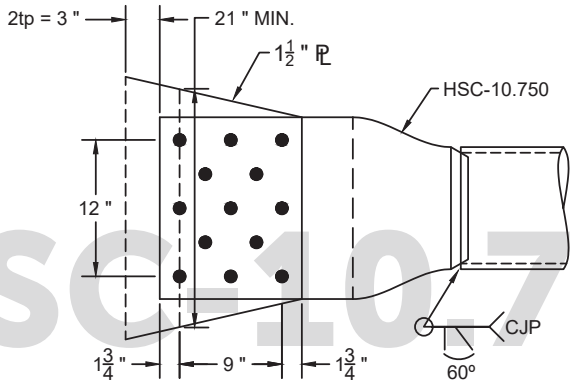
16-1"-325

LRFD 1264 kip
ASD 841 kip



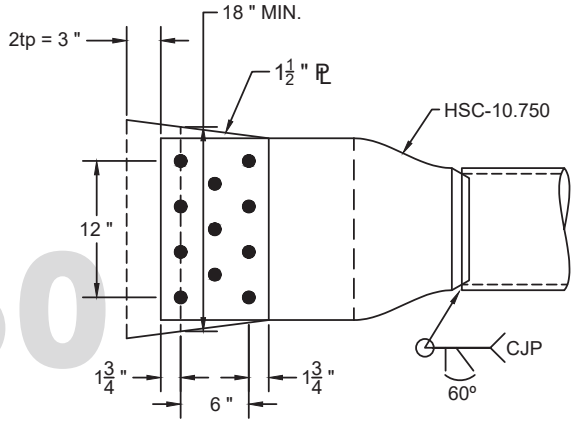
13-1"-325

LRFD 1021 kip
ASD 679 kip



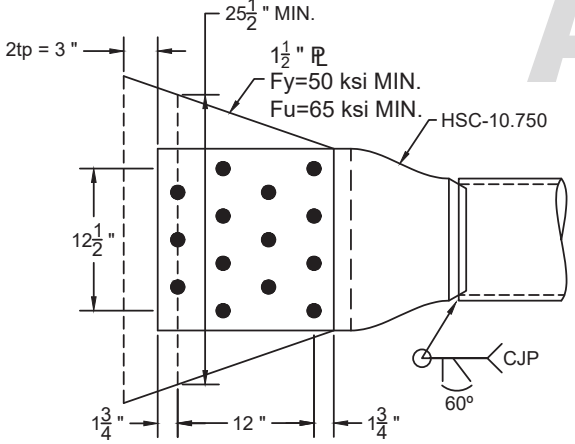
11-1"-325

LRFD 860 kip
ASD 573 kip



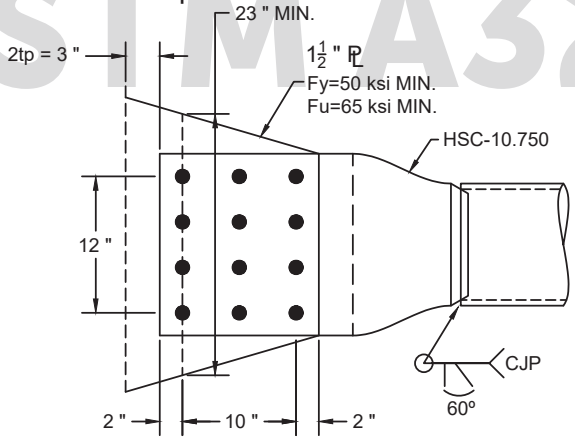
14-1 1/8"-325

LRFD 1404 kip
ASD 936 kip



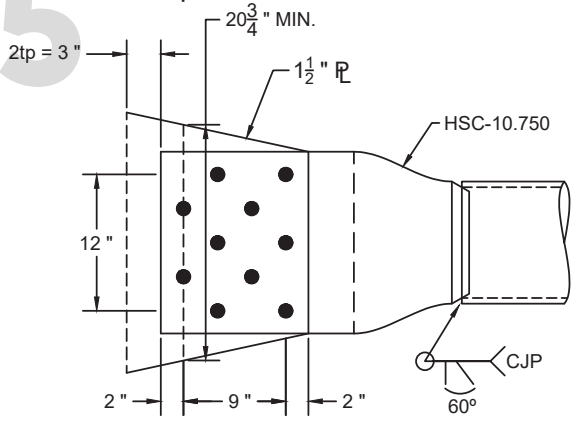
12-1 1/8"-325

LRFD 1217 kip
ASD 811 kip



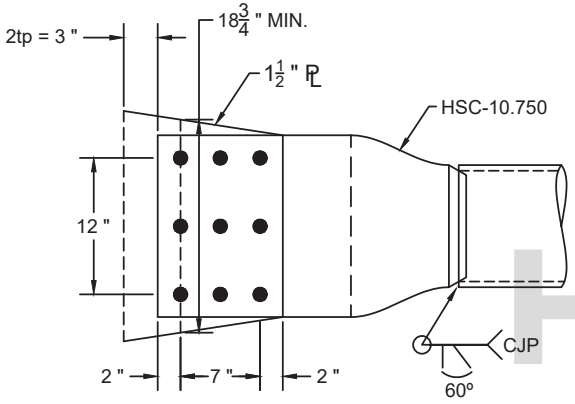
10-1 1/8"-325

LRFD 1008 kip
ASD 671 kip



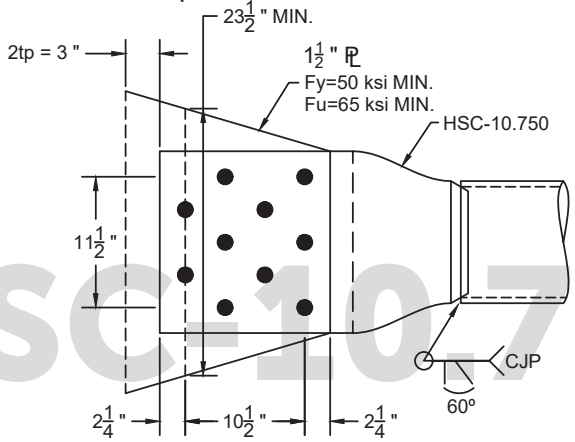
9-1 1/8"-325

LRFD 911 kip
ASD 606 kip



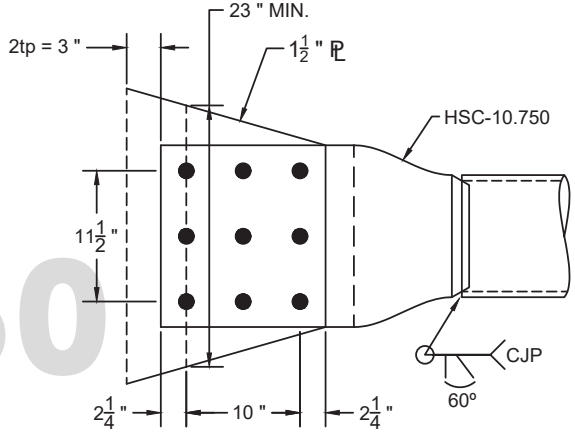
10-1 1/4"-325

LRFD 1252 kip
ASD 834 kip



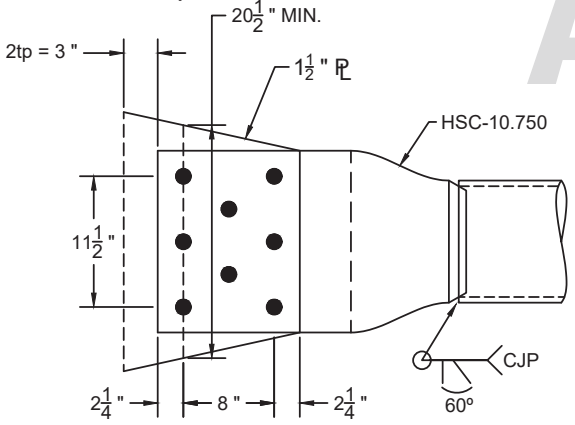
9-1 1/4"-325

LRFD 1118 kip
ASD 744 kip



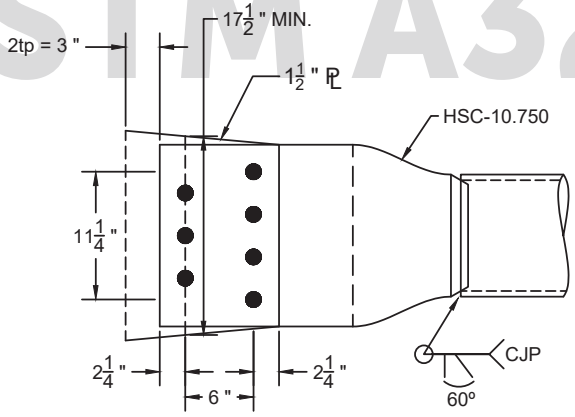
8-1 1/4"-325

LRFD 993 kip
ASD 662 kip



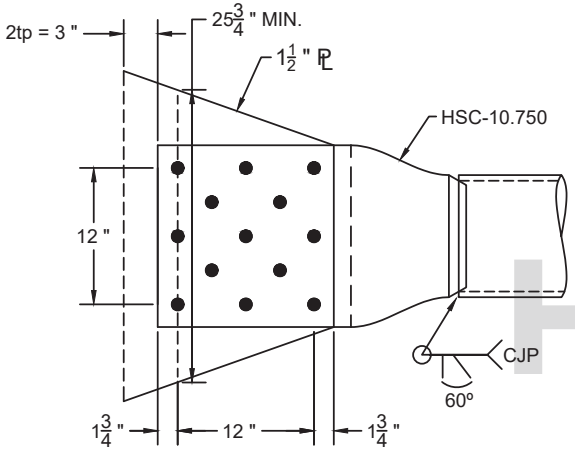
7-1 1/4"-325

LRFD 851 kip
