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Design of Bolts in Shear-Bearing Connections per AISC LRFD 3rd Edition (2001)

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Course Content

A) Bolted Connections

1. Introduction

Failure of structural members is not common, but most structural failures are caused by poorly designed or detailed connections. In times past, the pin device most often used was the rivet. Since the 1950's, the high-strength bolts have substituted the rivet as the primary connector for structural steel connection. This course will address the design of high-strength bolts in a shear bearing-type connection. This type of connection is used in a variety of steel assemblage applications such as truss joints for bridges, buildings, and transmission towers, beam and column splices, wind bracing systems, and built-up sections. For a detail discussion of design of riveted connection the reader can refer to other sources (Ref. 1).

A great number of types and sizes of bolt are available, and so are many connections in which they are used. We will cover a few of the most common bolting methods used in building structures. It is convenient to categorize the behavior of different types of connections according to the type of loading. The tension member lap splice shown on **Figure 1** produces forces that tend to shear the shank of the fastener. The hanger connection shown on **Figure 2** subjects the fastener to tension. The connection shown in **Figure 3** subjects the upper row of fasteners to both shear and tension. The strength of a fastener depends on whether it is stressed in shear or tension, or both.

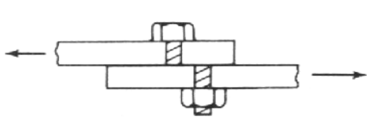


Figure 1

Figure 2

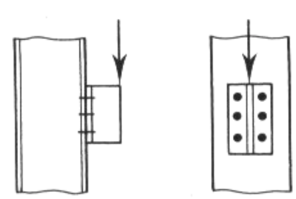


Figure 3

The LRFD provides updated bolting information consistent with the 2000 Research Council of Structural Connections (RCSC) specifications. The design philosophy of Load and Resistance Factor Design (LRFD) is primarily based on a consideration of failure conditions rather than working load conditions. Load factors are applied to the service loads, and members are selected with enough strength to resist the factored loads. Furthermore, the theoretical strength of the element is reduced by the application of a resistance factor.

The equation format for the LRFD method is stated as:

$$\Sigma\gamma_i Q_i = \phi\, R_n \qquad \textbf{(Eq. 1)}$$

Where:
Q_i = a load (force or moment)

γ_i = a load factor (LRFD section A4 Part 16, Specification)

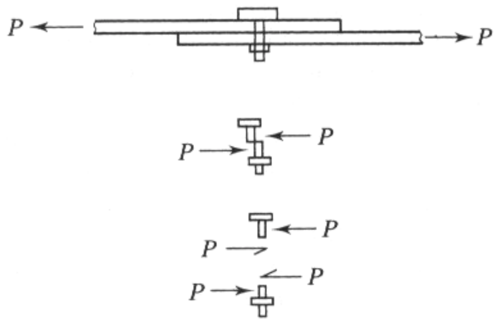
R_n = the nominal resistance, or strength, of the component under consideration

ϕ = resistance factor (for bolts given in LRFD Chapter J, Part 16)

The LRFD manual also provides extensive information and design tables for the design considerations of bolts in Part 7, Part 9, 10 and Part 16 Chapter J. Other parts of the manual cover more complex connections such as flexible moment connections (Part11), fully restrained moment connections (Part 12), bracing and truss connections (Part 13), column splices (Part 14), hanger connections, bracket plates, and crane-rail connections (Part 15). Our discussion will be limited to the basic shear bearing-type joints as presented in Part 7 and Part 16, Chapter J.

2. Failure Modes of Bolted Shear Connections

There are two broad categories of failure in connections with fasteners subjected to shear: failure of the fastener and failure of the parts being connected. The basic assumption is that equal size fasteners transfer an equal share of the load as long as the fasteners are arranged symmetrically with respect the centroidal axes of the connected members. In a lap joint as shown in **Figure 4**, failure of the fastener is assumed to occur as shown. This connection has only one shear plane of action, thus the bolt it's said to be in ***single shear,*** and although the loading is not perfectly concentric, the eccentricity is small and usually is neglected.



