Splice Performance of #6 Reinforcing Bars in Masonry with Self-Consolidating Grout

Annie Ruth Nielson
Brigham Young University

Follow this and additional works at: https://scholarsarchive.byu.edu/etd

BYU ScholarsArchive Citation
https://scholarsarchive.byu.edu/etd/7765

This Thesis is brought to you for free and open access by BYU ScholarsArchive. It has been accepted for inclusion in Theses and Dissertations by an authorized administrator of BYU ScholarsArchive. For more information, please contact scholarsarchive@byu.edu, ellen_amatangelo@byu.edu.
Splice Performance of #6 Reinforcing Bars in Masonry with Self-Consolidating Grout

Annie Ruth Nielson

A thesis submitted to the faculty of Brigham Young University in partial fulfillment of the requirements for the degree of Master of Science

Fernando S. Fonseca, Chair
David Jensen
Kyle Rollins

Department of Civil and Environmental Engineering Brigham Young University

Copyright © 2019 Annie Ruth Nielson All Rights Reserved
ABSTRACT

Splice Performance of #6 Reinforcing Bars in Masonry with Self-Consolidating Grout

Annie Ruth Nielson
Department of Civil and Environmental Engineering, BYU
Master of Science

Reinforced masonry grouted using self-consolidating grout (SCG) is a relatively new and economically competitive option for construction, providing advantages such as reduced construction time, decreased noise and vibration, and reliable consolidation. However, SCG has different properties than conventional grout and its performance should be verified using current governing code requirements. The purpose of this research program was to determine the development length of spliced reinforcing bars in masonry grouted with SCG.

Twelve masonry panels, four courses high and two and a half blocks wide, were constructed using 8-inch concrete masonry units, each with two pairs of vertically spliced #6 reinforcing bars. Six of the panels had splice lengths that met current code provisions to verify that the code requirements are adequate for use with SCG. The remaining panels had shorter splice lengths than required to investigate the possibility of shorter splices in SCG. The ultimate bond strengths were compared to the design requirement for a splice to develop 125% of the yield strength of the reinforcing bars.

All lap splices developed the required stress, including those with shorter lengths. This indicates that the current code provisions are adequate to determine the development length of reinforcement splices in masonry grouted with SCG and reinforced with #6 bars in the specific configurations tested. According to this study, a development length reduction factor may be viable when SCG is used in masonry.

Keywords: self-consolidating grout, development length, masonry, splice
ACKNOWLEDGMENTS

This research project and written thesis came with a lot of support from others. Aaron Roper handed off the second half of what was originally his project to me. He did his best to acclimate me to the whole operation and answered all my questions. Because of all the effort he put into designing and setting up the project, my job was easier than his. Dr. Fonseca was very encouraging to me and always showed understanding and patient confidence in me throughout the process of getting a masters degree and starting my family at the same time. He met with me regularly and helped me with any research questions I had. He also did his best to make sure I had scholarships and funded research.

Dave Anderson and Rodney Mayo are the hands-on heroes of the BYU Structures lab. Under my direction they set up and ran all the physical testing and operations as well as the computers for continuous measurement. They made sure that necessary but often overlooked details such as safety measures and the timely emptying of dumpsters were taken care of. They always had a good idea to address any hiccups during the research process. Rodney both stayed late and arrived early in order to help me finish all the masonry panel and prism testing in the time we had allotted to use the lab. Andrew Cheney, the student lab assistant, Maggie Peterson, and Michael Reynolds also helped with various research tasks such as capping specimens.

I would especially like to thank Kim Glade for helping me graduate by making sure I met deadlines and got paperwork signed when needed, and for her general care and encouragement. Finally, my husband Mark was a great support in many ways including childcare and encouragement throughout the writing process and my masters degree coursework.
<table>
<thead>
<tr>
<th>TABLE OF CONTENTS</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>List of Tables</strong></td>
</tr>
<tr>
<td><strong>List of Figures</strong></td>
</tr>
<tr>
<td><strong>Chapter 1  Introduction</strong></td>
</tr>
<tr>
<td>1.1 Background</td>
</tr>
<tr>
<td>1.2 Literature Review</td>
</tr>
<tr>
<td>1.3 Contributions</td>
</tr>
<tr>
<td><strong>Chapter 2  Procedure</strong></td>
</tr>
<tr>
<td>2.1 General Testing Strategy</td>
</tr>
<tr>
<td>2.2 Initial Material Selection and Testing</td>
</tr>
<tr>
<td>2.2.1 Steel Reinforcement</td>
</tr>
<tr>
<td>2.2.2 Grout and CMU</td>
</tr>
<tr>
<td>2.3 Specimen Construction and Testing</td>
</tr>
<tr>
<td>2.3.1 Masonry Panel Construction</td>
</tr>
<tr>
<td>2.3.2 Mortar Testing</td>
</tr>
<tr>
<td>2.3.3 Grout and Masonry Prism Testing</td>
</tr>
<tr>
<td>2.3.4 Reinforced Masonry Panel Testing</td>
</tr>
<tr>
<td><strong>Chapter 3  Results and Analysis</strong></td>
</tr>
<tr>
<td>3.1 Preliminary Testing</td>
</tr>
<tr>
<td>3.1.1 Steel Reinforcement</td>
</tr>
<tr>
<td>3.1.2 Preliminary Grout and Masonry Specimens</td>
</tr>
<tr>
<td>3.2 Final Masonry Specimens</td>
</tr>
<tr>
<td>3.2.1 Mortar</td>
</tr>
<tr>
<td>3.2.2 Concrete Masonry Units</td>
</tr>
<tr>
<td>3.2.3 Grout</td>
</tr>
<tr>
<td>3.2.4 Grouted Masonry Prisms</td>
</tr>
<tr>
<td>3.2.5 Reinforced Masonry Panels</td>
</tr>
<tr>
<td><strong>Chapter 4  Conclusions</strong></td>
</tr>
<tr>
<td>4.1 Summary</td>
</tr>
<tr>
<td>4.2 Conclusions</td>
</tr>
<tr>
<td>4.3 Recommendations for Future Research</td>
</tr>
<tr>
<td><strong>Appendix A  Results</strong></td>
</tr>
<tr>
<td><strong>Appendix B  Specimen Schematics and Photographs</strong></td>
</tr>
</tbody>
</table>
# LIST OF TABLES

