DEFORMATIONAL COMPATIBILITY IN WELD GROUPS

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ABSTRACT

Both the AWS Welding Code and AISC Specification allow a strength increase for fillet welds based on the direction of loading. This strength increase can often be used to improve the economy of structural steel connections. However, deformational compatibility must also be maintained when designing concentrically loaded fillet weld groups with the elements loaded in different directions or with differing leg sizes. Several examples of common connections pertaining to this topic will be discussed.

INTRODUCTION

Since 1993 the AISC Specification has included a provision that allows for a directional strength increase for fillet welds. The provision in its full form produces acceptable results and is consistent with test results. Historically the procedure was used primarily to calculate the capacity of eccentrically loaded weld groups, though the procedure is equally suited to concentrically loaded weld groups. The directional strength increase and the load-deformation relationship of the weld elements are determined from an approximate equation based on an equation presented by Lesik and Kennedy (Lesik and Kennedy 1990).

In the 2005 AISC Specification the directional strength increase was moved from an appendix into the main body of the Specification. Also added were a number of equations to simplify the application of the strength increase to concentrically loaded weld groups. The equations added are (J2-5) and (J2-9b). The Specification states that (J2-5) is applicable a single line weld or weld group that are aligned linearly and (J2-9b) is applicable to weld groups with elements oriented both longitudinally and tranversely to the load.

This paper will demonstrate that additonal requirements must also be satisfied if either (J2-5) or (J2-9b) is to be applied. Those requirements are that all weld elements must be of the same strength and size. This paper will also demonstrate the proper application of J2.4(b) to concentrically loaded weld groups consisting of fillet weld elements of varying leg sizes.

AN EXAMPLE

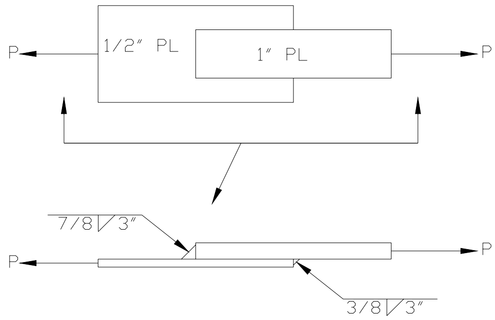


Figure 1

Assume two plates connected as shown in Figure 1 subjected to an axial load. Any eccentricity due to the line of action of the force is neglected for this example. With the maximum size transverse welds applied, the capacity of the welds is to be calculated. Based on section J2.4(a) of the AISC Specification the strength of each weld can be calculated as:

$R_n = A_w 0.60 F_{EXX} \left(1.0 + 0.50 \sin^{1.5}\theta\right)$ where,
F_{EXX} = the electrode classification number, ksi
A_w = the effective area of the weld, in.
θ = the angle of loading measured from the weld longitudinal axis, degrees

For convenience the term $\left(1.0 + 0.50 \sin^{1.5}\theta\right)$ will be called η. Since the welds are both loaded transversely, η, in accordance with the Manual, will be 1.5 for both welds. The capacity, assuming 70 ksi weld metal, is then calculated as:

$$R_n = (0.875 + 0.375)(\cos 45)0.60(70\ ksi)(1.5)(3") = 167\ kips$$

However, one crucial parameter has been missed. If we assume the plates to be infinitely rigid, then the deformation of the smaller weld and the larger weld must be equal. However, the 3/8" weld cannot accomodate the same degree of deformation as the 7/8" weld. This can be seen in Figure 2. AISC gives the deformation of a weld at fracture as:

