Exposed Column-Base Plate Connections Bending About Weak Axis: II. Experimental Study

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Abstract

Four exposed-type column-base plate connection sub-assemblages (two for 6-bolt connection and two for 4-bolt connection) were tested under the SAC Phase II loading history in the direction of column weak axis. Mainly investigated in this experimental study were global cyclic performance of each test specimen and behavior of major connection elements under large column lateral displacements. Also examined were effects of different number of anchor bolt, different relative strength between base plate and anchor bolts, and different filler metal and welding detail on cyclic ductility in the connection. Only one of the four specimens completed the applied entire loading history without significant strength degradation and formed a plastic hinge at the bottom of the column. The other three specimens showed a limited ductility in the connection. Through the experimental study, the Drake and Elkin's design method that was used for design of the test specimens was evaluated and several major findings from the numerical parametric study presented in the companion paper, which is published together, were verified.

Keywords: Connection, Base Plate, Weak Axis, Cyclic Performance, Relative Strength Ratio

1. Introduction

An experimental study is conducted at the University of Michigan to evaluate the Drake and Elkin’s design method (Drake and Elkin, 1999) (referred to hereafter as the D&E method) for the exposed-type column-base plate connections bending about weak axis and to verify several major findings from the numerical parametric study presented in the companion paper (Lee et al., 2008), which is published together. For this study, a total of four column-base plate connection sub-assemblages (two for 6-bolt connection and two for 4-bolt connection) are designed as per the D&E method and fabricated using two different notch-tough filler metals, both satisfying the minimum requirements specified in the AISC Seismic Provisions (AISC, 2005). These specimens are cyclically tested under the SAC Phase II loading history (SAC, 1997).

Through the numerical parametric study conducted by the authors (Lee and Goel, 2001, Lee et al., 2002 and 2008), it has been shown that different number (arrangement) of anchor bolt could significantly change base plate yielding pattern on the tension side and also affect bearing stress distribution on the compression side.

In order to reconfirm the above observation, two different number of anchor bolt cases (i.e., 6 bolt and 4 bolt cases) are studied in this test program. The above finite element analysis results also noted a significance of the relative strength ratio among the connection elements (i.e., column, base plate, and anchor bolts) in connection behavior under large column lateral displacements. In order to further investigate the relative strength ratio effects under reversal cyclic loading, two different total anchor bolt sizes (stiffnesses) are tested for the same column element and base plate thickness. Larger total anchor bolt size (stiffness) is selected for the 4-bolt specimens.

Cyclic behavior of the four test specimens is analyzed and global connection responses are compared with predictions from the numerical analyses presented in the companion paper (Lee et al., 2008). Effects of different number of anchor bolt, different relative strength between base plate and anchor bolts, and different filler metal and welding detail on cyclic performance of the connection are investigated through these comparisons and analyses. Several factors that may significantly affect cyclic ductility in the connection are also examined. Further, mostly based on study of strain distribution on the surface of the base plate and base plate behavior on both the tension and compression sides, the D&E method is experimentally evaluated and several possible limitations of this design approach are explored. This paper presents

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analysis results of cyclic performance of the four test specimens and also discusses major findings from this experimental study.

2. Design of Test Specimen

2.1. Base plate and anchor bolt

An assembly of 80 in. long W12x96 A572 Grade 50 column and 20 in. × 20 in. A36 base plate is selected for the four exposed-type column-base plate connection sub-assemblies (two for 6-bolt connection and two for 4-bolt connection) tested in this experimental study. For the given connection geometry presented in Fig. 1 and by choosing 6 ksi for $f'c$ of the grout, 2.25 in. thick base plates are designed by the D&E method for both the 6-bolt and 4-bolt test specimens. This base plate thickness is denoted herein as “$t_{po}$” for convenience.

ASTM A354 Grade BD, which has $F_{u,bolt} = 150$ ksi and $F_{y,bolt} = 130$ ksi, is selected for anchor bolts to resist the design tensile forces calculated from the D&E method as well as the design shear forces in the connection. The total anchor bolt size on the tension side, designed following the ASTM A490 high strength bolt design procedure within the 2005 AISC LRFD Specifications (AISC, 2005), is denoted in this paper as “$K_o$”. The designed minimum diameter of each anchor bolt is 1.107 in. for the 6-bolt connection, whereas it is 1.356 in. for the 4-bolt connection. As indicated in Fig. 1, however, 1.25 in. and 2.0 in. anchor bolts are chosen for the 6-bolt and 4-bolt specimens, respectively, in this experimental study. A 30% stiffer (i.e., 1.3 $K_o$) anchor bolt is selected for the 6-bolt specimens, whereas 120% stiffer (i.e., 2.2 $K_o$) anchor bolt is selected for the 4-bolt specimens. Such increase is intended to provide stronger anchor bolts so as not to yield at the ultimate state of the connection during the test. In addition, the difference in the anchor bolt stiffness between the 6-bolt and 4-bolt specimens will facilitate study of the relative strength ratio effects on cyclic performance of the connection.

