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Course Description

Unmasking the Ductility Factor, Shear Rupture, and Element Capacity in Welded Connections

January 24, 2018

Several considerations need to be made while in the process of designing welds and welded connections. This webinar will provide guidance on some of the more challenging considerations. Specifically, when (1) should the ductility factor be applied to a weld; (2) is the load path from the weld to the connecting element(s) unclear in regard to shear rupture checks, and; (3) should a weld be sized to develop the strength of a connecting plate? This presentation provides background into the development of these design considerations, and discusses the application of these limit state checks.

Learning Objectives

- Describe the background related to the development of shear rupture in base material adjacent to welds.
- Describe the background related to the development of the weld ductility factor (the Richard factor).
- When designing welds identify when it is intended to develop the strength of a connecting element.
- Identify the appropriate application of the weld ductility factor.
- Describe how to consider shear rupture in your welded connection design when shear rupture cannot be directly calculated.

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Unmasking the Ductility Factor, Shear Rupture, and Element Capacity in Welded Connections

written and presented by:
Patrick J. Fortney, Ph.D., P.E., S.E., P.Eng
Associate Professor – Educator
University of Cincinnati
Department of Civil and Architectural Engineering and Construction Management

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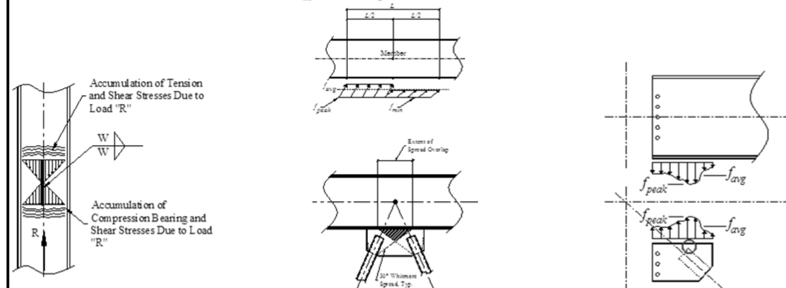
Unmasking the Ductility Factor, Shear Rupture, and Element Capacity in Welded Connections



Note that the material presented is based on a paper submitted to EJ:

Fortney, P. J., Muir, L.S., Thornton, W.A. Thornton (2018), "Guidance on Shear Rupture, Ductility, and Element Capacity in Welded Connections," *Engineering Journal*, American Institute of Steel Construction, (submitted for review)

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AGENDA

- ❖ Shear Rupture Near Welds
  - ➢ Background
  - ➢ Discussion
- ❖ Weld Ductility Factor
  - ➢ Background
  - ➢ Discussion
- ❖ Element-Capacity Welds
  - ➢ Background
  - ➢ Discussion

All discussion based on 2016 Steel Construction Manual

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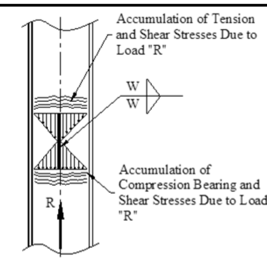
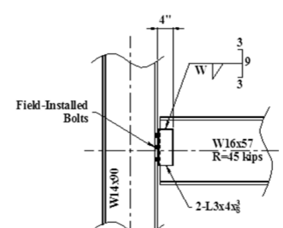


Shear Rupture Near Welds



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Shear Rupture Near Welds

Part 9 of the AISC *Manual* provides a brief discussion of how to address base material rupture strength at welds entitled,

"**Connecting Element Rupture Strength at Welds**."

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Shear Rupture Near Welds

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"**Connecting Element Rupture Strength at Welds**."

As briefly discussed in the AISC *Manual*, this section is intended to be applied in cases where the demand on the base material adjacent to the welds is "***not readily known***."

I - 14

Shear Rupture Near Welds

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"**Connecting Element Rupture Strength at Welds**."

As briefly discussed in the AISC *Manual*, this section is intended to be applied in cases where the demand on the base material adjacent to the welds is "***not readily known***."

In this case, the shear rupture strength of the base material adjacent to the weld is ensured to at least match the strength of the weld.

I - 15

Shear Rupture Near Welds

Part 9 of the AISC *Manual* provides a brief discussion of how to address base material rupture strength at welds entitled,

"**Connecting Element Rupture Strength at Welds**."

As briefly discussed in the AISC *Manual*, this section is intended to be applied in cases where the demand on the base material adjacent to the welds is "***not readily known***."

In this case, the shear rupture strength of the base material adjacent to the weld is ensured to at least match the strength of the weld.

- Let's look at those two equations, and see how they were derived.

I - 16

Shear Rupture Near Welds

Part 9 of the AISC *Manual* provides a brief discussion of how to address base material rupture strength at welds entitled,

"**Connecting Element Rupture Strength at Welds**."

The equations given in the AISC *Manual* (Equations 9-2 and 9-3) are repeated here for convenience.

$$t_{\min} = \frac{3.09D}{F_u} \qquad \text{(EQ. 9-2)}$$

$$t_{\min} = \frac{6.19D}{F_u} \qquad \text{(EQ. 9-3)}$$

I - 17

Shear Rupture Near Welds

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$$t_{\min} = \frac{3.09D}{F_u} \qquad \text{(EQ. 9-2)}$$

$$t_{\min} = \frac{6.19D}{F_u} \qquad \text{(EQ. 9-3)}$$

D is the weld size in sixteenths of an inch

F_u is tensile strength of the base material

t_{min} is minimum required thickness of the base material

I - 18

Shear Rupture Near Welds

Again, the intent of these equations is to ensure that the base material adjacent to the weld can develop the strength of the weld material.

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Shear Rupture Near Welds

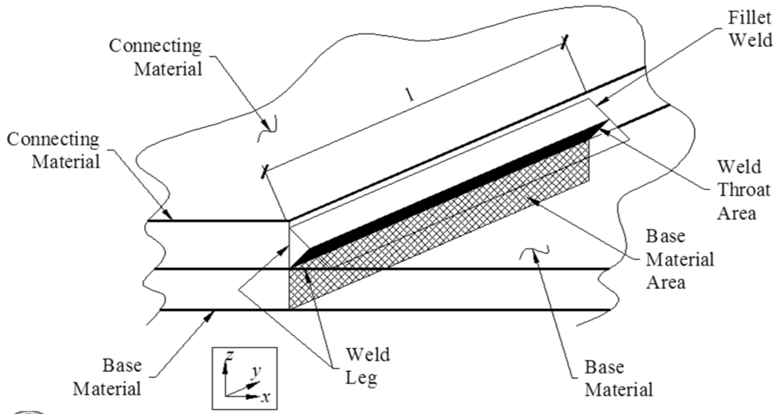
Again, the intent of these equations is to ensure that the base material adjacent to the weld can develop the strength of the weld material.

The derivation of the two equations is relatively straightforward…

I - 20

Shear Rupture Near Welds

For a *one-sided weld* condition…



Shear Rupture Near Welds

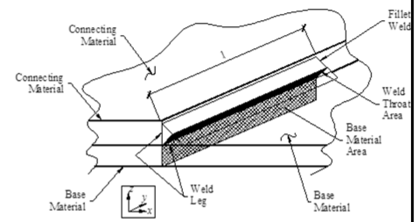
The *fillet weld throat* (shown with solid shading) is given by the following…

$$R_{nw} = 0.60F_{EXX}\cos(45)\left(\frac{D}{16}\right)l$$

If $F_{EXX} = 70$ ksi

$$R_{nw} = (0.60)(70)\cos(45)\left(\frac{D}{16}\right)l$$

$$R_{nw} = 1.856Dl$$



one-sided weld

Shear Rupture Near Welds

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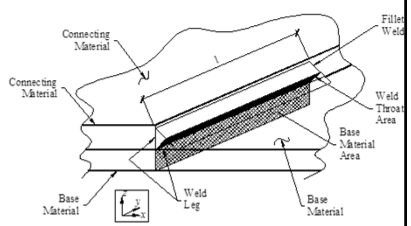
If $F_{EXX} = 70$ ksi

$$R_{nw} = (0.60)(70)\cos(45)\left(\frac{D}{16}\right)l$$

$$R_{nw} = 1.856Dl$$

The rupture area of the base material (shown with the cross-hatching) is…

$$R_{np} = 0.60F_u l t_{min}$$



one-sided weld

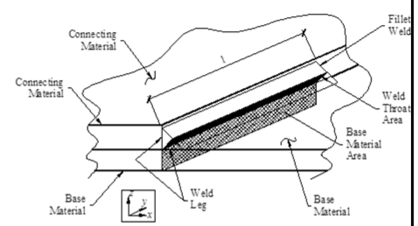
Shear Rupture Near Welds

Setting R_{nw} and R_{np} equal to each other gives the following…

$$R_{nw} = R_{np}$$

$$1.856Dl = 0.60F_u l t_{min}$$

$$t_{min} = \frac{1.856Dl}{0.60F_u l}$$



one-sided weld

Shear Rupture Near Welds

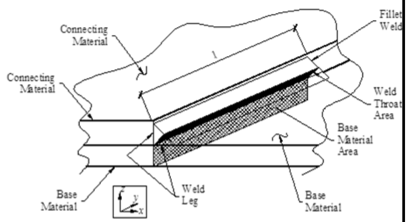
Setting R_{nw} and R_{np} equal to each other gives the following…

$$R_{nw} = R_{np}$$

$$1.856Dl = 0.60F_u l t_{min}$$

$$t_{min} = \frac{1.856Dl}{0.60F_u l}$$

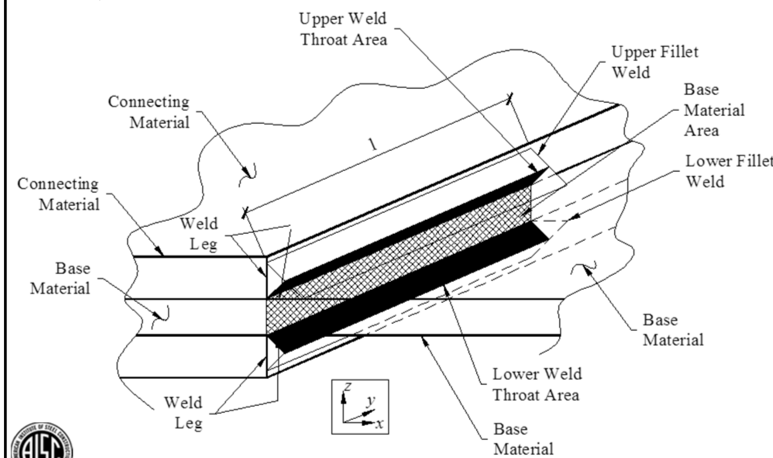
$$t_{min} = \frac{3.09D}{F_u} \quad \text{(as given by EQ. 9-2)}$$



one-sided weld

Shear Rupture Near Welds

Similarly, for a *two-sided weld* condition…



Shear Rupture Near Welds

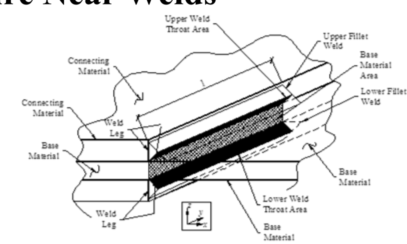
The *fillet weld throat* (shown with solid shading) is given by the following, considering that there are now two throats…

$$2R_{nw} = R_{np}$$

$$(2)3.712Dl = 0.60F_u l t_{min}$$

$$t_{min} = \frac{(2)3.712Dl}{0.60F_u l}$$

$$t_{min} = \frac{6.19D}{F_u} \quad \text{(as given by EQ. 9-3)}$$



two-sided weld

Shear Rupture Near Welds

Summary

One-Sided Weld

$$t_{min} = \frac{3.09D}{F_u} \quad \text{(EQ. 9-2 of } \textit{Manual}\text{)}$$

Two-Sided Weld

$$t_{min} = \frac{6.19D}{F_u} \quad \text{(EQ. 9-3 of } \textit{Manual}\text{)}$$

Shear Rupture Near Welds

Summary

One-Sided Weld

$$t_{\min} = \frac{3.09D}{F_u} \quad \text{(EQ. 9-2 of } \textit{Manual}\text{)}$$

Two-Sided Weld

$$t_{\min} = \frac{6.19D}{F_u} \quad \text{(EQ. 9-3 of } \textit{Manual}\text{)}$$

➢Let's take a closer look at Equation 9-2

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Shear Rupture Near Welds

One-Sided Weld

$$t_{\min} = \frac{3.09D}{F_u} \quad \text{(EQ. 9-2 of } \textit{Manual}\text{)}$$

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Shear Rupture Near Welds

One-Sided Weld

$$t_{\min} = \frac{3.09D}{F_u} \quad \text{(EQ. 9-2 of } \textit{Manual}\text{)}$$

This is simply the beginning of the derivation of the equation used to size a fillet weld.

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Shear Rupture Near Welds

One-Sided Weld

$$t_{\min} = \frac{3.09D}{F_u} \quad \text{(EQ. 9-2 of } \textit{Manual}\text{)}$$

This is simply the beginning of the derivation of the equation used to size a fillet weld.

$$R_{nw} = 0.60F_{EXX}\cos(45)\left(\frac{D}{16}\right)l$$

If $F_{EXX} = 70$ ksi

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$$R_{nw} = 1.856Dl$$

I - 32

Shear Rupture Near Welds

One-Sided Weld

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This is simply the beginning of the derivation of the equation used to size a fillet weld.

$$R_{nw} = 0.60F_{EXX}\cos(45)\left(\frac{D}{16}\right)l$$

$$\text{If } F_{EXX} = 70 \text{ ksi}$$

$$R_{nw} = (0.60)(70)\cos(45)\left(\frac{D}{16}\right)l$$

$$R_{nw} = 1.856Dl$$

- If we multiply $1.856Dl$ times the LRFD strength reduction factor, ϕ, or divide by the ASD strength reduction factor, Ω, we get the well-know equations.