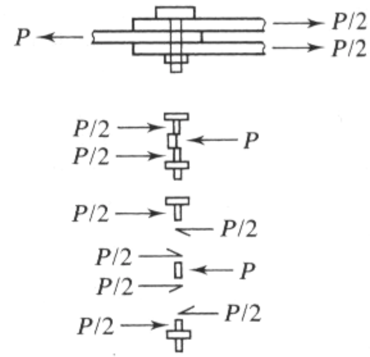


Figure 4 **Figure 5**

The connection in Figure 5 is similar, except that portions of the fastener shank is subject to half the total load, meaning that two cross sections are effective in resisting the total load. The bolt for this condition is in ***double shear.***

Another failure mode for the bolt is that of bearing failure at the bolt hole as shown in **Figure 6.**

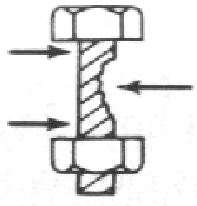


Figure 6

Other modes of failure in shear connections include failure of the members being connected and are categorized as:

a) Failure resulting from excessive tension, shear, or bending in the parts being connected, i.e. tension members may fail by tension on both the gross area and effective net area. Block shear might also need to be investigated depending on the configuration of the connection **(Figure 7).**

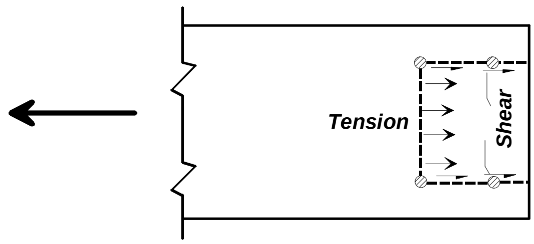


Figure 7

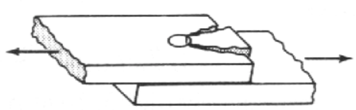


Figure 8

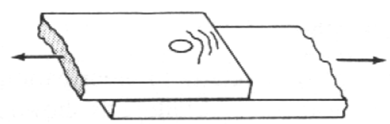


Figure 9

b) Failure of the connected members due shear failure of plate (**Figure 8**), and large bearing exerted by the bolt **(Figure 9).**

Other items affecting the bearing problem may be the presence of a nearby bolt or the proximity to an edge in the direction of the load. Therefore, the bolt spacing and edge distance will affect the bearing strength of a connection.

3. <u>Most Common Types of Fasteners in Structural Joints</u>

a) High-Strength Bolts:

The three basic types of high strength listed in the LRFD are:
ASTM designations A325, F1852, and A-490.

The new ASTM specification F1852 refers to the fasteners frequently referred to as tension control, TC, twist-off, or torque-and-snap fasteners. High-Strength bolts range in diameter from ½ to 1 ½ in. For bolts larger than 1 ½ in the AISC allows the use of ASTM A449 provided that they are not used in slip-critical connections.

b) Unfinished Bolts

These bolts are made from low-carbon steel, designated as ASTM A307, and are available with both hex and square heads in diameters from ¼ to 4 in. in grade A for general applications. They are sometimes referred as common, machine, or rough bolts. These bolts are used primarily in light structures, secondary or bracing members, platforms, catwalks, purlins, girts, light trusses, and other structures with small loads and static in nature. The A307 bolts are used predominantly in connections for wood structures.

c) Rivets

For many years rivets were the preferred means of connecting structural steel members, but now are practically obsolete in the United States. The AISC LRFD still provides method to evaluate these fasteners, mainly for review of existing old structures. Rivet steel is a mild carbon steel designated by ASTM as A502 Grade 1 (F_y = 28 ksi) and grade 2 & 3 (F_y = 38 ksi). The principal causes for the obsolescence of rivets have been the development of high-strength bolts and welding techniques. Another disadvantage that hastened the rivet demise was the high cost of field riveting.