$$\Delta_u = 1.087(\theta + 6)^{-0.65} w \le 0.17w$$

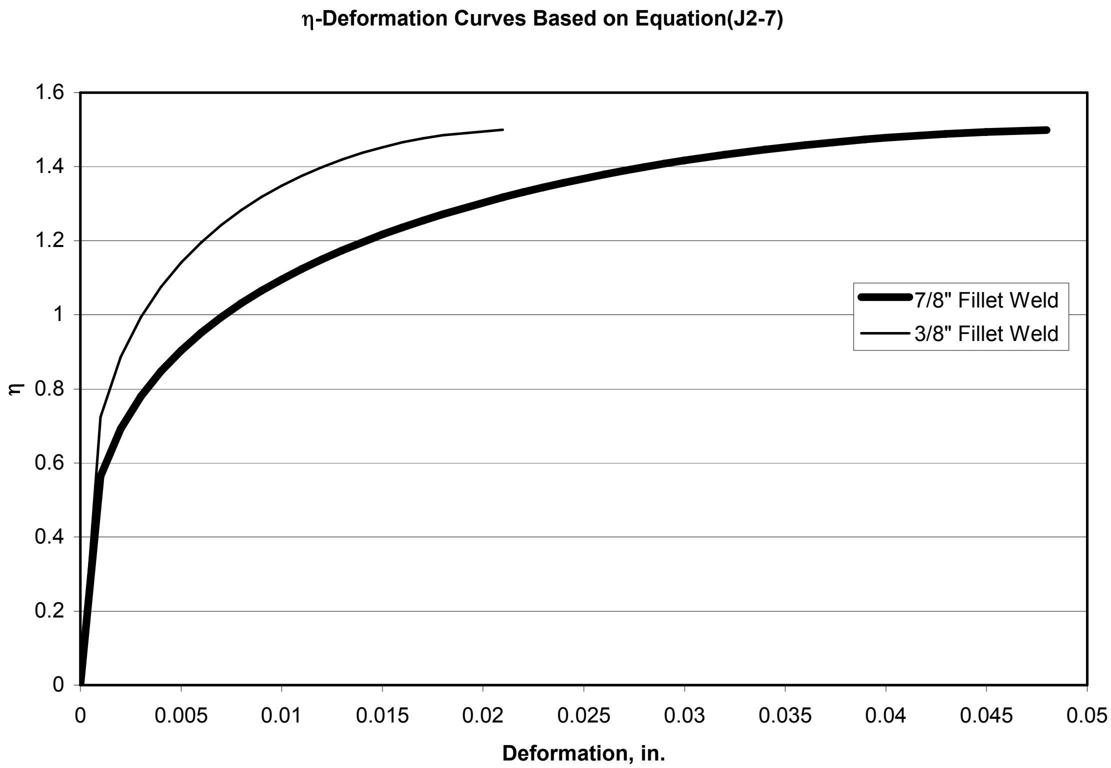


Figure 2

This can be simplified to 0.056w assuming transverse loading. For the 3/8” weld this results in a maximum deformation of 0.021”. For the 7/8” weld this results in a maximum deformation of 0.049”. Therefore the 3/8” weld will fracture prior to the 7/8” weld developing its full strength. Equation (J2-7) from the AISC Specification gives the strength of a weld element as:

$F_{wi} = 0.6F_{EXX}\left(1.0 + 0.50\sin^{1.5}\theta\right)f(p)$ where,

$f(p) = [p(1.9 - 0.9p)]^{0.3}$

p = the ratio of the deformation to the deformation at maximum stress, Δ_m.

$\Delta_m = 0.209(\theta + 2)^{-0.32}w$, for transverse loading $\Delta_m = 0.049w$

For the 7/8” weld $\Delta_m = 0.043''$. With the limiting deformation based on the 3/8” weld of 0.021”, p=0.021/0.043 = 0.488. An effective η can be calculated as:

$$\eta_{eff} = (1.5)[0.488(1.9 - 0.9(0.488))]^{0.3} = 1.36$$

From this the capacity of the weld considering the deformational compatibility can be calculated as:

$$R_n = [0.875(1.36) + 0.375(1.5)](\cos 45)0.60(70\ ksi)(3") = 156\ kips$$

This is somewhat smaller (6.6%) than the capacity of 167 kips predicted by (J2-4).

The problem become somewhat more complicated when combining longitudinally and transversely loaded welds of varying leg sizes, as shown in Figure 3.

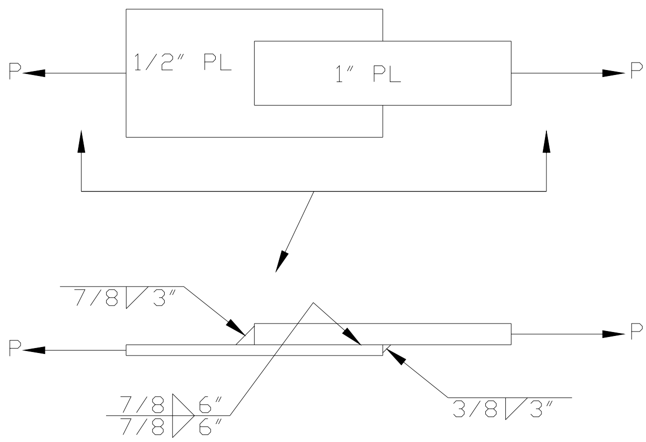


Figure 3

Since the additional welds are both larger than the limiting 3/8" weld and longitudinally loaded, it is obvious that the added welds will not limit the deformation. However, it is also obivous that they will not develop their full strength. AISC Equation (J2-9b) would predict the strength as:

$$R_n = [0.85(0.875)(12") + 1.5(0.875 + 0.375)(3")][(\cos 45)0.60(70\ ksi)] = 432\ kips$$

Equation (J2-9b) assumes η equal to 1.5 for the transversely loaded welds and 0.85 for the longitudinally loaded welds. However, again the deformational compatibility of the varying sizes of welds is neglected.

