ORTHOGONAL AND SKEWED SHEAR CONNECTIONS DESIGN AND DETAILING REQUIREMENTS

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ABSTRACT

Simple shear connections for beam to beam and girder to column connections are one of the keys to safe economical fabrication, erection and structure performance. This paper explores basic connection design requirements and provides guidance on safety and economy of various types of connections. It discusses standard orthogonal and skewed shear connections for wide flange beams to wide flange girders and columns and to hollow structural sections (HSS) and concrete supports.

INTRODUCTION

One major advantage structural steel has over other materials is the ability to easily provide safe reliable and economical connections. The majority of members in a steel frame can be connected using connection details that can be designed for the vertical reaction of the beam or girder only. Typically minor eccentricities and restraint can be neglected but it is important to know the limits of these simple shear connections. The design basis for these simple shear connections is a combination of basic engineering mechanics and empirical rules. Selection of the connection detail for a particular application however also involves the magnitude of the load, the geometry of the connection, the fabricator's equipment capabilities, and erection requirements.

The need to include these cost and constructablility considerations has lead many structural engineers of record (SEOR) to delegate the design of connections to the fabricator's connection design engineer. This delegation of connection design responsibility along with the use of computer aided design (CAD) programs such as RAM steel has made it difficult for the SEOR to develop a sense of proportion when it comes to establishing connection design requirements. This can result in design details that are inappropriate and / or loads that are so conservative it is impossible to use standard connection details. The on going controversy over design responsibility for structural connections has obscured the fact that regardless of who is ultimately responsible for connection design much of the same information must be shown on the design drawings.

When the SEOR elects to design the connections it will be impossible to show all of the various detail dimensions locating all of the welds and bolts for every individual connection. It will be necessary to show representative connections and their strength schedules along with the required connection design forces so the fabricator's detailer can develop the required shop drawing information.

When connection design responsibility is delegated to the fabricator's engineer it will still be necessary to provide representative details indicating the SEOR's design requirements for force transfer along with the connection design forces. These representative connection details are also needed to enable the fabricator's estimator to price the work without doing connection design. Developing good representative connections gives the SEOR an opportunity to review problem areas where member sizing, connection geometry and the magnitude of the reaction may make it very difficult to design a good economical connection. It may still be possible to change the framing system at this time to provide for a connection that is easier to fabricate and erect and consequently is more economical.

CONNECTION DESIGN REQUIREMENTS

Connection design starts with appropriate design loads. Engineers tend to regard connections as the weak link in their framing system. This often results in mandatory loads for connections that can be several times the actual design load. While some conservatism may be advisable if the designer is uncertain about the actual load distribution in the future, there is no need to increase design shear reactions for connection reliability. A study of the reliability or "safety index" β for fillet welds and high strength bolts was presented in a paper by Fisher, Galambos, Kulak and Ravindra entiltled The Criteria for Connectors in Load and Resistance Factor Design. The safety index for fillets welds and high strength bolts in shear updated for the 1999 AISC LRFD Specification is approximately 4. When compared to a safety index of 2.6 for compact beams, it is evident that the connectors have extra safety incorporated into their design strengths. Interestingly the above paper lists the fundamental requirements for a well designed connection as strength, ductility and economy. These requirements have not changed.

The connection design load should be the actual factored or service design load for the particular member. Experience has shown that designing for factored loads is generally more economical and the economy increases as the dead load / live load ratio increases. SEOR's especially on fast track projects worry about design changes that might change the loading pattern on certain beams. While most code mandated loads are already conservative, where absolutely necessary a reasonable increase in live load can be added to account for possible future changes in the design-loading pattern with a minimum of impact on constructability and economy.

These design loads for simple shear connections should be shown for each member on the framing plans similar to the example in Figure 1. This will help the connection designer, detailer, and shop drawing reviewer make sure the proper connection detail is used and it will also provide a record of the design reaction for the owner if future changes are required. Structural design programs such as RAM Steel can down load these beam design reactions to the framing plan and can even allow the SEOR to apply an increase the live load when required for future changes.

Schedules listing beam reactions should not be used. In order to avoid being unconservative it becomes necessary for the schedule to require the connections of all similar members be designed for the worst load case. The result is connections for typical members become uneconomical and may even require reinforcement of the member at the connection due to a load that is in excess of the actual member capacity. . For example a recent project had two W18x106 girders spanning 30ft and over 200 W18x35 beams spanning 40ft with a schedule that gave the same load requirements for all W18 beams. The scheduled reaction was the equivalent of a code live load for the W18x35 beams of over 300psf on a typical office floor. Designing the connections for this reaction would have required all of the beam webs to be reinforced for shear.

A similar problem exists when the contract documents require connections to be designed for a percentage of the uniform design load (UDL) capacity of the beam. This concept will work only in the very unlikely case where all beams are designed at or near their bending capacity. Most projects have numerous short span beams where using a percentage of UDL will result in the connection having to be designed for a load several times the actual design load and again requiring reinforcement at the connection. Again a recent project using the factored UDL requirement had over 30 beams on one level that could not develop the specified load using a full depth double angle connection. Using a factored UDL reaction can also be unconservative when there are concentrated loads close to the support.

The importance of providing the connection detailer / designer the actual reaction that each member is sized for can not be over emphasized. Using schedules or a UDL factor to mandate design loads will almost always make it necessary for the connection detailer or designer to initiate numerous Requests for Information (RFI) in order to obtain the information needed to design and detail a connection that can be readily fabricated and erected. Unfortunately extensive use of the RFI process not only delays the start of shop drawings it can result in an adversarial relationship when a good cooperative relationship would benefit everyone.

After determining the design loads the next step in connection design is selecting the type of connection to be used. Traditionally a fabricator's preference for a particular type of welded or bolted connection detail varied depending upon the equipment he had and where he bought his material. Now days almost all shops have saws for cutting to accurate length and numerical control equipment for punching and drilling. This equipment coupled with extensive use of F1852, tension control bolts, makes shop bolted connections the preferred choice for standard wide flange to wide flange connections. Skewed connections, connections to HSS and connections to concrete however, typically require some form of welded connection.

DESIGN AND DETAILING OF STANDARD SHEAR CONNECTIONS

The design of double angle and single angle connections is discussed in detail in Chapter 10 of the AISC Manual of Steel Construction –LRFD, Third Edition. Double angle connections are typically applied to the supported member while single angles are normally located on the supporting member. Bolts typically should be snug tightened 3/4 inch or 7/8 inch diameter ASTM A325 bolts with threads included. Snug tightened bolts have the same design shear strength as fully tension bolts, snug tightening however reduces installation and inspection costs and avoids future disputes regarding pretension requirements. The basic requirement for snug tightened bolts is that all plies of the connections should be brought into firm contact. Bolts that are specified as snug tightened can however be tensioned and still be inspected and classified as snug tight. For example F1852 bolts could be specified and / or used even though the bolts are designed as snug tightened. For connections on larger members with heavier loads it may be possible to design special connections with thicker angles so the threads are excluded and the higher "X" values can be used.

Figures 1 and 2 are examples of typical bolted / bolted double and single angle connection details along with a table of design strengths that could be used by a detailer to select connections based on loads shown on the framing plan. Standard details like this should be developed by the SEOR using the connection design information in the AISC Manual of Steel Construction.

Short slotted holes should be used in outstanding legs of double angle connections to allow for variances in web thickness thereby permitting the use of the same angle for several beam sizes. It is also permitted to use short slots in the legs of the angles attached to the supported beam web and still neglect the eccentricity from the support to the bolt line. The adjustment these short slots provide may useful when double angle connections are used on beams with large cambers. Most fabricating shops cut beams to length and punch or drill the connections before cambering. This not only results in a connection slope at the end of the member it will change the out to out length slightly. The short slots in the angles can be used to compensate for this slope and length increase.

Single angle connections must be designed for the eccentricity from the bolt line on the supporting member to the centerline of the web of the supported member. This will normally be the governing strength limit state for this connection, although it is possible to design the bolts in this shop connection as a bearing connection with threads excluded to help compensate for the effect of eccentricity. Another advantage of single angle connections is that when using ¾ inch diameter bolts, block shear rupture usually only governs when using thin web W8 and W10 beams. Normally it is possible to utilize the full bolt strength when designing connections for other sections. Bolt bearing on the supported member may govern when there are connections on both sides of thin web girders. When this is the case the angles can be offset from side to side as shown in Figure 4 to provide for single shear on the bolts. Standard holes should be always be used on the supporting girder to limit out of plane rotation. Short slotted holes however, should be used on outstanding leg of the angle that connects to the supported member. The short slot here, not only helps compensate for variations in girder web thickness it also helps prevent any possible field fit up problem due to end slope and length change that might result from cambering the supported member.

While block shear rupture seldom governs the design of single angle connections it often is the governing limit state for coped double angle connections. A typical cope detail for either a single angle or double angle bolted / bolted connection is shown in Figure 5. The SEOR can develop a conservative table similar to Figure 6,that allows the connection detailer to check if the strength of cope or the block shear rupture strength limit state governs connection design for standard cope sizes. When the cope depth has to increase due to the girder flange thickness or width it will be necessary to provide calculation based on actual detail dimensions. While a table based on minimum web thickness for each depth like this is easy for the detailer to use, a more efficient table could be developed based on actual web thickness.

Single plate connections can be used as an alternate to the single angle connection and their design is covered in more detail below in skewed connections.

Seated connections once were popular with erectors and some fabricators. However, with thin beam webs and 50ksi material, web crippling often governs the design making seated connections less efficient. Seated connections are used primarily at expansion joints.

APPLICATIONS OF STANDARD SHEAR CONNECTIONS

Double angle bolted / bolted-framed connections are recommended for beam to column connections. This type of connection not only has substantial shear capacity it typically has enough stiffness and axial strength to help align and brace the structure. It is important to note that OSHA Erection Safety Rules Sub Part R requires that "matching" or double connections to column webs must be detailed so that a minimum of two bolts remain in the connection of the first member erected at all times. Typically this is accomplished as shown in Figure 7, by offsetting the connection angles of the members framing to the column web. This offsetting can not be done if there is a mandatory requirement that connections extend the full depth of the beam. When this is a requirement the additional expense of using seat angles as an erection aid is incurred. These seat angles may prevent the entry of beams into the column web from above. When wide flange columns are less then 12 inches deep it may be necessary to offset the bolts in the beam web or even shop weld the angles as shown in Figure 8 to avoid interference with the bolts to the column. All of this points up the importance of being able to size these connections for actual design loads rather than some mandatory connection depth, connection schedule or detail requirement.