ASD 566 kip



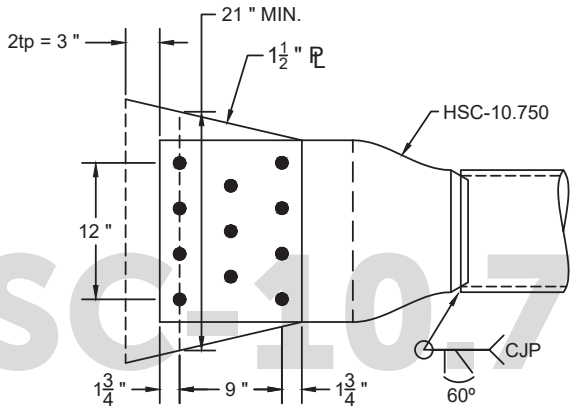
13-1"-490

LRFD 1251 kip
ASD 833 kip



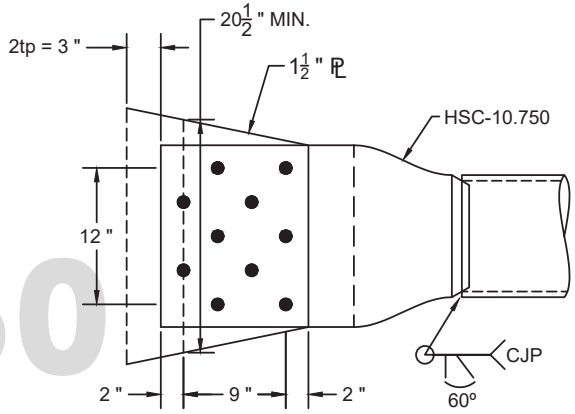
11-1"-490

LRFD 1021 kip
ASD 679 kip



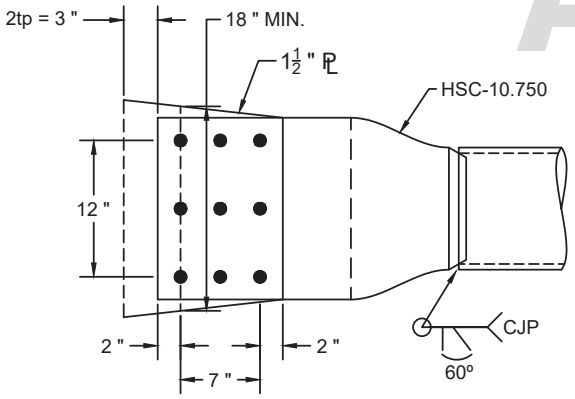
10-1"-490

LRFD 990 kip
ASD 660 kip



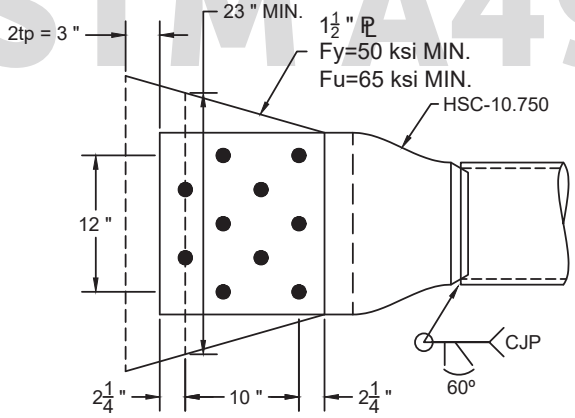
9-1"-490

LRFD 875 kip
ASD 582 kip



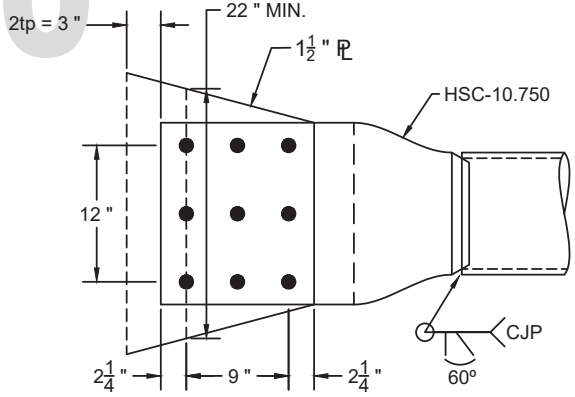
10-1 1/8"-490

LRFD 1252 kip
ASD 835 kip



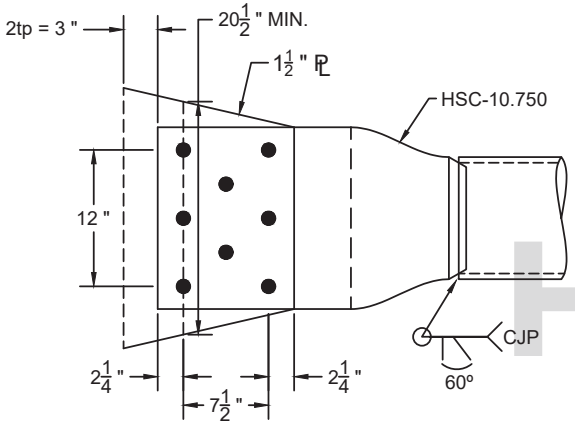
9-1 1/8"-490

LRFD 1069 kip
ASD 711 kip



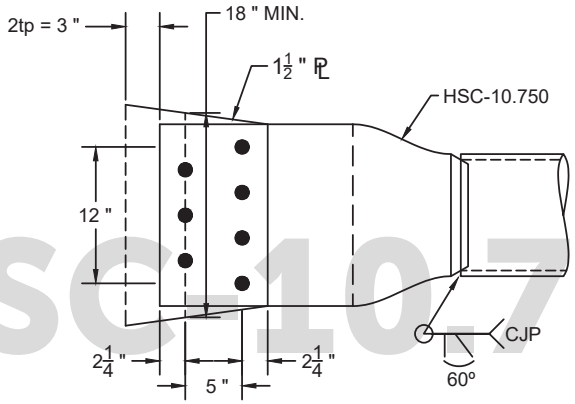
8-1 1/8"-490

LRFD 996 kip
ASD 663 kip



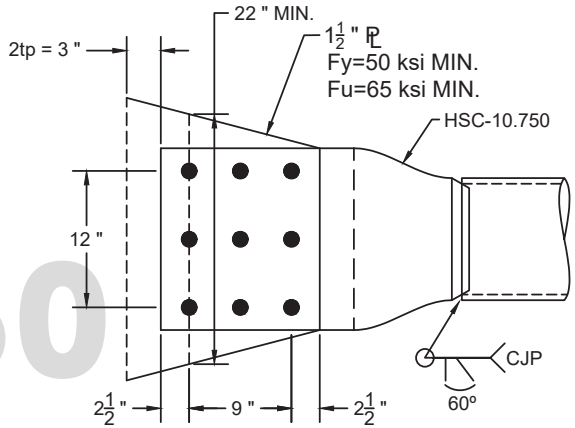
7-1 1/8"-490

LRFD 864 kip
ASD 575 kip



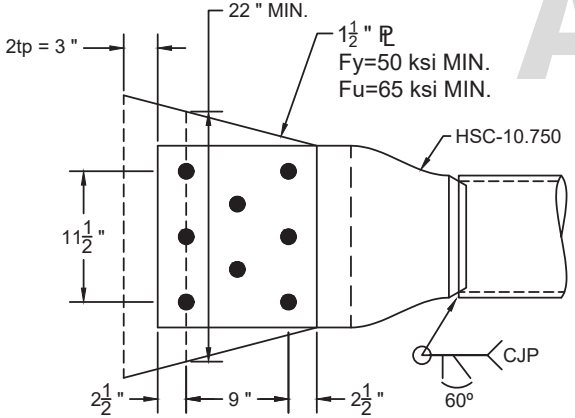
9-1 1/4"-490

LRFD 1258 kip
ASD 839 kip



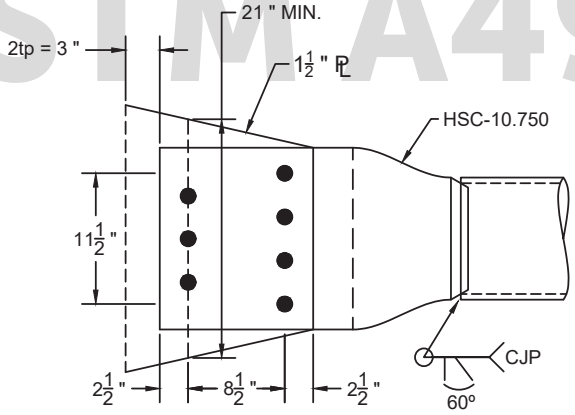
8-1 1/4"-490

LRFD 1237 kip
ASD 825 kip



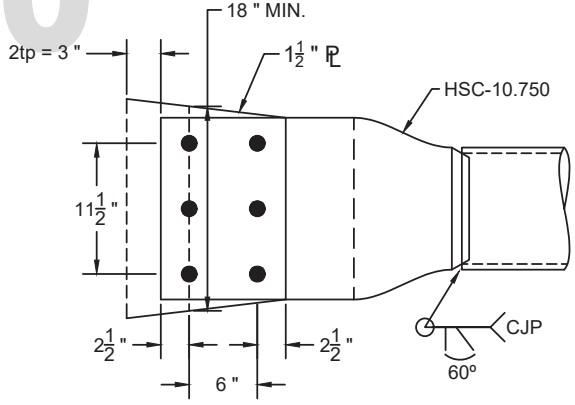
7-1 1/4"-490

