Preliminary Analysis of Doubler Plate Attachment Details for Steel Moment Frames

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SUMMARY:
Beam-column joints in steel moment resisting frames frequently require the addition of doubler plates to increase the shear strength of the column panel zone. The addition of doubler plates often requires a substantial amount of welding, and the details used for welding the doubler plate to the column can affect both cost and structural performance. However, there is little past research available that provides guidance for the design of welds for attachment of doubler plates.

This paper summarizes the results of preliminary analysis of doubler plate attachment details for the steel moment resisting frames. This study involved finite element modeling of a simplified representation of beam-to-column joint subjected to monotonic loading. Analysis cases with different doubler plate attachment details were studied. Issues that were investigated included the effect of welding different edges of the doubler plate to the column, the effect of extending the doubler plate beyond the panel zone region, and the effect of providing two thinner doubler plates of equivalent total thickness on both sides of the column web instead of one thick doubler plate on one side of the column web. In addition, the forces developed in the doubler plate welds were computed from the finite element analysis and compared with current building code requirements for the design of these welds.

Keywords: Doubler plate attachment, panel zone, steel moment resisting frames

1. INTRODUCTION

When a steel moment resisting frame is subjected to lateral load, either due to wind or seismic loads, the region of the column located within the beam-column joint known as *panel zone* is generally subjected to high shear. In some cases, the shear in the panel zone is sufficiently high that the panel zone must be reinforced to increases its strength and/or stiffness and one of the most common approaches to do so is to weld a plate known as *doubler plate* to the column. Welding doubler plates to columns can add significant cost to the beam-column joint in steel moment resisting frames and very little research has been conducted in the past to investigate economical and structurally effective ways of attaching a doubler plate to a column.

Although a considerable amount of past research has studied the load-deformation behaviour of panel zones under both monotonic and cyclic loads, very little past research has studied the attachment of doubler plates to columns in the panel zone regions. Limited studies of doubler plates and attachment details were conducted by Becker (1975), Slutter (1982), Mays (2000), Lee et al (2005a, 2005b), and by Ciutina and Dubina (2008). However, this past research has not resulted in clear conclusions on the most effective methods of welding the doubler plate to the column.

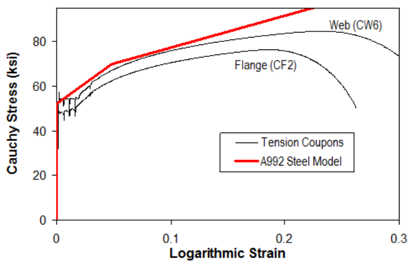
Research using finite element analyses was conducted to provide insights into several key issues related to the attachment of doubler plates to columns. The key questions addressed were: (a) where should doubler plate welds be located: top and bottom only, vertical sides only, or on all four sides? (b) are there benefits of extending the doubler plate above and below the beam? (c) are there benefits of providing two thinner doubler plates of equivalent thickness instead of one thick doubler plate? (d) what are the forces developed in the doubler plate welds? These issues were studied by developing finite element models of the panel zone region in Abaqus/CAE 6.9-EF2. This paper provides highlights of this research study. More complete details are reported in Shirsat (2011).

2. MODELING TECHNIQUES

In Abaqus the models were intended to represent the behaviour of steel elements loaded well into the inelastic range where the behaviour is dominated by shear. To evaluate the capabilities of the Abaqus model, results of selected experiments involving shear yielding of steel members were compared with model predictions. In Abaqus the wide flange members, plates and welds were modelled as 3D deformable solids with homogeneous section properties. An 8-node solid brick element with linear geometric order and reduced integration was used for the wide flange members, plates and welds. This element which has 8 nodes and 6 faces is called as hexahedra and is designated as C3D8R in Abaqus. From the literature review it was seen that the C3D8R element has been used successfully in past research with Abaqus to study the response of structural steel members loaded into the inelastic range.

The material model developed by Okazaki (2004) to represent A992 steel members was adopted for this study, as Okazaki showed good correlation with experiments where members were loaded well into the inelastic range in both flexure and shear. Although Okazaki's research involved cyclic loading experiments on shear yielding links, he developed an Abaqus model intended only for monotonic loading. For this, he chose a tri-linear model with isotropic hardening. The input of the material data in Abaqus is required to be in terms of true stress (Cauchy stress) and true strain (logarithmic strain) *(from Getting Started with Abaqus: Interactive Edition, Section 10.2.2).*

