STEEL STRENGTH OF ANCHOR BOLTS IN STAND-OFF BASE PLATE CONNECTIONS

By

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For Grandpa, Granny, Louise, Bonnie, and Kevin
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# TABLE OF CONTENTS

<table>
<thead>
<tr>
<th>Section</th>
<th>Page</th>
</tr>
</thead>
<tbody>
<tr>
<td>ACKNOWLEDGMENTS</td>
<td>4</td>
</tr>
<tr>
<td>LIST OF TABLES</td>
<td>12</td>
</tr>
<tr>
<td>LIST OF FIGURES</td>
<td>13</td>
</tr>
<tr>
<td>ABSTRACT</td>
<td>29</td>
</tr>
<tr>
<td>CHAPTER</td>
<td></td>
</tr>
<tr>
<td>1 BACKGROUND</td>
<td>31</td>
</tr>
<tr>
<td>1.1 Problem Statement</td>
<td>31</td>
</tr>
<tr>
<td>1.2 Components in Stand-off Base Plate Connections</td>
<td>33</td>
</tr>
<tr>
<td>1.3 Load Transfer through Anchored Connections to Concrete</td>
<td>37</td>
</tr>
<tr>
<td>1.4 Load Transfer through Anchor Bolts</td>
<td>39</td>
</tr>
<tr>
<td>1.5 Objectives</td>
<td>40</td>
</tr>
<tr>
<td>1.6 Dissertation Organization</td>
<td>41</td>
</tr>
<tr>
<td>2 LITERATURE REVIEW</td>
<td>43</td>
</tr>
<tr>
<td>2.1 Introduction</td>
<td>43</td>
</tr>
<tr>
<td>2.2 Background</td>
<td>43</td>
</tr>
<tr>
<td>2.2.1 Steel Material Characteristics and Behavior</td>
<td>43</td>
</tr>
<tr>
<td>2.2.2 Steel Failure Criteria</td>
<td>44</td>
</tr>
<tr>
<td>2.2.3 Cross-sectional Plastic Limits</td>
<td>50</td>
</tr>
<tr>
<td>2.2.3.1 Normal forces</td>
<td>52</td>
</tr>
<tr>
<td>2.2.3.2 Bending moments</td>
<td>53</td>
</tr>
<tr>
<td>2.2.3.3 Shear forces</td>
<td>54</td>
</tr>
<tr>
<td>2.2.3.4 Combined bending moment and normal force</td>
<td>56</td>
</tr>
<tr>
<td>2.2.3.5 Combined normal force and shear force</td>
<td>58</td>
</tr>
<tr>
<td>2.2.3.6 Combined bending moment and shear force</td>
<td>59</td>
</tr>
<tr>
<td>2.2.3.7 Combined bending moment, normal force, and shear force</td>
<td>64</td>
</tr>
<tr>
<td>2.2.4 Plastic Hinge Behavior</td>
<td>66</td>
</tr>
<tr>
<td>2.3 Anchor Bolt Steel Strength</td>
<td>68</td>
</tr>
<tr>
<td>2.3.1 Normal Strength Capacity of Threaded Rods and Anchor Bolts</td>
<td>68</td>
</tr>
<tr>
<td>2.3.2 Bolt Steel Strength in Flush-mounted Base Plates and Steel-to-steel Connections</td>
<td>70</td>
</tr>
<tr>
<td>2.3.2.1 Shear strength</td>
<td>70</td>
</tr>
<tr>
<td>2.3.2.2 Combined normal and shear forces</td>
<td>73</td>
</tr>
<tr>
<td>2.3.3 Bolt Steel Strength in Ungrouted Stand-off Base Plate Connections</td>
<td>77</td>
</tr>
<tr>
<td>2.3.3.1 Pure shear force</td>
<td>77</td>
</tr>
<tr>
<td>2.3.3.2 Combined normal and shear force</td>
<td>83</td>
</tr>
<tr>
<td>2.3.3.3 Summary and open areas of research in ungrouted stand-off base plates</td>
<td>86</td>
</tr>
<tr>
<td>Section</td>
<td>Page</td>
</tr>
<tr>
<td>------------------------------------------------------------------------</td>
<td>------</td>
</tr>
<tr>
<td>2.3.4 Bolt Steel Strength in Grouted Stand-off Base Plate Connections</td>
<td>87</td>
</tr>
<tr>
<td>2.3.4.1 Combined normal and shear force</td>
<td>87</td>
</tr>
<tr>
<td>2.3.4.2 Summary and open areas of research in grouted stand-off base plate connections</td>
<td>92</td>
</tr>
<tr>
<td>2.4 Influence of Other Connection Conditions</td>
<td>93</td>
</tr>
<tr>
<td>2.4.1 Base Plate Rigidity</td>
<td>93</td>
</tr>
<tr>
<td>2.4.2 Connection Pretensioning</td>
<td>94</td>
</tr>
<tr>
<td>2.4.3 Influence of Grout on Stand-off Base Plate Connection Behavior</td>
<td>94</td>
</tr>
<tr>
<td>2.5 Summary</td>
<td>95</td>
</tr>
<tr>
<td>3 CODE COMPARISONS</td>
<td>97</td>
</tr>
<tr>
<td>3.1 Introduction</td>
<td>97</td>
</tr>
<tr>
<td>3.2 Steel Strength of Anchor Bolts in Tension</td>
<td>97</td>
</tr>
<tr>
<td>3.3 Steel Strength of Anchor Bolts in Shear</td>
<td>99</td>
</tr>
<tr>
<td>3.4 Steel Strength of Anchor Bolts in Combined Tension and Shear</td>
<td>101</td>
</tr>
<tr>
<td>3.5 Provisions for Steel Strength in Stand-off Base Plates</td>
<td>102</td>
</tr>
<tr>
<td>3.6 Hole Sizes</td>
<td>103</td>
</tr>
<tr>
<td>3.7 Summary</td>
<td>104</td>
</tr>
<tr>
<td>4 DEVELOPMENT OF EXPERIMENTAL PROGRAM</td>
<td>105</td>
</tr>
<tr>
<td>4.1 Introduction</td>
<td>105</td>
</tr>
<tr>
<td>4.2 Materials and Methods</td>
<td>107</td>
</tr>
<tr>
<td>4.2.1 Bolt Specimens</td>
<td>107</td>
</tr>
<tr>
<td>4.2.2 Concrete Blocks</td>
<td>107</td>
</tr>
<tr>
<td>4.2.3 Grout</td>
<td>109</td>
</tr>
<tr>
<td>4.2.4 Instrumentation and Data Acquisition</td>
<td>112</td>
</tr>
<tr>
<td>4.3 Direct Shear Testing</td>
<td>113</td>
</tr>
<tr>
<td>4.3.1 Experimental Design</td>
<td>113</td>
</tr>
<tr>
<td>4.3.2 Test Setup</td>
<td>114</td>
</tr>
<tr>
<td>4.3.2.1 Structural components</td>
<td>114</td>
</tr>
<tr>
<td>4.3.2.2 Instrumentation</td>
<td>116</td>
</tr>
<tr>
<td>4.3.3 Test Preparation and Procedure</td>
<td>117</td>
</tr>
<tr>
<td>4.4 Torsion Testing of Scaled Anchor Bolt Groups</td>
<td>118</td>
</tr>
<tr>
<td>4.4.1 Experimental Design</td>
<td>119</td>
</tr>
<tr>
<td>4.4.2 Test Setup</td>
<td>120</td>
</tr>
<tr>
<td>4.4.2.1 Structural components</td>
<td>120</td>
</tr>
<tr>
<td>4.4.2.2 Instrumentation</td>
<td>125</td>
</tr>
<tr>
<td>4.4.3 Tests Requiring Special Treatment</td>
<td>127</td>
</tr>
<tr>
<td>4.4.3.1 T10: Fiber-reinforced polymer (FRP) wrap around grouted stand-off base plate</td>
<td>127</td>
</tr>
<tr>
<td>4.4.3.2 T26: Absence of leveling nuts in grouted stand-off base plate connection</td>
<td>128</td>
</tr>
<tr>
<td>4.4.3.3 T27: Shim stacks supporting base plate</td>
<td>129</td>
</tr>
<tr>
<td>4.4.3.4 T28: Dry-packed mortar around bolt circle</td>
<td>129</td>
</tr>
<tr>
<td>4.4.4 Test Preparation and Procedure</td>
<td>130</td>
</tr>
</tbody>
</table>
4.4.4.1 Stage 1: initial setup ...................................................................................130
4.4.4.2 Stage 2: final instrumentation placement and bolt pre-loading ..........131
4.4.4.3 Stage 3: loading ..........................................................................................131
4.4.5 Treatment of Experimental Data ..........................................................................132
4.5 Large-scale Anchor Bolt Groups under Predominantly Torsion Loading ..........136
4.5.1 Experimental Design ............................................................................................136
4.5.2 Test Setup .............................................................................................................136
4.5.2.1 Structural components ................................................................................136
4.5.2.2 Instrumentation ...........................................................................................139
4.5.3 Test Preparation and Procedure ............................................................................140
4.5.4 Treatment of Experimental Data ..........................................................................141
4.6 Eccentric Shear Loading of Rectangular Bolt Groups ...................................................143
4.6.1 Experimental Design ............................................................................................143
4.6.2 Test Setup .............................................................................................................145
4.6.2.1 Structural components ................................................................................145
4.6.2.2 Instrumentation ...........................................................................................149
4.6.3 Tests Requiring Special Treatment ......................................................................150
4.6.3.1 ES21, ES22, ES31, ES32: four-bolt tests ...................................................151
4.6.3.2 ES30: effect of friction in grouted installation ...........................................151
4.6.4 Test Preparation and Procedure ............................................................................152
4.6.5 Treatment of Experimental Data ..........................................................................153
4.6.5.1 Ungrouted tests ...........................................................................................153
4.6.5.2 Grouted tests ...............................................................................................155
4.7 Summary .........................................................................................................................157
5 EXPERIMENTAL RESULTS .............................................................................................158
5.1 Introduction .....................................................................................................................158
5.2 Ductility of Bolt Materials ............................................................................................158
5.3 Direct Shear Testing .......................................................................................................162
5.3.1 Influence of Test Geometry on Bolt Tension Forces ...........................................162
5.3.2 Summary of Results .............................................................................................164
5.4 Torsion Testing of Scaled Anchor Bolt Groups .............................................................167
5.4.1 Summary of Results .............................................................................................167
5.4.2 Load-displacement Behavior of Ungrouted Tests ................................................168
5.4.2.1 Size effect ...................................................................................................169
5.4.2.2 Effect of pretensioning .................................................................................170
5.4.2.3 Effect of concrete strength .........................................................................171
5.4.2.4 Effect of bolt material ................................................................................172
5.4.2.5 Comparisons between Torsion and Direct Shear testing .........................174
5.4.3 Load-displacement Behavior of Grouted Tests ....................................................175
5.4.3.1 Effect of leveling nuts in a grouted installation .........................................178
5.4.3.2 Effect of dry-packed low-strength mortar around the bolt line in place of a nonshrink grout pad ..................................................179
5.4.3.3 Effect of shim stacks in place of a grout pad ..............................................180
5.4.3.4 Effect of an FRP wrap around a grout pad ...............................................181
5.5 Large-scale Anchor Bolt Groups under Predominantly Torsion Loading ..........183
### 5.6 Eccentric Shear Loading of Rectangular Bolt Groups

- **5.6.1 Summary of Results**
- **5.6.2 Load-displacement Behavior of Ungrouted Two-bolt Eccentric Shear Tests**
  - **5.6.2.1 Effect of exposed length at different eccentricities**
  - **5.6.2.2 Effect of pretensioning**
  - **5.6.2.3 Effect of bolt ductility**
  - **5.6.2.4 Illustration of total load vs. displacement**
- **5.6.3 Load-displacement Behavior of Grouted Tests**
  - **5.6.3.1 Effect of exposed length**
  - **5.6.3.2 Effect of grout friction**
- **5.6.4 Load-displacement Behavior of Four-bolt Tests**

### 5.7 Summary

### 6 ANALYSIS AND DISCUSSION

- **6.1 Introduction**
- **6.2 Statistical Analysis of Direct Shear Tests**
- **6.3 Development of Predictive Models for Anchor Bolt Steel Strength**
  - **6.3.1 Empirical Determination of Plastic Section Modulus**
  - **6.3.2 Interactions between Bending Moments, Shear Forces, and Normal Forces**
    - **6.3.2.1 Lower-bound interaction**
    - **6.3.2.2 Upper limit interaction**
  - **6.3.3 Prediction of Anchor Bolt Steel Strength in Ungrouted Stand-off Base Plates**
    - **6.3.3.1 First-order equilibrium of applied normal and shear forces**
    - **6.3.3.2 Second-order equilibrium of applied normal and shear forces**
    - **6.3.3.3 Proposed analytical model**
    - **6.3.3.4 Model sensitivity to variable assumptions**
  - **6.3.4 Prediction of Anchor Bolt Steel Strength in Stand-off Base Plates with Grout and other Stiff Intermediate Layers**
    - **6.3.4.1 Proposed analytical method**
    - **6.3.4.2 Model sensitivity to variable assumptions**
- **6.4 Model and Code Comparisons to Experimental Results**
  - **6.4.1 Ungrouted Tests**
  - **6.4.2 Grouted Tests**
- **6.5 Effect of Minor Test Variables**
- **6.6 Application of Analytical Models to Multiple-bolt Connections**

### 7 SUMMARY, CONCLUSIONS, AND RECOMMENDATIONS

- **7.1 Dissertation Summary**
- **7.2 Conclusions**
- **7.3 Design Recommendations**
  - **7.3.1 Recommendations for Steel Strength Design of Anchor Bolts in Ungrouted Stand-off Base Plate Connections**
  - **7.3.2 Recommendations for Steel Strength Design of Anchor Bolts in Grouted Stand-off Base Plate Connections**
7.3.2.1 Simple method ...........................................................................................................266
7.3.2.2 Detailed method ........................................................................................................267
7.3.3 General recommendations for steel strength design of anchor bolts in standoff base plates ...............................................................................................270
7.4 Recommendations for Future Work .............................................................................270

APPENDIX

A LOAD-DISPLACEMENT BEHAVIOR OF DIRECT SHEAR TESTS ..................................271
B LOAD-DISPLACEMENT BEHAVIOR OF TORSION TESTS ..................................................278
C LOAD-DISPLACEMENT BEHAVIOR OF LARGE-SCALE TESTS .......................................321
D LOAD-DISPLACEMENT BEHAVIOR OF ECCENTRIC SHEAR TESTS .............................331
E DERIVATIONS OF INTERACTIONS BETWEEN NORMAL FORCE, SHEAR FORCE, AND BENDING MOMENT ON CIRCULAR CROSS-SECTIONS ...............................364
   E.1 Introduction ..................................................................................................................364
   E.2 Interaction of Normal Force and Bending Moment ......................................................364
   E.3 Interaction of Normal and Shear Forces .....................................................................368
   E.4 Interaction of Bending and Shear Forces ..................................................................369
   E.5 Three-dimensional Axial/Shear/Bending Interaction ..................................................371

REFERENCES ........................................................................................................................373

BIOGRAPHICAL SKETCH ....................................................................................................378
<table>
<thead>
<tr>
<th>Table</th>
<th>Description</th>
<th>Page</th>
</tr>
</thead>
<tbody>
<tr>
<td>1-1</td>
<td>Tension and shear failure modes defined by ACI (2011).</td>
<td>41</td>
</tr>
<tr>
<td>2-1</td>
<td>Threaded rod section properties.</td>
<td>69</td>
</tr>
<tr>
<td>3-1</td>
<td>AASHTO (2013a), ACI (2011), and AISC (2010) bolt tension strengths.</td>
<td>98</td>
</tr>
<tr>
<td>3-2</td>
<td>AASHTO (2013a), ACI (2011), and AISC (2010) bolt shear strengths.</td>
<td>100</td>
</tr>
<tr>
<td>3-3</td>
<td>Comparison of AASHTO (2013a,b), AISC (2010), and FDOT hole sizes.</td>
<td>104</td>
</tr>
<tr>
<td>4-1</td>
<td>Direct Shear test matrix.</td>
<td>113</td>
</tr>
<tr>
<td>4-2</td>
<td>Torsion test matrix.</td>
<td>119</td>
</tr>
<tr>
<td>4-3</td>
<td>Summary of scaling results for 5/8 in. diameter Torsion bolt specimens.</td>
<td>121</td>
</tr>
<tr>
<td>4-4</td>
<td>Large-scale test matrix.</td>
<td>136</td>
</tr>
<tr>
<td>4-5</td>
<td>Eccentric shear test matrix.</td>
<td>144</td>
</tr>
<tr>
<td>5-1</td>
<td>Summary of Direct Shear anchor bolt ultimate load and displacement results.</td>
<td>162</td>
</tr>
<tr>
<td>5-2</td>
<td>Summary of Direct Shear anchor bolt ultimate load and displacement results.</td>
<td>165</td>
</tr>
<tr>
<td>5-3</td>
<td>Summary of torsion testing anchor bolt ultimate load and displacement results.</td>
<td>168</td>
</tr>
<tr>
<td>5-4</td>
<td>Summary of Large-scale anchor bolt ultimate load and displacement results.</td>
<td>183</td>
</tr>
<tr>
<td>5-5</td>
<td>Summary of eccentric shear ultimate load and displacement results.</td>
<td>186</td>
</tr>
<tr>
<td>6-1</td>
<td>Analysis of variance between selected Direct Shear datasets.</td>
<td>205</td>
</tr>
<tr>
<td>6-2</td>
<td>Evaluation of statistical differences between selected Direct Shear datasets.</td>
<td>206</td>
</tr>
<tr>
<td>6-3</td>
<td>Comparisons between F1554 bolt data and the proposed ungrouted analytical model.</td>
<td>245</td>
</tr>
<tr>
<td>6-4</td>
<td>Comparisons between 354BD and stainless steel bolt data and the proposed ungrouted analytical model.</td>
<td>246</td>
</tr>
<tr>
<td>6-5</td>
<td>Comparisons between experimental results for grouted stand-off base plates, rational model, ACI (2011) interpretation, and Gresnigt et al. (2008) models</td>
<td>252</td>
</tr>
<tr>
<td>B-1</td>
<td>Vertical base plate displacements in Torsion tests.</td>
<td>279</td>
</tr>
<tr>
<td>C-1</td>
<td>Vertical and horizontal base plate displacements (in.) in Large-scale tests.</td>
<td>322</td>
</tr>
<tr>
<td>Figure</td>
<td>Description</td>
<td>Page</td>
</tr>
<tr>
<td>--------</td>
<td>---------------------------------------------------------------------------------------------------</td>
<td>------</td>
</tr>
<tr>
<td>1-1</td>
<td>Definition of components illustrated for an ungrouted annular stand-off base plate.</td>
<td>33</td>
</tr>
<tr>
<td>1-2</td>
<td>Schematic illustrations of base plate connection types.</td>
<td>34</td>
</tr>
<tr>
<td>1-3</td>
<td>Field installation of flush-mounted base plate.</td>
<td>35</td>
</tr>
<tr>
<td>1-4</td>
<td>Field installation of ungrouted stand-off base plate.</td>
<td>36</td>
</tr>
<tr>
<td>1-5</td>
<td>Field installation of grouted stand-off base plate.</td>
<td>36</td>
</tr>
<tr>
<td>1-6</td>
<td>Transfer of base plate forces into individual shear forces on anchor bolts in a cantilever sign structure with an ungrouted stand-off base plate.</td>
<td>38</td>
</tr>
<tr>
<td>1-7</td>
<td>Schematic illustration of shear force transfer through double-nut ungrouted stand-off base plate connection.</td>
<td>39</td>
</tr>
<tr>
<td>1-8</td>
<td>Schematic illustration of shear force transfer through double-nut grouted stand-off base plate connection.</td>
<td>40</td>
</tr>
<tr>
<td>2-1</td>
<td>State of stress on a differential three-dimensional element.</td>
<td>45</td>
</tr>
<tr>
<td>2-2</td>
<td>Three-dimensional von Mises failure criterion in Haigh-Westergaard principal stress space.</td>
<td>49</td>
</tr>
<tr>
<td>2-3</td>
<td>Two-dimensional von Mises yield criterion in principal stress space.</td>
<td>49</td>
</tr>
<tr>
<td>2-4</td>
<td>Cross-sectional spread of plasticity due to pure bending moments.</td>
<td>54</td>
</tr>
<tr>
<td>2-5</td>
<td>Assumed spread of cross-sectional plasticity due to shear force based on strength observations of steel bolts.</td>
<td>55</td>
</tr>
<tr>
<td>2-6</td>
<td>Fully plastic combination of normal and shear stresses on cross-section.</td>
<td>56</td>
</tr>
<tr>
<td>2-7</td>
<td>Equivalent distribution of bending and normal stresses on fully plasticized cross-section.</td>
<td>57</td>
</tr>
<tr>
<td>2-8</td>
<td>Combined effects of normal stress and assumed fully plastic shear stress.</td>
<td>59</td>
</tr>
<tr>
<td>2-9</td>
<td>Combined effects of semi-plastic bending and the elastic limit of shear stress attributed to Bezukhov (1936).</td>
<td>61</td>
</tr>
<tr>
<td>2-10</td>
<td>Fully plastic combined bending and shear stress distribution attributed to Palchevsky (1948).</td>
<td>62</td>
</tr>
<tr>
<td>Page</td>
<td>Description</td>
<td></td>
</tr>
<tr>
<td>------</td>
<td>-------------</td>
<td></td>
</tr>
<tr>
<td>2-11</td>
<td>Development of plastic hinge region in a doubly moment-restrained beam subjected to shear force.</td>
<td></td>
</tr>
<tr>
<td>2-12</td>
<td>Shear force acting on a flush-mounted base plate.</td>
<td></td>
</tr>
<tr>
<td>2-13</td>
<td>Illustration of steel shear failure of an anchor bolt in a flush-mounted base plate.</td>
<td></td>
</tr>
<tr>
<td>2-14</td>
<td>Combined normal and shear forces acting on a flush-mounted base plate.</td>
<td></td>
</tr>
<tr>
<td>2-16</td>
<td>Shear force acting on an ungrouted stand-off base plate.</td>
<td></td>
</tr>
<tr>
<td>2-17</td>
<td>Simplified illustration of direct shear test setup used by Scheer et al. (1987).</td>
<td></td>
</tr>
<tr>
<td>2-18</td>
<td>Dimensions used in Eligehausen et al. (2006b) for anchor bolt bending stresses.</td>
<td></td>
</tr>
<tr>
<td>2-19</td>
<td>Simplified illustration of the test schematic used in Lin et al. (2011).</td>
<td></td>
</tr>
<tr>
<td>2-20</td>
<td>Combined normal and shear forces acting on an ungrouted stand-off base plate.</td>
<td></td>
</tr>
<tr>
<td>2-21</td>
<td>Simplified illustration of combined tension and shear test setup used by Scheer et al. (1987).</td>
<td></td>
</tr>
<tr>
<td>2-22</td>
<td>Combined shear and normal forces acting on a grouted stand-off base plate.</td>
<td></td>
</tr>
<tr>
<td>2-23</td>
<td>Anchor bolts after 14 weeks of wetting cycles.</td>
<td></td>
</tr>
<tr>
<td>4-1</td>
<td>Schematic of formwork used for casting anchor bolt specimens upside-down in Direct Shear, Torsion, and Eccentric Shear blocks.</td>
<td></td>
</tr>
<tr>
<td>4-2</td>
<td>Wet-curing process used for Direct Shear, Torsion, and Eccentric Shear blocks.</td>
<td></td>
</tr>
<tr>
<td>4-3</td>
<td>Grout installation procedure.</td>
<td></td>
</tr>
<tr>
<td>4-4</td>
<td>Grout pad and 2 in. cubes after wet-wrapping.</td>
<td></td>
</tr>
<tr>
<td>4-5</td>
<td>Labeled schematic of Direct Shear test setup.</td>
<td></td>
</tr>
<tr>
<td>4-6</td>
<td>Connection details for Direct Shear tests.</td>
<td></td>
</tr>
<tr>
<td>4-7</td>
<td>Inside of Direct Shear form before concrete placement.</td>
<td></td>
</tr>
<tr>
<td>4-8</td>
<td>Instrumentation in Direct Shear testing.</td>
<td></td>
</tr>
<tr>
<td>4-9</td>
<td>Torsion test setup.</td>
<td></td>
</tr>
<tr>
<td>4-10</td>
<td>Labeled schematic of Torsion test setup.</td>
<td></td>
</tr>
</tbody>
</table>
4-36 Two-bolt compression and shear test. .................................................................150
4-37 Four-bolt test. .......................................................................................................151
4-38 Eccentric shear test setup. ...................................................................................152
4-39 Profile schematic of displaced two-bolt tension and shear Eccentric Shear test. ....153
4-40 Free-body illustrations of Eccentric Shear tests.......................................................154
5-1 Schematic of ductility tensile test setup.................................................................159
5-2 Applied load vs. crosshead displacement of tensile test specimens for ductility. ....160
5-3 Calculated true stress vs. extension of ductility tensile specimens............................161
5-4 Schematic of exaggerated displaced shape of single-bolt Direct Shear specimens......163
5-5 Schematic of shear and unintended normal forces in double-bolt Direct Shear tests...164
5-6 Examples of specimens used to measure $l_b$. ........................................................166
5-7 Load-displacement behavior of representative Direct Shear tests............................166
5-8 Load-displacement behavior comparing 5/8 in. and 1 in. dia. ungrouted Torsion tests...169
5-9 Failed Torsion anchor bolts with ungrouted $2d_b$ base plates. .........................170
5-10 Load-displacement behavior of comparable ungrouted Torsion tests with normal-strength and low-strength concrete. .............................................................171
5-11 Load-displacement behavior of comparable ungrouted Torsion tests with F1554, 354BD, and Type 316 stainless steel materials. .......................................................173
5-12 T24 (stainless steel Torsion test) after anchor bolt failure. ........................................174
5-13 Comparisons between ungrouted Direct Shear and Torsion tests. .........................174
5-14 Failed grouted Torsion specimens. .............................................................................176
5-15 Load-displacement behavior of grouted six-bolt Torsion tests...............................177
5-16 Load-displacement behavior of grouted (and shimmed) three-bolt Torsion tests........177
5-17 Three-bolt grouted Torsion tests after anchor bolt failure. .......................................179
5-18 T27 (no grout, shimmed Torsion test) after anchor bolt failure. .............................181
5-19 Plan view of T10 (FRP-retrofitted $2.8d_b$ exposed length) after bolt failure. ........182
5-20 Load-displacement behavior of Large-scale tests..........................................................184
5-21 LS1 after anchor bolt failure. .........................................................................................184
5-22 Failed Large-scale grouted specimens. ........................................................................185
5-23 Representative ungrouted two-bolt tension/shear Eccentric Shear specimen after anchor bolt failure. .................................................................187
5-24 Representative ungrouted two-bolt compression/shear Eccentric Shear specimen after anchor bolt failure. .................................................................187
5-25 Load-displacement behavior of ungrouted 6 in. eccentricity two-bolt ES tests. .........189
5-26 Load-displacement behavior of ungrouted 12 in. eccentricity two-bolt ES tests. .........189
5-27 Load-displacement behavior of ungrouted 18 in. eccentricity two-bolt ES tests. .............190
5-28 Load-displacement behavior of ungrouted 24 in. eccentricity two-bolt ES tests. ..........190
5-29 Load-displacement comparisons between equivalent ungrouted pretensioned and non-pretensioned two-bolt tension/shear ES tests. .................................................................191
5-30 Load-displacement comparisons between equivalent ungrouted F1554 and Type 316 stainless steel two-bolt ES tests. ..................................................................................192
5-31 Approximate “total load” of selected ungrouted Eccentric Shear tests. .................193
5-32 Representative grouted two-bolt tension/shear Eccentric Shear specimen after anchor bolt failure. .................................................................194
5-33 Representative grouted two-bolt compression/shear Eccentric Shear specimen after anchor bolt failure. .................................................................194
5-34 Load-displacement behavior of grouted 6 in. eccentricity two-bolt ES tests. ..............195
5-35 Load-displacement behavior of grouted two-bolt 24 in. eccentricity ES tests. .............196
5-36 ES (24 in. eccentricity compression/shear grouted test with Teflon layers) after anchor bolt failure. .................................................................................197
5-37 Load-displacement behavior of four-bolt ES tests.......................................................198
5-38 ES21 (6 in. eccentricity ungrouted four-bolt ES test) after anchor bolt failure. ..........199
5-39 ES22 (24 in. eccentricity ungrouted four-bolt ES test) after anchor bolt failure. ..........199
5-40 Test ES31 (6 in. eccentricity four-bolt grouted ES test) after bolt failure. A) Side view of cracked grout. B) Top view of cracked grout. ........................................200
6-24 Effect of friction on predicted grouted resistance to applied shear. ................................243
6-25 Effect of bolt rotation on predicted grouted resistance to applied shear..................243
6-26 Summary of pure shear results against predictive models........................................247
6-27 Ungrounded results normalized by first-order predicted values as a function of exposed length..................................................................................................................248
6-28 Ungrounded results normalized by first-order predicted values as a function of \( \rho v \) .............................................248
6-29 Ungrounded results normalized by second-order predictions as a function of exposed length.................................................................................................................................249
6-30 Ungrounded results normalized by second-order prediction as a function of \( \rho v \). ........249
6-31 Grouted results normalized by predictions as a function of \( \rho v \).................................252
6-32 Grouted results normalized by predictions as a function of exposed length................253
7-1 Anchor bolt in ungrounded stand-off base plate connection with applied normal and shear force.........................................................................................................................264
7-2 Anchor bolt in grouted stand-off base plate connection with applied normal and shear force.................................................................................................................................266
A-1 DS1 (flush-mounted base plate, 5/8 in. bolts, n = 1) applied load and bolt tension vs. displacement ..............................................................................................................................272
A-2 DS2 (ungrounded 1.2 dia. stand-off base plate, 5/8 in. bolts, n = 1) applied load and bolt tension vs. displacement ..............................................................................................................................272
A-3 DS3 (ungrounded 1.6 dia. stand-off base plate, 5/8 in. bolts, n = 1) applied load and bolt tension vs. displacement ..............................................................................................................................273
A-4 DS4 (ungrounded 2 dia. stand-off base plate, 5/8 in. bolts, n = 1) applied load and bolt tension vs. displacement ..............................................................................................................................273
A-5 DS6 (ungrounded 4 dia. stand-off base plate, 5/8 in. bolts, n = 1) applied load and bolt tension vs. displacement ..............................................................................................................................274
A-6 DS7 (flush-mounted base plate, 5/8 in. bolts, n = 2) instrumentation schematic ............274
A-7 DS8 (ungrounded 2 dia. stand-off base plate, 5/8 in. bolts, n = 2) applied load and bolt tension vs. displacement ..............................................................................................................................275
A-8 DS9, DS10, and DS11 (flush-mounted, ungrounded 2 dia. stand-off base plate, and 4 dia. ungrounded stand-off base plate, 1 in. bolts, n = 1) applied load and bolt tension vs. displacement ..............................................................................................................................275
A-9 DS8 (flush-mounted base plate, 5/8 in. bolts, n = 1, adhesive installation) applied load vs. displacement with comparable datasets

A-10 DS13 (ungrouting 2 dia. stand-off base plate, 5/8 in. bolts, n = 1, adhesive installation) applied load vs. displacement with comparable datasets

A-11 DS14 (ungrouting 2 dia. stand-off base plate, 5/8 in. bolts, n = 1, adhesive installation) with DS6 CIP applied load vs. displacement

B-1 T1 (flush-mounted base plate, 5/8 in. bolts, n = 6) instrumentation schematic

B-2 T1 individual actuator loads vs. bolt displacement

B-3 T1 bolt tensions vs. bolt displacement

B-4 T1 average applied load vs. vertical displacements

B-5 T2 (flush-mounted base plate, 5/8 in. bolts, n = 6) instrumentation schematic

B-6 T2 individual actuator loads vs. bolt displacement

B-7 T2 bolt tensions vs. bolt displacement

B-8 T2 average applied load vs. vertical displacements

B-9 T3 (ungrouting 2 dia. stand-off base plate, 5/8 in. bolts, n = 6) instrumentation schematic

B-10 T3 individual actuator loads vs. bolt displacement

B-11 T3 bolt tensions vs. bolt displacement

B-12 T4 (ungrouting 2 dia. stand-off base plate, 5/8 in. bolts, n = 6) instrumentation schematic

B-13 T2-B individual actuator loads vs. bolt displacement

B-14 T4 bolt tensions vs. bolt displacement

B-15 T4 average applied load vs. vertical displacements

B-16 T6 (ungrouting 4 dia. stand-off base plate, 5/8 in. bolts, n = 6) instrumentation schematic

B-17 T6 individual actuator loads vs. bolt displacement

B-18 T6 bolt tensions vs. bolt displacement

B-19 T6 average applied load vs. vertical displacements
B-20  T5 (ungrouted 2 dia. stand-off base plate, 5/8 in. bolts, n = 6, no pretension) 
instrumentation schematic ...............................................................................................290

B-21  T5 individual actuator loads vs. bolt displacement .........................................................290

B-22  T5 bolt tensions vs. bolt displacement .............................................................................291

B-23  T5 average applied load vs. vertical displacements .........................................................291

B-24  T7 (grouted 2 dia. stand-off base plate, 5/8 in. bolts, n = 6) instrumentation schematic ..........................................................292

B-25  T7 individual actuator loads vs. bolt displacement ..........................................................292

B-26  T7 bolt tensions vs. bolt displacement .............................................................................293

B-27  T7 average applied load vs. vertical displacements .........................................................293

B-28  T8 (grouted 4 dia. stand-off base plate, 5/8 in. bolts, n = 6) instrumentation schematic ..........................................................294

B-29  T8 individual actuator loads vs. bolt displacement ..........................................................294

B-30  T8 bolt tensions vs. bolt displacement .............................................................................295

B-31  T8 average applied load vs. vertical displacements .........................................................295

B-32  T9 (grouted 4 dia. stand-off base plate, 5/8 in. bolts, n = 6) instrumentation schematic ..........................................................296

B-33  T9 individual actuator loads vs. bolt displacement ..........................................................296

B-34  T9 bolt tensions vs. bolt displacement .............................................................................297

B-35  T9 average applied load vs. vertical displacements .........................................................297

B-36  T10 (grouted 4 dia. stand-off base plate with FRP wrap, 5/8 in. bolts, n = 6) 
instrumentation schematic ...............................................................................................298

B-37  T10 individual actuator loads vs. bolt displacement ..........................................................298

B-38  T10 bolt tensions vs. bolt displacement .............................................................................299

B-39  T10 average applied load vs. vertical displacements .........................................................299

B-40  T11 (flush-mounted base plate, 1 in. bolts, n = 3) instrumentation schematic ................300

B-41  T11 individual actuator loads vs. bolt displacement ..........................................................300
<table>
<thead>
<tr>
<th>Page</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>B-42</td>
<td>T11 bolt tensions vs. bolt displacement</td>
</tr>
<tr>
<td>B-43</td>
<td>T11 average applied load vs. vertical displacements</td>
</tr>
<tr>
<td>B-44</td>
<td>T12 (ungrounded 2 dia. stand-off base plate, 1 in. bolts, n = 3) instrumentation schematic</td>
</tr>
<tr>
<td>B-45</td>
<td>T12 individual actuator loads vs. bolt displacement</td>
</tr>
<tr>
<td>B-46</td>
<td>T12 bolt tensions vs. bolt displacement</td>
</tr>
<tr>
<td>B-47</td>
<td>T12 average applied load vs. vertical displacements</td>
</tr>
<tr>
<td>B-48</td>
<td>T13 (ungrounded 4 dia. stand-off base plate, 1 in. bolts, n = 3) instrumentation schematic</td>
</tr>
<tr>
<td>B-49</td>
<td>T13 individual actuator loads vs. bolt displacement</td>
</tr>
<tr>
<td>B-50</td>
<td>T13 bolt tensions vs. bolt displacement</td>
</tr>
<tr>
<td>B-51</td>
<td>T13 average applied load vs. vertical displacements</td>
</tr>
<tr>
<td>B-52</td>
<td>T14 (ungrounded, 0 dia. exposed length, 0.625 in. F1554 Gr. 55 bolts, n = 3) schematic</td>
</tr>
<tr>
<td>B-53</td>
<td>T14 individual actuator loads vs. bolt displacement</td>
</tr>
<tr>
<td>B-54</td>
<td>T15 (ungrounded, 0.5 dia. exposed length, 0.625 in. F1554 Gr. 55 bolts, n = 3) schematic</td>
</tr>
<tr>
<td>B-55</td>
<td>T15 individual actuator loads vs. bolt displacement</td>
</tr>
<tr>
<td>B-56</td>
<td>T16 (ungrounded, 2.8 dia. exposed length, 0.625 in. F1554 Gr. 55 bolts, n = 3) schematic</td>
</tr>
<tr>
<td>B-57</td>
<td>T16 individual actuator loads vs. bolt displacement</td>
</tr>
<tr>
<td>B-58</td>
<td>T17 (ungrounded, 6 dia. exposed length, 0.625 in. F1554 Gr. 55 bolts, n = 3) schematic</td>
</tr>
<tr>
<td>B-59</td>
<td>T17 individual actuator loads vs. bolt displacement</td>
</tr>
<tr>
<td>B-60</td>
<td>T18 (low-strength concrete, ungrouted, 1 dia. exposed length, 0.625 in. F1554 Gr. 55 bolts, n = 3) schematic</td>
</tr>
<tr>
<td>B-61</td>
<td>T18 individual actuator loads vs. bolt displacement</td>
</tr>
<tr>
<td>B-62</td>
<td>T19 (low-strength concrete, ungrouted, 1 dia. exposed length, 0.625 in. F1554 Gr. 55 bolts, n = 3) schematic</td>
</tr>
<tr>
<td>Page</td>
<td>Description</td>
</tr>
<tr>
<td>------</td>
<td>-------------</td>
</tr>
<tr>
<td>B-63</td>
<td>T19 individual actuator loads vs. bolt displacement ........................................................311</td>
</tr>
<tr>
<td>B-64</td>
<td>T20 (low-strength concrete, ungrouted, 3 dia. exposed length, 0.625 in. F1554 Gr. 55 bolts, n = 3) schematic ........................................................312</td>
</tr>
<tr>
<td>B-65</td>
<td>T20 individual actuator loads vs. bolt displacement..........................................................312</td>
</tr>
<tr>
<td>B-66</td>
<td>T21 (low-strength concrete, ungrouted, 6 dia. exposed length, 0.625 in. F1554 Gr. 55 bolts, n = 3) schematic ..........................................................313</td>
</tr>
<tr>
<td>B-67</td>
<td>T21 individual actuator loads vs. bolt displacement..........................................................313</td>
</tr>
<tr>
<td>B-68</td>
<td>T22 (ungrouted, 1 dia. exposed length, 0.625 in. 354 BD bolts, n = 3) schematic..............314</td>
</tr>
<tr>
<td>B-69</td>
<td>T22 individual actuator loads vs. bolt displacement..........................................................314</td>
</tr>
<tr>
<td>B-70</td>
<td>T23 (ungrouted, 3 dia. exposed length, 0.625 in. 354 BD bolts, n = 3) schematic..............315</td>
</tr>
<tr>
<td>B-71</td>
<td>T23 individual actuator loads vs. bolt displacement..........................................................315</td>
</tr>
<tr>
<td>B-72</td>
<td>T24 (ungrouted, 1 dia. exposed length, 0.625 in. 316 stainless bolts, n = 3) schematic........316</td>
</tr>
<tr>
<td>B-73</td>
<td>T24 individual actuator loads vs. bolt displacement..........................................................316</td>
</tr>
<tr>
<td>B-74</td>
<td>T25 (grouted, 3 dia. exposed length, 0.625 in. F1554 Gr. 55 bolts, n = 3) schematic........317</td>
</tr>
<tr>
<td>B-75</td>
<td>T25 individual actuator loads vs. bolt displacement..........................................................317</td>
</tr>
<tr>
<td>B-76</td>
<td>T26 (grouted without leveling nuts, 3 dia. exposed length, 0.625 in. F1554 Gr. 55 bolts, n = 3) schematic ..........................................................318</td>
</tr>
<tr>
<td>B-77</td>
<td>T26 individual actuator loads vs. bolt displacement..........................................................318</td>
</tr>
<tr>
<td>B-78</td>
<td>T27 (shimmed without grout, 3.6 dia. exposed length, 0.625 in. F1554 Gr. 55 bolts, n = 3) schematic ..........................................................319</td>
</tr>
<tr>
<td>B-79</td>
<td>T27 individual actuator loads vs. bolt displacement..........................................................319</td>
</tr>
<tr>
<td>B-80</td>
<td>T28 (ungrouted with dry-packed mortar, 4.2 dia. exposed length, 0.625 in. F1554 Gr. 55 bolts, n = 3) schematic ..........................................................320</td>
</tr>
<tr>
<td>B-81</td>
<td>T28 individual actuator loads vs. bolt displacement..........................................................320</td>
</tr>
<tr>
<td>C-1</td>
<td>LS1 (ungrouted 2.3 dia. stand-off base plate, 1.25 in. bolts, n = 6) instrumentation schematic ..........................................................323</td>
</tr>
<tr>
<td>C-2</td>
<td>LS1 actuator applied load vs. average bolt displacement.....................................................323</td>
</tr>
<tr>
<td>C-3</td>
<td>LS1 bolt tensions vs. bolt displacement.................................................................................324</td>
</tr>
</tbody>
</table>
C-4 LS1 applied load vs. vertical and horizontal displacements ............................................324
C-5 LS2 (grouted 2.3 dia. stand-off base plate, 1.25 in. bolts, n = 6) instrumentation schematic ..........................................................325
C-6 LS2 individual actuator loads vs. average bolt displacement ............................................325
C-7 LS2 bolt tensions vs. bolt displacement ...........................................................................326
C-8 LS2 applied load vs. vertical and horizontal displacements ............................................326
C-9 LS3 (grouted 4.3 dia. stand-off base plate, 1.25 in. bolts, n = 6) instrumentation schematic ...........................................................................327
C-10 LS3 actuator load vs. average bolt displacement .............................................................327
C-11 LS3 bolt tensions vs. bolt displacement ...........................................................................328
C-12 LS3 applied load vs. vertical and horizontal displacements ............................................328
C-13 LS4 (grouted 4.3 dia. stand-off base plate, 1.25 in. bolts, n = 6) instrumentation schematic ...........................................................................329
C-14 LS4 actuator load vs. average bolt displacement .............................................................329
C-15 LS4 bolt tensions vs. bolt displacement ...........................................................................330
C-16 LS4 applied load vs. vertical and horizontal displacements ............................................330
D-1 ES1 (ungrouted, 0 dia. exposed length, 0.625 in. F1554 Gr. 55 bolts, n = 2, tension/shear, 6 in. nominal eccentricity, not pretensioned) test schematic ..........................................................332
D-2 ES1 actuator load vs. average bolt displacement .............................................................332
D-3 ES2 (ungrouted, 1 dia. exposed length, 0.625 in. F1554 Gr. 55 bolts, n = 2, tension/shear, 6 in. nominal eccentricity, not pretensioned) test schematic ..........................................................333
D-4 ES2 actuator load vs. average bolt displacement .............................................................333
D-5 ES3 (ungrouted, 1 dia. exposed length, 0.625 in. F1554 Gr. 55 bolts, n = 2, tension/shear, 6 in. nominal eccentricity, pretensioned) test schematic ..........................................................334
D-6 ES3 actuator load vs. average bolt displacement .............................................................334
D-7 ES4 (ungrouted, 1 dia. exposed length, 0.625 in. F1554 Gr. 55 bolts, n = 2, tension/shear, 6 in. nominal eccentricity, pretensioned) test schematic ..........................................................335
D-8 ES4 actuator load vs. average bolt displacement .............................................................335
D-9  ES5 (ungrouted, 3 dia. exposed length, 0.625 in. F1554 Gr. 55 bolts, n = 2, tension/shear, 6 in. nominal eccentricity, not pretensioned) test schematic .................336
D-10 ES5 actuator load vs. average bolt displacement.................................................................336
D-11 ES6 (ungrouted, 3 dia. exposed length, 0.625 in. F1554 Gr. 55 bolts, n = 2, tension/shear, 12 in. nominal eccentricity, pretensioned) test schematic ......................337
D-12 ES6 actuator load vs. average bolt displacement.................................................................337
D-13 ES7 (ungrouted, 1 dia. exposed length, 0.625 in. F1554 Gr. 55 bolts, n = 2, tension/shear, 18 in. nominal eccentricity, pretensioned) test schematic ......................338
D-14 ES7 actuator load vs. average bolt displacement.................................................................338
D-15 ES8 (ungrouted, 3 dia. exposed length, 0.625 in. F1554 Gr. 55 bolts, n = 2, tension/shear, 18 in. nominal eccentricity, pretensioned) test schematic ......................339
D-16 ES8 actuator load vs. average bolt displacement.................................................................339
D-17 ES9 (ungrouted, 1 dia. exposed length, 0.625 in. F1554 Gr. 55 bolts, n = 2, tension/shear, 24 in. nominal eccentricity, not pretensioned) test schematic .................340
D-18 ES9 actuator load vs. average bolt displacement.................................................................340
D-19 ES10 (ungrouted, 1 dia. exposed length, 0.625 in. F1554 Gr. 55 bolts, n = 2, tension/shear, 24 in. nominal eccentricity, pretensioned) test schematic ......................341
D-20 ES10 actuator load vs. average bolt displacement.................................................................341
D-21 ES11 (ungrouted, 3 dia. exposed length, 0.625 in. F1554 Gr. 55 bolts, n = 2, tension/shear, 24 in. nominal eccentricity, not pretensioned) test schematic .................342
D-22 ES11 actuator load vs. average bolt displacement.................................................................342
D-23 ES12 (ungrouted, 1 dia. exposed length, 0.625 in. 316 Stainless Steel bolts, n = 2, tension/shear, 24 in. nominal eccentricity, pretensioned) test schematic ......................343
D-24 ES12 actuator load vs. average bolt displacement.................................................................343
D-25 ES13 (ungrouted, 1 dia. exposed length, 0.625 in. F1554 Gr. 55 bolts, n = 2, compression/shear, 6 in. nominal eccentricity, not pretensioned) test schematic ...............344
D-26 ES13 actuator load vs. average bolt displacement.................................................................344
D-27 ES14 (ungrouted, 3 dia. exposed length, 0.625 in. F1554 Gr. 55 bolts, n = 2, compression/shear, 6 in. nominal eccentricity, not pretensioned) test schematic ...............345
D-28 ES14 actuator load vs. average bolt displacement.................................................................345
D-29  ES15 (ungrounded, 3 dia. exposed length, 0.625 in. F1554 Gr. 55 bolts, n = 2, compression /shear, 12 in. nominal eccentricity, pretensioned) test schematic ...............346

D-30  ES15 actuator load vs. average bolt displacement ........................................................................346

D-31  ES16 (ungrounded, 3 dia. exposed length, 0.625 in. F1554 Gr. 55 bolts, n = 2, compression /shear, 18 in. nominal eccentricity, pretensioned) test schematic ...............347

D-32  ES16 actuator load vs. average bolt displacement ........................................................................347

D-33  ES17 (ungrounded, 1 dia. exposed length, 0.625 in. F1554 Gr. 55 bolts, n = 2, compression /shear, 24 in. nominal eccentricity, not pretensioned) test schematic ........348

D-34  ES17 actuator load vs. average bolt displacement (note: error in data acquisition. This space is intentionally left blank.) .............................................................................348

D-35  ES18 (ungrounded, 1 dia. exposed length, 0.625 in. F1554 Gr. 55 bolts, n = 2, compression /shear, 24 in. nominal eccentricity, pretensioned) test schematic ......................349

D-36  ES18 actuator load vs. average bolt displacement ........................................................................349

D-37  ES19 (ungrounded, 3 dia. exposed length, 0.625 in. F1554 Gr. 55 bolts, n = 2, compression /shear, 24 in. nominal eccentricity, not pretensioned) test schematic ........350

D-38  ES19 actuator load vs. average bolt displacement ........................................................................350

D-39  ES20 (ungrounded, 3 dia. exposed length, 0.625 in. F1554 Gr. 55 bolts, n = 2, compression /shear, 24 in. nominal eccentricity, not pretensioned) test schematic ........351

D-40  ES20 actuator load vs. average bolt displacement ........................................................................351

D-41  ES21 (ungrounded, 3 dia. exposed length, 0.625 in. F1554 Gr. 55 bolts, n = 4, 6 in. nominal eccentricity, pretensioned) test schematic .........................................................352

D-42  ES21 actuator load vs. average bolt displacement ........................................................................352

D-43  ES22 (ungrounded, 3 dia. exposed length, 0.625 in. F1554 Gr. 55 bolts, n = 4, 24 in. nominal eccentricity, pretensioned) test schematic .........................................................353

D-44  ES22 actuator load vs. average bolt displacement ........................................................................353

D-45  ES23 (grouted, 0 dia. exposed length, 0.625 in. F1554 Gr. 55 bolts, n = 2, tension/shear, 6 in. nominal eccentricity, pretensioned) test schematic ..............................354

D-46  ES23 actuator load vs. average bolt displacement ........................................................................354

D-47  ES24 (grouted, 1 dia. exposed length, 0.625 in. F1554 Gr. 55 bolts, n = 2, tension/shear, 6 in. nominal eccentricity, pretensioned) test schematic ..............................355
D-48  ES24 actuator load vs. average bolt displacement...........................................................355

D-49  ES25 (grouted, 3 dia. exposed length, 0.625 in. F1554 Gr. 55 bolts, n = 2, 
tension/shear, 6 in. nominal eccentricity, pretensioned) test schematic ..........................356

D-50  ES25 actuator load vs. average bolt displacement...........................................................356

D-51  ES26 (grouted, 1 dia. exposed length, 0.625 in. F1554 Gr. 55 bolts, n = 2, 
tension/shear, 6 in. nominal eccentricity, pretensioned) test schematic ........................357

D-52  ES26 actuator load vs. average bolt displacement...........................................................357

D-53  ES27 (grouted, 3 dia. exposed length, 0.625 in. F1554 Gr. 55 bolts, n = 2, 
tension/shear, 24 in. nominal eccentricity, pretensioned) test schematic ......................358

D-54  ES27 actuator load vs. average bolt displacement...........................................................358

D-55  ES28 (grouted, 0 dia. exposed length, 0.625 in. F1554 Gr. 55 bolts, n = 2, 
compression/shear, 6 in. nominal eccentricity, pretensioned) test schematic...............359

D-56  ES28 actuator load vs. average bolt displacement...........................................................359

D-57  ES29 (grouted, 1 dia. exposed length, 0.625 in. F1554 Gr. 55 bolts, n = 2, 
compression/shear, 24 in. nominal eccentricity, pretensioned) test schematic..............360

D-58  ES29 actuator load vs. average bolt displacement...........................................................360

D-59  ES30 (grouted, 1 dia. exposed length, 0.625 in. F1554 Gr. 55 bolts, n = 2, 
compression/shear, 24 in. nominal eccentricity, pretensioned, Teflon) test schematic ...361

D-60  ES30 actuator load vs. average bolt displacement...........................................................361

D-61  ES31 (grouted, 3 dia. exposed length, 0.625 in. F1554 Gr. 55 bolts, n = 4, 6 in. 
nominal eccentricity, pretensioned) test schematic .......................................................362

D-62  ES31 actuator load vs. average bolt displacement...........................................................362

D-63  ES32 (grouted, 3 dia. exposed length, 0.625 in. F1554 Gr. 55 bolts, n = 4, 24 in. 
nominal eccentricity, pretensioned) test schematic .......................................................363

D-64  ES32 actuator load vs. average bolt displacement...........................................................363

E-1  Distribution of combined normal and bending stresses on circular section ..............365

E-2  Assumed distribution of combined normal and shear stresses on circular section ....368

E-3  Assumed lower-bound distribution of combined normal and shear stresses on 
circular section ..............................................................................................................369
E-4 Assumed distribution of combined normal and shear stresses on circular section........371
Steel structural elements are often anchored to concrete foundations by way of base plates that stand off of the foundation, leaving an exposed portion of anchor bolts that must carry connection loads. Shear forces on individual anchor bolts produce not only shear on the bolt cross-section, but additional bending moments whose magnitudes depend on the exposed length of the bolt. Grout pads installed below the stand-off base plate offer additional contributions to the resistance of applied forces. Motivated by gaps in information and lack of uniformity in addressing anchor bolt steel shear strength in stand-off base plates, the objective of this dissertation was to characterize the steel strength of anchor bolts in ungrouted and grouted stand-off base plate connections. To satisfy this objective, a four-part experimental study was undertaken. Direct Shear testing established relationships between stand-off distance and ultimate steel shear strength. Torsion and Large-scale testing applied torsion loads to circular bolt groups, uniquely generating pure shear force on bolts throughout the test process. Eccentric Shear testing produced varying combinations of normal and shear applied loads to anchor bolts. Within the four experimental arrangements, several variables were investigated for their influences on anchor bolt steel strength. Rational analytical models supported by the experimental results were proposed for the determination of steel strength in stand-off base plate connections exposed to generalized loading and exposed length conditions. The models were
governed by the interaction of normal force, shear force, and bending moment on the circular anchor bolt cross-section and, for grouted connections, the influence of interfacial friction. Simplified recommendations were provided for practical implementation of the findings presented within this dissertation.
CHAPTER 1
BACKGROUND

1.1 Problem Statement

Steel base plates connecting poles, columns, and other structural elements to concrete through anchor bolts often stand off the concrete foundation to accommodate leveling adjustments. Accompanying base plate stand-off, a length of anchor bolts remains exposed between the base plate and concrete surface over which combinations of shear and normal force may be transmitted in the structural load path. Depending on the specific application and local ordinances, the volume between stand-off base plates and concrete surface may or may not be filled with an intermediate grout or mortar layer.

The presence of a bolt length over which forces act may significantly weaken the steel strength of bolts due to the interaction of bending, shear, and normal stresses on the bolt cross-section, resulting in a lower bolt steel capacity. Eligehausen et al. (2006) state that no accepted theory is presently available for determining the steel strength of anchor bolts experiencing combinations of shear force, normal force, and bending moment. Various combinations of these loading conditions spring from the type of connection, stand-off distance of the base plate, and proportions of applied normal and shear forces on the bolt.