<table>
<thead>
<tr>
<th>Table</th>
<th>Description</th>
<th>Page</th>
</tr>
</thead>
<tbody>
<tr>
<td>2.1</td>
<td>Development Length Parameters</td>
<td>22</td>
</tr>
<tr>
<td>2.2</td>
<td>Loads to Determine Displacement Rate Change Thresholds</td>
<td>31</td>
</tr>
<tr>
<td>3.1</td>
<td>Steel Reinforcement Test Results</td>
<td>33</td>
</tr>
<tr>
<td>3.2</td>
<td>Preliminary Grout Compressive Strength Testing Results</td>
<td>34</td>
</tr>
<tr>
<td>3.3</td>
<td>Mortar Cube Test Results</td>
<td>35</td>
</tr>
<tr>
<td>3.4</td>
<td>Compressive Testing of Single Hollow CMU Blocks Results</td>
<td>36</td>
</tr>
<tr>
<td>3.5</td>
<td>Compressive Testing of Ungrouted Stacked CMU Prisms Results</td>
<td>36</td>
</tr>
<tr>
<td>3.6</td>
<td>Grout Prism Compression Test Results</td>
<td>38</td>
</tr>
<tr>
<td>3.7</td>
<td>Stacked Masonry Prism Compression Test Results</td>
<td>40</td>
</tr>
<tr>
<td>3.8</td>
<td>Reinforced Masonry Panel Testing Results</td>
<td>42</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>Reinforced masonry wall layout.</td>
<td>1</td>
</tr>
<tr>
<td>1.2</td>
<td>Reinforcing lap splice.</td>
<td>3</td>
</tr>
<tr>
<td>1.3</td>
<td>Soric and Tulin 1987 reinforced masonry specimens.</td>
<td>4</td>
</tr>
<tr>
<td>1.4</td>
<td>Hammons et al. 1994 reinforced masonry specimens.</td>
<td>6</td>
</tr>
<tr>
<td>1.5</td>
<td>Hammons et al. 1994 testing apparatus.</td>
<td>7</td>
</tr>
<tr>
<td>1.6</td>
<td>Thompson 1997 testing schematic.</td>
<td>8</td>
</tr>
<tr>
<td>1.7</td>
<td>Thompson regression compared to standards at the time.</td>
<td>9</td>
</tr>
<tr>
<td>1.8</td>
<td>NCMA 1999 testing apparatus, which was laid horizontally.</td>
<td>11</td>
</tr>
<tr>
<td>2.1</td>
<td>Masonry panel setup.</td>
<td>16</td>
</tr>
<tr>
<td>2.2</td>
<td>Headed steel reinforcement.</td>
<td>17</td>
</tr>
<tr>
<td>2.3</td>
<td>Rebar testing apparatus.</td>
<td>18</td>
</tr>
<tr>
<td>2.4</td>
<td>Headed rebar testing apparatus.</td>
<td>18</td>
</tr>
<tr>
<td>2.5</td>
<td>Grout prism molds. Paper towel is used to line the edges of each mold to prevent bonding while allowing seepage of water.</td>
<td>19</td>
</tr>
<tr>
<td>2.6</td>
<td>Elevated wooden bases with holes cut for the headed rebar.</td>
<td>20</td>
</tr>
<tr>
<td>2.7</td>
<td>Masonry panel construction.</td>
<td>21</td>
</tr>
<tr>
<td>2.8</td>
<td>26-in. splice length panel group schematic.</td>
<td>22</td>
</tr>
<tr>
<td>2.9</td>
<td>Panels before grouting.</td>
<td>23</td>
</tr>
<tr>
<td>2.10</td>
<td>SCG slump flow test, showing a flow of 21.5 inches.</td>
<td>24</td>
</tr>
<tr>
<td>2.11</td>
<td>Completed masonry panels.</td>
<td>25</td>
</tr>
<tr>
<td>2.12</td>
<td>Completed grout (left) and masonry (right) prisms.</td>
<td>25</td>
</tr>
<tr>
<td>2.13</td>
<td>Mortar cube after compression testing.</td>
<td>26</td>
</tr>
<tr>
<td>2.14</td>
<td>Capping masonry prisms.</td>
<td>26</td>
</tr>
<tr>
<td>2.15</td>
<td>Masonry (left) and grout (right) prism compression testing. Two plates were placed on the stacked masonry prisms to distribute the load.</td>
<td>27</td>
</tr>
<tr>
<td>2.16</td>
<td>Masonry prism modes of failure.</td>
<td>28</td>
</tr>
<tr>
<td>2.17</td>
<td>Grout prism modes of failure.</td>
<td>28</td>
</tr>
<tr>
<td>2.18</td>
<td>Reinforced masonry panel direct tension testing apparatus.</td>
<td>30</td>
</tr>
<tr>
<td>2.19</td>
<td>Panel prepared for testing in direct tension apparatus. Left: The fork lift has lowered the panel until it is hanging from the top wide flange beam. Middle: The headed rebar can be seen sitting on the top washer which is placed on the flanges of the rotated wide flange. Right: The headed rebar can be seen under the bottom washer which is placed on the web of the rotated wide flange.</td>
<td>31</td>
</tr>
<tr>
<td>3.1</td>
<td>Stress-strain curve for steel reinforcing bar sample 3. Visual observation of the yield plateau was used to determine a yield strength of 69 ksi.</td>
<td>34</td>
</tr>
<tr>
<td>3.2</td>
<td>Stress vs. strain curves for single hollow concrete masonry blocks and stacked ungrouted masonry prisms.</td>
<td>37</td>
</tr>
<tr>
<td>3.3</td>
<td>Failed hollow CMU block.</td>
<td>37</td>
</tr>
<tr>
<td>3.4</td>
<td>Failed ungrouted stacked CMU prism.</td>
<td>38</td>
</tr>
<tr>
<td>3.5</td>
<td>Stress vs. displacement plot for grout prisms at 47 and 48 days.</td>
<td>39</td>
</tr>
<tr>
<td>3.6</td>
<td>Stress vs. displacement plot for masonry prisms at 47 and 48 days.</td>
<td>41</td>
</tr>
</tbody>
</table>
3.7 Measured vs. predicted splice capacity. ................................................. 43
3.8 Failure pattern of 21” masonry panel #1, showing cracking through the mortar and two blocks. ......................................................... 44
3.9 Stress vs. displacement for the six panels with 31-in. splices. ................ 45
3.10 Stress vs. displacement for the three panels with 26-in. splices. .............. 45
3.11 Stress vs. displacement for the three panels with 21-in. splices. .............. 46
3.12 Stress vs. displacement for all panels. Increased splice length is clearly correlated with not only increased strength, but also higher ductility. ......................... 47
3.13 Average stress vs. displacement for each splice length. The two vertical lines for each group are located at average ultimate stress and average ultimate displacement. .. 48
3.14 SCG-rebar interface. No slippage is apparent. ....................................... 48
3.15 SCG-CMU interface. The smooth rounded edge of the grout within the masonry cavity indicates complete consolidation of the SCG. .......................... 49
3.16 Stair-step patterned cracking in 21-in. test group masonry panel #2. ....... 49
3.17 Catastrophic failure of 26-in. test group masonry panel #1. .................... 50
3.18 Measured vs. predicted splice capacity combined data with Roper (2018). Splices grouted with SCG tend to perform better than expected according to the linear regression (Equation 1.7), although the relationship is not linear between different bar sizes. .. 52
3.19 Measured vs. predicted splice capacity combined data with Roper (2018). This is the same as the previous plot with the axes beginning at zero. ................. 53
A.1 Stress-strain curve for steel reinforcing bar sample 1. ......................... 60
A.2 Stress-strain curve for steel reinforcing bar sample 2. .......................... 60
A.3 Stress-strain curve for Headed steel reinforcing bar sample. .................. 61
A.4 Stress vs. displacement for preliminary stacked masonry prisms grouted with SCG. . 61
B.1 21-in. splice length test group panel schematic. .................................... 63
B.2 31-in. splice length test group panel schematic. .................................... 64
B.3 31-in. panel #1. ........................................................................ 64
B.4 31-in. panel #2. ........................................................................ 65
B.5 31-in. panel #3. ........................................................................ 65
B.6 31-in. panel #4 with broken rebar. ..................................................... 66
B.7 31-in. panel #5. ........................................................................ 66
B.8 31-in. panel #6. ........................................................................ 67
B.9 26-in. panel #1. ........................................................................ 67
B.10 26-in. panel #2 with broken rebar. .................................................... 68
B.11 26-in. panel #3. ........................................................................ 69
B.12 21-in. panel #1. ........................................................................ 70
B.13 21-in. panel #2. ........................................................................ 70
B.14 21-in. panel #3. ........................................................................ 71
B.15 Stacked masonry prism #1, showing failure mode 2. ......................... 71
B.16 Stacked masonry prism #2, showing failure mode 2. ......................... 72
B.17 Stacked masonry prism #3, showing failure mode 3. ......................... 72
B.18 Stacked masonry prism #4, showing failure mode 3. ......................... 73
B.19 Stacked masonry prism #5, showing failure mode 1. ......................... 73
B.20 Stacked masonry prism #6, showing failure mode 3. ................................. 74
B.21 Grout prism #1, showing failure type 1. .................................................. 74
B.22 Grout prism #2, showing failure type 1. .................................................. 75
B.23 Grout prism #3, showing failure type 4. .................................................. 75
B.24 Grout prism #4, showing failure type 1. .................................................. 76
B.25 Grout prism #5, showing failure type 2. .................................................. 76
B.26 Grout prism #6, showing failure type 1. .................................................. 77
CHAPTER 1.  INTRODUCTION

1.1 Background

Masonry is one of the oldest and longest-lasting building materials. Some advantages of masonry include the ability to construct curved walls without complicated formwork, a vast array of available colors and textures for architectural purposes, durability, thermal conductivity for insulation purposes, and fire resistance (Hendry, 2001). Major modern advances in reinforced concrete have also benefited masonry. Reinforced masonry is an economically competitive choice for many applications, especially low- to medium-height residential, commercial, educational, and industrial buildings. In reinforced masonry, bars are placed in the cavities of the masonry at specified intervals and enclosed in grout, a cementitious material similar to concrete, as shown in Figure 1.1.