Figure 2.1 (a) shows the tri-linear model developed by Okazaki (2004) for the A992 steel used in his experiments. In this figure, the curve for the A992 steel model is in terms of true stress and logarithmic strain values. The figure also shows the tension coupon test results for coupons from the web and flange which are in the form of engineering stress and engineering strain. As seen in this figure, the A992 steel model represents closely the web of W12x120. In this research the main interest was to study the deformation and stresses in the panel zone i.e. in the column web area. Thus this tri-linear material model developed by Okazaki was used to model all the wide flange shapes in this research study. Figure 2.1 (b) shows the input command lines in Abaqus for this material model. This same material model was also used for the doubler plates. Plate material is not available in A992 steel, as this steel specification is only applicable to wide flange shapes. Plate material used for doubler plates would commonly be specified as A572 Gr. 50. However, the A992 and A572 Gr 50 specifications are quite close, so the A992 material model described above was also used for the doubler plates. The material model for the weld developed by Okazaki (2004) was used for all the analysis cases for groove welds and fillet welds wherever applicable. Figure 2.1 (c) shows the input command lines in Abaqus for weld material model.



(a)

```
*Material, name=Steel_inelastic
*Elastic
29000., 0.3
*Plastic
 52.09,     0.
 69.94, 0.04808
 125.7, 0.43508
```

(b)

```
*Material, name=inelastic_weld
*Elastic
29000., 0.3
*Plastic
 65.15,     0.
 83.65, 0.04804
145.91, 0.43467
```

(c)

Figure 2.1. (a) Tri-linear material model (from Okazaki (2004)); (b) Input command lines in Abaqus for material model; (c) Input command lines in Abaqus for weld material model

Panel zone response in steel moment frames is dominated by large shear forces. Consequently, it was of interest to evaluate how well the Abaqus modeling techniques used for this research can predict the response of steel members subjected to inelastic shear deformation. For this purpose, Abaqus was used to model experiments reported in the literature by Ryu (2005) and by Engelhardt et al (2000). These comparisons showed that the Abaqus model could reproduce the inelastic load-deformation response measured in the experiments with reasonable accuracy. Full details of these comparisons are provided in Shirsat (2011).

3. PARAMETRIC STUDIES ON DOUBLER PLATE ATTACHMENT DETAILS

3.1. Subassembly model in Abaqus

3.1.1. Finite element model in Abaqus
The analysis was performed using the finite element program Abaqus 6.9-EF2. A simplified representation of a beam-column joint was developed for the purposes of this study. In this simplified model, a segment of column was modelled with a length equal to a typical story height. The beams were replaced by "loading plates" located at the level of the beam flanges, as shown in Figure 3.1. The corresponding shear force and bending moment diagrams for the column model are also shown in Figure 3.1. The column, loading plates, end plates, doubler plates and welds were modelled using the 3D solid element C3D8R in Abaqus. The tri-linear material models developed by Okazaki (2004) were used to model structural steel and welds. The loading plates were modelled to behave elastically with a very high modulus of elasticity of 60000 ksi. 2-inch thick end plates were modelled at each end of the column.

The 'structured' mesh feature in Abaqus was used for the column, doubler plate, loading plates and the end plates. The 'sweep' meshing technique with 'medial axis' algorithm was used for meshing the groove welds and fillet welds. An approximate global seed size equal to 0.1-inch was used for groove welds and fillet welds. The column had a finer mesh in the panel zone region. The tie constraint was used between: (i) the column (s) and the end plates (m); (ii) column flanges (m) and the loading plates (s); (iii) doubler plate (m) and the welds (s); (iv) column web (m) and the welds (s); where (m) denotes master surface and (s) denotes slave surface for tie constraint. 'Hard' contact with allowing separation after contact was used between the column web and the doubler plate. In this contact the column web was selected as the 'master' and the doubler plate as the 'slave' allowing 'finite sliding' between the two. Abaqus does not allow a single node to be a part of two constraints thus the corners of the groove weld and the fillet weld were very slightly chamfered.

3.1.2. Loading and boundary conditions
Inflection points of the column were assumed to occur at mid-height of the columns with an assumed story height of 12-feet. In Abaqus, the column was hinged at one end and had a vertical roller at the other end as shown in Figure 3.1 (b). The top and bottom ends of the column were restrained to translate in the lateral direction to avoid torsion in the column. Point loads acting in opposite directions were applied to the column through the loading plates to simulate the beam flange forces. The distance between the point loads is denoted as "d" in Figure 3.1(b), and this represents the distance between mid-depth of the beam flanges. These concentrated loads generate a very high shear force in the panel zone.

