Appendix A: Contribution of Transverse Rods to the Lateral Stiffness of the Plate Assembly

A.1: Introduction

The effective inertia and hence the computed lateral stiffness of the interconnected dual-plates varies significantly when the rod diameter is varied. Because the moment of inertia of a given tie rod varies according to the fourth power of the diameter, the effective inertia of tie rods can vary greatly as the diameter is varied. A 1-in. diameter rod has been used successfully to interconnect dual-plates for other patented construction applications that utilize friction stir-welding to join the rods with the plates (Bowerman and Chapman, 2002). Because a three-eighths scale model was used in the experimental program of this research project, it was decided that a 0.25-in. to 0.75-in. diameter rod would be needed to effectively connect the two plates of the reduced-scale model if full fixity of the rod-to-plate connection is achieved.

A series of reduced-scale (3/8-scale) tests to investigate the contribution of the transverse connecting rods to the lateral stiffness of the plate assembly was performed. The plate assembly was 9-in. thick, 13.5-in. wide, and 22.5-in. high. The spacing between the transverse bars was 4.5 in. in both orthogonal directions. Transverse connecting rods (fully threaded rods) of ¼-in., ½-in., and ¾-in. diameters were considered while keeping all the other parts of the test set-up the same. Finite element analyses were performed for the three cases, as well as for a 3/8-in. diameter case. The analysis results were compared with the test results. Based upon the information gathered regarding the performance of the tie rods used in the reduced-scale tests, the tie rod diameter to be used in the three-eighths scale specimens was selected.
A.2: Experimental Investigation

Lateral load was applied to three plate assemblies with the 0.25, 0.5, and 0.75-in. diameter transverse rods in the direction perpendicular to the plate assembly. Lateral displacement and strains in the plates and rods were measured to calculate the lateral stiffness of each plate assembly and to study how load was transferred in the plate assemblies.

A.2.1: Test Set-up

A drawing and a photograph of the test set-up are shown in Figs. A.1 and A.2, respectively. Transverse rods were spaced at 4.5 in. (which is a scaled spacing comparable to the 12 in. spacing in the prototype). Hinges were used to attach the loading plate (13.5 in. x 15.5 in. x 1 in.) to the 3/16-in. plates. The 3/16-in. plates were made of grade A36 steel. Hinges helped to transfer the horizontal load to the plate assembly specimen without also transferring significant moments. The transverse rods were attached to the 3/16-in. plates using four nuts per each bar—two inside the plate assembly and two outside. These nuts were tightened using a typical wrench; neither a cheater bar nor an impact wrench was used for tightening the nuts. Listed minimum tensile strength of the transverse bar was 53 ksi. However, coupon tests of similar rods tested later showed the tensile strength to be 70 ksi. The 3/16-in. plates were bolted at the bottom to 5 in. x 5 in. x 3/4 in. single angles by turn-of-the-nut method (1/3 rotation). These 5 in. x 5 in. x 3/4 in. angles were tightly bolted (by turn of the nut method) to a base plate which was anchored to the strong floor using high-strength PT bars that were post-tensioned (see Figs. A.1 and A.2). The load was applied by rotating a turnbuckle attached to the loading plate on the top of the plate assembly.
A.2.2. Instrumentation

Lateral loads, lateral deflections, and strains in the plates and transverse connecting rods were measured. The outer side of the 3/16-in. plates was whitewashed to detect any evidence of yielding in the plates. A 5-kip load cell, attached between the turnbuckle and loading plate (see Fig. A.2), was used to measure applied lateral load. A linear potentiometer was installed to measure lateral deflections (see Fig. A.2). Lateral load and lateral deflection measurements were then used to calculate the lateral stiffness of the plate assembly to assess the behavior and effectiveness of the tie rods to make the plates function together as an integral unit. Strain gauges were attached to the 3/16-in. plates to infer the second moment of area of the cross section of the plate assembly (from the pure bending formula, \( \sigma = \frac{M\cdot y}{I} \)). Strain gauges were also attached on top and bottom of the transverse rods to determine how much bending or axial force is transferred by the rods from one plate to the other. Strain gauges were attached to transverse bars only for the ½-in. diameter case. Strain gauge layouts for the test set-ups with 3/4-in., 1/2-in., and 1/4-in. diameter bars are shown in Figs. A.3, A.4, and A.5, respectively. Strain gauges which were installed, but did not function properly, are not shown in the figures.