LRFD 1021 kip
ASD 679 kip



6-1 1/4"-490

LRFD 875 kip
ASD 582 kip



HSC-12.75

ANSI/AISC 341-22

$$\frac{D}{t} \leq \frac{0.053 \cdot E}{R_y F_y}$$

ASTM A500
Grade B

Fy = 46 ksi

Ry·Fy = 64 ksi

thus D/t ≤ 23.9

Shape	Wall Thickness, t		D/t	Area, A	Ry·Fy·A	BEARING-TYPE CONNECTIONS[1] Detail Number					
						Group 120 (eg. A325)			Group 150 (eg. A490)		
	Nominal	Design[2]				Bolt Size			Bolt Size		
	in.	in.		in.^{2}	kip	1	1⅛	1¼	1	1⅛	1¼
HSS 12.75	0.625	0.581	21.9	22.22	1,431	20-1"-325	15-1 1/8"-325	12-1 1/4"-325	16-1"-490	12-1 1/8"-490	10-1 1/4"-490

1. Connections must have pretensioned high-strength bolts. The following are suggested bolt lengths:
   5¾" Long bolt for 1" Group 120 (eg. A325) or Group 150 (eg. A490)
   6" Long bolt for 1¼" and 1⅛ Group 120 (eg. A325) or Group 150 (eg. A490)
2. Design wall thickness for ASTM A500 HSS sections taken as $0.93 \cdot t_{nominal}$.