Over-sized bolt holes, as per the 2005 AISC LRFD Specifications (AISC, 2005), are designed to make it easy to place the connection sub-assemblies on their intended location. 1.5 in. (−1.25 in. +4/16 in.) diameter bolt holes are thus prepared for the 6-bolt specimens while 2.3125 in. (−2.0 in. +5/16 in.) diameter bolt holes are prepared for the 4-bolt specimens. For washers, ASTM F436 circular washers with 3.0 in. nominal outside diameter are used only for the 6-bolt specimens. The top and bottom ends of each anchor rod are threaded. Threads on the top of the anchor rods are used to fasten the base plate on the grout effectively. In addition, as shown in Fig. 2, bottom of each anchor rod is fixed to the concrete foundation using supplemental steel plates and double nuts. All nuts are hand-tightened and thus, effects of anchor bolt prestressing can be neglected in this experimental study.

2.2. Filler metal and welding detail

Figure 3 illustrates the connection welding detail used to fabricate the test specimens. Two welding materials, satisfying the general requirements within Section 7.3 of the 2005 AISC Seismic Provisions (AISC, 2005), are used to connect the column to the base plate. These filler metals are ER70S-3 (Lincoln SuperArc L-50 SuperGlide...
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S3) and E70TG-K2 (Lincoln Innershield NR-311Ni). Typical mechanical properties of the two welding materials, provided by the manufacturer, are compared in Table 1. As indicated in Fig. 3, ER70S-3 is used for the first pair of the specimens (i.e., SP 6-1 and SP 4-1) while E70TG-K2 is used for the second pair of the specimens (i.e., SP 6-2 and SP 4-2). Effects of these two different welding materials on cyclic ductility in the connection will be discussed later in this paper.

The connections tested by Fahmy (1999) formed a basis of the welding detail in this experimental study. As shown in Fig. 3, column flanges are welded to the base plate using the Partial Joint Penetration (PJP) groove welds that consist of a 30° single bevel with 1/8 in. root face. 1/4 in. fillet welds connecting the inside of the column flanges and the column web to the base plate are also placed using ER70S-3 filler metal in SP 6-1 and SP 4-1 and using E70TG-K2 filler metal in SP 6-2 and SP 4-2. In order to prevent an early fracturing in the welds at column flange tips, the PJP groove welds are reinforced by 3/16 in. fillet welds only in SP 6-1, SP 6-2, and SP 4-1. Comparison of the test results between SP 4-1 and SP 4-2 will thus make it possible to investigate effects of the fillet reinforcement over the PJP groove welds on cyclic column-base plate connection performance in case of the weak axis bending. Table 2 summarizes major parameters that will be investigated through this experimental study.

### 3. Material Property and Test Setup

In order to estimate the actual moments transferred from the column to the connection at its ultimate state, it is necessary to obtain mechanical properties of the column. For this reason, tensile coupon tests were performed on selected two column elements which consisted of the test specimens; one from SP 6-1 and one from SP 4-1. For each W-section, two coupons were taken from one of the two column flanges and tested in accordance with ASTM E8. These material test results are summarized in Table 3. It should be noted that the expected column yield stress (i.e., 58 ksi) used for the design calculations and the numerical parametric study is clearly over-estimated as compared with the measured yield strength of the column.

As shown in Fig. 4, a test setup is prepared for this experimental study so that cyclic lateral displacements can be applied at the top end of the column, forcing plastic hinge formation at the bottom of the column member. An MTS hydraulic actuator that is capable of reversal cyclic loads up to 100 kips and stroke range of +/- 5 in. is installed in this test setup. The SAC Phase II loading history (SAC, 1997) is applied to ensure the results can be compared with numerous beam-to-column moment connection test results conducted during the SAC investigations. The drift in this loading history is defined as the ratio between the column lateral displacement and the column height.

#### Table 1. Mechanical properties of welding material

<table>
<thead>
<tr>
<th>Welding Material</th>
<th>Yield Strength (ksi)</th>
<th>Tensile Strength (ksi)</th>
<th>CVN at -20°F (ft-lbs)</th>
<th>Elongation (%)</th>
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</thead>
<tbody>
<tr>
<td>ER70S-3</td>
<td>63.7</td>
<td>76.2</td>
<td>65</td>
<td>30</td>
</tr>
<tr>
<td>E70TG-K2</td>
<td>69</td>
<td>86</td>
<td>35</td>
<td>27</td>
</tr>
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</table>

*The data in this table are given by Lincoln Electric company.