A.2.3: Test Method

The reduced-scale specimen, which resembled a cantilever beam, was subjected to a lateral load at the top to induce bending of the interconnected 3/16-in. thick plates. Lateral load was applied by slowly rotating the turnbuckle being careful not to apply any impact load. The load cell, potentiometer, and strain gauges were monitored and recorded using a Vishay Micro Measurements data acquisition system (system 7000). Lateral load was applied to a certain load level (2163 lbs for ¾-in. diameter rod case, 1679 lbs for ½-in. diameter rod case, and 343 lbs for ¼-in. diameter rod case) so that at least a significant part of the linear curve was obtained. For the ½-in. and ¼-in. diameter cases, load, deflection, and strain measurements were recorded also for the unloading response. Load,
deflection, and strain measurements were zeroed before starting each test. During each test, whitewash on the steel plates was monitored for flaking to detect yielding of the steel plates.

A.2.4: Test Results

Lateral Load versus Lateral Deflection from Laboratory Tests

Lateral load versus lateral deflection curves are shown in Fig. A.6 for each of the three tests (3/4-in. dia. rod, ½-in. dia. rod, and ¼-in. dia. rod). Calculation of lateral stiffness from the initial linear portion of the curves using a linear trend line is shown in Fig. A.7. The lateral stiffnesses are tabulated in Table A.1. Upon unloading, permanent deformations of 0.92 in. and 0.44 in. were observed for the ½-in. and ¼-in. diameter cases, respectively (refer Fig. A.6).

Strain Measurements in Plates and Transverse Rods from Laboratory Test

There was no observation of flaking of the whitewash during the tests. Strains developed in the plates versus lateral load plots for the test set-ups with 3/4-in., 1/2-in., and 1/4-in. diameter bars are shown in Figs. A.8, A.9, and A.10, respectively. Strain readings were consistently compressive on the loading side of the plate and tensile on the opposite side, and decreased with distance from the base of each plate assembly suggesting cantilever behavior of the assemblies. Strains in transverse rods versus lateral load plots for the test set-up with 1/2-in. diameter bars are shown in Fig. A.11. The strain gauges on the transverse rods are shown in Fig. A.4. From Fig. A.11, it could be noted that strain gauges B2 and B3 measured compressive strains while strain gauge B1 measured tensile strains in the top level transverse rod. Similarly, strain gauges B5 and B6 measured compressive strains while strain gauges B4 and B7 measured tensile strains in the bottom level transverse rod. Hence, it could be concluded that the transverse rods
transferred significant bending from the plate on the loading side to the plate on the opposite side of the plate assembly.

**A.3: Finite Element Analysis**

Finite element analysis was performed to compare predicted response with the experimental results and to further expand the investigation to 3/8-in. diameter rods. Abaqus (Abaqus/CAE 6.10-1) was used to perform the finite element analysis for each of the four cases (1/4-in., 3/8-in., 1/2-in., and 3/4-in. diameter rods).

**A.3.1: Finite Element Modeling**

A finite element model of the specimen is shown in Fig. A.12. The 3/16-in. plates were modeled as shell elements (S4, a 4-node doubly curved general-purpose shell, finite membrane strains). The portion of the 3/16-in. plates which was clamped by the bolts and nuts to the ¾-in. angles at the base was assumed to be rigid and was therefore omitted.