Figure 1.1: Reinforced masonry wall layout.
The bond between the reinforcing bars and the grout makes reinforced masonry a composite material. Reinforced masonry and reinforced concrete benefit from the tensile strength and ductility of steel. Since steel behaves in a ductile manner and masonry and concrete behave in a brittle manner at failure, it is desirable for the composite material to be controlled by the ductile nature of the steel. Without a sufficient bond between the grout (or concrete) and the reinforcement, the structure would lose its ductile character, leading to catastrophic brittle failure. Bond strength is maximized by minimizing the voids in the grout (or concrete) and maximizing the contact surface between the bar and the grout (or concrete). It is vital to consolidate fresh grout and concrete to minimize voids. In laboratory experiments, fresh grout and concrete are often consolidated using tamping rods. In the field, vibratory rods or “stingers” are inserted vertically into the fresh grout (or concrete) to consolidate it, among other methods (Mindess, 2003).

Self-consolidating concrete (SCC) was originally developed as a technological advancement in underwater concrete placement (Khayat, 1999). SCC is much more fluid than conventional concrete and consolidates under its own weight, eliminating the need for mechanical consolidation. It was widely adopted for general structural use in Japan in the late 1980s to solve structural stability problems caused by a large population of unskilled workers who were not trained to consolidate conventional concrete properly.

Grout generally has lower strength than conventional concrete and contains smaller aggregate, although the strength and mix design can vary almost as much as concrete. Like concrete, grout is a cementitious material made from Portland cement and has similar ratios of compressive, tensile and shear strength to concrete. Self-consolidating grout (SCG), a subset of SCC, has an ideal application in the masonry industry because of the congested nature of the masonry cavities. These tight spaces around the reinforcement make it difficult for conventional grout to consolidate properly. SCG achieves significantly more fluidity than conventional grout by limiting aggregate size as well as using plasticizing and viscosity modifying admixtures. The paste or powder content of SCG forms the matrix in which particles are suspended. Because of the high fluidity of the material, the paste must be carefully designed and monitored to maintain a stable mix that will not segregate. Although the admixtures in SCG make the material more expensive, the cost is often more than accounted for by the decrease in labor and construction time. SCG can be placed using
the same procedure as conventional grout except that the mortar fins do not have to be removed and lifts can be much higher, up to 24 ft, without the need of any consolidation (NCMA, 2007).

A major benefit of SCG is the reliable bond between the reinforcing bars and the grout. Well-consolidated grout maximizes the contact surface between the bar and the grout. In order for grout and steel to act as a unit, the embedment of a bar in the grout must be long enough to result in a bond strength that is at least as strong as the bar so that the bar would yield instead of pulling out of the grout. This minimum embedment length requirement is called development length.

Development length in tension governs not only the embedment of a single bar in grout, but also the required bar overlap length in a splice. Splicing consists of longitudinally overlapping two bars and tying them together using wire before pouring the grout, as depicted in Figure 1.2. Splicing is common in reinforced masonry (as well as in reinforced concrete) because many structural elements need a continuous bar to span longer than the available lengths of reinforcing bars. Properly spliced reinforcing bars are structurally equivalent to a continuous bar. Splice length is also known as lap length. Lap length, splice length, and development length are terms that will be used interchangeably throughout the rest of this thesis.

![Figure 1.2: Reinforcing lap splice.](image)

The strength of a splice comes from the bond between the bars and the grout. If the splice is long enough, or has a sufficient lap length, the grout can transfer the tensile load from one bar to the other without the bars pulling apart under loading. The masonry code requires a development
length that results in a bond between the grout and the reinforcement that is at least 125% of the yield strength of the bar. This is to ensure a sufficiently ductile response (Thompson, 1997). The required development length depends on many factors, including diameter of the reinforcing bar, yield strength of the bar, grout clear cover around the bar, and strength of the grout and masonry.

1.2 Literature Review

The equation that estimates development length based on all of the aforementioned factors has evolved over years of research. A shift from an allowable stress design format to a limit states design format in the late 1900s gave rise to more research about those limit states, including masonry research.

In 1987, Soric and Tulin (1987) led the American part of a collaborative American-Japanese effort to study reinforced masonry. They tested 27 pull-pull 6-inch nominal concrete masonry stacked specimens reinforced with various lengths of spliced #4 and #7 bars. Their specimens are shown in Figure 1.3.

![](image1.png)

**Figure 1.3:** Soric and Tulin 1987 reinforced masonry specimens.

To model the bond and slip distributions in the splices, they represented the masonry as a thick-walled cylindrical pressure vessel and the grout inside as hydraulic pressure. The system failed when the tensile forces from the grout exceeded the confining stress. Radial tensile stresses built up because of an angle between the bond forces and the bar when the bars were loaded in tension. The authors developed mathematical expressions relating variables in the masonry assemblies to the resulting bond strength of the splices. From their experiments they proposed that the three most important variables in splice performance were 1) bar size, 2) splice length, and 3)
masonry unit thickness. From their mathematical relationships and the assumption that the angle between the bar and the bond forces was 45 degrees, Equation 1.1 for splice length was obtained (Thompson, 1997):

\[ l_d = \frac{Cd_b^2f_y}{(t-d_b)f_{gt}} \]  

(1.1)

where:

- \( t \) = masonry thickness, in.;
- \( f_{gt} \) = grout tensile strength, psi;
- \( d_b \) = diameter of reinforcing bar, in.;
- \( f_y \) = yield strength of reinforcing bar, psi; and
- \( C \) = empirical constant.

The constant \( C \) accounted for nonuniformity of bond stresses along the length of bar. Based on the requirement for the splice to develop at least 125% of the yield stress of the bar, the average value of \( C \) was 1.75 for the combinations of bar diameter, unit size, and grout strength used by Soric and Tulin (1987).

The Masonry Standards Joint Committee adopted this equation into the Masonry Limit-States Design Standard (MLSDS) in 1992, assuming that \( C \) was 1.75 and that the grout tensile strength was 400 psi (Hammons et al., 1994). Thus, Equation 1.1 was modified into the form of Equation 1.2.

\[ \phi l_d = \frac{0.0045d_b^2f_{ye}}{(t-d_b)} \]  

(1.2)

where:

- \( \phi = 0.8 \) (capacity reduction factor); and
- \( f_{ye} \) = expected yield strength of the steel.

In 1994, the U.S. Army Corps of Engineers and Atkinson-Noland and Associates participated in the Construction Productivity Advancement Research (CPAR) Program. One of their
goals was to verify or modify the MLSDS equation for lap splice length. They also compared the provisions for the 1992 Uniform Building Code (UBC) and the 1988 ACI building code requirements for masonry structures. Their testing program included a total of 124 stacked concrete masonry specimens with 64 combinations of splice lengths, unit widths, masonry unit types, and bar diameters. The range of splice lengths and specimen sizes of concrete masonry units are shown in Figure 1.4.

Researchers tested the specimens using a vertical testing apparatus shown in Figure 1.5. This test schematic was designed to test the specimens in pure tension, but a small global eccentricity was created by the adjacent reinforcing bars forming the lap splice.

The CPAR program concluded that the minimum cover of the rebar had a significant effect on the capacity of the splices, with bigger bars tending to fail sooner than smaller ones. An average value of 1.75 for C in the MLSDS proposed equation did not accurately predict the required splice length, but the MLSDS equation matched the CPAR experimental data when researchers assigned a unique value of the constant C to each bar size (Hammons et al., 1994).

In 1994, the Uniform Building Code (UBC) adopted a new strength design expression for development length given in Equations 1.3 and 1.4 (International Conference of Building Officials, 1994). According to Thompson (1997), the $52d_b$ limit often governed the specified development length.
\[ l_d = \frac{l_{de}}{\phi} \geq 12 \text{ inches} \] (1.3)

and:

\[ l_{de} = \frac{0.15d_b^2f_y}{K\sqrt{f'_m}} \leq 52d_b \] (1.4)

where:

- \( l_d \) = development length of reinforcing bar, in.;
- \( \phi \) = strength reduction factor; equal to 0.80;
- \( l_{de} \) = basic development length, in.;
- \( d_b \) = diameter of reinforcing bar, in.;
- \( f_y \) = tensile yield stress of reinforcing bar, psi;
- \( K \) = reinforcing bar clear cover or clear spacing, whichever is less, and not greater than \( 3d_b \), in; and
- \( f'_m \) = ultimate compressive strength of masonry assemblage, psi.