*The 2005 AISC Seismic Provisions (AISC, 2005) require a minimum CVN toughness of 20 ft-lbs at -20°F.

#### Table 2. Major parameters of the test specimens

<table>
<thead>
<tr>
<th></th>
<th>6-bolt Specimen</th>
<th>4-bolt Specimen</th>
</tr>
</thead>
<tbody>
<tr>
<td>Number of Anchor Bolt</td>
<td>6</td>
<td>6</td>
</tr>
<tr>
<td>Anchor Bolt Stiffness</td>
<td>1.3 $K_o$</td>
<td>1.3 $K_o$</td>
</tr>
<tr>
<td>Base Plate Thickness</td>
<td>$t_{po} (=2.25$ in.)</td>
<td>$t_{po}$</td>
</tr>
<tr>
<td>Weld Material</td>
<td>ER70S-3</td>
<td>E70TG-K2</td>
</tr>
<tr>
<td>3/16 in. Fillet Overlay on PJP Weld</td>
<td>Yes</td>
<td>Yes</td>
</tr>
</tbody>
</table>
displacement and its clear length. In order to observe the stiffness degradation in the connection during the test, two small cycles (i.e., 0.5% drift cycles) are inserted between two large inter-story drifts after 1.0% drift level. Effects of column axial loads on the connection behavior are not studied in this experiment.

Each specimen is supported by a pair of lateral braces as shown in Fig. 4 to prevent out-of-plane deformation of the column under large lateral displacements. These lateral braces are reinforced additionally by two threaded bars in the transverse direction. The concrete foundation is fixed to the strong floor by threaded anchor rods and anchor plates at the four corners of this foundation so as not to move vertically or horizontally during the test. A 2 in. thick layer of non-shrink grout is placed between the base plate and the concrete foundation to plum each specimen and to make a smooth contact interface.

4. Instrumentation

Extensive instrumentations, including post-yield strain gauges and Linear Variable Differential Transformers (LVDTs), are used to collect information about specimen behavior during the cyclic tests. YFLA-5, manufactured by TML Tokyo Sokki Kenkyyo Co., Ltd., is chosen for all strain gauges. In order to measure deformed shape of the base plate and anchor bolt elongations, TRS 50 and TRS 100 position transducers, manufactured by Novotechnik, are used in this experimental study. Especially for the 6-bolt specimens, one additional position transducer is installed to measure lateral movements of the base plate under large column moments. The majority of the instrumentation is concentrated on the surface of the base plate and anchor bolts as schematically presented in Fig. 5. Specific targeted regions in this study include the base plate centerline (line “A”), the assumed bending line in the base plate (line “B”), and the base plate edge line (line “C”). In addition, measured maximum strains at the point “a” in Fig. 5 can be used to monitor major yielding in the base plate along the assumed bending line on both the tension and compression sides.

In order to measure strain variation in the anchor bolt on the tension side, strain gauges are installed on the surface of the anchor bolt as schematically presented in Fig. 5. These are located in the middle of the 2 in. thick grout layer. In order to protect the strain gauges from moisture while the grout is being cured, N-1 (Moisture-proofing Material) from TML Tokyo Sokki Kenkyyo Co., Ltd. is covered over the gauges. 3M Scotch 2228 rubber mastic tape is also covered to protect the strain gauges from unexpected contact forces from the grout during the cyclic tests. In this experimental study, the total tensile bolt forces are directly calculated from the measured strain values on the surface of the anchor bolts and their areas.

5. Connection Performance

Two 6-bolt specimens (i.e., SP 6-1 and SP 6-2) and two 4-bolt specimens (i.e., SP 4-1 and SP 4-2) are tested under the SAC Phase II loading history (SAC, 1997). Only SP 4-1 completed the whole loading history up to the 4.0% interstory drift without significant strength degradation. The other three specimens showed a limited ductility in the connection. Especially, due to premature rupturing at the boundary between the column and the
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After completing the two cycles at 4.0% drift required by the SAC protocol (SAC, 1997), two additional 5.0% drift cycles are applied on SP 6-1, SP 6-2, and SP 4-1 before each test is completed. The plots of the moments versus drifts are presented in Fig. 6. In this figure, the measured moments are normalized by the plastic moment \( M_p \) of the column. The total plastic rotations compared in Fig. 7 are determined by subtracting measured elastic rotations from the total rotations calculated at the top end of the column. For the determination of the elastic stiffnesses, the values at the first peak of 0.75% drift cycle are used. As mentioned earlier, one position transducer is installed in the 6-bolt specimens to measure lateral movements of the base plate under large column lateral displacements. The measured maximum base plate movements during 3.0% drift cycles are 0.014 in in SP 6-1 and 0.019 in in SP 6-2. These amounts are very small and thus ignored in calculation of the interstory drifts and subsequent analyses.