Transverse bars were modeled as beam elements (B33, a 2-node cubic beam in space). The effective pitch diameter of the threaded transverse bars can be calculated using Equation A.1 (ASME B1.1, 2003). The calculated effective pitch diameters of the 1/4-in., 3/8-in., 1/2-in., and 3/4-in. diameter rods were used in the finite element modeling. The pitch and effective pitch diameters of the transverse bars are tabulated in Table A.2.

\[ D_p = D_{maj} - 0.64952 \times p \]  \hspace{1cm} \text{Eq. (A.1)}

Where,  

- \( D_p \) = Effective pitch diameter (in.)  
- \( D_{maj} \) = Major diameter (in.)  
- \( p \) = pitch (in.) = \( 1/ \) (number of threads/in.)
Material behavior for the steel plate was assumed to be elastic-perfectly plastic with $E = 29,000$ ksi, $\sigma_y = 36$ ksi, $\nu = 0.3$, and Mass density $= 0.00073386 \text{ lbf} \cdot \text{s}^{-2} \cdot \text{in}^4$. Material behavior for the transverse bars also was assumed to be elastic-perfectly plastic with $E = 29,000$ ksi, $\sigma_y = 70$ ksi, $\nu = 0.3$, and Mass density $= 0.00073386 \text{ lbf} \cdot \text{s}^{-2} \cdot \text{in}^4$.

The 1-in. thick loading plate was modeled as a solid element (C3D8R, an 8-node linear brick, reduced integration, hourglass control). Mesh parts were created and assembled. Connection between each transverse bar and 3/16-in. plates was modeled as a “tie” constraint with rotational degree of freedom restricted. The joint between the 3/16-in. plate and the loading plate was modeled as a “tie” constraint without rotational degree of freedom restricted.

A fixed boundary condition was considered at the bottom of the modeled specimen (at the top of the nuts). When the specimen is pulled (towards the left) by applying tension in the turnbuckle, the ¾-in. angle restricts the lateral movement of the 3/16-in. plate on the left side while the 3/16-in. plate on the right side is free to move laterally. Hence, the lateral movement was restricted ($U_3=0$) along the joint between the left side 3/16-in. plate and the 3/16-in. angle. Figure A.12 shows both boundary conditions.

Analysis was performed in two steps: (1) Static General and (2) Static Riks. In the first step, gravity loading (self weight) was considered. In the second step, lateral load was applied as a concentrated force at the location shown in Fig. A.12. Because all the sensors were zeroed at the start of the test (after gravity loading), the deflections and strains from the gravity loading of the finite element model were deducted from the final results of the finite element analysis that are reported in this report.
**A.3.2: Finite Element Analysis (FEA) Results**

*Lateral Load versus Lateral Deflection from Finite Element Analysis*

Lateral load versus lateral deflection curves from the finite element analyses are shown in Fig. A.13 for each of the four rod diameter cases (1/4-in., 3/8-in., 1/2-in., and 3/4-in. diameter rods). Calculation of lateral stiffness from the initial linear portion of the curves using a linear trend line is shown in Fig. A.14. The lateral stiffnesses are tabulated in Table A.3. Figure A.14 shows that the lateral stiffness of the plate assembly significantly increases with rod diameter.

**A.4: Results Analysis and Comparison**

The load-deformation curves for the specimens with 1/4-in., 1/2-in., and 3/4-in. diameter rods from the FE analyses and the tests are shown in Fig. A.15. The test results were in good agreement with the FEA results (except a small deviation for the ¾-in. diameter rod case, see Fig. A.15). Lateral stiffness versus effective pitch diameter of the transverse rods is plotted in Fig. A.16. It can be noted from Fig. A.16 that the lateral stiffness of the specimen increased significantly with the diameter of the rods.