X Connector tabs not large enough to accommodate the number of bolts required

HSC-12.75

ANSI/AISC 341-22

$$\frac{D}{t} \leq \frac{0.053 \cdot E}{R_y F_y}$$

ASTM A500
Grade C

Fy = 50 ksi

Ry·Fy = 65 ksi

thus D/t ≤ 23.6

Shape	Wall Thickness, t		D/t	Area, A	Ry·Fy·A	BEARING-TYPE CONNECTIONS[1] Detail Number					
						Group 120 (eg. A325)			Group 150 (eg. A490)		
	Nominal	Design[2]				Bolt Size			Bolt Size		
	in.	in.		in.^{2}	kip	1	1⅛	1¼	1	1⅛	1¼
HSS 12.75	0.625	0.581	21.9	22.22	1,444	20-1"-325	15-1 1/8"-325	12-1 1/4"-325	16-1"-490	12-1 1/8"-490	10-1 1/4"-490

1. Connections must have pretensioned high-strength bolts. The following are suggested bolt lengths:
   5¾" Long bolt for 1" Group 120 (eg. A325) or Group 150 (eg. A490)
   6" Long bolt for 1¼" and 1⅛ Group 120 (eg. A325) or Group 150 (eg. A490)
2. Design wall thickness for ASTM A500 HSS sections taken as $0.93 \cdot t_{nominal}$.