Use of double angle connections to column webs means the erector must either be able to tip the beam in or slide it down the column. If the access to the column web is restricted, an extended single plate connection welded to plates above and below would be the next choice. The SEOR will need to evaluate the possible eccentric reaction from the bolt group to the column centerline when sizing the column. This is discussed in more detail in the section on skewed connections below.

OSHA does not have the same requirement for matching connections at girders, however, for economy and erection safety, double connections should be avoided where possible. Single sided connections such as single angles or single plates are the recommended connection. Again staying with the bolted / bolted concept, the preferred connection is a single angle for typical beam to girder connections. Because this is the most common connection in the buildings the design requirements for these connections will probably determine the size and grade of the bolt used on the project. The typical bolt when used in a full depth single angle connection should provide adequate strength for majority of the beam connections. The supported beam can be detailed with either standard holes or short slots.

Where special heavy load cases occur due to either concentrated loads or header framing, standard double angle connections can be used. When using double angles on coped members block shear rupture often governs and should be checked as noted above using either a design aid such as Figure 6 or with by actual calculations per the AISC Specification.

SKEWED CONNECTIONS

The standard connection details assume that the members being connected frame at right angles to each other. In most structures there will be some members, which do not meet at right angles. These are referred to as skewed connections. They require special design considerations to provide for safety while providing an economic structure.

The preferred skewed connections for economy and safety are single plates (Figure 9) and end plates (Figure 10). Single bent plates (Figure 3) and eccentric end plates also work well at very acute angles. The old traditional double bent plate connections are difficult to accurately fit and are expensive to fabricate. There are also quality (safety) problems with plate cracking at the bend line as the angle becomes more acute.

Single plates (Figure 9) are the most versatile and economical skewed connection with excellent dimensional control when using short-slotted holes. While capacity is limited, this is usually not a problem because skewed members generally carry less tributary area. Using the standard 3-in. hole gauge from the AISC *LRFD Manual of Steel Construction* Connection Tables (AISC, 1994), single plates can be utilized for intersection angles of 90º to 30º. Snug-tight bolts are preferred because they are more economical and greatly simplify installation when there are adjacent beams. They also eliminate the "banging bolt" problem, which occurs in single plate connections when pretensioned bolts slip into bearing. The AISC tables can be used to select the required plate size and bolts along with the weld capacity for the required load. This connection has an eccentricity related to physical distance "*a*" between the bolts and the weld as shown in Figure 9. The actual eccentricity depends on support rigidity, hole type, and bolt installation. For a flexible support and standard holes, the eccentricity for the bolts is

$$e_b = \left|(n-1) - a\right| \geq a$$

where n is the number of bolts.

For a flexible support with short-slotted holes,

$$e_b = \left|\frac{2n}{3} - a\right| \geq a$$

For a rigid support and standard holes,

$$e_b = \left|(n - 1) - a\right|$$

For a rigid support and short slotted holes,

$$e_b = \left| \frac{2n}{3} - a \right|$$

When it cannot be determined whether the support is rigid or flexible, the larger value of e_b from the above equations can be used.

The eccentricity for the bolts, e_b, is measured from the face of the support. Therefore, the eccentricity for the weld, e_w, is $e_w = e_b + a$. However, rather than using this value, AISC recommends that the weld size be such that the plate yields before the welds yield. For A36 plate and E70 electrodes, this requires that the fillet weld size is a minimum of ¾ of the plate thickness. The actual weld detail does, however, have to be developed for the skewed joint geometry, as will be shown later.

End plates (Figure 2) designed for shear only are able to provide more capacity than single plates and if horizontal slots are utilized with snug-tight bolts in bearing, some dimensional adjustment is possible. Hole gages can be adjusted to provide bolt access for more acute skews. The only real constructability problem arises when there are opposing beams that limit access to the back side of the connection. These end plate connections can be sized using the AISC tables to select plate size, bolts, and weld capacity. Note that there is no eccentricity with this joint. The weld detail, however, has to be adjusted for the actual geometry of the joint in a manner similar to the shear plate.

Single bent plates (Figure 11A) can be sized for either bolted or welded connections using procedures similar to those in the AISC Manual (AISC, 1994) for single angle connections. These involve two eccentricities, e_1, and e_2 from the bend line. The eccentricities are measured from the bend line because the plate has effectively zero bending strength out of plane.

A variation on the single bent plate of Figure 3A is shown in Figure 11B, where an angle is used providing a 90º bend. The eccentricities e_1 and e_2 are again measured from the "bend line," which is at the intersection of the two legs of the angle. Note that this connection places an eccentric load on the carrying beam, which may need to be considered in the design of this beam and its connections.

Eccentric end plates (Figure 12) can be easily designed for the eccentricity e using the tables in the AISC Manual for eccentrically loaded bolt groups.

CONFIGURATIONS FOR SKEWED CONNECTIONS TO COLUMNS

Skewed connections to wide flange columns present special problems. Connections to webs have very limited access and except for columns where the flange width is less than the depth, or for skews less than 30º, connections to flanges are preferred.

When connecting to column webs, it may be possible to use either a standard end plate or eccentric end plate as shown in Figures 13 and 14. Single plate connections should not be used unless the bolts are positioned outside the column flanges. This will make the connection so eccentric that top and bottom plates, as shown in Figure 15, may be needed. Extending the single plate increases the connection cost and, unless the connection is designed for the increased eccentricity (e of Figure 15), the column must be designed for it. Except for Figure 15, the eccentricities for these connections are the same as similar connections to beam webs.

Skewed connections to the column flange will also be eccentric when the beam is aligned to the column centerline. However, if the beam alignment is centered on the flange, as shown in Figure 16, the minor axis eccentricity is eliminated and the major axis eccentricity will not generally govern the column design. The connection eccentricity is related to the parameter a here in the same way as was discussed for Figure 9. It is reasonable to assume that the column provides a rigid support in Figure 16.

When the beam is aligned to the column centerline either single plates (Figure 17), eccentric end plates (Figures 18 and 19), or single bent plates (Figure 20) can be used. The eccentricity for each of these connections is again similar to that for the same connection to a beam web. An additional eccentricity e_y, which causes a moment about the column weak axis, is present in these connections as shown in Figures 17 through 20. The column design should be checked for this moment. In many cases, other members framing to the column may provide enough restraint to offset this eccentric moment. Note that the column eccentricities discussed here are considered about the column weak axis only. Non-skew shear connections to column flanges are not normally considered to induce any significant moment about the column strong axis, and thus the strong axis component of a skewed shear connection is ignored also.

The column eccentricities shown in Figures 17 through 20 perhaps need some further explanation. Generally, the eccentricity e_y in these figures is the distance from the point where the beam controidal axis intersects the line of the column flange face. This is clearly the case in Figures 17, 18, and 19. In Figure 20, because of the bent plate, the eccentricities are measured from the bend line as discussed earlier for the skewed connections to beams. The shear load is delivered as a shear only load at this point. As discussed subsequently in the example section of the paper, this point is usually taken at the face of the bent plate at the inside of the bend. This is what is shown in Figure 20.

A special skewed connection is often required when there is another beam framing to the column flange at 90º. If the column flange is not wide enough to accommodate a side by side connection, a bent plate can be shop welded to the column with matching holes for the second beam as shown in Figure 21. The plate weld is sized for the eccentricity e_2 plus any requirement for development as a fill plate in the orthogonal connection, and the column sees an eccentric moment due to e_y, which equals e_2 in this case.

METHODS FOR DETERMINING STRENGTH OF SKEWED FILLET WELDS

The AISC Manual Tables for single plates and end plates are based on using standard AWS equal leg fillet welds. The single plate weld is sized to equal or exceed the strength of the plate which results in a fillet weld size of 0.75 x t for orthogonal connections. The end plate weld is sized to carry the applied load. These standard orthogonal fillet welds of leg size W (Figure 22) need to be modified as the skew becomes more acute in order to maintain the required capacity. There are two ways to do this. The AWS D1.1 Structural Welding Code (AWS, 2000) provides a method to calculate the effective throat for skewed T-joints with varying dihedral angles, which is based on providing equal strength in the obtuse and acute welds. This is shown in Figure 22a. Table II-1 of Annex II of AWS D1.1 provides coefficients based on the formulas of Figure 22a to size welds of equal strength for various dihedral angles. The AISC Method (AISC, 1994, pp. 9-232 and 9-233) is simpler, and simply increases the weld size on the obtuse side by the amount of the gap as is shown in Figure 22c.

Both methods can be shown to provide strength equal to or greater than the required orthogonal weld size of W. The main difference with regard to strength is that the AWS method maintains equal strength in both fillets, whereas, the AISC method increases the strength on the acute side by maintaining a constant fillet size W while the increased size $W + g$ on the obtuse side actually loses strength because of the gap g and the geometrical reduction in effective throat, due to the dihedral angle being greater than 90°. Nevertheless, it can be shown that the sum of the strengths of these two fillet welds W and $W + g$ is always greater than the $2W$ of the required orthogonal fillets. It should be noted that the gap g is limited by AWS to a maximum value of 3/16 inch for both methods.

The effects of the skew on the effective throat of fillet welds can be very significant as shown in Figures 23A and 23B. These figures also show how fillet legs W_o and W_a are measured in the skewed configuration. Note that in non-orthogonal fillets, the "leg size" is not the contact length of the leg as would be the case for orthogonal fillets, but rather it is the projection of the contact length of one leg on a line perpendicular to the other leg. This is done to enable these fillet welds to be measured. On the acute side of the connection the effective throat for a given fillet weld size gradually increases as the connection intersection angle ϕ changes from 90º to 60º. From 60º to 30º, the weld changes from a fillet weld to a skewed T-joint (Figure 24) and the effective throat decreases due to the allowance Z (AWS D1.1, 2000, Table 2.2) for the unwelded portion at the root. While this allowance varies based on the welding process and position, it can conservatively be taken as the throat less 1/8 inch for 60º to 45º and ¼ inch for 45º to 30º. Joints less than 30º are not prequalified and generally should not be used.

AWS D1.1 recommends that the contract drawings either specify the required strength (load) or the effective throat required. The fabricator then sizes the weld based on the process and position that will be used.