4. New Joint Type Definitions

The 2000 RCSC specifications made significant changes in the way that joint types are defined for structural joints using ASTM A325 or A490 bolts. These changes were incorporated into the LRFD.

In the new specification, the engineer is responsible for designating the joint type in the contract documents, only the methodology has been revised.
The three new joint types are:
i) snug-tightened
j) pretensioned
k) slip-critical
Each joint type is to be specified in accordance with the required performance in the structural connection.

The snug-tightened joint will resist shear by shear / bearing (old bearing joint type). Tension may also be present with or without shear but only static tension. Exception is made for the A490 bolts, they are not allowed in snug-tightened joints subjected to tensile loads. Faying surface preparation is not required for these joints.

Pretensioned joints are allowed to resist shear by shear/bearing, bolt pretension is required due to significant load reversal, fatigue with no reversal load of the loading direction, A325 or A490 bolts subject to tensile fatigue, and/or A490 bolts subject to tension or combined shear and tension. The LRFD requires specific connections to be designed using fully pretensioned high-strength bolts, specifies in Chapter J, section J1.11. Faying surface preparation is not required for these joints.

Slip-critical joints were called in the past “friction type” connections. These joints resist shear loads by friction on faying surfaces of the connected parts. They are mostly required in the presence of fatigue with reversal of the loading, oversized holes, slotted holes (except when the load is normal to the slot), and when slipping at the faying surface would be detrimental to the structure’s performance. Faying surface preparation ***is required*** for these joints.

The minimum pretension load for A325 and A490 bolts are listed on LRFD Table J3.1. This load is equal to 70% of the minimum tensile strength of bolts.

Connections in ordinary building-type structures will most likely be that of the snug-tightened joint type.

5. Selected LRFD General Provisions

a) The minimum factored load to be used in designing a connection is 10 kips, except for lacing, sag rods, or girts (LRFD Section J1.7).

b) Load sharing for new work is not permitted in bearing-type connections between high strength or A307 bolts and welds in the connection (LRFD J1.9).

c) The installation and inspection of high strength bolts shall be done in accordance with the 2000 RCSC specifications sections 7, 8 & 9 (included in the LRFD manual).

d) Bolt length shall be properly selected to ensure adequate thread engagement per 2000 RCSC specifications, section 2.3.2.

6. Design Strength of Fasteners

The design shear strength of fasteners is specified in AISC LRFD Section J3.6 and Table J3.2

i) Design Tension Strength

$\phi R_n = \phi\, A_b\, F_n$ **(Eq. 2)**

Where:
ϕ = Resistance Factor = 0.75 (Table J3.2)
A_b = Nominal unthreaded body area of bolt or threaded part, in^2
F_n = Nominal tensile strength, $F_t = 0.75\ F_u$
F_u = minimum tensile strength of bolt material, LRFD Table 2-3

A307 Grade A: F_u = 60 ksi
A325: F_u = 120 ksi (for 0.5 in. to 1 in. bolts); 105 ksi (over 1 in. to 1.5 in.)
F1852: F_u = 120 ksi (for 0.5 in. to 1 in. bolts); 105 ksi (for 1.125 in.)
A490: F_u = 150 ksi (for 0.5 in. to 1.5 in. bolts)

ii) Design Shear Strength in Bearing-type Connection

The general form of equation 2 above applies with the following items (see LRFD Tables J3.2 and 7-10 below):

F_n = Nominal shear strength, $F_v = 0.40\ F_u$ for bolts when threads are not excluded from shear planes, i.e. A325-N or A490-N

F_n = Nominal shear strength, $F_v = 0.50\ F_u$ for bolts when threads are excluded from shear planes, i.e. A325-X or A490-X

In addition, when a bolt carrying load passes through fillers or shims in a shear plane, the provisions of LRFD section J3.6 apply.

