CHAPTER 7 COLUMN WEB DOUBLER PLATE WELDING REQUIREMENTS

7.1 INTRODUCTION

The literature is replete with experimental testing, finite element modeling, and analytical modeling of panel zones under cycled inelastic loading. As a result, the importance of modeling panel-zone deformations in high-seismic structural design and assessing whether such yielding is benign or detrimental is well understood. Surprisingly though, very little available research considers the welding requirements when doubler plates are used to stiffen the panel zone. The cost associated with welding doubler plates is significant, and research aimed at economically optimizing a safe welding procedure is warranted.

The purpose of this chapter is to define what role the horizontal welding plays in the load path for concentrated forces across the column web. Using the finite element method, it is shown that when the doubler is fitted between continuity plates, a minimum-size fillet or partial-joint-penetration groove weld can be safely used for all horizontal welding (i.e., doubler to stiffener weld), unless a larger-size weld is required to transmit the doubler portion of the unbalanced force from the transverse stiffener to the web doubler plate. It is also shown that welding the top and bottom of the doubler (i.e., doubler to column web weld) is not necessary when the doubler is extended past the location of the concentrated force, unless it is necessary to prevent shear buckling. Although the results presented herein are also applicable to wind and low-seismic applications, special emphasis is placed on the *AISC Seismic Provisions for Structural Steel Buildings* (1997), which applies in high-seismic applications.

7.2 DESIGN REQUIREMENTS AND CURRENT PRACTICE

According to the *LRFD Specification for Structural Steel Buildings* (1993), when the strength of a column web is less than required for the limit state of panel zone shear, doubler plates can be designed to carry the excess force. Due to the fact that a significant amount of welding is required, the selection of a larger column is frequently a more economical solution. However, in some cases this cannot be done and web doubler plates are necessary.

When doubler plates are used jointly with column stiffeners, there are two possible configurations. The doubler plate can be sized to fit between the top and bottom stiffeners or allowed to extend some distance beyond both stiffeners. For directly welded flange or flange-plated moment connections, this distance is usually 2.5 times the "k-distance" of the column. For extended end-plate moment connections, this distance is usually 3 times the "k-distance" of the column plus the end-plate thickness. For all cases, welding philosophies and fabrication techniques vary among designers and fabricators.

For the doubler plate-to-column flange weld (i.e., vertical weld), either a complete-joint-penetration groove welded detail or a fillet-welded detail is used. In the former case, the web doubler is stopped short of the column fillet area and the resulting gap is filled with weld metal. In the latter case, the web doubler plate is beveled to clear the column fillet and facilitate the fillet weld. The type and size of doubler plate-to-column stiffener weld or column web weld (i.e., horizontal weld) vary. Most details include some form of groove or fillet welding to account for shear forces, and in some cases, to resist local buckling of the web doubler plate. The magnitude of the shear force transmitted to the horizontal weld, however, remains unclear.

7.3 PROCEDURE

It can be argued that since the full resistance of the column web panel zone is required (otherwise a doubler plate would not be provided), no load transfer occurs between the doubler plate and the column web, and thus horizontal welds are not required except to prevent doubler plate buckling. To demonstrate this hypothesis, finite element studies were conducted for a particular girder-to-column joint. A W14x311 column section was chosen because of its relatively weak panel zone. The connecting girders were taken as nominal W36 sections and four doubler configurations were studied:

(IA) doubler plate between stiffeners with horizontal welds,

(IB) doubler plate between stiffeners without horizontal welds,

(IIA) doubler plate extended 2.5k beyond stiffeners with horizontal welds,

(IIB) doubler plate extended 2.5k beyond stiffeners without horizontal welds.

Doubler plate configurations I and II are shown in Fig. 7-1. The loading shown was used for all four cases. The distance between stiffeners is 35.5 in. to model the nominal center-to-center distance between the flanges of a W36 section. The column is pinned at the ends and is 180 in. long. The doubler plate and continuity plates shown have a thickness of 0.705 in. and 0.50 in., respectively. The unusual doubler thickness was chosen for convenience as one-half the W14x311 column web thickness. The continuity plate thickness was chosen arbitrarily to be equal to the assumed beam flange thickness used to calculate the distance between stiffeners. The horizontal deflection of Node X is used to show the effect of the horizontal weld configuration on the lateral drift of the column.