5.1. Performance of SP 6-1

SP 6-1 is distinguished by the use of three anchor bolts on the tension side (three on the compression side) and the use of ER70S-3 filler metal for the Partial Joint Penetration (PJP) groove welds and the 3/16 in fillet reinforcement. In order to prevent significant yielding in the anchor bolts during the test, 30% stronger anchor bolts than what the D&E method provides (i.e., 1.3 \( K_o \)) are designed for the two 6-bolt specimens (i.e., SP 6-1 and SP 6-2). As shown in Fig. 6(a), strength of SP 6-1 started to decrease near the second peak of 3.0% drift cycle due to significant crack propagation at one of the four column flange tips and this specimen showed only half of its moment capacity during the second cycle at 4.0% drift. As presented in Fig. 7, the measured total plastic rotation of SP 6-1 is 0.0139 radian during the first cycle at 3.0% drift.

In SP 6-1, the primary failure mode is cracking and subsequent rupturing at the boundary between the column and the base plate. Due to high strain gradient, initial cracks in this specimen were originated at the three column flange tips indicated in Fig. 8(a). Major crack openings at these locations were commonly observed at the level of toe of the 3/16 in fillet welds, away from the upper surface of the base plate. The initial crack at the outside of the column flange “R1” tip which was first visible during the 1.5% drift cycles was significantly propagated during the first cycle at 3.0% drift, and this directly caused strength degradation near the second peak of 3.0% drift cycle. Figure 9 presents crack opening at the column flange “R1” tip during the second cycle at 3.0%
drift. SP 6-1 lost almost half of its moment capacity during the second cycle at 4.0% drift because of rupturing at the column flange “R1” tip and crack propagation at the column flange “L1” and “L2” tips.

5.2. Performance of SP 6-2

Despite the different filler metal connecting the column to the base plate, SP 6-2 showed very similar cyclic responses as compared with those of SP 6-1 up to 2.0% drift. As indicated in Fig. 3, E70TG-K2 filler metal is used for the fabrication of SP 6-2. No strength degradation has been observed in this specimen until two load cycles at 3.0% drift were completed. However, as shown in Fig. 6(b), SP 6-2 showed a gradual strength drop during the first cycle at 4.0% drift and lost most of its moment capacity during the next load cycle. The measured total plastic rotation of SP 6-2 during the first load cycle at 3.0% drift is 0.0128 radian.

Moderate yielding of the column in SP 6-2 was visible near the column flange tip area, initiating during 0.75% drift cycles. Based on analysis of the measured strains, the column flanges fully yielded near the boundary area during the first cycle at 3.0% drift. Due to premature fracturing in this specimen, yield pattern and distribution in the base plate could not be thoroughly investigated. Up to 3.0% drift cycles, only local base plate yields were observed on its surface near the four column flange tips.

In SP 6-2, initial cracks at the column flange tips were observed at the level of toe of the 3/16 in. fillet welds or at the level of toe of the 1/4 in. fillet welds. Locations of these initial cracks are indicated in Fig. 8(b). The crack at the column flange “L2” tip, initially observed during the first cycle at 1.0% drift, grew continuously with the increase of the drift level. Figure 10 shows a deep crack at this location during the first cycle at 3.0% drift. Interestingly, this crack opening did not degrade the moment capacity of the connection up to the second cycle at 3.0% drift, as shown in Fig. 6(b). The column flange “L2” tip, however, was significantly fractured near the first peak of 4.0% drift cycle and this crack opening coupled with crack propagation at the column flange “L1” tip caused the gradual strength drop during the first cycle at 4.0% drift.

5.3. Performance of SP 4-1

SP 4-1 is characterized by the use of two anchor bolts on the tension side (two on the compression side) and the use of 120% stronger anchor bolts (i.e., 2.2 $K_o$). In order to connect the column to the base plate, ER70S-3 filler metal is used. As shown in Fig. 6(c), SP 4-1 showed a very ductile connection behavior through the applied entire loading history. In spite of the initial cracks mainly formed at toe of the 3/16 in. fillet welds during 1.5% through 2.0% drift cycles, as indicated in Fig. 8(c), this specimen exhibited excellent energy dissipation capacity up to 5.0% drift cycles. Minor crack openings and their limited propagations at the column flange tips forced a plastic hinge formation at the bottom of the column during higher drift cycles, as shown in Fig. 11. Even though SP 4-1 reached the plastic moment capacity ($M_p$) of the column, the base plate in this specimen did not fully yield across its whole width. Instead, only small yielded regions were observed on the surface of the base plate. This indicates that the D&E method results in stiffer base plates. The measured total plastic rotation of SP 4-1 is 0.0302 radian during the first cycle at 5.0% drift.
5.4 Performance of SP 4-2