I - 33

Shear Rupture Near Welds

Fillet Weld Strength (LRFD)

$$\phi R_{nw} = \phi 1.856Dl$$

$$\phi R_{nw} = (0.75)(1.856)Dl$$

$$\phi R_{nw} = 1.392Dl$$

Eq. 8-2a of the *Manual*

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Shear Rupture Near Welds

Fillet Weld Strength (LRFD)

$$\phi R_{nw} = \phi 1.856Dl$$

$$\phi R_{nw} = (0.75)(1.856)Dl$$

$$\phi R_{nw} = 1.392Dl$$

Eq. 8-2a of the *Manual*

Fillet Weld Strength (ASD)

$$\frac{R_{nw}}{\Omega} = \frac{1.856Dl}{\Omega}$$

$$\frac{R_{nw}}{\Omega} = \frac{1.856Dl}{2.00}$$

$$\frac{R_{nw}}{\Omega} = 0.928Dl$$

Eq. 8-2b of the *Manual*

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Shear Rupture Near Welds

When are the equations intended to be used?

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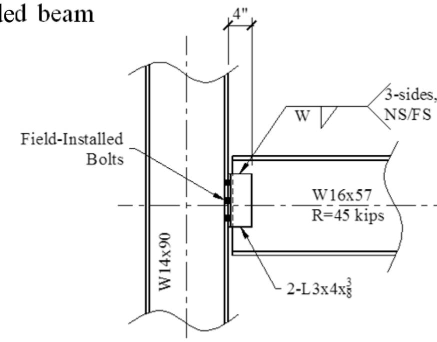
Shear Rupture Near Welds

When are the equations intended to be used?

These equations were originally developed to address shop welded beam end angle connections



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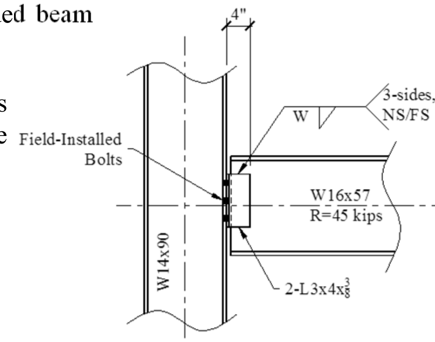
Shear Rupture Near Welds

When are the equations intended to be used?

These equations were originally developed to address shop welded beam end angle connections

The demand on the weld is "readily known" from the geometry and loading…



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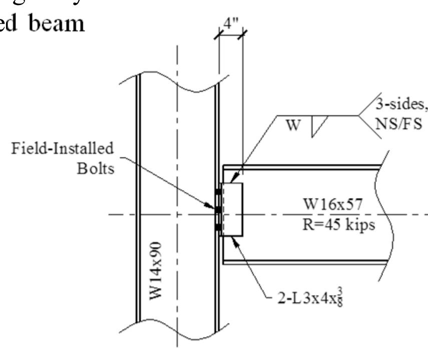
Shear Rupture Near Welds

When are the equations intended to be used?

These equations were originally developed to address shop welded beam end angle connections

The demand on the weld is "readily known" from the geometry and loading…

The demand on beam web, adjacent to the weld, is not "readily known."



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Shear Rupture Near Welds

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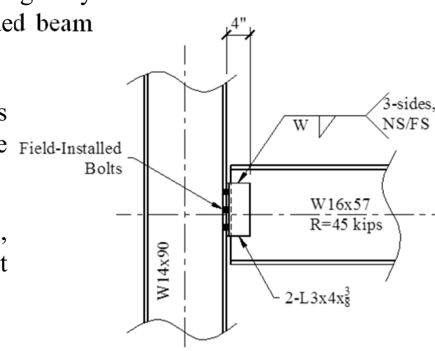

Equations 9-2 and 9-3 were developed for cases such as this…

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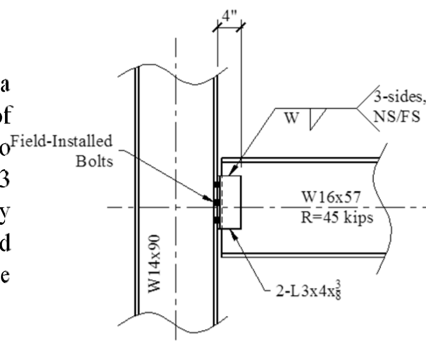
Shear Rupture Near Welds

When are the equations intended to be used?

In lieu of trying to develop a rigorous conservative model of the demand on the web adjacent to the weld, Equations 9-2 and 9-3 were developed to conservatively ensure that the base material could at least develop the strength of the weld.



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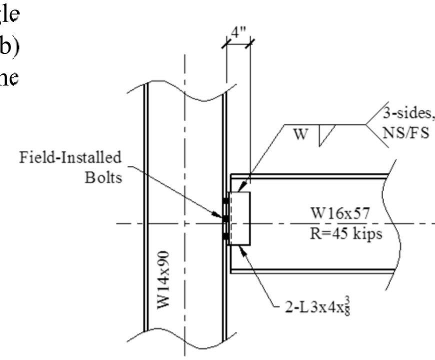
Shear Rupture Near Welds

When are the equations intended to be used?

For example, Equation 9-3 (angle on each side of the beam web) would be used to check the thickness of the beam web.



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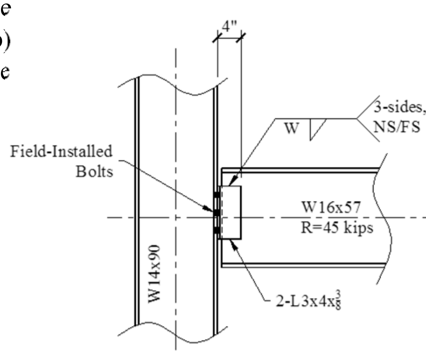
Shear Rupture Near Welds

When are the equations intended to be used?

For example, Equation 9-3 (angle on each side of the beam web) would be used to check the thickness of the beam web.

A W16x57 (A992-50) has a t_w = 0.430 in.
Assume D=3



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Shear Rupture Near Welds

When are the equations intended to be used?

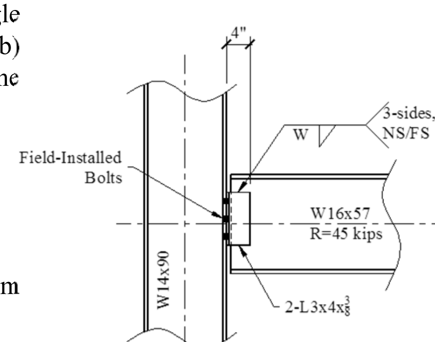
For example, Equation 9-3 (angle on each side of the beam web) would be used to check the thickness of the beam web.

A W16x57 (A992-50) has a t_w = 0.430 in.
Assume D=3

Equation 9-3 gives a minimum thickness of…

$$t_{\min} = \frac{6.19D}{F_u} = \frac{(6.19)(3)}{65}$$

$$t_{\min} = 0.287 \text{ in.} < t_w = 0.43 \text{ in.} \qquad \text{o.k.}$$



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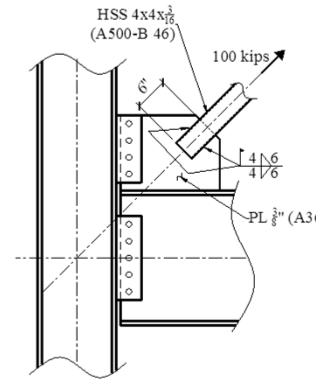
Shear Rupture Near Welds

When should these equations **not be used**?

Shear Rupture Near Welds

When should these equations **not** be used?

When you know the demand on the base material.

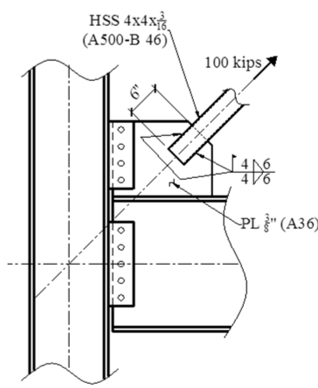


Shear Rupture Near Welds

When should these equations **not** be used?

When you know the demand on the base material.

Suppose you want to check shear rupture on the HSS walls…



Shear Rupture Near Welds

When should these equations **not** be used?

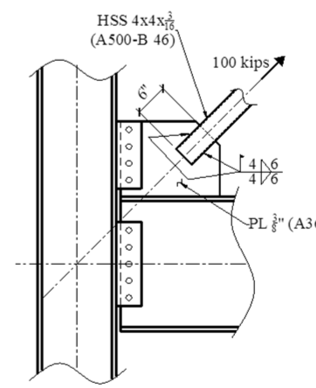
When you know the demand on the base material.

Suppose you want to check shear rupture on the HSS walls…

If you use Equation 9-2 (one-sided weld), you would get

$$t_{\min} = \frac{3.09D}{F_u} = \frac{(3.09)(4)}{58}$$

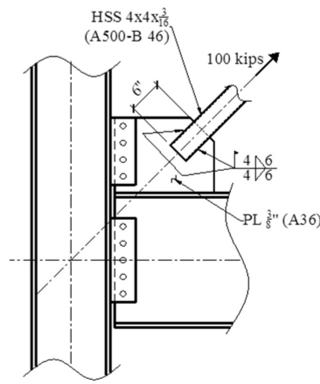
$$t_{\min} = 0.213 \text{ in.} > t_{des} = 0.174 \text{ in.} \qquad \text{n.g.}$$



Shear Rupture Near Welds

When should these equations **not** be used?

But we actually know the demand on the legs of the HSS walls (100 kips)



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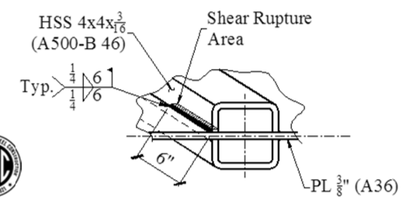
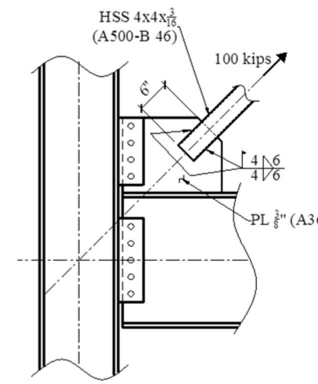
Shear Rupture Near Welds

When should these equations **not** be used?

But we actually know the demand on the legs of the HSS walls (100 kips)





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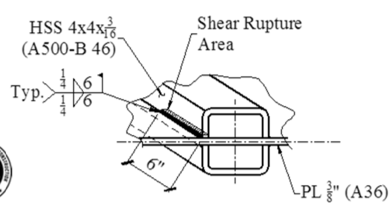
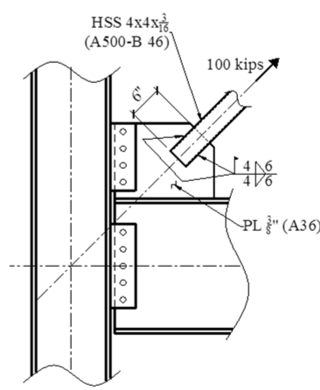
Shear Rupture Near Welds

When should these equations **not** be used?

But we actually know the demand on the legs of the HSS walls (100 kips)

$$\phi R_n = 4\phi(0.6F_u)t_{des}l$$
$$\phi R_n = (4)(0.75)(0.6)(58)(0.174)(6)$$
$$\phi R_n = 109 \text{ kips} > 100 \text{ kips} \qquad \text{o.k.}$$





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Shear Rupture Near Welds

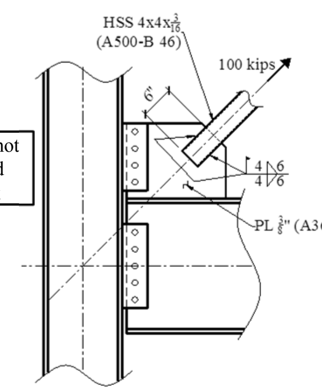
When should these equations **not** be used?

But we actually know the demand on the legs of the HSS walls (100 kips)

$$\phi R_n = 4\phi(0.6F_u)t_{des}l$$
$$\phi R_n = (4)(0.75)(0.6)(58)(0.174)(6)$$
$$\phi R_n = 109 \text{ kips} > 100 \text{ kips} \qquad \text{o.k.}$$

This is not a valid check

$$t_{min} = \frac{3.09D}{F_u} = \frac{(3.09)(4)}{58}$$
$$t_{min} = 0.213 \text{ in.} > t_{des} = 0.174 \text{ in.} \qquad \text{n.g.}$$



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Shear Rupture Near Welds

When should these equations **not** be used?

But we actually know the demand on the legs of the HSS walls (100 kips)

$$\phi R_n = 4\phi(0.6F_u)t_{des}l$$
$$\phi R_n = (4)(0.75)(0.6)(58)(0.174)(6)$$
$$\phi R_n = 109 \text{ kips} > 100 \text{ kips} \quad \text{o.k.}$$

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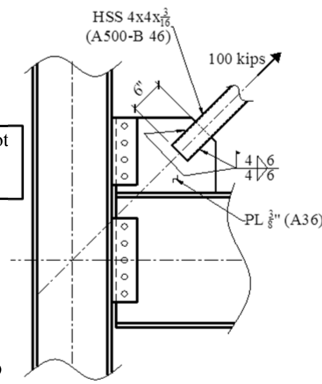
This is not a valid check

$$t_{min} = 0.213 \text{ in.} > t_{des} = 0.174 \text{ in.} \quad \text{n.g.}$$

What if we used equation 9-2 to check shear rupture, but used the actual weld size required for D??



I - 53

Shear Rupture Near Welds

When should these equations **not** be used?

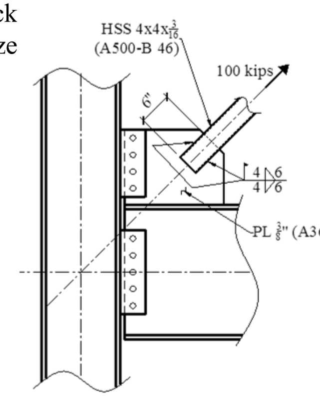
What if we used equation 9-2 to check shear rupture, but used the actual weld size required for D??