Thompson (1997) and his team at Washington State University tested several 8-inch concrete masonry panel specimens reinforced with bar sizes 4 and 7. The vertical testing schematic is shown in Figure 1.6. Thompson then combined his data with data from previous and concurrent
studies into a single database of over 150 specimens for numerical analysis. Thompson performed a multiple nonlinear regression model on the combined data from the test specimens of Soric and Tulin (1987), CPAR (Hammons et al., 1994), and the first two phases of a concurrent NCMA research program (Thomas et al., 1999), using variables of splice length, bar diameter, compressive strength of the masonry assemblage, and clear cover. The analysis resulted in Equation 1.5 for splice length.
\[ l_s = \frac{1.25A_b f_y + 23103.54 - 18472.85d_b^2 - 319.68\sqrt{f_m} - 3658.41c_{cl}}{554.81} \]  

where:

- \( l_s \) = length of lap splice, in.;
- \( A_b \) = area of the bar, \( \text{in}^2 \);
- \( f_y \) = tensile yield stress of reinforcing bar, psi;
- \( d_b \) = reinforcing bar diameter, in.;
- \( f_m' \) = ultimate compressive strength of masonry assemblage, psi;
- \( c_{cl} \) = minimum clear cover.

Thompson discovered that the previous code equation slightly overestimated required splice length for smaller bars and underestimated splice length for larger bars. A comparison of his multiple regression equation with that of other codes is shown in Figure 1.7, with the multiple nonlinear regression line bolded. The MLSD equation (Equation 1.2) is closer to Thompson’s model than the other standards.

Figure 1.7: Thompson regression compared to standards at the time.
Thompson proposed a few modifications to the UBC 1994 equation (Equation 1.4) resulting in Equation 1.6. This equation increased the maximum clear cover to bar diameter ratio from 3 to 5, adopted a gamma factor to account for the difficulty of achieving ductility with larger bars, and omitted the upper bound of 52\(\text{db}\) for the splice length (Thompson, 1997).

\[
\phi l_s = \frac{0.15 d_b f'_m \gamma}{K \sqrt{f'_m}} \geq 12 \text{ inches}
\]

where:

\(\gamma = 1.0\) for #3 through #6 reinforcing bars;
\(\gamma = 1.4\) for #7 through #11 reinforcing bars;
\(\phi = 0.8\);
\(K = \) the lesser of reinforcement cover or \(5d_b\), in.;
\(c_{cl}\) = minimum clear cover, in.;
\(d_b\) = reinforcing bar diameter, in.;
\(f'_m\) = ultimate compressive strength of masonry assemblage, psi; and.
\(l_s\) = length of lap splice, in.

Thompson also conducted research sponsored by the National Concrete Masonry Association (NCMA), in which the research team performed concurrent and almost identical research to that of Thompson (1997) with the added variable of reinforcing cover. Clay brick masonry was tested as well as concrete masonry. Concrete masonry panels made of both 8-inch and 12-inch CMU’s were tested in groups of three identical panels per set, using reinforcing bar sizes 4 through 9 and also varying grout strength, splice length, and reinforcing cover. A total of 108 concrete masonry panels were tested over four phases of research (Thomas et al., 1999). The testing frame, laid horizontally, is shown in Figure 1.8. The bars were configured symmetrically to minimize global eccentricity. Local eccentricity still existed at each splice, but the symmetry allowed direct tension during testing and minimized induced moment on the masonry panel.

For the multiple nonlinear regression analysis, the NCMA researchers chose to include specimens from all four phases of their own research as well as those from WSU (Thompson, 1997) and CPAR (Hammons et al., 1994). The authors included only specimens that failed by masonry splitting to focus on that particular mode of failure. The resulting data set consisted of
Figure 1.8: NCMA 1999 testing apparatus, which was laid horizontally.

177 individual specimens reinforced with Grade 60 steel bars, with concrete masonry units from 4 to 12 inch nominal thickness, bar sizes from #4 to #11, lap lengths from 12 inches to nearly 130 inches, and clear covers from 1.44 inches to 5.5 inches.

The NCMA research team noticed that when panels failed by masonry splitting (the usual failure mechanism), the failure path followed the path of least cover. They also investigated what would be the best definition of $f_m'$ for the multiple nonlinear regression analysis. Block strength, grout strength, and masonry strength all correlated well (resulting in an $r^2$ value of greater than 0.9) and could have been used, so they chose the masonry strength, defined as the compressive strength of a grouted masonry prism, according to tradition from similar research. This definition has the added benefit of being a composite measure of both masonry unit and grout strength. The multiple regression equation that best predicted the observed capacities of the splices is given in Equation 1.7.

$$T_r = -17624 + 305l_s + 25204d_b^2 + 322\sqrt{f_m'} + 3332c_{cl}$$  \hspace{1cm} (1.7)
where:

\[ T_r = \text{predicted load capacity of the splice, lb}; \]
\[ l_s = \text{tested lap length of splice, in.}; \]
\[ d_b = \text{diameter of reinforcement, in.}; \]
\[ f_{mt} = \text{tested compressive strength of masonry, psi, and} \]
\[ c_{cl} = \text{clear cover to reinforcement, in.}. \]

Based on code requirements that the load capacity of a splice be at least 1.25 times the yield strength of the bar, NCMA researchers replaced the predicted load capacity with 1.25\(A_b f_y\) and the solved for the development length to obtain Equation 1.8.

\[
l_r = \frac{1.25A_b f_y + 17624 - 25204d_b^2 - 322\sqrt{f'_{m}} - 3332c_{cl}}{305}
\]  

(1.8)

where:

\[ l_r = \text{basic development length based on regression analysis, in;} \]
\[ A_b = \text{area of reinforcing bar, in}^2; \]
\[ f_y = \text{yield strength of reinforcing steel, psi;} \]
\[ f'_{m} = \text{specified compressive strength of masonry, psi, and} \]
\[ c_{cl} = \text{clear cover of reinforcement, in.}. \]

Equation 1.8 was simplified to resemble the UBC equation, resulting in Equation 1.9.

\[
l_{de} = \frac{0.13d_b^2 f_y \gamma}{K \sqrt{f'_{m}}} \geq 12 \text{ inches}
\]  

(1.9)
where:

\[ l_{de} = \text{basic development length, in.}; \]
\[ d_b = \text{diameter of reinforcement, in.}; \]
\[ f_y = \text{specified yield strength of reinforcement, psi}; \]
\[ \gamma = \text{reinforcement size factor}; \]
\[ = 1.0 \text{ for #3 through #5 reinforcing bars}; \]
\[ = 1.4 \text{ for #6 through #7 reinforcing bars}; \]
\[ = 1.5 \text{ for #8 through #11 reinforcing bars}; \]
\[ K = \text{the lesser of minimum masonry cover or } 7d_b, \text{ in.}; \]
\[ f_m' = \text{specified compressive strength of masonry, psi}. \]

In 2002, the Masonry Standards Joint Committee (MSJC) adopted Equation 1.9 into their standard (Masonry Standards Joint Committee, 2002). In 2005, the reinforcement size factor \( \gamma \) was reduced to 1.3 for #6 and #7 bars, which was less conservative without significantly affecting the resulting \( l_d \) (Masonry Standards Joint Committee, 2005). The design standard from Building Code Requirements for Masonry Structures (TMS 402 2016) has not changed since 2005. The current equation for development length of uncoated bars in masonry is given in Equation 1.10 (American Concrete Institute, 2016).

\[
l_d = \frac{0.13d_b^2f_y\gamma}{K\sqrt{f_m'}} \geq 12 \text{ inches} \tag{1.10}
\]

where:

\[ l_d = \text{required development length, in.}; \]
\[ d_b = \text{diameter of reinforcing bar, in.}; \]
\[ f_y = \text{specified yield strength of reinforcement, psi}; \]
\[ \gamma = \text{reinforcement size factor}; \]
\[ = 1.0 \text{ for #3 through #5 reinforcing bars}; \]
\[ = 1.3 \text{ for #6 through #7 reinforcing bars}; \]
\[ = 1.5 \text{ for #8 and larger reinforcing bars}; \]
\[ K = \text{the lesser of minimum masonry cover or } 9d_b, \text{ in.}; \]
\[ f_m' = \text{specified compressive strength of masonry, psi}. \]
1.3 Contributions

All of the experiments leading to the current governing equation for the development length of reinforcing bars in masonry involved conventional grout that was mechanically consolidated. Self-consolidating grout has different properties than conventional grout, and so it is prudent to investigate the existing requirements with this newer building material. Roper (2018) conducted this type of research by investigating the splice length of #5 bars in masonry using SCG.