Values of design shear strength for A325, A490, and A307 are listed in LRFD Table 7-10

7. Geometric Layout of Structural Bolts

a) Size and Use of Hole (LRFD section J3.2)

The maximum sizes of holes for rivets and structural bolts are given in Table J3.3. These holes are classified as:

i) Standard holes – 1/16 in. larger than the nominal bolt diameter
ii) Oversized holes – not allowed in bearing-type connections
iii) Short-slotted holes – allowed in both slip-critical and bearing-type connections, but the length have to be normal to the direction of the load in bearing type connections.
iv) Long-slotted holes – allowed in only one of the connected parts of either a slip-critical or bearing type connection at an individual faying surface. They are permitted without regard to direction of loading in slip-critical connections, but should be normal to the direction of loading in bearing-type connections.

b) Minimum Spacing (LRFD section J3.3), **Figure 10**

The distance between the centers of standard, oversized, or slotted holes should be $2^{2}/_{3}$ times the nominal diameter of the fastener, d. LRFD also states that this minimum distance should be preferably 3 times d.

c) Minimum Edge Distance (LRFD section J3.4), **Figure 10**

The distance from the center of a standard hole to an edge of a connected part should not be less than the values from Table J3.4.

d) Maximum Spacing and Edge Distance (LRFD section J3.5)

The maximum edge distance is given as 12 times the thickness of the connected part under consideration, but less than 6 inches. The maximum longitudinal spacing of connectors is specified as:

i) For painted members or unpainted members not subject to corrosion, the maximum spacing is 24 times the thickness of the thinner part or 12 in.

ii) For unpainted members of weathering steel subject atmospheric corrosion, the maximum spacing is limited 14 times the thickness of the thinner plate or 7 in.

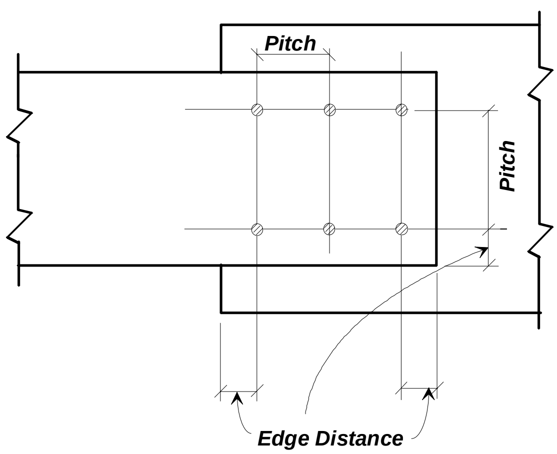


Figure 10

8. Design Bearing Strength (LRFD J3.10)

The strength of connection in bearing is taken at the bolt holes per AISC LRFD section J3.10.

Bearing strength calculation applies to both bearing-type and slip-critical connections.

The design bearing strength at the bolt hole is $\phi R_{n.}$
a) The design bearing strength is for service load when deformation is a design consideration (the hole edge deformation is limited to a maximum of ¼”). The bolt is also in a connection with standard, oversized, and short-slotted holes independent of the direction of loading, or a long-slotted hole with the slot parallel to the direction of the bearing force:

$R_n = 1.2L_c tF_u \leq 2.4dtF_u$ **(Eq. 3 ; LRFD J3-2a)**

b) When deformation at the bolt hole at service load is not a design consideration (hole ovalization, deformation greater than ¼”)

$R_n = 1.5L_c tF_u \leq 3.0dtF_u$ **(Eq. 3 ; LRFD J3-2b)**

c) For a bolt in a connection with long-slotted holes with the slot perpendicular to the direction of force:

$R_n = 1.0L_c tF_u \leq 2.0dtF_u$ **(Eq. 4 ; LRFD J3-2c)**

Where:
ϕ = Resistance Factor = 0.75
F_u = minimum tensile strength of the connected material, ksi
L_c = Clear distance in the direction of the force, between the edge of the hole and the edge of the adjacent hole or edge of the material, in.
d= bolt diameter, in
t = thickness of connected material, in

Another provision is given when deformation at the bolt hole at service load is not a design consideration (LRFD equation J-3-2b)

The design bearing strength for connections is computed as the sum of the bearing resistance of the individual bolts.