A 3/16-in. root gap occurs for 3/8-in. thick material at a connection intersection angle ϕ (Figures 23A and 23B) of 30°, i.e. $g = t\sin\phi = 0.375 \text{ x } \sin 30° = 0.1875$. For thicker material or larger angles, the contact point of the skewed material will need to be chamfered to reduce the gap to 3/16 inch or less.

The joint of Figure 24 is shown as a partial joint penetration (PJP) weld by the weld symbol. This is done for convenience in order to allow the required effective throat to be specified. As noted above, it is a skewed T-joint and, as such, is prequalified by AWS D1.1. It should be noted, however, that the obtuse side weld which is placed in the opening between the end of the skewed material and the other material will not be able to be achieved if $Z + t_e > t$. In this case, the skewed material will need to be chamfered and the weld on the obtuse placed as shown in Figure 23B.

AN EXAMPLE

Figure 25 shows an arrangement of beams framing to a column, two of which are skewed and one which is off center. This often occurs to accommodate architectural features and curtain wall requirements. Consider the design of the connection of the W24x76 on Line A to the column. The bolts are A325-N, 7/8-in. diameter, in standard 15/16-in. diameter holes. The connection material is A36, and the members are A572 Grade 50 or A992. The connection is similar to that shown in Figure 20 where the eccentricities for the connection, e_1 and e_2 of Figure 20, are taken from the bend line. Bent plates are usually dimensioned to the inside of the bend. Thus, in Figure 25, the bent plate dimensions for the beam on Column Line A are 4 1/8 in. and 4 3/8 – 1 = 3 3/8 in., which add up to the 7½-in. dimensioned length. It is usual practice to derive the eccentricities from these dimensions. Thus, for the bolts, e_1 = 2.5625 and for the weld, e_2 = 3.375 - xl where l = 21 in. and x will be determined from AISC Manual Table 8-42. The capacity of this connection will now be determined by checking the following Limit States.

1. Bolt Shear: The design strength of one bolt is ϕr_v = 0.75 x 48 x 0.6013 = 21.6 kips (AISC Spec., Sect. J3.6 (AISC, 1993)). With the eccentricity e_1 = 2.5625 in., the AISC Manual Table 8-18 gives C = 6.24 by interpolation. The design strength of the seven bolts is thus ϕR_n = 21.6 x 6.24 = 135 kips.

2. Weld Design Strength (AISC Manual Table 8-42): A 5/16-in. fillet weld of "C" shape is indicated in Figure 17. With l = 21 in., kl = 8/2 – 1 = 3 in., and $al + xl$ = 3.375 in., k = 0.143, x = 0.017 by interpolation in Table 8-42, p. 8-187. Therefore, al = 3.375 - 0.017 x 21 = 3.018 in., a = 0.144, and C= 1.85. The weld design strength is thus ϕR_n = 1.85 x 5 x 21 = 194 kips.

3. Bearing on the W24x76 Web (AISC Spec., Sect. J3.10): ϕR_n = 0.75 x 2.4 x F_u x t_w x d x C = 0.75 x 2.4 x 65 x 0.440 x 0.875 x 6.24 = 281 kips. This calculation assumes that the edge distances equal or exceed 1.5d and the spacing equals or exceeds 3d, which is the case here.

4. Bearing on the Bent Plate (AISC Spec., Sect. J3.10): ϕR_n = 0.75 x 2.4 x 58 x 0.5 x 0.875 x 6.24 = 285 kips. Again, this calculation assumes that edge distances and spacing in the plate equal or exceed 1.5d and 3d, respectively, which is the case here.

5. Gross Shear - Bent Plate (AISC Spec., Sect. J5.3): ϕR_n = 0.9 x 0.6 x F_y x t_w x l = 0.9 x 0.6 x 36 x 0.5 x 21 = 204 kips.

6. Net Shear - Bent Plate (AISC Spec, Sect. J4.1): ϕR_n = 0.75 x 0.6 x F_u x A_{net}. A_{net}= 0.5(21 - 7 x (0.9375 + 0.0625)) = 7.0 in^2. ϕR_n = 0.75 x 0.6 x 58 x 7.0 = 183 kips.

7. Net Bending Strength of Bent Plate: From AISC Manual Table 12.1, the net section modulus is S_n = 24.8 in^3. $\phi R_n = \dfrac{0.75\times F_u \times S_n}{e_1} = \dfrac{0.75\times 58\times 24.8}{2.5625}$ = 421 kips.

8. Gross Bending Strength of Bent Plate: The gross section modulus near the bolts is S = 1/6 x 0.5 x 21^2 = 36.7. $\phi R_n = \dfrac{0.9\times F_y \times S}{e_1} = \dfrac{0.9\times 36\times 36.7}{2.5625}$ = 464 kips. The design strength of the

connection is the least of the Limit State values given above, or ϕR_n = 135 kips, and the bolts control. The last two Limit States, Numbers 7 and 8, may not seem very important because they yield such large design strengths, i.e. 421 kips and 464 kips, respectively. For shallow connections, i.e., when fewer rows of bolts are used (l smaller) or eccentricities are larger, they can become the controlling Limit States. The AISC Manual Table 9-10 could have been used to check the bolted side of this connection. With a flexible support, standard holes, and an eccentricity of 3 in. (greater than actual eccentricity of e_1 = 2.5625 in.), Table 9-10 gives ϕR_n = 131 kips.

WIDE FLANGE BEAM CONNECTIONS TO HSS COLUMNS

Hollow structural sections (HSS) because of their closed shape require a shop-welded connection. The AISC Hollow Structural Sections Connection Manual covers the design of a wide variety of connections to HSS and has tables can be used to design and detail all of these connections.

When a beam has to frame to the face of an HSS column a single plate connection welded to the face of the column is the recommended connection detail. The same single plate design procedure described in skewed connections above can be used. Tests (Sherman and Ales 1991) established that standard single plate design procedures can be used as long as the wall of a rectangular HSS is not classified as a slender element. When the width–thickness ratio of the rectangular HSS column connection face less than 35 ($B/t \leqq 35$) standard AISC single plate design procedures can be used. The same is true for circular HSS as long as the section is compact under axial load ($D/t \leqq 3300Fy$) and the ASTM A500 production limit of $D/t \leqq 72$. Figure 26 is an example of a typical single plate connection detail to the face of a rectangular HSS along with the required wall thickness for each for standard wall widths and a table of design strengths for different connection depths. The SEOR can find the information needed to develop a detail and schedule like this in the AISC HSS Connections Manual. The old style connection detail where the plate is slotted through the column should be avoided except where axial forces have to be transferred through the column. This slotted type of connections is approximately four times more expensive than the standard single plate that is welded to the face. Quality of the slot cut can also be a concern because the two slots normally have to be cut manually and there can be problems due to the width of the slot and notches especially on the re-entrant corners.

When the required connection strength for framed connections to rectangular HSS columns can not be developed with a single plate connection a double angle connection can be shop welded to column (Figure 27). The beam bottom flange is then coped so the beam can be knifed between the angles in the field. When detailing double angle connections to HSS ideally the angle size selected should provide a connections width less the face width minus 4 times the column thickness and 2 times the required weld ($W \leqq B - 4t - 2w$). This will allow room for the standard fillet weld detail on the face of the HSS. If HSS face is too small the angles should be sized to provide a connection width that is slightly wider than the column so flare bevel grove welds can be provided along with any reinforcement if it is required.

WIDE FLANGE BEAM CONNECTIONS TO CONCRETE SUPPORTS

Design of beam connections to concrete supports with starts with knowing the tolerance requirements for the concrete member. ACI 117-90 Section 4.2 Lateral Alignment gives a tolerance of 1inch for concrete members. Field experience has shown that for basement walls, pilasters and piers; this type of tolerance may very well be required. If possible this is one place that a seated type connection would the needed adjustment. If however the beam is not at the top of the concrete member, forming contractors are reluctant to provide notches or voids in their form. The typical connection is an embedded plate flush with the wall.

Assuming that the concrete support consists of an embedded plate and if the beam connection has to allow for the full concrete tolerance the connection must be able to adjust plus or minus 1 inch or as near to this as possible. This amount of adjustment is almost impossible to supply with a bolted type of connection. The most that can be supplied by using a double angle connection field welded to the embedded plate and field bolted to long slots in the beam (Figure 28). is approximately 5/8 inch tolerance. While this may be enough for most conditions, using short slots in the angles can increase the tolerance by another 1/8 inch. The use of long slots requires that the connection be designed for the eccentricity from the bolt line to the face of the embedded plate and that the connection be designed slip critical. The connection angles act as the plate washer that is required over the long slot and limit the eccentricity in the connection. It is important that the bolts in these connections be noted as slip critical on the erection diagram so that proper installation and inspection procedure be followed. If the other end of the beam is connected to a steel column or girder if may be possible to provide a similar connection at that end and double the field adjustment.

The combination of designing the connection as eccentric and slip critical results in a reduced capacity for the bolts in this connection. When more design strength is required or additional length adjustment is needed, the connection can be changed to field welded instead of bolted similar to Figure 29. In this case a pair erection bolts can be provided using long slots in both the beam and angle if more tolerance adjustment is required. The angles and the welds should be sized for the maximum possible eccentricity.

Single plates are not recommended for connections to concrete except positioning the steel is not critical because the use of short slots in both the beam web and connection plate can not provide the required length adjustment. When using single plates if concrete tolerances do not allow the bolts to be installed the single plate should not be welded to the beam. Standard single plate design involves empirical rules for bolt size and plate thickness to allow some deformation at the bolt hole to achieve the required simple beam end rotation. The field fix in this case would be to add a single angle on the other side or the beam web and design the welds for the actual eccentricity.

The tolerance requirements for connections to concrete require skewed connections to be made with either single or double bent plate connections. Single bent plate connections were discussed above and the same general rules apply. Double bent plates while difficult to fabricate are still the preferred connection when length adjustments like these are required. The double bent plates can be designed and detailed similar to the double angles and slots discussed above.

The embedded plate part of the connection should also be sized for placement tolerances. It is recommended that the plate be 12 inches wider than the connection width. This extra width will ensure room for the connection, reduce the effect of eccentricity of the plate due to plate placement tolerances and also make it easier to develop the full strength of the shear connectors.