Shear rupture strength of walls

$$D_{req} = \frac{109}{(1.392)(4)(6)} = 3.2627$$

$$t_{min} = \frac{(3.09)(3.2627)}{58}$$

$$t_{min} = 0.174 \text{ in.} = t_{des} = 0.174 \text{ in.} \quad \text{o.k.}$$



I - 54

Shear Rupture Near Welds

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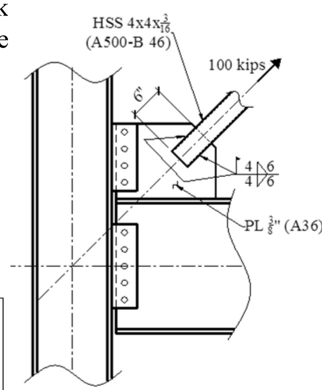
$$t_{min} = 0.174 \text{ in.} = t_{des} = 0.174 \text{ in.} \quad \text{o.k.}$$

Always compute shear rupture directly when you can reasonably understand the demand the base material.



I - 55

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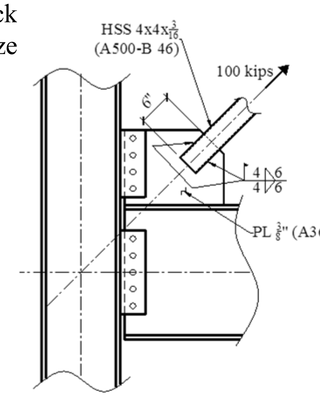
You run the risk of unnecessarily increasing thickness by a ratio of D_{prov} / D_{req}



I - 56

Shear Rupture Near Welds

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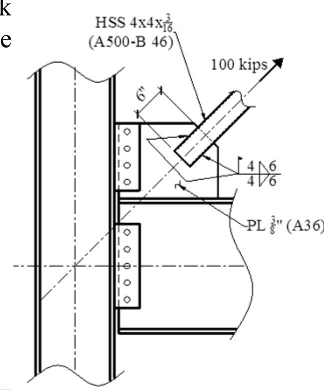


You run the risk of unnecessarily increasing thickness by a ratio of D_{prov} / D_{req}

This can be compounded if D_{min} controls the size of the weld

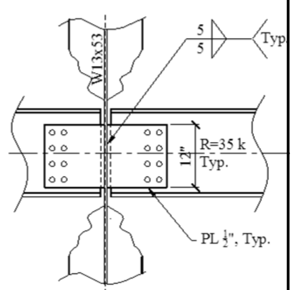
Shear Rupture Near Welds

Using 9-2 and 9-3 for sanity checks?

Shear Rupture Near Welds

Using 9-2 and 9-3 for sanity checks?

There are those who choose to check the web thickness of such a connection as shown.



Shear Rupture Near Welds

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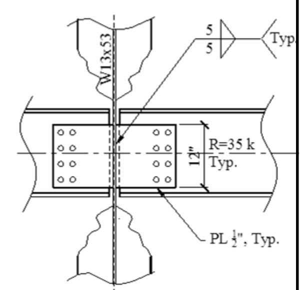
There are those who choose to check the web thickness of such a connection as shown.

In the absence of a clear understanding of how the column web actually absorbs the shear from the connection plate, some will do the following…



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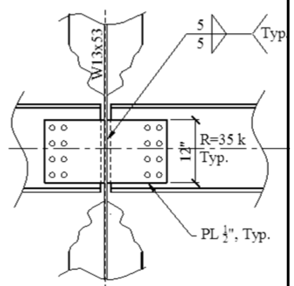

In the absence of a clear understanding of how the column web actually absorbs the shear from the connection plate, some will do the following…

Note that t_w for a W14x53 is 0.370"

$$t_{\min} = \frac{(6.19)(5)}{65} = 0.476 \text{ in}$$

$$t_{\min} = 0.476 \text{ in.} > t_w = 0.370 \text{ in.} \quad \text{n.g.}$$

I - 61

Shear Rupture Near Welds

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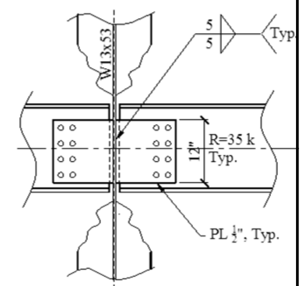

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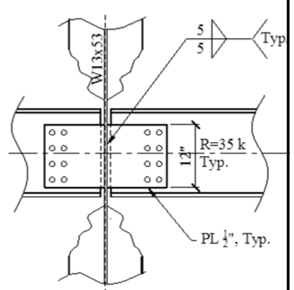
So is the column web really no good for these required loads??

I - 62

Shear Rupture Near Welds

Using 9-2 and 9-3 for sanity checks?

First of all, the weld size is a result of the $5/8t_p$ requirement; not based on the required load!



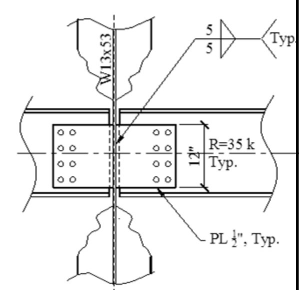
I - 63

Shear Rupture Near Welds

Using 9-2 and 9-3 for sanity checks?

First of all, the weld size is a result of the $5/8t_p$ requirement; not based on the required load!

So using these equations would be really conservative!



I - 64

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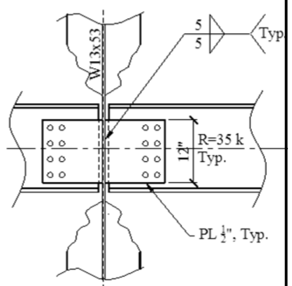
We could just directly compute shear rupture assuming that "failure" occurs as shear rupture along the depth of the connection



I - 65

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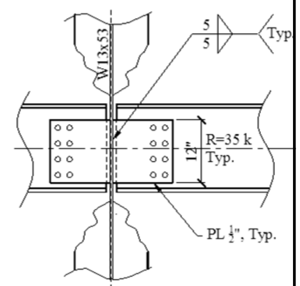

$$\phi R_n = (0.75)(0.6)(65)(12)(0.370)(2)$$

$$\phi R_n = 260 \text{ kips} > R_u = 35 \text{ kips} + 35 \text{ kips} = 70 \text{ kips} \quad \text{o.k.}$$

I - 66

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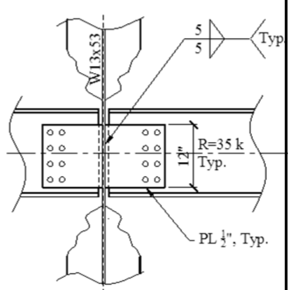

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But, is this really how the web absorbs this load??

I - 67

Shear Rupture Near Welds

Using 9-2 and 9-3 for sanity checks?

What if the previous calculation showed the web was too thin?

I - 68

Shear Rupture Near Welds

Using 9-2 and 9-3 for sanity checks?

What if the previous calculation showed the web was too thin?

Would the column size need to be changed; or the column reinforced?

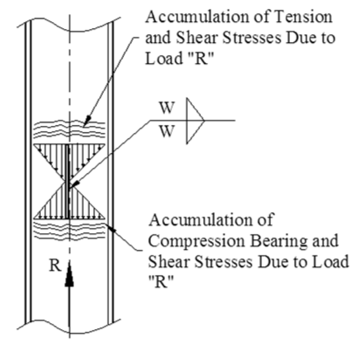
Shear Rupture Near Welds

Using 9-2 and 9-3 for sanity checks?

What if the previous calculation showed the web was too thin?

Would the column size need to be changed; or the column reinforced?

Depending on column properties (depth, thickness, etc.), the load transferred from the plate to the web will tend to hang from the web above and bear on the web below.



Shear Rupture Near Welds

Using 9-2 and 9-3 for sanity checks?

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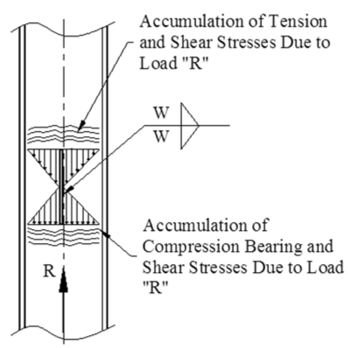
Would the column size need to be changed; or the column reinforced?

Depending on column properties (depth, thickness, etc.), the load transferred from the plate to the web will tend to hang from the web above and bear on the web below.

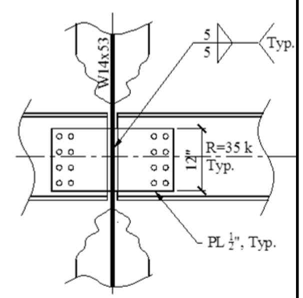
Some combination of these phenomena



Shear Rupture Near Welds

Using 9-2 and 9-3 for sanity checks?

If you absolutely feel you need to do something to check the web thickness, conservative options may be…

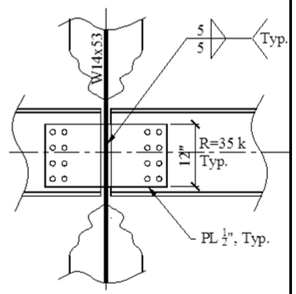


Shear Rupture Near Welds

Using 9-2 and 9-3 for sanity checks?

If you absolutely feel you need to do something to check the web thickness, conservative options may be…

Use Equation 9-2 or 9-3, but use the required weld size, based on load, rather than the provided weld size.



I - 73

Shear Rupture Near Welds

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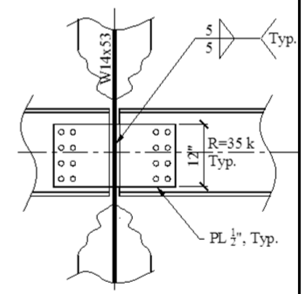
Use Equation 9-2 or 9-3, but use the required weld size, based on load, rather than the provided weld size.

$$D_{req} = \frac{35}{(1.392)(12)(2)} = 1.047$$

$$t_{\min} = \frac{(6.19)(1.047)}{65}$$

$$t_{\min} = 0.100 \text{ in.} < t_w = 0.370 \text{ in.} \quad \text{o.k.}$$



I - 74

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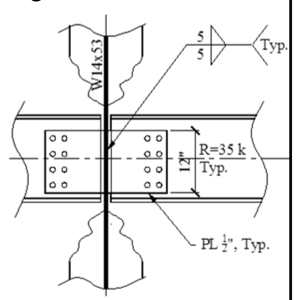


But, recognize that is still very conservative!

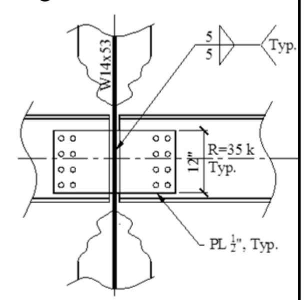
I - 75

Shear Rupture Near Welds

Using 9-2 and 9-3 for sanity checks?

If you absolutely feel you need to do something to check the web thickness, conservative options may be…

Or another option is to directly compute shear rupture assuming the effective rupture area is only along the connection length adjacent to each weld.



I - 76

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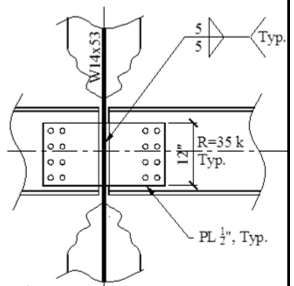

As calculated previously…

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I - 77

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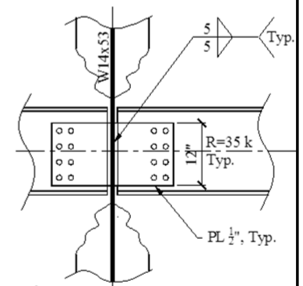

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But, recognize that even this is very conservative!

I - 78

Shear Rupture Near Welds

Poll Question 1

If Equations 9-2 or 9-3 given in Part 9 of the Manual is used to check shear rupture…

… when the intent is not to at least develop the strength of the weld,…

…what weld size should be used in order to not unnecessarily increase the plate thickness?

a) Use D_{min}
b) Use $D_{provided}$
c) Use $D_{required}$ (without D_{min} considerations)
d) Use $(5/8)t_p$

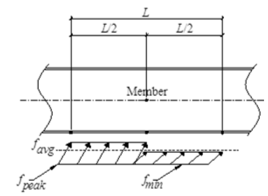
I - 79

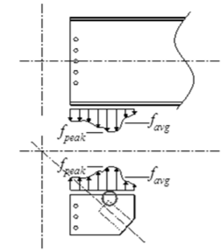
Shear Rupture Near Welds

End Part 1

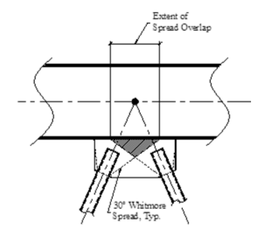
I - 80

Part II





Weld Ductility Factor



Weld Ductility Factor

The ductility factor for welds (some refer to this as the *Richard factor*), first showed up in AISC documents in the 1992 Manual of Steel Construction, Volume II: Connections.

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One of the assumptions in the development of the UFM is that interface forces are distributed uniformly along the interfaces…

II- 5

Weld Ductility Factor

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The ductility consideration arose during the development of the uniform force method (UFM); now commonly used for distributing forces in vertical brace connections framing to beam-column joints.

One of the assumptions in the development of the UFM is that interface forces are distributed uniformly along the interfaces…

…regardless of interface length, proximity of connected members, or other variables such as frame action (distortion).