8. Design Strength of Connecting Elements

The design strength of the connected parts in a connection is covered under LRFD sections J4. & J5.

a) Design Rupture Strength, LRFD J4.

Block Shear Rupture Strength is the limit-state resistance determined by the sum of the shear strength on a failure path(s) and the tensile strength on perpendicular section.

The criteria stated in the following formulations:

When: $F_u A_{nt} \geq 0.6 F_u A_{nv}$
$\phi R_n = \phi[0.60F_y A_{gv} + F_u A_{nt}] \leq \phi[0.60F_u A_{nv} + F_u A_{nt}]$ (Eq. 5 ; J4-3a)

When: $F_u A_{nt} < 0.6 F_u A_{nv}$
$\phi R_n = \phi[0.60F_u A_{nv} + F_y A_{gt}] \leq \phi[0.60F_u A_{nv} + F_u A_{nt}]$ (Eq. 6 ; J4-3b)

Where
$\phi = 0.75$
A_{gv} = gross area subject to shear, in^2
A_{gt} = gross area subject to tension, in^2
A_{nv} = net area subject to shear, in^2
A_{nt} = net area subject to tension, in^2

The block shear strength measures the tearing out the edge of one of the attached members.

b) Design Strength of Connecting Members in Tension, LRFD J5.

The design strength, ϕR_n, of bolted connecting elements statically loaded in tension shall be the lower limit-state value of yielding, rupture of the connecting elements, and block shear rupture.

i) Tension yielding of the connecting members:

$$\phi = 0.90$$
$$R_n = A_g F_y \qquad \text{(Eq. 7 ; LRFD J5-1)}$$

ii) Tension rupture of the connecting members:

$$\phi = 0.75$$
$$R_n = A_n F_u \qquad \text{(Eq. 8 ; LRFD J5-2)}$$

A_n is the net area, not to exceed 0.85 A_g

Example – Design of a Bolted Tension Bearing-Type Connection

The connection shown in **Figure 11** consists of two plates that transfer a dead load of 28 kips and a live load of 55 kips in tension to a single 12 in plate. The material specification of all plates is ASTM A36, with F_u = 58 ksi, and F_y = 36 ksi. The bolts are ¾ in A325-N placed in two rows.

Find:
a) The number of bolts required
b) The width and thickness of the narrow plates
c) The thickness of the 12” wide plate
d) The design bearing strength of the connection
e) The block shear rupture strength of the tension members and gusset plate

Solution:

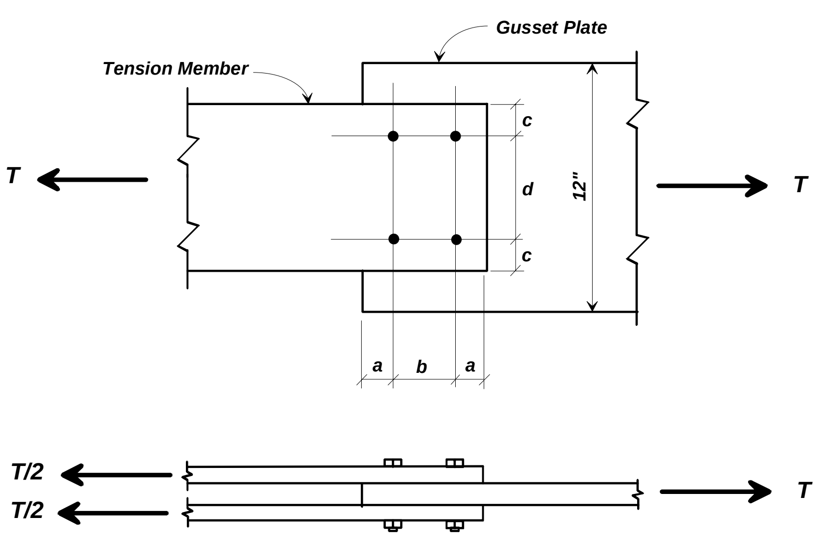


Figure 11

a) Factored Design Loads Per LRFD Part 2, load combination per ASCE 7-98

$U = 1.2\ D + 1.6\ L$

$T_u = 1.2(28) + 1.6\ (55) = 121.6\ k$

The bolts resist the loads is a “double shear” load transfer, the joint being a “snug-tightened” type, with the threads included in the shear plane