X Connector tabs not large enough to accommodate the number of bolts required

HSC-12.75

ANSI/AISC 341-22

$$\frac{D}{t} \leq \frac{0.053 \cdot E}{R_y F_y}$$

ASTM A53
Grade B

Fy = 35 ksi

Ry·Fy = 56 ksi

thus D/t ≤ 27.4

Shape	Wall Thickness, t		D/t	Area, A	Ry·Fy·A	BEARING-TYPE CONNECTIONS[1] Detail Number					
						Group 120 (eg. A325)			Group 150 (eg. A490)		
	Nominal	Design[2]				Bolt Size			Bolt Size		
	in.	in.		in.^{2}	kip	1	1⅛	1¼	1	1⅛	1¼
Pipe 12 XS	0.500	0.465	27.4	17.95	1,005	13-1"-325	11-1 1/8"-325	9-1 1/4"-325	11-1"-490	9-1 1/8"-490	7-1 1/4"-490

1. Connections must have pretensioned high-strength bolts. The following are suggested bolt lengths:
   5¾" Long bolt for 1" Group 120 (eg. A325) or Group 150 (eg. A490)
   6" Long bolt for 1¼" and 1⅛ Group 120 (eg. A325) or Group 150 (eg. A490)
2. Design wall thickness for ASTM A53 Pipe sections taken as $0.93 \cdot t_{nominal}$.