As shown in Figure 3.1, if the total force applied at each load plate level is F (with F/2 applied one each side of the column), the horizontal reactions at the top and bottom of the column is R (equal to the shear in the portion of the column outside of the panel zone), then the shear force within the panel zone, V_{pz}, can be computed from equilibrium as: V_{pz} = F- R = F(l-d)/l, where l is the distance between the inflection points. The nominal shear strength (V_n) of the column panel zone, when no doubler plate is used, is given by Eq. (J10-11) of the *2005 AISC Specification for Structural Steel Buildings*. For all of the parametric studies, the load F was applied up to a level that would produce a panel zone shear equal to 1.25 times the nominal panel zone shear strength. This load was increased gradually starting from zero with an increment of $0.0625V_{pz}$ up to $1.25V_n$. The value of $1.25V_n$ was chosen somewhat arbitrarily to represent a relatively large panel zone shear force that might be representative of the panel zone shear developed under seismic loading. Even when doubler plates were present in an analysis case, the value of 1.25 V_n was based on the value of V_n for a column without a doubler plate. Consequently, the maximum panel zone shear applied to the Abaqus model of the column was the same for all analysis cases with the same value of d. Finally, for the given target value of Vpz = 1.25Vn, the force F applied to the column can be computed from: $F = (V_{pz}l)/(l-d)$.

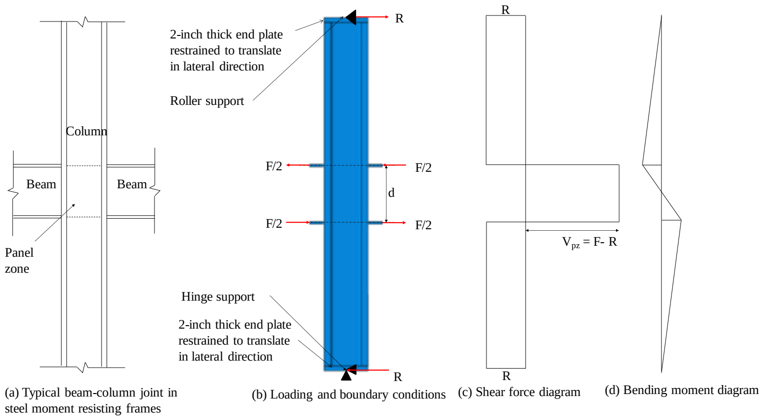


Figure 3.1. Loading, boundary conditions, shear force and bending moment diagrams

3.2. Analysis Cases

A simplified model of a column with a doubler plate was developed, as shown in Figure 3.1(b). Various key variables like column size, location of doubler plate to column welds, thickness of doubler plate, distance between loading plates, extension of doubler plate beyond loading points, replacing a thicker doubler plate on one side by two thin doubler plates of equivalent total thickness on both sides of column web, etc. were considered in the parametric studies.

Table 3.1. Analysis cases for W14x298 column section

Analysis Cases	d (inch)	Doubler plate				Weld location	Figure
		l_{dp} (inch)	b_{dp} (inch)	t_{dp} (inch)	N		
Case 1	24	-	-	-	-	-	
Case 2	24	24	10	1/2	1	Vertical welds only	Fig. 3.2(a)
Case 3		24	10	1/2	1	Horizontal welds only	Fig. 3.2(b)
Case 4		24	10	1/2	1	Vertical & horizontal welds	Fig. 3.2(c)
Case 5	24	24	10	9/8	1	Vertical welds only	Fig. 3.2(a)
Case 6		24	10	9/8	1	Horizontal welds only	Fig. 3.2(b)
Case 7		24	10	9/8	1	Vertical & horizontal welds	Fig. 3.2(c)
Case 8	24	24	10	9/16	2	Vertical welds only	Fig. 3.2(d)
Case 9	24	36	10	9/8	1	Vertical & horizontal welds	Fig. 3.2(e)
Case 17	24	36	10	1/2	1	Vertical welds only	Fig. 3.2(a)
Case 18		36	10	1/2	1	Vertical & horizontal welds	Fig. 3.2(c)
Case 19	36	36	10	1/2	1	Vertical welds only	Fig. 3.2(a)
Case 20		36	10	1/2	1	Vertical & horizontal welds	Fig. 3.2(c)
Case 21		-	-	-	-	-	

Table 3.2. Analysis cases for W33x263 column section

Analysis Cases	d (inch)	Doubler plate				Weld location	Figure
		l_{dp} (inch)	b_{dp} (inch)	t_{dp} (inch)	N		
Case 10	24	-	-	-	-	-	
Case 11	24	24	29.625	1/2	1	Vertical & horizontal welds	Fig. 3.2(c)
Case 12		24	29.625	1/2	1	Vertical welds only	Fig. 3.2(a)
Case 13	24	36	29.625	1/2	1	Vertical & horizontal welds	Fig. 3.2(e)
Case 14	24	24	29.625	9/8	1	Vertical & horizontal welds	Fig. 3.2(c)
Case 15		24	29.625	9/8	1	Vertical welds only	Fig. 3.2(a)
Case 16	24	24	15	9/8	1	Vertical & horizontal welds	Fig. 3.2(f)