NCHRP 494 (Fouad et al. 2003) presented a survey of state Department of Transportation representatives related to the performance of cantilever sign and signal support structures, which are ubiquitously installed with stand-off base plate connections. In their survey it was found that the predominant cause of foundation failure was anchoring structural inadequacies. The survey did not detail failure modes but it can be inferred that some portion of the failures were attributable to bolt weakening due to base plate stand-off, especially in connections without grout below the stand-off base plate.
Anchor bolt steel strength in ungrouted stand-off base plate connections is directly affected by bending moments produced by shear force transfer over the length between the concrete surface and the bottom of the base plate or, if leveling nuts are used, the bottom of the leveling nut, hereafter called the “exposed length,” a term taken from Lin et al. (2011). In the American Association of State Highway and Transportation Officials (AASHTO) LTS-6 *Standard Specifications for Structural Supports for Highway Signs, Luminaires, and Traffic Signals* (AASHTO 2013b), these bending moments are allowed to be ignored for exposed lengths of up to one bolt diameter. The American Concrete Institute (ACI) 318-11 *Building Code Requirements for Structural Concrete* (ACI 2011) and American Institute of Steel Construction (AISC) *Steel Construction Manual: 13th Edition* (AISC 2010) do not include provisions for ungrouted stand-off base plates.

Existing behavior and strength models in the body of literature for anchor bolt steel strength in ungrouted stand-off base plates are based on limited experimental work conducted in steel-to-steel connections in Scheer et al. (1987) and Lin et al. (2011). In the studies that used steel-to-steel loading apparatuses, test setups did not necessarily represent end conditions that would be present in field installations. As a result, both the shear strength and combined shear and normal strength remain open problems for investigation. Material type, influence of concrete, pretensioning, and the potential for size effect have not been studied based on the literature review. In grouted connections, AASHTO (2013b) does not allow grout to contribute to design strength. ACI (2011) specifies that the shear strength of equivalent flush-mounted connections be taken as 0.8 of the flush-mounted steel strength. AISC (2010) states that anchor bolt bending must be considered when the design calls for shear transfer through bolts. Some research has been conducted on anchor bolt steel strength in grouted stand-off base plate
connections (e.g., Adihardjo and Soltis 1979, Nakashima 1998, Gresnigt et al. 2008, Gomez et al. 2011), but experimental gaps remain and behavioral models do not fully account for forces experienced by anchor bolts and do not explicitly define criteria for ultimate strength.

The absence of a complete information in the body of experimental work and uniformity in the design approaches addressing anchor bolt steel strength in stand-off base plates serve as the motivating factor for this work.

1.2 Components in Stand-off Base Plate Connections

For the purposes of this dissertation, anchor bolt connections to concrete are broken into three categories defined by base plate support condition: those installed in flush-mounted base plates, ungrouted stand-off base plates, and grouted stand-off base plates. Figure 1-1 shows common definitions used throughout this dissertation and Figure 1-2 provides a profile schematic illustration of each support condition and. Both figures are illustrated for a connection where annular base plates connect round poles to concrete foundations. However, the concepts presented in this dissertation apply generally to any connection between a stiff steel base plate and concrete, including building column connections with rectangular base plates. Subsequent paragraphs to Figure 1-2 describe installation methods for each base plate support condition.

Figure 1-1. Definition of components illustrated for an ungrouted annular stand-off base plate.
Flush-mounted base plates are set directly on a finished concrete surface. Top nuts are placed above the base plate on the anchor bolts. Tightening the nuts prestresses the bolts in tension that is transferred through bond and/or bearing forces into the foundation concrete. An example of a field installation of a flush-mounted base plate is provided in Figure 1-3.

Figure 1-3. Field installation of flush-mounted base plate. (Photo courtesy of author.)

Ungrutted stand-off base plates are set on leveling nuts that have been pre-threaded onto anchor bolts. Tightening nuts (also referred to as “top nuts” in this dissertation) are threaded above the base plate to complete what is often referred to as a “double-nut” connection. Tightening the nuts imparts tension in the bolt only within the thickness between top nuts and leveling nuts. An “ungrutted” stand-off base plate connection can also be achieved without leveling nuts by setting the base plate on shim stacks or a setting plate. Examples of ungrutted stand-off base plate connections with shim stacks are rare in the literature and leveling plates have not been found. In reality, the behavior of shimmed base plates more closely represents grutted base plate behavior because the shims restrain downward base plate movement. For this reason shimmed connections included in the discussion of “grutted” connections in this dissertation. Unless otherwise specified, then, it is implied throughout this dissertation that ungrutted stand-off base plates contain leveling nuts as shown in Figure 1-4.
Grouted stand-off base plates are used in both column and signal/sign connections. The base plate is first placed on leveling nuts, shim stacks, or a setting plate as described in AISC Design Guide 1 (Fisher and Kloiber 2006). After the base plate has been erected and fastened, grout is placed to fully fill the void between the base plate and the concrete surface. Tightening top nuts prestresses bolts within the thickness of the base plate in double-nut connections and through the grout into the concrete in single-nut grouted connections. An example of a field installation of a grouted stand-off base plate is shown in Figure 1-5.
1.3 Load Transfer through Anchored Connections to Concrete

Base plate support condition dictates the mechanisms of load transfer from the base plate to the concrete foundation. In flush-mounted and grouted stand-off base plate connections, as detailed by Cook and Klingner (1992), direct compression forces acting on the structure are transferred from the base plate to the concrete or the grout. Direct tension forces are transferred fully through the anchors and into the supporting concrete foundation. Overturning moments are resolved into compression forces acting on the concrete alone and tension forces in the anchors, with base plate rigidity playing a role in the distribution of overturning moment to the bolts. Direct shear and torsional forces are resisted by a combination of the anchor steel and friction between the base plate and the concrete surface.

Ungrouted stand-off base plates transfer all compression and tension forces through anchor bolts. Overturning moment, then, is resisted by both tension and compression in bolts depending on the position of the bolts relative to the neutral axis of bending. Normal force in individual bolts is proportional to the distance from the neutral axis over the moment of inertia of the bolt group.

To illustrate the transfer of load to anchor bolts in stand-off base plate connections, a sign structure subjected to self-weight and a wind load acting on a cantilevered arm is illustrated in Figure 1-6 with an ungrouted base plate. The resultant wind load on the structure produces global overturning moment, torsion, and direct shear forces, while self-weight of the structure produces global axial and overturning forces. All of the forces transfer through the annular base plate, anchor bolts, and the foundation. Base plate forces resolve to combinations of normal and shear forces on individual anchor bolts within the group, normal forces in individual bolts resulting from the superimposition of equally shared base plate axial loads and distributed axial forces from overturning moments. As with flush-mounted installations, the magnitude of normal force
from overturning moment is related to the anchor bolt position relative to the neutral axis of the base plate moment. Shear force on individual anchor bolts is the resultant vector sum of equally distributed direct shear forces and resolved base plate torsion forces as shown in Figure 1-6.

Wind loads acting on cantilever sign structures produce individual bolt forces from torsion $V_t$ on the order of ten times the direct shear force.

Building column connections are most often designed to carry axial compression loads to the foundation (Fisher and Kloiber 2006). While examples of building column connections are not shown in this introductory chapter, their behavior is no different from that of annular base plate connections. In grouted and flush-mounted connections experiencing enough compression to produce friction resistance greater than the shear load on the column, anchors are not needed to transfer load. In some cases, however, columns experience tension, moment, and shear forces, all of which transfer combinations of shear and tension forces through the anchors.

<table>
<thead>
<tr>
<th>Global Forces</th>
<th>Connection Forces</th>
<th>Bolt Forces</th>
</tr>
</thead>
<tbody>
<tr>
<td><img src="image1" alt="Global Forces Diagram" /></td>
<td><img src="image2" alt="Connection Forces Diagram" /></td>
<td><img src="image3" alt="Bolt Forces Diagram" /></td>
</tr>
</tbody>
</table>

Figure 1-6. Transfer of base plate forces into individual shear forces on anchor bolts in a cantilever sign structure with an ungrouted stand-off base plate.
1.4 Load Transfer through Anchor Bolts

Anchor bolt shear forces act over a lever arm in stand-off connections. As a result of the shear force acting over this distance, bending moments to develop in the bolts in addition to the bolt shear force. Where rotation is restrained on both ends of the lever arm, which for the purposes of this dissertation is taken as the exposed length of the anchor bolt, the most extreme bending stresses are observed at the top and bottom of the exposed length. If one end is free to rotate (e.g. certain single-bolt connections), however, the bolt acts as a “flagpole” with extreme bending moment at the point of restraint.

Figure 1-7 illustrates the transfer of applied shear force through anchor bolt shear force and bending moment with double moment restraint for the ungrouted stand-off base plate condition. In a perfectly plumb anchor bolt, axial forces transfer directly through its centroid. Initial out-of-plumbness and second-order geometry, however, cause the transfer of axial force to act eccentrically to the bolt centroid, generating additional bending moments. Beyond elastic bending strains, plastic hinges at the top and bottom of the bolt exposed length develop on the way to anchor bolt failure.

Figure 1-7. Schematic illustration of shear force transfer through double-nut ungrouted stand-off base plate connection.

In grouted connections, direct compression forces are transferred from the base plate to the grout pad and anchor bolts in proportion to the relative stiffnesses of the materials. Direct tension forces are taken fully by the anchors. Overturning moments may be resolved into compression
forces acting on the grout and concrete and into tension forces in the anchors. Direct shear forces are transferred through a combination of the resistance of the steel anchors and systemic friction forces in the load path from the base plate through the grout through the concrete concrete surface as illustrated in Figure 1-8. Friction may develop from the restraint of vertical translation in the angled geometry of the displaced anchor bolts and/or through compressive forces. While some compressive forces may be taken by the bolt proportional to the stiffness of the bolt relative to the surrounding grout pad, as the bolt deforms and loses stiffness, grout pads will take on a much more significant portion, if not all, of the load due to very high relative stiffness to the bolts. This concept is explored in detail in Chapter 6.

Figure 1-8. Schematic illustration of shear force transfer through double-nut grouted stand-off base plate connection.

1.5 Objectives

This dissertation addresses the steel strength of anchor bolts in ungrouted and grouted stand-off base plate connections under generalized static loading conditions. However, several concrete failure modes related to anchor bolt embedment depth are defined by ACI 318-11 Appendix D (ACI 2011). All potential failure modes are shown in Table 1-1 must be considered in the design of anchored connections to concrete. The theoretical basis and design approaches for concrete failure modes are illustrated in Fuchs et al. (1995), Eligehausen and Balogh (1995), and Eligehausen et al. (2006). Design information on embedment failure modes is available in ACI 318-11.
Table 1-1. Tension and shear failure modes defined by ACI (2011).

<table>
<thead>
<tr>
<th>Tension Failure Modes</th>
<th>Shear Failure Modes</th>
</tr>
</thead>
<tbody>
<tr>
<td>Bolt steel</td>
<td>bolt steel</td>
</tr>
<tr>
<td>concrete breakout</td>
<td>concrete breakout</td>
</tr>
<tr>
<td>adhesive bond</td>
<td>concrete pryout</td>
</tr>
<tr>
<td>anchor pullout</td>
<td>concrete splitting</td>
</tr>
<tr>
<td>concrete side-face blowout</td>
<td></td>
</tr>
</tbody>
</table>

Information gaps and absence of unifying theory and design approaches for addressing anchor bolt steel shear strength in stand-off base plates served as the motivating factors for this dissertation, which was driven by the following overarching objective: to develop structural models of anchor bolt steel strength in ungrouted and grouted stand-off base plate connections for generalized loading conditions over a practical range of exposed lengths.

1.6 Dissertation Organization

Seven chapters and five appendices comprise this dissertation. Chapter 1 introduced the motivation, problem, objective, and structure of this dissertation. Chapter 2 reviews the literature on background theory and published research from related experimental and analytical research programs focusing on the tension, shear, and combined steel strength and behavior of bolted connections. Chapter 3 discusses how forces experienced by anchor bolts in flush-mounted, ungrouted, and grouted stand-off base plate connections are addressed in existing design codes. Chapter 4 describes the experimental program undertaken in this dissertation studying the steel strength of anchor bolts in four experimental arrangements: Direct Shear, Torsion, Large-scale, and Eccentric Shear. Chapter 5 provides a summary of the experimental results and load-displacement behavior from tests. In Chapter 6, analytical models are proposed and evaluated against experimental data. Chapter 7 summarizes the work conducted in the dissertation and presents its conclusions. Finally, Appendices A through D provide individual load-displacement
results for tests and Appendix E provides derivations for the interaction of normal forces, shear forces, and bending moments acting on circular cross-sections.
CHAPTER 2
LITERATURE REVIEW

2.1 Introduction

This chapter provides a review of the state-of-the-art research and theory concerning steel strength of anchor bolts. General discussions of steel behavior and failure at material, cross-sectional, and structural member levels are first introduced, followed by research and theory relevant to anchor bolt steel strength in connections to concrete. A summary of the literature and open areas for further pursuit relating to the steel strength of anchor bolts in stand-off base plate connections is provided at the end of the chapter.

2.2 Background

2.2.1 Steel Material Characteristics and Behavior

Steel materials are alloys consisting of, at minimum, iron and carbon in a crystalline atomic lattice structure (Bhadeshia and Honeycombe 2006). Imperfections, or misaligned interatomic connections in the lattice, are known as dislocations; it is the ability of dislocations to move without causing material failure that allows high-strain behavior in steel and other ductile metals. During steel production, many individual crystals called grains generate simultaneously at random geometric orientations as the steel coalesces from liquid to solid. Beyond carbon, the combination of additional alloying elements and production methods influence the characteristics of the crystal lattice, grain sizes, and distribution of dislocations to produce a wide spectrum of strengths, ductilities, resistance to chemical attack, resistance to temperature effects, and many other properties that can be tailored to the myriad unique applications of steel.

The response of any material to load is an expression of interatomic attraction and often idealized as a system of interconnected springs. Based on the attraction between individual atoms, the theoretical strength of a material can be estimated as approximately 10% of the...
Young’s modulus (Bhadeshia and Honeycombe 2006). Strengths approaching this theoretical value have been achieved in the laboratory setting using small-diameter material “whisker” specimens (Gordon 2006); however, much lower strengths are observed on the macroscopic level due to stress concentrations from dislocations, microcracks, grain boundaries, and other deviations from the ideal atomic structure of a material. Initial yielding of a material occurs when stress concentrations overcome the interatomic bond at any discrete location. In brittle materials such as concrete and ceramics, cracks formed during initial yielding propagate without material recovery, resulting in sudden failure; this topic is covered in depth by the field of fracture mechanics, pioneered by Griffith (1921).

Ductility in metals results from dislocations moving through the crystalline structure along slip (shear) planes after the material has begun to yield (Bruneau et al. 2011). As steel is loaded past initial yield, the movement of dislocations is slowed by the presence of grain boundaries and alloying elements, causing dislocations to aggregate and allowing higher loads to be taken on the material level. This high-strain capacity increase is known as “strain-hardening.” Fracture in steel occurs when the aggregations of dislocations do not allow further movement, resulting in stress concentrations sufficient to separate the interatomic bonds on a macroscopic level.

The effects of the randomly distributed dislocations and randomly oriented grains blend to form the material’s stress/strain constitutive properties, allowing steel structures and components to be effectively treated and analyzed as a continuum (McGinty 2014). Constitutive models of steel are often represented as equivalent elastic-perfectly plastic or other multilinear approximations that greatly simplify complex analytical problems, often at relatively small error.

2.2.2 Steel Failure Criteria

Stress acting on a differential element of a material can be transformed to component shear and tensile stresses by rotating the plane of reference; i.e., given any state of stress, tension and
shear exist simultaneously. Principal normal stresses are obtained by rotating the element until shear stresses have canceled. Figure 2-1 illustrates a transformation from a general state of shear and normal stress in three-dimensional Cartesian coordinates to principal stresses in principal coordinates. The state of stress can be expressed in tensor form, given in (2-1) for a general state of shear and normal stresses and in (2-2) for principal stresses.

\[
\sigma = \begin{bmatrix}
\sigma_x & \tau_{xy} & \tau_{zx} \\
\tau_{xy} & \sigma_y & \tau_{yz} \\
\tau_{zx} & \tau_{yz} & \sigma_z
\end{bmatrix}
\]  

(2-1)

\[
\sigma = \begin{bmatrix}
\sigma_1 & 0 & 0 \\
0 & \sigma_2 & 0 \\
0 & 0 & \sigma_3
\end{bmatrix}
\]  

(2-2)

where \(\sigma_x, \sigma_y, \sigma_z\) = directional normal stresses on differential element
\(\tau_{xy}, \tau_{yz}, \tau_{zx}\) = directional shear stresses on the element
\(\sigma_1, \sigma_2, \sigma_3\) = principal normal stresses
For steel and other ductile metals, the most widely accepted model for material failure is the von Mises (1913) criterion, by which failure is defined by the state of stress corresponding to the distortional energy, energy associated with only shape change (i.e., excluding volume change) required for material failure. Distortional energy is found by subtracting the energy due to volume change from the total energy associated with a given state of stress.

An excellent derivation of the von Mises criterion can be found in Jong and Springer (2009), whose logical progression of the derivation is summarized in (2-3) through (2-13) below. To determine the total energy on a differential element subjected to a state of stress, the components of the principal stress tensor are transformed to strain energy using Hooke’s law. Each component value of strain can be represented as given in (2-4), where the respective subscripts indicate the relationship for the three cases. Substituting (2-4) throughout (2-3), the total energy can be written as (2-5).

\[
\begin{align*}
   u_{tot} &= \frac{1}{2}(\sigma_1 \varepsilon_1 + \sigma_2 \varepsilon_2 + \sigma_3 \varepsilon_3) \\
   \varepsilon_{1/2/3} &= \frac{1}{E} \left( \sigma_{1/2/3} - \nu (\sigma_{2/3/1} + \sigma_{3/1/2}) \right) \\
   u_{tot} &= \frac{1}{2E} \left( \sigma_1^2 + \sigma_2^2 + \sigma_3^2 - 2\nu(\sigma_1 \sigma_2 + \sigma_2 \sigma_3 + \sigma_3 \sigma_1) \right)
\end{align*}
\]

where
- \( u_{tot} \) = total strain energy acting on a differential element
- \( \sigma_1, \sigma_2, \sigma_3 \) = principal normal stresses
- \( \varepsilon_1, \varepsilon_2, \varepsilon_3 \) = principal normal strains
- \( E \) = Young’s modulus of material
- \( \nu \) = Poisson’s ratio of material

The stress due to volume change, or hydrostatic stress, is defined as the average of the principal stresses. By taking each value of stress in the principal stress tensor and, analogously in (2-6), as equal to the hydrostatic stress, the hydrostatic strain energy can be determined similarly to total energy above:
\[ u_{\text{hyd}} = \frac{1 - 2\nu}{6E} \left( \sigma_1^2 + \sigma_2^2 + \sigma_3^2 - 2(\sigma_1\sigma_2 + \sigma_2\sigma_3 + \sigma_3\sigma_1) \right) \]  
(2-6)

where \( u_{\text{hyd}} \) = hydrostatic strain energy acting on a differential element

Distorsional energy is simply the total energy less the hydrostatic energy, mathematically given as Equation (2-7). Applying this to the special case of uniaxial tension applied to a material presents a case where \( \sigma_1 \) is equal to the yield stress, \( \sigma_y \), of the material and \( \sigma_2 \) and \( \sigma_3 \) equal zero, the distortional energy reduces to Equation (2-8).

\[ u_{\text{dis}} = \left( \frac{1 - \nu}{3E} \right) \left( \frac{(\sigma_1 - \sigma_2)^2 + (\sigma_2 - \sigma_3)^2 + (\sigma_3 - \sigma_1)^2}{2} \right) \]  
(2-7)

\[ u_{\text{dis}} = \frac{1 - \nu}{3E} \sigma_y^2 \]  
(2-8)

where \( u_{\text{dis}} \) = distortional energy acting on a differential element

By the von Mises failure criterion, for any given state of stress, when the energy associated with uniaxial tension capacity is exceeded, the material fails. Thus, in terms of principal stresses, the stress limit can be taken as when the right-hand side of Equation (2-7) exceeds the right-hand side of (2-8) as given in (2-9). Rearranging (2-9), the criteria for failure can be found (2-10). Below the limiting value of the ultimate stress of the material, the values produced by the right-hand side of (2-10) are called the “effective,” or von Mises, stresses (2-11). An alternate, more generalized, form of the von Mises stress expressed in terms of shear and tension stresses is often used to express the criterion (2-12). The equivalency of the two respective forms in (2-11) and (2-12) is derived in Jong and Springer (2009).
\[ \sigma_o = \frac{1}{\sqrt{2}}((\sigma_1 - \sigma_2)^2 + (\sigma_2 - \sigma_3)^2 + (\sigma_3 - \sigma_1)^2) \quad (2-10) \]

\[ \sigma_{VM,3} = \frac{1}{\sqrt{2}}((\sigma_1 - \sigma_2)^2 + (\sigma_2 - \sigma_3)^2 + (\sigma_3 - \sigma_1)^2) \leq \sigma_o \quad (2-11) \]

\[ \sigma_{VM,3} = \frac{1}{\sqrt{2}}((\sigma_x - \sigma_y)^2 + (\sigma_y - \sigma_z)^2 + (\sigma_z - \sigma_x)^2 + 6(\tau_{xy}^2 + \tau_{yz}^2 + \tau_{zx}^2)) \leq \sigma_o \quad (2-12) \]

where \( \sigma_o \) = ultimate normal stress capacity of material

\( \sigma_{VM,3} \) = three-dimensional von Mises stress

Figure 2-2 provides a graphical illustration of the von Mises yield criterion as a function of principal stresses in three-dimensional Haigh-Westergaard stress space. Failure is given as the combination of principal stresses that lies on the red curves; any state of stress within the curve has not failed. In Haigh-Westergaard space, in which all of the principal space axes are positioned isometrically, it can be readily seen that failure is taken as a constant distance from the hydrostatic axis (i.e. the line defined by \( \sigma_1 = \sigma_2 = \sigma_3 \)). Rotating the view, it can be seen that the criterion forms a cylinder parallel to the hydrostatic axis, where the material can theoretically hold infinite amounts of hydrostatic stress. While this is not true, the low hydrostatic stress components in anchor bolt applications allow the von Mises criterion to remain accurate despite this limitation.

Reducing the criterion to a two-dimensional state of stress, (2-13) is produced. Figure 2-3 shows the von Mises criterion in two-dimensional principal stress space, which can be viewed as a planar cut through the three dimensional cylinder shown in Figure 2-2. Anchor bolts subjected to combinations of normal and shear forces experience two-dimensional stress states, meaning that the simplified failure criteria defined by (2-13) and Figure 2-2 apply. The most widely acknowledged experimental confirmation of the von Mises criteria was performed by Taylor and...
Quinney (1932) who subjected copper, steel, and aluminum specimens to precise combinations of tensile and shear stress through applied torsion and tension force.

$$\sigma_{VM,2} = \sqrt{\sigma_x^2 + \sigma_y^2 + 3\tau_{xy}^2} \leq \sigma_o$$

where \(\sigma_{VM,2D} = \) two-dimensional von Mises stress

Figure 2-2. Three-dimensional von Mises failure criterion in Haigh-Westergaard principal stress space.

Figure 2-3. Two-dimensional von Mises yield criterion in principal stress space.
Beyond the specific case of ductile isotropic metals, for which the von Mises criterion applies, criteria exist accounting for brittle failure, anisotropy, and accounting for high amounts of hydrostatic stress, all of which are impertinent to this dissertation. Christensen (2013) authoritatively discusses the most prevalent historical failure models and proposes a generalized model for all materials based on the ratio of tensile to compressive material strength.

### 2.2.3 Cross-sectional Plastic Limits

While this section and dissertation in general are concerned with the plastic limit of beam cross-sections, an acknowledgement of beam elasticity is briefly introduced. In the elastic range of beam behavior, a plane cut through a beam can be used to visualize the strains in a material subjected to normal and shear stresses. Normal forces transpose and bending forces rotate the original plane. Without considering the effects of shear, the effects of tension and bending allow the initial plane cut to maintain its planarity in a new orientation.

Shear strain distortions from shearing and torsional forces warp the initial plane, which in turn causes elastic bending stresses to distribute non-linearly about a neutral axis that is shifted from the centroid of the section. Euler-Bernoulli beam theory does not account for shear distortion, simplifying analysis for many practical circumstances with sufficient accuracy, as the effects of shear distortions are relatively small for beams with span-to-depth ratios of six or more (Young 1989). In the elastic behavior of anchor bolts with exposed length, which have span-to-depth ratios approaching zero, shear distortions can be much more pronounced in the elastic range of behavior. At span-to-depth ratios less than one, maximum elastic stresses due to shear distortion exceed equivalent Euler-Bernoulli predicted stresses by an order of magnitude. Timoshenko beam theory, the derivation of which can be found in Graff (1975), accounts for these effects to provide a more accurate distribution of elastic stresses and strains.
Because this dissertation focuses on ultimate plastic capacity of anchor bolts, any discussion and illustration of the elastic behavior of beams is represented with the assumptions of Euler-Bernoulli beam theory for illustrative clarity and simplicity. Nonetheless, it is readily acknowledged that shear distortions greatly impact the elastic response of anchor bolts in standoff base plate connections.

Beyond the elastic response, stresses are redistributed until full plasticity is taken up by a section. The fully plastic distribution of stresses is assumed to develop favorable and similar fully plastic stress distributions irrespective of span-to-depth ratio. As a result, short and long beams are unified in the analysis contained in this dissertation. In practical plastic theory, several simplifying assumptions are often made. Bruneau et al. (2011) provide the following:

- Plasticity in a structure occurs at plastic hinges of zero length modeled as perfectly rigid-perfectly plastic entities.
- Geometric deformations due to large strains are not considered.
- Strain-hardening is not accounted for.
- Plastic deformations allow ultimate strength of a structure to develop.
- Load is applied in quasi-static fashion without reversal.

A beam cross-section can experience a variety of three-dimensional stress states from bending, normal, shear, and torsional forces. Anchor bolts, which are subjected to combinations of externally applied tension and shear force as described in Chapter 1, may experience shear stresses, axial stresses, and, in the case of bolts with exposed lengths, bending stresses about an axis perpendicular to the direction of applied shear force. The circular cross-section of bolts allows bending to be considered equally from any angle of application. In the following subsections, normal, bending, and shear stresses acting on a solid, prismatic cross-section of an isotropic material are first considered individually, followed by the interactions of their two- and
three-way combinations. For thin-walled sections and other states of stress on a cross-section including torsion and the torsional interactions with the stress states mentioned above, the reader is referred to Jirasek and Bazant (2001), Bruneau et al. (2001), and Zyczkowski (1981).

2.2.3.1 Normal forces

Material strength is most often expressed in terms of the uniaxial material strength in tension, which, when divided by the cross-sectional area, results in the ultimate tensile stress capacity. In the inelastic portion of the tensile test, the specimen first undergoes yield and cross-sectional loss across the entire gauge length due to the Poisson effect, followed by a high-strain process called necking, where the cross-sectional area is further reduced locally at a critical section that undergoes significant plastic flow until fracture. The greatest total load observed by a tensile specimen will occur at the onset of necking, after which the localized loss in cross-section produces stress concentrations in the necked region.

True stresses in the reduced cross-section are higher than the apparent, or engineering, stresses, which assume that the load acts over the undeformed cross-section. It is frequently assumed that up to the necking point, the cross-sectional reduction is constant along the entire gauge length of a tensile specimen (Ling 1996), resulting in (2-14) and (2-15). The reader is referred to Ling (1996) if true stresses beyond necking are desired.

\[ A_0 l_o = A_l l_i \quad (2-14) \]
\[ \sigma_{true} = \frac{T_i l_i}{A_o l_o} \quad (2-15) \]

where
- \( A_o \) = undeformed cross-sectional area
- \( l_o \) = original gauge length of tensile specimen
- \( A_l \) = cross-sectional area at an instance during loading
- \( l_i \) = measured gauge length at an instance during loading
- \( \sigma_{true} \) = true stress up to the point of striction due to plastic deformations
- \( T_i \) = measured tension force at an instance during loading
The constitutive stress-strain relationship of steel is the same in compression as in tension; however, on the member level, the cross-section increases under compression due to the Poisson effect, yielding higher overall compressive strengths. While this is acknowledged, it may not be practical to account for the differences due to compressive Poisson’s effects and is generally conservative to ignore the additional normal strength capacity in compression.

2.2.3.2 Bending moments

Bending moments produce axial stresses about the neutral axis of a cross-section. The maximum elastic moment occurs when the extreme fiber about the neutral axis of bending reaches yield, the well-known equation given in (2-16). At bending moments beyond the elastic limit and assuming elastic-perfectly plastic behavior, the extreme fibers of the cross-section deform without increasing in stress, the region of plasticity progressing until the entire section has yielded, in theory, requiring infinite cross-sectional curvature.

While infinite curvature cannot be produced due to material strain limits, the strains allowed by ductile metals allow for near-complete development of the theoretical plastic moment. In the fully plastic condition the maximum moment for the cross-section has been reached; this maximum moment is related to the moment at the elastic limit by a cross-sectional shape factor, \( \kappa \). Figure 2-4 shows the progression of plasticity in a cross-section subjected to pure moment for an elastic-perfectly plastic material and a high-strain representation of the moment distribution for a realistic constitutive material model.

\[
M_Y = \sigma_o S \quad (2-16)
\]

\[
M_o = \sigma_o \kappa S = \sigma_o Z \quad (2-17)
\]

where
- \( M_Y \) = elastic limit of moment; first yield of extreme fiber in bending
- \( M_o \) = theoretical ultimate moment capacity of section
2.2.3.3 Shear forces

The yield stress of a steel element exposed to shear can be derived using the von Mises formulation in Equation (2-12). With unidirectional shear, all normal stresses as well as the shear stresses in the other two directions reduce to zero. Setting the effective stress to the maximum permissible stress, the reduced formulation may be expressed as in (2-18) and (2-19).

\[
\sigma_t = \sqrt{\frac{1}{2} 6 \tau_t^2} = \sqrt{3} \tau_o \quad (2-18)
\]

\[
\tau_o = \frac{1}{\sqrt{3}} \sigma_o = \alpha \sigma_o \approx 0.577 \sigma_o \quad (2-19)
\]

where

- \( \sigma_o \) = ultimate normal stress capacity of material
- \( \tau_o \) = ultimate shear stress capacity of material
- \( \alpha \) = relationship between ultimate shear and tensile stress

Beyond the well-known elastic distribution of shear stress on a cross-section, \( VQ/Ib \), no information was found in the literature regarding the theoretical determination of the theoretical ultimate shear capacity of a cross-section other than by Jirasek and Bazant (2001), who state that this is “difficult to find.” In the absence of referential theory and based on the observed values of shear capacity above 0.577 times the tensile capacity in tests of bolts as discussed in Section 2.3.2.1, it is assumed hereafter that beyond the elastic shear stress limit, the maximum allowable...
shear stress spreads plastically away from the neutral axis of the cross section until the entire cross-section has yielded in shear as illustrated in Figure 2-5.

As a result of the assumption that shear plasticity spreads equally across the entire cross-section, the theoretical shear strength of a cross-section is simply equal the maximum shear stress applied uniformly over the cross-sectional area as given in (2-20). It is worth noting that while cross-sectional area decreases during a tensile test, a cross-section under pure shear loading undergoes negligible area change. Thus, to accurately predict the shear strength of a cross-section using the assumptions presented, the true stress (using the reduced cross-sectional area at failure) and not the engineering stress (using the original cross-sectional area) from a uniaxial tension test should be applied to the cross-section experiencing shear. It follows that the true shear strength of a bolt will be greater than 0.577 times the tensile strength, a probable explanation for experimental $\alpha$ values of 0.62 and 0.68 reported in Kulak et al. (1987). For practical engineering purposes relating to concrete based on experimental observations, $\alpha$ is often conservatively taken as 0.6 (e.g. by ACI (2011), AASHTO (2013b), and Eligehausen et al. (2006)); this value is used in the analysis of the data presented in this dissertation.

Figure 2-5. Assumed spread of cross-sectional plasticity due to shear force based on strength observations of steel bolts.
\[ V_o = \alpha \sigma_t A \]  
\[ V_{o,\text{true}} = \alpha \sigma_{o,\text{true}} A \]

where 
- \( V_o \) = shear capacity of a cross-section
- \( \alpha \) = proportion of shear to tensile stress material capacity
- \( V_{o,\text{true}} \) = true shear capacity of a cross-section
- \( \sigma_{o,\text{true}} \) = true uniaxial stress at onset of necking

### 2.2.3.4 Combined bending moment and normal force

Jirasek and Bazant (2001) describe the interaction of bending and normal force. Figure 2-6 shows a cross-section under variable combinations of normal and bending forces producing a fully plastic state. The resultant fully plastic stress diagram can be decomposed into equivalent components of stresses due to bending and normal forces as shown in Figure 2-7, where the bending stresses are moved to the extreme ends of the cross-section about the plastic neutral axis and the stresses due to normal force are placed in the remaining core.

![Figure 2-6. Fully plastic combination of normal and shear stresses on cross-section.](image-url)
For all possible neutral axis positions the cross-sectional normal forces and moments are found using their generic definitions as given in (2-22) and (2-23), respectively. The interaction of normal and bending forces can then be determined by satisfying cross-sectional equilibrium for all combinations of fully plastic moment and normal stresses and plotting the resulting values of \( N \) and \( M \). The equation describing the best fit curve of these plotted points defines their interaction, generally expressed in the form given in (2-24). Jirasek and Bazant demonstrate that exponents \( a \) and \( b \) are equal to 1 and 2, respectively, for a rectangular cross-section.

\[
N = \int_A \sigma_0 dA = \sigma_0 (A^+ - A^-) = N^+ + N^-
\]  

(2-22)

\[
M = \int_A \sigma_0 y dA = N^+ d_R^+ + N^- d_R^-
\]  

(2-23)

\[
\left( \frac{M}{M_0} \right)^a + \left( \frac{N}{N_0} \right)^b = 1
\]

(2-24)

where
- \( N \) = total normal force acting on cross-section for given state of stress
- \( A^+ \) = area of cross-section experiencing positive (tensile) stress
- \( A^- \) = area of cross-section experiencing negative (compressive) stress
- \( N^+ \) = total positive (tension) force acting on cross-section
- \( N^- \) = total negative (compression) force acting on cross-section
- \( M \) = bending moment acting on cross-section for given state of stress

Figure 2-7. Equivalent distribution of bending and normal stresses on fully plasticized cross-section.
\( d_R^+ \) = distance to centroid of tensile area

\( d_R^- \) = distance to centroid of compressive area

### 2.2.3.5 Combined normal force and shear force

The two-dimensional von Mises formulation (2-13) can be rearranged to derive the limiting combinations of axial and shear stresses on a cross-section. Setting all terms in (2-13) but a single normal stress and a single shear stress component to zero, then setting the effective stress to \( \sigma_o \), the criterion becomes (2-25) for axial and shear stresses acting on a common plane, which can then be rearranged to form the equation given in (2-26) and (2-27) defining a circular interaction relationship between normal and shear stresses.

\[
\sigma_o = \sqrt{\sigma^2 + 3\tau^2} \tag{2-25}
\]

\[
\left( \frac{\sigma}{\sigma_o} \right)^2 + \left( \frac{\tau}{\alpha \sigma_o} \right)^2 = 1 \tag{2-26}
\]

\[
\left( \frac{\sigma}{\sigma_o} \right)^2 + \left( \frac{\tau}{\tau_o} \right)^2 = 1 \tag{2-27}
\]

where

- \( \sigma \) = normal stress acting on cross-section
- \( \tau \) = shear stress acting on cross-section

Using numerical methods Shen (1995) found that for a rectangular section the interaction between shear and tension forces at the cross-section forces is described by the circular elliptical relationship shown below in (2-28). Experimental data on bolts and rivets (circular sections) under combinations of shear and tension force is discussed in Section 2.3.3.2, where this circular interaction was found to match experimental data. Given the numerical and experimental bases, it can be extrapolated that in the fully plastic state of a cross-section subjected to both shear and normal forces, the limiting magnitudes of both shear and normal stresses are able to spread uniformly across the entire cross-section, similar to what was described in Section 2.2.3.1 for the pure shear case.
\[
\left( \frac{N}{N_o} \right)^2 + \left( \frac{V}{V_o} \right)^2 = 1
\]  \hspace{1cm} (2-28)

where

- \( N \) = normal force acting on cross-section
- \( V \) = shear force acting on cross-section
- \( N_o \) = normal force capacity of cross-section
- \( V_o \) = shear force capacity of cross-section

Numerically, assuming uniformity of shear and normal stresses throughout the cross-section as illustrated in Figure 2-8 allows a transition from the interaction of stresses at a discrete point (2-27) to the interaction of total shear and tension forces acting on a cross-section given in (2-28).

![Figure 2-8. Combined effects of normal stress and assumed fully plastic shear stress.](image)

### 2.2.3.6 Combined bending moment and shear force

In order to combine the effects of normal stresses from bending moments with shear stresses, the shear stresses may be transformed into equivalent normal stresses using the von Mises failure criterion in (2-13). Plastic admissibility is satisfied when, at any point on the cross-section, the total effective normal stresses do not exceed the ultimate normal stress of the material.
For a cross-section experiencing bending moments and shear stresses less than or equal to the elastic limit of the material, the greatest magnitude of normal stress, of course, is at the extreme fibers of the section, while the greatest magnitude of shear stress is at the center of the section. It can be demonstrated that the respective elastic limits of shear and bending stresses can exist simultaneously on a cross-section; i.e., there is no elastic interaction between shear and bending forces (Zyczkowski 1981).

Beyond the elastic limit, assumptions about the distribution of bending and shear stresses dictate their resulting mathematical expressions of interaction. Two methods are described below, one with lower-bound assumptions and a second using upper-limit assumptions. Both methods may be used to produce the interaction of bending and shear forces in the general form presented in (2-29).

\[
\left(\frac{M}{M_0}\right)^a + \left(\frac{V}{N_0}\right)^b = 1
\]  

(2-29)

**Lower-bound Method.** Consider a beam loaded in pure bending with moments between the elastic limit and the fully plastic condition. Plastic regions of the cross-section experience the material normal stress limit and cannot carry additional stress. If, after a semi-plastic bending moment is already experienced by the cross-section, shear force is applied, plastically admissible shear stresses can only exist in the remaining elastic core of the cross-section from the original bending stresses. According to Zyczkowski (1981), Jirasek and Bazant (2001), and Shen (1995), Bezukhov (1936) was the first to propose this stress distribution and proposed a criterion for failure, conservatively assumed to be the first instance at which shear stress reaches the elastic limit; i.e., partial plastification of the cross-section. Bezukhov’s stress distribution is illustrated in Figure 2-9.
According to Zyczkowski (1981), a fully plastic variation of this method was provided by Palchevsky (1948). In this variation, it is assumed that at full sectional plasticity, bending stresses are distributed entirely to the extreme regions of the cross-section with respect to its neutral axis. It was assumed that shear acting on the remaining elastic core reaches full plasticity. Figure 2-10 shows the fully plastic cross-sectional stress distribution assumed for the lower-bound method; it can be seen that when converted to effective stresses, this distribution presents the same mathematical relationship as that between fully plastic bending moments and normal forces illustrated in Figure 2-7. Thus, in (2-29), $a$ and $b$ equal 1 and 2, respectively, for a rectangular cross-section.

Despite the fully plastic assumption Zyczkowski (1981) states that this method provides a lower-bound estimate of the bending-shear interaction; it will be seen that this produces values well under the interaction from the subsequently presented upper-limit energy method. As such, the Palchevsky (1948) model, and not that attributed to Bezukhov (1936), is used as the lower-bound method in the derivation of cross-sectional limits for a circular section later in this dissertation.
Figure 2-10. Fully plastic combined bending and shear stress distribution attributed to Palchevsky (1948).

**Upper-limit Method.** A fully plastic upper-limit solution for the interaction between moment and shear acting on a section is provided by Jirasek and Bazant (2001), whose derivation of the interaction is provided in (2-30) through (2-35). In this formulation, a functional equation is set up such that a combination of permissible sums of moment and shear forces is maximized by taking the most favorable combination of shear and moment stresses at every point in the cross-section to maximize the total energy resisted by the cross-section. In order to combine shear and bending terms with unit consistency, a factor, $k$, expressed in units of length, is multiplied with the shear term. (2-30) shows the functional relationship between bending moments and shear forces expressed in terms of $k$.

\[
M + kQ = \int_A z\sigma(z)\,dA + k \int_A z\tau(z)\,dA \tag{2-30}
\]

where

- $M$ = moment experienced by the cross-section
- $Q$ = shear experienced by the cross-section
- $k$ = adjustable factor for unit consistency
- $\sigma(z)$ = normal stress at distance $z$ from the neutral axis of bending
- $\tau(z)$ = shear stress at distance $z$ from the neutral axis of bending
To combine effects of stresses from moment and shear forces and set an upper limit on the sum total of possible stress, shear stresses are transformed using the von Mises yield criterion (2-13) to the remaining available equivalent normal stresses given a simultaneous state of normal stress at a point on the cross-section due to bending (2-31). Thus, the von Mises failure criterion is satisfied at all points in the cross-section. To maximize (2-31), its derivative, given in (2-32), is set equal to zero.

\[
\Phi(\sigma) = \int_A z \sigma(z) dA + \int_A \frac{1}{\alpha_0} \sqrt{\sigma_0^2 - \sigma^2(z)} dA
\] (2-31)

\[
\delta\Phi(\bar{\sigma}, \delta\sigma) = \int_A z \delta\sigma(z) dA + \frac{k}{\alpha_0} \int_A \frac{-\bar{\sigma}(z) \delta\sigma(z)}{\sqrt{\sigma_0^2 - \sigma^2(z)}} dA
\] (2-32)

where

- \(\Phi(\sigma)\) = functional equation describing total normal stress
- \(\alpha_0\) = relationship between ultimate shear and normal stress = \(\sqrt{3}\)
- \(k\) = adjustable factor for unit consistency
- \(\sigma_0\) = ultimate normal stress
- \(\bar{\sigma}(z)\) = most favorable distribution of normal stress

Rearranging terms, the expression for the normal stress at any point in the cross-section that allows full plasticity for a given value of \(k\) is given by (2-33). This value, in turn, may be inserted into the general formulations for moment and shear in the cross-section, (2-34) and (2-35), respectively, to produce their respective maximized values for a given value of \(k\). The resulting interaction curve between shear in moment for a given cross-section can then be found by varying \(k\) between values producing maximum shear and maximum moment on the cross-section. Jirasek and Bazant do not provide a resulting equation but state that the resulting interaction equation is nearly circular, i.e. exponents \(a\) and \(b\) both equal to 2 in (2-29).

\[
\bar{\sigma}(z) = \frac{\alpha_0 \sigma_0 z}{\sqrt{k^2 + \alpha_0^2 z^2}}
\] (2-33)
\[
\bar{M} = \int_A z \bar{\sigma}(z) dA = \alpha_0 \sigma_0 \int_A \frac{z^2}{\sqrt{k^2 + \alpha_0^2 z^2}} dA 
\]  
(2-34)

\[
\bar{Q} = \int_A z \bar{t}(z) dA = \frac{k \sigma_0}{\alpha_0} \int_A \frac{1}{\sqrt{k^2 + \alpha_0^2 z^2}} dA 
\]  
(2-35)

where \( \bar{M} \) = maximized moment for a given \( k \)
\( \bar{Q} \) = maximized shear for a given \( k \)

### 2.2.3.7 Combined bending moment, normal force, and shear force

Under combinations of simultaneously acting shear forces, normal forces, and bending moments, Shen (1995) proposed that a three-dimensional interaction hypersurface bounded by the three two-dimensional interactions discussed above can be used practically to define the interaction surface between the three types of forces. In the interaction between forces expressed by equivalence to unity, the respective magnitudes of the force are often expressed normalized by their ultimate values, as given in (2-36) through (2-38) for normal, shear, and bending forces.

\[
n = \frac{N}{N_o} 
\]  
(2-36)

\[
v = \frac{V}{V_o} 
\]  
(2-37)

\[
m = \frac{M}{M_o} 
\]  
(2-38)

where \( n \) = normal force normalized by capacity
\( v \) = shear force normalized by capacity
\( m \) = bending moment normalized by capacity

Using the logic presented in Shen (1995), in general a three-variable hypersurface with known two-variable relationships can be formed as given in (2-39) through (2-44). Generic variables \( A, B, \) and \( C \) are used, with (2-39), (2-40), and (2-41) providing their interactive two-variable unity relationships. The effect of any of the three variables, illustratively \( C \) below, can
be accounted for by normalizing the other two, \( A \) and \( B \), by their limiting relationships with \( C \). To illustrate this below, \( C \) is inserted into the relationship between \( A \) and \( B \) (2-39). For some value of \( C \), the values of \( A \) and \( B \) are constrained by (2-42) and (2-43), respectively, from the relationships defined by (2-40) and (2-41). The respective terms for \( A \) and \( B \) are normalized by their limiting value for a given \( C \), resulting in (2-44).

\[
A^a + B^b = 1 \tag{2-39}
\]

\[
A^c + C^d = 1 \tag{2-40}
\]

\[
B^e + C^f = 1 \tag{2-41}
\]

\[
A = (1 - C^e)^{1/d} \tag{2-42}
\]

\[
B = (1 - C^f)^{1/e} \tag{2-43}
\]

\[
\left( \frac{A}{(1 - C^d)^{1/c}} \right)^a + \left( \frac{B}{(1 - C^f)^{1/e}} \right)^b = 1 \tag{2-44}
\]

To apply the hypersurface generation technique above to the interaction of normal, bending, and shear forces on a cross-section, \( A, B, \) and \( C \) can be replaced with \( n, m, \) and \( v \), as given in (2-45) through (2-50). While the final interaction equation, (2-50), is expressed in terms of the original interaction equation between normal and shear forces (due to the fact that anchor bolts are generally only subjected to combinations of applied shear and normal forces), of course, the same equation could be expressed equivalently with the moment term in either component numerator; i.e., \( m \) could be substituted for \( A \) or \( B \) in the generic relationships given above.

\[
n^a + v^b = 1 \tag{2-45}
\]

\[
n^c + m^d = 1 \tag{2-46}
\]

\[
v^e + m^f = 1 \tag{2-47}
\]

\[
n = (1 - m^e)^{1/d} \tag{2-48}
\]
\[ v = (1 - m_f)^{1/e} \]  \hspace{2cm} (2-49)

\[ \left( \frac{n}{(1 - m^d)^{1/c}} \right)^a + \left( \frac{v}{(1 - m_f)^{1/e}} \right)^b = 1 \]  \hspace{2cm} (2-50)

### 2.2.4 Plastic Hinge Behavior

As a beam experiences bending forces beyond the elastic limit, the plasticized extreme fibers of the critical cross-section no longer contribute to the resistance of additional bending moments. The remaining resistance of this cross-section to additional bending forces is proportional to the reduced moment of inertia of the elastic core of the section. Because the ratio of lost moment of inertia to lost section area is quadratic, the distribution of the plasticized region of the beam along the length takes on a parabolic distribution to maintain static equilibrium. Plasticity continues to spread along the beam until the critical section becomes fully plastic. At this point, the rotation of the beam can be calculated by taking the integral of the curvatures along its length as in (2-51).

\[ \varphi = \int_{0}^{l_{rot}} \frac{M(x)}{EI(x)} dx \]  \hspace{2cm} (2-51)

where
- \( \varphi \) = initial rotation due to onset of full section plasticity
- \( l \) = length over which rotation is being measured
- \( x \) = position along length of rotation
- \( M(x) \) = moment at position \( x \)
- \( E \) = Young’s modulus of material
- \( I(x) \) = moment of inertia at position \( x \)

Beyond the onset of (theoretical) full plasticization, the distribution of strain along the length of the beam is less clear. As a result, for cases where ultimate beam rotation is of interest, assumptions are made about the strain distribution contributing to the ultimate rotation. Figure 2-11 illustrates for a doubly moment restrained beam the progression from (1) the limit of elastic bending moments to (2) semi-plastic moments to (3) the onset of fully plastic bending moments and finally (4) a highly deformed length experiencing double constant curvature over a plastic
Figure 2-11. Development of plastic hinge region in a doubly moment-restrained beam subjected to shear force. Details at A) the yield limit, B) a semi-plastic state, C) the onset of fully plastic bending, and D) a high-strain fully plastic condition.

Lin et al. (2011), which is discussed in greater detail in Section 2.3.3.1, assumed that for anchor bolts with exposed length, a plastic hinge length of one bolt diameter or one-half of the exposed length experiences constant curvature until the strain limit is reached. Only tensile strains were accounted for in the model; rotations due to compressive strains were not considered, likely due to the Poisson’s effects discussed in Section 2.2.3.1. Rotation of the plastic hinge was calculated by multiplying the constant curvature, defined as the inverse tangent
of the strain limit of the material divided by the diameter of the bolt, by the assumed length of the hinge.

2.3 Anchor Bolt Steel Strength

Beyond the general cross-section and beam theory presented above, several experimental programs and analyses have been dedicated specifically to anchor bolt and steel rod applications. The remaining sections in this chapter focus on the evaluation of anchor bolt steel strength in all base plate support conditions to concrete to illuminate gaps in knowledge about the specific case of anchor bolts in stand-off base plate connections.

2.3.1 Normal Strength Capacity of Threaded Rods and Anchor Bolts

Without accounting for contributions to strength from bearing against concrete and grout surfaces in compression, steel strength of anchor bolts and threaded rods subjected purely to normal loads is unaffected by whether the bolt is installed in a flush-mounted base plate connection, ungrouted stand-off base plate connection, or grouted stand-off base plate connection. Axial stiffness, of course, decreases with increasing exposed length, but the implications of changes in axial stiffness are extremely small by comparison to the changes in deformations from shear force with equivalent changes in exposed length.

The tensile strength of the threaded portion of bolts is equal to the ultimate tensile stress capacity of the bolts multiplied by the net tensile area, given in (2-52) based on the provisions of ANSI B1.1 (ASME 2004) as provided in Table 7-17 in AISC (2010). According to Yura et al. (1987), (2-52) underrepresents the tensile stress area owing to an errant assumption that the thread diameter is symmetrical about the pitch line of the bolt and (2-53) should be used as a more correct version accounting for this asymmetry. Nevertheless, (2-52) is used in this dissertation because of its general acceptance in practice and relatively small deviation from (2-53).
\[ A_{nt} = \frac{\pi}{4} \left( d_b - \frac{0.9743}{n} \right)^2 \]  
\[ A_{nt,Y} = \frac{\pi}{4} \left( d_b - \frac{0.9381}{n} \right)^2 \]

where 

- \( A_{nt} \) = net tensile area of threaded rods commonly used in practice
- \( A_{nt,Y} \) = net tensile area of threaded rods proposed by Yura et al. (1987)
- \( d_b \) = unthreaded bolt diameter, in.
- \( n \) = number of threads per inch

Common threaded rod sizes as well as the gross and net tensile areas are presented in Table 2-1 as given by Table 7-17 of AISC (2010). Table 2-1 also contains ratios of the threaded area to the gross bolt area for these bolt sizes, which range from 0.72 to 0.80 for 0.5 in. to 2 in. diameter bolts, respectively.

<table>
<thead>
<tr>
<th>Bolt Dia., ( d_b ) (in.)</th>
<th>Gross Bolt Area, ( A_b ) (in.(^2))</th>
<th>Threads per in., ( n )</th>
<th>Net Tensile Area, ( A_{nt} ) (in.(^2))</th>
<th>( A_{nt}/A_b )</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.5</td>
<td>0.196</td>
<td>13</td>
<td>0.142</td>
<td>0.72</td>
</tr>
<tr>
<td>0.625</td>
<td>0.307</td>
<td>11</td>
<td>0.226</td>
<td>0.74</td>
</tr>
<tr>
<td>0.75</td>
<td>0.442</td>
<td>10</td>
<td>0.334</td>
<td>0.76</td>
</tr>
<tr>
<td>0.875</td>
<td>0.601</td>
<td>9</td>
<td>0.462</td>
<td>0.77</td>
</tr>
<tr>
<td>1</td>
<td>0.785</td>
<td>8</td>
<td>0.606</td>
<td>0.77</td>
</tr>
<tr>
<td>1.125</td>
<td>0.994</td>
<td>7</td>
<td>0.763</td>
<td>0.77</td>
</tr>
<tr>
<td>1.25</td>
<td>1.227</td>
<td>7</td>
<td>0.969</td>
<td>0.79</td>
</tr>
<tr>
<td>1.375</td>
<td>1.485</td>
<td>6</td>
<td>1.155</td>
<td>0.78</td>
</tr>
<tr>
<td>1.5</td>
<td>1.767</td>
<td>6</td>
<td>1.405</td>
<td>0.80</td>
</tr>
<tr>
<td>1.75</td>
<td>2.405</td>
<td>5</td>
<td>1.899</td>
<td>0.79</td>
</tr>
<tr>
<td>2</td>
<td>3.142</td>
<td>4.5</td>
<td>2.498</td>
<td>0.80</td>
</tr>
</tbody>
</table>

At high enough ratios of exposed length to bolt diameter it is conceivable that an anchor bolt or threaded rod could fail in a buckling mode. Bae et al. (2005) stated that inelastic buckling under concentric compression has been produced in rebar at ratios of length to diameter as low as 7 for higher-strength material and 8 for lower-strength material in their survey of existing literature. However, ungrouted bolts with greater than 3 bolt diameter exposed lengths have not been found in field observations or in the literature. In their experimental work, Bae et al. (2005) studied, among other factors, the effect of eccentricity of applied compressive load to the strength of specimens, showing that at higher levels of eccentricity and the same length-to-
diameter ratio of specimens, strength was decreased, indicating an interaction between the
applied compressive load and the moments from the eccentricity of applied load.

2.3.2 Bolt Steel Strength in Flush-mounted Base Plates and Steel-to-steel Connections

2.3.2.1 Shear strength

To determine the shear strength of bolt materials experimentally, Wallaert and Fisher
(1965) conducted tests on individual 7/8 in. diameter and 1 in. diameter ASTM A325, A354, and
A490 bolts installed in two test configurations. Bolt steel grade, diameter, and grip length were
found to have no significant influence on the ratio of shear to tensile strength. The two jigs used
produced different values of shear-to-tensile strength, 0.62 and 0.68. Kulak et al. (1987) stated
that ratio from this testing of 0.62 should be used as a basis for determining pure shear strength
experimentally of bolt materials. This ratio is also used by The American Institute of Steel
Construction Steel Construction Manual (AISC 2010). Kulak et al. (1987) also stated that
pretensioning the bolts had no influence on shear strength, as the low-magnitude axial strains
incurred in the preload process are overcome and their stresses discharged in tests of ultimate
shear strength.