As indicated in Fig. 3, E70TG-K2 filler metal is used for the fabrication of SP 4-2. In this specimen, the 3/16 in. fillet reinforcement is not added over the PJP groove welds. SP 4-2 showed a sudden brittle failure at early stage of the loading history. Figure 6(d) shows a remarkable strength drop near the first peak of 1.5% drift cycle. This specimen lost most of its moment capacity during the first load cycle at 2.0% drift. As presented in Fig. 7, the measured total plastic rotation of SP 4-2 is 0.0017 radian during the first cycle at 1.0% drift.

Initial cracks in SP 4-2 were first visible at the column flange “R1”, “R2”, and “L1” tips during the first load cycle at 1.0% drift as indicated in Fig. 8(d). These crack openings were observed at the level of 1/4 in. above the top surface of the base plate. This was about the same location of toe of the 1/4 in. fillet welds placed inside the column flanges. Near the first peak of 1.5% drift cycle, the crack at the column flange “L1” tip was significantly propagated and this caused the sudden strength degradation in SP 4-2. A gradual crack opening was also observed at the column flange “R1” tip during the first load cycle at 1.5% drift. This crack propagation at the column flange “R1” tip was captured in Fig. 12. The “L1” and “R1” column flanges were completely fractured during the first load cycle at 2.0% drift.

6. Test Results and Discussion

Test results of the four specimens are analyzed and major findings from this experimental study are discussed. First, total connection lateral forces and total tensile bolt forces of these four specimens are compared each other to investigate global connection responses during the test. The experimental outputs are also compared with the Finite Element Analysis (FEA) results presented in a companion paper (Lee et al., 2008). In addition, an effort has been made to examine effects of the relative strength between the base plate and anchor bolts for a given column element on cyclic performance of the connection. Second, strain variations on the surface of the base plate in the longitudinal direction are investigated to find location of the maximum curvature in the base plate due to its bending. Base plate yielding along the assumed bending lines has also been monitored based on peak strains measured at the point “a” presented in Fig. 5. Third, effects of different number (or arrangement) of anchor bolt on base plate deformation and strain
distribution in the base plate are examined. Lastly, based on experimental observations, effects of different welding material and welding detail on failure modes and cyclic connection ductility are studied. Effects of two different filler metals are investigated from the comparison of the test results between SP 6-1 and SP 6-2. Comparison of the cyclic connection performance between SP 4-1 and SP 4-2 facilitates to investigate effects of two different welding details selected in this experimental study.

6.1. Total lateral force

Measured total lateral forces of the four specimens are plotted and compared in Fig. 13. In general, it is not easy to compare continuous experimental output data directly to the results of numerical analysis especially in the experiments which are controlled by cyclic displacements. For this reason, a total of twelve drift points are selected herein for an efficient comparison between the test results and the FEA estimations. These drift levels are 0.05, 0.10, 0.15, 0.25, 0.35, 0.55, 0.75, 1.15, 1.70, 2.60, 3.85, 5.00%. In Fig. 13, specimen responses measured from only one column displacement direction are plotted. Direction of the column displacement that is forcing tension in the “L1” and “L2” column flanges is selected for this analysis. Unstiffened exposed-type column-base plate connection is usually classified as a partially-restrained joint. For the investigation of partial fixity of the connection in the linear range, lateral stiffness of each specimen is compared with the cantilever elastic stiffness marked as the dot-line in Fig. 13.

As shown in Fig. 13(a), stiffnesses and strengths of the two 6-bolt specimens (i.e., SP 6-1 and SP 6-2) show a good agreement with the FEA predictions up to around 1.70% drift, after which the test results show gradual strength degradation due to crack propagation in the welds connecting the column to the base plate. In the linear response range, lateral stiffnesses of these two specimens are commonly 66% of the elastic stiffness of the cantilever. Figure 13(b) compares experimental outputs of the two 4-bolt specimens (i.e., SP 4-1 and SP 4-2) with the FEA predictions. By using the larger anchor bolt size (i.e., 2.2 \(K_o\)), it has been expected that the 4-bolt specimens develop stiffer initial connection responses as compared with those of the 6-bolt specimens, which consist of smaller anchor bolt stiffness (i.e., 1.3 \(K_o\)). However, the measured lateral stiffness of SP 4-1 showed only 57% of the elastic stiffness of the cantilever. This amount is clearly lower than that of the 6-bolt specimens. Only SP 4-1 completed the applied entire loading history up to 5.0% interstory drift and thus, the test results of this specimen could be able to be compared with the FEA results at higher drifts (after 3.0% drift). In the range of 3.85% through 5.0% drift, as shown in Fig. 13(b), the measured total lateral forces of SP 4-1 are clearly smaller than the FEA predictions. Major reason of these differences can be easily found in Table 3. The tensile coupon test results shown in this table indicate that the yield strength of SP 4-1 is found to be around 50 ksi while, as mentioned earlier, an over-estimated column yield stress