II- 6

Weld Ductility Factor

Part 13 of the *Manual*, **BRACING CONNECTIONS**, **Available Strength** (page 13-11) states…

II- 7

Weld Ductility Factor

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"*Note that when the gusset is directly welded to the beam or column, the connection should be designed for the larger of the peak stress and* ***1.25*** *times the average stress, but the weld size need not be larger than that required to develop the strength of the gusset…*"

II- 8

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"*…this 25% increase is recommended to allow adequate redistribution of transverse stresses in the weld group...*"

II- 9

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"*…this 25% increase is recommended to allow adequate redistribution of transverse stresses in the weld group...*"

"*…This adjustment should not be applied to welds that resist only shear forces (Hewitt and Thornton, 2004).*"

II- 10

Weld Ductility Factor

In the 1992 Manual of Steel Construction, Volume II: Connections, the weld ductility factor was first introduced but, the factor was recommended to be **1.40**.

II- 11

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The 40% increase was a consensus requirement of the AISC COM at that time, based on a report submitted by Williams (a Ralph Richard student).

II- 12

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II- 13

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- Proximity, and
- Frame Distortion

…effects.

II- 14

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- Proximity, and
- Frame Distortion

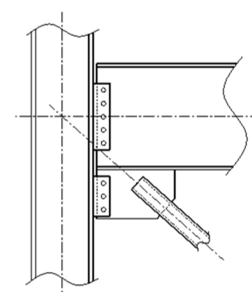
We'll look at this a little later.

…effects.

II- 15

Weld Ductility Factor

Williams' work was a nonlinear inelastic FEA study of braces framing to beam-column joints

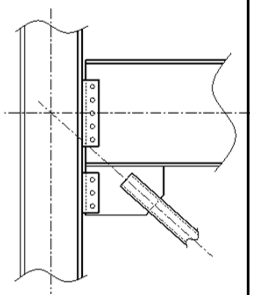


II- 16

Weld Ductility Factor

Williams' work was a nonlinear inelastic FEA study of braces framing to beam-column joints

Forty-five (45) specimens with varying geometry was considered. For each model, a plot of the ratio of the peak-to-average stress along the welded interface was plotted, as shown in the plot here.



II- 17

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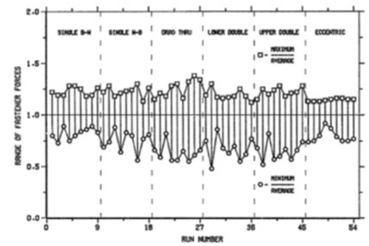
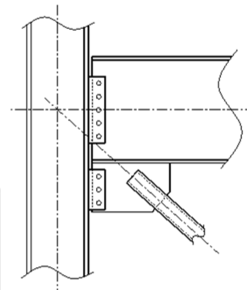
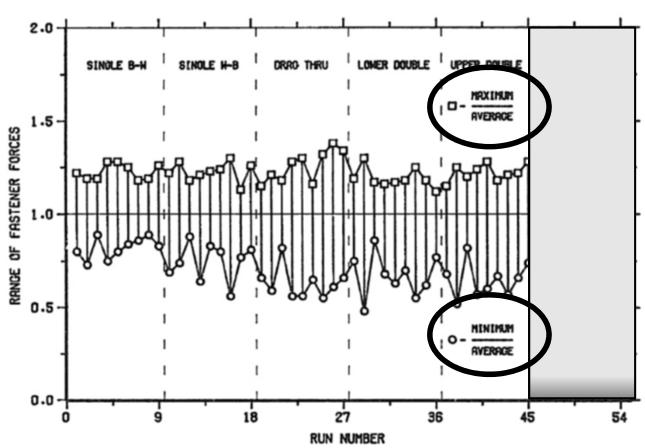


Figure 48. Uniformity of Fastener Force Distributions at Yield Load.



II- 18

Weld Ductility Factor



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II- 19

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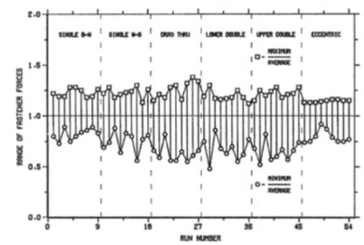

Figure 48. Uniformity of Fastener Force Distributions at Yield Load.

The largest ratio was 1.39

AISC COM rounded up to 1.40.

Run	Stress Ratio	Run	Stress Ratio	Run	Stress Ratio	Run	Stress Ratio	Run	Stress Ratio
1	1.22	10	1.22	19	1.16	28	1.20	37	1.16
2	1.20	11	1.31	20	1.22	29	1.32	38	1.26
3	1.19	12	1.19	21	1.19	30	1.18	39	1.20
4	1.30	13	1.22	22	1.29	31	1.17	40	1.26
5	1.29	14	1.24	23	1.32	32	1.18	41	1.30
6	1.28	15	1.26	24	1.18	33	1.19	42	1.19
7	1.19	16	1.32	25	1.33	34	1.27	43	1.22
8	1.20	17	1.14	26	$\boxed{1.39}$	35	1.18	44	1.22
9	1.29	18	1.26	27	1.37	36	1.12	45	1.30

II- 20

Weld Ductility Factor

In 2004, Hewitt and Thornton did a statistical analysis of the data.

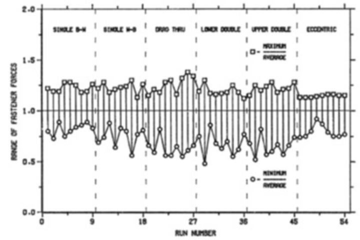

Figure 48. Uniformity of Fastener Force Distributions at Yield Load.

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1	1.22	10	1.22	19	1.16	28	1.20	37	1.16
2	1.20	11	1.31	20	1.22	29	1.32	38	1.26
3	1.19	12	1.19	21	1.19	30	1.18	39	1.20
4	1.30	13	1.22	22	1.29	31	1.17	40	1.26
5	1.29	14	1.24	23	1.32	32	1.18	41	1.30
6	1.28	15	1.26	24	1.18	33	1.19	42	1.19
7	1.19	16	1.32	25	1.33	34	1.27	43	1.22
8	1.20	17	1.14	26	1.39	35	1.18	44	1.22
9	1.29	18	1.26	27	1.37	36	1.12	45	1.30

II- 21

Weld Ductility Factor

In 2004, Hewitt and Thornton did a statistical analysis of the data.

Assuming a 90% confidence interval, they recommended that the increase be reduced from 40% to 25%.

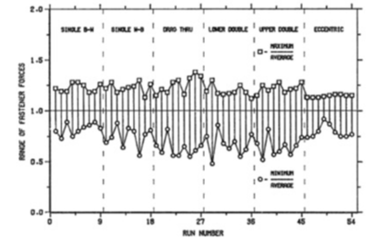

Figure 48. Uniformity of Fastener Force Distributions at Yield Load.

Run	Stress Ratio	Run	Stress Ratio	Run	Stress Ratio	Run	Stress Ratio	Run	Stress Ratio
1	1.22	10	1.22	19	1.16	28	1.20	37	1.16
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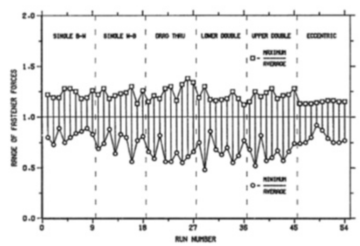
II- 22

Weld Ductility Factor

In 2004, Hewitt and Thornton did a statistical analysis of the data.

Assuming a 90% confidence interval, they recommended that the increase be reduced from 40% to 25%.

The AISC COM acted on that recommendation and lowered the increase to today’s value of 1.25.



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II- 23

Weld Ductility Factor

In 2004, Hewitt and Thornton did a statistical analysis of the data.

Assuming a 90% confidence interval, they recommended that the increase be reduced from 40% to 25%.

The AISC COM acted on that recommendation and lowered the increase to today’s value of 1.25.

On an aside, even at a 95% confidence, the value still comes out as ~1.25.

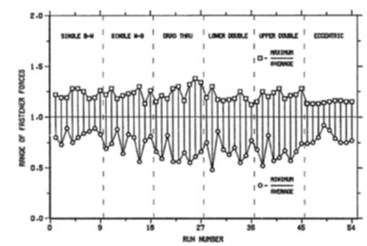

Figure 48. Uniformity of Fastener Force Distributions at Yield Load.

Run	Stress Ratio	Run	Stress Ratio	Run	Stress Ratio	Run	Stress Ratio	Run	Stress Ratio
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8	1.20	17	1.14	26	1.39	35	1.18	44	1.22
9	1.29	18	1.26	27	1.37	36	1.12	45	1.30

II- 24

Weld Ductility Factor

Recall that the COM concluded that adequate redistribution needed to be provided to account for

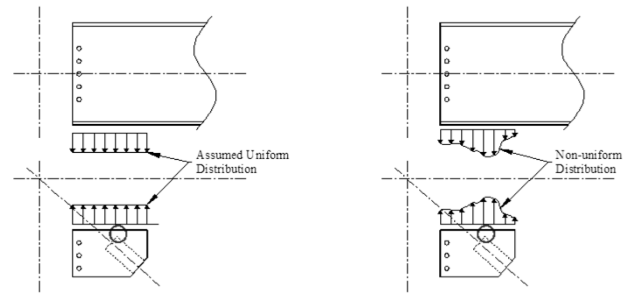
- ❖ Proximity and
- ❖ Frame Distortion

Weld Ductility Factor

Recall that the COM concluded that adequate redistribution needed to be provided to account for

- ❖ Proximity and
- ❖ Frame Distortion

Proximity



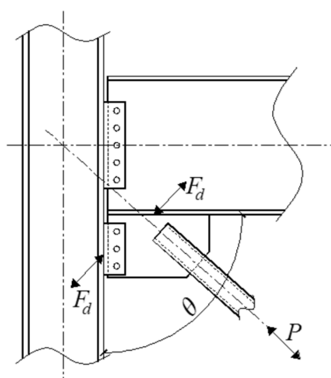
Weld Ductility Factor

Recall that the COM concluded that adequate redistribution needed to be provided to account for

- ❖ Proximity and
- ❖ Frame Distortion

Frame Distortion

When brace is in tension, θ tends to decrease; when the brace is in compression, θ tends to increase.



Weld Ductility Factor

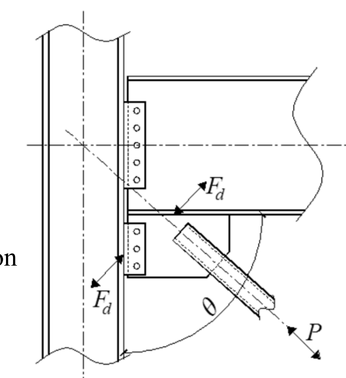
Recall that the COM concluded that adequate redistribution needed to be provided to account for

- ❖ Proximity and
- ❖ Frame Distortion

Frame Distortion

When brace is in tension, θ tends to decrease; when the brace is in compression, θ tends to increase.

Changes in θ induce distortional forces on the gusset interfaces

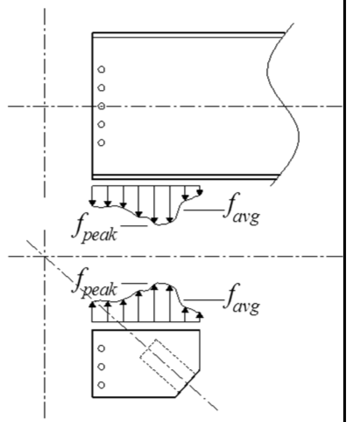


Weld Ductility Factor

Recall that the COM concluded that adequate redistribution needed to be provided to account for

- ❖ Proximity and
- ❖ Frame Distortion

Thus, as a result of <u>proximity</u> or frame <u>distortion</u>, or a <u>combination of both</u>, a ductility factor may need to be used to "correct" for the assumed uniform distribution.



Weld Ductility Factor

Although the ductility factor was originally developed for corner gussets, it has been used on many other various types of connections…

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- ❖ Hanger connections,
- ❖ Chevron gusset connections,
- ❖ Bracket connections, and
- ❖ Various other types of connections

It may very well be reasonable to do so when interface loads are assumed to be uniformly distributed and proximity or distortion can be reasonably assumed to be a consideration.

Let's look at proximity!

Weld Ductility Factor

Although the ductility factor was originally developed for corner gussets, it has been used on many other various types of connections…

- ❖ Hanger connections,
- ❖ Chevron gusset connections,
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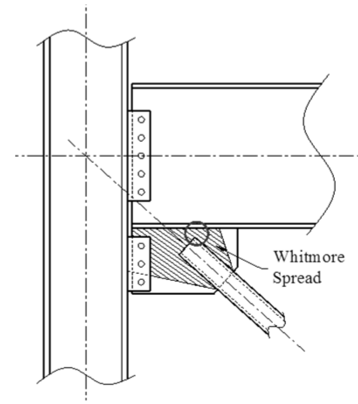
It may very well be reasonable to do so when interface loads are assumed to be uniformly distributed and proximity or distortion can be reasonably assumed to be a consideration.

II- 33

Weld Ductility Factor

Consider, again, the corner gusset connection. How might proximity be evaluated?

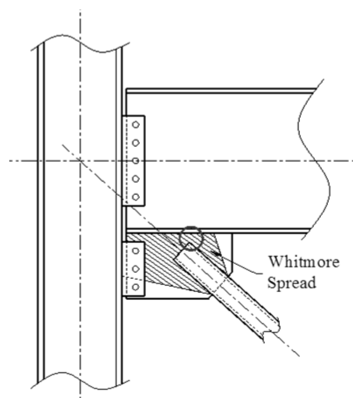


II- 34

Weld Ductility Factor

Consider, again, the corner gusset connection. How might proximity be evaluated?

The spread of the load through the gusset from the introduction of the load from the brace-to-gusset connection to the welded interface can be evaluated base on an assumed Whitmore spread



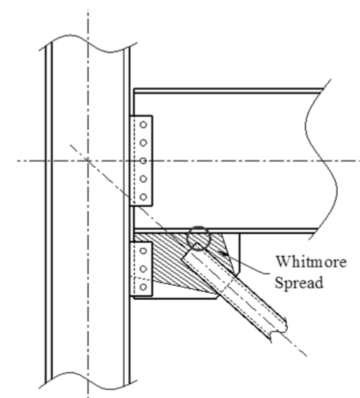
II- 35

Weld Ductility Factor

Consider, again, the corner gusset connection. How might proximity be evaluated?