This experimental program was designed to compliment and add to the research of Roper (2018) by testing the required splice length of #6 reinforcing bars in masonry using SCG. Preliminary studies on this topic have suggested that SCG may perform better than conventional grout, requiring a shorter development length to meet the 125% yield strength code requirement. It is possible that a reduction factor could be applied to the design equation for splice length when SCG is used.
CHAPTER 2. PROCEDURE

2.1 General Testing Strategy

The testing program was arranged as follows. Twelve masonry panels were constructed with two vertical reinforcement splices placed at 16 inches on center in each panel. The bars projected past the top and bottom of the masonry panel such that they could be connected to an apparatus to apply a tensile force, as shown in Figure 2.1. A tensile force was chosen because that was the method used to determine the current design requirements.

Of the twelve panels, six had the design splice length according to the current code. The other two sets of three masonry panels each had reduced splice lengths to investigate the possibility of SCG requiring a shorter splice length to develop 125% of the yield strength of the bar.

Scheduling made it so that not all the panels were tested on the same day. This could have potentially skewed the data slightly because the grout and mortar gain strength over time. To adjust for this inconsistency, grouted stacked masonry prisms were constructed on the same day as the panels, using the same grout, mortar, and CMU as the panels. Three of these stacked prisms were tested in compression on each day that the panels were tested. Correlating the strength of the panels with the compressive strength of the prisms tested that day, $f_m'$, allowed the results to be normalized.

2.2 Initial Material Selection and Testing

Design splice length depends on the compressive strength of masonry and the yield strength of the bar; therefore, the first step in the research was to select the materials and test these properties.
2.2.1 Steel Reinforcement

Grade 60 #6 steel bars were selected for the reinforcement for two reasons. First, because they have a large enough diameter that their splice length would not be controlled by the 12-in. minimum requirement. Second, because similar research has been done with #5 bars (Roper, 2018). This research would augment that body of knowledge. Headed rebar conforming to ASTM A970 (2017), Standard Specification for Headed Steel Bars for Concrete Reinforcement, class A and B, was supplied in 4 ft. lengths for the project. The heads allowed the tensile load to be applied using the available equipment. Figure 2.2 shows a photograph of the 4-ft. headed steel bars.

ASTM A615 (2016), Standard Specification for Deformed and Plain Carbon-Steel Bars for Concrete Reinforcement, states that Grade 60 bars must have minimum yield and ultimate
strengths of 60 and 90 ksi, respectively. The supplier tested the bars according to ASTM A370 (2017), Standard Test Methods and Definitions for Mechanical Testing of Steel Products, Method A9, and certified them as Grade 60. The bars were also tested by researchers in tension to confirm their specified yield strength. Four twelve-inch lengths were cut from the reinforcing bars and tested according to ASTM E8 (2016), Standard Test Methods for Tension Testing of Metallic Materials, at a strain rate of 0.3 in./min until tensile failure. Three of the samples were simple bar sections, while the final sample had the head connected. Figures 2.3 and 2.4 show the testing apparatus used for the ordinary bar sections and headed bar section, respectively. Stress-strain curves were developed for each tested specimen, and the yield strength was determined. These tests confirmed that the bars were stronger than the specified yield strength, but the specified yield strength was still used to calculate the required splice length.
2.2.2 Grout and CMU

Because of volume limitations in laboratory equipment, it was determined to use grout from a ready-mix supplier. The mix design was obtained from the supplier and mixed in-house for the preliminary testing. Preliminary testing of the grout, CMU, and masonry prisms was conducted to determine the compressive strength of these materials in order to design the required splice length according to the current design equation (Equation 1.10). Nine grout prisms were constructed according to ASTM C1019 (2016), Standard Test Method for Sampling and Testing Grout. Masonry units were arranged as shown in Figure 2.5, with paper towels placed between the grout and the
Figure 2.5: Grout prism molds. Paper towel is used to line the edges of each mold to prevent bonding while allowing seepage of water.

faces of the masonry forming the mold, to prevent bonding while allowing water to be drawn into the CMU. A plexiglass plate with form release oil applied was placed at the bottom of each grout prism mold.

SCG was poured into the molds in a single lift without tamping or vibration, and the surface was struck off. After an additional 15 minutes of self-consolidation, any shrinkage was refilled and the tops were resurfaced. The grout prisms were removed from the molds and transferred to a fog room to cure between 24 and 48 hours after being cast.

Seven masonry prisms were constructed using nominal 8-in. single core masonry units with Type S mortar from ready-mix bags, according to ASTM C1314 (2016), Standard Test Method for Compressive Strength of Masonry Prisms. The hollow stacked masonry prisms were then placed in watertight bags and grouted, and the bags were sealed. One grouted masonry prism and three grout prisms were tested at 14 days, and six grouted masonry prisms and six grout prisms were tested at 28 days.
2.3 Specimen Construction and Testing

2.3.1 Masonry Panel Construction

Masonry panels were constructed on raised wooden 2x12 DF-L#2 bases to allow the reinforcement to extend past the bottom of the masonry panels. The boards were marked with lines to ensure correct placement of the masonry, and holes were cut in the boards for the reinforcement heads to pass through. The cutouts were kept for the purpose of plugging the holes before grouting. The bases were elevated by placing them on 8-in. half CMU blocks, as shown in Figure 2.6.

![Elevated wooden bases with holes cut for the headed rebar.](image)

Professional masons constructed the masonry panels. All mortar was made by masons by combining bagged Type S mortar and water in a concrete floor mixer. Mortar was mixed for a sufficient time to prevent false set caused by rehydration of the gypsum in the mixture. The mortar was transported to the construction area by wheelbarrow and placed on stands.

Twelve panels were assembled with 8-in. CMUs and Type S mortar in running bond. The panels were nominally identical, each 4 courses tall, with a mortar joint beneath the first course to facilitate a level plane. Levelness was checked throughout the entire construction process. Mortar joints were finished with a concave tool. Figure 2.7 is a photograph from construction day. Mortar fins and droppings were removed from the masonry cells after construction.
On the same day as masonry panel construction, twelve masonry prisms were also constructed according to ASTM C1314 (2016) using the same CMUs and mortar. These were the prisms that were tested concurrently with the panels to determine the masonry compressive strength on the day of testing. Five mortar cubes were also cast according to ASTM C109 (2016), Standard Test Method for Compressive Strength of Hydraulic Cement Mortars (Using 2-in. or [50-mm] Cube Specimens).

After construction, the twelve panels were divided into three test groups, each with different splice lengths. The first group consisted of six panels with the splice length according to current design requirements. The required splice length was calculated based on a yield stress of 60 ksi and a compressive strength, $f'_m$, obtained from the preliminary masonry prism tests. The other two
groups consisted of three panels each and had shorter splice lengths. Parameters from the design Equation 1.10, as well as the development length used, are summarized in Table 2.1. A schematic of the panels with a 26-in. splice length is shown in Figure 2.8. Schematics of the other two test groups are included in Appendix B.