X Connector tabs not large enough to accommodate the number of bolts required

HSC-12.75

ANSI/AISC 341-22

$$\frac{D}{t} \leq \frac{0.053 \cdot E}{R_y F_y}$$

ASTM A1085
Grade A

Fy = 50 ksi

Ry·Fy = 63 ksi

thus D/t ≤ 24.6

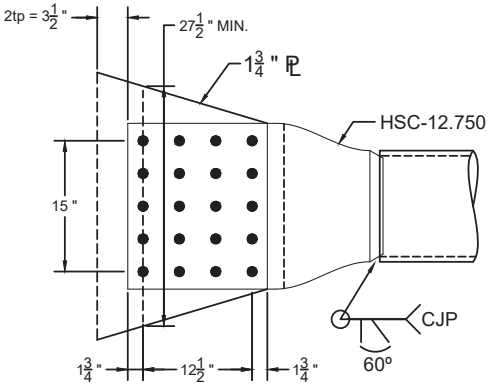
Shape	Wall Thickness, t		D/t	Area, A	Ry·Fy·A	BEARING-TYPE CONNECTIONS[1] Detail Number					
						Group 120 (eg. A325)			Group 150 (eg. A490)		
	Nominal	Design[2]				Bolt Size			Bolt Size		
	in.	in.		in.^{2}	kip	1	1⅛	1¼	1	1⅛	1¼
HSS 12.75	0.625	0.625	20.4	23.81	1,488	20-1"-325	15-1 1/8"-325	12-1 1/4"-325	16-1"-490	12-1 1/8"-490	10-1 1/4"-490

1. Connections must have pretensioned high-strength bolts. The following are suggested bolt lengths:
   5¾" Long bolt for 1" Group 120 (eg. A325) or Group 150 (eg. A490)
   6" Long bolt for 1¼" and 1⅛ Group 120 (eg. A325) or Group 150 (eg. A490)
2. Design wall thickness for ASTM A1085 HSS sections taken as $1.0 \cdot t_{nominal}$.