![Figure 13. Comparison of lateral force-drift response envelopes.](image-url)

<table>
<thead>
<tr>
<th>Coupon Identification</th>
<th>Coupon Size</th>
<th>Width (in.)</th>
<th>Thickness (in.)</th>
<th>Yield Strength (ksi)</th>
<th>Tensile Strength (ksi)</th>
<th>Y/T (%)</th>
<th>Elongation (%)</th>
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</thead>
<tbody>
<tr>
<td>SP 6-1</td>
<td>LF1</td>
<td>0.752</td>
<td>0.854</td>
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<td></td>
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<td>69.7</td>
<td>72.6</td>
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<td>SP 4-1</td>
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<td>74.1</td>
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<td>72.0</td>
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(i.e., 58 ksi) is used for the finite element modeling. The higher column yield stress in the numerical analysis, of course, results in the higher connection strength under large column lateral displacements.

6.2 Total tensile bolt force

In Fig. 14, total tensile bolt forces of the four specimens are compared with the FEA results. As mentioned, bolt forces are calculated directly from the strains on the surface of the anchor bolts at the multiple positions, schematically presented in Fig. 5, and the area of anchor bolts. For convenience, mean values of these strain readings are used for the calculation. Figure 14(a) presents the results of the 6-bolt specimens. Adjusting small shifts at the early stage of the loading history, the measured total tensile bolt forces of SP 6-1 and SP 6-2 are in a good agreement with the FEA results up to around 1.70% drift. The test results of SP 4-1, shown in Fig. 14(b), are better predicted by the FEA results through the applied entire loading history.

A significant point which should be mentioned in Fig. 14 is that the maximum tensile bolt forces for the 4-bolt specimen at higher drifts predicted by the numerical analysis are clearly higher than the FEA predictions for the 6-bolt specimen. This gap is mainly originated from the difference in the anchor bolt stiffness between the 6-bolt and 4-bolt specimens. In the case of stiffer anchor bolt (i.e., 2.2 \( K_o \)) in the 4-bolt specimen, due to constraint of the base plate rotation on the tension side, the overall moment arm between the total tensile bolt force and the resultant bearing force becomes shorter as compared with the 6-bolt specimen case (i.e., 1.3 \( K_o \)). In order to resist the same amount of the design moment, transferred from the same column section, the total tensile bolt force must increase with the shorter moment arm. Unfortunately, this numerical observation could not be verified successfully in this experimental study due to premature connection failure in the two 6-bolt specimens.

In Fig. 14, the flat portion of the curve at higher drifts (after 3.0% drift) implies that the specimen reaches its capacity due to significant yielding in a weak link in the connection, instead of yielding in the anchor bolts. In the case of SP 4-1, for instance, column was the weak link and thus, a plastic hinge formation was observed at the bottom of the column at the ultimate state of the connection.

6.3 Strain variation in base plate

Strain variations on the surface of the base plate in the longitudinal direction (along the line “A” in Fig. 5) are plotted in Figs. 15 and 16 to investigate strain distribution in the base plate and to find location of the maximum base plate curvatures on both the tension and compression sides. Figure 15 presents the base plate strain variations in the two 6-bolt specimens (i.e., SP 6-1 and SP 6-2) as well as the FEA predictions up to 2.6% drift level, and the experimental outputs of SP 4-1 are compared with the FEA results up to 5.0% drift level in Fig. 16. In the above figures, +4.8 in. or –4.8 in. location in the base plate indicates the center of the assumed bending line, marked as “a” on the line “A” shown in Fig. 5. The strains at this point have been used to monitor yielding in the base plate along the assumed two bending lines; one on the tension side and one on the compression side.