From LRFD Table 7-10, the design shear strength is

$\phi R_{n.} = 31.8$ kips / bolt

No. of Bolts required = 121.6 / 31.8 = 3.8 thus **use 4 – ¾” A325-N bolts**

b) The ¾-in bolts require a minimum edge distance of 1 ¼-in per LRFD Table J3.4 (at a sheared edge), the recommended spacing is taken as 3 x bolt diameter = 2.25 in., let’s use 3”

The minimum width of the tension members can be found as:

$$W = 2(1.25) + 2.25 = 4.75 \text{ in}$$

Considering no additional constraints, let’s try a width = 5 in. for these members

Design strength of connecting elements in tension (LRFD J5.2)

Design the tension members for yielding in the gross section, the design tension strength in yielding is ϕR_n with $\phi = 0.90$

Equating ϕR_n to the applied load, where $R_n = F_y A_g$

$$0.90 F_y A_g = 0.9(36)(5t) = 121.6 \text{ kips}$$

Solving for the thickness required, t = 121.6 / (2 x 0.90 x 180) = 0.375 in

Therefore, the thickness required based on yielding of the gross section is 3/8 in.

Check the plates for the limit-state of tension fracture in the net section:

$\phi = 0.75$

$\phi R_n = \phi F_u A_n = 0.75(58)1.22 \times 2 =$ **106.1 kips < 121.6 kips NG**

Where, $A_n = 5(0.375) – 2(3/4 + 1/8)\ 0.375 = 1.22\ in^2$

Note: LRFD Chapter B, section B2, requires that in computing the net area for tension and shear, the width of the bolt hole be taken as 1/16-in greater than the nominal bolt hole.

Thus increase the plate thickness or the width of the narrow plate,
Let's increase the width to 6"

The revised, $A_n = 6(0.375) – 2(3/4 + 1/8)\ 0.375 = 1.59\ in^2 < 0.85A_g$
$\phi R_n = \phi F_u A_n = 0.75(58)1.59 \times 2 =$ **138.3 kips > 121.6 kips ∴OK**

Use 3/8" x 6" Plates for the Tension Members

c) For the 12-in wide gusset plate, the thickness for gross yielding and fracture in the net section is computed as for the narrow plates

Gross yielding:

$t = 121.6 / (0.90 \times 12 \times 36) =$ **0.312 in < 3/8-in ∴OK**

Fracture in the net section:
$\phi R_n = \phi F_u A_n = 0.75(58)\ 3.82 =$ **166.1 kips > 121.6 kips ∴OK**

Where, $A_n= 12(0.375) – 2(3/4 + 1/8)\ 0.375 = 3.84\ in^2$ use $0.85A_g = 3.82\ in^2$

To prevent block shear rupture, try a ½" thick gusset plate since this plate has to resist the full load, and the tension members resist only half of the load.

Try 1/2" x 12" Gusset Plate

d) The design bearing strength of the connection

The deformation of the bolt hole at service load is a design consideration

The bolt pattern of the tension members is the same as the gusset (see Figures 12, & 13) and since their combined thickness exceed the gusset plate thickness, the latter controls the bearing strength of the connection.
The edge distance is 1.25" and the distance between the bolts is 3" (See Figure 13)

Per LRFD section J3.10, the bearing strength at the bolts is ϕR_n

Use Equation 3 (LRFD Eq. J3-2a):
$\phi = 0.75$ and $R_n = 1.2L_c t F_u \leq 2.4dtF_u$ (LRFD Eq. J3-2a)
d = 0.75 in
t = 0.5 in