The spread of the load through the gusset from the introduction of the load from the brace-to-gusset connection to the welded interface can be evaluated base on an assumed Whitmore spread

Assuming a uniform distribution at the welded interface may not be appropriate if the spread indicates that the entire interface is not engaged



II- 36

Weld Ductility Factor

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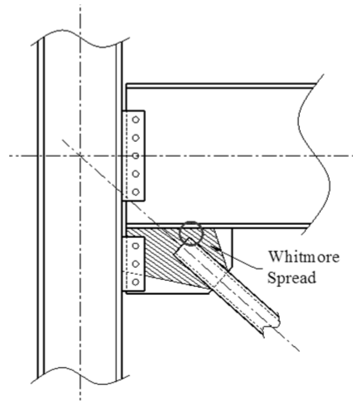


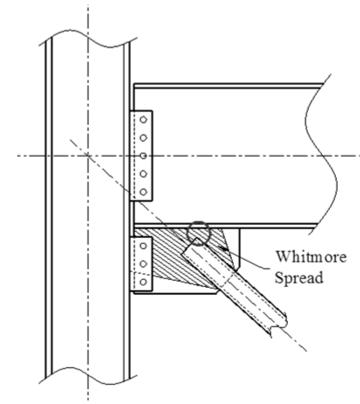
This could suggest that proximity is an issue

II- 37

Weld Ductility Factor

Consider, again, the corner gusset connection. How might proximity be evaluated?

What if the brace-to-gusset connection length was relatively longer while the distance from the end of the brace to the interface remained the same?



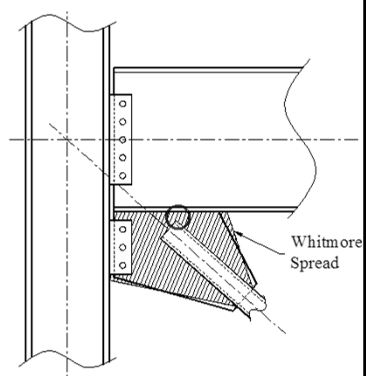
II- 38

Weld Ductility Factor

Consider, again, the corner gusset connection. How might proximity be evaluated?

What if the brace-to-gusset connection length was relatively longer while the distance from the end of the brace to the interface remained the same?

Something like this!



II- 39

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Consider, again, the corner gusset connection. How might proximity be evaluated?

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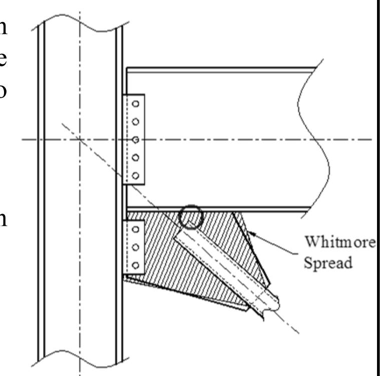
Something like this!

Proximity now does not seem to be an issue



II- 40

Weld Ductility Factor

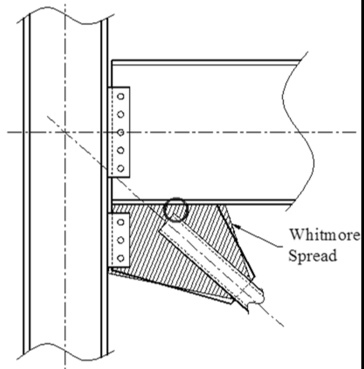
Consider, again, the corner gusset connection. How might proximity be evaluated?

What if the brace-to-gusset connection length was relatively longer while the distance from the end of the brace to the interface remained the same?

Something like this!

Proximity now does not seem to be an issue

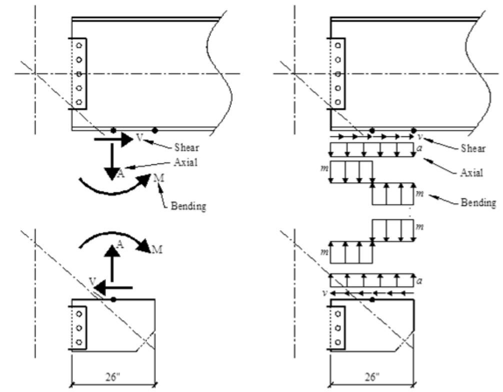
However, frame distortion will still be present under loading, and therefore, a ductility factor would be prudent.



II- 41

Weld Ductility Factor

In both the 1992 Manual and the 2004 Hewitt and Thornton paper, the average stress and peak stress is calculated assuming a uniform distribution of shear and normal forces and a plastic stress distribution of interface moment.



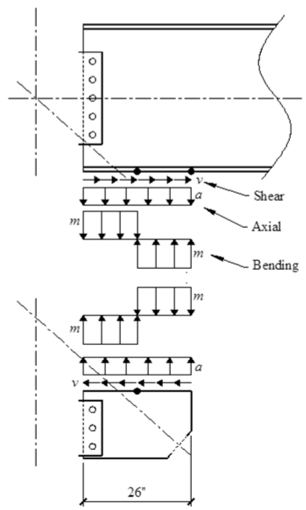
II- 42

Weld Ductility Factor

With this distribution…, the maximum stress is…

…the maximum stress is:

$$f_{\max} = \sqrt{(a+m)^2 + v^2}$$



II- 43

Weld Ductility Factor

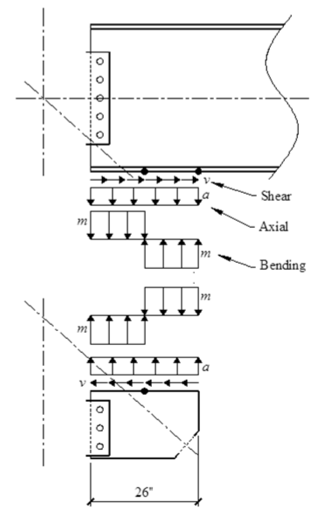
With this distribution…, the maximum stress is…

…the maximum stress is:

$$f_{\max} = \sqrt{(a+m)^2 + v^2}$$

…the minimum stress is:

$$f_{\min} = \sqrt{(a-m)^2 + v^2}$$



II- 44

Weld Ductility Factor

With this distribution…, the maximum stress is…

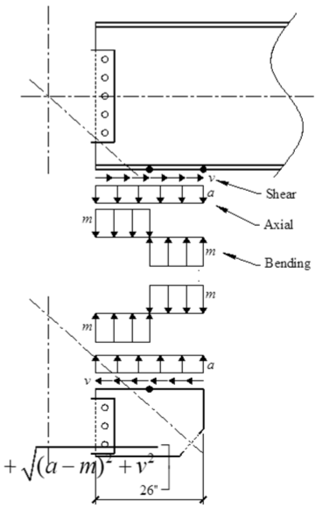
…the maximum stress is:

$$f_{\max} = \sqrt{(a+m)^2 + v^2}$$

…the minimum stress is:

$$f_{\min} = \sqrt{(a-m)^2 + v^2}$$

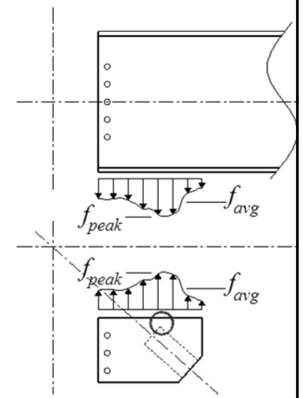
The average stress is the average of f_{max} and f_{min}.



$$f_{avg} = \frac{1}{2}\left(f_{\max} + f_{\min}\right) = \frac{1}{2}\left[\sqrt{(a+m)^2 + v^2} + \sqrt{(a-m)^2 + v^2}\right]$$

Weld Ductility Factor

Fundamentally, if f_{peak} (f_{max}) is larger than 1.25 f_{avg}, a ductility factor is not required.



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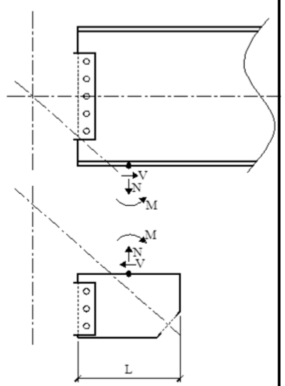
It is important to recognize that we typically design interface welds loaded as shown for the peak (maximum) forces/stress.



$$R = \sqrt{N_{eq}^2 + V^2}$$

Where N_{eq} is equal to…

$$N_{eq} = \frac{2M}{L} + \frac{A}{2}$$

Weld Ductility Factor

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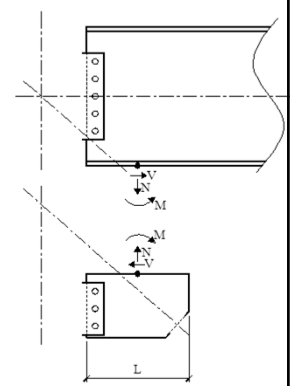


$$R = \sqrt{N_{eq}^2 + V^2}$$

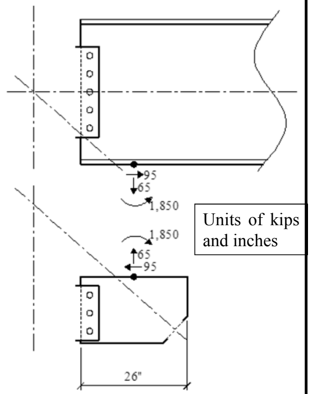
Where N_{eq} is equal to…

$$N_{eq} = \frac{2M}{L} + \frac{A}{2}$$

Although only one-half of the interface is loaded at this level, the entire interface weld is designed for this level of force.

Weld Ductility Factor

Suppose a welded interface is loaded as shown.



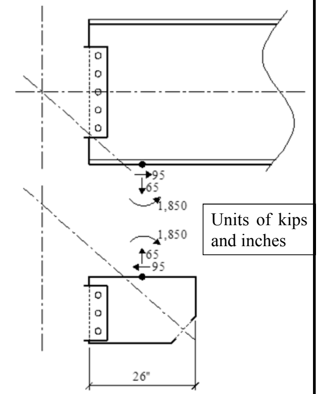
Weld Ductility Factor

Suppose a welded interface is loaded as shown.

The peak force is calculated as:

$$R_{peak} = \sqrt{\left(\frac{(2)(1{,}850)}{26} + \frac{65}{2}\right)^2 + \left(\frac{95}{2}\right)^2}$$

$$R_{peak} = 181 \text{ kips}$$



Weld Ductility Factor

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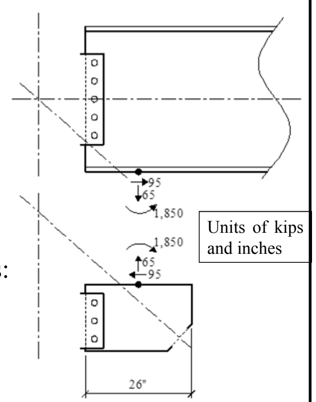
$$R_{peak} = \sqrt{\left(\frac{(2)(1{,}850)}{26} + \frac{65}{2}\right)^2 + \left(\frac{95}{2}\right)^2}$$

$$R_{peak} = 181 \text{ kips}$$

The minimum resultant force is calculated as:

$$R_{\min} = \sqrt{\left(\frac{(2)(1{,}850)}{26} - \frac{65}{2}\right)^2 + \left(\frac{95}{2}\right)^2}$$

$$R_{\min} = 120 \text{ kips}$$



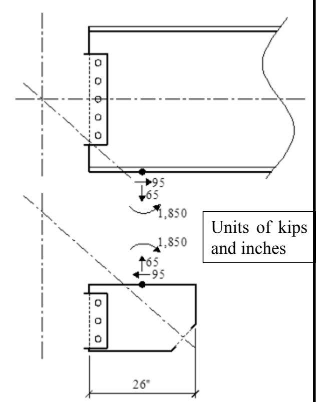
Weld Ductility Factor

Suppose a welded interface is loaded as shown.

The average resultant force is calculated as:

$$R_{avg} = \frac{181 + 120}{2}$$

$$R_{avg} = 151 \text{ kips}$$



Weld Ductility Factor

Suppose a welded interface is loaded as shown.

The average resultant force is calculated as:

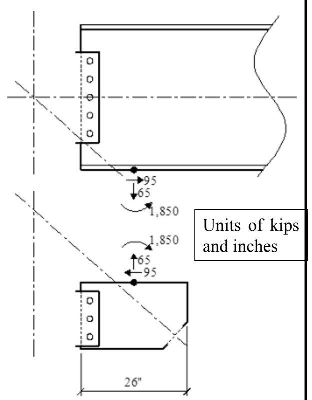
$$R_{avg} = \frac{181+120}{2}$$

$$R_{avg} = 151 \text{ kips}$$

1.25 times R_{avg} is:

$$1.25R_{avg} = (1.25)(151)$$

$$1.25R_{avg} = 188 \text{ kips} > R_{peak} = 181 \text{ kips}$$



II- 53

Weld Ductility Factor

Suppose a welded interface is loaded as shown.