It can be noted from Table A.1 and Table A.2 that the lateral stiffness of the dual-plate assembly with ¼-in. (2/3 in. in prototype) rods at 4.5-in. spacing (12 in. in prototype) is very small (193.2 lb/in from laboratory test and 256.3 lb/in from FEA) compared to the other specimens. Hence, the earlier consideration of using a ½-in. diameter rod in the prototype (which would have been 3/16 in. in the 3/8-scale lab models) was decided to be inadequate due to low lateral stiffness. Instead, a prototype with 1-in. diameter rods at 12-in. spacing (which provides 3/8-in. diameter bars at 4.5-in. spacing in the 3/8-scale lab models) was selected for the stability investigation to check whether the wall panels are
able to resist construction loads. Furthermore, in the patented production of the dual-plate assembly (Corus/TATA Bi-Steel), 1-in. diameter bars were used.
### Table A.1 Variation of lateral stiffness (from the initial linear portion of the curve) with transverse rod diameter- Laboratory test

<table>
<thead>
<tr>
<th>Rod Diameter (in.)</th>
<th>Lateral Stiffness – From Test (lb/in.)</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.25</td>
<td>193.2</td>
</tr>
<tr>
<td>0.50</td>
<td>1547.0</td>
</tr>
<tr>
<td>0.75</td>
<td>4450.1</td>
</tr>
</tbody>
</table>

### Table A.2 Effective pitch diameters of the transverse bars calculated for finite element analysis (using Equation A.1)

<table>
<thead>
<tr>
<th>Transverse bar Diameter, $D_{maj}$ (in.)</th>
<th>Threads/in. (1/in.)</th>
<th>$p$ = Pitch (in.)</th>
<th>Effective pitch diameter (in.)</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.25</td>
<td>20</td>
<td>0.050</td>
<td>0.218</td>
</tr>
<tr>
<td>0.375</td>
<td>16</td>
<td>0.062</td>
<td>0.334</td>
</tr>
<tr>
<td>0.50</td>
<td>13</td>
<td>0.077</td>
<td>0.450</td>
</tr>
<tr>
<td>0.75</td>
<td>10</td>
<td>0.100</td>
<td>0.685</td>
</tr>
</tbody>
</table>

### Table A.3 Variation of lateral stiffness (from the initial linear portion of the curve) with transverse rod diameter- Finite element analysis

<table>
<thead>
<tr>
<th>Rod Diameter (in.)</th>
<th>Lateral Stiffness – From FEM (lb/in.)</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.25</td>
<td>256</td>
</tr>
<tr>
<td>0.375</td>
<td>807</td>
</tr>
<tr>
<td>0.50</td>
<td>1780</td>
</tr>
<tr>
<td>0.75</td>
<td>3530</td>
</tr>
</tbody>
</table>
Figure A.1 Test set up for the investigation of transverse connecting rods (Front View)
Figure A.2 Test set-up for the investigation of transverse connecting rods

Figure A.3 Strain gauge layout for 3/4-in. bar diameter case (Test-1)
Figure A.4 Strain gauge layout for 1/2-in. bar diameter case (Test-2)

Figure A.5 Strain gauge layout for 1/4-in. bar diameter case (Test-3)
Figure A.6 Lateral load versus lateral deflection curves for 1/4, 1/2, & 3/4-in. rod diameters

Figure A.7 Lateral load versus lateral deflection for 3/4, 1/2, & 1/4-in. dia. Rod tests - lateral stiffness calculation
Figure A.8 Strain on plates versus lateral load-for specimen with 3/4-in. rod

Figure A.9 Strain on plates versus lateral load-for specimen with 1/2-in. rod
Figure A.10 Strain on plates versus lateral load—for specimen with 1/4-in. rod

Figure A.11 Strain on rods versus lateral load—for specimen with 1/2-in. rod
Figure A.12 Finite element model

Figure A.13 Lateral load versus lateral deflection-3/4, 1/2, 3/8 & 1/4-in. rod-FEM
Figure A.14 Lateral Load versus Lateral Deflection-3/4, 1/2, 3/8 & 1/4-in. Rod-FEA-Lateral Stiffness Calculation