One of basic design assumptions in the D&E method is that the base plate develops its full moment capacity at the ultimate state of the connection. It has been thus expected in this experimental study that major yielding in the base plate be observed along the assumed bending line either on the tension or on the compression side under large column cyclic displacements. Interestingly, however, no significant yielding was observed in the base plate even in SP 4-1 up to 5.0% drift. As shown in Figs. 15(b), 15(c), and 16(b), peak strains (measured at 2.6% drift) at the point “a" in SP 6-1 and SP 6-2 are within ±800 µε whereas peak strains (measured at 5.0% drift) at the same location are within ±1000 µε in SP 4-1. For comparison, the expected yield strain of grade 36 steel is 1240 µε. The above experimental results confirm one of major findings from the numerical analyses presented in the companion paper (Lee et al., 2008), which is published together: The D&E method results in stiff base
plates. These observations indicate that the exposed-type column-base plate connections designed by the D&E method may not behave as intended in the case of weak axis bending.

Based on comparison of the experimental outputs with the FEA predictions in Figs. 15 and 16, it has been found that the three-dimensional FEA model developed for the numerical parametric study is very useful to investigate strain distribution in the base plate and to find location of the maximum base plate curvature. The FEA analyses plotted in Figs. 15(a) and 16(a) indicate that maximum base plate strains occur near the point “a” on both the tension and compression sides in case of no column axial loads. Even though some of experimental outputs (e.g., on the tension side in SP 6-1 and on the compression side in SP 4-1) show minor inconsistencies at higher drifts, it seems to be reasonable to conclude that the assumed bending lines predict well the location of the maximum strain in the base plate. However, as noted in the companion paper (Lee et al., 2008), the above conclusion is less reliable in the case of 4-bolt connections due to the non-uniform base plate deformation in the transverse direction.

6.4. Effects of different number of anchor bolt

The numerical parametric study (Lee et al., 2008) has shown that different number (arrangement) of anchor bolt could significantly change the deformed shape and yielding pattern of the base plate on the tension side and bearing stress distribution on the compression side. In order to verify the above observation, deformed shape of the base plate at each drift level is plotted and compared in Figs. 17 and 18. Base plate deformations only on the

![Figure 15. Strain variation on the base plate surface along line A (6-bolt specimens).](image1)

![Figure 16. Strain variation on the base plate surface along line A (4-bolt specimen).](image2)
tension side are compared in these two figures. Test results of the two 6-bolt specimens (i.e., SP 6-1 and SP 6-2) show significantly different base plate deformations as compared with those of SP 4-1. The 6-bolt specimens, as presented in Figs. 17(a) and 17(b), develop flat or concave shape of the base plate due to restraint of the inner anchor bolts, whereas the base plate in SP 4-1 is clearly convex-shaped at higher drifts as presented in Fig. 18. Because of the out-of-plane base plate deformation, much complicated stress distributions are observed at the corner of the base plate and anchor bolts in SP 4-1. The convex-shaped base plate deformation in the 4-bolt connections can be easily transferred to the compression side, especially under no column axial loads. This can be clearly seen in Fig. 19. When coupled with reduced bearing area due to relatively large anchor bolt sizes, the convex shape of the base plate deformation in the 4-bolt connections can increase bearing stresses at the corner of the grout. Due to such high stress concentration, a corner of the grout in SP 4-1 was crushed during the first load cycle at 4.0% drift. Figure 11 presents this grout crushing at the left corner of the grout.

6.5. Effects of different filler metal and welding detail

Based on performance of the two 6-bolt specimens (i.e., SP 6-1 and SP 6-2), effects of the two different filler metals on cyclic connection ductility are examined. From Table 2, the only difference between SP 6-1 and SP 6-2 was the welding material. Column and base plate of SP 6-1 were connected using ER70S-3 (Lincoln SuperArc L-50 SuperGlide S3) electrode, whereas SP 6-2 was fabricated using E70TG-K2 (Lincoln Innershield NR-31Ni) electrode.

As already presented in Fig. 8(a), initial cracks in SP 6-1 were first visible at three of the four column flange tips during the first load cycle at 1.5% drift. Initial crack in SP 6-2, on the other hand, was observed earlier than the SP 6-1 case at the column flange “L2” tip, as indicated in Fig. 8(b). However, between these two specimens, no significant differences were observed in cyclic performance and crack propagation thereafter up to 3.0% drift. Further, these two specimens lost most of their strengths commonly during the second load cycle at 4.0% drift. Hence, based on this limited number of experiments, it
can be concluded that ER70S-3 and E70TG-K2 electrodes equally perform under the cyclic loads in the direction of weak axis bending when A572 Grade 50 column and A36 base plate combinations are chosen for the connection.