The ratio of threaded shear strength to unthreaded shear strength in bolts is frequently
assumed to be equal to the ratio of the net tensile area, (2-52), to the gross bolt area. Yura et al.
(1987), however, conducted several tests on ASTM A325 and A490 bolts with threads included and excluded from the shear plane, finding that the ratio of threaded to unthreaded strength was 0.83, higher than the net tensile area to nominal area of 0.77 for the 7/8 in. and 1 in. bolts tested due to a larger effective shear area than tensile area for the threaded portion of bolts.

Eligehausen et al. (2006) describe the general transfer of shear forces in a flush-mounted base plate. Any existing pretensioning in the anchor bolt generates compressive frictional resistance to shear between the base plate and concrete surface. Shear force greater than the frictional resistance causes the base plate to slip until the shear is transferred to the bolt. The bolt bears against the foundation until a concrete spall is formed, lowering the effective surface of the concrete and developing bending stresses and catenary tensile stresses in the anchor bolt in addition to the predominating shear stress. When steel failure is observed, the interaction of shear, tensile, and flexural forces within some anchor bolt length causes rupture as demonstrated in Figure 2-13 and described in Paulay and Park (1974).

![Figure 2-13. Illustration of steel shear failure of an anchor bolt in a flush-mounted base plate.](image)

Models accounting for the effects of the concrete support to an anchor bolt exposed to shear load have been developed assuming the anchor bolt is a beam supported by an elastic/elasto-plastic foundation. The first such model was proposed by Friberg (1940), which, according to Eligehausen (2006), predicts failure loads on the order of half of experimental
values. Given the complexity of these formulations, the inaccuracy of predictions, and that anchor bolt strength does not vary widely from steel-to-steel connections in these flush-mounted connections, it is standard practice to characterize steel shear failure in anchor bolt connections proportionally to the tensile strength of the steel as in steel-to-steel connections. Eligehausen et al. (2006) proposes a constant of 0.6 to describe the relationship between shear and tensile capacity of anchor bolts installed in normal strength concrete as a result of the aggregation of German and American shear strength tests, a value also adopted by ACI 318-11 (ACI 2011) and AASHTO LTS-6 (AASHTO 2013), also in good agreement with the steel-to-steel strength of 0.62 given by Kulak et al. (1987).

Grosser (2012) performed anchor bolt steel shear tests as part of a comprehensive research program on the connection strength of flush-mounted base plates studying concrete, steel, and combined failure modes. Material strength, bolt ductility, concrete strength, and embedment depths were investigated for their influence on steel shear strength.

In one set of tests, medium-strength concrete (4600 psi), high-strength concrete (9700 psi), and a rigid steel base were used as foundation materials for anchor bolts of three grades and three sizes. While concrete strength influenced the load-displacement and ultimate load, its influence on shear strength was not quantified due to the complexity of bolt bending forces within the depth of spall, particularly in the low-strength concrete, as discussed above. By inspection of reported COV values, the differences in ultimate strength due to concrete material do not appear to be statistically significant. Material strength and ductility, however, were correlated with the ultimate shear strengths; proposed correction factors were established with anchor bolt shear capacity inversely proportional to steel strength and directly proportional to steel ductility. These
results support the postulation in Section 2.2.3.1 that the shear area is larger than the net tensile area at failure due to Poisson’s effects during tensile tests.

In a separate set of tests by Grosser (2012), the effect of embedment depth on anchor bolt shear strength was studied for low-strength (3300 psi) and medium-strength (5000 psi) concrete. In the low-strength concrete, anchor bolt shear strengths for embedment depths of 5 nominal diameters were approximately 20% lower than bolts installed with embedment depths of 8 bolt diameters. No difference, however, was observed in strength in the medium-strength concrete between the same embedment depth ratios. Ultimately, Grosser (2012) proposed (2-54) to characterize flush-mounted anchor bolt shear strength, \( V_{0Rk,S} \), in design.

\[
V_{0Rk,S} = k_{s1} \cdot k_{s2} \cdot k_{s3} \cdot A_s \cdot f_{uk}
\]

where

- \( k_{s1} = 0.6 \) for \( f_u < 500 \) MPa
- \( k_{s1} = 0.5 \) for \( 500 \) MPa \( \leq f_u \leq 1,000 \) MPa
- \( k_{s1} = 0.4 \) for \( f_u > 1,000 \) MPa
- \( k_{s2} = 1.0 \) for anchor bolts or unwelded headed studs
- \( k_{s2} = 1.2 \) for embed plates with headed studs
- \( k_{s3} = 0.8 \) for \( A_5 \) elongation \(< 16\%\)
- \( k_{s3} = 1.0 \) for \( A_5 \) elongation \(\geq 16\%\)
- \( A_5 \) = rupture elongation measured across 5 bolt diameters
- \( A_s \) = cross-sectional area of steel
- \( f_{uk} \) = nominal characteristic ultimate tensile strength
- \( f_u \) = actual ultimate tensile strength

2.3.2.2 Combined normal and shear forces

![Combined normal and shear forces acting on a flush-mounted base plate.](image)
Chesson et al. (1965) conducted bolt steel strength tests on 3/4 in. and 1 in. dia. ASTM A325 and A354 bolts under combinations of tension and shear loading in a steel-to-steel connection testing apparatus. Tests were conducted with seven unique tension-to-shear ratios ranging from pure tension to pure shear in an angled load application experimental. In addition to the ratio of applied tensile to shear force, the presence of threads in the failure plane, bolt material, test block material, and bolt diameter were also considered for their influence on the tests. While Chesson et al. (1965) offered different constants to shear values based on bolt type and presence of threads in the failure plane, Kulak et al. (1987) suggested the following relationship to describe the strengths of all steel-to-steel fasteners subjected to combined tension and shear given in (2-55) to unify the results.

\[
\left(\frac{x}{0.62}\right)^2 + y^2 = 1.0
\]  \hspace{1cm} (2-55)

where 
- \(x\) = ratio of shear stress to tensile stress capacity
- \(y\) = ratio of tensile stress to tensile stress capacity

For anchor bolts in flush-mounted base plates connected to concrete foundations experiencing combined tension and shear, slight variations in the exponent values given for steel-to-steel connections in (2-55) have been proposed. McMackin et al. (1973) performed tests of combined tension and shear on anchor bolts, finding that an exponent of 5/3 fit the data appropriately. Similarly, Lotze et al. (2001) performed an extensive test program of individual anchor bolts subjected to combined tension and shear forces. The test setup contained an angled member that reacted against a concrete block hosting anchor bolt specimens. Results were evaluated against different exponent values in the tension-shear interaction equation, the authors concluding that exponent values between 1.67 and 1.8 are appropriate for anchored connections to concrete. ACI (2011) currently uses the 1.67 exponent to envelope all failure modes in combined tension and shear loading.
Cook and Klingner (1992) described the work conducted in Cook (1989) on anchor bolt behavior and strength on a variety of multiple-bolt connections with rectangular flush-mounted base plates. Varying combinations of shear and overturning moment using the test setup shown in Figure 2-15. Tests ranged from nearly pure shear applied to the base plate to predominantly overturning moment. ASTM A193 B7 threaded rod was used for all tests.

![Figure 2-15. Schematic of test setup given in Cook and Klingner (1992). (Figure courtesy of Cook and Klingner 1992)](image)

Beyond moment-to-shear ratio, other parameters studied included various arrangements of anchor bolts, number of bolts, base plate flexibility, and friction contributions to connection strength. An analytical model characterizing the shear strength of a group of anchor bolts, $V_{ut}$, applied at an eccentricity $e$ was produced to validate the experimental program. Based on critical values of $e$, ranges of shear- and moment-dominated behavior were defined. Shear-dominated behavior ($e < e''$) was defined as the range in which applied shear forces exceeded the resultant steel-to-concrete friction forces caused by the moment. For the shear-dominated range of eccentricities, (2-56) was developed, which includes the elliptical representation of anchor bolt tension and shear forces in (2-28). In moment-dominated behavior ($e > e''$), bolt shear did not contribute substantially to failure and the bolt group strength was characterized by Equation
While it did not affect the calculated value of bolt group steel strength, a second critical eccentricity \( e' \) defined the threshold between base plate slip and lack of slip due to friction.

\[
V_{ut} = \gamma T_0 ma + \frac{\sqrt{n^2(a^2 + b^2) - m^2b^2}}{a^2 + b^2}
\]  
\[ (2-56) \]

\[
V_{ut} = \frac{nT_0 d}{e}
\]  
\[ (2-57) \]

where
- \( \gamma \) = ratio of shear to tensile strength (assumed to be 0.5 for anchor bolts)
- \( T_0 \) = ultimate tensile strength of the anchor
- \( m \) = number of rows of anchors in the compression zone of bending
- \( a = 1 - \frac{\mu e}{d} \)
- \( n \) = number of bolts
- \( b = \frac{ye}{d} \)
- \( d \) = distance from compressive reaction to tension anchors
- \( \mu \) = coefficient of friction between the base plate and concrete
- \( e \) = eccentricity of applied load; equal to ratio of base plate moment to shear

**Critical eccentricities:**
- \( e'' \) = minimum eccentricity for connections with no tension-shear interaction
  \[ e'' = \frac{nd}{n\mu + ny} \]
- \( e' \) = minimum eccentricity for connections experiencing no base slip
  \[ e' = \frac{d}{\mu} \]

Based on tests of concrete friction resistance in Cook (1989), a coefficient of friction between a steel base plate and concrete foundation of 0.43 was found, with 0.4 proposed for practical applications. As high hole tolerances, low steel ductility, and the number of anchors in line with applied load can all affect the ability for the system to distribute load equally between multiple anchors, Cook and Klingner (1992) recommend that \( \gamma = 0.5 \) for adhesive and cast-in-place anchors and 0.6 for sleeved anchors. The 0.5 factor was proposed to account for the fact that anchor bolts may not be distributed equally within their oversize holes, resulting in one anchor failing at lower levels of connection displacement than the others and a lower average load between the bolts.
2.3.3 Bolt Steel Strength in Ungrooved Stand-off Base Plate Connections

2.3.3.1 Pure shear force

Figure 2-16. Shear force acting on an ungrouted stand-off base plate.

Experimental investigation of the strength of anchor bolts has been limited to specimens installed in steel-to-steel loading apparatuses. The first such study was conducted by Scheer et al. (1987), who performed singly moment-restraining tests of anchor bolts with varying exposed lengths from a steel base plate to a point of load application in a direct shear and combined tension and shear test apparatus. The direct shear test setup is illustrated in Figure 2-17 and the combined tension and shear setup is described and illustrated in the next section. A steel base plate hosted two identical bolt specimens, which were then attached to individual loading plates, one connected to an applied load, the other to a reaction load. As a result, equal load was applied the individual specimens in opposite directions. The connection between the loading plates and the anchor bolt specimens allowed bolt rotation, hence specimens were loaded in single curvature. In practice, the rotation of bolts is often restrained on both ends of the exposed length.
Based on the work by Scheer et al. (1987), Eligehausen et al. (2006) and *fib* (2011) use (2-58) to characterize the theoretical shear strength, $V_{t,t}$, of a stand-off base plate assembly purely as a function of tensile stresses due to the bolt moment. Figure 2-18 provides dimensions used in this formulation. It was stated that where the distance from the concrete surface to the center of shear loading is greater than one bolt diameter, any shear influence on bending moment at failure may be neglected. This simplifying assumption is clearly made possible by the conservative representation of the bending lever arm from a distance one-half diameter below the concrete surface (to account for concrete spalling) to the center of load application on the base plate irrespective of the presence of a leveling nut below the base plate.

$$V_{t,t} = \alpha V_M M_{u,s}$$  \hspace{1cm} (2-58)

where \( V_{u,s} \) = theoretical shear strength  
\( \alpha_M \) = correction factor for base plate restraint condition  
\( = 1.0 \) when rotation is not restrained (single curvature)
= 2.0 when rotation is restrained (double curvature)

\( M_{u,s} \) = possible bending moment on the section = \( M_{u,s}^0 \) for pure shear

\( M_{u,s}^0 \) = fully plastic bending moment = \( 1.7W_{el}f_y \)

\( W_{el} \) = elastic section modulus of threaded portion of the bolt

\( f_y \) = measured steel yield stress

\( l \) = shear load lever arm = \( e_1 + a_3 \)

\( e_1 \) = distance from concrete surface to base plate mid-thickness

\( a_3 \) = 0.5d when no nut is fastened to the concrete surface

= 0 when nut fastened to concrete surface

Figure 2-18. Dimensions used in Eligehausen et al. (2006b) for anchor bolt bending stresses.

Lin et al. (2011) conducted physical and simulated analytical tests of bolts with exposed length in a different steel-to-steel testing approach. The physical test setup included two 5/8 in. dia. B7 threaded rods loaded simultaneously in shear varying exposed lengths varying from 0.2 to 8 bolt diameters with finger-tightened nuts. Effects of oversize holes were also tested. The experimental setup included vertical restraint of the deformed bolts, similar to installations with grout pads, shim stacks or other stiff materials placed below the base plate, but not necessarily
installations with ungrouted stand-off base plates. The vertical restraint caused unmeasured tensile forces to act on the bolts, changing the state of bending stress.

Figure 2-19. Simplified illustration of the test schematic used in Lin et al. (2011).

Three zones of load displacement behavior were characterized for bolts with 2 diameter exposed lengths or greater: 1) an initial linear-elastic segment, 2) an approximately linear strain-hardening segment, and 3) tensile engagement of the bolt until rupture. Tensile contributions (Zone 3) to the observed failure strengths were significant due to the vertical restraint of the test specimens, as those tension forces act over the horizontal displacement of the bolts, releasing bending moments and thereby increasing the strength and stiffness of the specimens. Additional rotation within oversize holes allowed further tensile engagement, resulting in ultimate strengths between 16% percent and 36% higher than snug-hole counterparts for 2 diameter and 4 diameter exposed lengths, respectively.
Equation (2-59) characterizing the shear strength, $V_{se}$, of bolts with exposed lengths was proposed, adding a conservative linear interaction of shear and moment (2006) and accounting for the tensile contribution to the resistance of the applied load through bolt tension. The rotation angle, $\beta$, is estimated by summing potential rotations from an oversize hole and from the rotations that would correspond to bold rupture from tensile strains due to bending within a plastic hinge. As described in Section 2.2.4, plastic hinge rotation was estimated by assuming constant curvature over the length of the hinge. Any rotations allowed by the oversize holes were added to the plastic curvature for the $\beta$ value. For values of $l$ less than two anchor bolt diameters, it is suggested that $\beta$ be taken as 0, simplifying to (2-60).

$$V_{se} = f_{ya}A_{se,v} \sin(\beta) + \frac{f_{ya} \cos(\beta)}{1.0} + \frac{l}{0.9A_{se,v} + 3.4S}$$ (2-59)

$$V_{se,\beta=0} = \frac{f_{ya}}{0.9A_{se,v} + 3.4S}$$ (2-60)

where

- $V_{se}$ = predicted steel strength based on ultimate rotation of anchor bolt
- $V_{se,\beta=0}$ = predicted steel strength for no bolt rotation
- $f_{ya}$ = yield strength of the bolt material
- $A_{se,v}$ = cross-sectional area of bolt
- $\beta$ = bolt rotation with respect to initial shape (see above)
- $l$ = exposed length of the bolt
- $S$ = section modulus of the bolt

The proposed equations fit the finite element analysis data well with the given constraints, with experimental data from oversized holes falling between two assumed values of bolt rotation. However, the following limitations to the analytical equation have been identified by the author of this dissertation:

- *Yield* stress capacity, assumed to be approximately 0.6 of the ultimate stress capacity, was used to model *ultimate* strength behavior.
• The ratio of shear capacity to tensile capacity, given in this dissertation as \( \alpha \) and approximately equal to 0.6, is not directly accounted for; instead, the model relies on the ratio of yield to ultimate stress to approximately equal the shear-to-tensile strength ratio.

• Tensile stresses from the tension force that develops are not accounted for in the interaction equation.

• The linear interaction equation used between shear and moment stresses is conservative compared to even the lower-bound methods described in Section 2.2.3.1.

Liu (2014) performed finite element analyses on a cantilever sign structure with an annular base plate subjected to a realistic design torsion load, which resulted in pure shear applied to individual anchor bolts in the model from the resolved torsion force. Base plate thickness and number of bolts were investigated. Significant bending stresses were reported in bolts, even for exposed lengths equal to and less than one anchor bolt diameter. As base plates became more flexible, maximum bending moments on bolts increased non-proportionally due to the loss in rotational resistance at the base plate, magnifying the moments at the bottom of the exposed length (toward the single moment restraint condition). A simplified version of the interaction equation between moment and tensile forces was proposed with limiting values of allowable moment. The effects of shear stresses and their interaction with normal stresses, however, were not discussed. It was concluded that experimental study is needed to investigate the AASHTO (2009) allowance for ignoring bending stresses in bolts in ungrouted stand-off base plate connections with up to one diameter of exposed length based on the work completed by Kaczinski et al. (1998). It was further concluded that base plates with greater than one anchor bolt diameter of thickness should be used to preclude the possibility of moment amplification on anchor bolts due to base plate flexibility.
2.3.3.2 Combined normal and shear force

Figure 2-20. Combined normal and shear forces acting on an ungrouted stand-off base plate.

Scheer et al. (1987) also performed tests on specimens loaded in combined tensile and shear force in a similar experimental approach to their direct shear tests described in the previous section. A schematic of the combined tension and shear test setup is provided in Figure 2-21, where a circular steel base plate allowed specimens to be made with various combinations of applied tension and shear force. As with the direct shear setup, the loading plates on this test setup allowed rotation of the bolts, resulting in single-curvature test specimens.

Again based on this work by Scheer et al. (1987), Eligehausen et al. (2006) and fib (2011) provided a design equation for the permissible value of moment, $M_{u,s}$, based on a conservative linear interaction between moment and axial forces (2-61). The reduced value of $M_{u,s}$ is then placed into (2-58) to obtain a design value of allowable shear force.

$$M_{u,s} = M^0_{u,s}(1 - N/N_u)$$  \hspace{1cm} (2-61)

where

- $M_{u,s}$ = possible moment
- $N$ = applied tension
- $N_u$ = tensile capacity of anchor bolt
No steel strength tests have been conducted on anchor bolts in ungrouted stand-off base plate connections including concrete. As part of a much larger study focusing on the fatigue performance of cantilever sign and signal structures, however, Kaczinski et al. (1998) applied low levels of load to ungrouted stand-off base plates in realistic cantilever sign assemblies to determine the effects on the elastic distribution of stress on anchor bolts. The following were identified as variables affecting the performance of anchored base plate connections: misalignment of anchor bolts (i.e. out-of-plumbness), exposed length of the bolts, and nut tightness (i.e. pretensioning). Test specimens included a group of eight 1.5 in. diameter Grade 55 bolts installed at a 21 in. bolt circle diameter with bolt exposed lengths of 1 in. Two anchor bolt plumbness values (perfectly plumb and 1:20 out-of-plumb) and two base plate thicknesses (1 in. and 1.5 in.) were studied, producing four unique assemblies. Four unique Moment:Torsion:Shear ratios were applied to each of the four assemblies, producing sixteen loading cases. Using two of
the load ratios and two separate values of anchor bolt installation plumbness, additional tests were conducted with 3 in. bolt exposed lengths. Load was applied monotonically to what was reported as half of the yield strength for every test.

To measure the effect of applied load on the anchor bolts, four uniaxial strain gages were installed at 90 degree intervals on the exposed portion of each anchor bolt. The authors of the study acknowledged the presence of bolt bending from shear forces produced by direct shear and torsional base plate forces. To isolate bolt bending in tests with applied overturning moment and no torsion to the connection, strain gage readings from bolts positioned most closely to the neutral axis of the base plate overturning moment were used. Calculated bolt bending stresses from strain gage readings were compared to theoretical bending stresses assuming the bolts to be fixed-fixed beams with the direct shear load distributed equally between the eight bolts. The theoretical bending moments were reported at levels between 40% and 90% higher than calculated moments from strain gauge readings for the bolts with 3 in. exposed length. Greater discrepancy was reported between calculated and theoretical bolt bending moments in the 1 in. exposed length tests.

On bolts with axial stress due to base plate overturning moment, bending stresses were less than ten percent of the total stresses for these bolts, leading to the conclusion that bending stresses may be ignored for anchor bolt exposed length less than one bolt diameter. Although tests were performed with a relatively high magnitude of applied torsion relative to connection overturning moment, it does not appear that the resulting bending stresses in the bolts from these types of tests were included in the assessment of the magnitude of bending stresses resulting from all sources of anchor shear compared to anchor axial stresses from overturning moment.
Beyond anchor bolt exposed length, it was also concluded that base plate thickness should be at least one bolt diameter to prevent prying action from a relatively flexible (to the bolts) base plate.

**2.3.3.3 Summary and open areas of research in ungrouted stand-off base plates**

Anchor bolts in field installations of ungrouted stand-off base plate connections, e.g. an annular base plate connection between a steel pole and a concrete foundation as described in Kaczinski et al. (1998) are generally 1) doubly moment restrained over the exposed length (leveling nut and concrete surface) and 2) without axial restraint. In the laboratory, application of a shear force to anchor bolt in an ungrouted stand-off base plate connection presents an experimental challenge, as the shear force acting over the lever arm of the bolt produces overturning moments that must be restrained to satisfy force equilibrium. Scheer et al. (1987) and Lin et al. (2011) developed unique and innovative solutions to this challenge in their steel-to-steel testing schemes discussed earlier. However, neither Scheer et al. (1987) nor Lin et al. (2011) contained both double moment restraint and lack of vertical restraint in their approaches. In addition, both studies utilized steel-to-steel connections to represent steel-to-concrete connections. The influence of concrete was assumed in Eligehausen et al. (2006) based on observations of concrete spalling and not accounted for in the model presented by Lin et al. (2011). No tests have been conducted with combinations of compression and shear forces. Furthermore, simplifying assumptions made in the analytical models from Eligehausen et al. (2006) and Lin et al. (2011) leave room for refinement to describe the steel strength of anchor bolts in ungrouted stand-off base plate connections.

The gaps in experimental testing of anchor bolts in ungrouted stand-off base plate connections exposed to shear force from the experiments presented in the preceding paragraphs result in open areas for research. Specifically, gaps include 1) tests with combined double moment restraint and freedom to displace downward, 2) tests with steel base plates connected to
concrete base material, 3) tests with applied compression and shear loading, and 4) analytical models of steel strength that include shear, normal, and flexural deformations.

2.3.4 Bolt Steel Strength in Grouted Stand-off Base Plate Connections

2.3.4.1 Combined normal and shear force

Figure 2-22. Combined shear and normal forces acting on a grouted stand-off base plate.

All of test programs found investigating anchor bolt steel strength in grouted stand-off base plates used many combinations of shear and tension force. Thus, this section is not split into “pure shear” and “combined axial and shear” studies as in previous sections.

Adihardjo and Soltis (1979) studied the steel strength of single 0.5 in. dia. anchor bolts with 1 in. grout pads subjected to varying angles of applied load. Failure was defined as yielding of the anchor bolt, not ultimate strength. Following this definition, tests were not run to anchor bolt steel rupture. Results were plotted against variants of the elliptical tension/shear interaction equation given in (2-28). Tests with a low ratio of applied shear to tension agreed well with the interaction curves; however, as the ratio of shear load to tension load increased to 1 and beyond,
initial yielding was lower than predicted by the interaction equations assuming a shear yield strength of 0.6 of the tensile yield strength. In the shear-only cases, Adihardjo and Soltis (1979) yield strength of approximately 0.55 of the tension strength.

Nakashima (1998) performed single-anchor ultimate strength tests with combined tension and shear at various angles of applied load, including cases with direct tension and direct shear. All tests contained a 20 mm. base plate and 20 mm. grout pad with no leveling nut below the base plate. The effect of embedment depth and the influence of threads in the shear plane on the strength of anchors exposed to combined loads was also studied. It was found that test values exceeded the capacity predicted by the elliptical interaction equation between applied tension and shear stresses. No observable difference was found between tests where the shear plane was located within the threaded portion and where the shear plane was located at the intersection of the threaded and unthreaded portion of the bolts, meaning that potential stress concentrations from the transition did not appear to play a significant role in ultimate strength. Where the shear plane was in the unthreaded portion of the bolt, interestingly, the values appear to be higher than would be predicted by the increase in net area of the bolt.

Gresnigt et al. (2008) summarized the shear strength of two- and four-anchor bolt grouted connections to square base plate with from tests conducted at TU-Delft in 1992 with and without applied tension on the connection. Four-bolt tests were performed with 20 mm (0.79 in.) dia., Grade 4.6 bolts at 130 mm x 130 mm spacing, while two-bolt tests were performed with 20 mm dia. grade 8.8 bolts at 130 mm spacing. Grout pad thickness varied from 15 mm to 60 mm. No leveling nut was placed below the base plate. Results from 22 tests were reported, six with no applied tension to the bolt group and 16 with applied tension ranging from 121 kN to 200 kN.
All but six of the tests resulted in steel rupture, while in the others failure was attributed to concrete modes.

Predictive elastic limits and subsequent plastic load-displacement behavior of anchor bolt shear resistance, $F_h$, was derived in (2-62), where the resistance to applied shear load depended on tension forces development by geometry of bolt bending and contributions from grout pad friction and elastic compression. The model assumes a linear relationship between the applied load and displacement until bolt yielding corresponding to the value of $\delta_h$, provided below. Coefficients of friction, $f_w$, of 0.2 and 0.3 were proposed for sand-cement mortar and “special grout (e.g. Pagel IV),” respectively.

$$F_h = \frac{f_y b A_{b,s}}{\sqrt{\delta_h^2 + v_r^2}} (\delta_h + f_w v_r) - f_w F_t$$  \hspace{1cm} (2-62)

where

- $f_{y,b}$ = yield strength of the bolt material (assumed as 0.8 times ultimate)
- $A_{b,s}$ = cross-sectional area of bolt
- $\delta_h$ = base plate horizontal displacement; elastic limit = $v_r \sqrt{\frac{2 F_{y,b}}{E}}$
- $E$ = elastic modulus of the bolt
- $f_w$ = coefficient of friction between the grout pad and the base plate
- $v$ = grout pad thickness
- $v_r$ = effective grout pad thickness = $v + 0.5d_b$
- $d_b$ = nominal anchor bolt diameter
- $F_t$ = applied tensile force (assumed known)

This model provides load-displacement behavior for a bolt in a grouted stand-off base plate but does not directly predict ultimate strength. With post-failure knowledge of the measured values of ultimate deflection corresponding to ultimate loads for each test, a corresponding predictive value of ultimate load can be found. While the rational basis for this model is strong, the author of this dissertation identifies the following limitations:

- No methodology for determining the ultimate displacement, which is essential for determining the ultimate load with the model, are provided beyond an acknowledgement that the displacement is related to the ductility of the material.
• Bolt material yield strength was used (arbitrarily chosen as 0.8 times the ultimate strength) to describe ultimate load behavior.

• The model does not account for values of tension force high enough to overcome the frictional resistance of the grout pad; a bolt experiencing full tension loading, for instance, is allowed to maintain full shear force by the model.

• The influence of shear stresses on the bolt cross-section, which more negatively impact the strength of the bolt than tensile stresses, was not integrated into the model.

It was concluded that the ultimate shear resistance of an anchor bolt is nearly independent of the thickness of the grout layer. Given the constraints of the model in predicting ultimate strength (i.e. no criteria for ultimate deflection) and that no trend was observed in bolt strength relative to grout pad thickness, a simplified approach was proposed for design where constants accounting for the ductility of the two materials were multiplied by the ultimate tensile strength of the bolt to approximate shear strength. Applied tension was not accounted for, allowing extreme cases such as simultaneously applied full tension and shear load by the model. Thus, gaps are left in the generality of the simplified approach.

Experimental results ranged from 1.4 to 2.6 times the predicted values of the simplified method. Results were also compared against existing European standards utilizing the approach given in (2-58), which takes the “lever arm” of bolt bending over a length from the center of the base plate to 0.5 bolt diameters below the concrete surface and does not account in any way for the presence of grout. Unsurprisingly, experimental results from Gresnigt et al. (2008) failed at 3.3 to 26.2 times the values predicted by the European standard method.

In a similar experimental approach to Gresnigt et al. (2008), Gomez et al. (2011) studied cyclic shear transfer of a large column connection to a concrete foundation under constant axial load in two tests with 0.875 in. and 1.25 in. F1554 Gr. 55 anchor bolts, respectively. As with all other tests, no nuts were placed below the base plate, which was set and leveled using actuators in the test setup. Constant tension was applied to the bolt groups at approximately $0.3T_o$ and
0.4T_o, respectively. A cyclic displacement-controlled shear loading regimen was then applied to the bolt group until failure.

Results in Gomez et al. (2011) were compared to two calculations, both using an elliptical interaction equation given in between shear and tensile stresses, where tensile stresses included the applied axial force and maximum bolt bending stresses over the distance from the center of a welded plate washer to the top of the grout pad. It was acknowledged that the combination of bending and normal tensile stresses is inadmissible for the fully plastic failure condition. Bolt rupture occurred at the bottom of the grout pad, so it is not immediately obvious what the calculated moment value represents in the failure of the bolt. This method produced a predicted value of failure on the order of ½ of the test values.

To improve the predictive calculation, a tensile component of load resistance of the bolts due to their displaced shape was added to the resistance of shear force. This contribution was taken as the applied tensile force, a constant, multiplied by the proportion of the horizontal deflection over the length of bolt bending from top of the grout pad to the middle of the welded plate washer. As in Gresnigt et al. (2008), the model predicted post-yield behavior - albeit through a different mechanism of tensile contribution from the applied loads instead of from the clamping action of the grout pad - but not ultimate strength as there is no methodology was provided for determining ultimate displacement. Applied tension was not considered for its influence on the assumed bending stresses in the calculations provided by Gomez et al. (2011) and no discussion of clamping action of the displaced base plate against the grout surface and its resulting was discussed.

In addition to the tests that studied anchor bolt capacity, Gomez et al. (2008) conducted three tests investigated the friction characteristics of the base plate assembly with and without
shim stacks, finding that previously established coefficients of friction are appropriate for steel base connections to grout with and without shim stacks. A grout-to-base plate coefficient of friction was found to be 0.46, very close to that found by Cook and Klingner (1992).

Fichtner (2012), in a German dissertation, conducted several tests bolts installed in rectangular stand-off base plate connections using grout pads. Tests were conducted at various loading eccentricities and grout pad heights. As with the other experiments discussed in this subsection, leveling nuts were not used below the base plate. Practical design-oriented equations were proposed with a reduction in shear strength accompanying increases in grout pad height.

2.3.4.2 Summary and open areas of research in grouted stand-off base plate connections

In the available literature for grouted stand-off base plate connections, as with ungrouted connections, opportunities for further study remain. None of the studies included a leveling nut below the base plate, a very common installation condition in the field; without a leveling nut, the base plate would have to be installed directly onto a pre-set grout pad or leveled using some other method. The implications of adding a leveling nut include the rotational behavior of the exposed length of the bolt and the exposed length itself. No tests have been conducted on the steel strength of anchor bolts with shim stacks used for leveling the base plate, which may or may not replicate the behavior of the grouted stand-off base plate condition. As with ungrouted stand-off base plate connections, existing analytical models have room for improvement, most notably in the inclusion of shear stresses on bolts and criteria for the ultimate displacement of the bolts. In line with the gaps, opportunities for further study include 1) tests with leveling nuts below the base plate, 2) tests investigating the effects of, e.g., low-strength dry-packed grout, shim stacks, and an FRP retrofit around grout pads, and 3) development of more robust analytical models of bolt steel strength in grouted stand-off base plate connections.
2.4 Influence of Other Connection Conditions

2.4.1 Base Plate Rigidity

The resolution of load transfer from the base plates to anchor bolts is dependent on the rigidity/flexibility of the base plate as described by Cook and Klingner (1992). In a fully rigid base plate, axial connection forces transfer to anchor bolts in proportion to their relative stiffnesses, while overturning moments transfer to anchor bolts in proportion to their distance from the neutral axis of bending and their relative stiffnesses. Where the base plate bears against concrete (flush-mounted base plate connection) or grout (grouted stand-off base plate connection), the equilibrium of forces can be determined with a compressive reaction from overturning base plate forces at the extreme bearing distance on the concrete or grout.

A “flexible” base plate can be defined as one in which significant bending occurs within the base plate before failure of anchor bolts, shifting the location of the neutral axis of bending. The hinged base plate causes a redistribution of loads and reactions depending on support conditions, often resulting in “prying” action on the anchor bolt from the less favorable connection moment resistance. In flush-mounted and grouted connections, hinging of the base plate moves the compressive reaction, at its most extreme, to the location of the hinge, decreasing the depth of the moment arm resisting bending and thereby increasing resultant compression forces on the concrete/grout and tensile forces on anchor bolts.

As discussed in earlier sections, Kaczinski et al. (1998) and Liu (2014) both suggest that base plates be designed to be rigid to avoid negative consequences including unfavorable moment distribution on bolts and prying action. If more information on the distribution of forces through flexible base plates is desired, the reader is pointed to Wald et al. (2008), which provides comprehensive models for base plate stiffness and force transfer to bolts.
2.4.2 Connection Pretensioning

While it has been shown to have no impact on ultimate tensile strength of bolts, pretensioning by the “turn-of-the-nut” method brings the tensioned length of bolt into the initial stages of yield behavior (Kulak et al. 1987) and is recommended in NCHRP 469 (Dexter and Ricker 2002) for better anchor bolt performance under fatigue loading for sign and signal structures. The Federal Highway Administration Guidelines for the Installation, Maintenance, and Repair of Structural Supports for Highway Signs, Luminaires, and Traffic Signals (FHWA 2002) provides a procedure for tightening anchor bolts in a star pattern using the turn-of-the-nut method, which is outlined in greater detail in Chapter 4 for how pretensioned test specimens were treated.

2.4.3 Influence of Grout on Stand-off Base Plate Connection Behavior

Cook et al. (1995) and Cook et al. (2000a) studied linear load-displacement behavior of stand-off base plate connections for sign/signal structures with and without grout pads below the base plate under high ratios load of moment to shear on the base plate. Cook and Bobo (2001) synthesized this work, providing guidelines and methodology for determining the required base plate thickness, required tensile stress area of bolts, and contributions to the overall rotation of the structural member from base plate components.

To explore the corrosion potential of a grout pad installed with a flowable, formed grout, Cook et al. (2000b) studied the corrosion of bolts with and without grout pads exposed to 14 weeks of rotating between saltwater immersion and free air exposure up to the base plate. The two grouts studied exhibited different water sorptivity levels. It was found that after 14 weeks, the bolts with grout lost nearly no section to corrosion, while the bolts that were ungrouted rusted significantly. Between the two grouts studied, no difference in weight loss was found. It was concluded that the properly installed grout protected the bolts rather than exacerbating corrosion.
Properly-installed, non-shrink, flowable grout installed in a form was recommended to mitigate corrosion of anchors in standoff base plates.

![Anchor bolts after 14 weeks of wetting cycles. A) Bolts in ungrouted stand-off base plate. B) Bolts in grouted stand-off base plate. (Photos courtesy of Cook et al. 2001b).](image)

Kaczinski et al. (1998) stated that dry-pack grout or mortar pads; i.e. those installed with a stiff grout or mortar and finished with a trowel, should not be used based on the potential for corrosion from water pooling. Dry-packed grout pads, in addition to the potential for interstices between packed layers through which water may flow and be retained, are often installed against a duct-taped perimeter around the outside of the bolt circle, providing even greater corrosion potential.

### 2.5 Summary

Based on the review of applicable literature, the variables most significantly affecting the steel strength of anchor bolts in stand-off base plates are 1) the exposed length of anchor bolts and 2) base plate support condition (i.e. flush-mounted, ungrouted stand-off, and grouted stand-off base plates) and 3) the ratio of applied axial to shear load. Other variables found in the literature that may influence anchor bolt steel strength in stand-off base plate connections include hole oversize, bolt diameter, installation type (i.e. cast-in-place (CIP) vs. adhesive anchors), pretensioning of bolts, bolt steel strength/ductility, location of the bolt threaded/untethered
interface, concrete strength, base plate flexibility, grout material strength, grout pad installation
technique (i.e. dry-packed vs. formed), and various grout pad size and shape parameters.

Many combinations of base plate support condition, stand-off distance, and ratios of
normal to shear force applied to individual bolts are possible in field installations and
experiments simulating them. While several of the variables given above have been investigated
in the literature, major gaps that this dissertation addresses were identified in the combinations of
the variables and the analytical models used to represent both ungrouted and grouted stand-off
base plates.

In ungrouted stand-off base plates, no tests have been conducted on anchor bolt steel
strength with concrete. In the studies that used steel-to-steel loading apparatuses, test setups did
not necessarily represent end conditions that would be present in field installations. It is
concluded that both the shear strength and combined shear and normal strength remain open
problems for investigation. Beyond anchor bolt exposed length, material type, influence of
concrete, pretensioning, and the potential for size effect have also not been studied.

Grouted stand-off base plates have been studied more extensively in the existing literature,
including behavioral models that account for grout contributions to anchor bolt steel strength
from vertical restraint, friction, and anchor bolt displaced geometry. While these models contain
strong analytical bases, they do not include criteria for ultimate anchor bolt steel strength and
none account for the interaction of stresses on the critical cross-section of the anchor bolt. No
tests of bolt steel strength have included leveling nuts on the bolts below the base plate, which
are used extensively in field installations.
CHAPTER 3
CODE COMPARISONS

3.1 Introduction

The American Association of State Highway and Transportation Officials (AASHTO 2013a), the American Concrete Institute (ACI 2011), and the American Institute of Steel Construction (AISC 2010) provide Load and Resistance Factor (LRFD) code provisions for the design of anchor bolts. Design bolt steel strength values including LRFD $\Phi$ factors from these code bodies are compared within this chapter for axial, shear, and combined axial and shear loads. In addition to these LRFD standards, AASHTO also provides an Allowable Stress Design (ASD) approach for highway sign, signal, and lighting structures (AASHTO 2013b). For the sake of comparison, all values are given for the threaded portion of the anchor bolt. If unthreaded properties are desired, the nominal area could be substituted (or reduction factors accounting for threads removed) in calculations to obtain gross section strengths. Beyond strength comparisons, allowable hole sizes for each code body and existing provisions accounting for the presence of both ungrouted and grouted stand-off base plates are discussed.

3.2 Steel Strength of Anchor Bolts in Tension

AASHTO (2013a) Section 6.13.2.10.2 specifies the design tensile strength, $\phi T_n$, as given in Equation (3-1). This assumes the net tensile area is 0.76 of the gross cross-sectional area.

\[
\phi T_n = \phi 0.76 A_p F_{ub}
\]  

(3-1)

where $\phi = 0.8$, $A_p = $ unthreaded bolt cross-sectional area, $F_{ub} = $ specified minimum tensile strength of the bolt

ACI (2011) Section D.5 specifies the design tensile strength, $\phi N_s \alpha$, as given in Equation (3-2). For the sake of comparison to other codes, $A_{se,N}$ is taken as 0.75 of the gross area (the full
Ductile elements are defined by Appendix D as having a tensile test elongation equal to or exceeding 14% and an area loss of at least 30%. Brittle elements are those that do not meet one or both of the elongation or area reduction requirements in tensile testing.

\[
\phi_{N_{sa}} = \phi A_{se} f_{uta} 
\]

(3-2)

where

- \( \phi \) = 0.75 for ductile steel as defined above
- \( \phi \) = 0.65 for brittle steel as defined above
- \( A_{se,N} \) = threaded bolt cross-sectional area = \( \pi \left( d_a - \frac{0.9743 n_t}{n_t} \right)^2 \) [units in inches]
- \( f_{uta} \) = specified tensile strength of anchor steel that shall be taken as no greater than 125,000 psi or 1.9\( f_{ya} \)
- \( f_{ya} \) = specified yield strength of anchor steel

AISC (2010) Section J6 and Table J3.2 specify the design tensile strength, \( \phi R_{nt} \), as given in Equation (3-3). The tensile area is assumed to be 0.75 of the unthreaded area.

\[
\phi R_{nt} = \phi F_{nt} A_b
\]

(3-3)

where

- \( \phi \) = 0.75
- \( F_{nt} \) = nominal tensile stress capacity = 0.75\( F_u \)
- \( A_b \) = unthreaded bolt cross-sectional area
- \( F_u \) = specified minimum tensile strength

Table 3-1 provides a summary of resistance factors, nominal tensile strengths, and resulting design tensile strengths in terms of an assumed net tensile area, \( A_{se} \), equal to 0.75 of the gross area and specified tensile strength \( F_{ut} \).

<table>
<thead>
<tr>
<th>Code</th>
<th>Resistance (Strength Reduction) Factor</th>
<th>Nominal Tensile Strength</th>
<th>Design Strength</th>
</tr>
</thead>
<tbody>
<tr>
<td>AASHTO (2013a)</td>
<td>0.80</td>
<td>1.0( A_{se} F_{ut} )</td>
<td>0.80( A_{se} F_{ut} )</td>
</tr>
<tr>
<td>ACI (2011)</td>
<td>0.75</td>
<td>1.0( A_{se} F_{ut} )</td>
<td>0.75( A_{se} F_{ut} )</td>
</tr>
<tr>
<td>Ductile</td>
<td>0.75</td>
<td>1.0( A_{se} F_{ut} )</td>
<td>0.75( A_{se} F_{ut} )</td>
</tr>
<tr>
<td>Brittle</td>
<td>0.65</td>
<td>1.0( A_{se} F_{ut} )</td>
<td>0.65( A_{se} F_{ut} )</td>
</tr>
<tr>
<td>AISC (2010)</td>
<td>0.75</td>
<td>1.0( A_{se} F_{ut} )</td>
<td>0.75( A_{se} F_{ut} )</td>
</tr>
</tbody>
</table>
3.3 Steel Strength of Anchor Bolts in Shear

AASHTO (2013a) Section 6.13.2.12 specifies the design shear strength, $\phi R_n$, for ASTM F1554 anchor bolts with threads included in the shear plane as given in Equation (3-4). The basis for Equation (3-4) is discussed in AASHTO C6.13.2.7. The 0.48 multiplier for threads in the shear plane is based on a shear strength of 0.60 times the tensile strength of the bolt for threads not in the shear plane multiplied by 0.80 to account for the threads in the shear plane as observed by Yura et al. (1987). In Commentary Section C6.13.2.12 it is recommended to apply an additional 0.8 factor to the shear strength to account for bolt hole oversize and other factors from group effects using engineering judgment.

$$\phi R_n = \phi 0.48 A_p F_{ub}$$ \hspace{1cm} (3-4)

where

- $\phi$ = 0.75
- $A_p$ = unthreaded bolt cross-sectional area
- $F_{ub}$ = specified minimum tensile strength of the bolt

ACI (2011) D.6.1 specifies the design shear strength, $\phi V_{da}$, as given in Equation (3-5). See Section 3.2 for definitions of ductility.

$$\phi V_{da} = \phi 0.6 A_{se,v} f_{uta}$$ \hspace{1cm} (3-5)

where

- $\phi$ = 0.65 for ductile steel as defined in Section 3.2
- $\phi$ = 0.60 for brittle steel as defined in Section 3.2
- $A_{se,v} = \frac{\pi}{4} (d_a - 0.9743 \frac{n_r}{n_t})^2$ [for units in inches]
- $f_{uta}$ = specified tensile strength of anchor steel and shall be taken as no greater than 125,000 psi or $1.9 F_{ya}$
- $f_{ya}$ = specified yield strength of anchor steel

AISC (2010) Section J9 and Table J3.2 specify the design shear strength, $\phi R_{nv}$, for anchor bolts as given in Equation (3-6). The 0.45 multiplier for the calculation of $R_{nv}$ is a result of a 0.9 factor for group and hole oversize effects similar to the 0.8 factor discussed above in the AASHTO (2013a) approach. However, in AISC (2010) this factor was moved from 0.8 to 0.9.
after several years of satisfactory connection performance. The 0.9 factor is applied to the 0.625 average bolt strength from Chesson et al. (1965) discussed in Section 2.3.2. To account for threaded area, the ratio of threaded shear strength to unthreaded shear strength is taken as 0.8 following recommendations by Yura et al. (1987) discussed in Section 2.3.2 in contrast to the AASHTO (2013a) and ACI (2011) assumptions that the shear area can be taken as the net tensile area in threaded regions. Thus, it follows that 0.625*0.9*0.8 = 0.45.

\[
\phi R_{nv} = \phi F_{nv} A_b 
\]

where

- \( \phi = 0.75 \)
- \( F_{nv} \) = nominal shear stress capacity = 0.45\( F_u \)
- \( A_b \) = unthreaded bolt cross-sectional area
- \( F_u \) = specified minimum tensile strength

Table 3-2 provides a summary of resistance factors, nominal shear strengths, and resulting design shear strengths for the threaded area in terms of the nominal bolt area, \( A_b \), which is used in place of \( A_{se} \) because of the different assumptions about the ratio of unthreaded to threaded strength by the different codes. For comparison, this ratio is represented for ACI (2011) as 0.75, the approximate value of the threaded to unthreaded bolt area.

Table 3-2. AASHTO (2013a), ACI (2011), and AISC (2010) bolt shear strengths

<table>
<thead>
<tr>
<th>Code</th>
<th>Resistance (Strength Reduction) Factor</th>
<th>Nominal Shear Strength</th>
<th>Design Strength</th>
</tr>
</thead>
<tbody>
<tr>
<td>AASHTO (2013a)</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>No group effect</td>
<td>0.75</td>
<td>0.48( A_b F_{ut} )</td>
<td>0.36( A_b F_{ut} )</td>
</tr>
<tr>
<td>With group effect</td>
<td>0.75</td>
<td>0.38( A_b F_{ut} )</td>
<td>0.29( A_b F_{ut} )</td>
</tr>
<tr>
<td>ACI (2011)</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Ductile</td>
<td>0.65</td>
<td>0.6( A_b F_{ut} )</td>
<td>0.39( A_b F_{ut} )</td>
</tr>
<tr>
<td>Brittle</td>
<td>0.60</td>
<td>0.6( A_b F_{ut} )</td>
<td>0.36( A_b F_{ut} )</td>
</tr>
<tr>
<td>AISC (2010)</td>
<td>0.75</td>
<td>0.45( A_b F_{ut} )</td>
<td>0.34( A_b F_{ut} )</td>
</tr>
</tbody>
</table>
3.4 Steel Strength of Anchor Bolts in Combined Tension and Shear

AASHTO (2013a) Section 6.13.2.11 specifies a reduced value of $\phi T_n$ as given in Equation (3-7) accounting for shear present in the bolt. This is a factored rearrangement of the elliptical relationship between tension and shear from (2-28). For $\frac{P_t}{R_n}$ less than 0.33, the full value of $T_n$ given in Equation (3-1) is allowed. No provisions are given for the shear strength under low levels of tension load, contrary to ACI (2011) and AISC (2010).

$$\phi T_n = \phi 0.76 A_p F_{ub} \sqrt{1 - \left(\frac{P_t}{\phi_s R_n}\right)^2}$$  \hspace{1cm} (3-7)

where  
$\phi$ = 0.8  
$A_p$ = unthreaded bolt cross-sectional area  
$F_{ub}$ = specified minimum tensile strength of the bolt  
$P_t$ = factored shear force on the bolt  
$\phi_s$ = 0.75  
$R_n$ = nominal shear resistance of the bolt (3-4)

ACI (2011) Section D.7 uses a linear interaction between bolt tension and shear forces as given in (3-8). At values of $\frac{V_{ua}}{\phi V_n} \leq 0.2$ the full tension strength of the anchor bolt may be taken as given in (3-2). Similarly, at values of $\frac{N_{ua}}{\phi N_n} \leq 0.2$ the full shear strength of the anchor bolt may be taken as given in Equation (3-5).

$$\frac{N_{ua}}{\phi N_{sa}} + \frac{V_{ua}}{\phi V_{sa}} \leq 1.2$$ \hspace{1cm} (3-8)

where  
$N_{ua}$ = factored tension force in the anchor bolt  
$\phi N_{sa}$ = tension capacity as calculated in (3-2)  
$V_{ua}$ = factored shear force in the anchor bolt  
$\phi V_{sa}$ = shear capacity as calculated in (3-5)

AISC (2010) Section J7 specifies the design available tension strength, $\phi R_{nv}$, in (3-9), using a linear relationship between tension and shear forces as in ACI (2011). Similar to ACI (2011), full tension and shear strength as calculated in Equation (3-3) and Equation (3-6),
respectively are allowed for the anchor bolt if the other component of stress is less than 30% of its available non-combined stress capacity.

\[
\phi R_n = \phi F'_{nt} A_b
\]  

(3-9)

where \( \phi = 0.75 \)

\( F'_{nt} = \) available tensile stress = \( 1.3F_{nt} - \frac{F_{nt}}{\phi F_{nv}} f_{rv} \leq F_{nt} \)

\( F_{nt} = \) nominal tensile stress as defined in (3-3)

\( F_{nv} = \) nominal shear stress as defined in (3-6)

\( f_{rv} = \) factored shear stress

### 3.5 Provisions for Steel Strength in Stand-off Base Plates

AASHTO (2013a) does not provide guidance on designing anchor bolt steel strength for ungrouted or grouted stand-off base plates. AASHTO (2013b) 5.17.4.3, however, accounts for stand-off explicitly based on work performed by Kaczinski et al. (1998), stating that where the distance from the concrete surface to the bottom of the leveling nut exceeds one bolt diameter, bending in the anchor should be considered. Within the commentary, it is stated that bending moments developing in the anchor bolt from shear forces may be determined by modeling a doubly moment-restraining beam with length equal to the distance between the concrete surface and the bottom of the leveling nut. Stresses due to bolt bending are added to the tensile stress component of the elliptical interaction relationship provided in Equation (3-10) (or the compressive stresses in a similar compression/shear interaction). The commentary also states explicitly that bolt bending from bolt shear forces may be ignored for exposed lengths less than or equal to one anchor bolt diameter. AASHTO (2013b) 5.17.3.3 states that grout installed beneath the base plate may not be designed to contribute to connection strength in double-nut connections (those using leveling nuts). Thus, two double-nut connections, one with and one without a grout pad, would be designed using the beam model described above.
\[
\left( \frac{f_v}{F_v} \right)^2 + \left( \frac{f_t}{F_t} \right)^2 \leq 1.0
\]  

(3-10)

where
\[ f_v = \text{factored shear stress} \]
\[ F_v = \text{allowable shear stress} \]
\[ f_t = \text{factored tensile stress from both axial bolt tension and bolt bending} \]
\[ F_t = \text{allowable tensile stress} \]

ACI (2011) D.6.1.3 states that 80% of the shear strength of an anchor group is maintained in the presence of a grout pad irrespective of grout pad height (base plate stand-off distance). It can be inferred that this provision applies to both pure shear and combined tension and shear applications. No provisions currently exist in ACI (2011) for anchor bolt steel strength in ungrouted stand-off base plate connections.

Beyond stating in commentary that anchor bolt bending must be considered when shear is transferred through base plate bearing against anchor bolts, AISC (2010) does not provide guidance for ungrouted or grouted stand-off base plates. However, AISC Design Guide 1 (Fisher and Kloiber 2006) provides an example using the interaction of tension forces in the anchor from direct tension and bolt bending with shear forces in the anchor in a base plate supported by a grout pad. Their calculation uses the same beam bending model as in AASHTO (2013b), but takes the length of the beam as the distance from the top of the grout to the middle of the grout pad. This assumes that anchor bolt length within the thickness of the grout pad does not contribute to bolt bending and also implies that grout pads shall not contribute to anchor bolt strength.

### 3.6 Hole Sizes

Table 3-3 provides anchor bolt hole size comparisons between AASHTO (2013a), AISC (2010), and current FDOT standards. ACI (20111) does not provide guidance on hole size.
AASHTO (2013a,b) provides hole dimensions for both “shear holes” and “normal holes,” the former defined as those with anchor bolts designed to resist shear due to direct shear or torsion forces on the base plate. AISC hole sizes for anchors, given in Table C-J9.1 (AISC 2010) align with the dimensions for AASHTO normal holes while AISC oversize holes for steel-to-steel connections align with AASHTO shear holes. To account for this relatively large oversize, AISC (2010) Section C-J9 states that if anchor bolts are to resist shear forces bolt bending must be considered, but does not provide any further guidance. FDOT hole sizes are taken as 0.5 inches greater than the bolt diameter following “General Notes” in Design Standards 11310 and 17745.

Table 3-3. Comparison of AASHTO (2013a,b), AISC (2010), and FDOT hole sizes.

<table>
<thead>
<tr>
<th>Bolt Dia. (in.)</th>
<th>Hole Dimensions (in.)</th>
<th>Hole Oversize (in.)</th>
<th>Hole:Bolt Ratio</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>AASHTO Shear</td>
<td>AISC</td>
<td>FDOT</td>
</tr>
<tr>
<td>0.5</td>
<td>0.63</td>
<td>1.06</td>
<td>-</td>
</tr>
<tr>
<td>0.625</td>
<td>0.81</td>
<td>1.19</td>
<td>-</td>
</tr>
<tr>
<td>0.75</td>
<td>0.94</td>
<td>1.31</td>
<td>-</td>
</tr>
<tr>
<td>0.875</td>
<td>1.06</td>
<td>1.56</td>
<td>-</td>
</tr>
<tr>
<td>1</td>
<td>1.25</td>
<td>1.81</td>
<td>1.5</td>
</tr>
<tr>
<td>1.25</td>
<td>1.56</td>
<td>2.06</td>
<td>1.75</td>
</tr>
<tr>
<td>1.5</td>
<td>1.81</td>
<td>2.31</td>
<td>2</td>
</tr>
<tr>
<td>1.75</td>
<td>2.06</td>
<td>2.75</td>
<td>2.25</td>
</tr>
<tr>
<td>2</td>
<td>2.31</td>
<td>3.25</td>
<td>2.5</td>
</tr>
</tbody>
</table>

3.7 Summary

This chapter identified existing code specifications for the treatment of anchor bolt strength in flush-mounted, ungrouted stand-off, and grouted stand-off base plate connections subjected to combinations of externally applied shear and normal forces. Gaps in code treatment of different cases were identified in the determination of strength in both ungrouted and grouted stand-off base plate connections. The results and analytical methods from this dissertation are used to provide recommendations for the codified treatment of the steel strength of anchor bolts in stand-off base plate connections.
CHAPTER 4
DEVELOPMENT OF EXPERIMENTAL PROGRAM

4.1 Introduction

The development of the experimental program was driven by the primary objectives of quantifying anchor bolt steel strength for generalized loading and exposed length conditions in ungrouted and grouted stand-off base plate connections. Based on the results of the literature review, three major variables were identified in determining the strength of anchor bolts in stand-off base plate connections:

- Anchor bolt exposed length
- Base plate support condition (i.e. flush-mounted, ungrouted stand-off, and grouted)
- Ratio of applied axial-to-shear force

In the experimental program, the greatest focus and priority was given to the comprehensive treatment of ranges of practical values of the three major variables. In addition, the following minor variables were investigated in specific tests: method for determining shear strength, installation type in Direct Shear tests, bolt pretensioning, size effect, bolt material type, hole size, grout installation condition, fiber-reinforced polymer (FRP) retrofit around a grouted stand-off base plate, shim stacks as base plate support condition, absence of leveling nuts in a grouted installation, and the effect of friction in grouted installation.