Figure A.15 Lateral Load versus Lateral Deflection-3/4, 1/2, & 1/4-in. Rod
Figure A.16 Lateral stiffness of the plate assembly versus effective pitch diameter of the transverse rod— from laboratory test and finite element analysis
Appendix B: Investigation of Weld between the Reinforcing bar and the Foundation Connection Plate

B.1: Introduction

Tension force (developed from flexure) in each 3/16-in. shear wall plate has to be transferred to the foundation block first via the ¾-in. foundation connection plate and then via the no.7 reinforcing bars (see Fig. 3.16). The weld size and length of weld between the no.7 reinforcing bars and ¾-in. thick connection plate were determined to permit development of the yield strength of the no.7 bar. For stability test and cyclic loading test, the size and spacing of the no.7 bar were designed for yielding of the 3/16-in. shear wall plate and for tensile fracturing of the 3/16-in. shear wall plate, respectively.

AWS Specification D1.4 (AWS, 1998 and 2011) specifies the effective weld size to be 0.2*Bar diameters (refer to Fig. B.1) and the effective weld area to be the effective weld size multiplied by the weld length. Furthermore, AWS D1.4 (1998 and 2011) specifies an allowable weld stress of “0.3 times the nominal tensile strength of filler metal, except shear stress on the base metal shall not exceed 0.4 times the yield strength of the base metal for shear on the effective area”. For the purpose of LRFD design, this design approach may be modified to consider a weld stress of 0.6 times the nominal tensile strength of filler metal with applicable strength reduction factor, 0.75, and applicable load factors. Hence, the shear strength of the weld (on either side of the reinforcing bar) can be calculated by using Equation B.1.

\[ \phi R_n = 0.6 \times F_{E_{XX}} t_e L_{weld} \]  

Eq. (B.1)

where 
\( \phi \) = strength reduction factor (0.75 for shear) 
\( R_n \) = shear strength 
\( F_{E_{XX}} \) = electrode strength (ksi) 
\( t_e \) = effective weld size (0.2*Bar diameter) 
\( L_{weld} \) = weld length
AWS D1.4 (1998 and 2011) specifies the type of electrodes (E8015, E8016, and E8018 for the SMAW process) to be used to weld ASTM A706 reinforcing bars to plates. However, the welding procedure is not specified. As a result, a number of weld pass combinations were attempted and each cross section was examined to identify the proper combination that would provide the necessary weld throat. Then the shear strength of the weld made by that combination was determined via tension tests and compared with the shear strength calculated using Equation B.1.

**B.2: Test Set-up and Test Method**

Two weld passes were considered for the weld between the ¾-in. foundation connection plate and the no.7 reinforcing bar. A 1/8-in. E8018 electrode was used for the first pass, while a 5/32-in. or a 3/16-in. E8018 electrode was used for the second pass. The DC voltage (on the Miller welding machine) was set to 30V. A number of samples were made with different Amperages to investigate the optimal welding procedure. After welding, the samples were cut and etched with a 2% Nital Solution. An example of the etched sample is shown in Fig. B.2. Figure B.2 (a) shows a sample weld made with a 1/8-in. electrode root pass and a 5/32-in. electrode used for the second pass. A good weld (with minimum included discontinuities and minimal burning of the reinforcing bar and the plate) was obtained with 110 amps for the 1/8-in. electrode and 140 amps for the 5/32-in. electrode. Figure B.2(b) shows a weld specimen made with a 1/8-in. electrode root pass and a 3/16-in. electrode for the second pass. A good weld (with minimum defects) was obtained with 110 amps for the 1/8-in. electrode and 160 amps for the 3/16-in. electrode. From the observations and measurements of the weld sizes of the specimens shown in Figs. B.2(a) and B.2(b), it was determined that a first weld pass on each side of the bar using a 1/8-in. electrode followed by a second pass using a 3/16-in. electrode produced the necessary weld throat. Although the procedure was not a formal welding procedure specification (WPS), it was adequate to demonstrate that the welds were suitable.
To demonstrate the structural performance, two specimens like those shown in Figs. B.3 and B.4 with different weld lengths ($L_w$ in Fig. B.3) produced with E8018 electrodes were tested in a universal test machine to determine the weld length needed to develop the yield strength of the no.7 reinforcing bars. The first specimen had an average weld length of 1.81 in. and the second specimen had an average weld length of 1.00 in.