X Connector tabs not large enough to accommodate the number of bolts required

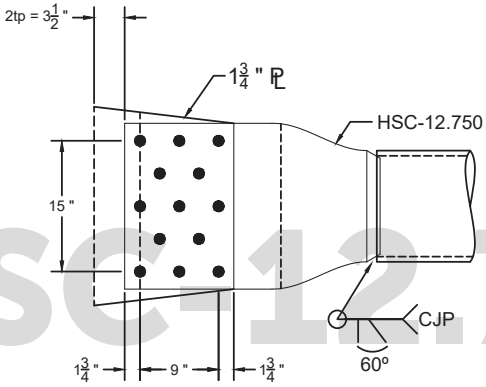
20-1"-325

LRFD 1559 kip
ASD 1037 kip



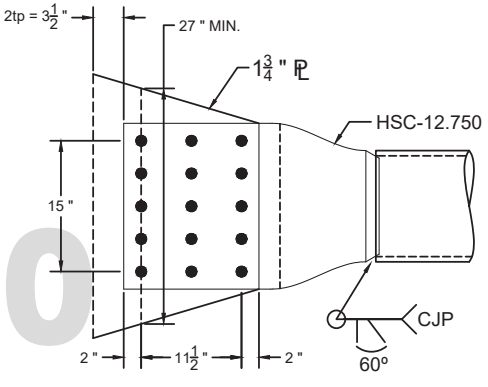
13-1"-325

LRFD 1041 kip
ASD 694 kip



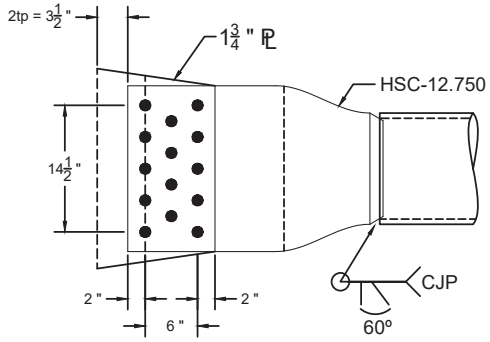
15-1 1/8"-325

LRFD 1501 kip
ASD 1000 kip



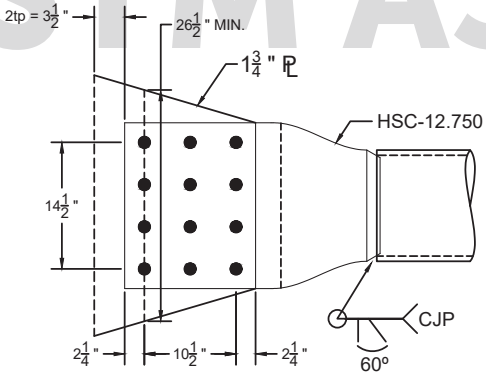
11-1 1/8"-325

LRFD 1047 kip
ASD 698 kip



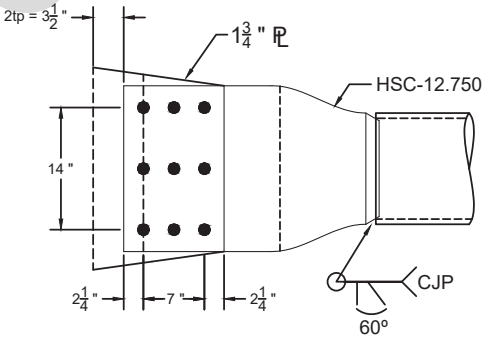
12-1 1/4"-325

LRFD 1498 kip
ASD 999 kip



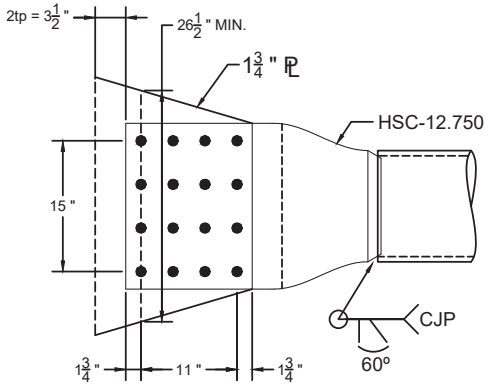
9-1 1/4"-325

LRFD 1077 kip
ASD 717 kip



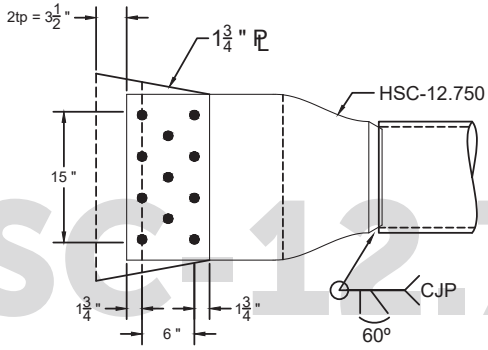
16-1"-490

LRFD 1503 kip
ASD 1000 kip



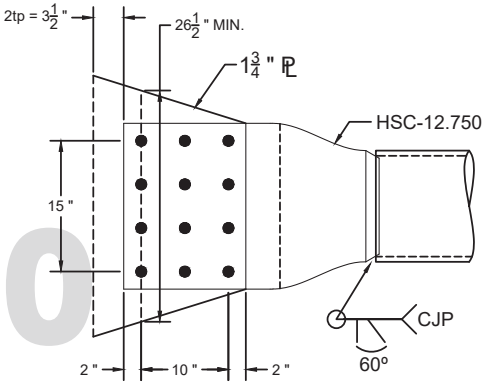
11-1"-490

LRFD 1077 kip
ASD 717 kip



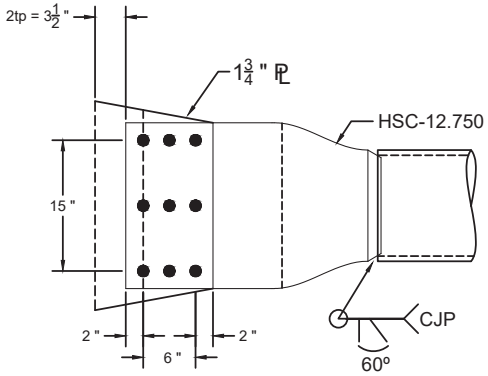
12-1 1/8"-490

LRFD 1503 kip
ASD 1000 kip



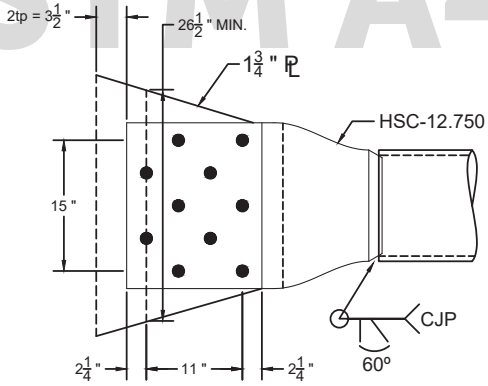
9-1 1/8"-490

LRFD 1077 kip
ASD 717 kip



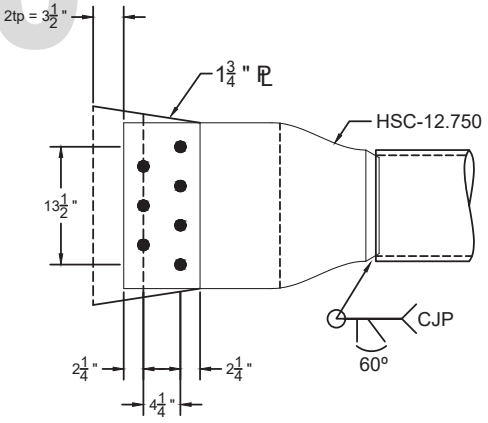
10-1 1/4"-490

LRFD 1493 kip
ASD 996 kip



7-1 1/4"-490

LRFD 1023 kip
ASD 682 kip



HSC-14.00

ANSI/AISC 341-22

$$\frac{D}{t} \leq \frac{0.053 \cdot E}{R_y F_y}$$

ASTM A1085
Grade A

Fy = 50 ksi

Ry·Fy = 63 ksi

thus D/t ≤ 24.6