CONCLUSIONS

Simple beam shear connection design is important to safe economical fabrication and erection and to the satisfactory performance of typical building structures. Good connection design starts with accurate design load information for each member and this can be best shown on the framing plans. Standard orthogonal or 90° connections can be designed for typical framing using the procedures and tables in the 3rd Edition AISC Manual of Steel Construction. Skewed connections are very common in steel construction. Very little guidance is available for their economic selection, analysis and design. This paper provides suggested connections and methods to provide a consistent limit state approach to their analysis and design as shown in the design example. As noted in the paper, the selected connection configuration in some cases may affect the design of the beam or column, which supports the skewed beam. Connections to HSS required some type of shop welded connection material. Single plates welded to the face of the HSS are the preferred connections for typical beams. Double angles shop welded to the HSS can be use for heavy reactions. Connections to concrete require adjustment to accommodate concrete tolerances. Use of long slotted connections with either slip critical bolts or erection bolts with field welds work best.

Representative details and design strength schedules for the above connections should be part of the construction documents on every project in order to allow the fabricator to competitively bid and detail the project.

ACKNOWLEDGEMENTS

The skewed connections part of this paper was taken from a paper “Design of Skewed Connections” by Larry Kloiber and William Thornton. Victor Schneur and the staff at LeJeune Steel Co assisted in the development of the standard connection details. This paper is dedicated to the many hard working dedicated “Chief Engineers” that I have had the privilege knowing and working with who devoted their engineering careers to building safe economical steel structures. People such as John Griffis, Ned Young, Jim Holesapple, Jim Wooten, Ed Becker, John Koch, Andy Courtney, Gene Miller and Art Arndt.

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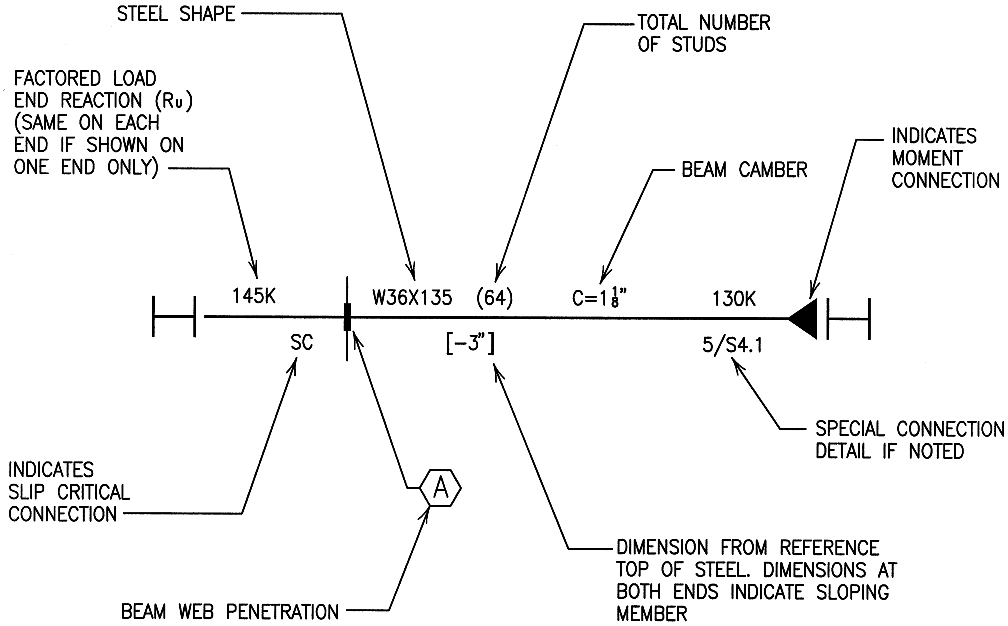
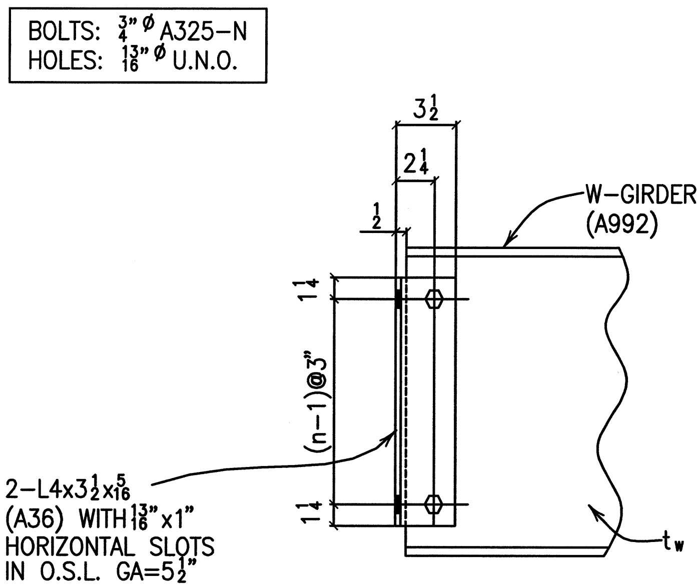


Fig. 1. Reactions and Detail Information Shown on Framing Plans

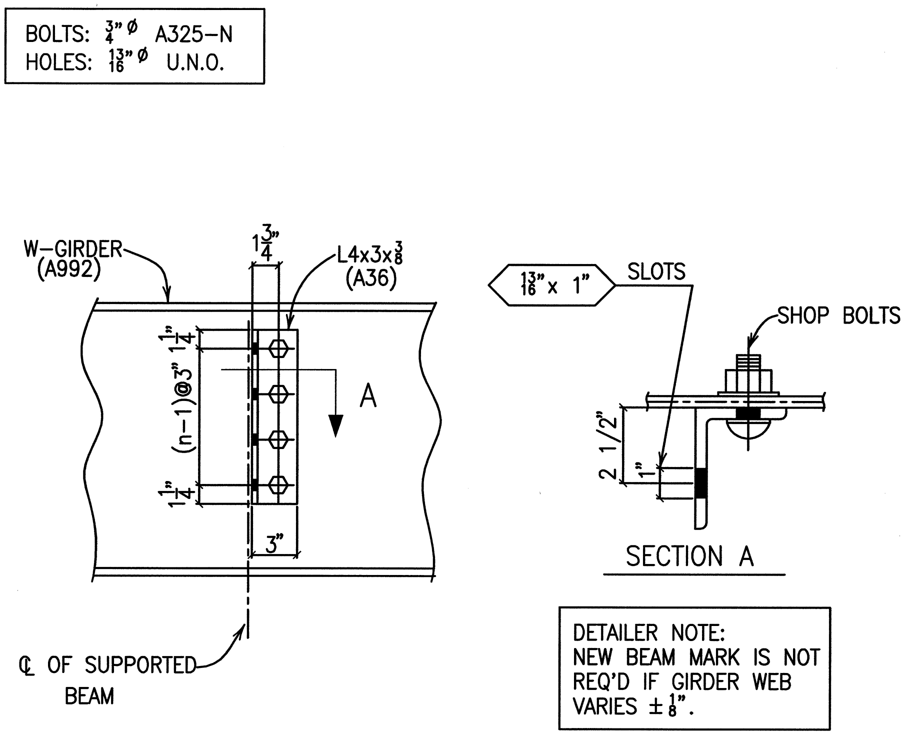


NUMBER OF ROWS, n	CONNECTION DESIGN STRENGTH FOR WEB THICKNESS, (ϕR_n) KIPS		
	$t_w=\frac{3}{16}$"	$t_w=\frac{1}{4}$"	$t_w>=\frac{5}{16}$"
2	41.1	54.9	61.2
3	61.7	82.3	95.4
4		110	127
5			159
6			191
7			223
8			254
9			286
10			318

NOTES:

1. REF. LRFD, 3RD ED.
2. WHEN THIS CONNECTION IS USED FOR BEAM TO GIRDER CONNECTION, SEE TABLE FOR COPES.
3. BOLT BEARING MUST BE CHECKED AT MATCHING CONNECTIONS WHEN WEB THICKNESS OF SUPPORTING MEMBER IS LESS THAN $\frac{5}{16}$".

Fig. 2. Double Angle Girder to Column Connection



BEAM SIZE	NUMBER OF ROWS, n	CONN. DESIGN STRENGTH, (ϕR_n) KIPS*
W8, W10	2	18.8
W12, W14	3	35.5
W16	4	52.8
W18	5	69.8
W21	6	86.7
W24	7	103

NOTES:
1. REF. LRFD, 3RD ED.
2. * IF SPECIFIED FACTORED END REACTION EXCEEDS CONNECTION DESIGN STRENGTH, USE DOUBLE ANGLE CONNECTION.

Fig. 3. Single Angle Beam to Girder Connection

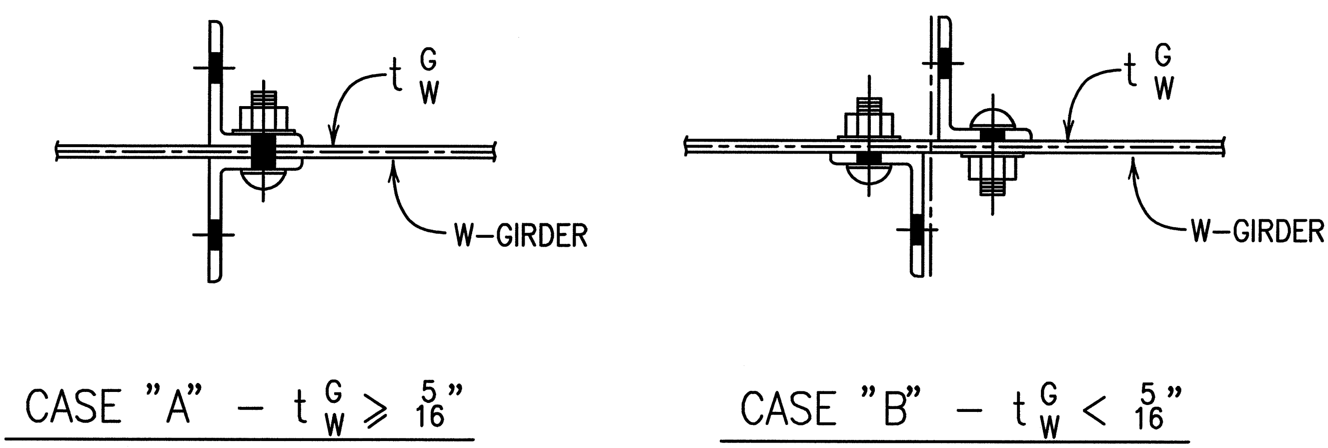
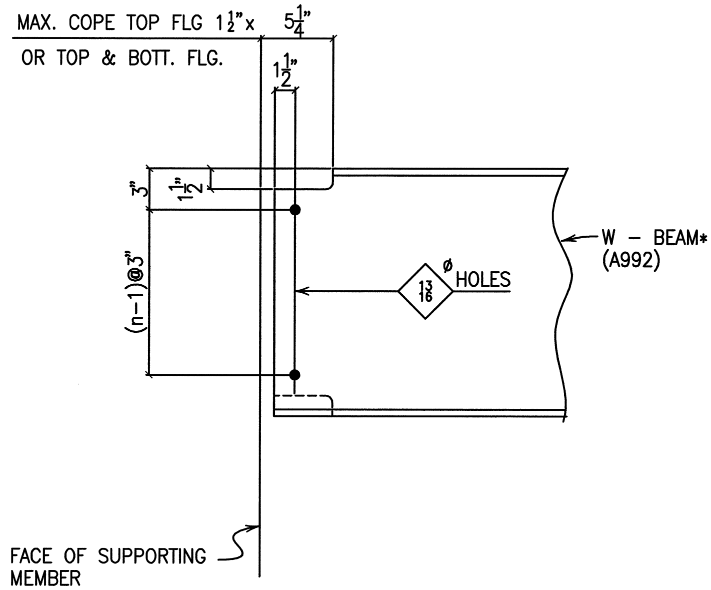