### B.3 Results and Conclusions

In the first specimen, the yielding of the reinforcing bar initiated at 73.2 kips (61 ksi), and in the second specimen failure of the weld initiated along the weld line at 56.2 kips. Hence, it was determined that a 1.31-in. weld length ($= 1.00 \text{ in.} \times \frac{73.2 \text{ kips}}{56.2 \text{ kips}}$) was needed to yield no. 7 bars used in the tension tests. The weld length calculated from Equation B.1, considering an 80 ksi electrode, excluding the strength reduction factor, and considering welds on both sides of the reinforcing bar, was 2.15 in. The shear strength to be resisted ($R_n$) was calculated by multiplying the yield strength of 60 ksi by the area of the no. 7 reinforcing bar. The lower weld length determined from the tension tests compared with that calculated from Equation B.1 may be primarily attributed to the weld size larger than 0.2*Bar diameter (as shown in Fig. B.1). Differences in deposited weld sizes between the two specimens also can be a reason for the lower weld length calculated from the tension tests.

For the experimental program a 2-in. weld length (a value between 1.31 in. and 2.15 in. determined from the tension tests and calculated using Equation B.1, respectively) was used to attach the hooked reinforcing bars to the connection plate for fabrication of the anchorage hardware in the base block. It is recommended to make the weld with two passes. For the first pass, a 1/8-in. electrode with Amperage of 110 and a deposition speed of 4.5 in./min. can be used. For the second pass, a 3/16-in. electrode with Amperage of 160 and a deposition speed of 5.6 in./min. can be used.
Figure B.1 Flare bevel groove weld (from AWS D1.4/D1.4M, 1998 and 2011)

Figure B.2 Etched weld samples: (a) Two weld passes with 1/8-in. electrode followed by 5/32-in. electrode and (b) Two weld passes with 1/8-in. electrode followed by 3/16-in. electrode
Figure B.3 Tensile test set-up to evaluate reinforcing bar - plate weld strength – Front view

Figure B.4 Tensile test set-up to evaluate reinforcing bar - plate weld strength – Cross Section
Appendix C: Investigation of Weld between Shear Wall Plate and Foundation Connection Plate

C.1: Introduction

The forces from the 3/16-in. shear wall plate have to be transferred to the ¾-in. foundation connection plate via the weld between the shear wall plate and the foundation connection plate (see Fig. 3.16). Even though the size of the weld was designed considering the static forces and static material properties for the in-plane cyclic loading test (which is detailed in Chapter 6), the behavior of the weld for cyclic loading had to be investigated. The purpose of this test was to verify that the failure will occur in the steel plates rather than in the weld.

In the study of “the behavior of double-skin composite walls for in-plane cyclic loading” by Eom et al. (2009), the shear wall plates were connected to the base plate using complete joint penetration groove welds. For the applied in-plane cyclic loading, the welded joint failed in tensile fracture before the wall plate reached flexural yielding (at 1.5% drift ratio). Motivated by this study, two types of welded connection were considered for the joint between 3/16-in. thick specimen plates and the ¾-in. thick foundation connection plates. Both were tested under tensile cyclic loading to select the necessary and sufficient weld connection.

C.2: Test Set-up and Test Method

For the two welded connections proposed for cyclic testing, the first welded connection option involved one load path, while the second involved two load paths. Details of the two options are outlined below.
(1) A 7/16-in. fillet weld was used between the 3/16-in. plate and ¾-in. plate as shown in Fig. C.1. The entire vertical tensile load from the 3/16-in. thick shear wall plate was intended to be transferred to the ¾-in. thick foundation connection plate via only a 7/16-in. fillet weld. The design philosophy was to size the weld so that it could develop the tensile capacity of the 3/16-in. plate.