The average resultant force is calculated as:

$$R_{avg} = \frac{181+120}{2}$$

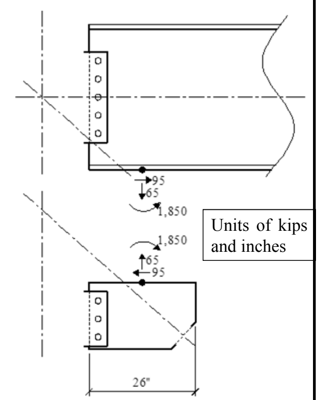
$$R_{avg} = 151 \text{ kips}$$

1.25 times R_{avg} is:

$$1.25R_{avg} = (1.25)(151)$$

$$1.25R_{avg} = 188 \text{ kips} > R_{peak} = 181 \text{ kips}$$

$$\therefore \text{ Design weld for } 1.25R_{avg} = 188 \text{ kips}$$



II- 54

Weld Ductility Factor

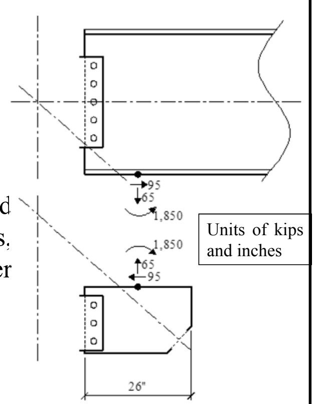
Suppose a welded interface is loaded as shown.

SUMMARY

$R_{peak} = 181$ kips

$1.25R_{avg} = 188$ kips $\Leftarrow$ controls

However, AISC publications (e.g., SDM and DG 29), the authors of example problems, and I prefer a more conservative and simpler approach.



II- 55

Weld Ductility Factor

Suppose a welded interface is loaded as shown.

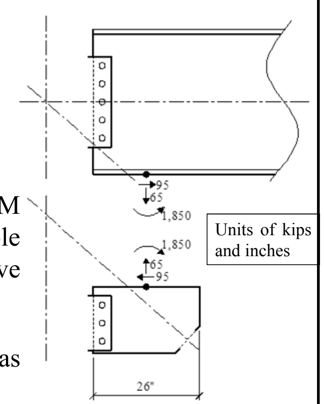
SUMMARY

$R_{peak} = 181$ kips

$1.25R_{avg} = 188$ kips $\Leftarrow$ controls

However, in AISC publications (e.g., SDM and DG 29), the authors of example problems, and I prefer a more conservative and simpler approach.

The calculation typically presented is as follows:



II- 56

Weld Ductility Factor

Suppose a welded interface is loaded as shown.

SUMMARY $R_{peak} = 181$ kips
$1.25R_{avg} = 188$ kips

$$R = \sqrt{N_{eq}^2 + V^2}$$

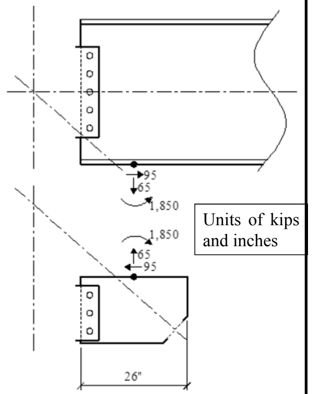
$$R = \sqrt{\left(\frac{(2)(1{,}850)}{26} + \frac{65}{2}\right)^2 + \left(\frac{95}{2}\right)^2}$$

$$R = \sqrt{(175)^2 + (47.5)^2} = 181 \text{ kips}$$

$$\phi = \tan^{-1}\left(\frac{175}{47.5}\right) = 74.8^{\circ}$$

$$\mu = 1.0 + 0.5\sin^{1.5}(74.8) = 1.47$$

$$D_{req} = \frac{(1.25)(181)}{(1.392)(26)(2)(1.47)} = 2.12$$



II- 57

Weld Ductility Factor

Suppose a welded interface is loaded as shown.

SUMMARY $R_{peak} = 181$ kips
$1.25R_{avg} = 188$ kips

$$R = \sqrt{N_{eq}^2 + V^2}$$

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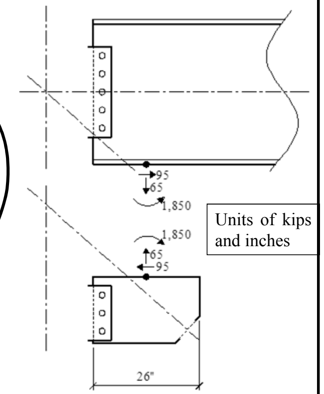
$$\mu = 1.0 + 0.5\sin^{1.5}(74.8) = 1.47$$

$$D_{req} = \frac{(1.25)(181)}{(1.392)(26)(2)(1.47)} = 2.12$$

This is $1.25R_{peak}$
(1.25)(181 kips)=226 kips

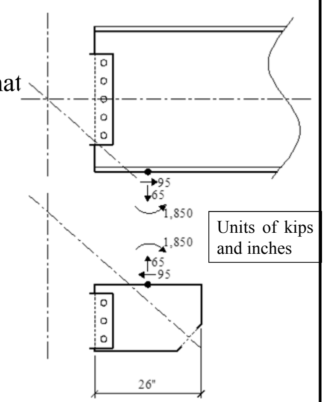


II- 58

Weld Ductility Factor

Suppose a welded interface is loaded as shown.

Most example problem authors know that they are using a ductility factor with R_{peak}.



II- 59

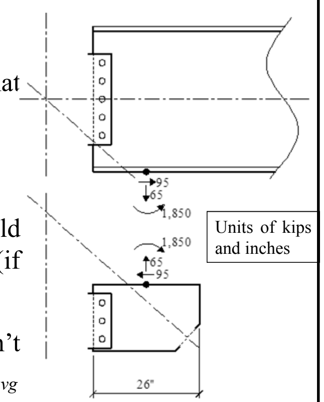
Weld Ductility Factor

Suppose a welded interface is loaded as shown.

Most example problem authors know that they are using a ductility factor with R_{peak}.

It's done for the following reasons:

- ❖ They know a ductility factor should always be used for corner gusset welds (if for any other reason, frame distortion)
- ❖ It's quick, conservative, and they don't have to take the time to calculate f_{min} & f_{avg}



II- 60

Weld Ductility Factor

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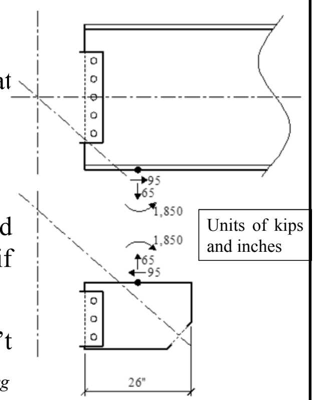


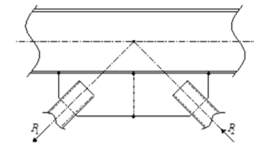
> **➢ But at least recognize that it's not how the ductility factor was intended to be evaluated.**

II- 61

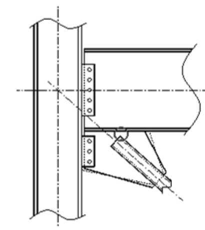
Weld Ductility Factor

Should the ductility factor be used on **chevron** interfaces?

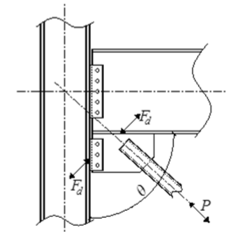
Some use it always; so never on this type of connection



Recall the two triggers for ductility evaluation



Proximity



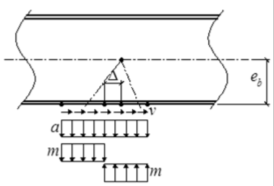
Distortion

II- 62

Weld Ductility Factor

Should the ductility factor be used on **chevron** interfaces?

Proximity



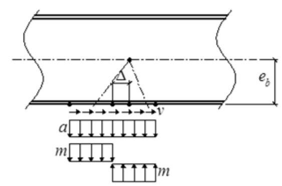
Recall that it is generally assumed that the load from both braces is transferred uniformly along the interface

II- 63

Weld Ductility Factor

Should the ductility factor be used on **chevron** interfaces?

Proximity



Recall that it is generally assumed that the load from both braces is transferred uniformly along the interface

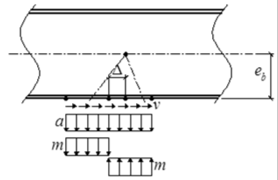
How does that assumption align with the transfer of forces through the gusset using a Whitmore evaluation?

II- 64

Weld Ductility Factor

Should the ductility factor be used on **chevron** interfaces?

Proximity



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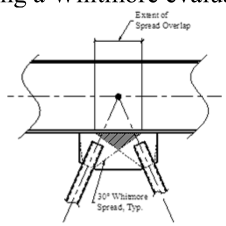


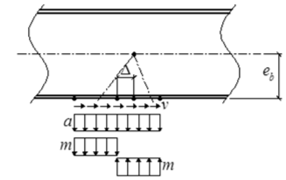
Consider this geometry.

II- 65

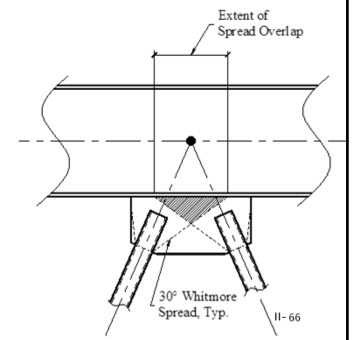
Weld Ductility Factor

Should the ductility factor be used on **chevron** interfaces?

Proximity



With this geometry, neither brace influences the entire interface

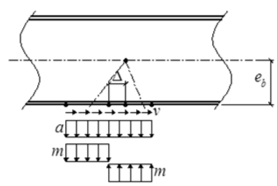


II- 66

Weld Ductility Factor

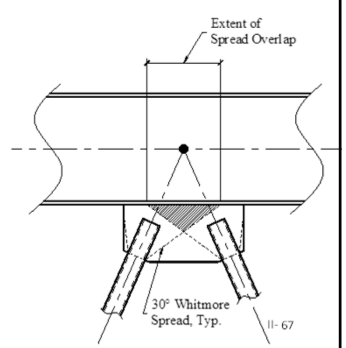
Should the ductility factor be used on **chevron** interfaces?

Proximity



With this geometry, neither brace influences the entire interface

It might be prudent to evaluate the need for a ductility factor for this weld!

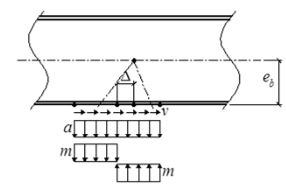


II- 67

Weld Ductility Factor

Should the ductility factor be used on **chevron** interfaces?

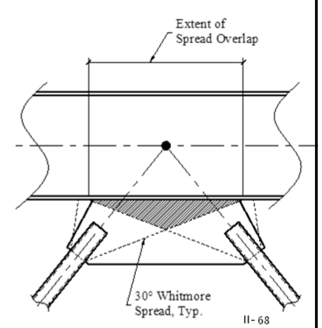
Proximity



What about this geometry?

With this geometry, both braces influence the entire interface

An evaluation of the need for a ductility factor may not be required.



II- 68

Weld Ductility Factor

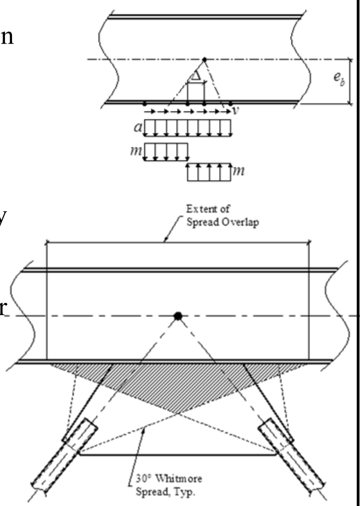
Should the ductility factor be used on **chevron** interfaces?

Proximity

What about this geometry?

The interface even more engaged by both braces

Based on proximity, a ductility factor evaluation is not required



II- 69

Weld Ductility Factor

Should the ductility factor be used on **chevron** interfaces?

Distortion

What about Distortion?

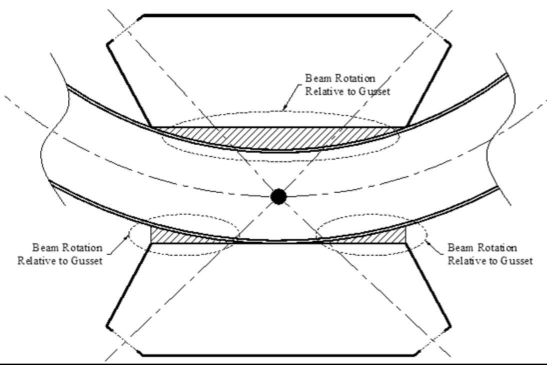
II- 70

Weld Ductility Factor

Should the ductility factor be used on **chevron** interfaces?

Distortion

What about Distortion?



II- 71

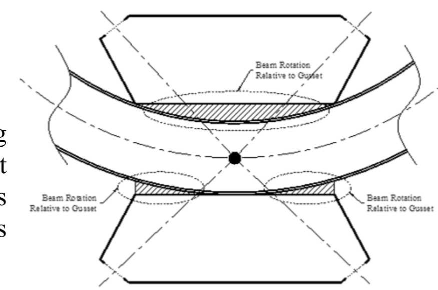
Weld Ductility Factor

Should the ductility factor be used on **chevron** interfaces?

Distortion

What about Distortion?

The beam curvature during bending will always be present under loading creating stress risers at both the gusset edges and middle of the interface



II- 72

Weld Ductility Factor

Should the ductility factor be used on **chevron** interfaces?

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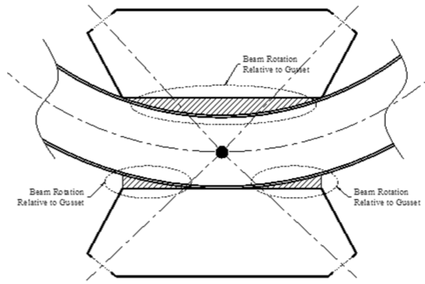
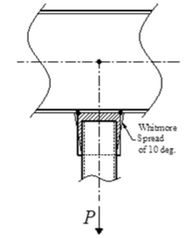


Regardless of proximity, distortion will always be present, and an evaluation of the ductility factor should always be done for a chevron connection

Weld Ductility Factor

Should the ductility factor be used on **hanger** connection interfaces?