Each combination of variables is referred to as an analysis case as shown in Table 3.1 and Table 3.2. In these tables, d is the centre to centre distance between loading plates, l_{dp} is the length of doubler plate, b_{dp} is the width of doubler plate, t_{dp} is the thickness of doubler plate, and N is the number of doubler plates. The various doubler plate arrangements are as shown in Figure 3.2.

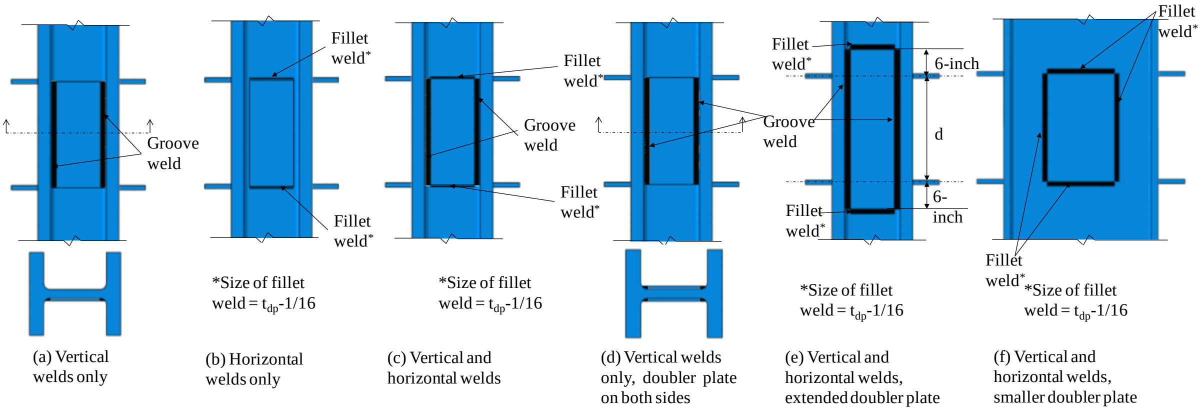


Figure 3.2. Various arrangements of doubler plate welds

4. DISCUSSION OF ANALYSIS RESULTS

4.1. Effect of Location of Doubler Plate Welds

The results of analysis cases 1 to 7, 19 to 21, 10 to 12 and 14, 15 were considered to better understand the most effective location of welding the doubler plate. Figure 4.1 (a) shows the panel zone shear versus panel zone rotation curves for the W14x398 column and Figure 4.2 (a) shows the Von Mises stress distribution in the column and the doubler plate at the mid-height for these cases. The stress distributions plotted in Figure 4.2 are for the point in the loading history of highest panel zone shear force considered in the analysis.

For the W14x398 column with 24-inch distance between the loading plates, the observations from these figures are: (a) the doubler plate is significantly less effective in taking the shear force when it is welded only at the top and bottom as done in case 3 and case 6; (b) welding the doubler plate only vertically is as effective as welding it on all four sides as seen from the panel zone shear versus rotation curves and Von Mises stress distribution for cases 2, 4 and cases 5, 7. It was also observed that in case 3 and case 6, the rotation of the doubler plate was more than the rotation of the column web in the panel zone region. Also slight buckling of the doubler plate was observed. Thus, welding the doubler plate only at the top and bottom is ineffective. In analysis cases 19, 20 and 21, W14x398 was again used as the column section but the distance between the loading points was 36-inches. A 1/2-inch thick doubler plate was used in analysis cases 19 and 20. From Figures 4.1 (b) and 4.2 (b) for these cases, it is seen that welding the doubler plate only vertically is as effective as welding it on all four sides even when the distance between the loading plates is increased.

For case 15 in which a 9/8-inch thick doubler plate was welded only vertically to the W33x263 column, buckling of the doubler plate was not observed. In this case the thickness of the doubler plate satisfies Equation (9-2) of *2005 AISC Seismic Provisions*. Whereas in analysis case 12 where a 1/2-inch doubler plate was welded only vertically, slight buckling of the doubler plate was observed. From Figure 4.1 (d), it is observed that, in case 15 in which the doubler plate is welded only vertically, in the elastic region the stiffness is larger as compared to the stiffness in case 14 in which the doubler plate is welded on all four sides. From a strength point of view also there does not seem to be any advantage in welding the double plate on all four sides.