Table 2.1: Development Length Parameters

<table>
<thead>
<tr>
<th>Test Group</th>
<th>(d_b) (in)</th>
<th>(f_y) (ksi)</th>
<th>(\gamma)</th>
<th>(K) (in)</th>
<th>(f_m') (psi)</th>
<th>(l_{d,calc}) (in)</th>
<th>(l_{d,used}) (in)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>0.75</td>
<td>60</td>
<td>1.3</td>
<td>3.4375</td>
<td>2875</td>
<td>30.95</td>
<td>31</td>
</tr>
<tr>
<td>2</td>
<td>0.75</td>
<td>60</td>
<td>1.3</td>
<td>3.4375</td>
<td>2875</td>
<td>30.95</td>
<td>26</td>
</tr>
<tr>
<td>3</td>
<td>0.75</td>
<td>60</td>
<td>1.3</td>
<td>3.4375</td>
<td>2875</td>
<td>30.95</td>
<td>21</td>
</tr>
</tbody>
</table>

Figure 2.8: 26-in. splice length panel group schematic.
Lap splices were assembled by tying the bars together with bailing wire and placing them through the holes of the wooden bases. The holes were patched with the original wood cutouts and industrial tape. Bars on the bottom were cut so that the bar ended at the top of the splice. Figure 2.9 is a photo of panels just before grouting.

Grouting of the panels and prisms occurred eight days after initial construction. A ready-mix truck delivered the SCG, which initially had a slump of 8 inches. Water was added incrementally, and the SCG was tested for slump and Visual Stability Index (VSI) with each addition of water until a slump flow of 22-in. was achieved. A VSI value of zero was observed for all tests. Figure 2.10 shows one of the slump flow tests.
The final grout mix was transferred from the ready-mix truck to the panels in large bins carried by forklift. Grout was then poured into the masonry cells in a single lift with no mechanical consolidation using buckets. The reinforcement was then centered and confirmed vertical with a level, and the grout surface was finished. The panels were labeled with the splice length. Two completed panels are shown in Figure 2.11.

The ready-mixed SCG was also used to grout the CMU prisms and to cast SCG prisms in accordance with ASTM C1019 (2016). The completed grout and masonry prisms are shown in Figure 2.12. After 24 hours, the grout prisms were removed from their molds. The prisms were cured in the same ambient temperature and humidity as that of the panels. They were not placed in a fog room. This was done so that the compressive strength of the prisms was as identical as possible to that of the reinforced panels.

2.3.2 Mortar Testing

Mortar cubes were prepared on the same day as that of masonry panel construction. Five days after being cast, the cubes were removed from their molds and placed in a fog room to cure. Compressive strength was tested at 36 days according to ASTM C109 (2016) using a Forney
compression machine. The compressive tests were conducted at a displacement-controlled rate of 0.05 in./min. Figure 2.13 shows a typical mortar cube after testing.
2.3.3 Grout and Masonry Prism Testing

All prisms during this research project were capped with high-strength gypsum according to ASTM C1552 (2016), Standard Practice for Capping Concrete Masonry Units, Related Units and Masonry Prisms for Compression Testing, before compression testing. All caps cured for a minimum of two hours before testing. Compression testing of prism specimens was conducted under monotonic loading at a displacement-controlled rate of 0.05 in./min. A group of freshly capped masonry prisms is shown in Figure 2.14.

Three grout prisms and three masonry prisms were tested each day that reinforced masonry panels were tested, to normalize the results of the panel testing. Masonry prisms were tested ac-
According to ASTM C1019 (2016), and grout prisms were tested according to ASTM C1314 (2016). A Baldwin Universal Testing Machine (UTM) was used for all prism tests. Because the circular bearing block of the Baldwin UTM was smaller than the top area of the masonry prisms, flat steel plates were placed between the specimen and the bearing block to distribute the load. Two plates were used to obtain the necessary plate thickness. Load was applied at a displacement-controlled rate of 0.05 in./min. until specimen failure. Load vs. displacement diagrams were recorded by the computer connected to the compression machine. The compression testing setup of masonry and grout prisms is shown in Figure 2.15.

![Compression testing setup](image)

**Figure 2.15**: Masonry (left) and grout (right) prism compression testing. Two plates were placed on the stacked masonry prisms to distribute the load.

After failure, each specimen was compared to Figure 2.16 or Figure 2.17 to determine the failure mode. These figures came from ASTM C1314 (2016), Standard Test Method for Compressive Strength of Masonry Prisms, and ASTM C39 (2015), Standard Test Method for Compressive Strength of Cylindrical Concrete Specimens, respectively.
Figure 2.16: Masonry prism modes of failure.

Figure 2.17: Grout prism modes of failure.
2.3.4 Reinforced Masonry Panel Testing

There is no standard method for the testing of splices in reinforced masonry in tension. The approach in this research program, however, is consistent with similar research programs (Thompson, 1997; Hammons et al., 1994). Even though most splices in masonry walls are loaded in tension due to bending in the wall, a pull-pull test represents an extreme loading condition and allows for observation of the failure mode without the possibility of concrete crushing.

The testing apparatus for the reinforced masonry panels was a Baldwin Universal Testing Machine (UTM) with a capacity of 300 kips. High-strength rods were attached to the top and bottom crossheads of the UTM and to wide-flange steel sections to transfer the tensile load evenly to the two projecting pieces of rebar. A W8x31 shape was chosen based on an anticipated loading per bar of 1.5 times the yield strength of the rebar. The wide-flange was oriented so that the web was horizontal. On the top, the bars were inserted through holes in the web of the wide flange and secured with a washer that sat on the flanges of the wide flange. Initial testing of the apparatus had smaller washers sitting on the web, but there was a problem with the web yielding during loading. On the bottom, the rebar was also inserted through holes in the wide flange and secured with a washer, but space was too limited to allow the washer to bear on the flanges. In this case, yielding of the web was avoided by selecting thicker washers with a larger bearing area. Figure 2.18 shows a diagram of the testing apparatus layout.

To mount a panel on the testing machine, the panel was first hung from industrial lifting straps and lifted by a forklift into approximately the correct location in the center of the apparatus. After correct alignment was ensured, the panel was slowly lifted until the top rebar heads passed through the oversized holes in the wide-flange and reached the top of the flanges. Washers were then placed across the flanges between the rebar heads and the steel W section. The forklift was lowered until the panel hung from the top wide flange. The bottom adjustable crosshead was then raised until the lower reinforcing bars passed through the holes in the bottom wide-flange. Washers were placed between the web of the bottom steel section and the rebar heads, and the adjustable crosshead was lowered again until the connection was snug but not tight. The two wide flange members were not completely tightened against the crossheads to leave allowance for any slight variance in elevation or angle of the reinforcement. This allowance was not sufficient for cases where the elevation of the bottom two rebar heads differed by more than 1/8-in. In this case, small
1/8-in. thick steel plates were placed on either side of the rebar between the large washer and the steel section, serving as an additional washer. Figure 2.19 shows photographs of a panel ready to be tested as well as the top and bottom connections.

Before testing, the minimum required tensile capacity of the splice was calculated as 1.25 times the yield stress of the bar. For #6 bars, this was approximately 33 kips per bar assuming the specified yield stress of 60 ksi, or approximately 40 kips per bar assuming the yield stress from the suppliers mill test of 72 ksi. These values marked the loads at which the displacement rate was to be changed during testing. This change in displacement rate was justified because the yielding of the bar proved that the splice could develop the minimum required capacity. After the minimum capacity was reached, the masonry panels were tested until failure, but with a slightly
Figure 2.19: Panel prepared for testing in direct tension apparatus. Left: The fork lift has lowered the panel until it is hanging from the top wide flange beam. Middle: The headed rebar can be seen sitting on the top washer which is placed on the flanges of the rotated wide flange. Right: The headed rebar can be seen under the bottom washer which is placed on the web of the rotated wide flange.

faster displacement rates to decrease the testing time. When calculating the thresholds for the displacement rate changes on the day of testing, however, the 1.25 factor was accidentally omitted from the calculation, making the loads at which the displacement rate changed slightly lower. The force per bar with and without the factor of 1.25 is listed in Table 2.2.

<table>
<thead>
<tr>
<th>Yield Stress (ksi)</th>
<th>$A_{\text{bar}}$ (in²)</th>
<th>Force per bar with 1.25 factor (kip)</th>
<th>Force per bar without 1.25 factor (kip)</th>
</tr>
</thead>
<tbody>
<tr>
<td>60</td>
<td>0.44</td>
<td>33</td>
<td>26.4</td>
</tr>
<tr>
<td>72</td>
<td>0.44</td>
<td>39.6</td>
<td>31.68</td>
</tr>
</tbody>
</table>

The masonry panels were loaded at a displacement rate of 0.1 in./min. until the tensile force reached 26.4 kips per bar, at which point the displacement rate was increased to 0.15 in./min. When the tensile force per bar reached 32.7 kips, the displacement rate was increased again to
0.3 in./min. until panel failure. Failure was defined as either significantly decreased load or visual observation of failure in the masonry.