The effective η for the transversely loaded 7/8" fillet weld will not change due to the addition of the new welds, since these welds do not limit the deformation of the system. However, an effective η for the longitudinally loaded fillet welds must be calculated. This is done in a manner similar to that for 7/8" weld in the original configuration.

$$\Delta_m = 0.209(0+2)^{-0.32}(0.875) = 0.146"$$

With the limiting deformation based on the 3/8" weld of 0.021", p=0.021/0.146 = 0.144. An effective η can be calculated as:

$$\eta_{eff} = (1.0)[0.144(1.9 - 0.9(0.144))]^{0.3} = 0.664$$

From this the capacity of the weld considering the deformational compatibility can be calculated as:

$$R_n = [0.875(1.36)(3") + 0.375(1.5)(3") + 0.875(0.664)(12")](\cos 45)0.60(70\ ksi) = 363\ kips$$

This is about 16% less than the capacity predicted by (J7-9b).

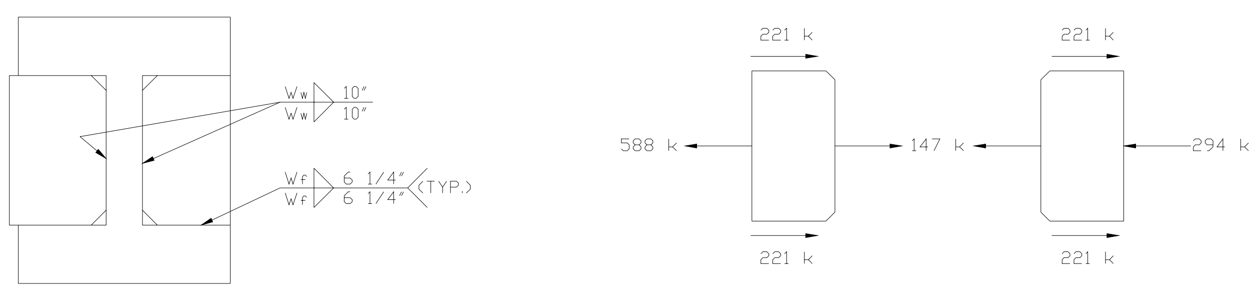
NEGLECTING THE DIRECTIONAL STRENGTH INCREASE

Prior to the inclusion of the directional strength increase in the Specifications, welds were obviously designed neglecting the increase. However, the deformation compatibility of the weld group was also neglected. The AISC Specification still allows this approach in Equation (J2-9a). It is commonly believed that neglecting the directional strength increase must surely be conservative, but this does not appear to be the case for the condition discussed. Neglecting the strength increase and the deformational compatibility, the capacity can be calculated as shown:

$$R_n = [(0.875)(12") + (0.875 + 0.375)(3")][(\cos 45)0.60(70\ ksi)] = 423\ kips$$

This is nearly as great as the capacity predicted by Equation (J2-9a) and exceeds the capacity predicted considering the deformation compatibility by 16.5%. This 16.5% increase is not negligible and is not justified.

A MOMENT CONNECTION EXAMPLE



(a) siffeners welded to column (b) free-body diagrams of stiffeners

Figure 4

The same problems arise in actual design situations, such as weak-axis moment connections. Assume the stiffeners for a moment connection to the weak-axis of a column are welded as shown in Figure 4a. The corresponding free-body diagrams are shown in Figure 4b.

The required weld size at the flanges in accordance with (J2-9a) can be calculated as:

$$W_f = \frac{221\,kips}{(\cos 45)0.60(70\ ksi)(2)(6.25")} = 0.595 \text{ Use 5/8" fillet welds}$$

The required weld size at the web in accordance with (J2-9a) can be calculated as:

$$W_w = \frac{147\ kips}{(\cos 45)0.60(70\ ksi)(2)(10")} = 0.247 \text{ Use 1/4" fillet welds}$$

The required weld size at the flanges in accordance with (J2-9b) can be calculated as:

$$W_f = \frac{221\,kips}{(0.85)(\cos 45)0.60(70\ ksi)(2)(6.25")} = 0.700 \text{ Use 3/4" fillet welds}$$

The required weld size at the web in accordance with (J2-9b) can be calculated as:

$$W_w = \frac{147\ kips}{(1.5)(\cos 45)0.60(70\ ksi)(2)(10")} = 0.165 \text{ Use 3/16" fillet welds}$$

Now we can check the welds obtained using (J2-9a) considering the directional strength increase and the deformational compatibility. The weld to the web being smaller and inherently less ductile will obviously limit the maximum deformation. The maximum deformation of this weld can be calculated as:

$$\Delta_u = 1.087(90+6)^{-0.65}(0.25) \le 0.17(0.25) = 0.014" < 0.043"$$

As has been demonstrated previously, the effective η for the longitudinally loaded welds at the flanges can be calculated as:

$$\Delta_m = 0.209(\theta+2)^{-0.32} w = 0.209(0+2)^{-0.32}(0.625) = 0.105"$$

$$p = \frac{0.014"}{0.105"} = 0.133$$

$$\eta_{eff} = f(p) = [0.133(1.9-0.9(0.133))]^{0.3} = 0.649$$

The strength of the weld group can be calculated as:

$$R_n = [0.625(0.649)(6.25)(4) + 0.25(1.5)(10)(2)](\cos 45)0.60(70\ ksi) = 524\ kips$$

Since this is less than the required load of 588 kips,the welds designed according to (J2-9a) are not sufficient.