Weld Ductility Factor

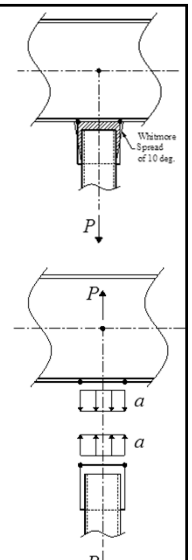
Should the ductility factor be used on **hanger** connection interfaces?

The maximum stress is: $f_{\max} = \sqrt{(a+0)^2 + 0^2}$
$f_{\max} = a$

The minimum stress is: $f_{\min} = \sqrt{(a-0)^2 + 0^2}$
$f_{\min} = a$

So, the average stress is: $f_{avg} = a$

$\therefore 1.25 f_{avg} > f_{peak}$ always



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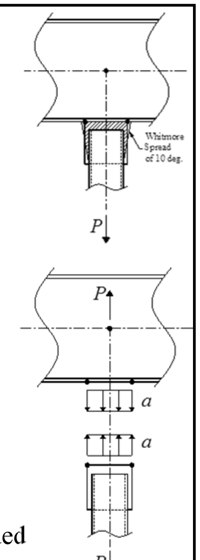
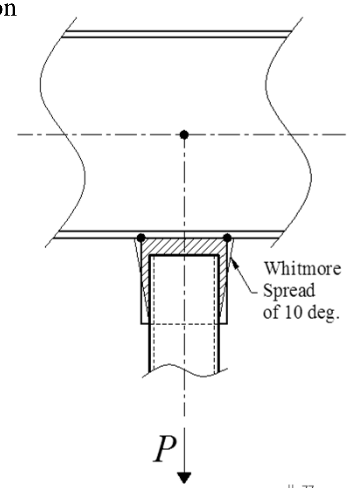
Suggesting that a ductility factor is always needed



Weld Ductility Factor

Should the ductility factor be used on **hanger** connection interfaces?

Proximity is not an issue here; even at very acute Whitmore angles



II- 77

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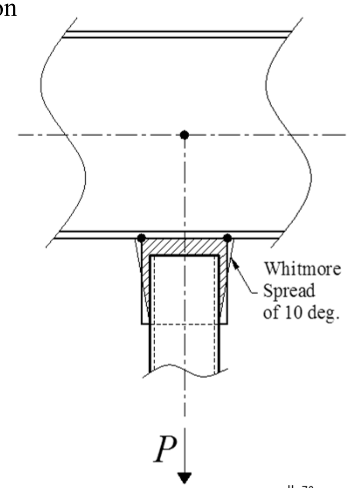
Proximity is not an issue here; even at very acute Whitmore angles

The interface length is usually small enough that distortion is not an issue



II- 78

Weld Ductility Factor

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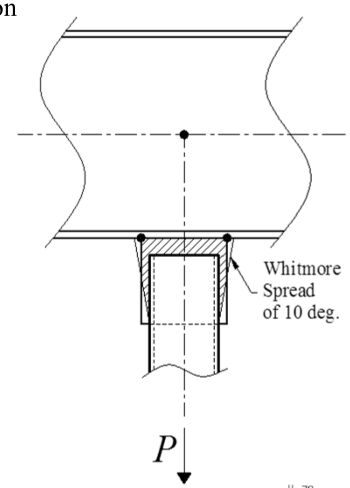
A ductility factor is not needed here!



II- 79

Weld Ductility Factor

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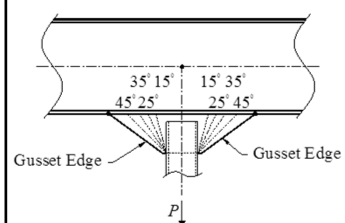
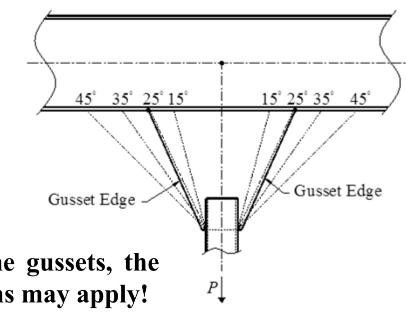


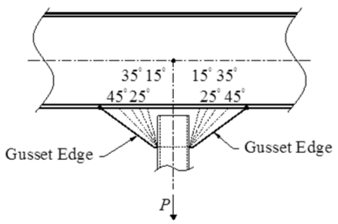


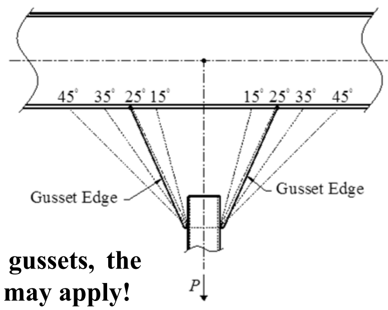
However, if one was to taper the gussets, the same issues discussed for chevrons may apply!

II- 80

Weld Ductility Factor

Should the ductility factor be used on **hanger** connection interfaces?





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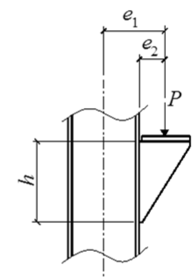
Proximity and distortion (interface length)

II- 81

Weld Ductility Factor

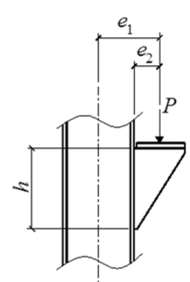
Should the ductility factor be used on **bracket** connection interfaces?



II- 82

Weld Ductility Factor

Should the ductility factor be used on **bracket** connection interfaces?



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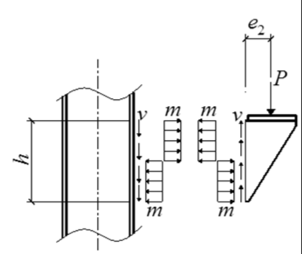
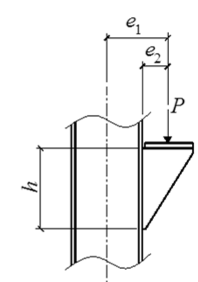


II- 83

Weld Ductility Factor

Should the ductility factor be used on **bracket** connection interfaces?



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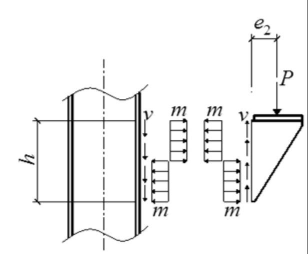


Suggesting that a ductility factor is always needed

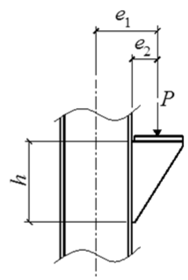
II- 84

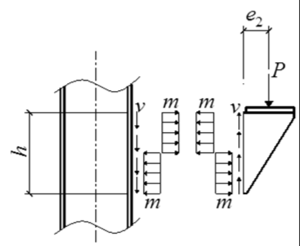
Weld Ductility Factor

Should the ductility factor be used on **bracket** connection interfaces?

However, there are no proximity or distortion issues with the geometry.

A ductility factor simply does not apply!





II- 85

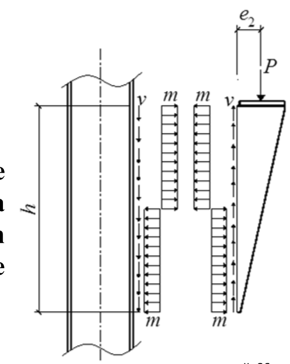
Weld Ductility Factor

Should the ductility factor be used on **bracket** connection interfaces?

However, there are no proximity or distortion issues with the geometry.

A ductility factor simply does not apply!

However, be aware that if *h* is large enough, column bending could produce a curvature that would make distortion something that may need to be considered.



II- 86

Weld Ductility Factor

Clearly, there is not enough time to cover every possible connection and geometry.

II- 87

Weld Ductility Factor

Clearly, there is not enough time to cover every possible connection and geometry.

Keep proximity and distortion in mind

II- 88

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Keep proximity and distortion in mind

❖ A ductility factor for corner gussets should always be evaluated!

II- 89

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Clearly, there is not enough time to cover every possible connection and geometry.

Keep proximity and distortion in mind

❖ A ductility factor for corner gussets should always be evaluated!

❖ Otherwise, use good judgment!

$$\because 1.25 f_{avg} > f_{peak} \ ??$$

II- 90

Weld Ductility Factor

Clearly, there is not enough time to cover every possible connection and geometry.

Keep proximity and distortion in mind

❖ A ductility factor for corner gussets should always be evaluated!

❖ Otherwise, use good judgment!

$$\because 1.25 f_{avg} > f_{peak} \ ??$$

Finally, never use a ductility factor on a weld that has been designed to develop the strength of the connecting element (including a "(5/8)t" weld)!

II- 91

Weld Ductility

Poll Question 2

For what reason(s) was the ductility factor reduced from 1.40 to 1.25?

a) The COM wanted to make welds more economical
b) The original 1.40 factor was before test data was available but, the 1.25 factor was based on test data
c) The 2004 Hewitt and Thornton paper proposed 1.25 based on an evaluation of a 95% confidence level
d) The 2004 Hewitt and Thornton paper proposed 1.25 based on an evaluation of a 90% confidence level

II- 92

ELEMENT CAPACITY WELDS

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Let's look at welds designed to develop the strength of the connecting element.

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ELEMENT CAPACITY WELDS

Let's look at welds designed to develop the strength of the connecting element.

- Generally, welds need only be designed to resist the loads transferred between the parts based on the structural analysis.
- Generally, welds need not be sized based on the available or expected strength of the joined parts.
- When welds are sized based on the strength of the joined parts, this is often referred to as "developing" as in "developing the plate" or "developing the strength of the beam".'

II- 97

ELEMENT CAPACITY WELDS

When would a designer choose to provide a capacity weld?

- The Seismic Provisions require it (e.g., brace gussets),
- The manual recommends it (e.g., $(5/8)t_p$)
- Joining elements of built-up members and judgment suggests that shear flow is not appropriate (e.g., built-up links in EBF systems).

❖ Rational thinking should be used to no unnecessarily specify capacity welds

II- 98

ELEMENT CAPACITY WELDS

- One option is to provide a complete-joint-penetration (CJP) groove weld. As indicated in Table J2.5 of the *Specification* at CJP groove welds "the strength of the joint is controlled by the base metal" not the strength of the weld.

II- 99

ELEMENT CAPACITY WELDS

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- Partial-joint-penetration (PJP) groove weld with or without reinforcing fillet welds can also be used to develop steel elements.

II- 100

ELEMENT CAPACITY WELDS

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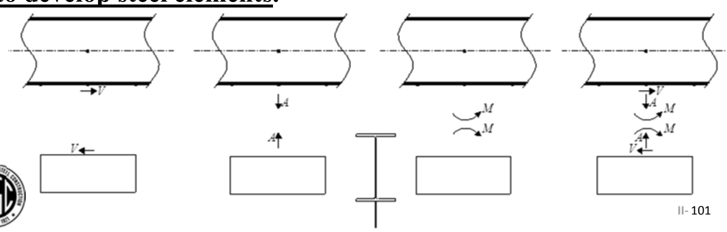
- Partial-joint-penetration (PJP) groove weld with or without reinforcing fillet welds can also be used to develop steel elements.

❖ **This discussion will concentrate on the design of fillet welds used to develop steel elements.**



ELEMENT CAPACITY WELDS

SHEAR

The required weld size to develop a part subjected to shear can be determined by setting the available shear strength of the weld equal to the available shear strength of the part.

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I'll do this using the LRFD method but, the same can be done for ASD as well. Also assuming a two-sided weld.

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I'll do this using the LRFD method but, the same can be done for ASD as well. Also assuming a two-sided weld.

$$\underbrace{\phi 0.6F_yA_g}_{\text{Shear strength of plate}} = \underbrace{1.392DL_p}_{\text{Shear strength of weld}}$$

ELEMENT CAPACITY WELDS

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The required weld size to develop a part subjected to shear can be determined by setting the available shear strength of the weld equal to the available shear strength of the part.

I'll do this using the LRFD method but, the same can be done for ASD as well. Also assuming a two-sided weld.

$$\underbrace{\phi 0.6 F_y A_g}_{\text{Shear strength of plate}} = \underbrace{1.392 D L_p}_{\text{Shear strength of weld}}$$

$$1.0(0.6)F_y t_p L_p = 1.392 D(2)L_p$$

II- 105

ELEMENT CAPACITY WELDS

SHEAR

The required weld size to develop a part subjected to shear can be determined by setting the available shear strength of the weld equal to the available shear strength of the part.

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$$1.0(0.6)F_y t_p L_p = 1.392 D(2)L_p$$

$$\therefore D = 0.216 F_y t_p$$

II- 106

ELEMENT CAPACITY WELDS

SHEAR

The required weld size to develop a part subjected to shear can be determined by setting the available shear strength of the weld equal to the available shear strength of the part.

I'll do this using the LRFD method but, the same can be done for ASD as well. Also assuming a two-sided weld.

$$\underbrace{\phi 0.6 F_y A_g}_{\text{Shear strength of plate}} = \underbrace{1.392 D L_p}_{\text{Shear strength of weld}}$$

$$1.0(0.6)F_y t_p L_p = 1.392 D(2)L_p$$

$$\therefore D = 0.216 F_y t_p$$

Note that if a one-sided weld was used (for some reason), D_{req} would be $0.431F_y t_p$.

II- 107

ELEMENT CAPACITY WELDS

Tension

The required weld size to develop a part subjected to tension can be determined by setting the available tensile strength of the weld equal to the available tensile strength of the part.

I'll do this using the LRFD method but, the same can be done for ASD as well. Also assuming a two-sided weld.

See Section J2.4(b) of the *Specification* for "directional strength increase."

$$\boxed{(1.0 + 0.5\sin^{1.5}\theta)}$$

$$\underbrace{\phi F_y A_g}_{\text{Tensile strength of plate}} = \underbrace{(1.5)1.392 D L_p}_{\text{Tensile strength of weld}}$$

$$0.9 F_y t_p L_p = (1.5)1.392 D(2)L_p$$

$$\therefore D = 0.216 F_y t_p$$

Note that if a one-sided weld was used (for some reason), D_{req} would be $0.431F_y t_p$.