Increasing the strain rate during compressive testing tends to increase the stress of the concrete at failure slightly. At low strain rates (less than $10^{-4}$ strain/sec), however, this relative increase in compressive stress is less than 10% and usually closer to 0% (Schwer, 2009). The highest strain that was induced on the splices in this experiment after the loading rate changed from 0.1 to 0.15 was $1.2 \times 10^{-4}$ strain/sec on the 21-in splice, a conservative number because it only includes the length of the splice in the calculation and ignores the length of the bar not included in the splice. This means that the worst case increase of reported ultimate strength after yielding of the bar due to the earlier change in displacement rate would be about 10% for the 21-in splices, and it is probable that there was no increase in strength at all. Therefore, the yield strengths reported in this thesis are not reduced to account for this increased strain rate error because it most likely did not affect the data.
CHAPTER 3. RESULTS AND ANALYSIS

3.1 Preliminary Testing

3.1.1 Steel Reinforcement

The steel reinforcement was tested in tension to confirm the specified yield stress. Stress-strain curves were developed for each of four samples: three straight bar sections and one headed bar section. The stress-strain curve for Sample 3 is shown in Figure 3.1. Stress-strain curves for the other three samples are included in Appendix A. It appears that Sample 3 and the other bar samples experienced small slippage near yielding which did not affect the yield stress of the bar but caused the 0.2\% offset method to produce falsely low values for the yield stress, averaging 59.9 ksi. Thus, the tensile yield strength $f_y$ was obtained by visual observation of the yield plateau on the stress-strain curve; results are shown in Table 3.1. Since the purpose of the bar testing was to ensure that the bars were at least as strong as the specified yield strength, the results obtained by visual inspection are reasonable.

<table>
<thead>
<tr>
<th>Sample</th>
<th>Specified Tensile Yield Strength, $f_y$ (ksi)</th>
<th>Measured Tensile Yield Strength, $f_y$ (ksi)</th>
<th>Specified Tensile Ultimate Strength, $f_u$ (ksi)</th>
<th>Measured Tensile Ultimate Strength, $f_u$ (ksi)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>60</td>
<td>68</td>
<td>90</td>
<td>94.1</td>
</tr>
<tr>
<td>2</td>
<td>60</td>
<td>69</td>
<td>90</td>
<td>94.4</td>
</tr>
<tr>
<td>3</td>
<td>60</td>
<td>69</td>
<td>90</td>
<td>94.1</td>
</tr>
<tr>
<td>Headed</td>
<td>60</td>
<td>68</td>
<td>90</td>
<td>94.2</td>
</tr>
<tr>
<td>Average</td>
<td>60</td>
<td>68.5</td>
<td>90</td>
<td>94.2</td>
</tr>
</tbody>
</table>
3.1.2 Preliminary Grout and Masonry Specimens

The testing results from the preliminary masonry and grout prisms are presented in Table 3.2. This includes the average area, average load, and average compressive strength for both types of prisms. Stress displacement curves for the preliminary masonry prisms are included in Appendix A.

<table>
<thead>
<tr>
<th>Prism Type</th>
<th>Average Area (in²)</th>
<th>Average Load (kip)</th>
<th>Average Compressive Strength, ( f'_m ) (psi)</th>
<th>Coefficient of Variation</th>
</tr>
</thead>
<tbody>
<tr>
<td>Masonry</td>
<td>57.80</td>
<td>166</td>
<td>2880</td>
<td>9.2%</td>
</tr>
<tr>
<td>Grout</td>
<td>16.17</td>
<td>76</td>
<td>4680</td>
<td>8.2%</td>
</tr>
</tbody>
</table>

The preliminary stacked masonry prisms grouted with SCG made from the supplier’s mix design were tested to design the splice length of the masonry panel specimens according to Equation 1.10. This design provision for development length includes the compressive strength of the masonry, \( f'_m \), which was determined in the final masonry prisms to standardize the results of panel testing.
3.2 Final Masonry Specimens

3.2.1 Mortar

Table 3.3 lists the mortar cube compressive testing results. Compressive strength was calculated using the cross sectional area and the maximum load at failure, which are listed in the table.

<table>
<thead>
<tr>
<th>Sample</th>
<th>Area (in²)</th>
<th>Maximum Load (kip)</th>
<th>Compressive Strength (psi)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>4</td>
<td>100</td>
<td>2500</td>
</tr>
<tr>
<td>2</td>
<td>4</td>
<td>95</td>
<td>2400</td>
</tr>
<tr>
<td>3</td>
<td>4</td>
<td>100</td>
<td>2500</td>
</tr>
<tr>
<td>4</td>
<td>4</td>
<td>99</td>
<td>2500</td>
</tr>
<tr>
<td>5</td>
<td>4</td>
<td>89</td>
<td>2200</td>
</tr>
</tbody>
</table>

Average Compressive Strength 2400
Coefficient of Variation 5.2%

3.2.2 Concrete Masonry Units

Three plain ungrouted single masonry blocks as well as three stacked ungrouted masonry prisms were tested in compression according to ASTM C140 (2018), Standard Test Methods for Sampling and Testing Concrete Masonry Units and Related Units and ASTM 1314 (2016), Standard Test Method for Compressive Strength of Masonry Prisms. These tests did not affect any research parameters but the results are included for completion. The CMU blocks were from the same batch as those used to construct the masonry panels. Compression testing of the single blocks and stacked prisms was conducted 35 days after construction of the panels. The stacked prisms were tested 43 days after the mortar was made. This was a different batch of mortar than the mortar used for the panels, but was also Type S mortar. Table 3.4 presents the results for the compression testing of the single CMU blocks and Table 3.5 presents the results for the stacked ungrouted masonry prisms. The average compressive strength of the single blocks was 2950 psi, and the average
compressive strength of the stacked ungrouted masonry prisms was 2970 psi. Figure 3.2 shows graphs of stress vs. strain during testing for all six specimens. The beginning of each curve was modified to match the linear segment of the stress-strain curve, removing the differences in seating at the beginning of testing and allowing a better comparison of the behavior of the specimens. A failed single CMU block and a failed hollow stacked CMU prism are shown in Figures 3.3 and 3.4, respectively.

Table 3.4: Compressive Testing of Single Hollow CMU Blocks Results

<table>
<thead>
<tr>
<th>Sample</th>
<th>Measured Area (in²)</th>
<th>Maximum Load (kip)</th>
<th>Compressive Strength (psi)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>36.05</td>
<td>104</td>
<td>2900</td>
</tr>
<tr>
<td>2</td>
<td>39.86</td>
<td>119</td>
<td>2980</td>
</tr>
<tr>
<td>3</td>
<td>34.22</td>
<td>102</td>
<td>2980</td>
</tr>
</tbody>
</table>

Average Compressive Strength 2950
Coefficient of Variation 1.7%

Table 3.5: Compressive Testing of Ungrouted Stacked CMU Prisms Results

<table>
<thead>
<tr>
<th>Sample</th>
<th>Measured Area (in²)</th>
<th>Maximum Load (kip)</th>
<th>Compressive Strength (psi)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>36.78</td>
<td>104</td>
<td>2810</td>
</tr>
<tr>
<td>2</td>
<td>41.62</td>
<td>103</td>
<td>2460</td>
</tr>
<tr>
<td>3</td>
<td>37.31</td>
<td>135</td>
<td>3630</td>
</tr>
</tbody>
</table>

Average Compressive Strength 2970
Coefficient of Variation 20.1%

3.2.3 Grout

Grout Results

The ready-mix SCG samples that were tested before grouting had a slump flow of 22-in. and a Visual Stability Index (VSI) value of 0. Table 3.6 presents the compression test results for the
grout prisms poured on the same day as the masonry panels and prisms. The table includes compressive strength, curing time before testing, cross-sectional area and the failure mode according to Figure 2.17. Figure 3.5 shows the stress vs. displacement plots for the grout prisms.
Figure 3.4: Failed ungrouted stacked CMU prism.