Fig. 4. Typical Single Angle Detail At Girder Web



NOTES:
1. SEE TABLES FOR BEAM END DESIGN STRENGTH FOR SPECIFIED COPES.
2. IF SPECIFIED FACTORED END REACTION EXCEEDS COPED BEAM END DESIGN STRENGTH OR LARGER COPE IS REQUIRED, PROVIDE CALCULATIONS OF BEAM END STRENGTH BASED ON DETAILED COPE SIZE AND WEB THICKNESS.

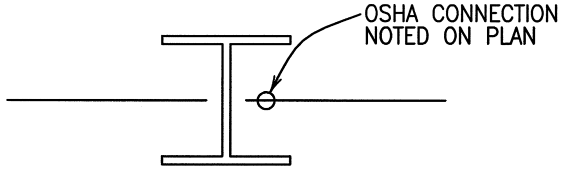
Fig. 5. Coped Beam Detail

BEAM SIZE	$1\frac{1}{2}$" x $3\frac{3}{4}$" COPES													
	COPE AT TOP FLANGE ONLY							COPE AT TOP AND BOTTOM FLANGE						
	NUMBER OF ROWS, n							NUMBER OF ROWS, n						
	2	3	4	5	6	7	8	2	3	4	5	6	7	8
W8	21.5							8.1						
W10	27.6							17.9						
W12	29	41.4						29	31.7					
W14		47.7							47.7					
W16		51.8	67.3						51.8	67.3				
W18		62.2	80.8	99.5					62.2	80.8	90.6			
W21			94.3	116	138					94.3	116	127		
W24			106	131	155	180				106	131	155	167	
W27				152	181	210	238				152	181	210	225

BEAM SIZE	$1\frac{1}{2}$"X$5\frac{1}{4}$" COPES													
	COPE AT TOP FLANGE ONLY							COPE AT TOP AND BOTTOM FLANGE						
	NUMBER OF ROWS, n							NUMBER OF ROWS, n						
	2	3	4	5	6	7	8	2	3	4	5	6	7	8
W8	15.3							5.8						
W10	27.6							12.8						
W12	29	41.4						22.6	22.6					
W14		47.7							37.6					
W16		51.8	67.3						51.8	57.6				
W18		62.2	80.8	99.5					62.2	80.8	90.6			
W21			94.3	116	138					94.3	116	127		
W24			106	131	155	180				106	131	155	167	
W27				152	181	210	238				152	181	210	225

NOTES:
1. THE DESIGN STRENGTH (ϕR_n) KIPS HAS BEEN CALCULATED FOR THE LIGHTEST BEAM WEB IN A GROUP, AND IT IS CONSERVATIVE FOR HEAVIER BEAMS IN THE SAME GROUP.
2. THESE TABLES ARE CONSERVATIVE FOR SMALLER COPES.
3. $1\frac{1}{2}$"– VERTICAL DIMENSION & $3\frac{3}{4}$" & $5\frac{1}{4}$"– HORIZONTAL DIMENSIONS.
4. SEE FIGURE 5 FOR DETAIL INFORMATION.

Fig. 6. Coped Beam Design Strength (ϕRn) Kips With ¾ in. Diameter Bolts in Standard Holes



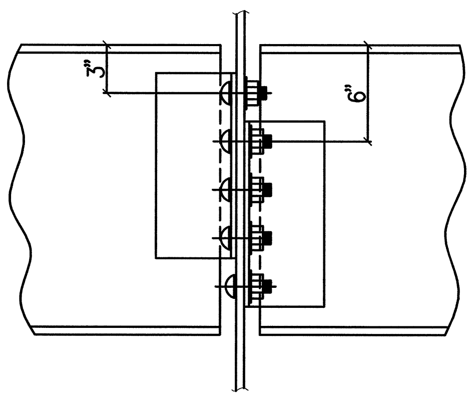
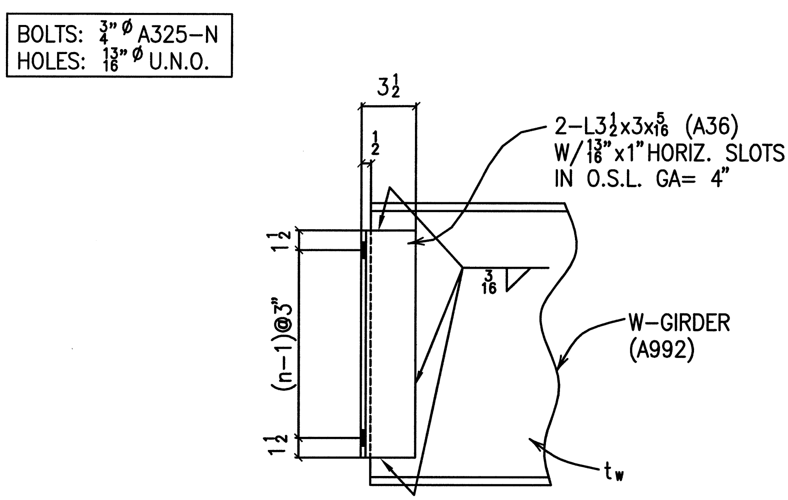


Fig. 7. Typical OSHA Connection At Column



NUMBER OF ROWS, n	CONNECTION DESIGN STRENGTH FOR WEB THICKNESS, (ϕR_n) KIPS		
	$t_w = \frac{3}{16}$"	$t_w = \frac{1}{4}$"	$t_w >= \frac{5}{16}$"
2	49.4	61.2	61.2
3	72.1	95.4	95.4
4		124	127
5			159
6			191
7			223
8			253
9			278
10			303

NOTES:
1. REF. LRFD, 3RD ED.
2. BOLT BEARING MUST BE CHECKED AT MATCHING CONNECTIONS WHEN WEB THICKNESS OF SUPPORTING MEMBER IS LESS THAN 5/16".

Fig. 8. Girder to Column Connection with GA = 4 in.

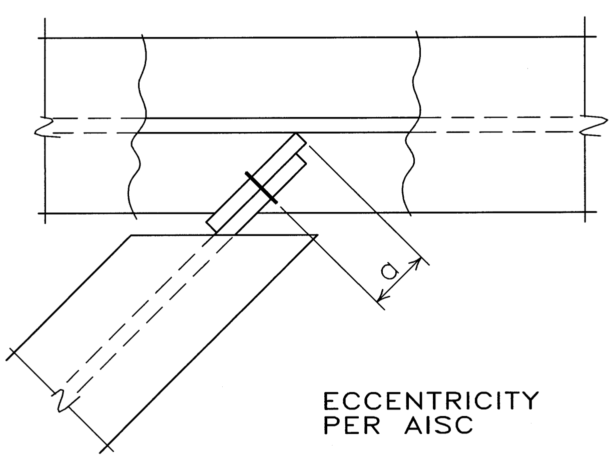


Fig. 9. Shear Tab (Single Plate)