L_c = clear distance, in the direction of the force, between the edge of the hole and the edge of the adjacent hole or edge of the material, in

The bolt hole dimension is taken as the bolt diameter + 1/16 in = h = 0.81 in

Edge bolts, $L_c = 1.25 – (0.81/2) = 0.85$ in
Interior bolts, $L_c = 3 – (0.81) = 2.19$ in

Therefore,
Edge bolts: $\phi R_n = 0.75 \times 1.2(0.85)(0.5)58 = 22.2$ kips
Interior bolts: $\phi R_n = 0.75 \times 1.2(2.19)(0.5)58 = 57.2$ kips use 39.1 kips

Limitation of capacity -→ $0.75 \times 2.4(0.75)(0.5)(58) = 39.1$ kips

Therefore, the total bearing strength at the bolt hole is

$\phi R_n = 2(22.2) + 2(39.1) =$ **122.6 kips > 121.6 kips ∴OK**

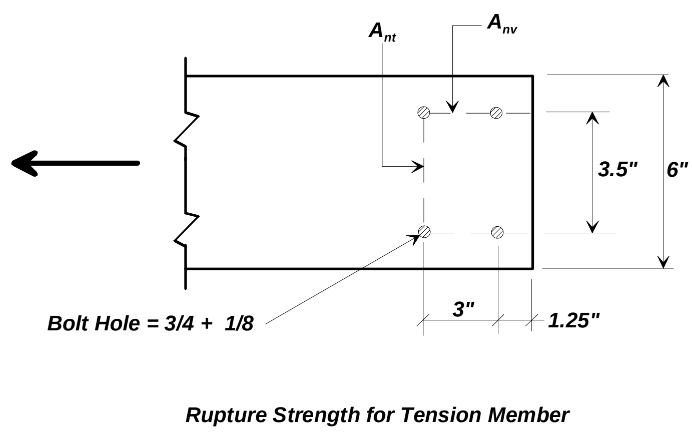
f) Design Rupture Strength of the connected plates (LRFD J4)

i) Shear Rupture Strength (J4.1)

$\phi R_n = \phi 0.6 F_u A_{nv}$ (LRFD J4-1)

The failure block for the gusset plate is identical as the block for the tension members, **Figures 12 and 13.**

The gusset plate resists the full factored tension, while each of the tension members take one-half of the total load, thus the gusset plate rupture design strength will control.



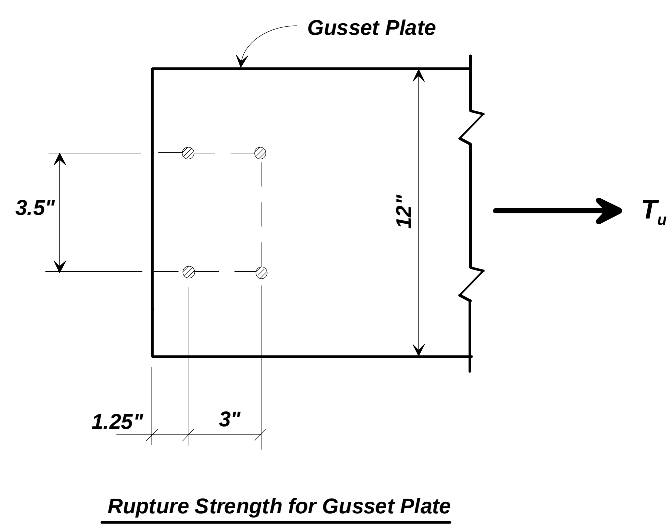


Figure 12 **Figure 13**

$\phi = 0.75$
$F_u = 58$ ksi
$A_{nv} = 2 \times 0.5\,[\,3 + 1.25 - 1.5(0.875)] = 2.94\ \text{in}^2$
Note, there is 1.5 x bolt hole in the path of the shear path

For the gusset plate:

$R_n = 0.6 \times 58 \times 2.94 =$ **102.3 kips**

ii) Tension Rupture Strength (LRFD J4.2):