(2) A 3/8-in. fillet weld and a bevel groove weld were used in addition to the 7/16-in. fillet weld as shown in Fig. C.2. The vertical tensile load from the 3/16-in. thick shear wall plate was intended to be transferred to the ¾-in. thick foundation connection plate via two load paths: (a) part of the load would be transferred from the 3/16-in. plate through the 7/16-in. fillet weld to the bevel groove weld, and (b) the remaining load would be transferred through the 3/8-in. fillet weld. Weld joint 2 was more time consuming and much more expensive to fabricate than weld joint 1 but it was expected to be stronger.

It was decided to test these two weld joints subjected to cyclic tensile loads. Cyclic compression was not introduced to avoid buckling of the specimen. Schematic diagrams of the two weld tests are shown in Fig. C.3. Specimens were made symmetric by combining with a mirror image of the actual weld joint so there would be no out-of-plane bending in the specimen during axial cyclic loading.

**Preparation of weld test specimens**

Innershield NR-232 flux-cored self shielded (FCAW-S) wires (0.068 in size) from Lincoln Electric were used to make the weld joint specimens. Prior to specimen fabrication, welding samples were prepared to check the penetration of the weld passes. Voltage and amperage were adjusted for each pass to obtain adequate weld penetration. Voltage, amperage, and deposit speed of each weld pass were recorded.
Progress made in the production of weld joint 2 is shown in the photos in Fig. C.4. Weld sample 1 had one pass for the 3/8-in. fillet weld and six passes for the bevel groove weld. The voltage ranged between 21.2 and 22.8 V, and the amperage ranged between 230 and 267 amps. It can be seen from Fig. C.4 (a) that the weld sample didn’t have sufficient penetration in the first 3/8-in. fillet weld and the first three passes of the bevel groove weld didn’t have adequate weld fusion. For sample 2 (Fig. C.4 (b)), the voltage and amperage were increased to 25.2 V and 300 amps (average), respectively, for the first 3/8-in. fillet weld and the first three passes of the bevel groove weld. Still, there was a large weld discontinuity in the first 3/8-in. fillet weld. Lastly, sample 3 (Fig. C.4 (c)) was made to check the penetration of the first 3/8-in. fillet weld and the first pass of the bevel groove weld. The voltage and amperage were increased to 25.7 V and 345 amps (average), respectively. The weld sample (made with only two passes) was satisfactory from observation. Furthermore, it was decided to back gouge (by grinding) the weld metal between the passes upon detecting any defect.

For the weld joint 1 specimen (Fig. C.3 (a)), the 7/16-in. weld was made with three passes. For the first pass, the voltage and amperage were 25.7V and 360 amps (average), respectively. For the other two passes, the voltage and amperage were 21.5V and 235 amps (average), respectively. After the first pass the weld metal was back gouged. The average measured weld size of the 7/16-in. fillet weld was 0.456 in. For the weld joint 2 specimen (Fig. C.3 (b)), the 7/16-in. weld was made with three passes. For the first pass, the voltage and amperage were 25.7V and 345 amps (average), respectively. For the other two passes, the voltage and amperage were 21.5V and 235 amps (average), respectively. After the first pass the weld metal was back gouged. The average measured weld size of the 7/16-in. fillet weld was 0.464 in.

The first 3/8-in. fillet weld in the weld joint 2 specimen was made with one pass. The voltage and amperage were 25.7V and 340 amps, respectively. The bevel groove weld was made with seven passes on one side and nine passes on the other side. The number of passes was dependent on the amount of back gouging. The voltage and
amperage for the first three passes of the bevel groove weld were 25.7V and 340 amps (average), respectively. The voltage and amperage for the fourth pass were 25.2V and 275 amps, respectively. The voltage and amperage for the remaining passes were 21.5V and 250 amps (average), respectively.