In order to investigate effects of the fillet weld reinforcement over the Partial Joint Penetration (PJP) groove welds, shown in Fig. 3, cyclic performance of the two 4-bolt specimens are compared. As indicated in Table 2, assuming effects from the two different filler metals are negligibly small, the major difference between SP 4-1 and SP 4-2 was existence of the fillet reinforcement outside the column flanges. The PJP groove welds in SP 4-1 was reinforced by 3/16 in. fillet welds, whereas only the minimum PJP groove welds were placed in SP 4-2 without the fillet reinforcement.

SP 4-1 and SP 4-2 showed a very different cyclic ductility in the connection. Due to lack of the fillet reinforcement, initial cracks in SP 4-2 were observed at three of the four column flange tips during the first load cycle at 1.0% drift, as indicated in Fig. 8(d). This specimen failed in a quite brittle manner at an early stage of the loading history. Figure 6(d) and Fig. 7 clearly present the poor performance of SP 4-2. SP 4-1, on the other hand, showed an excellent cyclic ductility in the connection through the entire loading history even though initial cracks in this specimen were first visible at the column flange “L2” tip during 1.5% drift cycles and at the column flange “R1” and “R2” tips during the 2.0% drift cycles. No significant strength degradation was observed up to 5.0% drift cycles and, as shown in Fig. 7, more than 0.03 radian of the total plastic rotation was measured in SP 4-1. The test results of the two 4-bolt specimens clearly indicate that the fillet reinforcement over the PJP groove welds should be added in case of the weak axis bending to develop a full moment capacity in the connection and to increase its cyclic ductility.

7. Observations and Conclusions

Four exposed-type column-base plate connection sub-assemblies (two for the 6-bolt connection and two for the 4-bolt connection) were fabricated using noitch-tough filler metals and cyclically tested in the direction of column weak axis under the SAC Phase II loading history (SAC, 1997). Only one of the four specimens (i.e., SP 4-1) completed the applied entire loading history without significant strength degradation and formed a plastic hinge at the bottom of the column. The other three specimens showed a limited ductility in the connection. Major findings from this experimental study are summarized in the following:

Unstiffened exposed-type column-base plate connection sub-assemblies showed a partial fixity in the connection in the linear response range under the applied loading history. In this range, lateral stiffness of the two 6-bolt specimens (i.e., SP 6-1 and SP 6-2) commonly showed 66% of the elastic stiffness of the cantilever while SP 4-1 developed a mildly lower lateral stiffness, i.e., 57% of the elastic stiffness of the cantilever.

Through the numerical parametric study presented in the companion paper (Lee et al., 2008), it has been shown that location of the resultant bearing force (Ru) as well as its amount varies with respect to the change of the anchor bolt stiffness for a given column element and base plate thickness. Stiffer anchor bolt increases amount of the total tensile bolt force (Tu) as well as amount of Ru due to a shortened overall moment arm between these two. The finite element analyses presented in Fig. 14 thus show a higher amount of Tu in the case of stiffer anchor bolt in the 4-bolt specimen (i.e., 2.2 Ku). Unfortunately, however, the above numerical observation could not be verified successfully in this experimental study because of early connection failure in the 6-bolt specimens, which consisted of a relatively smaller anchor bolt stiffness (i.e., 1.3 Ku).

One of basic design assumptions in the D&E method is that the base plate develops its full moment capacity at the ultimate state of the connection. Interestingly, no significant base plate yielding has been observed through the applied entire loading history in this experimental study. Even in SP 4-1, only small yielded areas were observed on the surface of the base plate near the column flange tips up to 5.0% drift cycles. This indicates that the exposed-type column-base plate connections designed by the D&E method may not behave as intended in the case of weak axis bending.

Despite the excellent energy dissipation capacity shown in SP 4-1, two potential problems have been noted for the 4-bolt connection design; complicated base plate yielding pattern on the tension side and undesirable concrete crushing at the corner of the grout on the compression side. In order to solve these problems and to develop more rational and reliable design methods, further study on the 4-bolt connections should be conducted. In particular, effects of column axial loads on force flow and stress distribution in the connection should be included in this study.

Based on comparison of the cyclic connection ductility between SP 4-1 and SP 4-2, it is recommended in this study to add the fillet reinforcement over the partial joint penetration groove welds outside the column flanges when the welding detail presented in Fig. 3 is used to connect the column to the base plate. Required size of the fillet reinforcement can be determined in proportioning the fillet welds of the tested specimen in this experimental study.

The two 6-bolt specimens (i.e., SP 6-1 and SP 6-2) showed a limited cyclic ductility in the connection at higher drifts even though the noitch-tough filler metals, satisfying the general requirements within Section 7.3 of the 2005 AISC Seismic Provisions (AISC, 2005), were used to fabricate the connections. In order to improve seismic performance of the connection, more ductile welding detail and/or stiffler detail that can mitigate high local stress concentration at the column flange tips should be provided.
References