$\phi R_n = \phi F_u A_{nt}$ (LRFD J4-2)

$\phi = 0.75$

$A_{nt} = 0.5\ (3.5 – 0.875) = 1.31\ in^2$

For the gusset plate:
$R_n = 58 \times 1.31 =$ **75.9 kips**

iii) Block Shear Rupture Strength (LRFD J4.3):

For the gusset plate, $F_u A_{nt} = 75.9$ kips $< 0.6\ F_u A_{nv} = 102.31$ kips
Since $F_u A_{nt} < 0.6\ F_u A_{nv}$ then LRFD Equation (J4-3b) governs block shear rupture strength

$\phi R_n = \phi[0.60F_u A_{nv} + F_y A_{gt}] \leq \phi[0.60F_u A_{nv} + F_u A_{nt}]$ (J4-3b)

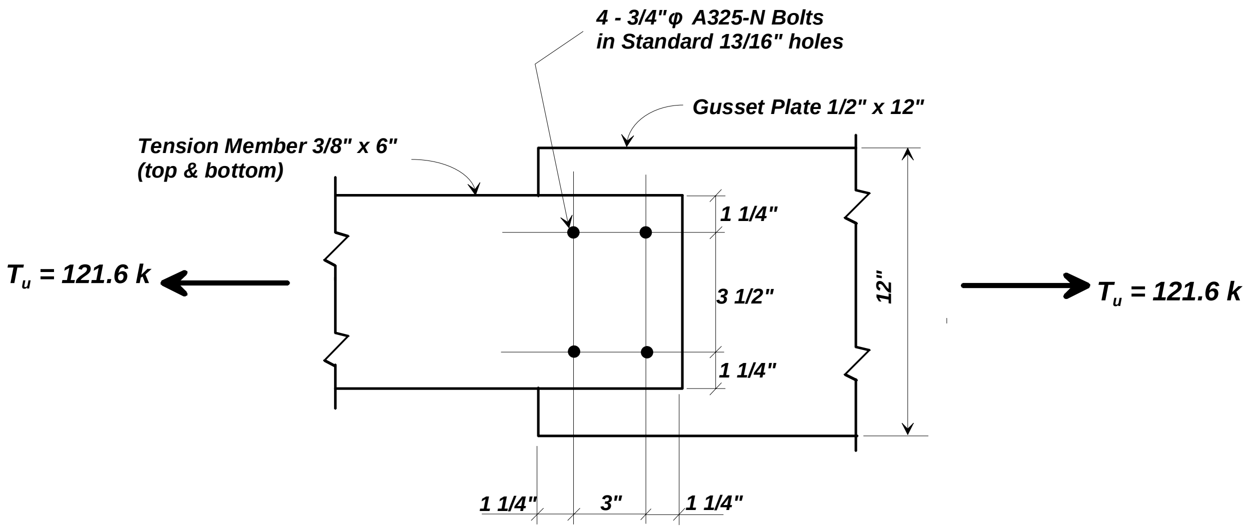
$\phi = 0.75$

$\phi R_n = 0.75[0.60 \times 58 \times 2.94 + 36 \times 3.5 \times 0.5] =$ **123.9 kips > 121.6 kips Ok**

Check for limit
$0.75\ [0.60 \times 58 \times 2.94 + 58 \times 1.31] = 133.7$ kips > 123.9 kips OK

The tension members are adequate since they will have a larger computed block shear strength than the gusset plate ($2\ t = 0.75\ in^2 > 0.50\ in^2$).
The final connection is shown in **Figure 14.**

Use ½” x 12” Gusset Plate


4 - 3/4"φ A325-N Bolts
in Standard 13/16" holes
Gusset Plate 1/2" x 12"
Tension Member 3/8" x 6"
(top & bottom)
T_u = 121.6 k
1 1/4"
3 1/2"
1 1/4"
12"
T_u = 121.6 k
1 1/4"
3"
1 1/4"

<u>Figure 14</u>