II- 108

ELEMENT CAPACITY WELDS

Compression

- The available strength of welds relative to compression load applied transverse to the longitudinal axis of the weld is generally assumed to be equal to that relative to tension load applied transverse to the longitudinal axis of the weld.

II- 109

ELEMENT CAPACITY WELDS

Compression

- The available strength of welds relative to compression load applied transverse to the longitudinal axis of the weld is generally assumed to be equal to that relative to tension load applied transverse to the longitudinal axis of the weld.
- There has been little testing of such conditions. There are reasons to believe that the strength of fillet welds subjected to compression will be greater than that for welds subjected to tension.

II- 110

ELEMENT CAPACITY WELDS

Compression

- The available strength of welds relative to compression load applied transverse to the longitudinal axis of the weld is generally assumed to be equal to that relative to tension load applied transverse to the longitudinal axis of the weld.
- There has been little testing of such conditions. There are reasons to believe that the strength of fillet welds subjected to compression will be greater than that for welds subjected to tension.
- Whereas applied tension will tend to open the root of the fillet, applied compression will tend to close the root of the weld.

II- 111

ELEMENT CAPACITY WELDS

Compression

- The available strength of welds relative to compression load applied transverse to the longitudinal axis of the weld is generally assumed to be equal to that relative to tension load applied transverse to the longitudinal axis of the weld.
- There has been little testing of such conditions. There are reasons to believe that the strength of fillet welds subjected to compression will be greater than that for welds subjected to tension.
- Whereas applied tension will tend to open the root of the fillet, applied compression will tend to close the root of the weld.
  - There also may be bearing between the parts over some portion of the joint, which is generally neglected, and <u>should be neglected, unless the parts are fit to bear (mill-to-bear)</u>.

II- 112

ELEMENT CAPACITY WELDS

Compression

Conservatively, and <u>for the sake of avoiding specifying joints to be mill-to-bear</u>, treat the tension and compression cases the same.

$$\therefore D = 0.216 F_y t_p$$

II- 113

ELEMENT CAPACITY WELDS

Compression

Conservatively, and <u>for the sake of avoiding specifying joints to be mill-to-bear</u>, treat the tension and compression cases the same.

$$\therefore D = 0.216 F_y t_p$$

Note that if a one-sided weld was used (for some reason), D_{req} would be $0.431F_y t_p$.

II- 114

ELEMENT CAPACITY WELDS

Bending

- The intended meaning of "developing" the element can be less clear when related to bending.

II- 115

ELEMENT CAPACITY WELDS

Bending

- The intended meaning of "developing" the element can be less clear when related to bending.
- Various criteria can and are commonly used in design: elastic strength (first yield), plastic strength, and plastic strength with continued rotation.

II- 116

ELEMENT CAPACITY WELDS

Bending

- The intended meaning of "developing" the element can be less clear when related to bending.
- Various criteria can and are commonly used in design: elastic strength (first yield), plastic strength, and plastic strength with continued rotation.
- The plastic strength of a rectangle bent about its strong axis will be discussed here. A similar approach can be used if one was to evaluate elastic strength.

II- 117

ELEMENT CAPACITY WELDS

Bending

The required weld size to develop a rectangular part subjected to bending about its strong axis can be determined by setting the available flexural strength of the weld equal to the available flexural strength of the part.

I'll do this using the LRFD method but, the same can be done for ASD as well. Also assuming a two-sided weld.

$$\underbrace{\phi F_y Z}_{\text{Flexural strength of plate}} = \underbrace{(2)(1.5)1.392D\frac{L_p^2}{4}}_{\text{Strength of weld}}$$

$$D = 0.862\left(\frac{F_y Z}{L_p^2}\right) \qquad Z = \frac{t_p L_p^2}{4}$$

$$D = 0.862\left(\frac{F_y}{L_p^2}\right)\left(\frac{t_p L_p^2}{4}\right) = 0.216F_y t_p$$

II- 118

ELEMENT CAPACITY WELDS

Summary

Limit State	*Weld Size (D)
Shear	$0.216F_y t_p$
Tension	$0.216F_y t_p$
**Compression	$0.216F_y t_p$
Bending	$0.216F_y t_p$

*Assuming two-sided fillet weld
**Assuming not mill-to-bear

II- 119

ELEMENT CAPACITY WELDS

Summary

Limit State	*Weld Size (D)
Shear	$0.216F_y t_p$
Tension	$0.216F_y t_p$
**Compression	$0.216F_y t_p$
Bending	$0.216F_y t_p$

*Assuming two-sided fillet weld
**Assuming not mill-to-bear

Combinations of shear, axial, and/or bending

Using similar procedures, it can be shown that any combination of these loads, the weld size required to develop the strength of the plate is:

$$D = 0.216F_y t_p$$

II- 120

ELEMENT CAPACITY WELDS

Developing the Strength of the Connecting Plate

Assuming A36 plate material

$D = 7.78t_p$

$\therefore W = 0.486t_p$

$\because W \cong 0.50t_p$

II- 121

ELEMENT CAPACITY WELDS

Developing the Strength of the Connecting Plate

Assuming A36 plate material

$D = 7.78t_p$

$\therefore W = 0.486t_p$

$\because W \cong 0.50t_p$

Assuming GR 50 plate material

$D = 10.8t_p$

$\therefore W = 0.675t_p$

$\because W \cong 0.75t_p$

II- 122

ELEMENT CAPACITY WELDS

Single Plate Shear Connections

Part 10 of the Manual recommends a (5/8)t_p weld for this type of connection (the same for both A36 or GR 50 material).

II- 123

ELEMENT CAPACITY WELDS

Single Plate Shear Connections

Part 10 of the Manual recommends a (5/8)t_p weld for this type of connection (the same for both A36 or GR 50 material).

❖ Note that there is no provision of the *Specification* requiring that the weld be stronger than the plate.

II- 124

ELEMENT CAPACITY WELDS

Single Plate Shear Connections

Part 10 of the Manual recommends a (5/8)t_p weld for this type of connection (the same for both A36 or GR 50 material).

- ❖ Note that there is no provision of the *Specification* requiring that the weld be stronger than the plate.
- ❖ Instead the ($^5/_8$)t_p recommendation is used as a means of satisfying Sections B3.4a and J1.2 of the *Specification*.

II- 125

ELEMENT CAPACITY WELDS

Single Plate Shear Connections

Part 10 of the Manual recommends a (5/8)t_p weld for this type of connection (the same for both A36 or GR 50 material).

- ❖ Note that there is no provision of the *Specification* requiring that the weld be stronger than the plate.
- ❖ Instead the ($^5/_8$)t_p recommendation is used as a means of satisfying Sections B3.4a and J1.2 of the *Specification*.
- ❖ Section B3.4a requires that, "A simple connection shall have sufficient rotation capacity to accommodate the required rotation determined by the analysis of the structure.

II- 126

ELEMENT CAPACITY WELDS

Single Plate Shear Connections

Part 10 of the Manual recommends a (5/8)t_p weld for this type of connection (the same for both A36 or GR 50 material).

- ❖ Note that there is no provision of the *Specification* requiring that the weld be stronger than the plate.
- ❖ Instead the ($^5/_8$)t_p recommendation is used as a means of satisfying Sections B3.4a and J1.2 of the *Specification*.
- ❖ Section B3.4a requires that, "A simple connection shall have sufficient rotation capacity to accommodate the required rotation determined by the analysis of the structure.
  - ❖ Section J1.2 requires that, "Flexible beam connections shall accommodate end rotations of simple beams. Some inelastic but self-limiting deformation in the connection is permitted to accommodate the end rotation of a simple beam."

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ELEMENT CAPACITY WELDS

Large Inelastic Rotations - Seismic

In some instances, primarily related to seismic design, the weld must not only develop the flexural strength of the joined parts, but must also maintain its strength through large inelastic rotations of one of the parts joined.

II- 128

ELEMENT CAPACITY WELDS

Large Inelastic Rotations - Seismic

In some instances, primarily related to seismic design, the weld must not only develop the flexural strength of the joined parts, but must also maintain its strength through large inelastic rotations of one of the parts joined.

- One such condition involves the welds of gusset plates attaching vertical braces used in special concentrically braced frames (SCBF).

II- 129

ELEMENT CAPACITY WELDS

Large Inelastic Rotations - Seismic

In some instances, primarily related to seismic design, the weld must not only develop the flexural strength of the joined parts, but must also maintain its strength through large inelastic rotations of one of the parts joined.

- One such condition involves the welds of gusset plates attaching vertical braces used in special concentrically braced frames (SCBF).
  - Per the *Provisions* SCBF are "expected to provide significant inelastic deformation capacity primarily through brace buckling and yielding of the brace in tension."

II- 130

ELEMENT CAPACITY WELDS

Large Inelastic Rotations - Seismic

In some instances, primarily related to seismic design, the weld must not only develop the flexural strength of the joined parts, but must also maintain its strength through large inelastic rotations of one of the parts joined.

- One such condition involves the welds of gusset plates attaching vertical braces used in special concentrically braced frames (SCBF).
  - Per the *Provisions* SCBF are "expected to provide significant inelastic deformation capacity primarily through brace buckling and yielding of the brace in tension."
  - When the buckling occurs out-of-plane, large inelastic rotations occur about approximately the longitudinal axis of the weld group, which could lead to premature rupture of the weld.

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ELEMENT CAPACITY WELDS

Large Inelastic Rotations - Seismic

Section F2.6c.4 of the *Provisions* is intended to address this concern and states,...

II- 132

ELEMENT CAPACITY WELDS

Large Inelastic Rotations - Seismic

Section F2.6c.4 of the *Provisions* is intended to address this concern and states,...

…"For out-of-plane brace buckling, welds that attach a gusset plate directly to a beam flange or column flange shall have available shear strength equal to $0.6R_yF_yt_p/\alpha_s$ times the joint length."

R_y is a correction factor on F_y to get from nominal specified to expected yield strength

II- 133

ELEMENT CAPACITY WELDS

Large Inelastic Rotations - Seismic

Section F2.6c.4 of the *Provisions* is intended to address this concern and states,...

…"For out-of-plane brace buckling, welds that attach a gusset plate directly to a beam flange or column flange shall have available shear strength equal to $0.6R_yF_yt_p/\alpha_s$ times the joint length."

R_y is a correction factor on F_y to get from nominal specified to expected yield strength

Even with the inclusion of R_y the required weld size is still $\frac{3}{4}t_p$.

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ELEMENT CAPACITY WELDS

Large Inelastic Rotations - Seismic

Note that the Provisions require element/member capacity weld in other places…

- Generally, however this is achieved with CJP welds!
  - Beam-to-column moment connections (OMF, IMF, SMF)
  - Continuity plate welds to flanges (OMF, IMF, SMF)
  - etc.

II- 135

Element Capacity Welds

Poll Question 3

It was shown that a weld size equal to $0.216F_yt_p$ would develop the strength of the connection plate for any combination of shear, axial, and bending load demands…

…This is true for both one-sided and two-sided fillet welds?

a) True

b) False

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Unmasking Welds…

REFERENCES

You may find the following references useful.

Carter, C. J., Muir, L. and Dowswell, B. (2016), "Establishing and Developing the Weak-Axis Strength of Plates Subjected to Axial Loads," *Engineering Journal,* AISC, Vol. 53, No. 3, pp. 147–158

Dowswell, B. and Muir, L. (2012), "Steelwise: Developing M_p," *Modern Steel Construction*, May

Dowswell, B, (2015), "Plastic Strength of Connecting Elements," Engineering Journal, AISC, Vol. 52, No. 2, pp. 47-66

Fortney, P.J., and Thornton, W.A. (2015). "The Chevron Effect – Not an Isolated Problem," Engineering Journal, *American Institute of Steel Construction*, V. 52, No. 2, pp. 125-164

Fortney, P.J., and Thornton, W.A. (2017). "The Chevron Effect and Analysis of Chevron Beams – A Paradigm Shift," Engineering Journal, *American Institute of Steel Construction*, V. 54, No. 4, pp. 253-296

II- 137

Unmasking Welds…

REFERENCES (cont.)

You may find the following references useful.

Hewitt, C.M., and Thornton, W.A., (2004). "Rationale Behaind and Proper Application of the Ductility Factor for Bracing Connections Subjected to Shear and Transverse Loading," Engineering Journal, AISC, Vol. 41, pp. 3-6

Muir, L.S., and Thornton, W.A., (2014), "Vertical Bracing Connections – Analysis and Design," *Design Guide 29*, American Institute of Steel Construction, Chicago, IL

Muir, L.S. and Hewitt, C. (2009), "Design of Unstiffened Extended Single-Plate Shear Connections," *Engineering Journal*, AISC, Vol. 46, No. 2., pp. 67-80

Richard, R.M. (1986), "Analysis of Large Bracing Connection Designs for Heavy Construction," *The National Engineering Conference*, Nashville, TN, June 12-14, 1986, pp. 31-1 - 31-44

Stillmaker, K., Kinvinde, A.M., Gallasso, C., (2015), "Follow-up Fracture Mechanics Analysis of Column Splice Connections with Partial Joint Penetration Welds Using the Master Curve Approach," Report submitted to the American Institute of Steel Construction, July

II- 138

Unmasking Welds…

REFERENCES (cont.)

You may find the following references useful.

Thornton, W.A. and Muir, L.S. (2009), "Vertical Bracing Connections for Moderate Seismic Demands," *Proceedings of the ASCE/SEI 2009 Structures Congress*

Williams, G.C., (1986), "Steel Connection Designs Based on Inelastic Finite Element Analysis," Dissertation, University of Arizona

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There's always a solution in steel.

Unmasking the Ductility Factor, Shear Rupture, and Element Capacity in Welded Connections

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?QUESTIONS?

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