Additional variables were identified by sources in the literature as having potential influence on the shear steel strength and behavior of anchor bolts in stand-off base plate connections and were not investigated in this study. It was determined that for a number of these additional variables, results from previously conducted experimental programs apply generally to anchor bolts in stand-off base plates, requiring no further study. These variables include the presence of threads in the shear plane (Yura et al. 1987, Nakashima 1998) and the effect of
embedment depth (Nakashima 1998). Other variables, including grout footprint parameters were considered to not have significant effects based on the model proposed in Gresnigt et al. (2008), which assumes that grout used in these connections is sufficiently strong to resist compressive forces from the base plate and sufficiently stiff to have negligible effects on assumptions of axial force transfer (see Section 6.3.4). These assumptions minimize the effects of grout strength and footprint parameters have little effect on anchor bolt steel strength. Finally, the influence of base plate flexibility was not considered due to recommendations that the base plate be sufficiently stiff to be rigid from, e.g., Kaczinski et al. (1998) and Liu (2014) and because this affects the distribution of force to, not the capacity of, individual anchor bolts.

A total of 132 tests were conducted in four experimental arrangements, itemized below. Direct Shear, Torsion, and Eccentric Shear tests were all conducted at the University of Florida Structures Laboratory, while Large-scale testing was conducted at the Florida Department of Transportation Marcus H. Ansley Structures Research Center in Tallahassee, Florida. A short description of each arrangement is given below:

- **Direct Shear:** 68 tests of single- and double-bolt ungrouted stand-off base plate connections at varying anchor bolt exposed lengths
- **Torsion:** 28 tests of circular groups of 5/8 in. and 1 in. diameter anchor bolts with the connection loaded in torsion (resulting in pure shear applied to individual bolts in the connections) under combinations of exposed length and support condition
- **Large-scale:** Four tests of circular groups of six 1.25 in. diameter bolts loaded predominantly in torsion (resulting in pure shear applied to individual bolts in the connections) with ungrouted and grouted support conditions
- **Eccentric Shear:** 33 tests of two- and four-bolt connections loaded with an eccentric shear force that produced overturning moment and shear force on the connection resulting in varying axial-to-shear force ratios on the individual bolts with combinations of exposed length and support condition.

To avoid redundancy, materials and methods shared by all experimental arrangements are first presented in the subsequent section, followed by sections for each test arrangement. The
geometrical treatment of experimental data resulting in reported values, where applicable, is described for each experimental arrangement.

4.2 Materials and Methods

4.2.1 Bolt Specimens

A majority of anchor bolt specimens were cast-in-place, consisting of fully threaded rods headed with ASTM A563 heavy hex nuts. The only exceptions to this specimen type were 1) in Direct Shear testing a series of unheaded bolts installed with a two-part epoxy adhesive and 2) Torsion test T28, where an adhesive installation was used for the final test after all cast-in-place specimens had been tested. The primary bolt material used was ASTM F1554 Grade 55 steel, with a limited number of tests containing ASTM A276 Type 316 stainless steel and ASTM 354BD materials to investigate the effects of material ductility.

Anchor bolt embedment depths within the concrete of approximately 10, 7, and 16 bolt diameters for 0.625 in., 1 in., and 1.25 in. bolts, respectively, were used to prevent concrete breakout failure modes. Given that the failure mode studied was anchor bolt steel fracture and that plastic bolt deformations were not observed beyond approximately one bolt diameter into the concrete surface, it was assumed that the embedment depths chosen had no effects on the steel strength of the bolts.

4.2.2 Concrete Blocks

With the exception of one block in Torsion testing with a low-strength mix with 2500 psi specified 28-day compressive strength, the concrete used was a Florida Department of Transportation Class IV Drilled Shaft mix with 4000 psi specified 28-day strength. Reinforcement within the blocks was provided strictly to prohibit failure modes other than anchor bolt steel fracture. In all cases, rebar was placed with a minimum cover of ¾ in. and at
least 3 in. from anchor bolt specimens to prevent influence on local spalling in front of the anchor bolts.

Concrete blocks for the three experimental arrangements conducted at the University of Florida structures research laboratory were placed with anchor bolts installed upside-down in formwork to achieve a flat concrete top surface without need for troweling or other finishing methods. Holes were drilled through both the bottom sheet of plywood and a second reference piece of plywood to set the position and plumbness of anchor bolts. Figure 4-1 shows a profile schematic of the system used to place anchor bolts.

![Figure 4-1. Schematic of formwork used for casting anchor bolt specimens upside-down in Direct Shear, Torsion, and Eccentric Shear blocks.](image)

Embedment depth of the anchors was set by placing a nut above and a nut below the reference sheet of plywood. Four female DYWIDAG coil loops were embedded at mid-depth of the slab to tie into for lifting and handling. ASTM A563 heavy hex nuts were placed on the embedded ends of the anchors and locked against rotation during concrete placement with a bead of quick-setting adhesive. Blocks were wet-cured for seven days after pouring using a drip hose covered by painter’s tarp and 4-mil plastic sheeting as shown in Figure 4-2. At 28 days, formwork was removed, after which the blocks remained in ambient laboratory conditions until all testing had been completed.
4.2.3 Grout

A nonshrink grout with specified 28-day strength of 9,000 psi was chosen for all grouted tests except for a single dry-packed mortar test. FDOT (2014) Section 934 for non-shrink grouts requires a 3-day compressive strength of 5,000 psi for 2 in. cube tests conducted in accordance with ASTM C109 (ASTM 2012), while the Technical Data Guide for the product chosen provides a minimum strength of 4,500 psi (corresponding to a 25-30 second flow cone time). It was determined that use of this nonshrink grout at three-day strength would provide conservative results compared to these FDOT standards. The lone dry-packed mortar test contained Type M mortar tested at an early age to achieve a low-strength condition.

For all grout pad installations a flow time of 20-30 seconds following ASTM C939 protocol (ASTM 2010b) was achieved following the methodology used in Cook and Bobo (2001). Grout pads were installed in accordance with the Manufacturer’s Printed Installation Instructions (MPII) and ASTM C1107 general requirements. To prepare the concrete surface for grout pad placement, it was brushed of debris and blown with an air hose. In grouted six-bolt Torsion tests and Large-scale tests, for 24 hours preceding grout installation the concrete surface was wetted with saturated shop rags placed on the area of concrete receiving the grout pad.
Approximately one hour before grout placement, the shop rags were removed to achieve a “Saturated Surface Dry” (SSD) condition during grout placement. This practice was discontinued for grouted three-bolt torsion tests and eccentric shear tests, as its effects were seen as negligible.

The amount of grout needed was determined for every test and weighed using a digital scale. For a given weight of grout, 17% water by dry grout weight was measured and placed in a five-gallon bucket. The dry grout was poured into this water and the mix was blended for a minimum of five minutes. Water was then added and mixed in thoroughly until the 20-30 second flow time was reached. It was found that approximately 19% water content by weight was ultimately needed for the ambient laboratory conditions to achieve the desired flow cone times. Because additional holes were present in the base plates for both Torsion and Large-scale tests, grout was installed in a single pour through one of the additional holes using the flow cone attachment from the ASTM C939 (ASTM 2010b) method as shown in Figure 4-3. Care was taken to install grout within 30 minutes from initial wetting as specified by the MPII.

Immediately after placing a grout pad, 2 in. cubes were cast in copper molds treated with a light coat of WD-40 applied with shop rags. Cubes were prepared as specified by ASTM C942 (ASTM 2010a). Plexiglas sheets were then placed atop the gang mold to retard water evaporation and the cubes were left near the grout pad. At 24 hours, the formwork for the grout pad and the cubes were removed. For all six-bolt torsion and Large-scale tests with grout, both the grout pad and grout cubes were painted with the manufacturer’s specified grout curing compound. The grout pad was wrapped in wet paper towels and left to sit until tested. Although a moist cure condition with specific curing parameters is specified by ASTM C109 (ASTM 2012), the cubes were treated the same as the grout to provide identical curing conditions. A representative Torsion grout pad and grout cubes is shown in Figure 4-4. Grout cubes were tested
in compression following ASTM C109 protocol (ASTM 2012) immediately after corresponding torsion tests. For a displacement-controlled test, the tests ranged from 250-350 lb./sec. in the linear-elastic phase of the tests, after which the displacement rate was kept consistent until grout cube failure.

Figure 4-3. Grout installation procedure. A) Performing the ASTM C939 flow cone test. B) grout pad placement. (Photos courtesy of author.)

Figure 4-4. Grout pad and 2 in. cubes after wet-wrapping. (Photo courtesy of author.)

In one grouted Torsion test, grout cubes were made and treated by wrapping with wet paper towels at 24 hours without painting the curing compound in addition to the normal painted specimens. It was found that there were no significant differences in the strengths between the
two treatment methods at the three-day mark at which all specimens were tested. Furthermore, it can be inferred from Gresnigt et al. (2008) and is later confirmed in the results that the grout strength is not a contributing factor to the steel strength of anchor bolts in grouted connections. For these reasons, painting the finished grout during form release at 24 hours was discontinued for three-bolt grouted Torsion tests and grouted Eccentric Shear tests.

4.2.4 Instrumentation and Data Acquisition

Every experimental arrangement contained a unique instrumentation scheme. Specific details for each experimental arrangement are discussed further in the respective sections for the arrangements. In general, however, the following measurements were taken:

- Applied load using through-hole, moment compensating load cells
- Horizontal displacement of base plate using LVDTs, linear potentiometers, and string potentiometers.
- Vertical displacement of base plate using LVDTs
- “Tension” experienced by anchor bolts with through-hole load cells

The bolt “tension” load cells were placed on the bolts directly above the base plate. While a majority of these load cells contained self-aligning, moment releasing spherical washers, these load cells did not have the capability to compensate for bending forces, which, in many cases, were pronounced, even in the pretensioning process. In addition, several of these instruments were loaded above their prescribed capacity. As a result, the readings cannot be taken as direct measurements of tension; instead, they must be interpreted as “relative” measurements throughout the loading process. All relative tension load cell readings are placed in the appendices, where the treatment of their data is described.

All readings from instrumentation were obtained using LabVIEW software and National Instruments modules in a Compaq DAQ chassis. Data were obtained at a 10 Hz rate for all
experiments with the exception of direct shear tests, where readings were taken at 2 Hz. Wires from all instrumentation fed into the DAQ chassis, which fed directly into a laboratory computer via USB.

4.3 Direct Shear Testing

4.3.1 Experimental Design

Shear strength of ungrouted anchor bolts with stand-off base plates was first studied using a direct shear loading configuration with bolts installed in a rectangular base plate. The complete direct shear matrix is provided in Table 4-1. In total, 68 tests in 15 series studied stand-off distance, single vs. double anchor bolts installed in a rectangular base plate, cast-in-place vs. adhesive anchor bolts, and 5/8 in. vs. 1 in. diameter bolts installed in tight (i.e. no oversize) holes.

Table 4-1. Direct Shear test matrix.

<table>
<thead>
<tr>
<th>Test Set</th>
<th>Reps</th>
<th>$d_p$ (in.)</th>
<th>$d_b$ (in.)</th>
<th>n</th>
<th>Concrete</th>
<th>Install. Method</th>
<th>Bolt Material</th>
<th>Connection Type</th>
<th>Top Nut Tightness</th>
<th>$l_{BP} / d_b$</th>
<th>$l_{LN} / d_p$</th>
</tr>
</thead>
<tbody>
<tr>
<td>DS1</td>
<td>8</td>
<td>0.625</td>
<td>0.63</td>
<td>1</td>
<td>N</td>
<td>CIP</td>
<td>F1554</td>
<td>FM</td>
<td>FT</td>
<td>0</td>
<td>na</td>
</tr>
<tr>
<td>DS2</td>
<td>6</td>
<td>0.625</td>
<td>0.63</td>
<td>1</td>
<td>N</td>
<td>CIP</td>
<td>F1554</td>
<td>U</td>
<td>FT</td>
<td>1.2</td>
<td>0</td>
</tr>
<tr>
<td>DS3</td>
<td>2</td>
<td>0.625</td>
<td>0.63</td>
<td>1</td>
<td>N</td>
<td>CIP</td>
<td>F1554</td>
<td>U</td>
<td>FT</td>
<td>1.6</td>
<td>0.4</td>
</tr>
<tr>
<td>DS4</td>
<td>10</td>
<td>0.625</td>
<td>0.63</td>
<td>1</td>
<td>N</td>
<td>CIP</td>
<td>F1554</td>
<td>U</td>
<td>FT</td>
<td>3</td>
<td>2.8</td>
</tr>
<tr>
<td>DS5</td>
<td>4</td>
<td>0.625</td>
<td>0.63</td>
<td>1</td>
<td>N</td>
<td>CIP</td>
<td>F1554</td>
<td>U</td>
<td>FT</td>
<td>4</td>
<td>2.8</td>
</tr>
<tr>
<td>DS6</td>
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<td>0.625</td>
<td>0.63</td>
<td>1</td>
<td>N</td>
<td>CIP</td>
<td>F1554</td>
<td>U</td>
<td>FT</td>
<td>2</td>
<td>0.8</td>
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<td>DS7</td>
<td>4</td>
<td>0.625</td>
<td>0.63</td>
<td>2</td>
<td>N</td>
<td>CIP</td>
<td>F1554</td>
<td>FM</td>
<td>FT</td>
<td>0</td>
<td>na</td>
</tr>
<tr>
<td>DS8</td>
<td>5</td>
<td>0.625</td>
<td>0.63</td>
<td>2</td>
<td>N</td>
<td>CIP</td>
<td>F1554</td>
<td>U</td>
<td>FT</td>
<td>2</td>
<td>0.75</td>
</tr>
<tr>
<td>DS9</td>
<td>4</td>
<td>1</td>
<td>1.01</td>
<td>1</td>
<td>N</td>
<td>CIP</td>
<td>F1554</td>
<td>FM</td>
<td>FT</td>
<td>0</td>
<td>na</td>
</tr>
<tr>
<td>DS10</td>
<td>5</td>
<td>1</td>
<td>1.01</td>
<td>1</td>
<td>N</td>
<td>CIP</td>
<td>F1554</td>
<td>U</td>
<td>FT</td>
<td>2</td>
<td>0.75</td>
</tr>
<tr>
<td>DS11</td>
<td>2</td>
<td>1</td>
<td>1.01</td>
<td>1</td>
<td>N</td>
<td>CIP</td>
<td>F1554</td>
<td>U</td>
<td>FT</td>
<td>4</td>
<td>2.75</td>
</tr>
<tr>
<td>DS12</td>
<td>4</td>
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<td>0.63</td>
<td>1</td>
<td>N</td>
<td>AD</td>
<td>F1554</td>
<td>FM</td>
<td>FT</td>
<td>0</td>
<td>na</td>
</tr>
<tr>
<td>DS13</td>
<td>5</td>
<td>0.625</td>
<td>0.63</td>
<td>1</td>
<td>N</td>
<td>AD</td>
<td>F1554</td>
<td>U</td>
<td>FT</td>
<td>2</td>
<td>0.8</td>
</tr>
<tr>
<td>DS14</td>
<td>2</td>
<td>0.625</td>
<td>0.63</td>
<td>1</td>
<td>N</td>
<td>AD</td>
<td>F1554</td>
<td>U</td>
<td>FT</td>
<td>4</td>
<td>2.8</td>
</tr>
<tr>
<td>DS15c</td>
<td>2</td>
<td>0.625</td>
<td>0.63</td>
<td>1</td>
<td>N</td>
<td>CIP</td>
<td>F1554</td>
<td>U</td>
<td>FT</td>
<td>2</td>
<td>0.8</td>
</tr>
</tbody>
</table>

$na$ = not applicable

$^a$DS = Direct Shear, T = Torsion, LS = Large-scale, ES = Eccentric Shear

$^b$N = Normal-strength Concrete, L = Low-strength Concrete

$^c$FM = Flush-Mounted base plate, U = Ungrouted stand-off base plate with leveling nut, G = Grouted stand-off base plate, GF = Grouted stand-off base plate with FRP retrofit

$^d$FT = Finger-Tightened, TOTN = Turn-of-the-Nut

$^e$Loading rod threaded directly into base plate (no clevis)
4.3.2 Test Setup

4.3.2.1 Structural components

A three-dimensional schematic of the test setup is shown below in Figure 4-5. The setup contained a 46.5 in. by 46.5 in. by 12 in. deep concrete block hosting multiple anchor bolt test specimens and resting on steel beam sections. A 7/8 in. diameter ASTM A193 B7 tension rod supplied load to the base plate through a telescoping through-hole hydraulic actuator supported by a reaction frame tied to a strong wall. The actuator was positioned on the back side of the steel frame composed of two steel C channel sections with a 2 in. gap to allow the tension loading rod to pass through. On the loading end the concrete block bore against a square steel reaction channel section to restrain concrete breakout stresses and provide spacing. A system of steel beams tied to the strong floor of the laboratory was used to keep the system restrained against overturning moments during tests.

Figure 4-5. Labeled schematic of Direct Shear test setup.
Checks were made on the double channel sections comprising the reaction frame to withstand shear forces, compressive forces, and bending moments produced by the applied load from the actuator and the bearing reaction produced by the steel square reaction section. To maintain equilibrium in the test setup shown in Figure 4-5, the tie-down system needed to resist a moment equal to the loading eccentricity distance between the center of the base plate and the location of the bearing reaction on the test block.

The rectangular base plate with dimensions of 4 in. by 14.5 in. by 1.18 in. thick was developed by Grosser (2012). In flush-mounted tests and double-bolt stand-off base plate tests the loading rod was threaded directly into the base plate with no additional restraint. Single-bolt stand-off base plate tests, however, included a roller to restrain base plate rotation as shown in Figure 4-6. A clevis connection was used to release the bending forces in the loading rod. Nuts were installed in a “finger-tight” condition at approximately 20 in.-lb. Checks were made for net section fracture, tear-out, and bearing failure modes of the steel base plate.

![Figure 4-6. Connection details for Direct Shear tests. A) Single bolt. B) Double-bolt.](image)

Concrete blocks were designed with sufficient anchor bolt edge distance and effective depth to prohibit concrete breakout per ACI 318-11 without reinforcement. A mid-section perimeter of number 3 rebar was installed capable of supporting handling forces from self-weight. A perimeter of #3 rebar was placed at mid-depth for handling precautions. Figure 4-7 shows a typical Direct Shear form before placing concrete.
4.3.2.2 Instrumentation

Figure 4-8 indicates the locations of instrumentation used for direct shear testing. Load was measured by a through-hole 100 kip moment-compensating load cell located at the back end of the loading actuator with steel plates on either end of the load cell. Displacement was measured at the back end of the rectangular base plate with a linear potentiometer. The potentiometer, which contained a spring-retracting plunger, was connected to the back of the base plate through a stiff steel cord that fastened magnetically to the center of the top surface. In 42 of the 66 tests conducted, a “tension” load cell was placed at the top of the bolts. The tests conducted without a tension load cell were all within datasets DS1, DS2, DS7 and DS8, which contained flush-mounted and $2d_b$ stand-off tests for single- and double-bolt tests and were run to determine if the load cell impacted test results (from the change in distance between leveling and tightening nuts). No difference in load-displacement behavior was found. Hence, all other 5/8 in. bolt tests contained tension load cells. A tension load cell was not used in 1 in. bolt tests.
4.3.3 Test Preparation and Procedure

To prepare every Direct Shear test, the concrete block was set into place onto two steel beams using an overhead crane, ensuring that the top surface of the block was level with steel shims below the block as necessary. Block position adjustments were made until the anchor bolt specimen was in line with the loading rod. The block was fastened on the back end using beams and tension ties in a snug-tight connection. Stand-off height of the test was then established and equal distance from the concrete surface to the bottom of the base plate at the front and back of the base plate was ensured. The loading rod and actuator were set collinearly with the mid-height of the base plate. All remaining instrumentation and test components (e.g. roller, clevis hinge) were installed. The tightening nut was set to a finger-tight condition of 20 lb-in., after which the system was preloaded to approximately 100 lb. to engage components within the test setup.

The displacement-controlled loading procedure followed ASTM E488 protocol (ASTM 2010c), which states that failure should occur between 1 and 3 minutes from the beginning of the test. Due to the changing stiffnesses and resulting ultimate displacements for test specimens with different stand-off heights, the test displacement rate was estimated for every test to fall within this range of failure times. Consistent displacement rate was applied until anchor bolt rupture, ending an individual test.
4.4 Torsion Testing of Scaled Anchor Bolt Groups

A second form of shear strength testing of anchor bolts was investigated by loading circular anchor bolt groups in torsion. Circular bolt groups consisted of three and six bolts using 5/8 in. and 1 in. diameter bolts in flush-mounted, ungrouted stand-off, and grouted stand-off configurations. Two primary reasons drove the development of a torsion loading apparatus to test the shear strength of anchor bolts: 1) Individual anchor bolts within the system were loaded in pure shear throughout the loading process, even beyond second-order displacement as bolts were stretched/compressed axially and as the geometry of the displaced bolts caused downward movement of the base plate (see Section 5.3.1 for a detailed description of second-order influences in Direct Shear testing) and 2) A circular bolt pattern replicates the common connection for sign and signal structures connecting circular poles to concrete foundations through annular base plates. Figure 4-9 shows a representative Torsion test with three 1 in. diameter bolts.

Figure 4-9. Torsion test setup. (Photo courtesy of author.)
4.4.1 Experimental Design

The complete Torsion testing matrix is provided below in Table 4-2. In total, ten tests with groups of six 5/8 in. diameter anchor bolts, three tests with three 1 in. diameter bolts, and 15 tests with three 5/8 in. diameter bolts were conducted. Tests with groups of six 5/8 in. diameter bolts were designed as scaled versions of existing Florida Department of Transportation mast arm and cantilever sign anchored base plate connections as discussed in the next section.

Table 4-2. Torsion test matrix.

<table>
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<tr>
<th>Set</th>
<th>$d_B$ (in.)</th>
<th>$\ell_B$ (in.)</th>
<th>$n$</th>
<th>Concrete</th>
<th>Install. Type</th>
<th>Bolt Material</th>
<th>Conn. Type</th>
<th>Top Nut Tightness</th>
<th>$l_{BP}$</th>
<th>$l_{LN}$</th>
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<tr>
<td>T1</td>
<td>0.625</td>
<td>0.81</td>
<td>6</td>
<td>N</td>
<td>CIP</td>
<td>F1554</td>
<td>FM</td>
<td>FT</td>
<td>0</td>
<td>na</td>
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<td>N</td>
<td>CIP</td>
<td>F1554</td>
<td>FM</td>
<td>FT</td>
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<td>na</td>
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<td>CIP</td>
<td>F1554</td>
<td>U</td>
<td>TOTN</td>
<td>2</td>
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<td>CIP</td>
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<td>G</td>
<td>TOTN</td>
<td>4</td>
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<td>N</td>
<td>CIP</td>
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<td>M</td>
<td>TOTN</td>
<td>4.2</td>
<td>3</td>
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</table>

$na = not applicable$

$^aDS =$ Direct Shear, $T =$ Torsion, $LS =$ Large-scale, $ES =$ Eccentric Shear

$^bN =$ Normal-strength Concrete, $L =$ Low-strength Concrete

$^cFM =$ Flush-Mounted base plate, $U =$ Ungrouted stand-off base plate with leveling nut, $G =$ Grouted stand-off base plate, $GF =$ Grouted stand-off base plate with FRP retrofit, $GO =$ Grouted stand-off base plate without leveling nut, $US =$ Ungrouted stand-off base plate with Shim stack supports, $M =$ low-strength dry-packed Mortar around the bolt circle

$^dTFT =$ Finger-Tightened, $TOTN =$ Turn-Of-The-Nut
The following variables were investigated in this study: (1) Exposed length of anchor bolts (major variable); (2) base plate support condition, i.e., flush-mounted base plates, ungrouted stand-off base plates, and grouted stand-off base plates (major variable); (3) bolt pretensioning, (4) concrete strength; (5) number of bolts in ungrouted and grouted grouted configurations; (6) FRP retrofit around a grouted stand-off base plate; (7) anchor bolt size effect; (8) absence of leveling nuts in grouted stand-off base plate connection; (9) low-strength concrete; (10) shim stacks supporting base plate; and (11) anchor bolt material: ASTM F1554, ASTM 354BD, and SAE 316 Stainless Steel.

4.4.2 Test Setup

4.4.2.1 Structural components

A schematic of the scaled torsion test setup is given in Figure 4-10. Anchor bolt groups were installed in 46.5 x 46.5 x 12 in. deep reinforced concrete blocks. Independently operated hand pumps supplied hydraulic fluid to the actuators, which connected to the base plate assembly “loading wings” through 1 in. diameter ASTM A193 B7 threaded rods and 1 in. diameter clevis pin connections to produce torsion on the base plate and, ultimately, the anchor bolt group. Rolling reaction frames reacted against the opposite side of the concrete block via a 1.5 in. diameter ASTM A193 B7 tension rod sent through a PVC duct embedded in the concrete block. Because the system was self-reacting, no external tie-down support was necessary or used.

The 5/8 in. diameter bolt tests were designed as scaled versions of existing FDOT structures. Ratios of the bolt group radius and base plate thickness to nominal anchor bolt diameters were used as scaling parameters. To determine common ratios, a field survey of five mast arm assemblies was conducted, standard sizes for base connections in mast arm assemblies from a major supplier were obtained, the FDOT Cantilever Sign Program V5.1 (FDOT 2007)
was used to produce several designs for highway cantilever signs, and a single cantilever sign specimen surveyed in BD545-54 were compiled.

Table 4-3 provides the results and chosen dimensions for the test specimen. In contrast to the finger-tight top nut condition in Direct Shear testing, bolts in all but one of the stand-off base plate tests were pretensioned by the “turn-of-the-nut” method given in FDOT Specification 649. Flush-mounted tests, which included a Teflon layer below the base plate as in Direct Shear testing, were not pretensioned to reduce additional contributions to strength from friction.

Table 4-3. Summary of scaling results for 5/8 in. diameter Torsion bolt specimens.

<table>
<thead>
<tr>
<th></th>
<th>Bolt Circle Radius to Bolt Dia. Ratio</th>
<th>Base Plate Thickness to Bolt Dia. Ratio</th>
</tr>
</thead>
<tbody>
<tr>
<td>UF Mast Arm Survey</td>
<td>6.1</td>
<td>1.0</td>
</tr>
<tr>
<td>Cantilever V5.1 Program</td>
<td>7.5</td>
<td>0.86</td>
</tr>
<tr>
<td>2012 FDOT Standard 17743</td>
<td>6.57-7.5</td>
<td>1.25-1.67</td>
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<tr>
<td>BD545-54 Specimen</td>
<td>9.0</td>
<td>0.6</td>
</tr>
<tr>
<td>Chosen Ratios</td>
<td>8</td>
<td>1</td>
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</table>

Figure 4-10. Labeled schematic of Torsion test setup.
Base plate assembly dimensions and details are provided in Figure 4-11. The 12 in. HSS 7.5 X 0.5 pipe section was made of 55 ksi Carbon Steel DOM Mechanical Tube. The circular base plate and steel for the loading wing were made of grade 36 steel. All connections between assembly components were made with 3/8 in. fillet welds. For 2-bolt diameter spacing to the 5 in. bolt circle radius, 3.75 in. and 6.25 in. radii to the outside of the pipe stub and the base plate, respectively, were chosen. To maintain the 1:1 base plate thickness to nominal anchor bolt diameter ratio for all tests, 3/8 in. thick plates were welded above holes for the 1 in. diameter bolts. Hole diameters for both 5/8 in. and 1 in. diameter bolt tests were chosen per AASHTO (2013b) Table 5-5 for shear holes, defined as those with anchors subjected to shear forces due to base plate direct shear or torsion. After components were welded and holes cut, the entire assembly was case-hardened.

Figure 4-11. Labeled schematic of base plate assembly components.

Base plate assembly components were checked for structural capacity using the AISC Steel Construction Manual (AISC 2010). The pipe was designed to resist torsion forces from the loading couple along with combined moment, shear, and torsion forces that could be caused by
extreme unequal loading from the actuators. The base plate was designed against net section fracture, tear-out, and bearing failure modes from anchor bolt forces on the holes. Loading wings were designed for moment and shear forces during loading in addition to the same bolt hole checks as performed on the base plate. Welds were all designed for ultimate shear stresses.

In the concrete block, top reinforcement was designed to restrain breakout forces from the anchor bolts. ACI 318 (ACI 2011) breakout calculations were modified to account for overlapping failure cones from shear forces from anchor bolts induced by the torsion load. These forces can produce breakout cones in the direction of or perpendicular to the direction of load produced by an individual anchor bolt. Bottom block reinforcement accommodated minimum ACI 318-11 reinforcement requirements, temperature and shrinkage requirements, and was sufficient to withstand handling forces. Tension ties running through the bottom portion of the block were designed to transfer loads from the rolling frames to a compression reaction on the other side of blocks. Thorough checks were made of the combined effects on the block from the anchor bolt breakout forces and bearing forces from frame and tension tie reactions.

Anchor bolt specimens were cast upside-down in formwork as described in Section 4.2.2. Specimens with groups of six 5/8 in. diameter bolts were installed four to a block while specimens with three 1 in. diameter bolts were cast one per block. 5/8 in. anchors were embedded 6 in. to the bottom of the bolt head \( h_{ef} = 5.375 \text{ in} \), while 1 in. anchors were embedded 8 in \( h_{ef} = 7 \text{ in} \). Number 4 rebar was used for all reinforcement bent into U shapes with 44 inch length and 8 in. legs. The bottom faces of all blocks contained five equally spaced bars in both horizontal directions to satisfy minimum temperature and shrinkage reinforcement. The 2 in. diameter PVC pipes for tension reaction ties were installed three inches from the bottom face of the block to the center of the PVC. Blocks containing 5/8 in. diameter anchor
specimens contained four bars in both horizontal directions on the top face, while those with 1 in. diameter anchors contained ten in each direction to restrain concrete breakout forces. All reinforcement contained a minimum cover of 1.5 in. Figure 4-12 shows completed formwork before the concrete pour and concrete blocks after formwork removal for blocks with 5/8 in. and 1 in. diameter bolt specimens.

Rolling reaction frames were composed of two channel sections with a 3 in. gap for load application and restraint. The frames also contained 0.5 in. thick steel bottom plates with casters and a 0.5 in. thick steel top plate to support identical telescoping through-hole hydraulic loading actuators. C10 x 15.3 channel sections were found sufficient to restrain bending moments assuming both a distributed load and a point load from the frame bearing reaction against the concrete block. The bottom platform was designed to balance the self-weight of the frames.

The torsion and Large-scale test setups containing circular bases plate most closely simulated annular base plates used in sign and signal connections. To simulate the presence of an annular base plate in grouted Torsion tests, a cardboard circle with a diameter equal to the pipe outer diameter was cut and attached to the center of the underside of the base plate with tape as shown in Figure 4-13 (a). Circular Plexiglas forms were placed with height equaling the distance
from the concrete surface to the top of the base plate. Heat was applied as the form was wrapped around the base plate to achieve a circular bend in the strip and avoid cracking. Grout head pressure was insignificant for the range of grout pad heights, so packing tape was sufficient to connect matching ends of the form. A layer of putty was placed around the perimeter of the joint between the form and the concrete surface. Figure 4-13B shows an installed form.

Figure 4-13. Grouted torsion base plate setup. A) Cardboard separator. B) Plexiglas formwork for grouted tests. (Photos courtesy of author.)

4.4.2.2 Instrumentation

Figure 4-14A, Figure 4-15A, and Figure 4-16A display a view of fully instrumented six-bolt 5/8 in. diameter, three-bolt 5/8 in. diameter, and three-bolt 1 in. diameter bolt test specimens, respectively. A plan view schematic of base plate instrumentation is provided in Figure 4-14B, Figure 4-15B, and Figure 4-16B. The following instrumentation was implemented for all Torsion testing in all configurations:

- Two moment-compensating load cells installed behind the actuators (see Figure 4-5)
- Through-hole load cells, one per anchor bolt, installed above the base plate with washers both above and below the cells. In the final 2 5/8 in. bolt tests conducted (T5 and T7), three new load cells were used with spherical washers placed above the cells (BT 1-6)
- Two horizontally oriented LVDTs installed behind base plate assembly loading wings
- Vertically oriented LVDTs (V1, V2, V3, V4)
Figure 4-14. Six-bolt 5/8 in. torsion setup. A) Fully instrumented specimen. B) Plan view detail of base plate assembly bolt numbers and instrumentation. (Photo courtesy of author.)

Figure 4-15. Three-bolt 5/8 in. torsion setup. A) Fully instrumented specimen. B) Plan view of base plate assembly bolt numbers and instrumentation. (Photo courtesy of author.)

Figure 4-16. Three-bolt 1 in. torsion setup. A) Fully instrumented specimen. B) Plan view detail of base plate assembly bolt numbers and instrumentation. (Photo courtesy of author.)
4.4.3 Tests Requiring Special Treatment

Several Torsion tests were conducted with a limited number of specimens to study the minor variables discussed at the beginning of this section. Four tests required unique treatment in their experimental setup beyond what was described in Section 4.3.1 and are discussed in further detail in subsequent sections.

4.4.3.1 T10: Fiber-reinforced polymer (FRP) wrap around grouted stand-off base plate

One torsion test contained a 4 dia. grout pad and an FRP retrofit. Guidelines provided by the FRP manufacturer were followed for preparation and installation. Design strength of the composite FRP system was 4.8 kips/in. of width per layer. The force required to be restrained by the FRP was calculated using two methods proposed in Cook and Halcovage (2007), which both produced a one-layer minimum. Three layers were conservatively used to ensure bolt steel rupture.

Fiber strips were cut to a height equal to the distance from the concrete surface to the top of the base plate and a length equal to the circumference of the base plate plus an additional six inches. To impregnate dry fibers, the strips were spooled around a small section of PVC tubing. An epoxy bath was made using a small plastic tub lined with plastic sheeting. A fiber strip was then unspooled into the tub, where a coat of epoxy was brushed on and rolled into the fibers using a toothed roller. The impregnated strip was then re-spooled. This process was repeated on the other side of the fibers, again unspooling and re-spooling in the same manner.

FRP application to the grout pad is illustrated in Figure 4-17A. The grout pad surface was first painted with a base coat of epoxy, after which the saturated layers of FRP were installed one at a time. To avoid material buildup, the starting point for each layer was set at 120 degrees from the previously installed layer. After the FRP had cured, it was found that excess epoxy material had settled at the interface between the concrete surface and the bottom of the grout pad, creating
an epoxy fillet. It was believed that this might unconservatively contribute strength to the retrofitted grout pad that would not be accounted for in design, so an oscillating fine-toothed cutting tool was used to separate the concrete surface from the epoxy material as shown in Figure 4-17B.

![Figure 4-17. Test T10 preparation. A) FRP application to base plate. B) Removal of epoxy fillet material at the FRP-to-concrete interface. (Photos courtesy of author.)](image)

4.4.3.2 T26: Absence of leveling nuts in grouted stand-off base plate connection

One test was conducted without leveling nuts supporting the base plate during grout installation, the base plate resting directly on the grout pad. This test condition is similar to those found within all grouted tests identified in the literature (see Section 2.3.4). To cast the grout pad while the base plate was already in place, the Torsion base plate assembly rested on supports from the concrete to its loading wings. After grout placement, the top not was not pretensioned (as in flush-mounted tests) to simplify the experimental conditions (note that pretensioning bolts in grouted connections with leveling nuts only pretensions the bolt between the top tightening nut and the leveling nut, while pretensioning bolts in connections without leveling nuts extends the pretension force a depth into the material supporting the base plate). Spherical washers were placed above the “tension” load cells as in all other torsion tests.
4.4.3.3 T27: Shim stacks supporting base plate

A single three-bolt Torsion test was conducted with shim stacks used as the method of supporting the stand-off base plate in place of a grout pad and leveling nuts. Figure 4-18 shows the installed shim stacks and instrumentation in this test. Three stacks of three 2 in. by 2 in. by 3/4 in. A36 steel plates were placed at the midpoints between the three 5/8 in. diameter bolt specimens. The metal plates that comprised each stack were welded together, leaving friction interfaces only between 1) the bottom of the base plate and the top of the shims and 2) the bottom of the shims and the concrete surface. Plates were polished to remove any dirt, paint, and rust. Spherical washers were placed above load cells.

![Figure 4-18. Test T27 after setting the base plate on shim stacks. (Photo courtesy of author.)](image)

4.4.3.4 T28: Dry-packed mortar around bolt circle

A single three-bolt torsion test was conducted with “worst-case” field installation conditions with a low-strength dry-packed mortar installed between the outer perimeter of the bolt circle and the edge of the base plate. To restrict mortar movement beyond the bolt circle, the core of the stand-off void between the base plate and the concrete surface defined by the inside of the bolt circle was filled with a compressible foam. Duct tape was then placed around the three bolts and foam interior. Mortar was mixed to a putty-like consistency, packed into the void, and reinforced with a Plexiglas form. The resulting annular mortar layer had an annular
width of approximately 1 in. and a height of 2.63 in. Figure 4-19 shows the duct-taped outer bolt circle before mortar packing and the form-supported dry-packed mortar bed.

Figure 4-19. Test T28 duct tape around outer bolt circle. (Photo courtesy of author.)

4.4.4 Test Preparation and Procedure

The test procedure used for Torsion testing was broken into three distinct stages: Initial Setup, Final Instrumentation and Bolt Preloading, and Loading. The three subsections below are outlined in list format to demonstrate the procedure in the stages.

4.4.4.1 Stage 1: initial setup

Initial setup consisted of the following procedural steps:

1. The concrete block with embedded anchor groups was placed supports, ensuring that the top surface of the concrete was level.

2. For tests other that were not flush-mounted, leveling nuts and washers were placed on the anchor bolts. For the flush-mounted tests, T1-A and T1-B, a 0.03 in. Teflon layer was placed on the concrete surface to reduce friction between the concrete and the base plate.

3. Stand-off distance was set as the distance from the top of the concrete surface to top of the leveling washer, with measurements made at the bearing end of every anchor bolt on the base plate. The base plate assembly was then seated on the washers. In the flush-mounted test, the base plate assembly was seated directly on the Teflon layer. For tests with grout, the grout pad was installed at this stage as described in Section 4.2.3.

4. The rolling reaction frames were secured to the concrete block by preloading the 1.5 in. diameter tension ties against the frames.
5. Actuators were attached to the reaction frames and loading rods were sent through the actuators and frames.

6. Actuator height was set by attaching levels to the loading rods, which were connected to the base plate assembly through a clevis connection.

4.4.4.2 Stage 2: final instrumentation placement and bolt pre-loading

Final instrumentation and bolt pre-loading consisted of the following procedural steps:

1. The LabVIEW program was run without recording data to view instrument readings.

2. LVDT plunger positions were set such that readings will stay in the linear range of the instrument output throughout the test.

3. Load cells were placed into testing position to finish preliminary setup.

4. Communication between instrumentation and the data acquisition system was verified.

5. Data recording began.

6. Anchor bolts were preloaded to FDOT specifications using the “turn-of-the-nut” method, defined as 1/3 turn past snug tight. For the flush-mounted test, T1, hand-tightness was used instead to minimize frictional forces in the base plate connection to the concrete. To execute the method, (a) snug tightness was performed as the AISC definition of the “full effort of an ironworker with an ordinary spud wrench.” Torque wrenches were not used due to compatibility constraints; therefore, snug tightness was performed as consistently as possible by the same test operator within and between tests with an approximate 100 lb. of effort applied at a 1 ft. torque arm. Bolts were snug tightened in a star pattern (e.g. bolts 1, 4, 2, 5, 3, 6); (b) The operator repeated the previous step process in the same order to ensure that all bolts satisfied the definition of snug tightness; (c) anchors were marked to indicate when an additional 1/3 turn of the nut would be achieved. Every nut was turned to the final position in same order as for snug tightening using an extended lever arm for torque.

7. The position of every LVDT was checked or reestablished.

8. Enough preload was applied to the actuator load cells to set their position.

4.4.4.3 Stage 3: loading

Loading consisted of the following procedural steps:

1. Two operators manned separate but identical hand pumps providing flow to their respective actuators as shown in Figure 4-20. The LabVIEW user interface displayed load values for the actuators. These values were used by the operators to maintain nearly equivalent load levels (within approximately 400 lb. at any given time) throughout the test.
2. The operators brought their loads into agreement between 1,000 and 2,000 lb.

3. Step loading was applied in the linear range of each test in approximately 1,000 lb.
   increments with short pauses at the end of each load step to recover from any loading
discrepancies. As the test transitioned into non-linear behavior, load was continually applied
until failure. In some cases tests were stopped to take pictures, readjust LVDTs, and examine
behavior.

Figure 4-20. Independent hydraulic hand pumps used in Torsion testing. (Photo courtesy of
author.)

4.4.5 Treatment of Experimental Data

To demonstrate the treatment of Torsion data, a single rotated arm on the base plate
loading apparatus is considered as illustrated in Figure 4-21. The load experienced by anchor
bolts results only from the tangentially applied load, \( P_{AT} \). Bolt displacement is taken as the
overall circumferential displacement around the bolt circle, which will be larger than the linearly
measured displacement.

As the apparatus is loaded equally by both actuators, the rotation of the loading wing about
the center of the apparatus can be found from the linear displacement measured by the horizontal
LVDTs as given in (4-1), with a horizontal displacement, \( \delta_h \), given by (4-2). The horizontal
displacement produces an angle between the loading actuator and the point of load application on
the loading wing. By geometry it can be found that angle of applied load with respect to its
tangential component to the bolt circle is the sum of $\theta_A$ and $\theta_w$. Thus, the tangentially applied
load, $P_{A,T}$, can be determined in equation (4-5). The bolt displacement, $\delta_h$, is taken as the
circumferential movement due to angle $\theta_w$, as given in (4-6).

$$\theta_w = \arcsin \left( \frac{\delta_{meas}}{r_{load}} \right)$$  \hspace{1cm} (4-1)

$$\delta_h = r_{load} \left( 1 - \cos(\theta_w) \right)$$  \hspace{1cm} (4-2)
\[ \theta_A = \tan \left( \frac{\delta_h}{l_{AW,i} - \delta_{meas}} \right) \]  

(4-3)

\[ \theta_T = \theta_A + \theta_w \]  

(4-4)

\[ P_{AT} = P_A \cos(\theta_T) \]  

(4-5)

\[ \delta_{bolt} = \theta_w r_{bolt} \]  

(4-6)

where

- \( \theta_w \) = angle of rotation of the loading wing about apparatus centroid
- \( \theta_A \) = rotation of applied load about actuator
- \( \theta_T \) = angle between applied load and tangential component to bolt circle
- \( r_{load} \) = radius from bolt circle centroid to load application = 5 in.
- \( r_{bl} \) = radius from bolt circle centroid to center of the bolt circle
- \( l_{AW,i} \) = initial length from applied load to loading wing \( \approx 38 \) in. and 56 in.
- \( P_A \) = applied load
- \( P_{AT} \) = tangential component to bolt circle of applied load
- \( \delta_{meas} \) = measured linear displacement of loading wing
- \( \delta_h \) = horizontal component displacement from apparatus rotation
- \( \delta_b \) = circumferential bolt displacement

As indicated in the definition of \( l_{AW,i} \) in the list above, the two loading rods corresponding to the two loading wings were of different lengths to accommodate the eccentric placement of test specimens with respect to the midline of the block. The maximum displacement measured in torsion testing was under two inches. If this extreme displacement is assumed with the lower value of \( l_{AW,i} \), (4-7) through (4-12) show the difference between the adjustment factor for \( P_{AT} \) with \( \theta_A \) as given in (4-3), resulting in (4-11) and with \( \theta_A = 0 \), resulting in (4-12).

\[ \theta_w = \sin \left( \frac{2 \text{ in.}}{5 \text{ in.}} \right) = 0.411 \text{ rad.} \]  

(4-7)

\[ \delta_h = (5 \text{ in.}) \left(1 - \cos(0.411 \text{ rad.})\right) = 0.417 \text{ in.} \]  

(4-8)

\[ \theta_A = \tan \left( \frac{0.417 \text{ in.}}{38 \text{ in.} - 2 \text{ in.}} \right) = 0.0116 \text{ rad.} \]  

(4-9)
\[ \theta_T = 0.411 \text{ rad} + 0.0116 \text{ rad} = 0.423 \text{ rad} \]  
(4-10)

\[ P_{AT} = P_A \cos(0.423\text{rad}) = 0.912P_A \]  
(4-11)

\[ P_{AEST} = P_A \cos(\theta_\omega) = P_A \cos(0.411 \text{ rad}) = 0.917P_A \]  
(4-12)

where \( P_{AEST} \) = estimated corrected load neglecting actuator rotation

The error in this extreme case for not considering \( \theta_A \) is 0.5%. The error will be smaller for the longer value of \( l_{AW,i} \), 56 in. Because of this relatively insignificant error, it was concluded that the difference in the two \( l_{AW,i} \) lengths did not play a significant role on the loading behavior of the anchor bolt group. Thus, the correction factor taken in the reduction of data can be taken without considering the effects of \( \theta_A \) with no loss in data integrity.

Given the simplifying assumptions from the previous paragraph, (4-13) through (4-16) show how data were treated for determining the load and displacement values reported for anchor bolts in the torsion test setup. The ultimate displacement is reported in the results as (4-15) and the ultimate load on a per-bolt basis is reported in the results as (4-16).

\[ \theta_{w,exp} = a \sin \left( \frac{\delta_{\text{meas.ave}}}{r_{load}} \right) \]  
(4-13)

\[ P_{AT,exp} = P_{A,ave} \cos(\theta_{w,exp}) \]  
(4-14)

\[ \delta_{\text{bolt}} = \theta_w r_{\text{group}} \]  
(4-15)

\[ V_{\text{bolt}} = \frac{2P_{AT,exp} r_{load}}{n r_{\text{group}}} \]  
(4-16)

where \( \theta_{w,exp} \) = angle of rotation of the loading wing taken for experimental data  
\( \delta_{\text{meas.ave}} \) = average of two LVDT measurements at each loading wing  
\( P_{AT,exp} \) = tangential applied load taken for experimental data  
\( P_{A,ave} \) = average applied load from two actuators  
\( V_{\text{bolt}} \) = shear load experienced by each bolt  
\( n \) = number of bolts in the circular group
\[ \tau_{\text{group}} = \text{bolt group radius} = 5 \text{ in.} \]
\[ \tau_{\text{load}} = \text{radius of applied load} = 5 \text{ in.} \]

4.5 Large-scale Anchor Bolt Groups under Predominantly Torsion Loading

4.5.1 Experimental Design

In Large-scale testing, circular bolt groups representing field installations of annular sign and signal field connections were conducted to investigate the possibility of size effects for larger bolts. Four tests were conducted on groups of six 1.25 in. diameter anchor bolts under predominantly torsion loading as shown in Table 4-4. As a baseline, test LS1 was left ungrouted with exposed lengths equal to one bolt diameter. Test LS2 was grouted with identical dimensions to LS1. An “extreme” exposed length value of 3 nominal anchor bolt diameters was chosen in grouted tests LS3 and LS4. Tests LS1-LS3 contained a circular base plate while LS4 contained an annular base plate.

Table 4-4. Large-scale test matrix

<table>
<thead>
<tr>
<th>Test ID ( \text{a} )</th>
<th>Hole Size (in.)</th>
<th>n</th>
<th>Concrete</th>
<th>Install. Method</th>
<th>Bolt Material</th>
<th>Conn. Type ( b )</th>
<th>Top Nut Tightness ( c )</th>
<th>( \ell_{\text{RF}} )</th>
<th>( \ell_{\text{LN}} )</th>
</tr>
</thead>
<tbody>
<tr>
<td>LS1</td>
<td>1.25</td>
<td>6</td>
<td>N</td>
<td>CIP</td>
<td>F1554</td>
<td>U</td>
<td>TOTN</td>
<td>2.3</td>
<td>1</td>
</tr>
<tr>
<td>LS2</td>
<td>1.25</td>
<td>6</td>
<td>N</td>
<td>CIP</td>
<td>F1554</td>
<td>G</td>
<td>TOTN</td>
<td>2.3</td>
<td>1</td>
</tr>
<tr>
<td>LS3</td>
<td>1.25</td>
<td>6</td>
<td>N</td>
<td>CIP</td>
<td>F1554</td>
<td>G</td>
<td>TOTN</td>
<td>4.3</td>
<td>3</td>
</tr>
<tr>
<td>LS4</td>
<td>1.25</td>
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<td>N</td>
<td>CIP</td>
<td>F1554</td>
<td>G</td>
<td>TOTN</td>
<td>4.3</td>
<td>3</td>
</tr>
</tbody>
</table>

\( ha = \) not applicable
\( a \) DS = Direct Shear, T = Torsion, LS = Large-scale, ES = Eccentric Shear
\( b \) N = Normal-strength Concrete, L = Low-strength Concrete
\( c \) FM = Flush-Mounted base plate, U = Ungrounded stand-off base plate with leveling nut, G = Grouted stand-off base plate, GF = Grouted stand-off base plate with FRP retrofit
\( d \) FT = Finger-Tightened, TOTN = Turn-of-the-Nut

4.5.2 Test Setup

4.5.2.1 Structural components

The test setup was designed to reuse the loading assembly from FDOT reports BD545-54, BDK75 977-04, and BDK75 977-32. The loading assembly, which was designed as a half-scale model in BDK545-54 for a failed cantilever sign structure, contained 12 1.75 in. diameter holes at a 10 in. bolt circle radius with a base plate thickness of 1 in. With FDOT’s allowable 0.5 in. hole oversize, 1.25 in. anchors were conservatively chosen. Among the mast arm signal base
plates surveyed in Gainesville, FL (discussed in Section 4.4.2.1), one specimen contained six 1.25 in. anchor bolts at a similar bolt circle radius to the loading assembly. The base plate thickness to bolt diameter ratio (0.8) fell within the range of values in Table 4-3. Thus, it was determined that the test design was conservative in representing existing structures.

Each test specimen was composed of a 6 ft. x 10 ft. x 3 ft. deep reinforced concrete block hosting a single anchor bolt group in the center as shown in Figure 4-22. Blocks were tied to the laboratory strong floor using steel beams and 1.5 in. diameter threaded rods. Load was applied by a hydraulic actuator placed at a 9 ft. torsion arm through a steel pin connection.

![Figure 4-22. Labeled Large-scale test setup. (Photo courtesy of author.)](image)

A front view schematic of the reinforcement scheme and support condition 10 ft. by 6 ft. by 3 ft. deep reinforced concrete blocks is provided in Figure 4-23. Anchor bolts were positioned such that the pipe loading assembly, which was offset 15 degrees from the nearest bolt circle radius through one of the base plate holes, would be initially parallel to the ground surface. An 8
by 15 grid of Number 4 top reinforcement in the concrete blocks was designed to accommodate anchor bolt breakout forces as described for Torsion block specimens in Section 4.4.2.1. Bottom reinforcement duplicated the top reinforcement grid and was adequate for creep, shrinkage, and handling. A minimum cover of 3 inches was used for reinforcement. Additional #8 bars were placed at the top and bottom of the blocks to restrain block bending moments from the applied load and reactions. Figure 4-24 shows the formwork, rebar, and anchor bolt placement of a representative Large-scale test before concrete placement.

![Diagram of reinforcement layout](image)

**Figure 4-23.** Front views of anchor placement details within Large-scale test blocks.

![Concrete formwork with rebar and anchor bolts](image)

**Figure 4-24.** Large-scale anchor bolt and rebar and rebar placement before concrete placement. (Photo courtesy of author.)
The loading assembly, illustrated in Figure 4-25, was composed of a 24 in. diameter, 1 in. thick circular base plate with 12 1.75 in. diameter holes. For test LS4, a 15.5 in. diameter circle was cut out of the center of the base plate. The base plate was welded to HSS 16.000 x 0.500 steel pipe assembly containing a 23 in. stub welded to a 120 in. loading arm.

4.5.2.2 Instrumentation

Figure 4-26 displays a view of a fully instrumented test specimen and a plan view schematic of base plate instrumentation within the test setup. The following instrumentation was implemented for all Large-scale testing:

- A load cell threaded into the loading actuator (see Figure 4-22)
- Six through-hole load cells installed on anchor specimens directly above the base plate assembly
- Two horizontally oriented LVDTs installed at base plate mid-height (H1 and H2)
- Three vertically oriented LVDTs (V1-V3)
4.5.3 Test Preparation and Procedure

To prepare for tests, the loading assembly was installed for every test with the block resting on the opposite face from the location of the anchor bolts. Procedural stages 1 and 2 described for Torsion testing were followed where applicable for setting stand-off distance and recording pre-loaded values in the bolt tension load cells. Concrete bug holes directly below the base plate were filled with putty so that grout would not fill these spaces and contribute mechanically to the shear strength of the grout pad (note: bug holes were not present in Torsion testing). After the anchor bolts were preloaded, grout was installed and treated in applicable tests following the methodology presented in 4.2.3. After the grout pad had cured for a minimum of 24 hours the concrete block was moved into the test position, instrumentation was installed, and the tie-down system was set in place.

Tests were displacement-controlled, with load supplied through an electric hydraulic pump manned by an operator. In the linear-elastic portion of each test the loading rate was set at 140
approximately 150 lb. per second on the 9 ft. loading arm, equivalent to a torque rate of approximately 100 kip-in./second. Displacement rate was increased through the much larger inelastic phase of test behavior. At the loading arm, displacements nearing 30 inches were observed as the anchor bolts deformed, requiring in-test adjustment of the loading actuator. During adjustment periods, the loading arm was supported by an overhead crane while the actuator was retracted and reset. A shorter actuator was used at the beginning of the test and replaced by a taller actuator after one or two stroke cycles. Other than stopping for actuator adjustment, the consistent quasi-static displacement was continued until anchor bolt rupture.

4.5.4 Treatment of Experimental Data

To demonstrate the treatment of Large-scale data to produce bolt forces and displacements, the rotated arm on the base plate loading apparatus is considered as illustrated in Figure 4-27 and Figure 4-28 showing the overall loading assembly and the rotated base plate and bolt line, respectively.