Next the welds designed in accordance with (J2-9b) will be investigated. Again the weld to the web will obviously limit the maximum deformation. The maximum deformation of this weld can be calculated as:

$$\Delta_u = 1.087(90+6)^{-0.65}(0.1875) \leq 0.17(0..1875) = 0.011'' < 0.0319''$$

The effective η for the longitudinally loaded welds at the flanges can be calculated as:

$$\Delta_m = 0.209(\theta+2)^{-0.32} w = 0.209(0+2)^{-0.32}(0.75) = 0.126''$$
$$p = \frac{0.011''}{0.126''} = 0.087$$
$$\eta_{eff} = f(p) = [0.087(1.9-0.9(0.087))]^{0.3} = 0.575$$

The strength of the weld group can be calculated as:

$$R_n = [0.75(0.575)(6.25)(4) + 0.1875(1.5)(10)(2)](\cos 45)0.60(70\ ksi) = 487\ kips$$

Again the capacity is less than the required load, so (J2-9b) is not suitable either.

In order to carry the required load some parameter of the weld group must change. Given that the weld to the web web is very small and is limiting the overall deformation of the group, increasing its size is the obvious choice. If the size is increased from 3/16" to 5/16", the largest single pass weld, then the limiting deformation becomes:

$$\Delta_u = 1.087(90+6)^{-0.65}(0.3125) \leq 0.17(0..3125) = 0.017'' < 0.053''$$

The effective η for the longitudinally loaded welds at the flanges will also increase to:

$$\Delta_m = 0.209(\theta+2)^{-0.32} w = 0.209(0+2)^{-0.32}(0.75) = 0.126''$$
$$p = \frac{0.017''}{0.126''} = 0.135$$
$$\eta_{eff} = f(p) = [0.135(1.9-0.9(0.135))]^{0.3} = 0.652$$

The strength of the weld group increases to:

$$R_n = [0.75(0.652)(6.25)(4) + 0.3125(1.5)(10)(2)](\cos 45)0.60(70\ ksi) = 641\ kips$$

Since the original weld size was small, There is little difference economically between the original configuration using 3/16" welds and the new configuration using 5/16" welds. However, the change in welds size represents a 66% increase. Using a 1/4" does not quite meet the capacity. If the optimal change was not quite so obvious, the relative benfits of all possible solutions must be weighed.

For instance, increasing the size of the weld to the flanges would be another option. If the 3/4" longitudinally loaded welds at the flanges were increased to 1-1/8”, the effective η can be calculated as:

$$\Delta_m = 0.209(\theta + 2)^{-0.32} w = 0.209(0 + 2)^{-0.32}(1.125) = 0.188"$$
$$p = \frac{0.011"}{0.188"} = 0.058$$
$$\eta_{eff} = f(p) = [0.058(1.9 - 0.9(0.058))]^{0.3} = 0.513$$

The strength of the weld group can be calculated as:

$$R_n = [1.125(0.513)(6.25)(4) + 0.1875(1.5)(10)(2)](\cos 45)0.60(70\ ksi) = 595\ kips$$

The weld is now sufficient to carry the required load of 588 kips. However, this option is obviously not the most economical choice.

CONCLUSIONS

Load-deformation compatibility must be considered in the design of weld groups. Both AISC and AWS adopt an instantaneous center of rotation procedure, which can be used to maintain compatibility. In the AISC Specification this procedure is outlined in J2.4 (b), and in AWS D1.1 it is outlined in 2.5.4.3. However, both AISC and AWS also allow the designer to neglect load-deformation compatibility if the increased directional strength is also neglected. This was done to allow the use of the traditional method of adding weld strengths with no increase in directional strength and no consideration of deformation compatibility. It also avoids a significant decrease in the predicted capacity for welds that are loaded longitudinally with small returns loaded transversely.

The design examples in this paper have been designed in accordance with the procedures shown in the AISC Specifications and AWS Code. The author was unable to locate results of physical tests of fillet weld groups consisting of weld elements of varying leg size. In the absence of confirming tests, it would seem prudent to design fillet welds groups accounting for load-deformation compatibility, as it is included in the AISC Specification and AWS Code.

REFERENCES

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