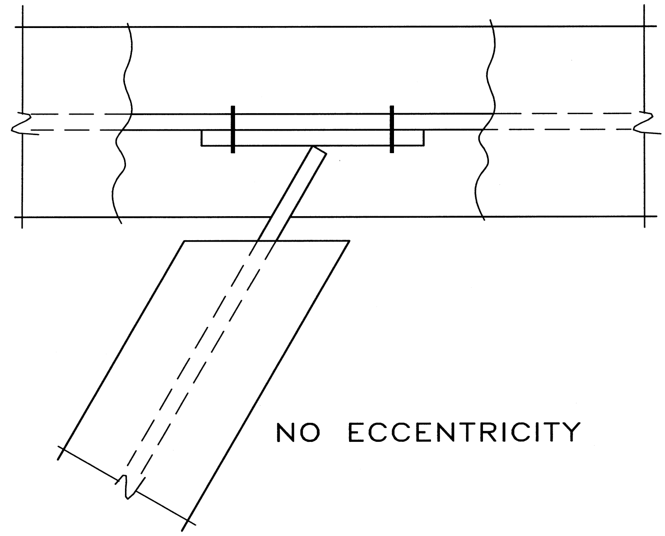


Fig. 10. Shear End Plate

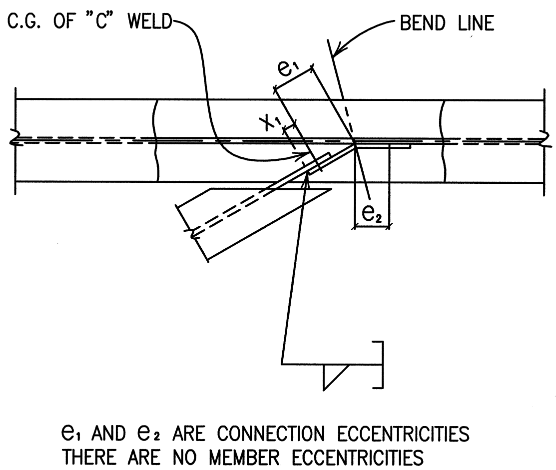


Fig. 11A. Bent Plate

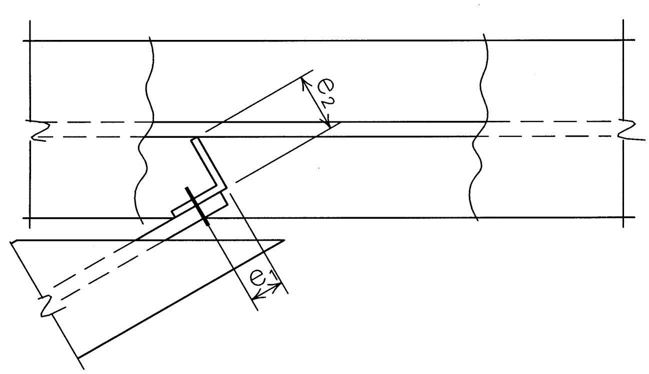


Fig. 11B. Use of Angle in Place of Single Bent Plate

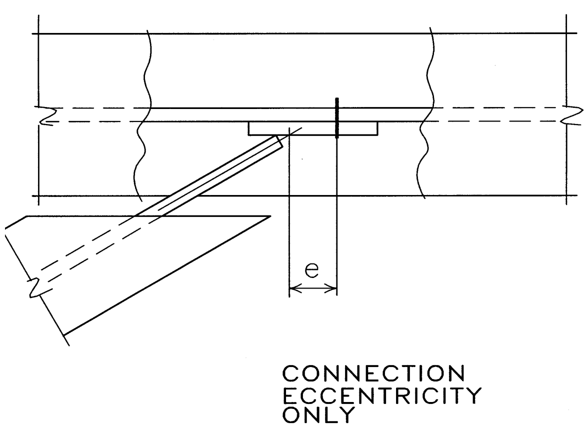


Fig. 12. Eccentric End Plate

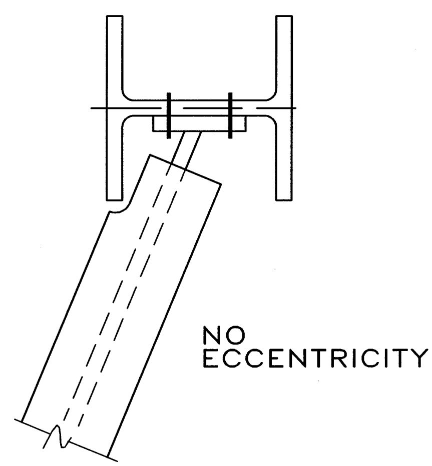


Fig. 13. End Plate

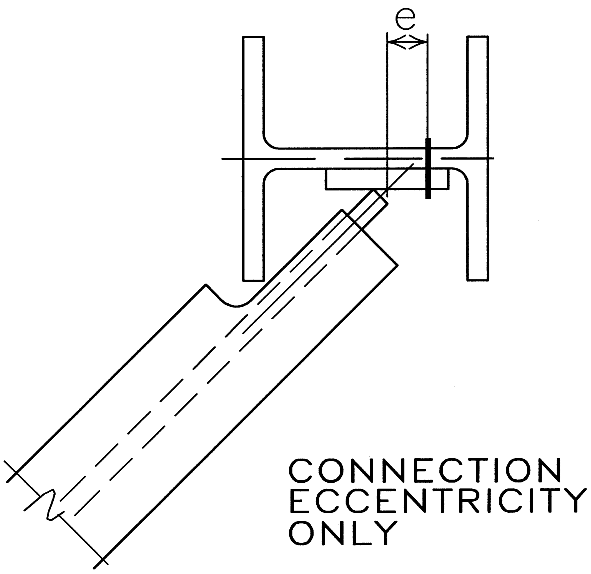


Fig. 14. Eccentric End Plate

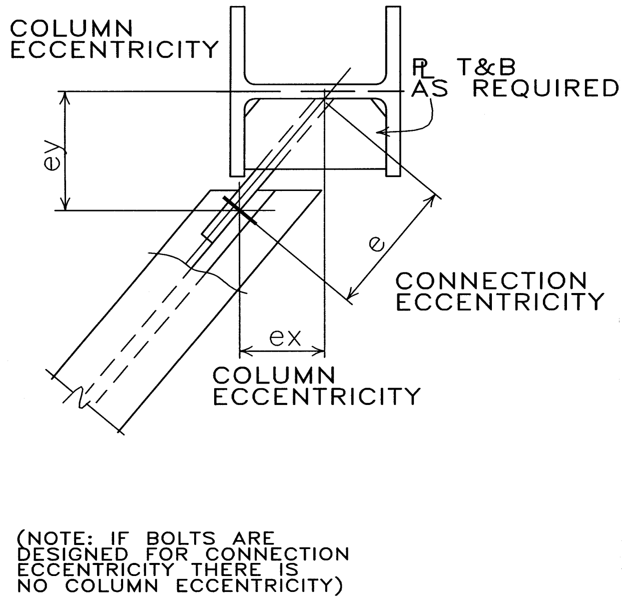


Fig. 15. Single Plate (Extended Shear Tab).