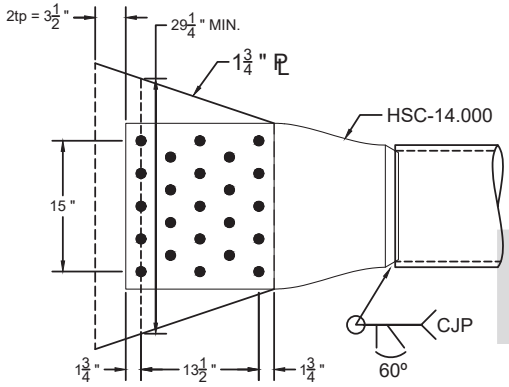
Shape	Wall Thickness, t		D/t	Area, A	Ry·Fy·A	BEARING-TYPE CONNECTIONS[1] Detail Number					
						Group 120 (eg. A325)			Group 150 (eg. A490)		
	Nominal	Design[2]				Bolt Size			Bolt Size		
	in.	in.		in.^{2}	kip	1	1⅛	1¼	1	1⅛	1¼
HSS 14.00	0.625	0.625	22.4	26.26	1,641	23-1"-325	18-1 1/8"-325	14-1 1/4"-325	18-1"-490	14-1 1/8"-490	12-1 1/4"-490

1. Connections must have pretensioned high-strength bolts. The following are suggested bolt lengths:
   6¼" Long bolt for 1" Group 120 (eg. A325) or Group 150 (eg. A490)
   6½" Long bolt for 1¼" and 1⅛ Group 120 (eg. A325) or Group 150 (eg. A490)
2. Design wall thickness for ASTM A1085 HSS sections taken as $1.0 \cdot t_{nominal}$.

X Connector tabs not large enough to accommodate the number of bolts required

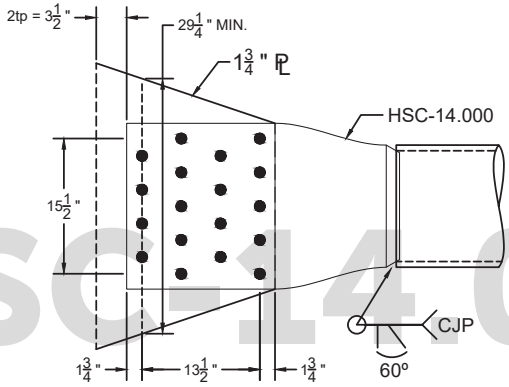
23-1"-325

LRFD 1645 kip
ASD 1097 kip



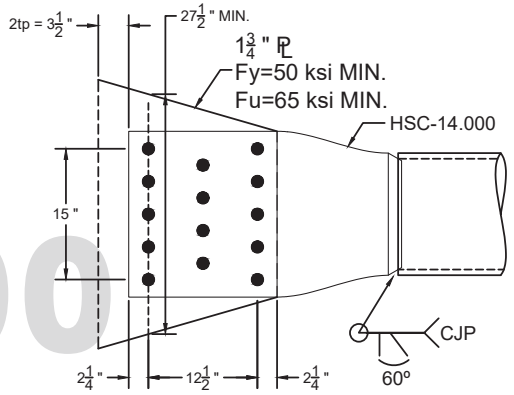
18-1 1/8"-325

LRFD 1658 kip
ASD 1103 kip



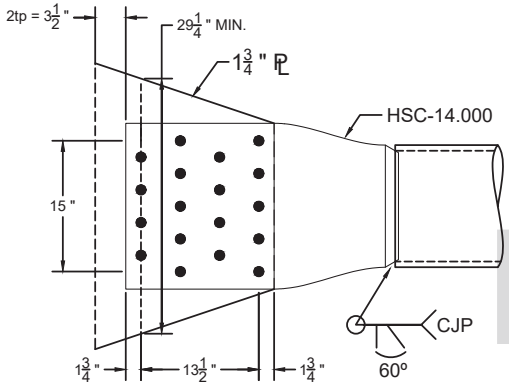
14-1 1/4"-325

LRFD 1672 kip
ASD 1114 kip



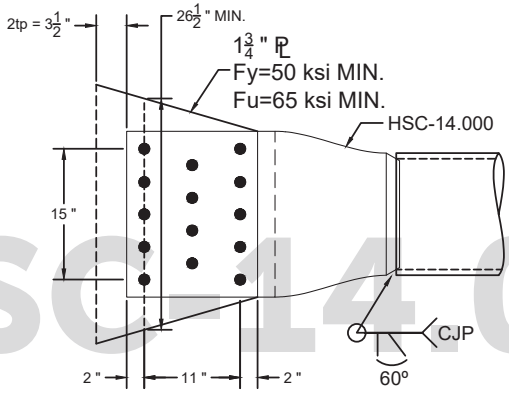
18-1"-490

LRFD 1658 kip
ASD 1103 kip



14-1 1/8"-490

LRFD 1644 kip
ASD 1096 kip



12-1 1/4"-490

LRFD 1652 kip
ASD 1101 kip