![Figure 4-27. Illustration of forces and dimensions in displaced Large-scale test.](image)
The overall rotation of the loading assembly about the bolt circle centroid, $\theta_{arm}$, can be found from the displacement measured by the string potentiometers, which wrapped around the base plate as it rotated, resulting in a circumferential measurement. $\theta_{arm}$ was ultimately be used to determine the individual bolt force and displacement in (4-19) and (4-20), respectively. The small overturning moment arm was found to have very little influence on results and was thus not considered in the treatment of the data.

\[
\theta_{arm} = \frac{\delta_{meas}}{r_{meas}} \tag{4-17}
\]

\[
P_{AT} = P_A \cos(\theta_{arm}) \tag{4-18}
\]

\[
V_b = \frac{P_T d_{arm}}{n r_{bolt}} \tag{4-19}
\]

\[
\delta_b = \theta_{arm} r_{bl} \tag{4-20}
\]

where

- $\theta_{arm}$ = rotation of the loading arm
- $\delta_{meas}$ = measured circumferential displacement
- $\delta_b$ = measured circumferential displacement
- $r_{meas}$ = radius from bolt circle centroid to $\delta_{meas}$
- $d_{arm}$ = radius from bolt circle centroid to load application = 96 in.
- $r_{bl}$ = radius from bolt circle centroid to center of the bolt circle

Figure 4-28. Illustration of displacements and dimensions on base plate and bolt circle.
4.6 Eccentric Shear Loading of Rectangular Bolt Groups

4.6.1 Experimental Design

The strength of anchor bolts experiencing combinations of axial and shear loading was investigated using an Eccentric Shear loading setup, where a horizontal load was applied at various eccentricities above bolt groups to produce various magnitudes of tension/compression to shear on bolts. Figure 4-29 shows the Eccentric Shear test setup.

![Eccentric Shear test setup. (Photo courtesy of author.)](image)

Table 4-4 details the 33 tests that were conducted on 0.625 in. diameter anchor bolts under combined axial and shear loading. Bolt groups were arranged in three configurations: two-bolt groups under combined tension and shear, two-bolt groups under combined compression and shear, and four-bolt groups, all with and without grout pads installed in the void between the base plate and concrete. The following variables were investigated in this portion of the study: 1) exposed length of anchor bolts (major variable); 2) base plate support condition:
ungrouted stand-off base plates, grouted stand-off base plates (major variable); 3) ratio of applied axial-to-shear load (major variable): various combinations of tension and shear (two-bolt), compression and shear (two-bolt), and both tension and compression (four-bolt); 4) bolt material – ASTM F1554, ASTM A354 BD, and ASTM A276 Type 316 stainless steel; 5) bolt pretensioning; and 6) effect of friction in grouted installation.

Table 4-5. Eccentric shear test matrix.

<table>
<thead>
<tr>
<th>Set</th>
<th>$d_b$ (in.)</th>
<th>Hole Size (in.)</th>
<th>$n$</th>
<th>Concrete</th>
<th>Install. Method</th>
<th>Bolt Material</th>
<th>Conn. Type b</th>
<th>Top Nut Tightness c</th>
<th>$l_{BP} / d_b$</th>
<th>$l_{LN} / d_b$</th>
<th>Load Type</th>
<th>Load Ecc. e (in.)</th>
</tr>
</thead>
<tbody>
<tr>
<td>ES1</td>
<td>0.625</td>
<td>0.81</td>
<td>2</td>
<td>N</td>
<td>CIP</td>
<td>F1554</td>
<td>U</td>
<td>FT</td>
<td>1.2</td>
<td>0</td>
<td>TS</td>
<td>5.3</td>
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<td>ES2</td>
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<td>0.81</td>
<td>2</td>
<td>N</td>
<td>CIP</td>
<td>F1554</td>
<td>U</td>
<td>FT</td>
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<td>N</td>
<td>CIP</td>
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<td>U</td>
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<td>U</td>
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<td>U</td>
<td>TOTN</td>
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<td>U</td>
<td>FT</td>
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<td>U</td>
<td>TOTN</td>
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<td>U</td>
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<td>3</td>
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<td>CS</td>
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<td>CIP</td>
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<td>U</td>
<td>FT</td>
<td>4.2</td>
<td>3</td>
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<td>4</td>
<td>N</td>
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<td>U</td>
<td>TOTN</td>
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<td>TCS</td>
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<td>N</td>
<td>CIP</td>
<td>F1554</td>
<td>U</td>
<td>TOTN</td>
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<td>TCS</td>
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<td>G</td>
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<td>G</td>
<td>TOTN</td>
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<td>G</td>
<td>TOTN</td>
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<td>G</td>
<td>TOTN</td>
<td>4.2</td>
<td>3</td>
<td>TCS</td>
<td>24</td>
</tr>
</tbody>
</table>

\(^a\)DS = Direct Shear, \(T\) = Torsion, \(LS\) = Large-scale, \(ES\) = Eccentric Shear
\(^b\)N = Normal-strength Concrete, \(L\) = Low-strength Concrete
\(^c\)FM = Flush-Mounted base plate, \(U\) = Ungrounded stand-off base plate with leveling nut, \(G\) = Grouted stand-off base plate, \(GF\) = Grouted stand-off base plate with FRP retrofit
\(^d\)FT = Finger-Tightened, \(TOTN\) = Turn-of-the-Nut
\(^e\)Measured to the top of the concrete surface
4.6.2 Test Setup

4.6.2.1 Structural components

A schematic of the Eccentric Shear loading setup is given in Figure 4-30. Load was applied to the steel base plate loading assembly through a hydraulic hand pump to a 120-kip telescoping actuator, which was restrained by a double c-channel steel frame tied into the laboratory strong wall/floor. The base plate assembly transferred load to anchor bolts installed in a 46.5 in. by 46.5 in. by 12 in. deep concrete block, which was restrained by a steel beam assembly, also tied to the laboratory strong floor.

Figure 4-30. Labeled schematic of Eccentric Shear test setup.
The base plate assembly consisted of a 2 in. thick by 16 in. long by 10 in. wide ASTM A572 Gr. 50 base plate, a 24 in. W8x28 A36 steel section, and a 1 in. by 3 in. by 24 in. steel loading “tab,” also made with A572 Gr. 50 steel. As with the torsion tests on 5/8 in. bolts, holes were oversized with a diameter of 0.81 in. To allow for the placement of tension load cells without inducing additional stretch length during the pretensioning process, anchor bolt holes were countersunk with 3 in. diameter holes to a thickness of 1 in. Load was applied laterally to the loading tab at approximate 6 in., 12 in., 18 in., and 24 in. eccentricities to the top of the concrete surface. Clusters of three overlapping 1 in. diameter holes were drilled at 5/8 in. center-to-center spacing into the loading tab to allow for anchor bolt exposed lengths of one, two, and three bolt diameters while maintaining a constant distance to the concrete surface for each loading eccentricity. Figure 4-31 and Figure 4-32 show profile and plan views of the base plate assembly, respectively, while Figure 4-33 shows the weld pattern used to connect the base plate assembly components.

Figure 4-31. Profile view of Eccentric Shear base plate assembly dimensions.
Figure 4-32. Plan view of Eccentric Shear base plate assembly dimensions.

Figure 4-33. Isometric schematic of Eccentric Shear base plate assembly component connection.
Rollers were implemented in two-bolt groups to achieve equilibrium in the transfer of load through the connection, resulting a defined internal moment arm between the test specimens and the roller reaction. In tension and shear tests, the roller was placed underneath the leading (nearest the point of load application) side of the base plate, while in compression and shear tests the roller was placed above the trailing side of the base plate. Roller locations can be seen in Figure 4-35A and Figure 4-36A for tension and compression, respectively.

Grout pads were cast to dimensions of 10 in. (the width of the base plate) by approximately 7 in. using Plexiglas formwork. Grout was introduced to the formed volume through the base plate’s central holes following the procedure outlined in Section 4.2.3.

The reinforcement and bolt placement scheme for concrete blocks are given in Figure 4-34. Blocks were designed to resist concrete breakout forces corresponding to full bolt tension and shear capacities as well as bending moments arising from the application and restraint of load through the concrete block. Tension forces from moment in various loading cases dominated the design, resulting in ten No. 4 bars at both the top and bottom faces of the blocks.

Figure 4-34. Eccentric Shear anchor bolt and rebar and rebar placement before concrete placement. (Photo courtesy of author.)
4.6.2.2 Instrumentation

Figure 4-26A, Figure 4-35A, and Figure 4-36A display views of representative fully instrumented test specimens in two-bolt combined tension and shear tests and combined compression and shear tests, respectively. Plan view schematics of the placement of instrumentation are also provided. The placement of rollers for moment reaction restraint is also shown in Figure 4-35 and Figure 4-36; tension/shear tests contained a roller initially placed below the base plate 12 in. horizontally from the centerline of the two test specimens, while compression/shear tests contained a roller initially placed above the base plate 12 in. horizontally from the centerline of the two test specimens. In all tests, the following instrumentation was present:

- A load cell threaded into the loading actuator (see Figure 4-30)
- 3 in. diameter through-hole load cells installed on test specimens
- Two horizontally oriented LVDTs installed at base plate mid-height (H1 and H2)
- Two vertically oriented LVDTs (V1 and V2)

Figure 4-35. Two-bolt tension and shear test. A) Fully instrumented specimen. B) Plan view detail of base plate. (Photo courtesy of author.)
Holes in the 3 in. diameter bolt tension load cells housed within the 3.1 in. diameter base plate holes were approximately 0.01 in. greater than the outside pitch diameter of the bolts. Given the tolerances from the outer and inner diameters of the annular load cells within the test setup, minor errors in anchor bolt placement (less than 1/16 in.) in some cases did not allow for multiple load cells to be present in tests. In these cases, annular “pucks” with an outer diameter of 2.9 in. and an inner diameter of 0.625 in. were placed above anchor bolt specimens in some cases in place of the tension load cells.

4.6.3 Tests Requiring Special Treatment

Several Eccentric Shear tests were conducted with a limited number of specimens to study the minor variables discussed at the beginning of this section. Five tests required unique treatment in their experimental setup beyond what was described in Section 4.6.2 and are discussed in further detail in subsequent sections.
4.6.3.1 ES21, ES22, ES31, ES32: four-bolt tests

Four tests, two grouted and two ungrouted, were conducted with four-bolt arrangements, all containing anchor bolt exposed lengths of three diameters. One of each of the grouted and ungrouted specimens was subjected to a lower eccentricity and one of each to a higher eccentricity. Two of the four bolts in each test resisted the tension component and two resisted the compression component of overturning moment, requiring no additional reaction restraint to maintain equilibrium. As such, no roller was implemented in four-bolt tests.

![Figure 4-37. Four-bolt test. A) Fully instrumented specimen. B) Plan view detail of base plate. (Photo courtesy of author.)](image)

4.6.3.2 ES30: effect of friction in grouted installation

In one grouted compression and shear test, double layers of Teflon were installed above and below the grout pad to demonstrate the effect of friction in resisting shear force in a grouted assembly. This test can be directly compared with ES29, which was identical less the presence of the Teflon sheets. The two double layers of Teflon are shown in Figure 4-38.
4.6.4 Test Preparation and Procedure

To prepare every Eccentric Shear test, the concrete block was set into place onto two steel beams using an overhead crane with adjustments made as necessary such that the top surface of the block was level with steel shims below the block. Horizontal block position adjustments were made until the centroid of the bolt group was in line with the loading rod. In two-bolt tension and shear and four-bolt tests the rectangular tube section restraining rotation of the block was set directly against the concrete surface, while in two-bolt compression and shear tests the tube section was placed directly above the back end of the base plate. The stand-off height of the test was established on both the front and the rear of the base plate. The loading rod and actuator were set collinearly with attachment point on the base plate to produce the desired loading eccentricity. All remaining instrumentation and test components (e.g. rollers, clevis hinge) were installed. Pretensioning as indicated in Table 4-5 for each test was then conducted, securing the position of the loading apparatus.

Load was applied to the single telescoping actuator through a hand pump until failure with most tests being lasting between five and ten minutes. As with other test setups, loading was paused for observational and instrument resetting purposes.
4.6.5 Treatment of Experimental Data

4.6.5.1 Ungrouted tests

A profile schematic of the ungrouted two-bolt combined tension and shear Eccentric Shear test setup is shown in Figure 4-39. The location of the free-body cut used to determine anchor bolt tensile forces was the point of anchor bolt contraflexure, assumed to be the midpoint of the exposed length, as indicated with a dashed line. Figure 4-40 shows the external and internal forces in the free body resulting from this cut for the three types of test conducted.

Through static moment and force equilibrium about the bottom of the anchor bolt, the forces within anchor bolts can be found. (4-21) and (4-22) show the internal moment arm used for calculating for all of the test types. In four-bolt tests it is assumed that the neutral axis of bending travels through the centroid of the four-bolt group and that shear forces are distributed equally between all four bolts.

Figure 4-39. Profile schematic of displaced two-bolt tension and shear Eccentric Shear test.
Figure 4-40. Free-body illustrations of Eccentric Shear tests. A) Two-bolt tension and shear. B) two-bolt compression and shear. C) Four-bolt.

\[ N_A = \frac{P e}{2d_{arm}} \]  
\[ V_A = \frac{P}{n} \]  

where  
\( d_{arm} \) = internal arm between compressive and tensile reactions = 12 in.  
\( e \) = eccentricity of applied load  
\( P \) = applied load  
\( V_A \) = calculated shear applied to an individual bolt  
\( N_A \) = calculated tension/compression applied to an individual bolt  
\( n \) = number of bolts

To visualize the effect of the roller in two-bolt tests on the internal moment reaction to the applied load, consider a roller section that is pinned in place but free to rotate. Now consider horizontally oriented plates above and below the roller connected to the roller at precisely 180 degrees from each other, both plates fixed against rotation and free to displace horizontally. Viewing a cross-section of this assembly, rotate the roller one radian clockwise. The roller imparts one roller radius of rightward horizontal displacement on the top plate and one radius of leftward horizontal displacement on the bottom plate. Relative to each other, the plates move two roller radii. Relative to each plate individually, the roller moves one radii. Irrespective of roller diameter, the relative movement between plates will double the movement between the roller and
either plate individually. Changing the boundary conditions such that the bottom plate is fixed and the roller can now move horizontally does not change the relative movement. Thus, the roller movement relative to the concrete surface will be half of the base plate relative to the concrete surface. Accordingly, the roller will only experience one-half of the measured base plate displacement, changing the internal moment arm between the roller and the centroid of the displaced bolt by this value.

The half-traveled distance of the roller compared to the base plate is additive to the internal moment arm resisting overturning moment in compression and shear tests and subtractive in the tension and shear tests. The distance traveled by the roller, however, is coincidentally offset by the placement of the cut on the midpoint of the anchor bolt exposed length, as the displacement of that point is also one-half of the measured base plate displacement. Thus, somewhat circuitously, the 12 in. initial internal moment arm circuitously remains constant throughout the loading process. Resulting values of $V_A$ and $N_A$ from the experimentally applied ultimate loads in Chapter 5 were tested against the predictions formed in Chapter 6. The assumptions of load distribution for the four-bolt tests are discussed and evaluated in these chapters.

4.6.5.2 Grouted tests

The presence of grout, as discussed in detail in the Chapters 2 and 6, often produces clamping forces on the grout pad, tensile forces in anchor bolts, and a friction contribution to the resistance of applied shear loads. Thus, $V_A$ and $N_A$ (i.e. $T_A$ or $C_A$) are resisted by anchor bolt steel and the grout, which is built into the analytical models in Chapter 6 for predicting the strength of grouted tests. Furthermore, the influence of the vertical reaction from grout must be considered when determining the assumed internal moment arm dimension. In two-bolt tests, where applicable, it was assumed that the vertical reaction from grout clamping occurred equally in front of and behind the bolt line, making the resultant moment arm equal to that in ungrouted
tests. Under this assumption, $N_A$ and $V_A$ were calculated the same as provided in (4-21) and (4-22). Where the applied compression takes all of the shear force in the fully rigid base plate assumption, the significance of the distance in the internal moment arm to the steel strength of anchor bolts vanishes and the problem becomes one of grout fracture mechanics outside of the scope of this dissertation.

Four-bolt tests, which did not contain the stiff roller reaction present in two-bolt tests, presented two unique scenarios for determining the internal moment arm resisting the overturning moment from the eccentrically applied load. In both cases the critical bolts were the tensile bolts, as the compressive reaction from the grout pad prevented significant load transfer into the compressive bolts. It is assumed that the base plate is fully rigid, that the edge of the grout pad is able to withstand and distribute concentrated compression forces, and that the grout pad is infinitely stiff. Under these assumptions, in both ES31 and ES32 the internal moment arm moved to the edge of the grout pad on the compressive side of the overturning moment and the neutral axis of bending shifted to the compressive reaction. As a result, all bolts engaged in tension in both tests.

For the lower-eccentricity test, ES31, the clamping force produced in the tension bolts was not exceeded by the applied tensile force, meaning that grout clamping still occurred in both lines of tensile bolts. In this test it was assumed that the shear force distributed evenly between the four bolts for the sake of comparison to analytical models. With the friction force overcome, the internal moment arm between extreme compressive and tensile forces becomes (4-23).

\[
 d_{arm_{ES31}} = d_{arm_t} + d_{bg,c} - \delta_h \tag{4-23}
 \]

where

- $d_{arm_{ES31}}$ = internal moment arm between extreme tensile and compressive reaction to eccentric shear load for ES31
- $d_{bg,c}$ = distance from bolt line nearest compressive reaction to the edge of the grout pad
For the higher-eccentricity test, ES32, the resultant friction from the compressed grout took approximately all of the applied shear load, engaging all bolts in tension only. Tensile forces experienced by the two lines of bolts, which have the same axial stiffness, were then taken as proportional to their distances from the compressive reaction. For simplicity and given the assumption that grout-to-base plate friction resisted all of the applied shear force, the displacement of the system was ignored in the calculation of the internal moment arm, which was taken as (4-24).

\[ d_{\text{arm}_{ES32}} = d_{\text{arm}} + d_{bg,C} \]  

(4-24)

where \( d_{\text{arm}_{ES32}} \) = internal moment arm between extreme tensile and compressive reaction to eccentric shear load for ES32

### 4.7 Summary

This chapter introduced the test program undertaken for determining the steel strength of anchor bolts in ungrouted and grouted stand-off base plate connections. Four experimental arrangements made up the test program: Direct Shear tests of ungrouted one- and two-bolt connections, Torsion testing of ungrouted and grouted circular bolt groups, Large-scale torsion testing of ungrouted and grouted circular bolt groups, and Eccentric Shear testing producing combinations of shear and normal forces on ungrouted and grouted two- and four-bolt groups. It is believed that this test program comprehensively addressed a wide range of representative loading, exposed length, and base plate support conditions in field installations of anchor bolts installed in stand-off base plate connections. Results of this testing are presented in Chapter 5 and analyzed in Chapter 6.
CHAPTER 5
EXPERIMENTAL RESULTS

5.1 Introduction

This chapter provides a summary and discussion of experimental test results. The treatment of ultimate load and displacement values as reported within this chapter was discussed in Chapter 4. All bolt failure values are expressed on a per-bolt basis in kips and normalized by tensile capacity for immediate comparison between methods with different numbers of bolts and different bolt materials/sizes. Ultimate displacement values are expressed both in inches and normalized by the nominal bolt diameter. Three steel failure locations were observed over the bolt length: directly below the leveling nut, directly above the leveling nut, and at the concrete surface; failure locations are provided for all test results.

Subsections within this chapter highlight the influence of different variables on the load-displacement behavior with accompanying commentary to describe load-displacement trends. This commentary is intentionally not analytically rigorous; rather, it is intended to conceptually introduce the development of models in Chapter 6 and provide basic descriptions of the behavior of tests.

Applied load-displacement results and all other data taken for individual tests are presented in Appendix A for the Direct Shear tests, Appendix B for Torsion tests, Appendix C for the Large-scale tests, and Appendix D for Eccentric Shear tests. Bolt tension and vertical displacement readings are also provided in applicable appendices.

5.2 Ductility of Bolt Materials

This section discusses special tensile tests and treatment of data for materials used in Torsion tests T14-T28 and Eccentric Shear tests for a brief discussion of material ductility. Outside of the ductility tests presented in this section, tensile tests were conducted on specimens
from all unique material types, sizes, and batches of anchor bolt specimens. Ultimate tensile strength for the batch from which each anchor bolt originated is provided within respective sections in “results summary” tables.

A schematic of the tensile specimens used for ductility testing is shown in Figure 5-1. A $5d_{nt}$ gauge length was used, where $d_{nt}$ was taken as the diameter corresponding to the net tensile area of a 5/8 in. diameter bolt, 0.537 in. To prepare specimens, two Grade 8 steel coupling nuts were turned onto the specimen until the gauge length was achieved. On the non-specimen side of either coupling nut, a 7/8 in. diameter ASTM A193 Gr. B7 rod was fastened. The outside ends of the B7 rods were clamped into self-clamping grips built into the Instron Universal Testing Machine used for conducting tests. Load was applied generally following the protocol outlined in ASTM A370, ASTM F606, ISO 898-1, and ISO 6892-1 with a pre-yield displacement rate of 0.04 in./min and a post-yield displacement rate of 0.9 in./min. Figure 5-2 shows the applied load vs. crosshead displacement of these tests, for which three repetitions each for F1554, 354 BD, and Type 316 stainless steel material were conducted.

Figure 5-1. Schematic of ductility tensile test setup.
Crosshead displacement readings, in addition to the extension experienced within the
gauge length of bolt specimens, include the initial systemic slip along with the elastic responses
of the coupling nuts, 7/8 in. diameter rods, and the components of the Instron test machine. To
extract the extension of the test specimens over their gauge length, the following steps were
taken:

1. The total systemic elastic stiffness, $K_{tot}$, was found for a 354BD test by dividing the applied
   load by displacement in the linear portion of test behavior after initial systemic slip and
   before initial yield of the bolt material.

2. The elastic stiffness of the five-diameter specimens, $K_{bolt, EL}$, was calculated with $AE/L$ of
   the bolt, assuming a steel Young’s modulus $E$ of 29000 ksi over the net tensile area $A$ of
   0.226 square inches and a length $L$ of 2.68 in.

3. Because the bolt specimen is in series, i.e. experiencing equal force, with the remainder of
   the test setup contributing to the overall displacement of the test, the inverse of the total
   stiffness, $K_{tot}$, was set equal to the sum of the inverses of the calculated bolt stiffness $K_{bolt, EL}$
   and the unknown “non-bolt” test apparatus stiffness, $K_{sys}$, which was determined by
   rearranging the equation.
4. The test apparatus displacement, $\delta_{sys}$, was calculated throughout the loading process by dividing the load by the stiffness.

5. The extension of the test specimen alone, $\delta_{bolt}$, was taken as the measured crosshead displacement, $\delta_{tot}$, less the test apparatus displacement found in the previous step beginning at 10 kips (the approximate load where initial systemic slip was no longer present). Displacements from the linear-elastic response are negligible, allowing the zero-displacement point to be taken at the 10-kip mark.

Using the extracted values of $\delta_{bolt}$, “true stress” of the specimen was found using the methodology discussed in Section 2.2.3.1, where it is assumed that the cross-section is reduced uniformly over the gauge length until the onset of necking at the ultimate load, after which the calculation of true stress is not valid due to localized strains from the necking process. The gauge length extension, however, is valid throughout rupture, allowing percentage elongation at both ultimate load and bolt steel rupture to be found. Figure 5-3 shows the extracted displacements vs. calculated values of true stress for the three materials used.

![Figure 5-3. Calculated true stress vs. extension of ductility tensile specimens.](image-url)
Table 5-1 summarizes the ultimate loads, ultimate engineering and true stresses, and percent elongations at ultimate load and bolt fracture for the nine tests presented in Figure 5-3. Both elongation measurements are expressed in inches and in terms of their percentage of the original gauge length of the specimens. It can be seen that the 354BD material is clearly significantly more brittle and the Type 316 stainless steel material is significantly more ductile than the F1554 material. The implications of the differences in ductility are discussed in Torsion and Eccentric Shear sections later in this chapter.

Table 5-1. Summary of Direct Shear anchor bolt ultimate load and displacement results

<table>
<thead>
<tr>
<th>Material</th>
<th>( T_o ) (kip)</th>
<th>( T_o ) COV (%)</th>
<th>( \sigma_{o, eng} ) (ksi)</th>
<th>( \sigma_{o, true} ) (ksi)</th>
<th>Ult. Elong. (in.)</th>
<th>Ult. Elong. (%)</th>
<th>Fract. Elong. in.</th>
<th>Fract. Elong. (%)</th>
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<td>354BD</td>
<td>38.3</td>
<td>0.3%</td>
<td>170</td>
<td>183</td>
<td>0.195</td>
<td>7.3%</td>
<td>0.305</td>
<td>11.4%</td>
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<tr>
<td>316</td>
<td>22.0</td>
<td>0.2%</td>
<td>97</td>
<td>117</td>
<td>0.448</td>
<td>16.7%</td>
<td>0.708</td>
<td>26.4%</td>
</tr>
<tr>
<td>F1554</td>
<td>20.0</td>
<td>0.1%</td>
<td>88</td>
<td>100</td>
<td>0.308</td>
<td>11.5%</td>
<td>0.487</td>
<td>18.2%</td>
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</tbody>
</table>

5.3 Direct Shear Testing

5.3.1 Influence of Test Geometry on Bolt Tension Forces

Results from Direct Shear (DS) tests were influenced by the geometry of their displaced shapes. High displacements in the Direct Shear experimental arrangement generated unintended rigid-body rotation of the direct shear setup, resulting in additional tension forces acting on displaced anchor bolts. To accurately compare results from direct shear tests with predictive models, then, second-order effects and the resulting interaction between moment, shear, and tension forces acting on the cross-section must be accounted for due to the tension loads.

To illustrate second-order tensile forces in single-bolt Direct Shear tests, a free-body schematic of a displaced test is provided in Figure 5-4. In this schematic it is assumed that the exposed length of the anchor bolts remains constant with zero-length plastic hinges at the top and the bottom of the exposed length. As the base plate moves horizontally, vertical restraint from the roller produces a vertical reaction, causing the base plate to rotate. As the base plate rotates, a
moment arm from the vertical movement of the point of load application develops. Taking moments about the top of the roller using the displaced shape of the rigid body, the tension in the bolts can be found.

Figure 5-4. Schematic of exaggerated displaced shape of single-bolt Direct Shear specimens

While the rigid body assumption conceptually illustrates well that tension forces developed in Direct Shear tests, it is not refined enough to accurately estimate the magnitudes of the tension forces for each test due to unaccounted-for contributions from a non-zero plastic hinge, leveling nut rotation, unknown exact values of concrete spall depth and bearing stress distribution, plastic deformations due to bending and axial force all impact lengths, Poisson’s effects, plastic shear displacements, etc., many of which can generate considerable effects on the calculated tensile force on the bolt within conceivable ranges of values. Significant effort was given to producing a generalizable format for determining the tension forces, but determining and implementing the unaccounted-for factors proved infeasible. As such, the results from Direct Shear tests can only be understood in relationship to each other and are not used in Chapter 6 for the development of recommended analytical and design models or as a basis for Chapter 7 conclusions.

Two-bolt tests also presented experimental shortfalls. In the absence of a roller, the second-order tension forces described above did not develop in tests. However, the applied load,
which traveled through the center of the base plate, produced an overturning moment within the base plate that translated to tension and compression forces within the rear and front bolts, respectively. Taking a cut at the point of bolt contraflexure similar that shown for Eccentric Shear tests in Figure 5-5, a relatively small, yet existent, overturning moment can be determined for the two-bolt Direct Shear experimental setup.

![Diagram of shear and unintended normal forces in double-bolt Direct Shear tests](image)

Figure 5-5. Schematic of shear and unintended normal forces in double-bolt Direct Shear tests.

In both single- and double-bolt Direct Shear tests, even if the axial forces are determined, it remains that the loading condition on anchor bolts is not pure shear as desired throughout the loading process, further demonstrating the difficulty in achieving this loading condition for bolts in ungrouted stand-off base plate connections discussed in Section 2.3.3.3 of the Literature Review.

### 5.3.2 Summary of Results

Qualified by the limitations discussed above, Table 5-2 provides a summary of Direct Shear test results. The $V_{u, bolt}$ value reported represents the ultimate shear load divided by the number of bolts. The 28-day concrete strength for Direct Shear testing was 4,990 psi. All 5/8 in. and 1 in. bolts were from the same batches. Ultimate tensile strengths, $T_o$, of anchor bolt threaded rod specimens, were determined according to ASTM F606 methodology (ASTM
2011a) using the UF Structures Laboratory Instron machine. Ultimate displacements, $\delta_u$, correspond to the value of $V_{u,bolt}$ for each test.

A limited number of bending length measurements were taken from cored anchor bolt specimens from datasets DS1 through DS8. The bending length, $l_b$, was defined as the distance from the concrete surface to the deepest point below the surface of the concrete where the anchor experienced any bending deformations. Two $l_b$ specimens are shown in Figure 5-6, one with 0.8 dia., the other with 2.8 dia. exposed length. In addition to $l_b$, profiles of concrete spalling in front of and behind the displaced anchor bolt are outlined; it can be seen on both bolts that more concrete spalled away on the trailing end of the bolt and, significantly, that curvature of the bolt occurred below the depth of the shallower spall profile on the leading edges of the bolts, the bolts bearing against the concrete at an angle. As a result, the cross-section of the bolt at the concrete surface effectively rotates into a more favorable orientation to resist load, a plausible explanation for failure predominantly occurring at the top of the exposed length below the leveling nut. The effects of cross-sectional rotation are further discussed in Section 6.3.3.4. Figure 5-7 displays load-displacement behavior for representative tests within the Direct Shear test program.

Table 5-2. Summary of Direct Shear anchor bolt ultimate load and displacement results

<table>
<thead>
<tr>
<th>Set</th>
<th>Reps</th>
<th>$n$</th>
<th>$l_{gp}$</th>
<th>$l_{lb}$</th>
<th>$T_u$ (kip)</th>
<th>$V_{u,bolt}$ (kip)</th>
<th>COV</th>
<th>$V_{u,bolt}$/$T_u$</th>
<th>$\delta_u$ (in.)</th>
<th>$\delta_u$/$d_b$</th>
<th>Failure Loc.</th>
<th>$l_b$ (in.)</th>
</tr>
</thead>
<tbody>
<tr>
<td>DS1</td>
<td>8</td>
<td>1</td>
<td>0</td>
<td>na</td>
<td>55.8</td>
<td>33.5</td>
<td>3%</td>
<td>0.60</td>
<td>0.31</td>
<td>0.31</td>
<td>BN</td>
<td>56</td>
</tr>
<tr>
<td>DS2</td>
<td>6</td>
<td>1</td>
<td>1.2</td>
<td>0</td>
<td>21.1</td>
<td>11.3</td>
<td>1%</td>
<td>0.59</td>
<td>0.18</td>
<td>0.29</td>
<td>BN</td>
<td>64</td>
</tr>
<tr>
<td>DS3</td>
<td>2</td>
<td>1</td>
<td>1.6</td>
<td>0.4</td>
<td>21.1</td>
<td>9.8</td>
<td>na</td>
<td>0.27</td>
<td>0.64</td>
<td>0.15</td>
<td>BN</td>
<td>32</td>
</tr>
<tr>
<td>DS4</td>
<td>10</td>
<td>1</td>
<td>2</td>
<td>0.8</td>
<td>21.1</td>
<td>7.9</td>
<td>6%</td>
<td>0.46</td>
<td>0.77</td>
<td>1.23</td>
<td>BN</td>
<td>66</td>
</tr>
<tr>
<td>DS5</td>
<td>4</td>
<td>1</td>
<td>3</td>
<td>1.8</td>
<td>21.1</td>
<td>5.6</td>
<td>6%</td>
<td>0.37</td>
<td>0.72</td>
<td>1.15</td>
<td>BN</td>
<td>73</td>
</tr>
<tr>
<td>DS6</td>
<td>5</td>
<td>1</td>
<td>4</td>
<td>2.8</td>
<td>21.1</td>
<td>4.4</td>
<td>10%</td>
<td>0.21</td>
<td>0.74</td>
<td>1.18</td>
<td>BN</td>
<td>47</td>
</tr>
<tr>
<td>DS7</td>
<td>4</td>
<td>2</td>
<td>2</td>
<td>0.8</td>
<td>21.1</td>
<td>11.4</td>
<td>5%</td>
<td>0.54</td>
<td>0.17</td>
<td>0.27</td>
<td>BN</td>
<td>54</td>
</tr>
<tr>
<td>DS8</td>
<td>5</td>
<td>2</td>
<td>4</td>
<td>2.8</td>
<td>21.1</td>
<td>6.9</td>
<td>3%</td>
<td>0.33</td>
<td>0.64</td>
<td>1.02</td>
<td>BN</td>
<td>72</td>
</tr>
<tr>
<td>DS9</td>
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<td>1</td>
<td>0</td>
<td>na</td>
<td>55.8</td>
<td>33.5</td>
<td>1%</td>
<td>0.60</td>
<td>0.31</td>
<td>0.31</td>
<td>BN</td>
<td>-</td>
</tr>
<tr>
<td>DS10</td>
<td>5</td>
<td>1</td>
<td>2.0</td>
<td>0.75</td>
<td>55.8</td>
<td>27.2</td>
<td>19%</td>
<td>0.49</td>
<td>1.64</td>
<td>1.64</td>
<td>AN</td>
<td>-</td>
</tr>
<tr>
<td>DS11</td>
<td>2</td>
<td>1</td>
<td>4.0</td>
<td>2.75</td>
<td>55.8</td>
<td>19.4</td>
<td>na</td>
<td>0.35</td>
<td>1.85</td>
<td>1.85</td>
<td>AN</td>
<td>-</td>
</tr>
<tr>
<td>DS12</td>
<td>4</td>
<td>1</td>
<td>0.0</td>
<td>na</td>
<td>21.1</td>
<td>12.2</td>
<td>3%</td>
<td>0.58</td>
<td>0.18</td>
<td>0.29</td>
<td>BN</td>
<td>-</td>
</tr>
<tr>
<td>DS13</td>
<td>5</td>
<td>1</td>
<td>2.0</td>
<td>0.8</td>
<td>21.1</td>
<td>9.2</td>
<td>5%</td>
<td>0.44</td>
<td>0.75</td>
<td>1.20</td>
<td>BN</td>
<td>-</td>
</tr>
<tr>
<td>DS14</td>
<td>2</td>
<td>1</td>
<td>4.0</td>
<td>2.8</td>
<td>21.1</td>
<td>5.1</td>
<td>5%</td>
<td>0.24</td>
<td>0.83</td>
<td>1.33</td>
<td>BN</td>
<td>-</td>
</tr>
<tr>
<td>DS15</td>
<td>2</td>
<td>1</td>
<td>2.0</td>
<td>0.8</td>
<td>21.1</td>
<td>7.7</td>
<td>na</td>
<td>0.36</td>
<td>0.66</td>
<td>1.07</td>
<td>BN</td>
<td>-</td>
</tr>
</tbody>
</table>

$na =$ not applicable
Figure 5-6. Examples of specimens used to measure $l_b$. A) DS8-2. B) DS6-5. (Photos courtesy of author.)

Figure 5-7. Load-displacement behavior of representative Direct Shear tests.
Flush-mounted tests performed nearly identically between 5/8 in. cast-in-place (CIP), 5/8 in. adhesive (AD), and 1 in. bolt tests. At $0.8d_b$ and $2.8d_b$ exposed length, however, differences emerged. Behavior and stiffness between CIP and AD single-bolt equivalents was approximately equivalent with higher $V_{u,bolt}$ values observed in the AD tests. It is believed that the higher strengths observed in adhesive tests were a result of greater surface area from the larger diameter annular adhesive ring bearing against the concrete surface. The effect of second-order p-delta tension stiffening of specimens is seen in all tests with exposed length, but is most obvious in the 1 in. dia. tests.

5.4 Torsion Testing of Scaled Anchor Bolt Groups

5.4.1 Summary of Results

Torsion test results are summarized in Table 5-3. One test, T8, was not run to ultimate load and two-tests, T3 and T6, are not reported due to calibration issues with instrumentation. (These were the first two tests conducted in Torsion testing.) Some discussion of the behavior of the tests, however, remains throughout this chapter, as useful information was still gleaned from their in-test behavior.

The 28-day concrete strength was 6,360 psi for the blocks containing specimens T1 through T13 and 5740 for blocks containing specimens T14 through T28. Grout strengths were 5,360 psi, 7,480 psi, and 6,230 psi for T5, T6-B, and T7, respectively. 5/8 in. bolt materials for T1 through T10 originated from the same batch with ultimate tensile strength of 20.7 kips, while T14 through 28 originated from the same batch with tensile strength of 20.0 kips. 1 in. bolts were from the same batch as in Direct Shear testing with a tensile strength of 55.8 kips. As illustrated in detail in Section 4.4.5, expressions of $V_{u,bolt}$ for Torsion testing reflect the average of the two load cell readings adjusted for loading geometry and distributed equally among the number of
bolts within a test. Values of \( \delta_u \) correspond to the circumferential displacement of the bolt corresponding to \( V_{u,bolt} \) for each test.

<table>
<thead>
<tr>
<th>Test</th>
<th>( t_{BP} ) ( d_b )</th>
<th>( t_{LM} ) ( d_b )</th>
<th>( T_o ) (kip)</th>
<th>( V_{u,bolt} ) (kip)</th>
<th>( \frac{V_{u,bolt}}{T_o} )</th>
<th>( \delta_u ) (in.)</th>
<th>( \frac{\delta_u}{d_b} )</th>
<th>Failure Loc.</th>
</tr>
</thead>
<tbody>
<tr>
<td>T1</td>
<td>0</td>
<td>na</td>
<td>20.7</td>
<td>11.5</td>
<td>0.56</td>
<td>0.32</td>
<td>0.52</td>
<td>CS</td>
</tr>
<tr>
<td>T2</td>
<td>0</td>
<td>na</td>
<td>20.7</td>
<td>10.6</td>
<td>0.51</td>
<td>0.28</td>
<td>0.44</td>
<td>CS</td>
</tr>
<tr>
<td>T3</td>
<td>2</td>
<td>0.8</td>
<td>20.7</td>
<td>6.8</td>
<td>0.33</td>
<td>0.49</td>
<td>0.79</td>
<td>BN</td>
</tr>
<tr>
<td>T4</td>
<td>2</td>
<td>0.8</td>
<td>20.7</td>
<td>7.4</td>
<td>0.36</td>
<td>0.60</td>
<td>0.96</td>
<td>BN</td>
</tr>
<tr>
<td>T5</td>
<td>2</td>
<td>0.8</td>
<td>20.7</td>
<td>11.0</td>
<td>0.53</td>
<td>0.56</td>
<td>0.90</td>
<td>AN</td>
</tr>
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<td>T6</td>
<td>4</td>
<td>0.8</td>
<td>20.7</td>
<td>10.6</td>
<td>0.51</td>
<td>0.44</td>
<td>0.79</td>
<td>BN</td>
</tr>
<tr>
<td>T7</td>
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<td>2.8</td>
<td>20.7</td>
<td>11.2</td>
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<td>0.42</td>
<td>0.67</td>
<td>BN</td>
</tr>
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<td>10.2</td>
<td>0.51</td>
<td>0.55</td>
<td>0.88</td>
<td>BN</td>
</tr>
<tr>
<td>T9</td>
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<td>2.8</td>
<td>20.7</td>
<td>13.3</td>
<td>0.64</td>
<td>1.03</td>
<td>1.65</td>
<td>BN</td>
</tr>
<tr>
<td>T10</td>
<td>4</td>
<td>2.8</td>
<td>20.7</td>
<td>14.8</td>
<td>0.71</td>
<td>0.44</td>
<td>0.44</td>
<td>BN</td>
</tr>
<tr>
<td>T11</td>
<td>0</td>
<td>Na</td>
<td>55.8</td>
<td>31.7</td>
<td>0.57</td>
<td>0.39</td>
<td>0.39</td>
<td>BN</td>
</tr>
<tr>
<td>T12</td>
<td>2</td>
<td>0.75</td>
<td>55.8</td>
<td>18.0</td>
<td>0.32</td>
<td>0.88</td>
<td>0.88</td>
<td>BN</td>
</tr>
<tr>
<td>T13</td>
<td>4</td>
<td>2.75</td>
<td>55.8</td>
<td>10.5</td>
<td>0.19</td>
<td>1.57</td>
<td>1.57</td>
<td>BN</td>
</tr>
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<td>T14</td>
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<td>0</td>
<td>20.7</td>
<td>11.2</td>
<td>0.56</td>
<td>0.42</td>
<td>0.67</td>
<td>BN</td>
</tr>
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<td>0.5</td>
<td>20.7</td>
<td>10.2</td>
<td>0.51</td>
<td>0.55</td>
<td>0.88</td>
<td>BN</td>
</tr>
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<td>20.7</td>
<td>3.2</td>
<td>0.16</td>
<td>0.57</td>
<td>0.92</td>
<td>BN</td>
</tr>
<tr>
<td>T17</td>
<td>7.2</td>
<td>6</td>
<td>20.7</td>
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<td>0.09</td>
<td>1.20</td>
<td>1.92</td>
<td>BN</td>
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<td>T18</td>
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<td>1</td>
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<td>0.79</td>
<td>BN</td>
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<td>0.89</td>
<td>BN</td>
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<td>0.15</td>
<td>0.72</td>
<td>1.15</td>
<td>BN</td>
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<td>20.7</td>
<td>1.7</td>
<td>0.09</td>
<td>1.20</td>
<td>1.92</td>
<td>BN</td>
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<td>38.3</td>
<td>10.8</td>
<td>0.28</td>
<td>0.45</td>
<td>0.73</td>
<td>BN</td>
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<td>T23</td>
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<td>3</td>
<td>38.3</td>
<td>4.9</td>
<td>0.13</td>
<td>0.73</td>
<td>1.17</td>
<td>BN</td>
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<td>16.9</td>
<td>0.77</td>
<td>1.64</td>
<td>2.62</td>
<td>AN</td>
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<td>20.7</td>
<td>9.5</td>
<td>0.48</td>
<td>0.70</td>
<td>1.12</td>
<td>BN</td>
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<td>17.3</td>
<td>0.86</td>
<td>1.11</td>
<td>1.78</td>
<td>BN</td>
</tr>
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<td>na</td>
<td>20.7</td>
<td>10.7</td>
<td>0.54</td>
<td>1.28</td>
<td>2.04</td>
<td>CS</td>
</tr>
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<td>3</td>
<td>20.7</td>
<td>9.1</td>
<td>0.45</td>
<td>0.82</td>
<td>1.31</td>
<td>BN</td>
</tr>
</tbody>
</table>

\( na = \) not applicable

ERR = error in data acquisition

DNF = did not fail

AN = Above Nut, BN = Below Nut, CS = Concrete Surface

### 5.4.2 Load-displacement Behavior of Ungrouted Tests

Flush-mounted tests aligned well with Direct Shear results and the well-established 0.6 ratio of experimental shear-to-tensile strength for anchor bolts installed in concrete. Initial linear-elastic behavior until concrete spalling at the location of anchor bolt bearing, after which stiffness degraded continuously until ultimate load. T2 stiffness degraded at a lower displacement and failed at lower ultimate load (\( 0.51T_o \)) than T1 (\( 0.56T_o \)), presumably from less...
favorable bolt position within oversized holes. Anchor bolt failure surfaces in flush-mounted tests were smooth over the entire cross-section, indicating shear slip over the entire cross-section and supporting the assertion that shear plasticity spreads across the entire anchor bolt cross-section at failure.

Tests with exposed length demonstrated linear-elastic behavior to approximately $0.15T_o$ and $0.08T_o$ for $0.8d_b$ and $2.8d_b$ exposed length tests, respectively, followed by a ductile plastic phase of behavior through ultimate load. Anchor bolts failed predominantly below the leveling nut. Failure planes were rough, clearly showing tensile rupture and a demarcation between locations of tension and compression fields from bending over the bolt cross-section.

5.4.2.1 Size effect

Figure 5-8 shows a comparison of the load-displacement behavior of equivalent 5/8 in. and 1 in. test specimens. Normalization of bolt shear by tensile strength allows visual comparison.

![Load-displacement behavior comparing 5/8 in. and 1 in. dia. ungrouted Torsion tests.](image)

Figure 5-8. Load-displacement behavior comparing 5/8 in. and 1 in. dia. ungrouted Torsion tests.
Strength and behavior between the two bolt sizes were nearly identical. Ultimate displacements of the $0.8d_b$ and $2.8d_b$ exposed length tests were on the order of one bolt diameter for both with the exception of the 1 in. diameter $2.8d_b$ Torsion test (T13), which showed greater deformation with respect to $d_b$. The normalization scheme for comparing strength and displacement clearly illustrates that there is no size effect between 5/8 in. and 1 in. dia. bolts.

5.4.2.2 Effect of pretensioning

Figure 5-9 shows anchor bolts in pretensioned test T4 and the non-pretensioned test T5 tests after failure. Leveling nuts in both tests underwent rotation en route to ultimate strength, but it is clear that the magnitude of T5 rotation exceeded that in T4. The linear-elastic stiffness of T5 was lower, likely due to additional bolt bending within the thickness of the base plate. While the difference in stiffness leading up to failure and the magnitude of $V_{u,bolt}$ are slight, it is hypothesized that the additional leveling nut rotation in the non-pretensioned test generated small tensile forces, resulting in a stiffer and stronger bolt response due to the more favorable stress distribution and small amount of moment release from second-order p-delta effects.

Figure 5-9. Failed Torsion anchor bolts with ungrouted $2d_b$ base plates. A) Pretensioned T2-B. B) Non-pretensioned T4. (Photos courtesy of author.)
5.4.2.3 Effect of concrete strength

Figure 5-10 shows comparable tests with specimens installed in the “normal-strength” concrete (approximately 6000 psi 28-day strength for the various batches tested) used throughout testing and a lower-strength concrete with 2710 psi 28-day strength. Tests in weak concrete were conducted at three exposed lengths, with two at 1 dia., one at 3 dia., and one at 6 dia.

![Load-displacement behavior of comparable ungrouted Torsion tests with normal-strength and low-strength concrete.](image)

The only direct comparison was for the six-diameter test, which aligned nearly exactly in load-displacement behavior throughout the loading process and in ultimate load. For the 1 and 3 diameter exposed lengths, the nearest normal-strength concrete equivalent tests contained exposed lengths of 0.8 and 2.8 bolt diameters, respectively. At the one diameter level, one low-strength test was loaded unevenly, resulting in a misshaped load-displacement curve with an ultimate load lower than the normal-strength 0.8 diameter near-equivalent (the lower ultimate strength, obviously, would be expected). The other low-strength 1-diameter test, however, was stronger than the 0.8 diameter exposed length equivalent. At the 3-diameter exposed length
level, tests between low-strength and normal-strength concrete nearly perfectly aligned. It is logical to assume that the “worst-case” orientation of applied load would be the pure shear case due to the direct bearing of the shear force on the concrete surface. Even with this limited dataset, it appears that for cases where the concrete is strong enough to prevent concrete failure modes, for a practical range of concrete strengths within which anchor bolts are embedded, the effect of concrete strength does not have a significant effect on the ultimate steel strength of anchor bolts in stand-off base plates for all ratios of applied normal-to-shear force.

5.4.2.4 Effect of bolt material

Figure 5-11 shows the load-displacement behavior for two tests with low-ductility ASTM 354BD material, one test with high-ductility Type 316 stainless steel material, and comparable tests with F1554 material. The material strengths and elongations for tensile specimens are provided in Section 5.2. The two tests with 354 BD material showed lower strengths throughout their load-displacement behavior than the F1554 near-equivalents, which both had 0.2$d_b$ lower exposed lengths and would be expected to be slightly stronger. Another contributing factor to the higher values for near-equivalent F1554 tests is that the third-turn-of-the-nut pretensioning force associated with the higher-strength, lower-ductility 354BD material was much higher than the F1554 material. No leveling nut rotation was observed in the 354BD tests, which can have significant positive effects on the ultimate strength of tests as discussed later in Chapter 6, whereas leveling nut rotation was often observed in F1554 tests, which could contribute to the marginally higher values observed in F1554 tests. Thus, the lower strengths relative to ultimate strength in 354BD material vs. F1554 material are not considered significant.

Only one stainless steel test was run at one bolt diameter of exposed length. It can be seen that up to approximately 0.3 anchor bolt diameters of displacement, the behavior of the stainless steel test roughly matched the F1554 equivalent. However, shortly afterward, the ductility of the
stainless steel allowed significant nut rotation, leading to an increased phase of stiffness between approximately 0.5 and 1.8 bolt diameters of displacement. A final phase then commenced at approximately 1.8 bolt diameters where the rotated leveling nut made contact with the concrete surface, producing additional friction force and a reaction acting against bending moments similar, in some ways, to the behavior of grouted and shimmed tests. The stainless steel test ultimately failed above the nut with a shear force equal to approximately 75% of the tensile capacity, well above the pure-shear failure capacity of $0.6 \, T_0$.

Figure 5-11. Load-displacement behavior of comparable ungrouted Torsion tests with F1554, 354BD, and Type 316 stainless steel materials.

Figure 5-12 shows the failed T24 stainless steel test specimen, the only ungrouted Torsion test both to fail above the leveling nut and to make contact with the concrete surface (other than zero diameter exposed length tests). A 3 dia. exposed length stainless steel test was not run based on the result from the 1 dia. test, as it was determined that the Torsion test setup would not be able to allow for the displacements that would be present at the higher exposed length.
5.4.2.5 Comparisons between Torsion and Direct Shear testing

Figure 5-13 compares the behavior of direct shear and torsion tests. Flush-mounted torsion tests T1 and T2 exhibited nearly identical load-displacement behavior to flush-mounted Direct Shear tests (dataset DS1) aside from initial slip in their 30% oversize holes. However, it is clear that in the tests with exposed lengths, the Direct Shear tests exhibited significantly higher end stiffness due to the second-order p-delta effects discussed in Section 5.3.1.

Figure 5-13. Comparisons between ungrouted Direct Shear and Torsion tests.
5.4.3 Load-displacement Behavior of Grouted Tests

Grouted stand-off base plate tests exhibited markedly different behavior than their flush-mounted equivalents. Initial load-displacement behavior in all cases was extremely stiff until interlaminar bond between the steel base plate and grout surface was broken, engaging the anchor bolts in shear at approximately $0.1T_o$. A phase of stiffness roughly equivalent to the linear-elastic portion of flush-mounted tests commenced. At this point, behavior diverged within both Torsion and Large-scale tests. T5 and LS2, which contained the lower levels of stand-off within their respective experimental arrangements, transitioned smoothly into an approximately linear inelastic range, the stiffness degrading only moderately until failure. Tests T6-A, T6-B, LS3, and LS4, those containing higher levels of stand-off within their respective experimental arrangements, were split equally into two distinct patterns of behavior. Shortly after grout cracking, T6-B and LS4 entered a phase of stiffness approximately parallel to the ungrouted $4d_b$ stand-off test, T3. Following significant deformation, these tests transferred into a final phase of elevated stiffness leading to ultimate load. T6-A and LS3, however, contained another phase of increased stiffness and associated strength gain immediately following grout cracking, reaching local maxima and subsequently losing load until displacements roughly equivalent to transition into a the final phase of elevated stiffness observed in tests T6-B and LS4. It is believed that as the bolts displaced, the grout pad restrained downward displacement of the base plate, enabling the anchors to develop axial strains as well as flexural strains and shear strains as they became more inclined. The tension in the inclined anchors then contributed to shear capacity in two ways. The vertical “clamping” component of the anchor tension provided a friction force at the interface while the horizontal component provided direct resistance to the shear. These resulting effects of the grout pad at large base plate rotations generated significant additional connection stiffness and strength until anchor bolt rupture, which occurred below the leveling nut in all cases.
except for test T3, in which failure occurred at the interface between the base plate and the grout pad. Post-cracking stiffness observed in T6-A and LS3 may have been caused by interlock between cracked grout pieces that, when overcome, caused the tests to retrograde to the baseline behavior observed in T6-B and LS4 equivalents. The ultimate loads of all grouted tests ranged from approximately $0.55T_u$ to $0.7T_u$, all above the ACI (2011) ultimate strength $0.48T_u$ resulting to an 0.8 factor applied to the $0.6T_u$ shear strength. These results suggest that the ACI (2011) factor is conservative and appropriate as a design basis. Figure 5-14 shows in-test grout cracking and post-test grout top surfaces following base plate removal for $0.8d_b$ and $2.8d_b$ exposed length grouted tests. Load-displacement behavior of grouted stand-off base plate tests with six 5/8 in. bolts and tests with three 5/8 in. bolts are provided in Figure 5-15 and Figure 5-16, respectively.

Figure 5-14. Failed grouted Torsion specimens. A) In-test grout cracking and B) post-test grout surface for T7. C) In-test grout cracking and D) post-test grout surface for T8. (Photos courtesy of author.)
Figure 5-15. Load-displacement behavior of grouted six-bolt Torsion tests.

Figure 5-16. Load-displacement behavior of grouted (and shimmed) three-bolt Torsion tests.
5.4.3.1 Effect of leveling nuts in a grouted installation

T25 and T26 were conducted with the same distance from the concrete surface to the bottom of the base plate, T25 supported by a leveling nut and T26 without a leveling nut. Referring to Figure 5-16, it can immediately be seen that the test without the leveling nut (with an x inside of the circular markers) failed at approximately twice the strength of the leveling nut test. It can also be seen that three-bolt grouted test with a leveling nut failed well below six-bolt tests with leveling nuts in Figure 5-15 and at a lower displacement. In T25, a relatively large gap between the top of the grout surface and the bottom of the grout pad was initially present; however, clamping was still present in the test during failure as evidenced by a brief increase in stiffness from approximately 0.6 dia. to 1.1 dia. of displacement. It is believed that the low strength of this test was anomalous, as the displacement at failure is much lower than the other tests with comparable exposed length from both ungrouted and grouted tests. As discussed in the Literature Review and the analytical model developed in this dissertation in Chapter 6, the ultimate displacement has a very large bearing on the ultimate strength of a grouted connection. Thus, the state of stress within the bolt reaches ultimate levels early in the displacement process. As the displacement increases, the stress state changes, still at ultimate levels, but at higher resistance to applied load due to friction and more favorable stress distribution on the bolt critical section. T25 took nearly an identical path to failure as the 2.8 dia. exposed length six-bolt tests in Figure 5-15 before abruptly failing at the low displacement.