7.4 PANEL ZONE INVESTIGATION USING THE LRFD METHOD

For the W14x311 column section with concentrated beam flange forces shown in Fig. 7-1, the nominal panel zone shear strength (assuming the factored column axial load, P_u, is less than or equal to 40% of the axial yield strength, or $0.4P_y$) is obtained from the *LRFD Specification for Structural Steel Buildings* (1993) as

$$V_n = 0.6F_{yw}d_c t_p \qquad \text{(AISC Eq. K1-9)}$$

where

F_{yw} = nominal panel zone yield stress,

t_p = total thickness of panel zone including doubler plate, and

d_c = column depth.

Here it is intended that the effect of panel zone deformations on frame stability will not be considered in the analysis.

Alternatively, to account for the additional strength contribution of thick column flanges, the *AISC Seismic Provisions for Structural Steel Buildings* (1997) provides the following equation for determining the nominal panel zone shear strength (assuming $P_u \leq 0.75P_y$):

$$V_n = 0.6F_{yw}d_c t_p\left[1+\frac{3b_{cf}t_{cf}^2}{d_b d_c t_p}\right] \qquad \text{(AISC Eq. 9-1)}$$

where

b_{cf} = column flange width,

t_{cf} = column flange thickness, and

d_b = beam depth.

This equation is identical to AISC Eq. K1-11 which is to be used when frame stability and plastic panel zone deformations are to be considered in the analysis.

For the column section alone (no doubler plate), and P_u = 0 kips, the following nominal shear strengths are obtained:

$$V_n = 0.6F_{yw}A_w = 0.6(50)(17.12*1.41) = 724 \text{ kips}$$

$$V_n = 0.6F_{yw}d_c t_p\left[1+\frac{3b_{cf}t_{cf}^2}{d_b d_c t_p}\right] = 0.6(50)(17.12*1.41)\left[1+\frac{3(16.23)(2.26)^2}{36(17.12)(1.41)}\right] = 931 \text{ kips}$$

from AISC Eq. K1-9 and AISC Eq. 9-1, respectively.

The first value of V_n is less than the factored loading of V_u = 600 + 300 = 900 kips. Hence, a doubler plate is required to prevent the panel zone from yielding under the given loading. Therefore, to insure elastic behavior of the panel zone, the appropriate doubler plate thickness must be provided. Rearranging AISC Eq. K1-9, and letting t_p include the thickness of the doubler plate (Fy = 50 ksi), the required doubler plate thickness is determined to be 0.35 in. or a 3/8 in. plate. The 0.705 in. doubler plate used in this study satisfies this requirement and results in a nominal shear strength of V_n = 1.5* 724 = 1086 kips.

First, cases IA and IIA include the doubler plate with horizontal welding. Next, cases IB and IIB include the doubler plate attached without any form of horizontal welding. This design may cause significant stresses to develop away from the center of the panel zone and decrease the design strength. Finally, cases IA-PL, IB-PL, IIA-PL, and IIB-PL consider the doubler plate with a yield stress of 36 ksi, in which case the design strength is decreased from 1086 kips to some unknown value greater than 724 kips.

Since 1086 kips exceeds the factored loading of V_u = 900 kips, it appears that column section may be considered adequate. However, this value of V_n allows for some yielding to take place in the panel zone (when the doubler has a yield stress of 36 ksi), and these plastic deformations must be considered in the analysis of the entire structure.

7.5 FINITE ELEMENT ANALYSIS

The finite element model used to analyze the panel zone is two-dimensional as shown in Fig. 7-2 (full length of column not shown). For the purposes of this study, the column and continuity plate elements have a yield stress of 50 ksi. Doubler plates with yield stresses of both 36 and 50 ksi are considered. The loading shown in Fig. 7-1 is chosen to cause stresses in the doubler plate to exceed 36 ksi. This will allow the doubler (when the yield stress is 36 ksi) to respond inelastically while the column web responds elastically to the given loading. Approximately 12,000 solid eight-node elements that allow plasticity effects are used to model the assembly. The meshing detail shown in Fig. 7-2 was found to be adequate by a convergence study. To avoid placing the loads on one node in the vicinity of the stiffeners, the stiffener and column web meshing in these areas are refined to allow the load to be applied as three loads at each location. For example, the 450 kip load shown in Fig. 7-1 is replaced by three 150 kip loads in the same region. Horizontal and vertical welds are nominal ½ in. fillet welds and are assumed to respond elasticly. Although fillet welds in shear are extremely ductile and may not necessarily remain elastic, any inelasticity in the welds only bolsters the conclusions made here.