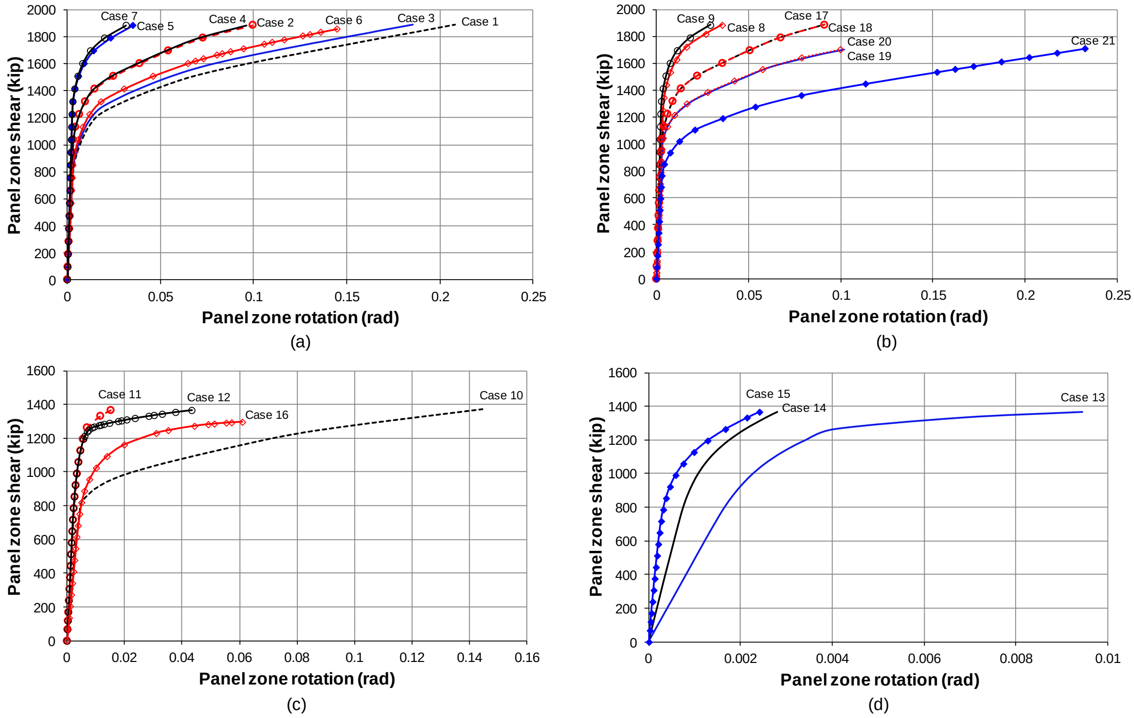


Figure 4.1. Panel zone shear vs. panel zone rotation

From the Abaqus analysis, it was possible to compute various force components in the welds attaching the doubler plate to the column. From this analysis it was observed that when the doubler plate was welded on all four sides, the vertical welds attracted more force as compared to the case where no welds were provided at the top and bottom of the doubler plate. It was also seen that the vertical force in the vertical weld in case 4 in which the doubler plate was welded on all four sides is larger than that in case 2 in which the doubler plate was welded only vertically.

Thus, from the above observations it can be concluded that providing only horizontal welds i.e. welding the doubler plate only at the top and bottom is ineffective. However, welding the doubler plate only along the vertical edges appears to be as effective as welding on all four sides. Thus, as long as the vertical edges are welded, there appears to be little advantage in welding the top and bottom edges of the doubler plate. A possible exception may be cases with relatively thin doubler plates that are prone to buckling. In such cases, welding along the top and bottom edges, in addition to the vertical edges, may help prevent or delay buckling of the doubler plate. Investigating buckling criteria for doubler plates was beyond the scope of this study, and should be considered in future research. Nonetheless, in cases where buckling of the doubler is not a concern, then it seems clear that welding the doubler plate only on the vertical edges is adequate.

4.2. Effect of Extension of Doubler Plate

The next issue of interest was to better understand if it would be beneficial to extend the doubler plate at the top and bottom beyond the loading points, i.e. beyond the depth of the connected beams. The other item of interest was to verify the observation from the previous section that even when the doubler plate is extended, welding the doubler plate only along the vertical edges is as effective as welding it on all four sides. From the literature review it is observed that in most of the past experimental research, the doubler plate was extended beyond the beam depth. The magnitude of extension varied in the various past studies. To study these issues, the results of analysis cases 4, 7, 9, 17, 18, 11 and 13 will be considered. From Figures 4.1(b) and 4.2(b) for cases 17 and 18, it is seen that even when the doubler plate is extended, welding it only vertically is as effective as welding it on all four sides. From these figures for cases 4, 18 and 7, 9 there appears to be no significant benefit in extending the doubler plate for the W14x398 column. It was also observed that the stresses in the column web and column flange were not significantly reduced by extending the doubler plate.