Comparing T26 to the grouted six-bolt tests from Figure 5-15 with leveling nuts, T8 and T9, at near-equivalent stand-off distance to the bottom of the base plate (4.2 dia. in T26 compared to 4.0 in T8 and T9), it can be seen that T26 failed at a strength of nearly 30% percent greater per bolt than six-bolt equivalents. It is believed that the cause of the greater strength relative to these tests is caused by the allowance of the cross-section to rotate in the leveling nut
tests within the thickness of the oversized base plate hole. As will be shown in Chapter 6, rotation of the cross-section can have profound strengthening effects on the ultimate state of stress within the bolt due to a shift from shear loading over the cross-section to more favorable tensile loading. It is also possible that anchor bolts, by chance, were able to bear against cracked and clamped concrete, producing further resistance against the applied torsion load. At any rate, this test suggests that there is no loss in strength from placing a base plate directly on a grout pad. Figure 5-17 shows top views of the failed grouted three-bolt test with and without leveling nuts.

![Figure 5-17. Three-bolt grouted Torsion tests after anchor bolt failure. A) T25 (with leveling nut). B) T26 (without leveling nut). (Photos courtesy of author.)](image)

5.4.3.2 Effect of dry-packed low-strength mortar around the bolt line in place of a nonshrink grout pad

Test T28 with a dry-packed mortar layer around the bolt line presented an interesting experimental case with a significantly weaker material and a much smaller volume of total material between the grout pad and the concrete surface. It is conceivable that the resulting loss in overall stiffness could translate to different bolt behavior within the test. In the load-displacement behavior of this test, shown in Figure 5-16 with a circular white inner marker to indicate the absence of grout within the bolt circle area, four general phases of load-displacement
behavior can be identified. The first, similar to the tests with nonshrink grout, was an initially very stiff phase until mortar cracking, which occurred, as would be expected, at a much lower strength than tests with stronger and a greater volume of grout material. Immediately after mortar cracking, the bolt strength carried the load until approximately 0.4 dia. displacement, when the test entered the clamping stage of behavior, stiffened by the frictional resistance from the clamping force. The stiffness of this phase of behavior, significantly, is roughly the same as the stiffnesses of the six-bolt grouted tests with 2.8 dia. exposed length and the three-bolt test with no leveling nut, meaning that in this stage of loading, the worst-case, very weak material placed only in a thin annular strip around the perimeter of the bolt line (with greater than a 2.5 depth-to-width ratio) performed the same as a high-strength, nonshrink grout installed throughout the stand-off void. The strength of the mortar test was equitable to that of the grouted tests at, for instance, the 1 dia. displacement mark, possibly due to lower or the absence of bolt bearing contributions against the intermediate material. At approximately 1.3 bolt diameters of displacement, the mortar failed in compression, but the bolts did not, beginning the fourth phase of load-displacement behavior where all load was again taken entirely by the bolts. It can be seen that the second phase and fourth phase can be connected between 0.4 and 1.4 bolt diameters of displacement. The test continued until approximately 2.2 bolt diameters of displacement, when the bolts finally ruptured. The parallel load-displacement curve from 0.6 to 1.3 dia. of displacement to the tests with stiffer grout materials clearly demonstrates that if the compressive strength of a low-strength mortar material is enough to resist the clamping force from bolt displacement, it can be expected to perform equivalently to a high-strength nonshrink grout.

5.4.3.3 Effect of shim stacks in place of a grout pad

The shim stack test, T27, was of comparable exposed length and stand-off distance (which were the same in the absence of a leveling nut) to the grouted test without a leveling nut. In the
absence of grout, the stiffness of the test remained nearly constant through failure with a small dip near the 1 dia. displacement mark and was noticeably lower than the clamped stiffness of grouted tests and the mortar test. Correspondingly, the ultimate strength of this test was slightly lower than the near-equivalent six-bolt 2.8 diameter exposed length tests. The ultimate displacement was comparable with other tests with similar stand-off distance. Figure 5-18 shows T27 after failure. It can be seen that the top of the bolt rotated significantly with the moment-releasing spherical washers. Failure occurred at the bottom of the exposed length of the bolt (top of the concrete surface), in contrast to most other tests in the experimental program, which suggests that the moment release from the spherical washers may have caused moment concentrations (toward the flagpole scenario as described in Liu (2014) in the Literature Review for flexible base plates), which could also account for the lower strengths reported for this test.

5.4.3.4 Effect of an FRP wrap around a grout pad

The load displacement behavior of the grouted 2.8\(d_b\) exposed length Torsion test with an FRP retrofit, T10, can be found in Figure 5-15. Initial behavior of T10 was similar to other
grouted tests. With load less than approximately 0.3\(T_u\), there was rotation of the base plate relative to the FRP wrap. After initial grout cracking at approximately 0.3\(T_u\), radial grout material displacement was restrained by the FRP perimeter. Subsequent test rotation was shared equally between the base plate assembly and the retrofitted grout pad until an explosive failure, with all of the anchor bolts completely sheared away at the top surface of the grout pad near 0.7\(T_u\). The higher result than flush-mounted shear strength was attributed to the combination of the restraint of the grout pad from crack propagation, the leveling nuts immediately below the grout pad surface producing greater bearing area (no local spalls in front of anchor bolts were observed), and friction between the grout pad and the base plate. While only one FRP-retrofitted test was performed, this result shows promise for bringing anchor bolts in existing ungrouted and grouted stand-off base plates to flush-mounted strength or higher at low levels of ultimate displacement. Figure 5-19 shows the grout top surface of T10. Evidence of microcracking patterns similar to the unretrofitted tests can be seen.

Figure 5-19. Plan view of T10 (FRP-retrofitted 2.8\(d_b\) exposed length) after bolt failure. (Photo courtesy of author.)
5.5 Large-scale Anchor Bolt Groups under Predominantly Torsion Loading

Large-scale results are summarized in Table 5-4. 28-day concrete strengths for the four blocks were 6,590 psi, 7,160 psi, 7,140 psi, and 8,460 psi and grout strengths for LS2 through LS4 were 9,100 psi, 9,010 psi, and 8,170 psi, respectively. All of the 1.25 in. bolts were from the same batch. Expressions of \( V_{bolt} \), as in Torsion tests, reflect geometrically adjusted values, in this case adjusted for the angle of loading with respect to the tangent of the pivoting loading arm. Contributions to bolt stresses by overturning moment from the short pipe section were negligible and are not reflected in the results. Again, ultimate displacements, \( \delta_u \), correspond to the value of \( V_{u,bolt} \) for each test. Displacement values were calculated by adjusting the average of the two string potentiometer readings from their base plate radius location to the bolt circle radius.

Table 5-4. Summary of Large-scale anchor bolt ultimate load and displacement results.

<table>
<thead>
<tr>
<th>Test</th>
<th>( l_{BP} )</th>
<th>( l_{LN} )</th>
<th>( T_u ) (kip)</th>
<th>( V_{u,bolt} ) (kip)</th>
<th>( \delta_u ) (kip)</th>
<th>( \delta_u ) (in.)</th>
<th>( T_0 ) (kip)</th>
<th>( T_0 ) (kip)</th>
<th>Failure Loc.</th>
</tr>
</thead>
<tbody>
<tr>
<td>LS1</td>
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<td>1</td>
<td>84.4</td>
<td>27.1</td>
<td>0.32</td>
<td>1.89</td>
<td>1.52</td>
<td>BN</td>
<td></td>
</tr>
<tr>
<td>LS2</td>
<td>2.3</td>
<td>1</td>
<td>84.4</td>
<td>59.1</td>
<td>0.7</td>
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<td>1.7</td>
<td>BN</td>
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<td>3</td>
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<td>46.9</td>
<td>0.56</td>
<td>3.57</td>
<td>2.86</td>
<td>AN</td>
<td></td>
</tr>
<tr>
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<td>4.3</td>
<td>3</td>
<td>84.4</td>
<td>52.8</td>
<td>0.63</td>
<td>2.53</td>
<td>2.02</td>
<td>BN</td>
<td></td>
</tr>
</tbody>
</table>

na = not applicable
ERR = error in data acquisition
DNF = did not fail
AN = Above Nut, BN = Below Nut, CS = Concrete Surface

Load-displacement behavior of all Large-scale tests is provided in Figure 5-20. The ungrouted test, LS1, demonstrated similar behavior to ungrouted Torsion results. The magnitude of ultimate load for this \( 1d_b \) exposed length test, \( 0.32T_u \), fell between the values for the \( 0.8d_b \) and \( 2.8d_b \) exposed length Torsion tests. Test LS4, which contained the modified base plate with the circle cut out of the center, demonstrated behavior similar to Torsion and Large-scale grouted tests with a circular base plate and a strength greater than its circular base plate counterpart LS3. From this comparison of tests it was determined that results from the circular base plate tests...
with the cardboard filler in the annular area are valid representations of annular base plates.

Grouted test behavior closely resembled the description given for

Figure 5-21 shows the displaced anchor bolts in ungrouted test LS1. While bolts were fully pretensioned using the turn-of-the-nut method, significant leveling nut rotation on the order of that observed in the non-pretensioned Torsion test T4 occurred. As discussed later in Chapter 6, it is expected that leveling nut rotation may produce higher failure load values and higher end-of-test stiffness than a fully rigid (rotationally) leveling nut.

![Graph showing load-displacement behavior](image)

Figure 5-20. Load-displacement behavior of Large-scale tests.

![Image of LS1 after anchor bolt failure](image)

Figure 5-21. LS1 after anchor bolt failure. (Photo courtesy of author.)
Figure 5-22 displays in-test grout pad cracking and post-test top views of the grout pads for tests LS2, LS3, and LS4. With the connection oriented 90 degrees to its in-service condition (i.e., anchor bolts parallel to the floor), fragments of grout material outside of the bolt circle fell away during post-test removal of the loading assembly. Remaining grout outside of the bolt circle was manually removed, explaining its absence in the top views.

Figure 5-22. Failed Large-scale grouted specimens. A) In-test grout cracking and B) post-failure grout surface for LS2. C) In-test grout cracking and D) post-failure grout surface for LS3. E) In-test grout cracking and F) post-failure grout surface for LS4. (Photos courtesy of author.)
5.6 Eccentric Shear Loading of Rectangular Bolt Groups

5.6.1 Summary of Results

Eccentric shear test results are summarized in Table 5-5. Included in this table are the ultimate applied shear load, $V_{bolt}$, and normal load, $N_{bolt}$, on a per-bolt basis. The full treatment of Eccentric Shear data is described and illustrated in Section 4.6.5.

Table 5-5. Summary of eccentric shear ultimate load and displacement results.

<table>
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<tr>
<th>Test</th>
<th>$\frac{t_{BR}}{d_b}$</th>
<th>$\frac{t_{LN}}{d_b}$</th>
<th>n</th>
<th>Type</th>
<th>Ecc. (in.)</th>
<th>$T_o$ (kip)</th>
<th>$P_u$ (kip)</th>
<th>$V_{u,bolt}$ (kip)</th>
<th>$N_{u,bolt}$ (kip)</th>
<th>$\frac{\delta_u}{d_b}$ (in.)</th>
<th>$\delta_u$ (in.)</th>
<th>Failure Location</th>
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<td>5.0</td>
<td>20</td>
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<td>20</td>
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<td>28.4</td>
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<tr>
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<td>2</td>
<td>CS</td>
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<td>20.8</td>
<td>10.4</td>
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<td>1.45</td>
<td>BN</td>
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<td>CS</td>
<td>22.3</td>
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<td>20.8</td>
<td>10.4</td>
<td>-19.2</td>
<td>0.91</td>
<td>1.45</td>
<td>BN</td>
</tr>
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<td>0.86</td>
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<td>4</td>
<td>TCS</td>
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<td>20</td>
<td>24.7</td>
<td>6.2</td>
<td>21.6</td>
<td>0.26</td>
<td>0.42</td>
<td>AN</td>
</tr>
</tbody>
</table>

na = not applicable
ERR = error in data acquisition
DNF = did not fail
AN = Above Nut, BN = Below Nut, CS = Concrete Surface
Ultimate horizontal displacements, $\delta_u$, correspond to the maximum applied load. 28-day concrete strengths for the four blocks was 5740 psi. Ultimate loads are not reported for two tests. A data reading error occurred during ES17 and test ES29 did not fail due to experimental setup limitations.

5.6.2 Load-displacement Behavior of Ungrounded Two-bolt Eccentric Shear Tests

Figure 5-23 and Figure 5-24 below show typical tension/shear and compression/shear specimens in the failed condition, respectively. At all eccentricities greater than 6 in., ungrouted tension/shear tests predominantly failed above the leveling nut, while the remaining tension/shear and compression/shear tests predominantly failed below the leveling nut. The load-displacement behavior of ungrouted two-bolt tests is presented in the following subsections.

Figure 5-23. Representative ungrouted two-bolt tension/shear Eccentric Shear specimen after anchor bolt failure. (Photo courtesy of author.)

Figure 5-24. Representative ungrouted two-bolt compression/shear Eccentric Shear specimen after anchor bolt failure. (Photo courtesy of author.)
5.6.2.1 Effect of exposed length at different eccentricities

Figure 5-25, Figure 5-26, Figure 5-27, and Figure 5-28 show the load-displacement results for two-bolt tests with F1554 material for nominal 6 in., 12 in., 18 in., and 24 in. eccentricities from the applied load to the top of the concrete surface, respectively. A majority of the experimental focus was directed at the 6 in. and 24 in. eccentricities at the extreme ends of the test apparatus range. Torsion tests with tension/shear are indicated with red lines, tests with compression/shear with blue lines. The y axis of the tests show the shear load applied on a per-bolt basis, which does not represent in any way the tensile forces associated with the eccentricity, or “total load” on individual tests, which is illustrated at the end of this section. Thus, comparisons between the ultimate loads given between different loading eccentricities should not be made using Figure 5-25 through Figure 5-28.

Tension/shear tests and compression/shear tests responded differently to the varying levels of applied load eccentricity. At every eccentricity, compression/shear tests were affected significantly by the presence of exposed length, as the second-order displacement of the bolt exacerbated bending moments from shear forces, maximum load occurring early in the loading process. For the same reason, all ungrouted compression/shear tests failed at lower loads than their tension/shear equivalents.

Tension/shear test response to load varied throughout the loading process. At the 6 in. eccentricity, tests were clearly weakened by an increase in exposed length. As would be expected, displacements at ultimate load increased with increasing exposed length. At the 18 in. and 24 in. eccentricities, the ultimate load and the stiffness leading up to it are nearly identical between tests run with 1 dia. and 3 dia. exposed lengths, suggesting that at some ratio of applied tension-to-shear, there is a transition from exposed length influencing ultimate strength and not.
Figure 5-25. Load-displacement behavior of ungrouted 6 in. eccentricity two-bolt ES tests.

Figure 5-26. Load-displacement behavior of ungrouted 12 in. eccentricity two-bolt ES tests.
Figure 5-27. Load-displacement behavior of ungrouted 18 in. eccentricity two-bolt ES tests.

Figure 5-28. Load-displacement behavior of ungrouted 24 in. eccentricity two-bolt ES tests.
5.6.2.2 Effect of pretensioning

Figure 5-29 shows the effects of pretensioning on two sets of equivalent tests, one at the 6 in. eccentricity, the other at the 24 in. eccentricity, both with 1 dia. exposed length and loaded in the tension/shear configuration. While at the lower eccentricity the strength was slightly lower in the non-pretensioned test and the displacement of the non-pretensioned test in the higher-eccentricity test was higher, combined with the results from Torsion testing, it was determined that not pretensioning, in general, will either have no overall negative influence or greater static loading capacity due to the increased potential for cross-sectional rotation favorable to the ultimate strength. As a result, in general, subsequently performed tests were conducted with pretensioning to match standard industry practice.

Figure 5-29. Load-displacement comparisons between equivalent ungrouted pretensioned and non-pretensioned two-bolt tension/shear ES tests.
5.6.2.3 Effect of bolt ductility

Figure 5-30 shows two tests with F1554 and Type 316 stainless steel equivalents, one pair in the compression/shear setup, the other in the tension/shear setup. All tests were pretensioned and run at the 24 in. eccentricity.

Other than differences in ultimate displacement, the tension/shear test was nearly identical between materials. For a majority of the test, the compression/shear test was also identical; however, in the Type 316 stainless steel test, the ductility of the bolt caused the leveling nut to make contact with the concrete surface, producing bearing and frictional forces. This test was not run to completion due to the leveling nut bearing on the concrete surface, but it can be inferred from Figure 5-30 and from the similar behavior of the Torsion test T24 that the test would have progressed at the increased stiffness until the bolt finally ruptured. Based on these tests and the
Torsion tests of varying ductility, it is concluded that increases in bolt ductility may have beneficial impacts on the behavior of tests, but only at very high displacements.

5.6.2.4 Illustration of total load vs. displacement

Figure 5-31 provides an illustration of the “total load” traveling through a selection of 1 in. dia. exposed length two bolt tests at varying loading eccentricities in both the tension/shear and compression/shear loading arrangements. Total load was determined by taking the square root of the sum of the squares of the applied shear and normal forces per bolt normalized by the tensile strength of bolts. It should be noted that, for simplicity, the nominal applied load eccentricity was used for determining the normal forces acting on the system at a given time; the implications of this are demonstrated, for example, in the 24 in. eccentricity test, which showed that the “total load” exceeded the tensile capacity of the bolt. Nonetheless, Figure 5-31 clearly demonstrates that as the ratio of normal-to-shear force increased, the total load capacity of the system increased due to the more favorable cross-sectional resistance to normal forces than shear forces.

Figure 5-31. Approximate “total load” of selected ungrouted Eccentric Shear tests.
5.6.3 Load-displacement Behavior of Grouted Tests

Figure 5-32 and Figure 5-33 below show side views of typical grouted tension/shear and compression/shear specimens, respectively, after grout and anchorbolt failure. The load-displacement behavior of grouted two-bolt tests is presented in the following subsections. In this section, shear load “per bolt” is to be understood simply as the total shear load divided by the number of bolts, recognizing that grout friction may contribute some percentage (up to 100%) of the applied shear load. Shear and tension load experienced in the critical section of anchor bolts in grouted connections is somewhat complex, as demonstrated in Gresnigt et al. (2008) and the analytical model developed in Chapter 6.
5.6.3.1 Effect of exposed length

Figure 5-34 and Figure 5-35 show the load-displacement behavior of 6 in. eccentricity and 24 in. eccentricity tests, respectively, of varying exposed lengths. At the 6 in. eccentricity, tests demonstrated similarity to Torsion equivalents. Unsurprisingly, the 0 diameter exposed length test contained nearly the same strength as its ungrouted equivalent from Figure 5-25. The high displacements of the 3 dia. exposed length test allowed a greater clamping force, producing the highest resistance to applied load. The lone compression/shear test showed similar behavior to tension/shear tests with evidence of post-grout cracking clamping from the final increase in specimen stiffness before ultimate load.

Figure 5-34. Load-displacement behavior of grouted 6 in. eccentricity two-bolt ES tests. Tension/shear 24 in. eccentricity tests, as with their ungrouted counterparts, performed nearly identically in ultimate load and stiffness leading to ultimate load. The behavior and ultimate loads of these tests also closely resembled the ungrouted equivalents with little evidence
of clamping behavior, suggesting that grout did not play a role in these tests with higher tension-to-shear ratios. From this high-tension behavior and the fact that grout certainly did play a role in pure shear (Torsion) tests and 6 in. eccentricity grouted tests, it is logical to believe that there would be a critical value of ratio of applied tensile-to-shear load below which grout contributes to strength and above which it does not. Compression/shear tests at the 24 in. eccentricity are discussed in the next subsection.

![Figure 5-35. Load-displacement behavior of grouted two-bolt 24 in. eccentricity ES tests.](image)

**5.6.3.2 Effect of grout friction**

The effect of grout friction was studied in two tests with 24 in. eccentricity and 1 dia. exposed length, ES30 (white inner marker square in Figure 5-35) and ES29 (gray inner marker square), that were identical except for the presence of two double layers of Teflon below and above the grout pad in ES30. Referring to the load-displacement behavior of the tests shown in Figure 5-35, the additional contribution to load and stiffness from friction in ES29 is
immediately apparent. In the test with Teflon, the bolts were allowed to displace after grout cracking, which occurred early in the absence of significant frictional resistance. In this test, bolt steel rupture was observed at approximately 1.4 bolt diameters of horizontal displacement. In the non-Teflon ES29, the strength of the grout allowed significantly greater resistance to applied load to develop until an abrupt failure of the edge of the grout pad at approximately 4 dia. horizontal displacement. After this initial failure, the location of the grout compressive reaction shifted and the test was able to again resist significant force at a stiffness nearly parallel to the stiffness experienced at equivalent load in the beginning of the test. At an applied load just greater than the tensile capacity of the bolts (by coincidence) the load restraint system experienced an internal failure, causing the test to be stopped. Nonetheless, the effect of grout friction is clearly demonstrated in this load-displacement behavior. Figure 5-36 shows the failed test with two double layers of Teflon.

![Failed test with two double layers of Teflon](image)

Figure 5-36. ES (24 in. eccentricity compression/shear grouted test with Teflon layers) after anchor bolt failure. (Photo courtesy of author.)

5.6.4 Load-displacement Behavior of Four-bolt Tests

The load-displacement behavior of all four-bolt tests is provided in Figure 5-37. It should be noted that the y axis results are reported on a “per-bolt” basis, i.e. total applied shear load
divided by four. However, as discussed earlier, this does not inherently allow direct comparison between tests with different eccentricities, as the tensile component is not accounted for. Furthermore, because of the high compressive load in the 24 in. grouted test, the friction from the grout pad likely took all of the applied shear force, resulting in the critical bolts farthest from the applied load experiencing pure tensile force. Thus, the y axis should be viewed as a means of comparing the total applied load between ungrouted and grouted tests with the same eccentricity to see the “net” effect of the grout on the resistance to an equivalent load.

Figure 5-37. Load-displacement behavior of four-bolt ES tests.

Figure 5-38 and Figure 5-39 show the post-failure condition of ungrouted tests ES21 and ES22, respectively, where it can be seen that failure occurred in the compression/shear bolts in both cases (the failure of tension bolts in ES21 was secondary). Ungrounded four-bolt tests both showed significant zero-stiffness ductility for an extended range of displacement, in contrast to the positive plastic stiffness of tension/shear tests and the negative plastic stiffness of
compression/shear tests due to second-order p-delta effects on the moment. If, as described in Section 4.6.5, the shear is distributed equally between the four bolts and the neutral axis of bending is assumed between the two lines of bolts, the compression bolts in the 6 in. eccentricity four-bolt test would experience the same forces as in the 12 in. eccentricity two-bolt test, while the 24 in. eccentricity four-bolt test would be equivalent to a 48 in. eccentricity two-bolt compression/shear test. The 12 in. eccentricity test failed at 2.5 kips, approximately 10% less than the four-bolt test, suggesting that the assumptions presented in Section 4.6.5 are both accurate and conservative.

Figure 5-38. ES21 (6 in. eccentricity ungrouted four-bolt ES test) after anchor bolt failure. (Photo courtesy of author.)

Figure 5-39. ES22 (24 in. eccentricity ungrouted four-bolt ES test) after anchor bolt failure. (Photo courtesy of author.)

Figure 5-40 and Figure 5-41 show the post-failure condition of grouted tests ES31 and ES32, respectively. In both grouted tests the failure was in the tension bolts furthest from the
applied load. The 6 in. eccentricity grouted test, ES31, demonstrated the characteristic grout cracking followed by increased stiffness due to friction from clamping forces from grouted Torsion tests, low-eccentricity tension/shear tests, and compression/shear tests with bolt failure. The 24 in. eccentricity test, ES32, failed in a near-pure tension mode with no effects of grout clamping, which will be analyzed in further detail along with all other tests in Chapter 6.

Figure 5-40. Test ES31 (6 in. eccentricity four-bolt grouted ES test) after bolt failure. A) Side view of cracked grout. B) Top view of cracked grout. (Photos courtesy of author.)

Figure 5-41. Test ES32 (6 in. eccentricity four-bolt grouted ES test) after bolt failure. A) Side view of cracked grout. B) Top view of cracked grout. (Photos courtesy of author.)

5.7 Summary

This chapter provided experimental results and load-displacement behavior of the test program outlined in Chapter 4. General trends in the interplay between the orientation of applied load, base plate support condition, and anchor bolt exposed lengths were observed.
It was found that in the pure shear and compression/shear loading conditions for bolts in ungrouted connections, strength decreased dramatically with increases in exposed length of anchor bolts. Strength loss was also observed in ungrouted tests with low ratios of tension-to-shear applied load at increasing exposed length, but with diminishing effects as the ratio of applied tension to shear force increased. At high ratios of tension-to-shear force, results between bolts in ungrouted connections with low and high exposed lengths were essentially indistinguishable. It can be inferred from these results that a transition point in tension/shear applied load ratio exists, below which bending moments from shear acting over the exposed length influence strength and behavior and above which they do not.

The general behavior of grouted connections was very similar for bolts experiencing pure shear and low ratios of tension-to-shear and compression-to-shear force. In these loading cases, second-order clamping action between the base plate and grout pad produced an additional component of applied load resistance. At high ratios of compression-to-shear applied load, friction contributions to strength and stiffness dominated the ultimate load-displacement behavior of tests. At high ratios of tension-to-shear applied load, tests closely resembled ungrouted tests with equivalent load ratio and exposed length in load-displacement behavior. As with ungrouted bolt behavior, it is clear that there are transitional points in the load response of grouted bolts to applied load. Steel strength of bolts in grouted stand-off base plates is described by the interaction of the ratio of applied normal to shear force, interfacial friction coefficients, and the angle of a bolt at rupture resulting from steel ductility.

Rationally derived mathematical formulae to predict transition points are provided for both ungrouted and grouted bolt strength in the analytical models presented in Chapter 7. The data
presented in this chapter are measured against the analytical models and conclusions are developed from the results of this analysis.
6.1 Introduction

The primary purpose of this chapter is to introduce analytical models for predicting the steel strength of anchor bolts in ungrouted and grouted stand-off base plate connections. In support of the proposed analytical models, comparisons are made between average strengths of Direct Shear datasets to statistically demonstrate strength loss with increasing exposed length of anchor bolts subjected to shear forces, and the results from Torsion, Large-scale, and Eccentric Shear testing are measured against predictive values from the analytical models. In addition, discussions are provided of theoretical characteristic failure angles for anchor bolts in bending and of the influences of the minor variables investigated in this dissertation on anchor bolt steel strength. Finally, based on results from four-bolt Eccentric Shear tests, recommendations for the application of the analytical models to multiple-bolt connections are proposed.

6.2 Statistical Analysis of Direct Shear Tests

Figure 6-1 presents normalized Direct Shear results against stand-off distance/exposed length, where it is clear that there is a significant reduction in steel shear strength with increasing exposed length. Despite the understanding that the Direct Shear experimental arrangement produced some unintended tensile forces in the bolts, parametric consistency between multiple repetitions within individual Direct Shear datasets allow statistical comparisons to determine with confidence whether increases in exposed length reduce the steel strength of anchor bolts in ungrouted stand-off base plates subjected to shear forces. Analysis of variance (ANOVA) methodology coupled with a Tukey-Kramer post-hoc test was used to statistically categorize the different datasets.
A one-factor ANOVA test evaluates the means of three or more samples sets (generically known as “treatments”). The statistical null hypothesis is that all means are statistically equivalent, but the test does not distinguish which sample sets are statistically different if the null hypothesis is rejected (hence the need for the Tukey-Kramer comparison below). To test the hypothesis, an $F$ statistic is obtained as shown below in (6-1) through (6-5). The $F$ statistic is compared against a tabulated critical $F$ value for a chosen acceptable probability of Type 1 Error, $\alpha$. If the calculated $F$ value exceeds the tabulated $F$ value (resulting in a P-value less than $\alpha$), the null hypothesis is rejected and the conclusion is made that not all means are statistically equivalent.

$$F = \frac{MST}{MSE} \quad (6-1)$$
\[ MST = \frac{SST}{t - 1} \]  
(6-2)

\[ MSE = \frac{SSE}{N - 1} \]  
(6-3)

\[ SST = n \sum_{ij} (\bar{y}_i - \bar{y})^2 \]  
(6-4)

\[ SE = \sum_{i} (y_{ij} - \bar{y}_i)^2 \]  
(6-5)

where

- \( F \) = statistic for determining statistical significance
- \( MST \) = Mean Square Treatment
- \( MSE \) = Mean Square Error
- \( SST \) = Between Treatment Sum of Squares
- \( SSE \) = Sum of Squares for Error
- \( t \) = number of treatments
- \( n \) = number of samples within each treatment
- \( N \) = total number of samples in the experiment
- \( y_{ij} \) = \( j^{th} \) sample taken from treatment \( i \)
- \( \bar{y}_i \) = mean of the samples in treatment \( i \)
- \( \bar{y} \) = mean of all samples in the experiment

All 5/8 in. dia. Direct Shear datasets excluding sets with fewer than three repetitions were considered in an ANOVA test with \( \alpha = 0.05 \) (i.e. 95% statistical confidence). Results of the analysis are given in Table 6-1. Unsurprisingly, the test produced a statistically significant result that at least one dataset is different from at least one other.

<table>
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<th>SS</th>
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<th>MS</th>
<th>F</th>
<th>P-value</th>
<th>F crit</th>
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<td>9</td>
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<td>312.464</td>
<td>1.68E-37</td>
<td>2.10</td>
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<td>0.0003</td>
<td></td>
<td></td>
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</tr>
<tr>
<td>Total</td>
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<td>54</td>
<td></td>
<td></td>
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</tr>
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</table>

If, as in Table 6-1, an ANOVA test results in statistical differences, further analysis is necessary to determine which dataset means are statistically different and equivalent. The Tukey-Kramer Honestly Significant Difference (HSD) is one such method that accounts for multiple
sample sizes as in Direct Shear datasets. Values of $\alpha$, $df_E$, and $MSE$ the ANOVA test are implemented in the HSD method, which produces a critical $SD$ value calculated by (6-6).

$$HSD = q_{dfE, t, \alpha} \sqrt{\frac{MSE}{n}} \quad (6-6)$$

where $HSD$ = critical value for determining statistical equivalence between datasets $q_{dfE, t, \alpha}$ = taken from the Student’s t-distribution table $df_E$ = number of degrees of freedom for error in ANOVA test $t$ = is the and is the total number of means in ANOVA test $MSE$ = Mean Square Error from ANOVA test $n$ = number of samples in each population

$HSD$ values are calculated for each dataset (differences are fully dependent on sample size). If the absolute value of the difference between any two datasets is greater than the combined $HSD$ values of the two datasets, the null hypothesis that any two means are statistically equivalent is rejected and it is concluded that the two dataset means are different from each other.

Table 6-2 provides a summary which datasets are statistically different or equivalent from conducting the Tukey-Kramer analysis. Individual cells delineate which datasets were statistically different (Y) and which were statistically equivalent (N) from corresponding datasets in the matrix.

Table 6-2. Evaluation of statistical differences between selected Direct Shear datasets.

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<th>Dataset</th>
<th>$l_e$</th>
<th>$d_b$</th>
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<th>$\frac{V_{u,bolt}}{T_u}$</th>
<th>Different from...</th>
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<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
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<tr>
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<td>0.625</td>
<td>AD</td>
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<td>Y</td>
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</table>
Within groups of datasets containing the same installation type (i.e. single-anchor cast-in-place datasets DS1 through DS6, double-anchor cast-in-place datasets DS7 and DS8, and adhesive anchor datasets DS12 and DS13), the strength of the bolts decreased with statistical significance between every increment of increase in exposed length. In other words, this statistical methodology clearly indicates statistically significant loss in anchor bolt strength as exposed length increases.

Flush-mounted and $0.8d_b$ exposed length tests were conducted using one- and two-bolt cast-in-place installations and one-bolt adhesive installations. All flush-mounted datasets were statistically equivalent except for the double-bolt DS7, where its normalized shear strength was lower than both the normalized shear strength of the 5/8 in. single-bolt cast-in-place (DS1) and 1 in. bolt (DS9) sets. At $0.8d_b$ exposed, however, adhesive anchors (DS13) are statistically stronger than 5/8 in. single anchors (DS1), which are in turn statistically stronger than the two-bolt tests (DS8), although the latter two are nearly statistically equivalent. The statistical analysis produces the following conclusions with 95% certainty: 1) strength decreased for all increments of increasing exposed length, 2) adhesive anchors in stand-off base plates were stronger than cast-in-place anchors, presumably from the significant decrease of concrete spalling at the bolt bearing location and 3) bolts tested in the one-bolt arrangement were stronger than bolts tested in the two-bolt arrangement. From 1) it is clear that the reduction in strength of anchor bolts in ungrouted stand-off base plate connections must be accounted for. Standard treatment of an increase in exposed length due to concrete spalling (e.g. by Eligehausen et al. 2006) for determining the strength of bolts in stand-off base plate connections is strongly supported by 2).

6.3 Development of Predictive Models for Anchor Bolt Steel Strength

This section describes the development of analytical models for predicting anchor bolt steel strength in ungrouted and grouted stand-off base plate connections. First, a justification of
the plastic section modulus implemented in the analytical models is given, followed by a summary of the cross-sectional limit states on circular sections subjected to combinations of shear forces, normal forces, and bending moments. Background theory for the respective bivariate interactions is discussed in Section 2.2.3 of the Literature Review and the full derivations specific to a circular cross-section are presented in Appendix E. Models for anchor bolt steel strength in ungrouted and grouted stand-off base plates are developed that account for the interplay between the angle of applied load, bolt ductility limit, and, in the grouted model, interfacial friction between grout, concrete, and steel components.

### 6.3.1 Empirical Determination of Plastic Section Modulus

The plastic section modulus, $Z$, of a circular section is equal to $d^3/6$. For threaded rods, no established methodology was found in the literature on the appropriate effective diameter, which will fall between the minimum root diameter and the gross bolt diameter, for determining bending capacity. The implication of the cubed diameter term is that relatively small changed in the assumed effective diameter produce significant effects on the resulting bending moment capacity of a threaded rod. As an illustration, for a 5/8 in. diameter bolt the minimum root diameter, 0.527 in., the diameter associated with net tensile area, 0.537 in., and the gross bolt diameter, 0.625 in., correspond to plastic section moduli of 0.0244 cubic in., 0.0257 cubic in., and 0.0407 cubic in., respectively, resulting in an increase in pure bending moment capacity from the minimum root diameter to the gross bolt diameter of 67%. While it is not likely that the effective bending diameter falls at or near either extreme, it is clear that the assumed value may have a profound impact on predictions of ultimate strength for which bending is a factor.

To provide the most accurate comparison between analytical models and experimental results, $Z$ was determined empirically using the two three-bolt torsion tests conducted with 6 dia. exposed lengths, T17 and T21. These tests failed with an average bolt shear force of 1.71 kips,
approximately 14% of the pure shear capacity of the cross-section. In both lower-bound and upper limit formulations of the interaction between shear and moment, the shear term is squared. Following these interaction equations, then, only 2% of the contribution to the failure of these bolts is attributable to shear with the remaining 98% coming from bending moment. If it is conservatively assumed that the failure of these two tests was only caused by bending moments, the ultimate moment from the tests, 3.21 kip-in., can be used as the reference value for \( M_o \).

Using \( \sigma_o = 88.48 \text{ ksi} \) from tensile tests and knowing that \( M_o = \sigma_o Z \), the empirical value of \( Z \) becomes 0.0362 in\(^3\), which corresponds to a diameter of 0.601 in., 4.3% greater than the average of the minimum root diameter and the gross bolt diameter (0.576 in.), suggesting that the average dimensions of a threaded rod cross-section may be appropriate for determining bending moment capacity.

### 6.3.2 Interactions between Bending Moments, Shear Forces, and Normal Forces

Appendix E presents complete derivations of the three bivariate and one three-variable interaction relationships between shear forces, normal forces, and bending moments acting on a circular section based on the methods summarized in Section 2.2.3 of the Literature Review. The interaction between shear forces and bending moments is complex, relying on assumptions of fully plastic stress distribution throughout the cross-section; as such, both “upper-limit” and “lower-bound” equations between shear force and bending moment are presented. With the two moment/shear interaction equations, two three-variable interaction relationships are also formed.

Normal force, bending moment, and shear force capacities of a circular cross-section are provided in (6-7), (6-9), and (6-11). In the interaction of forces and moments on a cross-section, it is convenient to express applied loads in terms of these univariate capacities as shown in (6-8), (6-10), and (6-12).
\[ N_o = A_{nt} \sigma_o \]
\[ = \pi r^2 \sigma_o \]  \hspace{1cm} (6-7)

\[ n = \frac{N}{N_o} \]  \hspace{1cm} (6-8)

\[ M_o = Z \sigma_o \]
\[ = \frac{4r^3 \sigma_o}{3} \]  \hspace{1cm} (6-9)

\[ m = \frac{M}{M_o} \]  \hspace{1cm} (6-10)

\[ V_o = \alpha N_o \]  \hspace{1cm} (6-11)

\[ v = \frac{V}{V_o} \]  \hspace{1cm} (6-12)

where

- \( N_o \) = ultimate uniaxial normal capacity of circular section
- \( M_o \) = ultimate moment capacity of circular section
- \( V_o \) = ultimate shear capacity of circular section
- \( A_{nt} \) = net tensile area (for threaded rod applications)
- \( r \) = radius
- \( Z \) = plastic section modulus of circular section
- \( \sigma_o \) = ultimate normal stress capacity of bolt material
- \( N \) = normal force acting on section
- \( V \) = shear force acting on section
- \( M \) = bending moment acting on section
- \( n \) = normal force normalized by capacity
- \( v \) = shear force normalized by capacity
- \( m \) = bending moment normalized by capacity

### 6.3.2.1 Lower-bound interaction

In the lower-bound approach, the stress-distribution between shear forces and bending moments attributed to Palchevsky (1948) as shown in Figure 2-10 and expanded in Appendix E is implemented. In this assumed relationship, fully plastic bending stresses are distributed strictly to the extreme areas of the cross-section with respect to the plastic neutral axis with fully plastic
shear takes up the remaining interior core. The resulting three-variable interaction between normal forces, shear forces, and bending moments is defined by (6-13).

\[ n^2 + v^2 + m = 1 \]  \hspace{1cm} (6-13)

**6.3.2.2 Upper limit interaction**

In the upper limit approach, the state of stress at every point on the circular cross-section is optimized for the greatest overall resistance to applied shear forces and bending moments using the technique presented in Jirasek and Bazant (2001). A conservative simplification, as described in Appendix E, results in the three-variable interaction equation given in (6-14).

\[
\left( \frac{n}{\sqrt{1-m}} \right)^2 + \left( \frac{v}{\sqrt{1-m^2}} \right)^2 = 1
\]  \hspace{1cm} (6-14)

Predicated on the assumption that a steel cross-section is able to plastically distribute stresses to the highest possible energy state, the upper-limit approach is considered by the author a more accurate representation of interaction between shear forces and bending moment. As such, the upper-limit equation is used in the derivation of the analytical model for bolt strength in ungrouted stand-off base plate connections. For ungrouted Cases, however, the grouted analytical model only depends on the interaction between shear and normal forces, for which the upper-limit and lower-bound methods produce the same interaction equations.

The three-variable upper-limit hypersurface is presented in Figure 6-2 for all possible values and sign orientations of \( n \), \( m \), and \( v \). Thicker black lines represent the three bivariate interaction equations and thinner gray lines represent interpolated values of the bivariate equations influenced by respective relationships with the third variable. Figure 6-3 displays the same hypersurface in terms of absolute values of \( n \), \( m \), and \( v \).
Figure 6-2. Upper-limit failure surface between bending moment, normal force, and shear force.

Figure 6-3. Failure surface octant produced by absolute magnitudes of bending moment, normal force, and shear force.
Anchor bolts within multiple-bolt connections are subjected only to externally applied shear and normal force resultants. The three-variable hypersurface from Figure 6-2 and Figure 6-3 can be further simplified to two-variable normal force vs. shear force space as shown in Figure 6-4; the exterior black line describes the interaction between shear and normal forces in the absence of bending moments, while interior gray lines describe the interaction of shear force vs. normal force bounded by the hypersurface for given values of moment.

![Figure 6-4. Shear and normal failure surface with interpolated limiting values from bending moments.](image)

### 6.3.3 Prediction of Anchor Bolt Steel Strength in Ungrouted Stand-off Base Plates

A simplified schematic of a displaced anchor bolt in an ungrouted stand-off base plate connection subjected to applied shear and normal forces is illustrated in Figure 6-5. Dashed lines display the location of the free-body cut in Figure 6-6, which provides the basis for the equilibrium of forces in the ungrouted analytical model. It should be noted that the actual exposed length is generally defined as the distance 0.5 bolt diameters below the concrete surface to the bottom of the leveling nut.
Using Figure 6-6, first-order (i.e., without considering force effects from displaced geometry) and second-order (i.e. implementing displaced bolt exposed length geometry in the determination of bending moments) equilibria of forces on the critical cross-section at the top and bottom of the exposed length can be developed. In both models it is assumed that the bolt is fully restrained in bending at the top and bottom of the exposed length and free to displace vertically and horizontally.

Figure 6-5. Simplified schematic of displaced bolt in grouted-stand-off base plate connection.

Figure 6-6. Free-body diagrams of forces on bolt exposed length in ungrouted stand-off base plate connection.
6.3.3.1 First-order equilibrium of applied normal and shear forces

In the first-order equilibrium of anchor bolt forces from Figure 6-6, the axial load, $N_A$, is assumed to act concentrically on an undeformed anchor bolt (i.e. $V_B = 0$), translating directly to bolt normal force $N_B$. Applied shear force, $V_A$, produces both shear forces $V_B$ and bending moments $M_B$ (6-17) over the effective exposed length, which can be found by summing moments about the point of contraflexure indicated with the black circle on the half-model of the assumed shape.

$$N_B = N_A = \rho_{NV} V_A \tag{6-15}$$

$$V_B = V_A \tag{6-16}$$

$$M_{B,1st} = \frac{V_A l_{eff}}{2} \tag{6-17}$$

where $V_A$ = applied shear force

$N_A$ = applied normal force (tension positive)

$\rho_{NV}$ = ratio of applied normal to shear force (tension positive)

$V_B$ = shear force experienced by bolt

$N_B$ = normal force experienced by bolt

$M_{B,1st}$ = first-order bending moment experienced by bolt

$l_{eff}$ = effective length of bolt

Substituting the values of normal force, shear force, and bending moment in terms of applied shear loads into (6-14), the corresponding upper limit first-order model for predicting strength of anchor bolts in ungrouted stand-off base plates becomes (6-18) and, similarly, the lower-bound first order substitution of values results in (6-19).

$$\left( \frac{N_B/N_o}{1 - \frac{M_{B,1st}}{M_o}} \right)^2 + \left( \frac{V_B/(\alpha N_o)}{1 - \left( \frac{M_{B,1st}}{M_o} \right)^2} \right)^2 = 1 \tag{6-18}$$
\[\left(\frac{\rho_{NV}V_A/N_n}{1 - \frac{V_A}{2M_n}}\right)^2 + \left(\frac{V_A/(\alpha N_n)}{1 - \left(\frac{V_A}{2M_n}\right)^2}\right)^2 = 1\]

\[\left(\frac{\rho_{NV}V_A/N_n}{1 - \frac{V_A}{2\sigma_Z B}}\right)^2 + \left(\frac{V_A/(\alpha N_n)}{1 - \left(\frac{V_A}{2\sigma_Z B}\right)^2}\right)^2 = 1\]

\[\left(\frac{\rho_{NV}V_A}{N_n}\right)^2 + \left(\frac{V_A}{\alpha N_n}\right)^2 + \frac{V_A l_{eff}}{2\sigma_Z B} = 1\]  
\hspace{1cm} (6-19)

where
- \(V_A\) = applied shear force
- \(N_n\) = applied normal force (tension positive)
- \(\rho_{NV}\) = ratio of applied normal to shear force (tension positive)
- \(V_B\) = shear force experienced by bolt
- \(N_B\) = normal force experienced by bolt
- \(M_{B,1st}\) = first-order bending moment experienced by bolt
- \(l_{eff}\) = effective length of bolt

### 6.3.3.2 Second-order equilibrium of applied normal and shear forces

Second order equilibrium is identical with first-order equilibrium with the exception that normal forces acting on the horizontally displaced anchor bolt exposed length produce second-order bending moments, commonly referred to as “p-delta” effects in structural engineering vernacular. Tension p-delta moments counteract first-order bending moment from the shear force while compression p-delta moments exacerbate first-order moments. (6-20) demonstrates the second-order effects of the axial force, where tensile forces are positive and compressive forces are negative, on the bolt bending moment which is applied to the model for determining anchor
bolt failure in (6-21). The similar resulting lower-bound interaction equation with all test variables inserted is provided in (6-22).

\[ M_{B_{2nd}} = \frac{V_A l_{eff}}{2} - \frac{N_A \delta_h}{2} = \frac{V_A l_{eff} - \rho_{NV} V_A \delta_h}{2} \]  

\[
\left( \frac{\rho_{NV} V_A / N_o}{1 - \frac{V_A l_{eff} - \rho_{NV} V_A \delta_h}{2\sigma_o Z_B}} \right)^2 + \left( \frac{V_A / (\alpha N_o)}{1 - \left( \frac{V_A l_{eff} - \rho_{NV} V_A \delta_h}{2\sigma_o Z_B} \right)^2} \right)^2 = 1
\]  

(6-21)

\[
\left( \frac{\rho_{NV} V_A}{N_o} \right)^2 + \left( \frac{V_A}{\alpha N_o} \right)^2 + \frac{V_A l_{eff} - \rho_{NV} V_A \delta_h}{2\sigma_o Z_B} = 1
\]  

(6-22)

where \( M_{B_{2nd}} \) = bending moment experienced by bolt considering first- and second-order moments

**6.3.3.3 Proposed analytical model**

In an anchor bolt with ungrouted exposed length subjected to pure shear force, the limiting horizontal displacement will depend on the ductility of the bolt material that is experiencing high bending strains. As proposed by Lin et al. (2011), it is assumed that curvature is constant over \( l_e \), which is conceptually illustrated in Figure 6-7. It can be seen that the horizontal displacement acts not over the entire exposed length, but predominantly in the intermediate length between plastic hinges.

If it is assumed that that plasticization occurs over a constant length \( l_p \) irrespective of \( l_{eff} \), the limiting angle produced by the ultimate bending strains in the pure shear loading scenario will remain constant for all values of exposed length, a “characteristic” angle \( \phi_{char} \) defined solely by the ductility of the material. Figure 6-8 illustrates the plastic hinge length concept for two lengths of anchor bolts experiencing shear load over their exposed lengths, demonstrating...
the constant plastic hinge concept that results in $\phi_{char}$ applying to all values of exposed length. Also included is a simplified schematic of the two plastic hinge regions and the interior non-plasticized region. The red and blue triangles on either end of the exposed length represent the total plasticity that is assumed to be absorbed equally over $l_p$ based on the idealized constant curvature assumption presented in Figure 6-7, with the greatest strains coming at the extreme fibers of the neutral axis over distance $l_s$ in Figure 6-8. With the simplified exposed length, Figure 6-30 and Figure 6-31 can be developed for the mathematical determination of strain over $l_p$.

Figure 6-7. Assumed moment and curvature distributions producing $\phi_{char}$.

Figure 6-8. Illustration of $\phi_{char}$ for a range of exposed lengths.

\[ l_s = 0.5d_b \cos(\phi_{char}) \quad (6-23) \]
\[ \varepsilon_{ult} = l_s/l_p \quad (6-24) \]
where \( l_s \) = total strained length
\( l_p \) = plastic hinge length
\( d_p \) = bolt diameter
\( \phi_{char} \) = characteristic angle associated with material ductility
\( \varepsilon_{ult} \) = constant strain acting over plastic hinge

At lower ratios of beam length to depth, shear strains begin to significantly affect, and ultimately dominate, the displacement behavior or a beam. In addition, at effective lengths less than \( 2l_p \), the hinge region from the two sides of the exposed length will overlap. For the purposes of this model, however, \( \phi_{char} \) due to pure bending on the cross-section is still assumed to hold true for exposed lengths less than \( 2l_p \).

Separately, the ratio of applied normal-to-shear applied load on the bolt, \( \rho_{NV} \), corresponds to a resultant loading angle \( \phi_{load} \), defined for the purposes of this dissertation with respect to the primary axis of the undeformed anchor bolt exposed length and equal in magnitude to

\[
\text{atan} \left( \frac{1}{\rho_{NV}} \right) \cdot \phi_{load}
\]

for a tension/shear test is illustrated in the cut-out circle in Figure 6-5. Pure tension and compression are defined as having a loading angle of zero degrees and pure shear a loading angle of 90 degrees, while tension/shear and compression/shear combinations produce positive and negative loading angles, between 0 and ±90 degrees.

In a pure tension test, it is clear that no bending moments exist. Increasing \( \phi_{load} \) slightly, it is reasonable to assume that the bolt will orient itself to \( \phi_{load} \) until failure. The bolt orientation is permitted by material strain capacity for tensile strains from resultant normal force on the cross-section to overcome bending strains. In other words, \( \rho_{NV} \) will equal \( l_{eff} / \delta_h \), causing the second-order bending moment from \( T_A \) acting over \( \delta_h \) to cancel the moment from \( V_A \) acting over \( l_{eff} \).

Thus, the forces acting on the bolt will be some combination of tension and shear. If \( \phi_{load} \) is set to a greater angle than \( \phi_{char} \), however, critical tensile strains from bending will be reached before compressive strains and stresses are relieved and failure will occur at an angle less than \( \phi_{load} \). For the
development of both the ungrouted and grouted analytical models of anchor bolt steel strength, it is assumed that the critical angle of applied load above which bending must be considered is equal to $\phi_{char}$. At this point $\phi_{char}$ is left undefined, to be revisited later after applying the model to results. In tension/shear cases, the use of second-order equilibrium is necessary for model accuracy.

Anchor bolts experiencing combinations of compression and shear are expected to reach maximum capacity at low levels of displacement due to the moment-amplifying p-delta effects of the compression force. The second-order strength will necessarily be less than the first-order strength and, as such, the first-order formulation should provide the limit state for ultimate load capacity with bolts experiencing compressive normal forces with shear.

The logic developed in the preceding paragraphs produces three separate mathematical cases for characterizing the ultimate strength of an ungrouted anchor bolt with exposed length based on the ratio of applied normal force to shear force: Case 1) a first-order interaction of moment, shear, and compression forces for pure shear and compressive loading cases ($\phi_{load} \leq 0$), Case 2) a second-order interaction of moment, shear, and tension forces for loading angles above the characteristic angle ($0 < \phi_{char} \leq \phi_{load}$) and Case 3) a first-order interaction of shear and tension forces on the anchor bolt cross-section for loading angles below the characteristic angle ($\phi_{load} < \phi_{char}$). In the absence of methodology for determining $\phi_{char}$, experimental results may be compared with the analytical model by replacing $\phi_{char}$ with $atan\left(\frac{\delta_p}{t_{eff}}\right)$ for a reference pure shear test. Cases 1, 2, and 3 are defined formally below.

**Case 1: Anchor bolt experiencing pure shear or combined compression and shear forces.** Figure 6-9 shows the pure shear loading condition and Figure 6-10 shows a generic combined compression and shear load acting on the anchor bolt cross-section. As the bolt displaces horizontally from the shear component of applied load, second-order moments act in
the direction of first-order moments, decreasing capacity. Hence, the capacity of the bolt is achieved early in the load-displacement process and first-order moments are appropriate for determining ultimate load. (6-28) and (6-29) show the resulting predictive equations for the upper limit and lower-bound equations produced by (6-25) through (6-27).

Figure 6-9. Displaced bolt under pure shear Case 1 loading.

Figure 6-10. Undisplaced bolt (for first-order analysis) under Case 1 compression/shear loading.

if \( \phi_{load} \leq -90 \ deg \) (compression or pure shear)

\[
V_B = V_A \tag{6-25}
\]

\[
N_B = N_A = \rho_N V_A \tag{6-26}
\]
\[ M_B = M_{B,1st} = \frac{V_A l_{eff}}{2} \]  

\[ \left( \frac{\rho_{NV} V_A / N_o}{1 - \frac{V_A l_{eff}}{2\sigma_o Z_B}} \right)^2 + \left( \frac{V_A / (\alpha N_o)}{1 - \frac{V_A l_{eff}}{2\sigma_o Z_B}} \right)^2 = 1 \]  

\[ \left( \frac{\rho_{NV} V_A}{N_o} \right)^2 + \left( \frac{V_A}{\alpha N_o} \right)^2 + \frac{V_A l_{eff}}{2\sigma_o Z_B} = 1 \]

### Case 2: Anchor bolt experiencing combined tension and shear forces – shear dominant

Figure 6-11 shows a generically applied load in the Case 2 range of behavior, illustrating that the ultimate displacement is associated with the limiting displacement from shear alone Figure 6-9. (6-33) and (6-34) show the resulting predictive equations for the upper limit and lower-bound equations produced by (6-30) through (6-32).

\[ \phi_{load} \geq \phi_{char} \]

![Figure 6-11. Displaced bolt (for second-order analysis) under Case 2 tension/shear loading.](image)

\[ if \quad \phi_{load} \geq \phi_{char} > 0 \]

\[ V_B = V_A \]  

\[ N_B = N_A = \rho_{NV} V_A \]  

222
\[ M_B = M_{B,2nd} = \frac{V_A l_{eff} - \rho_{NV} V_A \delta_h}{2} \]  
\[ (\frac{\rho_{NV} V_A}{N_o})^2 + \left(1 - \frac{V_A l_{eff} - \rho_{NV} V_A \delta_h}{2 \sigma_o Z_B} \right)^2 = 1 \] 
\[ \left(\frac{\rho_{NV} V_A}{N_o}\right)^2 + \left(\frac{V_A}{\alpha N_o}\right)^2 + \frac{V_A l_{eff} - \rho_{NV} V_A \delta_h}{2 \sigma_o Z_B} = 1 \]  

**Case 3: Anchor bolt experiencing combined tension and shear forces – tension dominant.** Figure 6-12 shows a generically applied load in the Case 3 range of behavior, illustrating that the ultimate displacement is dictated by the angle of applied load; i.e. the displaced shape of the bolt aligns with the applied load as afforded by the ductility limits of the bolt. (6-42) shows the resulting predictive equations for both upper limit and lower-bound equations produced by (6-39) through (6-41). Figure 6-13 summarizes all ungrouted cases.
if $\phi_{load} < \phi_{char}$

$$V_B = V_A$$  \hspace{1cm} (6-35)

$$N_B = N_A = \rho_{NV}V_A$$  \hspace{1cm} (6-36)

$$M_B = 0$$  \hspace{1cm} (6-37)

$$\left(\frac{N_B}{N_0}\right)^2 + \left(\frac{V_B}{\alpha N_0}\right)^2 = 1$$

$$= \left(\frac{V_A}{N_0}\right)^2 + \left(\frac{\rho_{NV}V_A}{\alpha N_0}\right)^2 = 1$$  \hspace{1cm} (6-38)

Figure 6-13. Aggregate delineation between ungrouted model Cases.