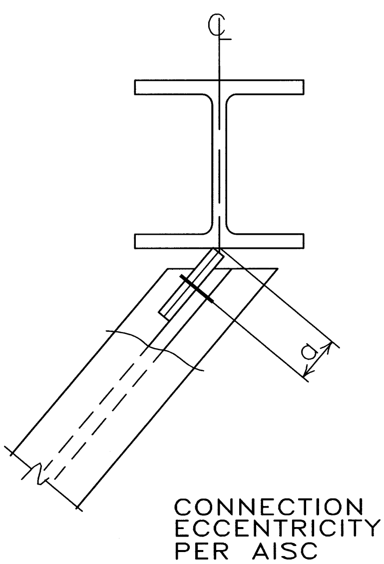


Fig. 16. Single Plate (Shear Tab) Centered on Column Flange

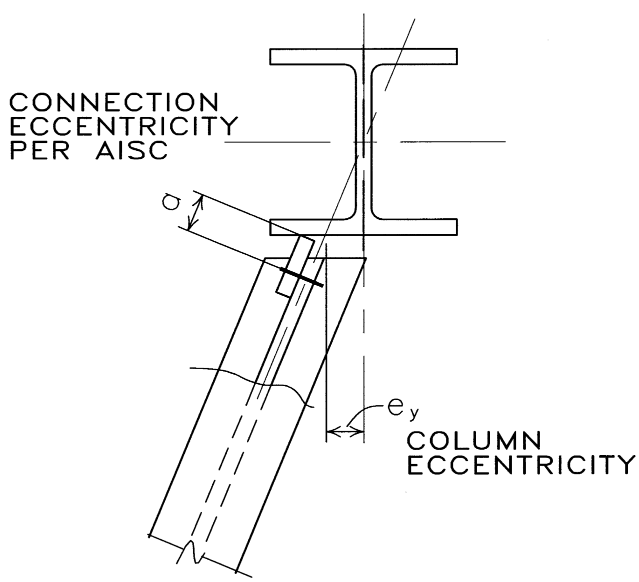


Fig. 17. Single Plate (Shear Tab) Gravity Axis Configuration

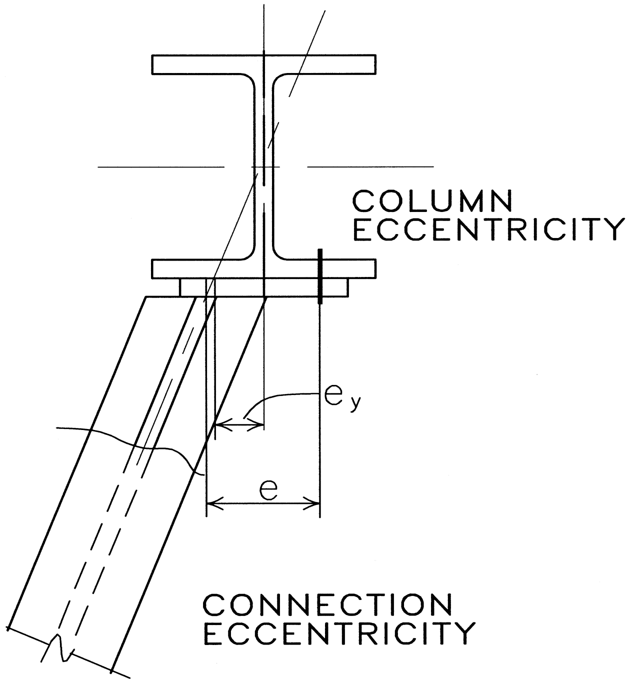


Fig. 18. Eccentric Shear End Plate Gravity Axis Configuration

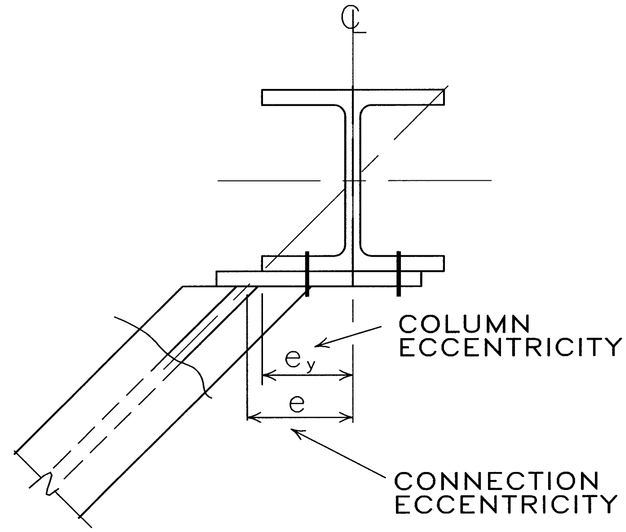


Fig. 19. Eccentric Shear End Plate for High Skew

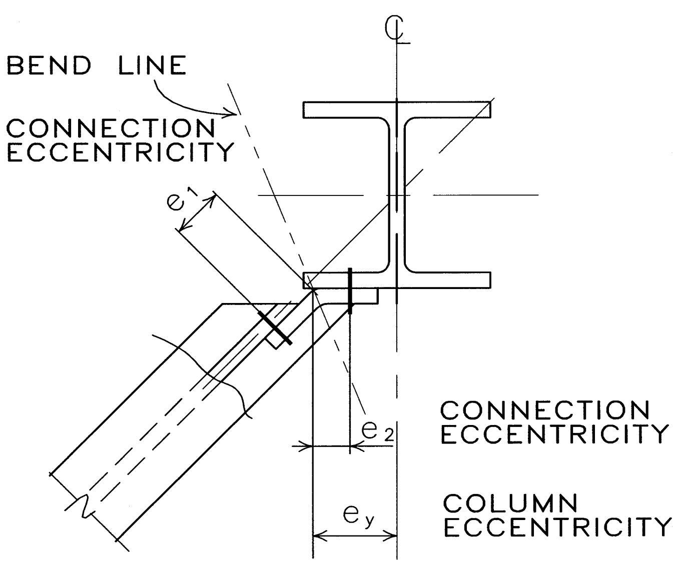


Fig. 20. Single Bent Plate One Beam Framing to Flange

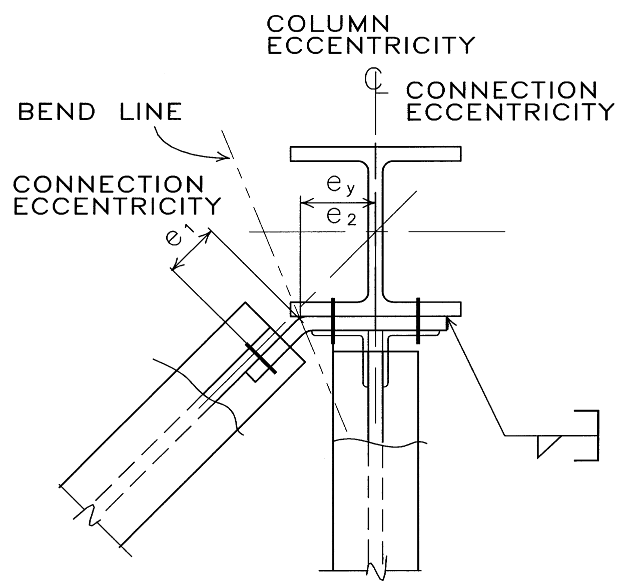


Fig. 21. Single Bent Plate—Two Beams

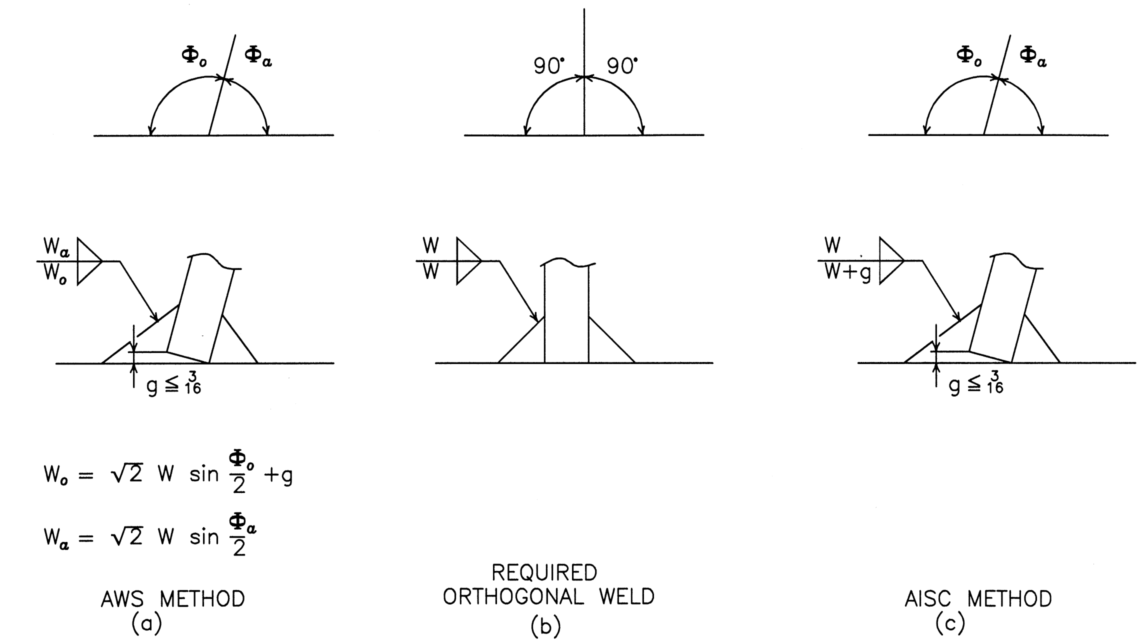


Fig. 22. Skewed Fillet Weld Sizes Required to Match Strength of Required Orthogonal Fillets

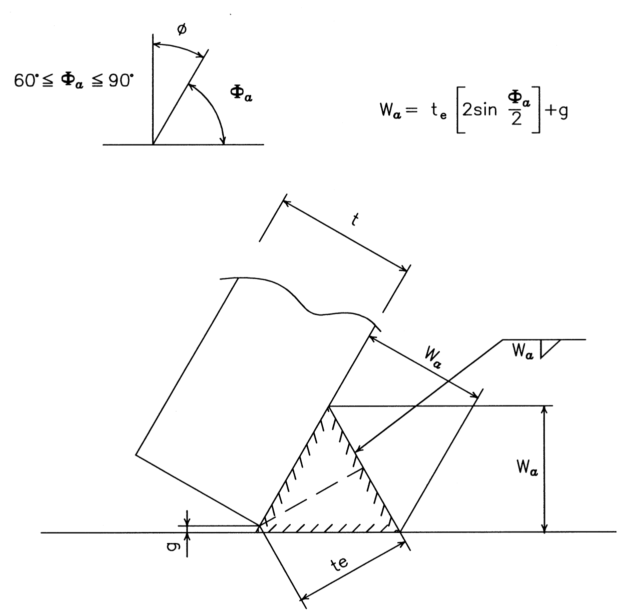


Fig. 23A. Geometry of Skewed Fillet Weld Acute Side

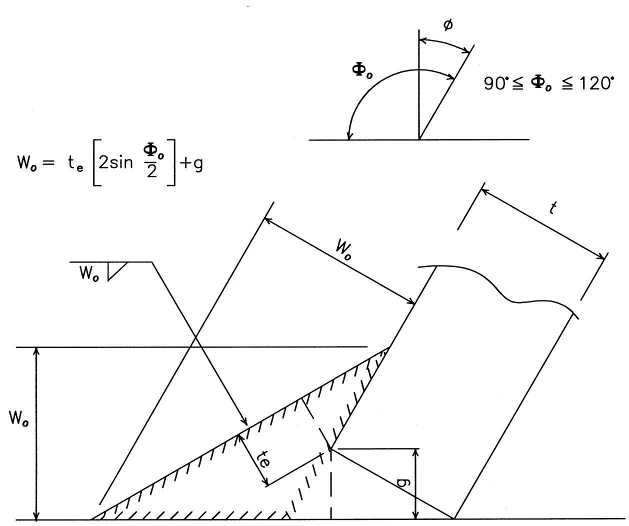


Fig. 23B. Geometry of Skewed Fillet Weld Obtuse Side

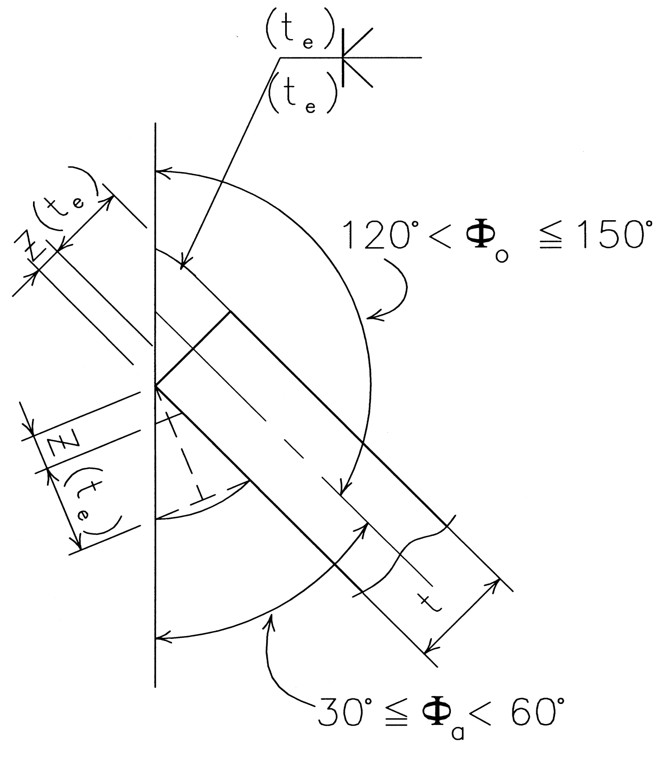


Fig. 24. Skewed T-Joint Acute Angles Less Than 60°, Obtuse Angles Greater than 120°