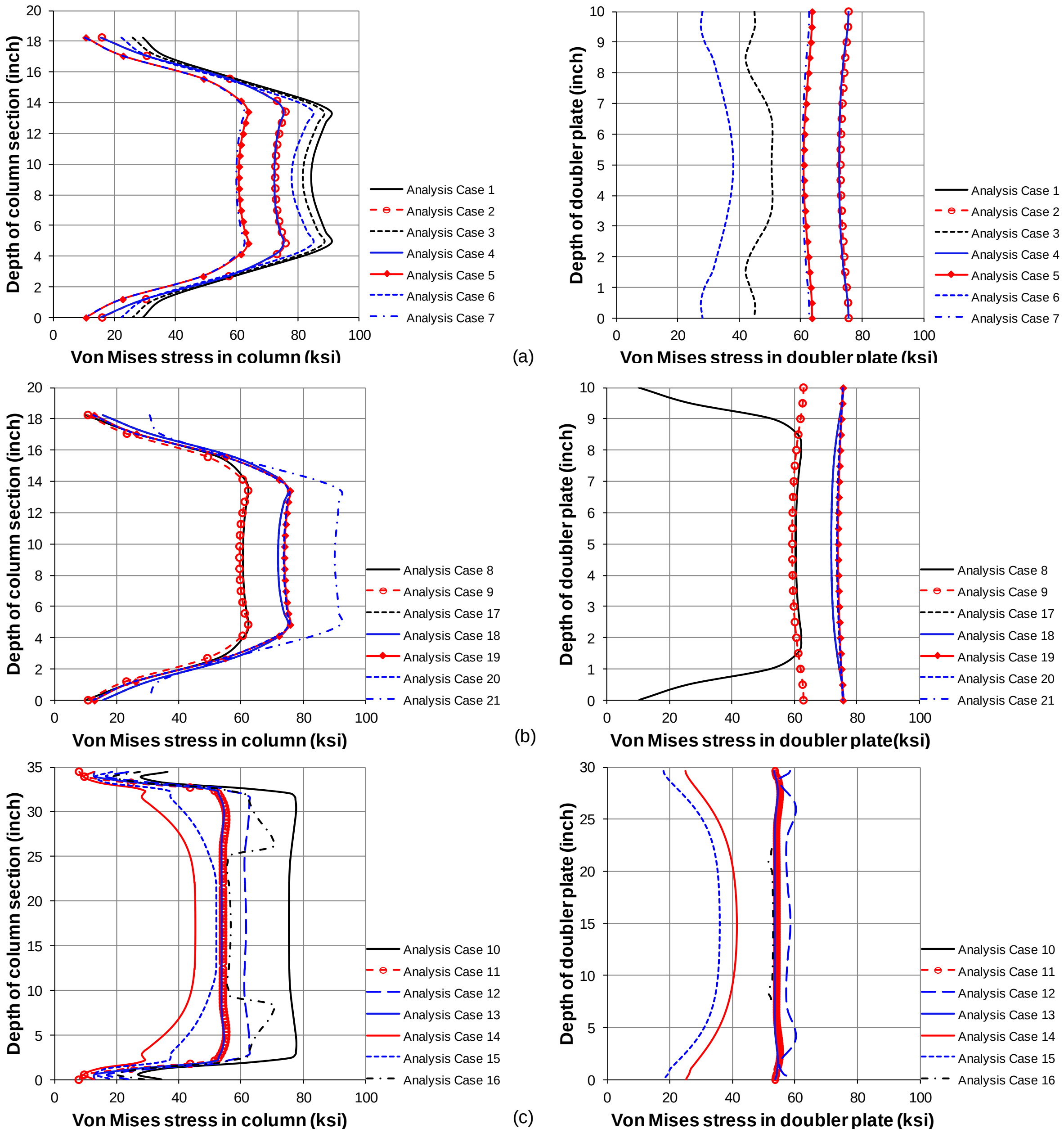


Figure 4.2. Von Mises stress in column and doubler plate at mid-height

As seen from Figures 4.1 and 4.2 for cases 11 and 13, in the elastic region, the stiffness in case 13 is larger as compared to that in case 11 and the strength at low levels of inelastic rotation also appears to be somewhat larger. But the Von Mises stress in both the cases is almost the same. It was also observed that the stresses in the column web and column flange were not reduced significantly when the doubler plate was extended as compared to when it was not extended. Nonetheless, extending the doubler plate provides some advantages of improved strength and stiffness for the deeper W33x263 column.