To illustrate the interplay between major parameters in the analytical model, Figure 6-14 and Figure 6-15 provide the total resistance to externally applied load as a function of major variables $l_e$ and $\rho_{nv}$, respectively. To account for the presence of both shear and normal forces in representing the load-carrying capacity of the bolts, the y axis is expressed as the resultant vector of applied shear and normal forces, calculated as the square root of the sum of the squares.
of the two components, normalized by the tensile capacity of the bolt. Red lines and blue lines indicate tension/shear and compression/shear loading scenarios, respectively, while the black line in Figure 6-14 indicates the absence of normal force and in Figure 6-15 demonstrates that tension/shear and compression/shear behavior is theoretically equivalent at $l_e = 0$.

In both Figure 6-14 and Figure 6-15, it can be seen that the worst-case loading scenario for the total load resistance of an anchor bolt is in pure shear, represented by the black line. When $\rho_{nv}$ and $l_e$ both equal zero, the familiar shear strength of 0.6 times the tensile strength is produced. Strength decreases significantly with increases from zero exposed length from the interaction of shear and moment forces on the cross-section. At higher levels of exposed length, as the shear required to produce high levels of moment diminish and the limiting strength reaches an asymptote. At every stepwise increase in $\rho_{nv}$, the tension/shear capacity and compression/shear capacity are unified at zero bolt exposed length in the absence of bending moments. In each case, as the exposed length increases, the tension/shear capacity increases compared to the compression/shear capacity due to second-order moment relief from the tension force. Depending on $\phi_{char}$, there will be a transition point in tension/shear behavior where the loading angle allows full moment relief, transitioning from Case 2 to Case 3 behavior; this is indicated by red horizontal lines in Figure 6-14 and the unification of tension/shear total load with pure shear total load at $\rho_{nv} = 2$ in Figure 6-15. The maximum total load, of course, occurs for pure tension and compression loading with theoretical capacity equal to the tensile strength.

$\phi_{char}$ also has an effect on the ultimate capacity presented in these models; a more ductile material will allow a greater ultimate angle and thus higher second-order reduction in bending moment. General behavioral trends in Figure 6-14 and Figure 6-15, however, still apply to all bolt ductilities.
Figure 6-14. Effect of exposed length on predicted ungrouted total load resistance.

Figure 6-15. Effect of applied load ratio on predicted ungrouted total load resistance.
6.3.3.4 Model sensitivity to variable assumptions

For anchor bolt steel strength in ungrouted stand-off base plate connections, the greatest sources of potential model error between experimental and analytical determination of anchor bolt steel strength in ungrouted stand-off base plate connections were identified as the adopted value of plastic section modulus, $Z$, and the cross-sectional rotation, $\phi_{cs}$. The term “cross-sectional rotation” refers to the possible rotation at the top and the bottom of the exposed length due to leveling nut rotation (e.g. as in Figure 5-21) and distributive shear force transfer into concrete (see Figure 5-6). In the analytical models, values of $Z$ and $\phi_{cs}$ were assumed. To demonstrate potential effects of different values for both potential sources of error, however, the impacts on bolt strength in the pure shear loading scenario for conceivable values of $Z$ and $\phi_{cs}$ are demonstrated in Figure 6-16 and Figure 6-17.

Figure 6-16 shows how the assumed plastic section modulus impacts the predicted strength using the ungrouted analytical model. Positions on the x axis represent the ratio of a given value of $Z$ to the range of permissible values. The y axis represents values of predicted shear force normalized by the predictive value associated with the empirically derived plastic section modulus, $Z_{emp}$, obtained as described in Section 6.3.1. It can be seen that for an exposed length of 3 dia., within the permissible range of values between minimum root and gross bolt diameters the calculated resistance to shear load can vary from 0.7 to 1.1 of the value associated with $Z_{emp}$. Nonetheless, it is believed that the $Z$ value was chosen with strong rational basis. Vertical black lines on Figure 6-16 indicate notable values of $Z$, with $Z_{nt}$ corresponding to the net tensile area from (2-52), $Z_{min}$ to the minimum root area, $Z_{gross}$ to the gross bolt diameter, and $Z_{ave}$ to the average diameter between the root mean and the gross bolt. Based on the conservative proximity of $Z_{ave}$ to $Z_{emp}$ and the logical assumption that bending occurs over an “average” cross-section,
it is believed that $Z_{ave}$ is appropriate for determining the bending moment capacity of threaded circular cross-sections in the absence of empirical results.

![Figure 6-16. Influence of assumed plastic section modulus on predicted pure shear bolt resistance.](image)

Cross-sectional rotation can favorably influence anchor bolt steel strength for bolts loaded in shear. Critical cross-sections at the top and bottom of the exposed length may both effectively rotate due to leveling nut rotation and distributive shear transfer into concrete, respectively. Figure 6-17 demonstrates the impact of cross-sectional rotation on the prediction of anchor bolt steel strength for the pure shear loading condition. Three values of exposed length are given; it can be seen that at the 3 bolt dia. exposed length, a 20 degree cross-sectional rotation can produce a 30 percent increase in strength compared to the zero-rotation baseline. Leveling nut rotation and anchor bolt curvature below the spalled concrete surface were both observed during the course of testing, indicating that there was some amount of effective cross-sectional rotation;
this has been identified as the most likely source of unmeasured experimental error between the analytical models and tests.

Figure 6-17. Influence of cross-sectional rotation on predicted pure shear bolt resistance.

If the bending stiffness of bolts is different at the top and the bottom of the exposed length due to the allowance of the leveling nut to rotate, base plate flexibility, or complex bending transfer into concrete, it is possible that one end of the exposed length may experience greater bending moments, translating to an overall reduced strength of the anchor bolt as demonstrated analytically by Liu (2014) in Section 2.3.3.1. Unequal bending stiffnesses at the top and bottom of the exposed length may account for some data points that failed at magnitudes of load below predictive models. In general, because the values produced by the models were, on average, very accurate and slightly conservative, it is believed that, to a degree, capacity-reducing errors from unequal moment distribution at the top bottom of the cross section and capacity-enhancing errors from cross-sectional rotation were either minimal or semi-offsetting.
6.3.4 Prediction of Anchor Bolt Steel Strength in Stand-off Base Plates with Grout and other Stiff Intermediate Layers

6.3.4.1 Proposed analytical method

This section describes the assumptions and processes leading to a model for anchor bolt steel strength in stand-off base plates with a stiff intermediate layer, typically grout, mortar, or steel shim stacks, placed between the concrete surface and the bottom of the base plate. For simplicity, all connections with a stiff intermediate layer are classified as “grouted.” This model bears similarity to the rational approach given by Gresnigt et al. (2008), which was reviewed in Section 2.3.4 of the Literature Review, with the following notable additions and exceptions:

- Failure of the anchor bolt is due to the combination of shear and tension stresses on the cross-section. The approach in Gresnigt et al. (2008) approach assumes that the bolt undergoes pure tensile failure.

- All values of applied tension-to-shear ratio are accounted for. The Gresnigt et al (2008) approach assumes that shear capacity is independent of applied tensile load, which is valid (due to the presence of the clamping force and relative stiffnesses of the bolt and intermediate layer as described below) for many, but not all, circumstances.

- Three states of friction contribution to the resistance of horizontal loads are accounted for depending on the angle of applied load and angle of the anchor bolt: resistance from friction alone, combined resistance from friction and bolt shear, and resistance from bolt shear alone. The Gresnigt et al. (2008) approach inherently assumes that both friction and bolt strength contribute to the resistance of horizontal force, which, again, accounts for many, but not all, circumstances.

- The ultimate strength of the model is related to the ultimate stress of the bolt material, while the Gresnigt et al. (2008) model relates ultimate strength to yield strength of the bolt without supporting explanation.

A schematic of a displaced anchor bolt in a grouted stand-off base plate connection is shown in Figure 6-18. Dashed lines indicate the location of the free-body cut for which forces are illustrated in Figure 6-19 and in the following subsections. To maintain its original length as the bolt displaces horizontally due to the applied shear force, the bolt will tend to displace downward. As shown in Figure 6-19, the stiff layer between the base plate and concrete surface
resists this downward displacement, producing a net clamping force $F_c$ on the intermediate layer and a corresponding contribution to the tensile force $T_B$ on the displaced bolt cross-section, where tension is positive.

Figure 6-18. Simplified schematic of displaced bolt in grouted-stand-off base plate connection.

Figure 6-19. Free-body diagram showing clamping force resulting from displaced anchor.

It is assumed that as the plate moves and the restraining grout pad causes the bolt to stretch, bending moment equilibrium is satisfied by the newly developed tension force acting over the horizontal displacement equaling the shear force acting over the exposed length,
resulting in zero bending moment on the bolt cross-section. It should be noted that in Figure 6-19 $F_c$ forces at the top and the bottom of the cut are collinear and thus cancel in bending moment equilibrium. Essentially, the bolt becomes a tension tie and as a result the bolt cross-section only experiences shear and tensile forces. In the absence of bending moment, upper limit and lower-bound approaches for the interaction of forces on the anchor bolt cross-section produce the same tension vs. shear interaction equation that governs steel failure, similar to ungrouted Case 3 above.

Relative stiffness between an anchor bolt and the surrounding intermediate layer impacts the transfer of applied normal forces, $N_A$. Tensile forces will release $F_c$ and compressive forces will contribute to $F_c$ in relation to the relative stiffnesses of the bolt and the surrounding grout material. Similarly, tensile force in the bolt will decrease as expressed in (6-42) with applied compressive force. It can be seen that for grout layers with infinite relative stiffness to the bolts they surround, $F_c$ will directly absorb the applied normal force and that, so long as $F_c > 0$, bolt tension will remain constant irrespective of the magnitude of applied normal force.

\[
k_G = \frac{A_{G,e}E_G}{h_G} \quad (6-39)
\]

\[
k_B = \frac{A_BE_S}{h_G} \quad (6-40)
\]

\[
F_{c,f} = F_{c,i} - \frac{k_GN_A}{k_B + k_G} \quad (6-41)
\]

\[
T_{B,f} = T_{B,i} + \frac{k_BN_A}{k_B + k_G} \quad (6-42)
\]

where $k_G$ = axial stiffness of effective grout pad clamping

$A_{G,e}$ = effective grout pad clamping area

$E_G$ = grout Young’s modulus

$h_G$ = height of the grout pad

$k_B$ = axial stiffness of bolt

$A_B$ = bolt net tensile area
$E_s$ = steel Young’s modulus
$F_{c,f}$ = final clamping force after normal force
$F_{c,i}$ = initial clamping force from vertical restraint of bolt
$N_A$ = applied normal force (tension positive)
$T_{B,f}$ = tension in bolt after applied normal force
$T_{B,i}$ = initial tensile force in bolt = $|F_{c,i}|$

(6-39) through (6-42) are applied to the grouted Eccentric Shear test dimensions in (6-43) through (6-46). To produce a “worst-case” influence of relative stiffness, a lower-bound grout stiffness is assumed with a 4000 psi compressive strength and experimental maximum stand-off distance/grout pad height of 2.5 in. Dividing the 10 in. width of the bolt into two segments, a lower-bound effective grout bearing area of 25 in.$^2$ is produced. Approximating grout stiffness as $57,000 \sqrt{f_c'} = 57,000 \sqrt{4000 \text{ psi}} = 3610$ ksi, the effective grout pad stiffness per bolt is equal to (6-43). The bolt area of 0.226 in.$^2$ corresponds to an axial stiffness as given in (6-44).

$$k_G = \frac{A_{G,E}E_G}{h_G} = \frac{(25 \text{ in.}^2)(3610 \text{ ksi})}{2.5 \text{ in.}} = 36,100 \text{ kip/in.}$$

(6-43)

$$k_B = \frac{A_BE_S}{h_G} = \frac{(0.226 \text{ in.}^2)(29000 \text{ ksi})}{2.5 \text{ in.}} = 2620 \text{ kip/in.}$$

(6-44)

$$F_{c,f} = F_{c,i} - \frac{k_GN_A}{k_B + k_G} = F_{c,o} - \left( \frac{36,100}{36,100 + 2620} \right)N_A = F_{c,o} - 0.93N_A$$

(6-45)

$$T_{B,f} = T_{B,i} + \frac{k_BN_A}{k_B + k_G} = F_{c,o} + \left( \frac{2620}{36,100 + 2620} \right)N_A = F_{c,o} + 0.07N_A$$

(6-46)

(6-45) and (6-46) indicate that under these “worst-case” assumptions, the clamping force absorbs 93% of an applied normal force, the remaining 7% traveling through the anchor bolt to maintain systemic equilibrium. If a tensile force is applied, the less-than-complete uptake of applied normal force by clamping force compared to an infinite ratio of grout-to-bolt stiffness will result in higher contributions to overall strength from friction while the additional force on the bolt will cause reductions in overall strength. The inverse is true of applied compressive
forces; as a result, errors arising from an assumption of an infinite grout-to-bolt stiffness ratio offset to some degree.

Due to the relatively high stiffness of grout to bolts in typical connections, offsetting errors from implementing true stiffness ratios, and the practical complexity of incorporating relative stiffness into the grouted model, the grout is mathematically taken as infinitely relatively stiff to the bolts in the subsequently presented analytical model. The following implications emerge from this assumption: 1) the displaced length of a stick-model bolt is proportional to the sum of squares of the horizontal displacement and the original length of exposed length and 2) the net clamping force $F_C$ is simply equal to the bolt tension force $T_B$ less the applied normal force $N_A$ as shown in (6-47). It is worth noting that $N_A$ may not exceed $T_B$; when $N_A = T_B$ there is simply no clamping force on the grout pad. For simplicity in derivation and, later, comparison to test results, the applied normal force will often be referred to in terms of its ratio proportion to the applied shear force, the ratio defined as $\rho_{nv}$. Use of $\rho_{nv}V_A$ in place of $N_A$ is especially useful in the derivation of Cases due to the elimination of a dependent variable.

$$F_C = T_B - N_A = T_B - \rho_{nv}V_A$$  \hspace{1cm} (6-47)

where
- $F_C$ = net clamping force on the grout pad
- $T_B$ = tension experienced by anchor bolt cross-section
- $N_A$ = applied normal force, tension positive
- $V_A$ = applied shear force
- $\rho_{nv}$ = ratio of applied axial force to shear force = $N_A/V_A$

Given the assumptions stated above and referring again to Figure 6-18 and Figure 6-19, the model can be developed. It is assumed in this model that the anchor bolt bends over a zero-length hinge, resulting in a simplified “stick model” for the geometry of the displaced bolt. The relationship between the displaced bolt and forces in the section follows:
\[
\frac{\delta_h}{l_{\text{eff}}} = \frac{V_A}{T_B} = \tan(\phi)
\]  \hspace{1cm} (6-48)

where

- \(\delta_h\) = horizontal bolt displacement
- \(l_{\text{eff}}\) = effective length of displaced bolt
- \(\phi\) = final stick-model rotation of the displaced bolt

While the stick model allows a reasonable simplification of the displaced shape of the bolt, it is unconservative to assume that the cross-section of the bolt has rotated at the critical section, as shear stresses would not be present under this assumption. In reality, the top and the bottom of the anchor bolt exposed length experience near-zero cross-sectional rotation, where horizontally applied forces translate to shear stresses on the cross-section. Thus, the applied shear force, \(V_A\), is resisted by some combination of friction force and shear force, \(V_B\), acting on the bolt cross-section according to (6-49), where the effective friction force is produced by the clamping compressive force less the applied tension force (or, if compression is applied, adding the compressive force) multiplied by the least coefficient of friction, \(\mu\), in the horizontal transfer of forces. Shear force on the bolt acts parallel to the applied horizontal force at its critical section. The friction force could be viewed as “relieving” the shear component of the angled bolt force from the critical bolt section. Combining (6-48) and (6-49) to relate tension forces to the applied shear load, (6-50) is produced. Rearranging (6-50), \(V_A\) is isolated to be expressed in terms of all other variables, resulting in (6-51).

\[V_A = V_B + \mu F_C = V_B + \mu(T_B - N_A) = V_B + \mu(T_B - \rho_{nv}V_A)\]  \hspace{1cm} (6-49)

\[\Rightarrow V_A = V_B + \mu \left(\frac{V_A l_{\text{eff}}}{\delta_h} - \rho_{nv}V_A\right) = V_B + \mu(V_A \cot(\phi) - \rho_{nv}V_A)\]  \hspace{1cm} (6-50)

\[\Rightarrow V_A = \frac{V_B}{1 - \mu\left(\frac{l_{\text{eff}}}{\delta_h} - \rho_{nv}\right)} = \frac{V_B}{1 - \mu\left(\cot(\phi) - \rho_{nv}\right)}\]  \hspace{1cm} (6-51)

where

- \(\mu\) = least coefficient of friction in horizontal force transfer
- \(V_B\) = shear force experienced by anchor bolt
At low displacements the resistance from the bolt varies not only due to geometry, but to the state of linear-elastic stress. In the Gresnigt et al. (2008) model simplifying the stresses experienced by the bolt to uniaxial tension, the elastic limit of the displacement was derived as (6-52). In an elastic-perfectly plastic system, this translates directly to (6-53) to provide the onset of fully plastic deformations.

\[
d_{hEL} = l_{eff} \sqrt{\frac{2\sigma_y}{E}} \\
d_{hPL} = l_{eff} \sqrt{\frac{2\sigma_o}{E}}
\]

(6-52) (6-53)

where

- \(d_{hEL}\) = horizontal bolt displacement corresponding to the elastic limit of behavior (Gresnigt et al. (2008))
- \(d_{hPL}\) = horizontal bolt displacement corresponding to the onset of plastic behavior (derived from Gresnigt et al. (2008))
- \(\sigma_y\) = tensile yield stress of bolt material
- \(\sigma_o\) = tensile ultimate stress of bolt material
- \(E\) = modulus of elasticity of bolt material

In the elastic-perfectly plastic material model assumption, beyond the limit defined by (6-53) the bolt is assumed to be in a fully stressed (but not necessarily fully strained) state; i.e. the interaction of tension and shear forces on the bolt critical cross-section will produce the relationship derived in Section 2.2.3.1, resulting in Equation (6-54).

\[
\left(\frac{T_B}{N_o}\right)^2 + \left(\frac{V_B}{\alpha N_o}\right)^2 = 1
\]

(6-54)

where

- \(T_B\) = tension force on anchor bolt at critical (unrotated) section
- \(V_B\) = shear on anchor bolt at critical (unrotated) section
- \(\alpha\) = ratio of shear to tension strength, taken as 0.6
- \(N_o\) = normal strength capacity of anchor bolt (from uniaxial tension test)

Rupture of the anchor bolt occurs when 1) the interaction of tension and shear on the cross-section are at a critical combination, i.e. (6-54) is true and 2) the bolt has reached a critical strain.
limit corresponding to an ultimate horizontal displacement/rotation of the bolt exposed length. Bolt ductility allows (6-54) to be satisfied without bolt rupture; indeed, it will be shown that after initial critical satisfaction of (6-54) the geometry allowed by bolt ductility promotes significant increases in overall resistance to shear load.

In the model proposed below, $\rho_{nv}$, $\mu$, and ultimate rotation of the displaced bolt generate combinations of tensile and shear forces on the bolt, which, at the ultimate strain, correspond to the ultimate capacity of the bolt/grout resistance to applied load.

Using (6-51) and (6-54), criteria for the presence/absence of friction and bolt shear can be developed and an ultimate predictive load can be found. Four specific cases emerge for the total resistance to applied shear load that can be categorized in terms of the mechanism of shear/horizontal load resistance: **Case 1** by friction alone with bolt rupture, **Case 2** by bolt shear alone, **Case 3** by a combination of friction and bolt shear, and **Case 4** by friction alone with no bolt rupture. The following expressions are derived for evaluating test results with a known horizontal displacement at ultimate load. It can be seen that the criteria for categorization of shear resistance rely on ratios of the ultimate horizontal displacement to the effective length of the bolt; these ratios could easily be expressed in terms of the corresponding angle of rotation, $\phi$, at failure. Thus, any use of $\frac{d_h}{l_{eff}}$ can be replaced by $\tan(\phi)$ and any use of $\frac{l_{eff}}{d_h}$ can be replaced by $\cot(\phi)$. As with the model for anchor bolt strength in ungrouted connections, it is believed that a characteristic angle, $\phi_{char}$, can be associated with a material without respect to exposed length. If this hypothesis holds, a predictive value (as opposed to an associative value with post-applied ultimate displacements) value can be produced by the model.

**Case 1: Shear resisted by friction alone after bolt yield.** Figure 6-20 shows the internal free body of Case 1 forces. When the quantity $\mu \left( \frac{l_{eff}}{d_h} - \rho_{nv} \right)$ is greater than 1, or,
equivalently, \( \frac{\delta_h}{l_{eff}} < \frac{\mu}{1 + \mu \rho_{nv}} \), the mathematical relationship between applied shear load and the bolt shear in (6-51) is negative, i.e., physically impossible. Less formally, this case may occur when the ductility of a bolt is very low, when the friction coefficient is very high, and in certain low-tension or high-compression circumstances. In this case, shear from the bolt cannot contribute to the resistance of the applied shear load, hence the shear term is removed from (6-49), resulting in all shear forces taken up by friction. In the absence shear stress on the bolt section, \( F_c \) simply becomes the tensile capacity, \( N_o \), of the bolt. Rearranging (6-57), the critical value of \( V_A \) becomes (6-58).

Figure 6-20. Free-body diagram for grouted Case 1 forces.

\[
\begin{align*}
T_B &= F_c = N_o \quad (6-55) \\
V_B &= 0 \quad (6-56) \\
V_A &= \mu(N_o - N_A) = \mu(N_o - \rho_{nv} V_A) \quad (6-57) \\
\Rightarrow V_A &= \frac{\mu N_o}{1 + \mu \rho_{nv}} \quad (6-58)
\end{align*}
\]
Case 2: Shear resisted by bolt shear alone. Figure 6-21 shows the internal free body of Case 2 forces. At tension-to-shear ratios $\rho_{nv}$ greater than $\frac{l_{eff}}{d_h}$, it can be seen from Equation (6-51) that the clamping force at ultimate load is fully overcome, resulting in no frictional contribution to the resistance of applied shear load. Removing all friction terms from Equation (6-51), $V_B$ becomes $V_A$, producing (6-62). Rearranging to solve for $V_A$, (6-63) results, which can be seen is a simple rearrangement of interaction equation between tensile and shear forces.

![Free-body diagram for grouted Case 2 forces.](image)

If $\rho_{nv} > \frac{l_{eff}}{\delta_h}$

\[
T_B = N_A = \rho_{nv} V_A
\]  \hfill (6-59)

\[
F_c = 0
\]  \hfill (6-60)

\[
V_B = V_A - \mu F_c = V_A
\]  \hfill (6-61)

\[
\left(\frac{T_B}{N_o}\right)^2 + \left(\frac{V_B}{\alpha N_o}\right)^2 = \left(\frac{\rho_{nv} V_A}{N_o}\right)^2 + \left(\frac{V_A}{\alpha N_o}\right)^2 = 1
\]  \hfill (6-62)

\[
\Rightarrow V_A = \frac{N_o}{\sqrt{\rho_{nv}^2 + \left(\frac{1}{\alpha}\right)^2}}
\]  \hfill (6-63)
Case 3: Applied resisted by both friction and bolt shear. Figure 6-22 shows the internal free body of Case 3 forces. For all scenarios outside of Cases 1, 2, and 4, the applied shear is resisted by both friction and bolt shear. Equation (6-51) can be rearranged in terms of bolt shear as Equation (6-67), which can then be placed in the interaction equation. Solving for the applied shear load, Equation (6-69) is produced.

Figure 6-22. Free-body diagram for grouted Case 3 forces.

\[
if \quad \rho_{nv} < \frac{l_{eff}}{\delta_h} \quad \text{and} \quad \frac{\delta_h}{l_{eff}} > \frac{\mu}{1 + \mu \rho_{nv}}
\]

\[
T_B = \frac{V_A l_{eff}}{\delta_h}
\]

(6-64)

\[
F_c = T_B - N_A = T_B - \rho_{nv} V_A
\]

(6-65)

\[
V_B = V_A - \mu F_c = V_A - \mu (T_B - N_A)
\]

(6-66)

\[
V_B = V_A \left(1 - \mu \left(\frac{l_{eff}}{\delta_h} - \rho_{nv}\right)\right)
\]

(6-67)

\[
\left(\frac{V_A l_{eff}}{\delta_h} \frac{\delta_h}{N_o^2}\right)^2 + \left(\frac{V_A \left(1 - \mu \delta_h \rho_{nv}\right)}{\alpha N_o}\right)^2 = 1
\]

(6-68)
\[ V_A = \frac{N_o}{\left( \frac{l_{\text{eff}}}{\delta_h} \right)^2 + \left( 1 - \mu \frac{l_{\text{eff}}}{\delta_h} - \rho_{nv} \right)^2} \] (6-69)

**Case 4: No shear force experienced by bolt.** Figure 6-24 shows an internal free body of Case 4 forces. Where the friction force produced from a compressive load is greater than the applied shear load, i.e. \( V_A < \mu (-N_A) \), or, equivalently, \( \mu \rho_{nv} < -1 \), the bolt will not experience shear forces. In this case, failure will be dictated by the grout material strength, which is outside of the scope of this dissertation.

![Free-body diagram for grouted Case 4 forces.](image)

**6.3.4.2 Model sensitivity to variable assumptions**

Figure 6-24 and Figure 6-25 show how the grouted analytical model responds to assumptions about the systemic coefficient of friction, \( \mu \), and the effective angle of bolt failure, \( \phi \), respectively. Kinks in both graphs delineate transitions between different Cases. In both figures, black lines represent a pure shear loading condition while blue and red lines represent compression/shear and tension/shear applied loading conditions, respectively. Predictive values
of applied shear force are normalized by the predicted shear force for zero values of the respective variables on the y axes, which is represented on the y axes as \( V_{A,o} \).

It is common practice to assume that the coefficient of friction between concrete/grout and steel materials is 0.4. Nonetheless, different materials and other factors may influence the friction transfer in grouted stand-off base plate connections. Figure 6-24 shows the influence on the model of \( \mu \) for different values of \( \rho_{nv} \). It can be seen, as would be expected, that when there is greater systemic tension, the resistance of the system is less influenced by changes in friction. However, even when high amounts of tension are present in the system, even relatively small changes in \( \mu \) can translate to predictions of systemic resistance to applied shear loads. The post-kink behavior of the curves at high values of mu indicate a transition from Case 1 to Case 2, beyond which the absence of shear on the bolt cross-section allows linear increases in strength with increases in \( \mu \). For Case 1 to occur, grout must initially crack before clamping action is established and shear force is absorbed from the bolt to the frictional resistance of the grout-to-base plate interface.

Figure 6-25 demonstrates that the final angle of anchor bolt rotation, \( \phi \), has a profound impact on the resistance of a steel-critical grouted stand-off base plate connection to applied shear forces from the additional response to load of the angled bolt. Higher \( \phi \) values translate to a more favorable stress distribution on the bolt cross-section. Plateaus at low levels of \( \phi \) represent Case 1 behavior, for which the bolt angle does not come into play in the mode until a value allowing anchor bolt contributions is reached, initiating Case 3 behavior. At high enough combinations of \( \rho_{nv} \) and \( \phi \), \( \mu \) no longer contributes to strength and the bolt is sufficiently ductile to align with the angle of applied load, transitioning from Case 3 to Case 2 behavior; for a given \( \rho_{nv} \), then, further increases in \( \phi \) do not contribute additional strength.
Figure 6-24. Effect of friction on predicted grouted resistance to applied shear.

Figure 6-25. Effect of bolt rotation on predicted grouted resistance to applied shear.
6.4 Model and Code Comparisons to Experimental Results

In this section, the analytical models developed in Section 6.3.3.4 and Section 6.3.4.1 are applied to experimental data for ungrouted and grouted tests with 5/8 in. dia. F1554 anchor bolts. For both analytical models, data are expressed in terms of measured values normalized by predictive values.

6.4.1 Ungrouted Tests

Table 6-3 aggregates results for all ungrouted tests with 5/8 in. F1554 anchor bolts conducted in the Torsion and Eccentric Shear experimental arrangements, comparing experimental results to both upper-limit and lower-bound methods. ES1 was not included due to the significant bearing forces produced against the concrete surface that produced an unrepresentatively high failure capacity and ES17 was not included due to the error in data acquisition. Also provided are the exposed length of the anchor bolts, the ratio of applied normal force to shear force, \( \rho_{nv} \), the ultimate shear and normal force of individual tests, and the loading Cases as defined by Section 6.3.4.1, where predicted values of shear force, \( V_A \), were calculated using (6-58), (6-63), and (6-69) for Cases 1, 2, and 3, respectively.

The separation between Cases was determined using a best-fit approach to data. It was found that applying Case 2 to all nominal 6 in. and 12 in. eccentricity tension/shear tests and Case 3 to all 18 in. and 24 in. tests matched experimental data well. Thus, the most extreme angle of applied load used in Case 3 with respect to the initial axis of the exposed length was approximately 36 degrees. All values of \( l_{eff} \) included a 0.5 dia. extension below the concrete surface. For tests that failed above the leveling nut, \( l_{eff} \) was extended further to directly above the leveling nut (equal to one anchor bolt diameter), as significant leveling nut rotations and the failure itself indicated bending moment transfer into this critical section of the bolt.
Table 6-3. Comparisons between F1554 bolt data and the proposed ungrouted analytical model

<table>
<thead>
<tr>
<th>Test</th>
<th>$\frac{l_{LN}}{d_b}$</th>
<th>$V_{u,bolt}$ (kip)</th>
<th>$N_{u,bolt}$ (kip)</th>
<th>$\rho_{nv}$</th>
<th>Fail. Loc.</th>
<th>Case</th>
<th>Upper Limit $V_{UL}$ (kip)</th>
<th>Lower Bound $V_{UL}$ (kip)</th>
<th>$\frac{V_{UL}}{V_{UL}}$</th>
<th>$V_{UL}$ (kip)</th>
<th>$\frac{V_{UL}}{V_{UL}}$</th>
<th>$f_{ib}$ (2011)</th>
</tr>
</thead>
<tbody>
<tr>
<td>T1</td>
<td>0.0</td>
<td>11.5</td>
<td>0</td>
<td>0</td>
<td>CS</td>
<td>1</td>
<td>12.4</td>
<td>0.93</td>
<td>12.4</td>
<td>0.93</td>
<td>12.42</td>
<td>0.93</td>
</tr>
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<td>T2</td>
<td>0.0</td>
<td>10.6</td>
<td>0</td>
<td>0</td>
<td>CS</td>
<td>1</td>
<td>12.4</td>
<td>0.85</td>
<td>12.4</td>
<td>0.85</td>
<td>12.42</td>
<td>0.85</td>
</tr>
<tr>
<td>T4</td>
<td>0.8</td>
<td>6.8</td>
<td>0</td>
<td>0</td>
<td>BN</td>
<td>1</td>
<td>6.9</td>
<td>1.00</td>
<td>6.2</td>
<td>1.11</td>
<td>3.82</td>
<td>1.79</td>
</tr>
<tr>
<td>T5</td>
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<td>0</td>
<td>0</td>
<td>BN</td>
<td>1</td>
<td>6.9</td>
<td>1.07</td>
<td>6.2</td>
<td>1.19</td>
<td>3.82</td>
<td>1.93</td>
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<tr>
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<td>0</td>
<td>0</td>
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<td>1</td>
<td>10.4</td>
<td>1.08</td>
<td>9.0</td>
<td>1.25</td>
<td>5.16</td>
<td>2.17</td>
</tr>
<tr>
<td>T15</td>
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<td>10.2</td>
<td>0</td>
<td>0</td>
<td>BN</td>
<td>1</td>
<td>7.8</td>
<td>1.31</td>
<td>6.9</td>
<td>1.48</td>
<td>4.13</td>
<td>2.48</td>
</tr>
<tr>
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<td>3.2</td>
<td>0</td>
<td>0</td>
<td>BN</td>
<td>1</td>
<td>3.0</td>
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<td>1.09</td>
<td>2.15</td>
<td>1.50</td>
</tr>
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<td>0</td>
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| Ave | 1.04 | 1.08 | 2.38 |
| Std. Dev. | 0.14 | 0.13 | 1.41 |
| COV (%) | 13% | 12% | 59% |

From Table 6-3 it can be seen that both the upper-limit and lower-bound methods fit experimental data well, with an average ratio of experimental-to-predicted values of 1.04 (COV = 14%) and 1.08 (COV = 13%), respectively. These results facilitate the conclusion that this analytical model performs satisfactorily and that upper-limit and lower-bound methods produce comparable and satisfactory results. Significantly, four-bolt tests ES21 and ES22 both matched the analytical model nearly perfectly, at 3% and 2% error, respectively, following the assumption...
that the neutral axis of bending in four-bolt tests lies at the midpoint between the tension and compression bolt lines and that failure is governed by the compression/shear bolts.

Table 6-4 shows how predictive models measured against Torsion tests with 354BD and Stainless steel materials; it can be seen that the limited number of 354BD tests indicated slightly lower-than-predicted values, but nothing worthy of conclusive arguments or immediate significance. The stainless steel test, T24, failed at a much higher-than-predicted load, owing, as discussed in Section 5.4.2.4, to the ductility of the material allowing the leveling nut to rotate significantly and bear against the concrete surface.

Table 6-4. Comparisons between 354BD and stainless steel bolt data and the proposed ungrouted analytical model

<table>
<thead>
<tr>
<th>Test</th>
<th>( \frac{l_{LN}}{d_b} )</th>
<th>( \rho_{nv} )</th>
<th>( V_{u, bolt} ) (kip)</th>
<th>( N_{u, bolt} ) (kip)</th>
<th>( V_{u, bolt} ) (kip)</th>
<th>Upper Limit</th>
<th>Lower Bound</th>
<th>( \frac{V_{u, bolt}}{V_{pred}} )</th>
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<td>2.58</td>
<td>6.0</td>
<td>2.8</td>
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Figure 6-26 shows how Torsion and LS tests responded to changes in exposed lengths. Data are accompanied by upper-limit and lower-bound curves, both assuming no additions to exposed length and with a 0.5 dia. extension due to concrete spalling. Figure 6-27 and Figure 6-28 show the measured-to-predicted ratio of first-order values of all 5/8 in. dia. F1554 tests as a function of exposed length and \( \rho_{nv} \), respectively using the upper-limit method. As \( \rho_{nv} \) increased, accompanying second-order release in moment is apparent, clearly indicating that for the most accurate determination of the capacity of bolts experiencing tension forces, second order effects must be taken into account. Higher error at the 3 dia. exposed length level comes as a result of the lower prediction associated with higher calculated first-order bending moments occurring over the exposed length. Figure 6-29 and Figure 6-30 provide the ratio of measured values to second-order upper-limit predictions as a function of major variables exposed length and \( \rho_{nv} \).
respectively, applying the analytical Cases as given in Table 6-3. It can be seen that the data are centered about the measured/predicted = 1 line with no sensitivity to either exposed length or $\rho_{nnv}$. As in Figure 6-27 and Figure 6-28, the stainless steel result (green fill) is much higher than predicted values due to the ductility of the test allowing contact between the displaced bolt and the concrete surface. In these plots, predictive values for all three materials tests are included; black circles indicate Torsion tests, while green and orange fill colors in black circles indicate stainless steel and 354BD materials, respectively. Unfilled circles represent F1554 tests. Blue and red diamonds represent, respectively, tension/shear and compression/shear Eccentric Shear tests. As in all other analysis of results, the exposed length over which first-order moments were calculated over a distance including a length of anchor bolt one-half diameter below the concrete surface to account for spalling.

Figure 6-26. Summary of pure shear results against predictive models.
Figure 6-27. Ungrounded results normalized by first-order predicted values as a function of exposed length.

Figure 6-28. Ungrounded results normalized by first-order predicted values as a function of $\rho_{nv}$.
Figure 6-29. Ungrouted results normalized by second-order predictions as a function of exposed length.

Figure 6-30. Ungrouted results normalized by second-order prediction as a function of $\rho_{nv}$.
6.4.2 Grouted Tests

Table 6-5 shows how experimental data from grouted tests compare to the analytical model derived in Section 6.3.4 using three different assumptions for the determination of the angle, to an interpretation of ACI (2011) provisions, and to the Gresnigt et al. (2008) model given in (2-62). The treatment of experimental data for the respective methodologies is given in subsequent paragraphs. Within Table 6-5, the failure location of the bolt is provided; it can be seen that for the Torsion/Large-scale tests and the compression/shear Eccentric Shear tests, the bolt failed predominantly below the leveling nut, while in tension/shear Eccentric Shear tests the bolts failed predominantly directly above the leveling nut. The shim stack test, T27, mortar test, T28, and the test without a leveling nut below the base plate, T26, were not included in this analysis to allow for statistical comparisons between data using the standard nonshrink grout with a leveling nut below the base plate.

For tests in which failure above the leveling nut, as with ungrouted tests, some amount of rotation occurred. If, hypothetically, the leveling nut were to rotate completely in line with the failure angle of the bolt, the measured horizontal displacement would act over the distance to the top of the nut as opposed to the anchor bolt exposed length. For this reason, the proposed model was evaluated in two ways, first considering the effective length to the bottom of the leveling nut, second considering the effective length to the top of the leveling nut. In both assumptions, as with the model for ungrouted tests, a one-half diameter length is added to the exposed length to account for concrete spalling.

In reality, the horizontal displacement included some amount of leveling nut rotation and slip of anchor bolts in oversized holes. In addition, the displaced bolt is not perfectly linear as assumed in the stick-model simplification. At either moment-restrained end of the exposed length there is a transitional length containing some amount of curvature; as a result, the angle of
bolt rotation is maximized in the center of the exposed length. The distribution of plasticity to
determine this angle is complex, but the assumption that a characteristic angle, $\phi_{char}$, can be
applied over all values of exposed lengths and loading conditions is evaluated in a third use of
the analytical model using a best-fit constant angle applied to all tests.

The ACI (2011) strength was derived using the interaction relationship between shear and
tensile forces given in (E-18), where the 2 exponent is reduced to 5/3 to match ACI provisions.
The shear capacity of the anchor bolt is simply reduced by a factor of 0.8 to account for
reductions in strength due to the presence of the grouted exposed length. This simplified model
does not account for grout clamping and friction forces. In the absence of direct instruction, for
compression/shear loading scenarios the ACI-predicted strength was simply taken as 0.8 times
the shear capacity.

Finally, values given using the Gresnigt et al. (2008) model are provided using two
assumptions, the first as recommended using the yield strength of the bolt steel material, $f_y$, the
second using the ultimate strength of the bolt material, $f_u$. $f_y$ was taken as 55 ksi, the prescribed
yield strength of the F1554 material (which was validated by the yield in the true stress
determination for the material given in Figure 5-3. To best compare to other models, the
effective length of the Gresnigt et al. (2008) model was taken as the distance from one half bolt
diameter below the concrete surface to the bottom of the leveling nut. Also not included is ES32,
in which anchor bolt failure occurred as a result of full shear uptake by friction, producing a
nearly pure tensile load on the critical bolts.

Figure 6-31 and Figure 6-32 provide data points normalized by the predictive model using
the best-fit angle assumption as a function of $\rho_{nv}$ and exposed length. Filled gray markers
indicate Torsion (circles) and Large-scale (hexagrams) data, while red, blue, and magenta diamond markers indicate tension/shear, compression/shear, and four-bolt tests, respectively.

Table 6-5. Comparisons between experimental results for grouted stand-off base plates, rational model, ACI (2011) interpretation, and Gresnigt et al. (2008) models

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<th>$\rho_{NV}$</th>
<th>$\delta_u$ (in.)</th>
<th>$V_{u,b}$ (k)</th>
<th>Fail. Loc.</th>
<th>Below $\frac{V_{u,b}}{V_{pred}}$</th>
<th>Above $\frac{V_{u,b}}{V_{pred}}$</th>
<th>Best Angle</th>
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<td>0.20</td>
<td>0.49</td>
<td>0.15</td>
</tr>
<tr>
<td>COV</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>17%</td>
<td>20%</td>
<td>15%</td>
<td>16%</td>
<td>32%</td>
<td>18%</td>
</tr>
</tbody>
</table>

Case from Section 6.3.4

Use of Fu for the Gresnigt et al. (2008) model was not recommended; these results are given strictly for illustration

Figure 6-31. Grouted results normalized by predictions as a function of $\rho_{NV}$.
The proposed model with the below-nut exposed length assumption produced a very accurate representation of the experimental data with 5% average error and 17% coefficient of variation of the errors, while the above-nut assumption produced generally conservative results with an average error of 32% and error COV of 26%. The proposed method with a best-fit constant failure angle applied to all tests, by definition of best-fit, contained an average error between measured and predicted values of 0%, but also contained the least coefficient of variation at 15% with for an angle, $\phi_{char}$, of 35 degrees. This strongly indicates that the characteristic angle assumption fits data well. The implications of this finding are significant, as the characteristic angle allows a predictive, rather than validatory model for determining accurate strengths of anchor bolts in grouted stand-off base plates using this F1554 Gr. 55 material. Despite its simplicity in modeling the complex interplay of forces, the ACI (2011) provisions provide a conservative estimate of the results with an average error of 26%. The Gresnigt et al. (2008) model using $f_p$ yielded a 52% average error, while using $f_u$ produced the only
unconservative result in Table 6-5, with the predictive model averaging 20% higher than the experimental data.

### 6.5 Effect of Minor Test Variables

While the interaction of minor test variables with major variables and each other cannot be quantified without further investigation, inferences can be gleaned from observation of failure load and load-displacement behavior from their testing. A discussion of each of the minor variables studied follows:

- **Effect of Test Method on Determining Shear Strength.** For the proper determination of the pure steel shear strength of anchor bolts in stand-off base plates, a test methodology that resists overturning moment without restraining vertical displacement and loads the bolt in pure shear in the displaced shape is essential. Direct Shear testing methodology did not successfully achieve these criteria, but Torsion testing methodology was found to satisfy the criteria due to the moment-balancing effects of two applied loads without need for additional vertical restraint.

- **Hole Oversize.** Flush-mounted torsion tests with oversize holes (T1 and T2) showed obvious slip through the hole oversize in the early stages of the tests. Flush-mounted shear strength may be adversely affected by unfavorable anchor bolt positioning within oversize holes. The lower value observed in test T1-B \((0.51T_u)\) is in good agreement with the AASHTO (2013a) commentary recommendation for a 0.8 factor for oversize holes. No adverse effects from hole oversize on ultimate strengths were observed in stand-off base plate tests.

- **Bolt Diameter.** Ultimate loads normalized by respective \(T_u\) values were comparable between equivalent ungrouted stand-off tests with 5/8 in. and 1 in. diameter bolts. It is concluded that the results are representative of the full range of anchor bolt diameters in ungrouted annular base plates. Grouted tests with 5/8 in. and 1.25 in. diameter bolts exhibited similar load-displacement behavior at comparable stand-off distances.

- **Bolt Pretensioning.** Pretensioning bolts was found to have no impact on ultimate strength of anchor bolts in stand-off base plate connections under varying ratios of applied normal-to-shear force and varying exposed lengths and minor impacts on load-displacement behavior. It is well-established, however, that pretensioning a bolt has several benefits in structural engineering practice. No recommendations deviating from standard pretensioning practices are proposed.

- **FRP Grout Pad Retrofit.** The use of an FRP wrap was tested in one Torsion test, T10. It was found that the overdesigned wrap confined the displacement of cracked grout, effectively transforming the behavior and capacity of the grouted stand-off base plate to a flush-mounted equivalent. This test suggests that implementation of an FRP wrap for
grouted stand-off base plates where serviceability may be desired at high loads. Further study may be needed to confirm this for different ratios of applied load.

- **SHIM STACKS AS LEVELING IMPLEMENT.** The Torsion shim stack test, T27, performed similarly to grouted Torsion tests with higher initial slip. The result of this test suggests that anchor bolts with shim stacks may be treated equivalently in design to grouted stand-off base plates.

- **MATERIAL DUCTILITY.** Compared to the baseline F1554 bolt material, the more brittle 354BD material did not perform significantly differently in Torsion tests T22 and T23 other than reductions in strength normalized by tensile capacity. Stainless steel test T24, however, performed markedly differently after the high ductility of the material allowed displacements that caused the leveling nut to make contact with the concrete surface, ultimately resulting in an extreme strength increase well above predicted load. ES12, however, which experienced a high ratio of tension-to-shear load, performed identically to its F1554 counterpart, supporting that for all materials, there is a critical angle above which material type does not factor into behavior.

- **GROUT PAD STRENGTH.** Torsion test T28 contained a low-strength, thin annular mortar layer to compare against equivalent tests with high-strength non-shrink grout. While the test failed at a lower load, before failure the behavior was nearly identical to that in high-strength grout tests. As a result, it is concluded that beyond the compressive strength being sufficient to withstand clamping and compressive loads, the strength of a cementitious layer between a base plate and a concrete surface has no impact on the steel strength of anchor bolts.

- **ABSENCE OF LEVELING NUT IN GROUTED CONNECTIONS.** Torsion test T26 failed at a significantly higher load than leveling nut and shim stack counterparts. It is believed that this came as a result of cross-sectional rotation enabled by the lack of the leveling nut and further enabled by the presence of a self-aligning spherical washer below the tightening nut. This result suggests that the existing body of experimental work on bolt strength in grouted connections, which has all been conducted without leveling nuts, may not be representative of the condition with a leveling nut.

### 6.6 Application of Analytical Models to Multiple-bolt Connections

Ungrouted Eccentric Shear tests ES21 and ES22 failed at conservative and comparable loads to the predictive model, in which it was assumed that the neutral axis of base plate bending was situated at the midpoint between tension and compression bolts. Furthermore, ES21 was directly comparable to two-bolt compression and shear test ES15, which contained the same ratio of applied compression to shear in the critical bolts. ES21 marginally outperformed ES15, suggesting that the centered neutral axis assumption is conservative and appropriate. Based on
the conservative ungrouted test results and engineering judgment, it is recommended that forces in multiple-bolt ungrouted stand-off base plate connections be distributed with normal and shear connection forces distributed equally between bolts and overturning moments distributed with a neutral axis through the centroid of the bolt group. Applying the resultant shear and normal forces on each bolt, the design should be governed by the critical bolt, e.g., that with the highest normalized combination of forces determined from applying the appropriate cases from the ungrouted model presented in 6.3.3.3.

Grouted Eccentric Shear test ES31 and ES32 performed in line with force distribution assumptions of a rigid base plate as discussed in Section 4.6.5. Along with the result from the Torsion test with mortar, it is concluded that grout strength does not affect force distribution to bolts and that the forces may be found in the same way as in flush-mounted base plate connections. The reader is referred to Cook and Klingner (1992) for determining forces on individual bolts. As for bolts in ungrouted connections, anchor bolt design should be governed by the critical bolt, applying the applicable cases from 6.3.4.1.

6.7 Summary

This chapter provided the development analytical models for the strength of anchor bolts in ungrouted and grouted stand-off base plate connections considering the interaction of shear force, normal force, bending moments, and, in the grouted model, interplay with friction and clamping forces from grout. Experimental results from Chapter 5 were found to be in good agreement with the analytical models for all tests, including in four-bolt Eccentric Shear tests where individual bolts experienced different combinations of applied normal and shear forces. Beyond evaluation of data against the models, observations of the behavior of the impact of minor variables were summarized and recommended applications of the analytical models to multiple-bolt connections was described. The results of the analysis lead to several conclusions
regarding determination of anchor bolt steel strength in ungrouted and grouted stand-off base plate connections.
CHAPTER 7
SUMMARY, CONCLUSIONS, AND RECOMMENDATIONS

7.1 Dissertation Summary

Anchor bolts in stand-off base plates connecting structures to concrete experience combinations of normal and shear forces. The shear forces act over the exposed length of the bolts, producing bending moments in addition to the shear stresses on the cross-section. In existing code standards, anchor bolt steel strength in ungrouted stand-off base plates is either not accounted for explicitly in AISC (2010) and ACI (2011) or they are designed using assumptions that are unconservative for certain loading conditions in AASHTO (2013b). Grout contributions to anchor bolt strength are not considered in the design of double-nut connections in AASHTO (2013b), left to the user to account anchor bolt bending in AISC (2010), or accounted for with a simple reduction factor in ACI (2011).

This dissertation studied the steel strength and behavior of anchor bolts in ungrouted and grouted stand-off base plate connections to concrete with emphasis on double-nut connections containing leveling nuts below the base plate. An experimental program was undertaken consisting of four experimental testing arrangements to study anchor bolt steel strength in stand-off base plate connections: Direct Shear, Torsion, Large-scale, and Eccentric Shear.

Direct Shear methodology established statistical relationships between stand-off distance and ultimate steel shear strength. Influences from installation method (cast-in-place vs. adhesive anchors) and number of bolts (single- vs. double-bolt connections) on bolt steel strength in stand-off base plates were also investigated. Statistically significant differences were found between anchor bolts at all levels of stand-off distance, including within the permissible range of up to one bolt diameter of exposed length by AASHTO (2013b) for ignoring strength reductions caused by bolt bending. Adhesive anchors were found to be statistically stronger than cast-in-
place anchors, likely due to the decreased relative bending length from the adhesive restraining bolt bending at the concrete surface. Minor differences in the behavior, but not the strength, of double-bolt vs. single-bolt tests attributable to a roller used to restrain base plate rotation in single-bolt but not double-bolt tests were observed. This behavioral difference did not translate to significant differences in ultimate strength. Both single- and double-bolt tests, however, did not contain pure shear load throughout the second-order displacement behavior of tests, initiating the development of the Torsion experimental arrangement.

Torsion testing consisted of evaluation of ungrouted and grouted circular groups of 5/8 in. and 1 in. diameter anchor bolts. Exposed length, influence of grout, concrete strength, bolt pretensioning, size effect, the presence of a fiber-reinforced polymer (FRP) wrap, the effect of a “worst-case” dry-packed mortar installation, shim stacks as a leveling implement, material ductility, and the absence of nuts in a grouted connection were all investigated for their influence on strength and behavior. The strengths of flush-mounted torsion tests were 9.5% lower than direct shear counterparts, which was attributed to anchor bolt placement within the oversize holes in Torsion testing. Grouted tests produced higher ultimate shear strengths than the ACI (2011)-specified ultimate load value at considerable displacements after grout cracking and subsequent clamping behavior. The influence of minor test variables is discussed at the end of Chapter 6.

Four Large-scale tests containing circular groups of six 1.25 in. diameter bolts were conducted, three with grout pads installed and one left ungrouted. Two tests were conducted at the AASHTO (2013b) maximum permissible leveling nut stand-off distance (exposed length) of one bolt diameter between the concrete surface and the bottom of the leveling nut, one ungrouted (LS1) and one grouted (LS2). The final two tests contained a more extreme exposed length of
three bolt diameters. The effects of a circular vs. annular grouted base plate were studied by removing a concentric circle from the circular base plate (used in LS1-LS3) in the final test (LS4). Similar reduction in the strength was seen in the ungrouted test LS1 to Direct Shear and Torsion results. Grouted test behavior and strength were consistent with Torsion tests, even in the annular base plate (all Torsion tests contained a circular base plate). It was determined that results from both experimental arrangements are valid representations of field annular base plate connections.

Eccentric shear testing investigated the influence of the ratio of applied normal force to shear force on both ungrouted and grouted stand-off base plate connections at varying exposed lengths. In addition to the load ratio, limited tests were conducted studying minor experimental values including bolt pretensioning, material ductility, and two- vs. four-bolt arrangements. Based on the results from this testing in conjunction with the Torsion and Large-scale test arrangements applying pure shear to individual bolts, it was determined that distinct categories of resistance to applied load emerge from combinations of loading angle, material ductility, and, where present, systemic friction. Again, discussion of minor variables is given at the end of Chapter 6.

Expanding on the observations from experimental testing, mathematical relationships were given to categorize different “Cases” for determining the strength of anchor bolts in ungrouted and grouted stand-off base plate connections. Separately, upper-limit and lower-bound failure criteria based on the interactions between applied shear force, normal force, and bending moments were derived for circular sections. The different Case treatments were combined with the failure criteria to present analytical models. It was found that both ungrouted and grouted stand-off base plate experimental results matched the data very well with an
assumption that bending moments occur over a depth equal to the original exposed length of the anchor bolt with an additional half-diameter extension below the concrete surface to account for concrete spalling. Four-bolt tests were found to align well with simplifying assumptions about the distribution of shear and normal forces to individual bolts, leading to conclusions and recommendations for the determination of anchor bolt strength in stand-off base plate connections.

7.2 Conclusions

Observations of the in-test behavior and measurement of data against analytical models in this dissertation generate strong evidence for the effects of various parameters on the steel strength of anchor bolts. The following conclusions are made:

- Steel strength of anchor bolts in ungrouted stand-off base plate connections is significantly better described using three analytical cases with interaction equations (6-13) and (6-14) between bending moment, shear, and tension than existing code- and literature-specified analytical models. The ratios of experimental values to predicted values using upper limit and lower-bound methods were 1.04 and 1.08, respectively, compared to fib (2011) methodology at 2.28. The three analytical cases are 1) first-order bending moment with shear and/or compressive force for pure shear and compression/shear applied loads, 2) first and second-order bending moment combined with shear and tensile forces for tensile loading angles greater than a characteristic angle corresponding to material ductility, and 3) shear and tensile forces with no bending moment considered for tensile loading angles less than the characteristic angle. Due to the symmetry of relationships between normal forces/bending moments and shear forces/bending moments, the marginally more conservative (6-14) may be more easily implemented in practical use, as recommended in the next section at minor consequence to predictive integrity.

- The characteristic angle of bolt rotation at failure for F1554 Gr. 55 material was found to be approximately 35 degrees from the initial position of the bolt in both ungrouted and grouted tests.

- Steel strength of anchor bolts in grouted stand-off base plate connections is significantly better described using equations (6-28), (6-33), and (6-38) (with exposed length taken to directly below the leveling nut) for three analytical cases than existing code- and literature-specified analytical models. The ratios of experimental values to predicted values using the proposed methodology was 1.05 compared to Gresnigt et al. (2008) methodology at 1.52. Using the ACI interaction equation and shear capacity reduction factor (7-4) produced a ratio of 1.32. The proposed equations account for the interplay between the interaction of shear and tension forces and grout frictional resistance from applied connection
compression and/or second-order clamping action from the grout’s resistance to anchor bolt downward movement. Based on the interplay between material ductility, angle of applied load, and systemic friction coefficient, the three analytical cases are: 1) shear force resistance from grout friction alone after grout cracking, 2) shear force resistance from the combination of anchor bolt shear and friction, and 3) shear force resistance from anchor bolt steel alone.