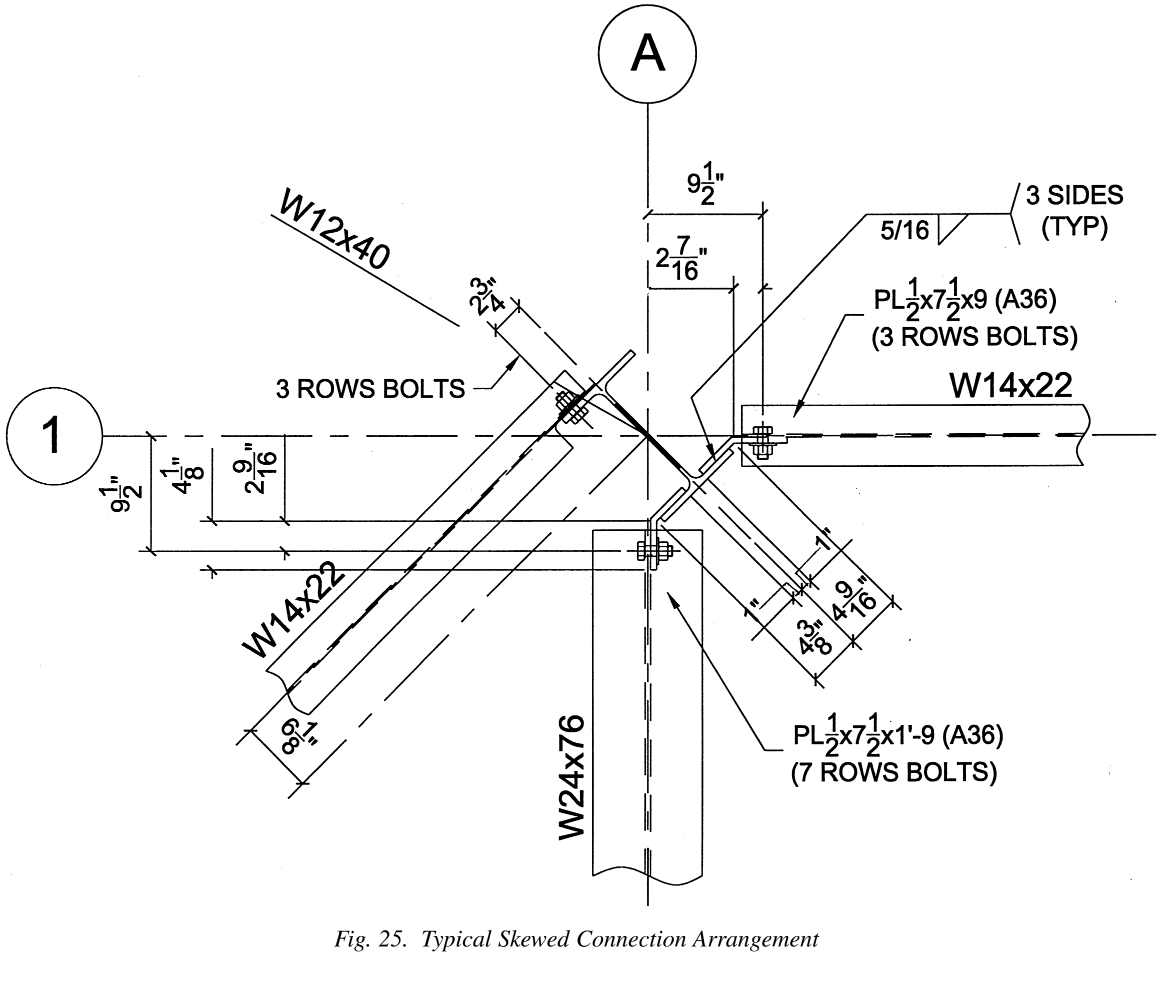
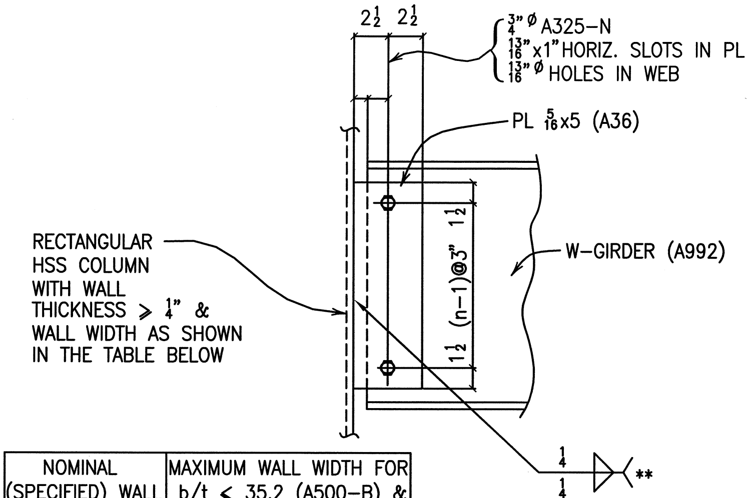


Fig. 25. Typical Skewed Connection Arrangement

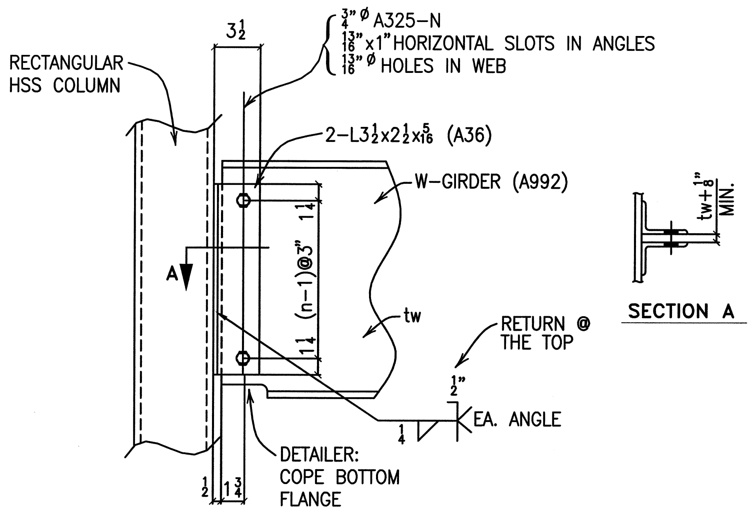


NOMINAL (SPECIFIED) WALL THICKNESS, IN.	MAXIMUM WALL WIDTH FOR b/t < 35.2 (A500-B) & b/t < 33.7 (A500-C)
$\frac{1}{4}$	8
$\frac{5}{16}$	10
$\frac{3}{8}$	12
$\frac{1}{2}$	18
$\frac{5}{8}$	22

GIRDER SIZE	NUMBER OF ROWS, n	CONN. DESIGN STRENGTH, (ϕR_n) KIPS*
W8, W10	2	16.4
W12, W14	3	31.6
W16	4	48.7
W18	5	65.9
W21	6	82.9
W24	7	99.7

NOTES:
1. REF. HSS CONNECTIONS MANUAL.
2. *IF SPECIFIED FACTORED END REACTION EXCEEDS CONNECTION DESIGN STRENGTH OR COLUMN t & b/t PARAMETERS ARE NOT MET, USE DOUBLE ANGLE CONNECTION.

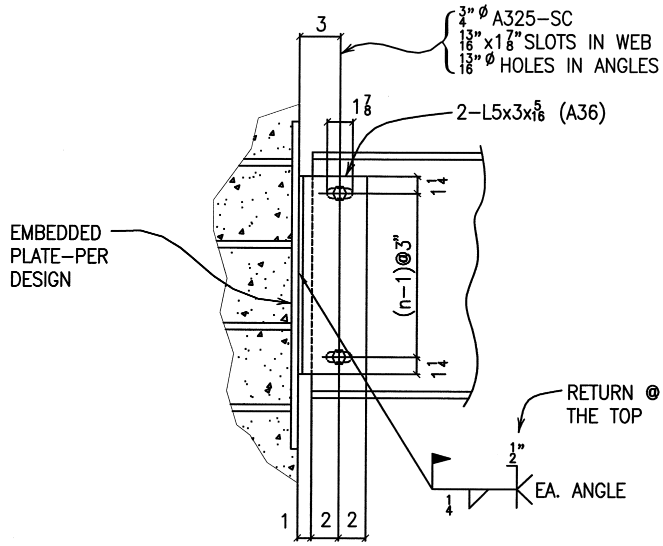
Fig. 26. Single Plate Connection to Rectangular HSS Column



NUMBER OF ROWS, n	CONNECTION DESIGN STRENGTH FOR WEB THICKNESS, (ϕR_n) KIPS	
	$t_w=\frac{3}{16}$	$t_w>=\frac{1}{4}$
2	31.9	31.9
3	49.4	65.0
4		101
5		137
6		173
7		209
8		244
9		279
10		314

NOTES:
1. REF. LRFD, 3RD ED.
2. IF $B-4t-\frac{1}{2}<5\frac{1}{2}$" INCREASE ANGLE WIDTH TO PROVIDE CONN. WIDTH OF $B+\frac{1}{2}$" MIM. & USE EQUIVALENT FLARE BEVEL GROOVE WELD.

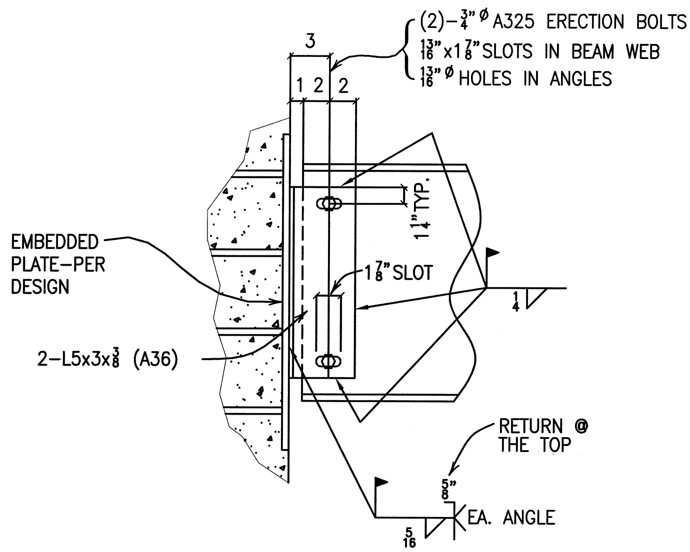
Fig. 27. Double Angle Connection to Rectangular HSS Column



BEAM/GIRDER SIZE	n* MAX	CONN. DESIGN STRENGTH, (ϕR_n) KIPS
W8, W10	2	12.8
W12, W14	3	25.6
W16	4	41
W18	5	56.9
W21	6	72.7
W24	7	88
W27	8	104
W30, W33	9	119
W36, W40, W44	10	134

NOTES:
1. REF. LRFD, 3RD ED.
2. IF BEAM/GIRDER IS SKEWED, USE BENT PL'S 5/16" (A36) INSTEAD OF ANGLES. SKEW NOT TO EXCEED 45°. BEND PLATES PERPENDICULAR TO THE GRAIN.
3. IF SPECIFIED FACTORED END REACTION IS LARGER THAN CONNECTION DESIGN STRENGTH, USE WELDED CONN.
4. * SELECT CONNECTION FOR SPECIFIED FACTORED END REACTION.

Fig. 28. Beam To Concrete Connection - SC Bolted



ANGLE LENGTH, INCHES	CONNECTION DESIGN STRENGTH FOR WEB THICKNESS, (ϕR_n) KIPS	
	tw=3/16"	tw>=1/4"
5.5	27.3	27.3
8.5	57.6	60.2
11.5		99.9
14.5		143
17.5		188
20.5		234
23.5		279
26.5		324
29.5		369

NOTES:
1. REF. LFRD, 3RD ED.
2. IF BEAM/GIRDER IS SKEWED, USE BENT PL'S 3/8" (A36) INSTEAD OF ANGLES. SKEW NOT TO EXCEED 45°. BEND PLATES PERPENDICULAR TO THE GRAIN.

Fig. 29. Beam to Concrete Connection - Welded