For each of the four cases discussed above, the loading configuration of Fig. 7-1 is applied with the doubler plate assumed to remain elastic (i.e., yield stress of 50 ksi). Next, the doubler plate is allowed to yield (i.e., yield stress of 36 ksi) for all four cases. These are tagged by the extension PL. For example, case IA with an inelastic doubler plate is denoted case IA-PL.

7.6 RESULTS

First, the column alone was subjected to the loading of Fig. 7-1 to determine the elastic behavior of the panel zone. This is shown in Fig. 7-3 which plots the von Mises stress distribution in the vicinity of the top stiffener. The deflections are exaggerated to enhance the detail of the figure. Stresses between 52 and 65 ksi are present in the center of the panel zone (based on elastic analysis) and the need for a doubler plate is apparent. It is interesting that stresses which are only about one-third of these values exist in the area of the panel zone close to the top stiffener. Also, very low stresses exist in the column web just above the stiffener, indicating a the lack of shear transfer in this region.

7.6.1 CASE IA: DOUBLER PLATE BETWEEN STIFFENERS WITH HORIZONTAL WELDS

Assuming that the doubler plate remains elastic, case IA is considered. After the loads have been applied to the structure, the resulting von Mises stress distribution across the column is shown in Fig. 7-4(a). The stresses shown in the panel zone are actually in the column web as the doubler is not shown here. When compared to Fig. 7-3, stresses in the column panel zone are significantly reduced. Stresses between 40 and 50 ksi result, indicating that the column remains elastic as assumed. The von Mises stress distribution in the doubler plate is shown in Fig. 7-4(b). The stresses are almost identical to those in the column web. With horizontal welds included in this case, stresses between 20 and 30 ksi are found near the top stiffener. Since the welds are designed to resist shear, the horizontal and vertical shear stress distributions across the doubler plate are shown in Figs. 7-4(c) and 7-4(d), respectively. Most of the horizontal shear stress is concentrated in the corners of the panel zone and seems to affect both welds. The significant vertical shear stress is concentrated primarily along the vertical weld.

7.6.2 CASE IB: DOUBLER PLATE BETWEEN STIFFENERS WITHOUT HORIZONTAL WELDS

With the doubler plate still assumed elastic, case IB is now considered and compared to case IA. The only difference between these two cases is that the horizontal weld is not included here. Comparing Fig. 7-5(a) with Fig. 7-4(a), it is clear that the absence of the horizontal welds somewhat alters the load path through the panel zone of the column. Slightly larger stresses can be seen in the center of the panel zone, but the main difference is near the top stiffener. The stresses here are about 10 percent larger than in case IA. Figure 7-5(b) explains the reasons for these changes. First of all, the analysis indicates the onset of localized yielding as shown in the figure. This is due to a high stress gradient in the proximity of the vertical weld termination. Also, very low stresses, shown by the white region, exist at the top of the doubler plate. Obviously, no significant shear stress exists there. Although a complete study of the effect of weld size on the high stress gradient is not provided, the results of this case study suggest that minimum size fillet welds would alleviate the high stress gradient problem and can be used conservatively in all cases, unless a larger-size weld is required to transmit the doubler portion of the unbalanced force from the transverse stiffener to the web doubler plate. If a groove-welded detail is used instead of fillet welds, it would be sufficient to fill the gap with a CJP groove weld or size a partial-joint-penetration groove weld.

Figures 7-5(c) and 7-5(d) show the horizontal and vertical shear stress distributions across the doubler plate, respectively. The absence of a horizontal weld only slightly alters the horizontal shear stress by increasing the values near the corner of the doubler plate. Nevertheless, the vertical shear stresses are changed significantly. Not only are larger stresses observable, but the high stress gradient near the corner of the doubler plate becomes a prime concern and a premature failure could result. Comparing Fig. 7-5(d) to Fig. 7-4(d), it is clear that the horizontal weld allows the vertical shear stress to be more evenly distributed at the corner of the doubler plate. Both problems are easily overcome by following the recommendations of the preceding paragraph.

7.6.3 CASE IIA: DOUBLER PLATE EXTENDED 2.5k BEYOND STIFFENERS WITH HORIZONTAL WELDS

For the extended elastic doubler plate problem of case IIA, conclusions similar to case IA are evident. With the doubler plate not shown, Fig. 7-6(a) shows the von Mises stress distribution across the column web. The stress distribution is almost identical to the distribution of Fig. 7-4(a) for case IA. This indicates that the extended portion of the plate has no significant effect on reducing stresses in or around the panel zone of the column. Figure 7-6(b) shows the von Mises stress distribution across the extended doubler plate. It is apparent that only a small amount of load is taken by the part of the doubler plate above the stiffener. The horizontal shear stress distribution of Fig. 7-6(c) indicates that no significant horizontal shear stresses are developed in the doubler plate. The vertical shear stress distribution is shown in Fig. 7-6(d). Most of the stress is concentrated along the vertical edge of the doubler plate. It is noted that the highest concentration is above the stiffener and that these values could increase for different k distances.