- The most accurate fit to data for both ungrouted and grouted analyses using the conditions above is found using the plastic section modulus corresponding to the average of the gross and minimum root bolt diameters for bending and the net tensile area for shear and tensile capacity.

- The most accurate fit to data for the steel strength of bolts with both ungrouted and grouted stand-off base plates implements an exposed length equal to the undisplaced exposed length of a bolt plus a half-diameter length below the concrete surface (to account for spalling). This finding matches standard industry assumptions.

- All values of anchor bolt exposed length in ungrouted double-nut connections result in reduced steel shear strength of the anchor bolts. The AASHTO (2013b) allowance for ignoring bolt bending for exposed lengths less than one bolt diameter is thus an unconservative assumption for predicting ultimate shear strength strength in ungrouted stand-off base plates with bolt exposed lengths less than one diameter. The beam bending model given by AASHTO (2013b) suggested for greater values of exposed length is appropriate, but assumptions about the superposition of normal stresses from normal forces and from bending moments is plastically inadmissible for the determination of strength.

- Two ungrouted tests of four-bolt connections matched predictions assuming that 1) shear, torsion, and normal forces were distributed equally between bolts in the connection and that 2) connection overturning bending moments were distributed about a neutral axis perpendicular to the overturning moment and traveling through the bolt group centroid.

- Two grouted tests of four-bolt connections matched predictions assuming that failure is governed by a critical bolt as in flush-mounted base plate connections with failure in the tension bolts.

- The AASHTO (2013a) group connection reduction factor of 0.8 is appropriate for ungrouted flush-mounted and stand-off base plate tests to account for unequal distribution of displacement to individual bolts. In stand-off base plate connections, however, systemic displacements will be distributed across the exposed lengths of the anchor bolts in the connection for equal load sharing between bolts. As bolts move into plastic ranges, forces are distributed favorably corresponding to the high ductility of afforded by the exposed length.

- Properly installed nonshrink grout pads significantly increase the steel shear capacity of anchor bolts in stand-off base plates, albeit at high levels of connection deformation. The ACI (2011) anchor bolt shear strength reduction factor of 0.8 for anchor bolt steel shear strength in grouted stand-off base plates compared to flush-mounted base plates is
conservative. Additionally, applying the shear vs. normal force interaction equation from ACI (2011) to grouted connections with the denominator in the shear term reduced by the 0.8 factor provides conservative and relatively accurate results. Existing AASHTO (2013b) specifications stating that a grout pad shall not be factored in the strength of the connection may be overly conservative.

- Beyond possessing the necessary compressive strength to withstand the combination of connection compression loads and clamping forces, bolt strength is independent of the strength of grout/mortar material. Nonetheless, a properly installed non-shrink grout is recommended as discussed in the next section.

- Adhesive anchors are at least as strong and stiff as cast-in-place anchors in stand-off base plates under short-duration static shear loading in stand-off base plate connections.

- While the absence of pretensioning in a double-nut connection allows slip at low levels of load, ultimate loads are not affected.

- An adequately designed FRP wrap around both the grout pad and the base plate perimeter restricts grout pad crack propagation, returning the steel shear strength of anchor bolts to at least the flush-mounted shear strength.

- Anchor bolts in connections with shim stacks and other intermediate layers at least as stiff as a low-strength cementitious material behaves equivalently to those in grouted stand-off base plate connections.

- Anchor bolt strength in connections to concrete is independent of a practical range of concrete foundation compressive strengths.

7.3 Design Recommendations

Springing from the conclusions presented in the previous section, several recommendations are made for practical use. Recommendations are broken into two categories: 1) steel strength of bolts in ungrouted stand-off base plate connections and 2) steel strength of bolts in grouted stand-off base plate connections. These recommendations present a simplified methodology to that used for the analytical models presented in the dissertation.

7.3.1 Recommendations for Steel Strength Design of Anchor Bolts in Ungrounded Stand-off Base Plate Connections

Figure 7-1 displays pertinent variables for determining the steel strength of anchor bolts in ungrouted stand-off base plates in design. Shear forces for individual bolts, $V$, in multiple-bolt
ungrouted stand-off base plate connections can be determined by 1) superimposing equally
distributed connection shear forces between individual bolts in the direction of the connection
load with resolved shear forces from torsion about the group centroid or 2) resolving shear forces
about the center of rotation. Normal forces in individual bolts, \( N \), can be determined by
superimposing equally distributed connection normal forces with connection bending moments
distributed linearly relative to their distance from the bolt group centroid.

![Figure 7-1. Anchor bolt in ungrouted stand-off base plate connection with applied normal and shear force.](image)

Knowing the shear and normal forces acting on each bolt in the connection, to determine
the design of the bolts for steel strength, every bolt should be evaluated against the three-variable
interaction equation between shear forces, normal forces, and bending moments as given in
(7-1). All of the bolts should be equally designed such that the maximum interaction equation for
all of the bolts (with safety factors applied) is less than 1.0.

\[
\left( \frac{N}{N_o} \right)^2 + \left( \frac{V}{V_o} \right)^2 + \left( \frac{M}{M_o} \right) \leq 1
\]

(7-1)

where \( N \) = normal force acting on section
\( V \) = shear force acting on section
\[ M = \text{bending moment acting on section} = \frac{V_{\text{eff}}}{2} \]
\[ I_{\text{eff}} = \text{effective exposed length of anchor bolt (see Figure 7-1)} \]
\[ N_o = \text{ultimate uniaxial normal capacity of circular section} = \pi r^2 \sigma_o \]
\[ V_o = \text{ultimate shear capacity of circular section} = a N_o \]
\[ M_o = \text{ultimate moment capacity of circular section} = \frac{4r^3 \sigma_o}{3} \]
\[ A_{\text{nt}} = \text{net tensile area (for threaded rod applications)} \]
\[ r = \text{radius corresponding to net tensile area} \]
\[ Z = \text{plastic section modulus of circular section} \]
\[ \sigma_o = \text{ultimate normal stress capacity of bolt material} \]

For connections with only tensile and shear forces on bolts with equivalent or higher ductility than F1554 grade 55 material, if it can be shown that 1) the tensile force will always accompany shear forces and 2) that the magnitude of the tensile force is greater than three times the shear force, the bolt may be designed for a simple combination of tension and shear as shown in (6-13).

\[
\left( \frac{N}{N_o} \right)^2 + \left( \frac{V}{V_o} \right)^2 \leq 1 \tag{7-2}
\]

If a more precise solution than (7-1) is desired, the reader may use (7-3) in its place. This equation provided a 4% better overall fit to experimental data in this dissertation, which may be insignificant compared to the additional effort required. Additionally, especially for bending-dominant situations, it is permissible to use a value of \( Z \) calculated using the average of the gross and minimum root bolt radii as opposed to the radius of the bolt associated with net tensile area.

\[
\left( \frac{N}{N_o} \right)^2 + \left( \frac{V}{V_o} \right)^2 = 1 \tag{7-3}
\]
7.3.2 Recommendations for Steel Strength Design of Anchor Bolts in Grouted Stand-off Base Plate Connections

7.3.2.1 Simple method

Figure 7-2 displays pertinent variables for determining the steel strength of anchor bolts in grouted stand-off base plates in design. Shear forces for individual bolts, $V$, and normal forces in individual bolts, $N$, should be determined as in standard practice for flush-mounted applications. It is assumed in the following equations that a properly installed high-strength (greater than 5000 psi), non-shrink, fluid grout is installed around anchor bolts filling the entire void between the base plate and the concrete surface.

![Diagram of anchor bolt in grouted stand-off base plate connection with applied normal and shear force.](image)

If any of the bolts in a group experience combined tensile and shear forces or pure shear forces, it is appropriate to design all bolts in the connection to the bolt diameter required to satisfy (7-4) for the worst-case bolt. The absence of a bending moment term in this equation results from the second-order balancing of moments by tension development in the anchor bolts.
from either/both applied tension force and second-order tension from the restraint of downward bolt movement acting over the horizontal displacement of the bolt.

\[
\left( \frac{N_A}{N_o} \right)^{5/3} + \left( \frac{V_A}{0.8V_o} \right)^{5/3} \leq 1
\]  

(7-4)

where  
- \( N_A \) = normal force acting on section  
- \( V_A \) = shear force acting on section  
- \( M \) = bending moment acting on section  
- \( N_o \) = ultimate uniaxial normal capacity of circular section = \( \pi r^2 \sigma_o \)  
- \( V_o \) = ultimate shear capacity of circular section = \( \alpha N_o \)  
- \( M_o \) = ultimate moment capacity of circular section = \( Z \sigma_o \)  
- \( A_{nt} \) = net tensile area (for threaded rod applications)  
- \( r \) = radius corresponding to net tensile area  
- \( Z \) = plastic section modulus of circular section  
- \( \sigma_o \) = ultimate normal stress capacity of bolt material  
- \( \alpha \) = ratio of shear to normal capacity of steel = 0.6

7.3.2.2 Detailed method

Equation (7-4) is a conservative solution stemming from existing specifications in ACI 318-11. In situations where a more precise solution is desired by the designer, particularly when all bolts will experience only compression forces, the methodology developed in Section 6.3.3.3 for four analytical cases is recommended. The author has chosen to express the equations solving for shear acting on an individual bolt, \( V_A \), expressed in terms of the ratio of applied normal to shear force and friction coefficient of the connection. The distinction between different types of analyses (cases) depends on the ratio of ultimate displacement, \( \delta_h \), to effective exposed length, \( l_{eff} \), of the bolt, which relate to the characteristic angle, \( \phi_{char} \), of the material as given in (7-5). In the derivations of the subsequently presented equations, the applied normal force \( N_A \) has been expressed as a ratio of the applied shear force, \( \rho_{nv} \) such that (7-6) is satisfied, which allowed a closed-form solution to each of the analytical cases provided below.
\[
\frac{\delta_h}{l_{eff}} = \tan(\phi_{char}) \quad (7-5)
\]

\[
\rho_{nv} = \frac{N_A}{V_A} \quad (7-6)
\]

where

- \(\delta_h\) = ultimate horizontal displacement of a bolt for a given length
- \(l_{eff}\) = effective exposed length of anchor bolt
- \(\phi_{char}\) = characteristic angle of the bolt specific to material
- \(\rho_{nv}\) = ratio of applied normal to shear force

The majority of the tests were performed with F1554 Gr. 55 material meeting the ductile steel element requirements of ACI 318. For F1554 Gr. 55 bolt material, \(\phi_{char}\) was found in this dissertation to be approximately 35 degrees; brittle steel materials would need to be tested to determine this value to implement this methodology. For the purposes of implementation into design, \(\phi_{char}\) should be conservatively taken as 27 degrees \((\frac{l_{eff}}{\delta_h} = 2.0)\).

The four cases for steel strength of anchor bolts in grouted stand-off base plate connections and their conditions are provided below. Each case is accompanied by a short description of its function along with the criteria and equations for implementation. It should be noted that for any of these cases to occur, bolt deformation causing grout cracking and initial yielding of the bolt is necessary; thus, the equations represent scenarios that may encounter large connection displacements and rotations. Nonetheless, they provide realistic and rational formulations of the ultimate steel strength of bolts in grouted stand-off base plate connections.

In order to use the subsequently presented equations for design, the following steps shall be taken for each combination of applied normal and shear force:

1. Ensure that the bolt material used is is a ductile steel element as defined in ACI 318.
2. Determine the design applied normal force, \(N_A\), and shear force, \(V_A\), for each bolt in the connection to produce the ratio of the two, \(\rho_{nv}\) by (7-6).
3. Using $\frac{l_{eff}}{\delta_h} = 2.0$, the value of $\rho_{nv}$ from 2., and $\mu$ (typically taken as 0.4), determine the applicable design case below.

4. Satisfy the equation for the Case for all loading combinations, designing all bolts in the connection to the diameter of the largest bolt produced.

**Case 1: Shear resisted by friction alone after bolt yield.** This case applies when a brittle steel is used and is conservative for typical ductile anchor materials. The shear resistance provided is a result of friction introduced by clamping action on the grout pad as the anchor displaces laterally but is restrained from vertical movement by the grout pad thus introducing a clamping force equal to the tensile capacity of the anchor. Case 1 is analogous to the ACI 318-11 shear friction provisions.

$$ if \quad \frac{\delta_h}{l_{eff}} < \frac{\mu}{1 + \mu \rho_{nv}} $$

$$ V_A \leq \frac{\mu N_o}{1 + \mu \rho_{nv}} \quad (7-7) $$

**Case 2: Shear resisted by bolt shear alone.** This case arises when the angle of tensile loading does not allow full bending deformation to occur and is analogous to (7-2) for ungrouted connections with a rearrangement of terms.

$$ if \quad \rho_{nv} > \frac{l_{eff}}{\delta_h} $$

$$ V_A \leq \frac{N_o}{\sqrt{\rho_{nv}^2 + \left(\frac{1}{\alpha}\right)^2}} \quad (7-8) $$

**Case 3: Applied resisted by both friction and bolt shear.** This case is valid for all other scenarios and represents shear resistance by a combination of bolt steel and interfacial friction forces.

$$ if \quad \rho_{nv} < \frac{l_{eff}}{\delta_h} \quad and \quad \frac{\delta_h}{l_{eff}} > \frac{\mu}{1 + \mu \rho_{nv}} $$
\[ V_A \leq \frac{N_o}{\left( \frac{l_{\text{eff}}}{\delta_h} \right)^2 + \left( 1 - \mu \left( \frac{l_{\text{eff}}}{\delta_h} - \rho_{nv} \right) \right)^2} \]  

(7-9)

**Case 4: No shear force experienced by bolt.** Where the friction force produced from a compressive load is greater than the applied shear load, which will necessarily be the case when \( \mu \rho_{nv} < -1 \), the bolt will never experience any forces. In this case, the shear strength will simply be \( \mu N_A \) with failure dictated by the grout material strength by existing design methods and assumptions.

7.3.3 **General recommendations for steel strength design of anchor bolts in stand-off base plates**

7.4 **Recommendations for Future Work**

Several possible directions of further study emerge from the results of the work in this dissertation. The following recommendations are made for future work related to the steel strength of anchor bolts in stand-off base plate connections:

- Further testing of FRP retrofits on grouted base plate connections investigating the effects of strength, stiffness, and crack mitigation of grouted stand-off base plate connections
- Cyclic loading with a range applied loading angles and exposed lengths for ungrouted stand-off base plate connections to determine seismic and fatigue performance
- Investigation of the influence of base plate flexibility on the strength and performance of ungrouted stand-off base plate connections
- Experimental or finite element investigation in anchor bolts in various common ungrouted stand-off base plate connections to evaluate design assumptions about the distribution of forces to individual anchor bolts
APPENDIX A
LOAD-DISPLACEMENT BEHAVIOR OF DIRECT SHEAR TESTS

This appendix contains the load-displacement behavior of Direct Shear tests. Tension load cell readings, while not necessarily representative of actual tensile stresses in the bolts, are also provided for 5/8 in. cast-in-place datasets DS1 through DS6. 1 in. bolt tests are presented together and adhesive anchor tests are superimposed with comparable datasets.
Figure A-1. DS1 (flush-mounted base plate, 5/8 in. bolts, n = 1) applied load and bolt tension vs. displacement.

Figure A-2. DS2 (ungrouted 1.2 dia. stand-off base plate, 5/8 in. bolts, n = 1) applied load and bolt tension vs. displacement.
Figure A-3. DS3 (ungrouted 1.6 dia. stand-off base plate, 5/8 in. bolts, n = 1) applied load and bolt tension vs. displacement.

Figure A-4. DS4 (ungrouted 2 dia. stand-off base plate, 5/8 in. bolts, n = 1) applied load and bolt tension vs. displacement.
Figure A-5. DS6 (ungrouted 4 dia. stand-off base plate, 5/8 in. bolts, n = 1) applied load and bolt tension vs. displacement.

Figure A-6. DS7 (flush-mounted base plate, 5/8 in. bolts, n = 2) instrumentation schematic.
Figure A-7. DS8 (ungrouted 2 dia. stand-off base plate, 5/8 in. bolts, n = 2) applied load and bolt tension vs. displacement.

Figure A-8. DS9, DS10, and DS11 (flush-mounted, ungrouted 2 dia. stand-off base plate, and 4 dia. ungrouted stand-off base plate, 1 in. bolts, n = 1) applied load and bolt tension vs. displacement.
Figure A-9. DS8 (flush-mounted base plate, 5/8 in. bolts, n = 1, adhesive installation) applied load vs. displacement with comparable datasets.

Figure A-10. DS13 (ungrouted 2 dia. stand-off base plate, 5/8 in. bolts, n = 1, adhesive installation) applied load vs. displacement with comparable datasets.
Figure A-11. DS14 (ungrouted 2 dia. stand-off base plate, 5/8 in. bolts, n = 1, adhesive installation) with DS6 CIP applied load vs. displacement.
APPENDIX B
LOAD-DISPLACEMENT BEHAVIOR OF TORSION TESTS

This appendix contains the locations of instrumentation and individual actuator loads producing torsion on the bolt group vs. bolt displacement for every Torsion test. Bolt tension vs. bolt displacement graphs for every bolt tension load cell and average applied actuator load vs. displacement of every vertically oriented LVDT are provided for T1 through T13. Due to initial improper calibration of actuator load cells, the first tests conducted (T1-A, T2-A, and T3) show imbalanced applied loads between the two actuators to varying degrees. The values presented represent corrected values after load cell recalibration; failure loads for these tests are the average of the two actuator readings at ultimate load. Despite many calibrations, tension load cells proved to be unreliable in providing consistent and reasonable readings between and within tests. However, their data are valuable for characterizing individual bolt behavior within tests. For this reason, readings are represented as values normalized by the maximum reading of all bolts within an individual test. Vertical displacement readings were not taken for T2-A. In some cases, vertical LVDTs may have been disturbed or fallen off of an edge (e.g. V3 in T1-A and V1 in T6-B). However, this was rare.

Table B-1 provides ultimate displacement values from vertically oriented LVDTs, with positive values indicating downward base plate movement. Readings were not taken for T2-A. Values were negligible in flush-mounted and ungrouted $2d_b$ tests, while ungrouted $4d_b$ tests showed much greater vertical displacement. In general, grouted vertical displacements were negligible, but slight upward movement was observed in $2d_b$ test T5. Load vs. vertical displacement graphs are provided for every test in Appendix B.
Table B-1. Vertical base plate displacements in Torsion tests.

<table>
<thead>
<tr>
<th>Test</th>
<th>V1</th>
<th>V2</th>
<th>V3</th>
<th>V4</th>
<th>Average</th>
</tr>
</thead>
<tbody>
<tr>
<td>T1</td>
<td>-0.011</td>
<td>0.000</td>
<td>0.035</td>
<td>0.001</td>
<td>0.006</td>
</tr>
<tr>
<td>T2</td>
<td>0.007</td>
<td>-0.036</td>
<td>0.013</td>
<td>-0.024</td>
<td>-0.005</td>
</tr>
<tr>
<td>T3</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>T4</td>
<td>0.038</td>
<td>-0.015</td>
<td>0.016</td>
<td>-0.012</td>
<td>0.007</td>
</tr>
<tr>
<td>T5</td>
<td>0.083</td>
<td>0.081</td>
<td>0.119</td>
<td>0.058</td>
<td>0.085</td>
</tr>
<tr>
<td>T6</td>
<td>-0.007</td>
<td>0.008</td>
<td>-0.014</td>
<td>0.001</td>
<td>-0.003</td>
</tr>
<tr>
<td>T7</td>
<td>-0.031</td>
<td>-0.079</td>
<td>-0.054</td>
<td>-0.054</td>
<td>-0.054</td>
</tr>
<tr>
<td>T8</td>
<td>0.008</td>
<td>-0.008</td>
<td>0.031</td>
<td>0.008</td>
<td>0.014</td>
</tr>
<tr>
<td>T9</td>
<td>0.042</td>
<td>-0.004</td>
<td>0.009</td>
<td>0.033</td>
<td>0.020</td>
</tr>
<tr>
<td>T10</td>
<td>-0.029</td>
<td>-0.008</td>
<td>-0.036</td>
<td>-0.013</td>
<td>-0.021</td>
</tr>
<tr>
<td>T11</td>
<td>-0.017</td>
<td>-0.013</td>
<td>0.012</td>
<td>na</td>
<td>-0.006</td>
</tr>
<tr>
<td>T12</td>
<td>0.006</td>
<td>-0.009</td>
<td>0.022</td>
<td>na</td>
<td>0.006</td>
</tr>
<tr>
<td>T13</td>
<td>0.266</td>
<td>0.271</td>
<td>0.378</td>
<td>na</td>
<td>0.305</td>
</tr>
</tbody>
</table>

*na = not applicable*
Figure B-1. T1 (flush-mounted base plate, 5/8 in. bolts, n = 6) instrumentation schematic.

Figure B-2. T1 individual actuator loads vs. bolt displacement.
Figure B-3. T1 bolt tensions vs. bolt displacement.

Figure B-4. T1 average applied load vs. vertical displacements.
Figure B-5. T2 (flush-mounted base plate, 5/8 in. bolts, n = 6) instrumentation schematic.

Figure B-6. T2 individual actuator loads vs. bolt displacement.
Figure B-7. T2 bolt tensions vs. bolt displacement.

Figure B-8. T2 average applied load vs. vertical displacements.
Figure B-9. T3 (ungrouted 2 dia. stand-off base plate, 5/8 in. bolts, n = 6) instrumentation schematic.

Figure B-10. T3 individual actuator loads vs. bolt displacement.
Figure B-11. T3 bolt tensions vs. bolt displacement.

Note: Vertical displacement not recorded for test T2-A.
Figure B-12. T4 (ungrouted 2 dia. stand-off base plate, 5/8 in. bolts, n = 6) instrumentation schematic.

Figure B-13. T2-B individual actuator loads vs. bolt displacement.
Figure B-14. T4 bolt tensions vs. bolt displacement.

Figure B-15. T4 average applied load vs. vertical displacements.
Figure B-16. T6 (ungrouted 4 dia. stand-off base plate, 5/8 in. bolts, n = 6) instrumentation schematic.

Figure B-17. T6 individual actuator loads vs. bolt displacement.
Figure B-18. T6 bolt tensions vs. bolt displacement.

Figure B-19. T6 average applied load vs. vertical displacements.
Figure B-20. T5 (ungrouted 2 dia. stand-off base plate, 5/8 in. bolts, n = 6, no pretension) instrumentation schematic.

Figure B-21. T5 individual actuator loads vs. bolt displacement.
Figure B-22. T5 bolt tensions vs. bolt displacement.

Figure B-23. T5 average applied load vs. vertical displacements.
Figure B-24. T7 (grouted 2 dia. stand-off base plate, 5/8 in. bolts, n = 6) instrumentation schematic.

Figure B-25. T7 individual actuator loads vs. bolt displacement.
Figure B-26. T7 bolt tensions vs. bolt displacement.

Figure B-27. T7 average applied load vs. vertical displacements.
Figure B-28. T8 (grouted 4 dia. stand-off base plate, 5/8 in. bolts, n = 6) instrumentation schematic.

Figure B-29. T8 individual actuator loads vs. bolt displacement.
Figure B-30. T8 bolt tensions vs. bolt displacement.

Figure B-31. T8 average applied load vs. vertical displacements.
Figure B-32. T9 (grouted 4 dia. stand-off base plate, 5/8 in. bolts, n = 6) instrumentation schematic.

Figure B-33. T9 individual actuator loads vs. bolt displacement.
Figure B-34. T9 bolt tensions vs. bolt displacement.

Figure B-35. T9 average applied load vs. vertical displacements.
Figure B-36. T10 (grouted 4 dia. stand-off base plate with FRP wrap, 5/8 in. bolts, n = 6) instrumentation schematic.

Figure B-37. T10 individual actuator loads vs. bolt displacement.
Figure B-38. T10 bolt tensions vs. bolt displacement.

Figure B-39. T10 average applied load vs. vertical displacements.
Figure B-40. T11 (flush-mounted base plate, 1 in. bolts, n = 3) instrumentation schematic.

Figure B-41. T11 individual actuator loads vs. bolt displacement.
Figure B-42. T11 bolt tensions vs. bolt displacement.

Figure B-43. T11 average applied load vs. vertical displacements.
Figure B-44. T12 (ungrooved 2 dia. stand-off base plate, 1 in. bolts, n = 3) instrumentation schematic.

Figure B-45. T12 individual actuator loads vs. bolt displacement.
Figure B-46. T12 bolt tensions vs. bolt displacement.

Figure B-47. T12 average applied load vs. vertical displacements.
Figure B-48. T13 (ungrooved 4 dia. stand-off base plate, 1 in. bolts, n = 3) instrumentation schematic.

Figure B-49. T13 individual actuator loads vs. bolt displacement.
Figure B-50. T13 bolt tensions vs. bolt displacement.

Figure B-51. T13 average applied load vs. vertical displacements.
Figure B-52. T14 (ungrouted, 0 dia. exposed length, 0.625 in. F1554 Gr. 55 bolts, n = 3) schematic.

Figure B-53. T14 individual actuator loads vs. bolt displacement.
Figure B-54. T15 (ungrouted, 0.5 dia. exposed length, 0.625 in. F1554 Gr. 55 bolts, n = 3) schematic.

Figure B-55. T15 individual actuator loads vs. bolt displacement.
Figure B-56. T16 (ungrounded, 2.8 dia. exposed length, 0.625 in. F1554 Gr. 55 bolts, n = 3) schematic

Figure B-57. T16 individual actuator loads vs. bolt displacement.
Figure B-58. T17 (ungрутовой, 6 диа. экпозиция длины, 0.625 в. F1554 Gr. 55 болты, n = 3) схематичный.

Figure B-59. T17 индивидуальные аккумуляторные нагрузки vs. болтовый смещение.

309
Figure B-60. T18 (low-strength concrete, ungrouted, 1 dia. exposed length, 0.625 in. F1554 Gr. 55 bolts, n = 3) schematic.

Figure B-61. T18 individual actuator loads vs. bolt displacement.
Figure B-62. T19 (low-strength concrete, ungrouted, 1 dia. exposed length, 0.625 in. F1554 Gr. 55 bolts, n = 3) schematic.

Figure B-63. T19 individual actuator loads vs. bolt displacement.
Figure B-64. T20 (low-strength concrete, ungrouted, 3 dia. exposed length, 0.625 in. F1554 Gr. 55 bolts, n = 3) schematic.

Figure B-65. T20 individual actuator loads vs. bolt displacement.
Figure B-66. T21 (low-strength concrete, ungrouted, 6 dia. exposed length, 0.625 in. F1554 Gr. 55 bolts, n = 3) schematic.

Figure B-67. T21 individual actuator loads vs. bolt displacement.
Figure B-68. T22 (ungrounded, 1 dia. exposed length, 0.625 in. 354 BD bolts, n = 3) schematic.

Figure B-69. T22 individual actuator loads vs. bolt displacement.
Figure B-70. T23 (ungrouted, 3 dia. exposed length, 0.625 in. 354 BD bolts, n = 3) schematic.

Figure B-71. T23 individual actuator loads vs. bolt displacement.
Figure B-72. T24 (ungрутов, 1 dia. exposed length, 0.625 in. 316 stainless bolts, n = 3) schematic.

Figure B-73. T24 individual actuator loads vs. bolt displacement.
Figure B-74. T25 (grouted, 3 dia. exposed length, 0.625 in. F1554 Gr. 55 bolts, n = 3) schematic.

Figure B-75. T25 individual actuator loads vs. bolt displacement.
Figure B-76. T26 (grouted without leveling nuts, 3 dia. exposed length, 0.625 in. F1554 Gr. 55 bolts, n = 3) schematic.

Figure B-77. T26 individual actuator loads vs. bolt displacement
Figure B-78. T27 (shimmed without grout, 3.6 dia. exposed length, 0.625 in. F1554 Gr. 55 bolts, n = 3) schematic.

Figure B-79. T27 individual actuator loads vs. bolt displacement.
Figure B-80. T28 (ungrouted with dry-packed mortar, 4.2 dia. exposed length, 0.625 in. F1554 Gr. 55 bolts, n = 3) schematic.

Figure B-81. T28 individual actuator loads vs. bolt displacement.
APPENDIX C
LOAD-DISPLACEMENT BEHAVIOR OF LARGE-SCALE TESTS

This appendix contains the locations of instrumentation, applied actuator load vs. bolt displacement from string potentiometer readings, bolt tension vs. bolt displacement graphs for every tension load cell, and applied actuator load vs. displacement for every vertically and horizontally oriented LVDT. Test LS1 contained a different vertical LVDT layout than the three subsequent tests; at the end of the test LVDT V1 was run into by the pipe loading apparatus, making its position unviable. Thus, the vertically oriented LVDTs were shifted away from the movement of the pipe in the remaining tests. For the same reasons presented in the Appendix B description, bolt tension readings are represented as values normalized by the maximum reading of all bolts within an individual test.

Displacements corresponding to ultimate load from the three vertically oriented (parallel to anchor bolts) and two horizontally oriented (orthogonal to anchor bolts) LVDTs arranged according to Figure C-1 are provided in Table C-1. Load vs. vertical and horizontal LVDT displacements are provided for every test in Appendix C. Positive values for vertical LVDTs indicate base plate movement toward the concrete at a given location. Positive readings for H1 and H2 indicate upward and rightward base plate movement, respectively. The greatest vertical and horizontal displacements were observed in LS3, which also experienced the greatest overall displacement. In contrast to the Torsion ungrouted stand-off test T2-B, the ungrouted stand-off test LS1 showed relatively large downward displacements, which were restrained in the grouted counterpart LS2.
<table>
<thead>
<tr>
<th>Test</th>
<th>V1</th>
<th>V2</th>
<th>V3</th>
<th>V Average</th>
<th>H1</th>
<th>H2</th>
</tr>
</thead>
<tbody>
<tr>
<td>LS1</td>
<td>0.103</td>
<td>0.182</td>
<td>0.039</td>
<td>0.108</td>
<td>0.279</td>
<td>0.101</td>
</tr>
<tr>
<td>LS2</td>
<td>0.048</td>
<td>-0.015</td>
<td>-0.010</td>
<td>0.008</td>
<td>0.215</td>
<td>-0.138</td>
</tr>
<tr>
<td>LS3</td>
<td>0.342</td>
<td>0.209</td>
<td>0.185</td>
<td>0.245</td>
<td>0.335</td>
<td>0.050</td>
</tr>
<tr>
<td>LS4</td>
<td>0.191</td>
<td>0.069</td>
<td>-0.019</td>
<td>0.080</td>
<td>0.190</td>
<td>0.006</td>
</tr>
</tbody>
</table>

Table C-1. Vertical and horizontal base plate displacements (in.) in Large-scale tests.
Figure C-1. LS1 (ungrouted 2.3 dia. stand-off base plate, 1.25 in. bolts, n = 6) instrumentation schematic.

Figure C-2. LS1 actuator applied load vs. average bolt displacement.
Figure C-3. LS1 bolt tensions vs. bolt displacement.

Figure C-4. LS1 applied load vs. vertical and horizontal displacements.
Figure C-5. LS2 (grouted 2.3 dia. stand-off base plate, 1.25 in. bolts, n = 6) instrumentation schematic.

Figure C-6. LS2 individual actuator loads vs. average bolt displacement.
Figure C-7. LS2 bolt tensions vs. bolt displacement.

Figure C-8. LS2 applied load vs. vertical and horizontal displacements.
Figure C-9. LS3 (grouted 4.3 dia. stand-off base plate, 1.25 in. bolts, n = 6) instrumentation schematic.

Figure C-10. LS3 actuator load vs. average bolt displacement.
Figure C-11. LS3 bolt tensions vs. bolt displacement.

Figure C-12. LS3 applied load vs. vertical and horizontal displacements.
Figure C-13. LS4 (grouted 4.3 dia. stand-off base plate, 1.25 in. bolts, n = 6) instrumentation schematic.

Figure C-14. LS4 actuator load vs. average bolt displacement.
Figure C-15. LS4 bolt tensions vs. bolt displacement.

Figure C-16. LS4 applied load vs. vertical and horizontal displacements.
APPENDIX D
LOAD-DISPLACEMENT BEHAVIOR OF ECCENTRIC SHEAR TESTS

This appendix contains schematics of test setups along with actuator load vs. bolt displacement readings for Eccentric Shear tests. Test schematics include the ungrouted/grouted condition, nominal location of applied load, and type of loading experienced by bolts (i.e. tension/shear, compression/shear, or both). Bolt displacements represent the average of two LVDT readings taken from the back of the base plate and are normalized by the gross bolt diameter.
Figure D-1. ES1 (ungrouted, 0 dia. exposed length, 0.625 in. F1554 Gr. 55 bolts, n = 2, tension/shear, 6 in. nominal eccentricity, not pretensioned) test schematic.

Figure D-2. ES1 actuator load vs. average bolt displacement.
Figure D-3. ES2 (ungrouted, 1 dia. exposed length, 0.625 in. F1554 Gr. 55 bolts, n = 2, tension/shear, 6 in. nominal eccentricity, not pretensioned) test schematic.

Figure D-4. ES2 actuator load vs. average bolt displacement.
Figure D-5. ES3 (ungrouted, 1 dia. exposed length, 0.625 in. F1554 Gr. 55 bolts, n = 2, tension/shear, 6 in. nominal eccentricity, pretensioned) test schematic.

Figure D-6. ES3 actuator load vs. average bolt displacement.
Figure D-7. ES4 (ungrouted, 1 dia. exposed length, 0.625 in. F1554 Gr. 55 bolts, n = 2, tension/shear, 6 in. nominal eccentricity, pretensioned) test schematic.

Figure D-8. ES4 actuator load vs. average bolt displacement.
Figure D-9. ES5 (ungrouted, 3 dia. exposed length, 0.625 in. F1554 Gr. 55 bolts, n = 2, tension/shear, 6 in. nominal eccentricity, not pretensioned) test schematic.

Figure D-10. ES5 actuator load vs. average bolt displacement.
Figure D-11. ES6 (ungrouted, 3 dia. exposed length, 0.625 in. F1554 Gr. 55 bolts, n = 2, tension/shear, 12 in. nominal eccentricity, pretensioned) test schematic.

Figure D-12. ES6 actuator load vs. average bolt displacement.
Figure D-13. ES7 (ungrouted, 1 dia. exposed length, 0.625 in. F1554 Gr. 55 bolts, n = 2, tension/shear, 18 in. nominal eccentricity, pretensioned) test schematic.

Figure D-14. ES7 actuator load vs. average bolt displacement.
Figure D-15. ES8 (ungrounded, 3 dia. exposed length, 0.625 in. F1554 Gr. 55 bolts, n = 2, tension/shear, 18 in. nominal eccentricity, pretensioned) test schematic.

Figure D-16. ES8 actuator load vs. average bolt displacement.
Figure D-17. ES9 (ungrouted, 1 dia. exposed length, 0.625 in. F1554 Gr. 55 bolts, n = 2, tension/shear, 24 in. nominal eccentricity, not pretensioned) test schematic.

Figure D-18. ES9 actuator load vs. average bolt displacement.
Figure D-19. ES10 (ungrouted, 1 dia. exposed length, 0.625 in. F1554 Gr. 55 bolts, n = 2, tension/shear, 24 in. nominal eccentricity, pretensioned) test schematic.

Figure D-20. ES10 actuator load vs. average bolt displacement.
Figure D-21. ES11 (ungrouted, 3 dia. exposed length, 0.625 in. F1554 Gr. 55 bolts, n = 2, tension/shear, 24 in. nominal eccentricity, not pretensioned) test schematic.

Figure D-22. ES11 actuator load vs. average bolt displacement.
Figure D-23. ES12 (ungrounded, 1 dia. exposed length, 0.625 in. 316 Stainless Steel bolts, n = 2, tension/shear, 24 in. nominal eccentricity, pretensioned) test schematic.

Figure D-24. ES12 actuator load vs. average bolt displacement.
Figure D-25. ES13 (ungruned, 1 dia. exposed length, 0.625 in. F1554 Gr. 55 bolts, n = 2, compression/shear, 6 in. nominal eccentricity, not pretensioned) test schematic.

Figure D-26. ES13 actuator load vs. average bolt displacement.
Figure D-27. ES14 (ungrouted, 3 dia. exposed length, 0.625 in. F1554 Gr. 55 bolts, \( n = 2 \), compression/shear, 6 in. nominal eccentricity, not pretensioned) test schematic.

Figure D-28. ES14 actuator load vs. average bolt displacement.
Figure D-29. ES15 (ungrounded, 3 dia. exposed length, 0.625 in. F1554 Gr. 55 bolts, n = 2, compression/shear, 12 in. nominal eccentricity, pretensioned) test schematic.

Figure D-30. ES15 actuator load vs. average bolt displacement.
Figure D-31. ES16 (ungrouted, 3 dia. exposed length, 0.625 in. F1554 Gr. 55 bolts, n = 2, compression/shear, 18 in. nominal eccentricity, pretensioned) test schematic.

Figure D-32. ES16 actuator load vs. average bolt displacement.
Figure D-33. ES17 (ungrouted, 1 dia. exposed length, 0.625 in. F1554 Gr. 55 bolts, n = 2, compression/shear, 24 in. nominal eccentricity, not pretensioned) test schematic.

Figure D-34. ES17 actuator load vs. average bolt displacement (note: error in data acquisition. This space is intentionally left blank.).
Figure D-35. ES18 (ungrouted, 1 dia. exposed length, 0.625 in. F1554 Gr. 55 bolts, n = 2, compression/shear, 24 in. nominal eccentricity, pretensioned) test schematic.

Figure D-36. ES18 actuator load vs. average bolt displacement.
Figure D-37. ES19 (ungrouted, 3 dia. exposed length, 0.625 in. F1554 Gr. 55 bolts, \( n = 2 \), compression/shear, 24 in. nominal eccentricity, not pretensioned) test schematic.

Figure D-38. ES19 actuator load vs. average bolt displacement.
Figure D-39. ES20 (ungrouted, 3 dia. exposed length, 0.625 in. F1554 Gr. 55 bolts, n = 2, compression/shear, 24 in. nominal eccentricity, not pretensioned) test schematic.

Figure D-40. ES20 actuator load vs. average bolt displacement.
Figure D-41. ES21 (ungrouted, 3 dia. exposed length, 0.625 in. F1554 Gr. 55 bolts, n = 4, 6 in. nominal eccentricity, pretensioned) test schematic.

Figure D-42. ES21 actuator load vs. average bolt displacement.
Figure D-43. ES22 (ungrouted, 3 dia. exposed length, 0.625 in. F1554 Gr. 55 bolts, n = 4, 24 in. nominal eccentricity, pretensioned) test schematic.

Figure D-44. ES22 actuator load vs. average bolt displacement.
Figure D-45. ES23 (grouted, 0 dia. exposed length, 0.625 in. F1554 Gr. 55 bolts, n = 2, tension/shear, 6 in. nominal eccentricity, pretensioned) test schematic.

Figure D-46. ES23 actuator load vs. average bolt displacement.
Figure D-47. ES24 (grouted, 1 dia. exposed length, 0.625 in. F1554 Gr. 55 bolts, n = 2, tension/shear, 6 in. nominal eccentricity, pretensioned) test schematic.

Figure D-48. ES24 actuator load vs. average bolt displacement.
Figure D-49. ES25 (grouted, 3 dia. exposed length, 0.625 in. F1554 Gr. 55 bolts, n = 2, tension/shear, 6 in. nominal eccentricity, pretensioned) test schematic.

Figure D-50. ES25 actuator load vs. average bolt displacement.
Figure D-51. ES26 (grouted, 1 dia. exposed length, 0.625 in. F1554 Gr. 55 bolts, n = 2, tension/shear, 6 in. nominal eccentricity, pretensioned) test schematic.

Figure D-52. ES26 actuator load vs. average bolt displacement.
Figure D-53. ES27 (grouted, 3 dia. exposed length, 0.625 in. F1554 Gr. 55 bolts, n = 2, tension/shear, 24 in. nominal eccentricity, pretensioned) test schematic.

Figure D-54. ES27 actuator load vs. average bolt displacement.
Figure D-55. ES28 (grouted, 0 dia. exposed length, 0.625 in. F1554 Gr. 55 bolts, n = 2, compression/shear, 6 in. nominal eccentricity, pretensioned) test schematic.

Figure D-56. ES28 actuator load vs. average bolt displacement.
Figure D-57. ES29 (grouted, 1 dia. exposed length, 0.625 in. F1554 Gr. 55 bolts, n = 2, compression/shear, 24 in. nominal eccentricity, pretensioned) test schematic.

Figure D-58. ES29 actuator load vs. average bolt displacement.
Figure D-59. ES30 (grouted, 1 dia. exposed length, 0.625 in. F1554 Gr. 55 bolts, n = 2, compression/shear, 24 in. nominal eccentricity, pretensioned, Teflon) test schematic.

Figure D-60. ES30 actuator load vs. average bolt displacement.
Figure D-61. ES31 (grouted, 3 dia. exposed length, 0.625 in. F1554 Gr. 55 bolts, n = 4, 6 in. nominal eccentricity, pretensioned) test schematic.

Figure D-62. ES31 actuator load vs. average bolt displacement.
Figure D-63. ES32 (grouted, 3 dia. exposed length, 0.625 in. F1554 Gr. 55 bolts, n = 4, 24 in. nominal eccentricity, pretensioned) test schematic.

Figure D-64. ES32 actuator load vs. average bolt displacement.
E.1 Introduction

Uniform axial stresses, bending moments, and shear stresses resulting from combinations of axial and shear loading on an anchor bolt interact in the cross-section of an anchor bolt in a stand-off base plate connection. This section provides the derivation of the ways that shear, moment, and axial stresses interact in the fully plasticized condition on a circular bolt cross-section, producing the basis for the recommended model for the steel strength of anchor bolts in ungrouted stand-off base plates. Limiting values of ultimate normal, shear, and moment capacity are given by (E-1) through (E-3) for a circular section.

\[ N_o = A_{nt} \sigma_o = \pi r_B^2 \sigma_o \]  
\[ M_o = Z_B \sigma_o = \frac{4 r_B^3 \sigma_o}{3} \]  
\[ V_o = \alpha N_o \]

where
\[ N_o \] = ultimate uniaxial normal capacity of anchor bolt  
\[ M_o \] = ultimate moment capacity of anchor bolt  
\[ V_o \] = ultimate shear capacity of anchor bolt  
\[ A_{nt} \] = net tensile area of anchor bolt  
\[ r_B \] = effective radius of bolt cross-section from net tensile area  
\[ Z_B \] = plastic section modulus of anchor bolt  
\[ \sigma_o \] = ultimate normal stress capacity of bolt material

E.2 Interaction of Normal Force and Bending Moment

Figure E-1 shows applicable variables to the interaction of normal force and bending moments. In this model, the distance \( \eta r_B \) and its corresponding value of \( \varphi \) delineate what portion of the cross-section is in compression and what portion is in tension. When \( \eta r_B \) is equal to the
bolt radius, \(r_B\), corresponding to \(\varphi = 0\), the cross-section is in pure compression. Similarly, when \(\eta r_B\) is equal to \(r_B\) in the opposite direction as drawn, \(\varphi = 2\pi\) and the section is in of pure tension. When \(\eta r_B = 0\), \(\varphi = \pi\) and the section is experiencing pure bending moment. All intermediate values contain combinations of axial and bending moments on the cross-section.

For any value of \(\varphi\) between 0 and \(\pi\), the tensile area of the circular segment below \(\eta r_B\) is represented by (E-4). The area in compression is found by subtracting the tension area from the area of the full circular section, as in (E-6).

\[
A^+ = \frac{r_B^2(\varphi - \sin(\varphi))}{2} \quad (E-4)
\]

\[
N^+ = \sigma_o A^+ = \frac{\sigma_o r_B^2(\varphi - \sin(\varphi))}{2} \quad (E-5)
\]

\[
A^- = \pi r_B^2 - \frac{r_B^2(\varphi - \sin(\varphi))}{2} \quad (E-6)
\]

\[
N^- = \sigma_o A^- = \sigma_o \left(\pi r_B^2 - \frac{r_B^2(\varphi - \sin(\varphi))}{2}\right) \quad (E-7)
\]
where

\( A^+ \) = area of bolt experiencing positive (tensile) stress

\( A^- \) = area of bolt experiencing negative (compressive) stress

\( \phi \) = angle describing location of tensile and compressive regions

\( N^+ \) = tension force on the cross-section

\( N^- \) = compressive force on the cross-section

The total normal force can then be obtained as (E-8). Normalizing this value by the

ultimate axial capacity of the section, (E-9) is produced.

\[
N_B = \int_A \sigma_o dA = \sigma_o (A^+ - A^-) = N^+ + N^- \\
= \sigma_o \left( \frac{r_B^2 (\phi - \sin(\phi))}{2} - \left( \pi r_B^2 - \frac{r_B^2 (\phi - \sin(\phi))}{2} \right) \right) \tag{E-8}
\]

\[
= \sigma_o r_B^2 (\phi - \pi - \sin(\phi))
\]

\[
n_B = \frac{N_B}{N_o} = \frac{\sigma_o r_B^2 (\phi - \pi - \sin(\phi))}{\pi r_B^2 \sigma_o} \tag{E-9}
\]

\[
= \frac{(\phi - \pi - \sin(\phi))}{\pi}
\]

where

\( N_B \) = resultant normal force on cross-section

\( n_B \) = ratio of normal force to normal capacity

The moment arm for the tensile stress is the distance from the centroid of the circular

section to the centroid of the tensile segment, given by (E-10). The distance to the centroid of the

compressive area, then, can be found using (E-11) and (E-12) by geometrically decomposing the

two areas about the centroid of the larger section.

\[
d^+_R = \frac{4 r_B \sin^3(\phi/2)}{3(\phi - \sin(\phi))} \tag{E-10}
\]

\[
A^+ d^+_R - A^- d^-_R = A_{nt}(0) = 0 \tag{E-11}
\]
\[
\begin{align*}
&= \left( \frac{r_B^2}{2} (\varphi - \sin(\varphi)) \right) \left( \frac{4r_B \sin^3(\varphi/2)}{3(\varphi - \sin(\varphi))} \right) \\
&\quad - \left( \pi r_B^2 - \frac{r_B^2(\varphi - \sin(\varphi))}{2} \right) d_R^- \\
d_R^- &= \frac{r_B \sin^3(\varphi/2)}{3(2\pi - \varphi - \sin(\varphi))} \\
\end{align*}
\]

(E-12)

where \( d_R^+ \) = distance to the resultant cross-section tensile force
\( d_R^- \) = distance to the resultant cross-section compressive force

Knowing the areas and their moment arms, the total bending moment can be obtained as

(E-13). Normalizing this value by the ultimate moment capacity of the cross-section results in

(E-14).

\[
M_B = \int_A \sigma_o y dA \\
= N^+ d_R^+ + N^- d_R^- \\
= \left( \sigma_o \frac{r_B^2 (\varphi - \sin(\varphi))}{2} \right) \left( \frac{4r_B \sin^3(\varphi/2)}{3(\varphi - \sin(\varphi))} \right) \\
\quad + \left( \sigma_o \left( \pi r_B^2 - \frac{r_B^2(\varphi - \sin(\varphi))}{2} \right) \right) \left( \frac{r_B \sin^3(\varphi/2)}{3(2\pi - \varphi - \sin(\varphi))} \right) \\
= \frac{\sigma_o 4r_B \sin^3 \left( \frac{\varphi}{2} \right)}{3} \\
\]

\[
m_B = \frac{M_B}{M_o} \\
= \sigma_o \frac{4r_B \sin^3 \left( \frac{\varphi}{2} \right)}{3} \\
= \frac{3}{4r_B^3 \sigma_o} \]

367
\[ = \sin^3 \left( \frac{\varphi}{2} \right) \]

where \( M_B \) = resultant bending moment on cross-section
\( n_B \) = ratio of bending moment to moment capacity

With values of \( m_B \) and \( n_B \) known for any value of \( \varphi \), the angle can be discretized from 0 to \( \pi \) to produce corresponding values of \( m_B \) and \( n_B \) and therefore their interaction. Fitting a curve to this relationship, (E-15) is produced, extremely close to the value for a rectangular section. Thus, (E-16) can be used practically for the interaction of moment and tension forces on a circular section.

\[ m_B + n_B^{2.01} = 1 \quad \text{(E-15)} \]

\[ m_B + n_B^2 = 1 \quad \text{(E-16)} \]

**E.3 Interaction of Normal and Shear Forces**

As described in Section 2.2.3.5, it is assumed that axial and shear forces acting on a cross-section are distributed uniformly, resulting in constant shear and tensile stress components over the entire section. As a result, all cross-sectional shapes, including circular, will be described by the interaction relationship between normal and shear stresses at all points in the cross-section, resulting in the Figure E-2 stress distribution and interaction relationships (E-17) and (E-18).

![Figure E-2. Assumed distribution of combined normal and shear stresses on circular section.](image-url)
\[
\left(\frac{N_B}{N_o}\right)^2 + \left(\frac{V_B}{\alpha N_o}\right)^2 = 1 
\]

(E-17)

\[
n_B^2 + v_B^2 = 1
\]

(E-18)

**E.4 Interaction of Bending and Shear Forces**

**Lower Bound.** An assumed stress distribution with all moment stresses section is provided in Figure E-3 for a circular section. Because of this assumption, shear stresses from the shear force and normal stresses from bending do not interact at any point in the cross-section. One may notice that, under this assumption, the combined effects of shear and moment present an analogous relationship to the case of combined moment and axial force, where the moment and axial “components” can be rearranged to the extreme distances from the neutral axis and centered about the neutral axis, respectively. Therefore, the mathematics necessarily align for the case of combined moment and shear in the assumed lower-bound distribution, resulting in (E-19).

![Figure E-3. Assumed lower-bound distribution of combined normal and shear stresses on circular section.](image)

\[
m_B + v_B^2 = 1
\]

(E-19)

**Upper Limit.** Applying the general form for the maximized bending and shear forces given by Jirasek and Bazant (2001) and described in Literature Review 2.2.3.6 to a circular
section results in (E-20) and (E-21). The value of $k$ in these equations is adjusted to produce different combinations of bending and shear forces compatible with the constraints of the equations.

$$
\bar{M}_B = \frac{\sigma_o}{\alpha} \int_A \frac{z_B^2}{\sqrt{k^2 + \left(\frac{z_B}{\alpha}\right)^2}} dA
$$

$$
= 2 \frac{\sigma_o}{\alpha} \int_{-r_B}^{r_B} \frac{z_B^2 b(z)}{\sqrt{k^2 + \left(\frac{z_B}{\alpha}\right)^2}} dz
$$

$$
= 2 \frac{\sigma_o}{\alpha} \int_{-r_B}^{r_B} \frac{z_B^2 \sqrt{r_B^2 - z^2}}{\sqrt{k^2 + \left(\frac{z_B}{\alpha}\right)^2}} dz
$$

$$
\bar{Q}_B = k \sigma_o \int_A \frac{1}{\sqrt{k^2 + \left(\frac{z_B}{\alpha}\right)^2}} dA
$$

(E-21)

where

- $\bar{M}_B$ = maximized bending moment on the cross-section
- $\bar{Q}_B$ = maximized shear force on the cross-section
- $\sigma_o$ = ultimate normal stress capacity of bolt material
- $k$ = shear weighting factor, expressed in length
- $z_B$ = plastic section modulus of bolt
- $r_B$ = effective radius of bolt
- $z$ = distance from bolt centroid to differential strip
- $b(z)$ = width of differential strip
- $\alpha$ = ratio of shear to tension capacity

Discretizing $k$ from 0 in. to 1 in., the interacting values of the maximized bending and shear forces are found. Again fitting a curve to the result using the least-squares method, (E-22) is produced for the interaction between bending and shear. For practical purposes the shear term exponent is conservatively reduced from 2.6 to 2, resulting in (E-23). It can readily be seen that this method will always produce a higher value than the lower-bound method presented above.
\[ m_B^2 + v_B^2 = 1 \]  \hspace{1cm} (E-22)
\[ m_B^2 + v_B^2 = 1 \]  \hspace{1cm} (E-23)

E.5 Three-dimensional Axial/Shear/Bending Interaction

**Lower Bound.** Using the assumed stress distribution from the lower-bound bending/shear interaction equation found in the previous section along with the bending/axial and axial/shear diagrams, a lower-bound combined bending/axial/shear stress diagram can be produced for the assumed distribution of stresses in Figure E-4. With this distribution, axial and shear stresses interact in the middle core of the cross-section, their combined effects interacting with the moment distribution on the extreme ends of the section from the neutral axis of bending. As a result, the three equations can be similarly derived to be combined to produce a three-dimensional interaction hypersurface between normal, moment, and shear forces. To produce this hypersurface mathematically, the methodology presented in (2-45) through (2-50). In this case, the relationships between shear/bending and normal bending forces can be rearranged as in (E-24) and (E-25), respectively. Normalizing the corresponding values of shear and normal force in their interaction equation, (E-26) is produced, which can be rearranged to be presented as (E-27).

![Diagram](image)

Figure E-4. Assumed distribution of combined normal and shear stresses on circular section.
\[ n = \sqrt{1 - m} \]  
\[ v = \sqrt{1 - m} \] \hspace{1cm} (E-24) \hspace{1cm} (E-25)

\[ \left( \frac{n}{\sqrt{1 - m}} \right)^2 + \left( \frac{v}{\sqrt{1 - m}} \right)^2 = 1 \] \hspace{1cm} (E-26)

\[ n^2 + v^2 + m = 1 \] \hspace{1cm} (E-27)

**Upper Limit.** Using the same technique presented for the lower-bound method, the upper-limit bending/shear relationship derived above can be combined with the bending/axial and axial/shear interaction equations to produce an interaction hypersurface as presented in (E-28) through (E-30). The stress distribution associated with this state is not given, as it relies on the optimized stress distribution for moment and shear force.

\[ n = \sqrt{1 - m} \] \hspace{1cm} (E-28)

\[ v = \sqrt{1 - m^2} \] \hspace{1cm} (E-29)

\[ \left( \frac{n}{\sqrt{1 - m}} \right)^2 + \left( \frac{v}{\sqrt{1 - m^2}} \right)^2 = 1 \] \hspace{1cm} (E-30)
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BIOGRAPHICAL SKETCH

Kenton E. McBride was born in 1987 to Dennis and Carol McBride in Omaha, Nebraska, where he lived through graduation at Millard North High School in 2005 and the University of Nebraska-Lincoln (Omaha Campus), where he completed a Bachelor of Science in civil engineering, in 2009. From Omaha Kenton moved to Gainesville, where he completed his Master of Engineering in 2011 and Ph.D. in 2014 in Civil Engineering (structural). Kenton enjoys playing basketball, nature, being done with his Ph.D., and Scrabble.