**Testing Weld Joint Specimens**

The yield strain ($\varepsilon_y$) of the 3/8-in. plate material was calculated from the two coupons tested in tension. The stress-strain curve for a coupon specimen is shown in Fig. C.5. The average yield strain was 0.002070 (2070 µε). The weld joint specimen was first loaded until the strain reached $2\varepsilon_y$ then was unloaded until the load was zero. The specimen was then loaded until the strain reached $3\varepsilon_y$ and was subsequently unloaded till the load was zero. Similarly, the specimen was subjected to repetitions of loading to develop strain multiples of $\varepsilon_y$ ($4\varepsilon_y$, $5\varepsilon_y$ …) until failure occurred.

Weld joint 1 specimen was subjected to 67 loading cycles before failure: One cycle of $2\varepsilon_y$ increment, another 19 cycles of $\varepsilon_y$ increment (for each cycle), another 10 cycles of $2\varepsilon_y$ increment, another 10 cycles of $3\varepsilon_y$ increment, another 10 cycles of $4\varepsilon_y$ increment, and the remaining 16 cycles of $5\varepsilon_y$ increment. The weld joint 2 test specimen was subjected to 50 loading cycles before failure: One cycle of $2\varepsilon_y$ increment, another 9 cycles of $\varepsilon_y$ increment, another 10 cycles of $3\varepsilon_y$ increment, and the remaining 30 cycles of $5\varepsilon_y$ increment.

The test specimens were configured with the test weld at one end and a bolted anchor connection at the opposite end. The bolted connection involved four high-strength bolts with a shear capacity in excess of the capacity of the welded connection at the test end of the specimen.

The test set-up for the weld joint specimens is shown in Fig. C.6. The tests were performed in a four pole MTS servo-hydraulic testing machine. The strain was measured
using a clip gage that was attached to the specimen, as can be seen at the top of Fig. C.6. The controller was programmed to apply cycles of loading automatically.

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C.3: Results and Conclusions

The stress-strain response curves for weld joint specimens 1 and 2 are shown in Figs. C.7 and C.8, respectively. Figure C.9 shows the failure of a weld joint specimen. Failure occurred by fracturing the 3/8-in. thick plate above the 7/16-in. fillet weld in both specimens. The 3/8-in. thick plates stretched up to 0.394 and 0.395 in/in strains in specimens 1 and 2, respectively (see Fig. C.7 and C.8). No indication of failure was observed in the welds in either of the specimens. Both welds sustained load until the plate element failed. However, during the weld joint tests, the specimens were not subjected to compression loading. The compression part of the loading cycle was avoided so that the specimen would not fail due to buckling of the 3/8-in. thick plate. But in the intersecting wall element test, portions of the plate will be subjected to tension and compression as the specimen is pushed and pulled in the two loading directions.

Because both weld joints did not fail during the cyclic loading tests and because weld joint 1 is considerably less expensive than weld joint 2, weld joint 1 was selected for the joint between the 3/16-in. shear wall plates and the ¾-in. foundation connection plates in the specimen for the cyclic loading test.
Figure C.1 Weld Joint 1 - A 7/16-in. Fillet weld between wall specimen plates and the foundation connection plates

Figure C.2 Weld Joint 2 - A 3/8-in. fillet weld and bevel groove weld in addition to the 7/16-in. fillet weld
Figure C.3 Weld Joints (a) Weld Joint 1 specimen (b) Weld Joint 2 specimen

Figure C.4 Weld samples (a) Weld sample 1 (b) Weld sample 2 (c) Weld sample 3
Figure C.5 Stress versus Strain Curve –Static Loading– Coupon Specimen 1 and 2

Figure C.6 Weld Joint Specimen Tests - Cyclic Loading in Four Pole MTS Testing Machine (Specimen 1)
Figure C.7 Stress versus Strain Curve –Cyclic Loading– Weld Joint 1 Specimen

Figure C.8 Stress versus Strain Curve –Cyclic Loading– Weld Joint 2 Specimen
Figure C.9 Failure of the Weld Joint Specimen (Specimen 2)