7.6.4 CASE IIB: DOUBLER PLATE EXTENDED 2.5K BEYOND STIFFENERS WITHOUT HORIZONTAL WELDS

These results show that case IIB (without horizontal welds) alleviates any problems inherent in case IIA. First of all, the stress distribution shown in Fig. 7-7(a) is almost identical to that of case IIA shown in Fig. 7-6(a). Figure 7-7(b) indicates that no significant stresses are developed in the doubler plate above the column stiffener. The increased horizontal shear stresses at the top of the doubler plate of Fig. 7-7(c) are not of concern. The stresses are less than yield and no weld is in this region. Compared to Fig. 7-6(d), Fig. 7-7(d) shows a decrease in vertical shear stress at the weld and no significant stress gradient. This result is very important, as it manifests how the exclusion of horizontal welds can be beneficial to the load path of the structure in some cases.

7.6.5 PLASTIC BEHAVIOR OF DOUBLER PLATE

The doubler plate is now allowed to respond inelastically to the loading for cases IA-PL, IB-PL, IIA-PL, and IIB-PL. Figures 7-8(a) - 7-8(d) show the stress distribution across the doubler plate for each of the cases, respectively. Although in these figures, black indicates stresses between 28.8 and 36 ksi, about 95 percent of this area has yielded. All four figures imply the same conclusion. For each case, yielding begins in the center of the panel zone and extends outward towards the vertical welds, which is exactly the behavior observed in physical testing of panel zones. Based on this load path, it is apparent that the vertical welds must be designed to develop the thickness of the doubler for significant seismic loading. Whether the horizontal weld is in place or not, yielding does not take place in the proximity of the horizontal weld location. In fact, almost all the yielding takes place below the stiffener region.

7.6.6 DEFLECTIONS

Since the lateral drift of any column under loading is a consideration of the design engineer, it is important to determine the role played by the horizontal welds here. Table 7-1 shows the deflection of node X (see Fig. 7-1) for each case. The largest deflection, as expected, is for the column without a doubler plate. The addition of a doubler plate decreases the deflection approximately 13 percent in cases IA through IIB. As expected, slightly larger deflections result for the inelastic doubler plate of cases IA-PL through IIB-PL. Although there is a slight difference between cases involving A and B (with and without horizontal welding), the horizontal weld seems to play no special role here.

7.7 CONCLUSIONS

The results presented in this study lead to several conclusions and design recommendations. For the case of a doubler plate sized to fit between previously fabricated stiffeners, a large horizontal weld can be unnecessary. On the other hand, not including the horizontal weld could result in a detrimentally high stress gradient in the corners of the doubler plate. This can be remedied by using a minimum-size fillet weld in all cases, unless a larger-size weld is required to transmit the doubler portion of the unbalanced force from the transverse stiffener to the web doubler plate. If groove welded detail is used instead of fillet welds, it is sufficient to fill the gap with a CJP groove weld or size a partial-joint-penetration groove weld. The deflection of the stiffener and the deflection of the doubler plate in this region may be slightly different, but this slight relative displacement is allowed by the ductility of the weld.

All of the problems discussed above can be overcome by not using any type of horizontal welding and simply extending the doubler plate beyond the stiffener a distance of 2.5 times the k distance of the column. Indeed, in this case, not using a horizontal weld at the top of the doubler plate actually decreases the vertical shear stresses on the vertical welds. For fabrication purposes, the doubler plate should be welded first with the stiffeners placed on top of, and welded directly to, the doubler plate.

TABLE 7-1. Horizontal deflection of node X for each case considered.

Doubler Plate Configuration	Horizontal Deflection (in.)
No Doubler Plate	0.0838
IA (weld, 50 ksi fit doubler)	0.0728
IB (no weld, 50 ksi fit doubler)	0.0732
IIA (weld, 50 ksi extended doubler)	0.0731
IIB (no weld, 50 ksi extended doubler)	0.0733
IA-PL (weld, 36 ksi fit doubler)	0.0739
IB-PL (no weld, 36 ksi fit doubler)	0.0744
IIA-PL (weld, 36 ksi extended doubler)	0.0741
IIB-PL (no weld, 36 ksi extended doubler)	0.0743