In conclusion, for the W14x398 column, there appears to be little advantage in extending the doubler plate in terms of panel zone strength and stiffness. However, for the deeper W33x263 column, there was some increase in stiffness and strength of the panel zone when the doubler plate was extended beyond the load application points.

4.3. Splitting Doubler Plate on Two Sides

To study if there is any benefit in providing two thinner doubler plates of equivalent thickness instead of one thick doubler plate, the results of analysis cases 5 and 8 were considered. From Figure 4.1 the stiffness in case 5 in the elastic region seems to be slightly larger than in case 8. From the Von Mises stress distribution in the column and the doubler plate as shown in Figures 4.2, there appears to be little difference in the stress in the column and the doubler plate in both the cases. Thus, from this limited study, it appears that there is little advantage in splitting a thicker doubler plate into two thinner doubler plates from the point of view of panel zone stiffness and strength. Note, however, there may be advantages from a fabrication and welding point of view. A very thick doubler plate requires large welds. These large welds, in turn, can cause weld shrinkage induced distortion and residual stresses. Splitting a single thick doubler plate into two thinner plates will reduce the weld size, and therefore may reduce potential problems associated with very large welds. Thus, while there appears to be little advantage in splitting a doubler from a structural performance point of view, there may be advantages from a fabrication and welding point of view. Further, when splitting a thick doubler into two thinner doublers, buckling of the doubler plate may become a concern. Note that both welding issues and doubler plate buckling issues were beyond the scope of this study.

4.4. Effect of Narrow Doubler Plate for W33x263 Column

The next issue examined was to see if there was any advantage of providing a narrower doubler plate for a deep column like W33x263. If this arrangement is effective, it would help to reduce the material required for the doubler plate and may simplify welding by allowing the use of fillet welds along the vertical edges of the doubler plate. Results of analysis case 16 were considered for this purpose and were compared with analysis case 10 (no doubler plate) and analysis case 11 (full width doubler plate). As seen from Figures 4.1 and 4.2 for cases 10, 11 and 16, the panel zone stiffness and strength are greatly reduced with the narrow doubler plate compared with the more conventional full width doubler plate. The Von Mises stresses in the column web where the doubler plate is present were low but these stresses were high in the portion of the column web where no doubler plate was present. Consequently, a significant amount of shear deformation is contributed by the unreinforced portion of the column. Overall, the use of a narrow doubler plate results in a significant loss of panel zone stiffness and strength.

4.5. Forces in the Welds

4.5.1. Forces in vertical welds

An attempt was made to determine if the design requirements for welds along the vertical edges of the doubler plate given in Section 9.3c of the *2005 AISC Seismic Provisions* are reasonable based on the analysis results. From the analysis results, the predominant force observed in the vertical welds was the vertical force. The ratio of vertical force in the vertical weld to the shear strength of the doubler plate based on actual yield stress of doubler plate ($=0.6*R_y*F_y*t_{dp}*l_{dp}$) was calculated for each analysis case. It was observed that this ratio varied between 0.5 and 1.3. In analysis cases 4 and 20, where this ratio was maximum, the panel zone rotation was around 0.1 radians (see Figure 4.1 (a), (b)). Whereas in analysis cases 14 and 15 in which this ratio is minimum, the panel zone rotation was much smaller (see Figure 4.1 (d)) and also the stress in the doubler plate was below the actual yield stress (see Figure 4.2 (d)). It should be noted that the analyses were all load controlled. Thus, in some cases large inelastic deformation was not seen. In future research it is recommended that a displacement control analysis be performed instead of a load control analysis. Nonetheless, from the available results it may be concluded that for cases where the panel zone experienced large inelastic rotations, the force in the vertical weld is of the order of the shear strength of the doubler plate. However, based on these analyses, it may be preferable to design the vertical welds for the expected shear strength of the doubler plate ($=0.6*R_y*F_y*t_{dp}*l_{dp}$) rather than for the available shear strength. It may also be preferable to include a factor for strain hardening of the doubler plate, which increases the force demands on the welds. Based on this very limited analysis, it appears that a reasonable design approach for the vertical welds would be to define the required strength of the welds to be equal to

1.25 times the expected shear strength of the plate. However, further research and analysis is needed to confirm this approach.

4.5.2. Forces in horizontal welds

An attempt was made to establish a relation between the forces in the horizontal weld and the force transmitted to the doubler plate. The portion of the force transmitted to the doubler plate can be interpreted as the shear force in the doubler plate. This force (V_{dp}) was calculated as: $V_{dp} = V_{pz} – V_n$, where V_n and V_{pz} are defined in Section 3.1.2. It was observed that the ratio of horizontal force in the horizontal weld to the force transmitted to the doubler plate was not consistent for all the analysis cases in which horizontal welds were provided. One thing which is clear from the results is that the force in the horizontal weld when the doubler plate is extended beyond the load points (cases 9 and 18) was very low. Thus, it can be concluded that even if a designer wishes to extend the doubler plate, the welds at top and bottom do not develop significant forces. However, additional research is needed to examine forces in the horizontal welds when the welds are required to restrain buckling. For cases where the doubler plate does not extend beyond the load points, large horizontal forces can be developed in the horizontal weld. The results of these analyses suggest that taking the required strength of the horizontal weld equal to the force transmitted to the doubler plate appears to be conservative, and in many cases it appears to be very conservative.

An attempt was also made to see if there is any relation between the horizontal force in the horizontal weld and the shear strength of the doubler plate calculated using actual yield stress. But a consistent relation between the forces in the weld and the shear strength of the doubler plate was not seen. Thus, based on this limited analysis, it appears that the design criteria for the horizontal welds as given in the *2005 AISC Seismic Provisions* is not accurate, but is conservative. Additional research is needed to better characterize the force in the horizontal welds. However, it should be emphasized again that the research has shown there is little advantage to adding horizontal welds on doubler plates except possibly when buckling of thin doubler plates is a concern. So, in most cases, using only vertical welds on the doubler plate is adequate, and this precludes the need to develop a design criteria for the horizontal welds.

5. CONCLUDING REMARKS

This paper provided an overview of preliminary analysis of doubler plate attachment details for the steel moment resisting frames considering twenty-one analysis cases. For this purpose a section of column between inflection points was modelled in Abaqus using 3D solid elements. Relationships between the forces in the weld and the shear strength of the doubler plate were also studied.

It was concluded that welding the doubler plate only along its horizontal edges (i.e. only at the top and bottom) is not an effective way of attaching a doubler plate to the column web. Welding the doubler plate in this manner does little to reduce the stresses in the panel zone region of the column. The analyses also showed that welding the doubler plate only along its vertical edges allowed for the development of the full strength of the doubler plate in all cases that were examined. If the vertical edges are welded, there is little advantage of welding the top and bottom edges also. The only benefit of welding the top and bottom edges in addition to the vertical edges was to help restrain buckling of thin doubler plates. Further research is needed to investigate the benefits of top and bottom welds for doubler plate stability.

Extending the doubler plate 6-inches above and below the loading points showed little benefit in the case of the W14x398 column. For the deeper W33x273 columns, however, extending the doubler plate beyond the loading points resulted in a modest increase in panel zone stiffness and strength. From a strength and stiffness point of view, providing two thinner doubler plates is essentially the same as providing one thick doubler plate. Providing a narrower doubler plate for a deep column significantly reduced panel zone stiffness and strength compared to the more conventional full width doubler plate, and is not recommended.

Recommendations in the *2005 AISC Seismic Provisions* for designing the vertical welds for a doubler plate appear reasonable compared to the weld forces computed from the analysis, but may be not conservative. Recommendations in the *2005 AISC Seismic Provisions* for designing the horizontal welds for a doubler plate appear inaccurate compared to the analysis results, but are conservative.

The research on doubler plate attachment details conducted herein is considered preliminary in nature, as a simplified model of the panel zone region was used and only monotonic loading was consider. It is recommended that a detailed study of doubler plate attachment details should be done by evaluating performance of various doubler plate attachment details under cyclic loading. This will require the use of material stress-strain models for steel and for welds that are representative of cyclic loading response. Additional research is needed wherein the continuity plates should be modelled along with doubler plates and various options for welding the continuity plate to the doubler plate should be examined.

When continuity plates are provided, additional research is needed regarding the benefits of extending the doubler plate beyond the panel zone region. In this research, an interior beam-column joint of a steel moment resisting frame was considered and the conclusions were drawn for this interior joint. The same conclusions may not be applicable for an exterior joint where the column is subjected to loading only from one side. Thus, additional research is needed to investigate this condition. The detail in which groove welds were provided along the vertical edges of the doubler plate was used in this research. Additional research is needed for the case where fillet welds are used to attach the vertical edges of the doubler plate to the column. Also a comparison needs to be made to evaluate the relative merits of groove welds versus fillet welds. Further research is needed on the stability of doubler plates, and how doubler plate stability is affected by the attachment details, including the use of welds on the top and bottom edges of the doubler plate and by the use of plug welds to connect the doubler plate to the column web.

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