Table 3.6: Grout Prism Compression Test Results

<table>
<thead>
<tr>
<th>Sample</th>
<th>Cure Time (days)</th>
<th>Area (in²)</th>
<th>Maximum Load (kip)</th>
<th>Failure Mode</th>
<th>Compressive Strength, $f'_m$ (psi)</th>
<th>Coefficient of Variation</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>47</td>
<td>13.89</td>
<td>48</td>
<td>Type 1</td>
<td>3470</td>
<td></td>
</tr>
<tr>
<td>2</td>
<td>47</td>
<td>13.34</td>
<td>49</td>
<td>Type 1</td>
<td>3660</td>
<td></td>
</tr>
<tr>
<td>3</td>
<td>47</td>
<td>14.12</td>
<td>42</td>
<td>Type 4</td>
<td>2990</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>Average Compressive Strength</td>
<td>3370</td>
</tr>
<tr>
<td>4</td>
<td>48</td>
<td>13.14</td>
<td>43</td>
<td>Type 1</td>
<td>3310</td>
<td></td>
</tr>
<tr>
<td>5</td>
<td>48</td>
<td>14.17</td>
<td>47</td>
<td>Type 2</td>
<td>3320</td>
<td></td>
</tr>
<tr>
<td>6</td>
<td>48</td>
<td>14.24</td>
<td>47</td>
<td>Type 1</td>
<td>3270</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>Average Compressive Strength</td>
<td>3300</td>
</tr>
</tbody>
</table>

**Grout Analysis**

Coarse SCG meeting the gradation requirements of ASTM C476 (2016), Standard Specification for Grout for Masonry, and ASTM C404 (2016), Standard Specification for Aggregates for Masonry Grout, was used for all specimens. The average compressive strength of the grout prisms tested at 47 and 48 days was 3370 and 3300 psi, respectively. It would have been expected that the 48-day compressive strength averaged higher than the 47-day compressive strength. The averages are relatively close, however, and the small time difference may not have been enough to
override normal variation among the samples, especially with such a low sample size. This small inconsistency in sample strengths is considered inconsequential to the research outcomes as grout compressive strength was determined for quality control.

ASTM C476 (2016) requires a minimum slump flow of 24-in, a standard that the ready-mix grout did not meet. Water was therefore added to increase the flow. SCG is especially sensitive to any mix design changes, so segregation was a concern as well as over-correction by exceeding the slump flow limit. Some voids were observed along the corners of the grout prisms, but that was likely due to the 90-degree corner. Observation of the stacked masonry prisms and panels after failure confirmed that the grout was well consolidated. No voids were observed on the rounded interface between the grout and the masonry.
### 3.2.4 Grouted Masonry Prisms

**Grouted Masonry Prisms Results**

Six stacked masonry prisms were tested on panel testing days, and the test results are tabulated in Table 3.7. This includes the curing time before testing, cross sectional area, load at failure, calculated compressive strength and the failure mode according to Figure 2.16. The stress-displacement response for all the prisms is shown in Figure 3.6, with the beginning of the curves modified to match the linear segment, removing any differences in seating and capping. Photos of the failed prisms are included in Appendix B.

<table>
<thead>
<tr>
<th>Sample</th>
<th>Cure Time (days)</th>
<th>Area (in²)</th>
<th>Maximum Load (kip)</th>
<th>Failure Mode</th>
<th>Compressive Strength, $f'_m$ (psi)</th>
<th>Coefficient of Variation</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>47</td>
<td>57.86</td>
<td>214</td>
<td>Mode 2</td>
<td>3690</td>
<td></td>
</tr>
<tr>
<td>2</td>
<td>47</td>
<td>57.96</td>
<td>205</td>
<td>Mode 2</td>
<td>3540</td>
<td></td>
</tr>
<tr>
<td>3</td>
<td>47</td>
<td>57.79</td>
<td>203</td>
<td>Mode 3</td>
<td>3510</td>
<td></td>
</tr>
<tr>
<td>4</td>
<td>48</td>
<td>57.96</td>
<td>222</td>
<td>Mode 3</td>
<td>3820</td>
<td>2.7%</td>
</tr>
<tr>
<td>5</td>
<td>48</td>
<td>58.02</td>
<td>217</td>
<td>Mode 1</td>
<td>3740</td>
<td></td>
</tr>
<tr>
<td>6</td>
<td>48</td>
<td>58.20</td>
<td>194</td>
<td>Mode 3</td>
<td>3340</td>
<td></td>
</tr>
</tbody>
</table>

**Grouted Masonry Prisms Analysis**

The masonry prisms tested on days 47 and 48 averaged compressive strengths of 3580 and 3630 psi, respectively. As expected, the average strength of the 48-day prisms was slightly higher. These values were used in Equation 1.7 to calculate the predicted splice capacity according to the linear regression model.
3.2.5 Reinforced Masonry Panels

Reinforced Masonry Panels Results

Three groups of panels were tested. The first group had six panels, each with a splice length of 31-in. The second group had three panels with a splice length of 26-in. The last group had three panels with a splice length of 21-in. During testing, a computer attached to the UTM machine continuously recorded displacement and load. From this data, the maximum load was obtained and ultimate stress per splice was calculated. Table 3.8 summarizes the results from all the panel specimens and includes the ratio of ultimate stress to specified yield stress as well as the failure mode, which was either splitting of the masonry or fracture of the rebar. All the panels reached an ultimate stress greater than 1.25 times the specified yield stress of the steel. The ratio of ultimate stress to average measured yield stress is also included. These ratios are lower because the measured yield stress of the bars was slightly higher than specified. Specimens with failure at less than 1.25 times the measured yield stress of the bar generally failed more quickly and in a less
ductile manner, supporting the idea that failure at 1.25 times the yield strength of the reinforcing bar is considered ductile (Thompson, 1997).

Table 3.8: Reinforced Masonry Panel Testing Results

<table>
<thead>
<tr>
<th>Splice Length (in)</th>
<th>Sample</th>
<th>Failure Mode</th>
<th>Maximum Load per Splice (kip)</th>
<th>Ultimate Reinforcement Stress (ksi)</th>
<th>Ratio of Ultimate to Measured Yield Stress</th>
<th>Ratio of Ultimate to Specified Yield Stress</th>
</tr>
</thead>
<tbody>
<tr>
<td>31</td>
<td>1</td>
<td>Masonry Splitting</td>
<td>37.2</td>
<td>84.6</td>
<td>1.23</td>
<td>1.41</td>
</tr>
<tr>
<td></td>
<td>2</td>
<td>Masonry Splitting</td>
<td>39.0</td>
<td>88.7</td>
<td>1.29</td>
<td>1.48</td>
</tr>
<tr>
<td></td>
<td>3</td>
<td>Masonry Splitting</td>
<td>35.5</td>
<td>80.7</td>
<td>1.18</td>
<td>1.34</td>
</tr>
<tr>
<td></td>
<td>4</td>
<td>Rebar Fracture</td>
<td>40.6</td>
<td>92.2</td>
<td>1.35</td>
<td>1.54</td>
</tr>
<tr>
<td></td>
<td>5</td>
<td>Masonry Splitting</td>
<td>37.7</td>
<td>85.7</td>
<td>1.25</td>
<td>1.43</td>
</tr>
<tr>
<td></td>
<td>6</td>
<td>Masonry Splitting</td>
<td>37.4</td>
<td>85.0</td>
<td>1.24</td>
<td>1.42</td>
</tr>
<tr>
<td></td>
<td>Average</td>
<td></td>
<td>37.9</td>
<td>86.2</td>
<td>1.26</td>
<td>1.44</td>
</tr>
<tr>
<td>26</td>
<td>1</td>
<td>Masonry Splitting</td>
<td>38.2</td>
<td>86.9</td>
<td>1.27</td>
<td>1.45</td>
</tr>
<tr>
<td></td>
<td>2</td>
<td>Rebar Fracture</td>
<td>40.6</td>
<td>92.2</td>
<td>1.35</td>
<td>1.54</td>
</tr>
<tr>
<td></td>
<td>3</td>
<td>Masonry Splitting</td>
<td>34.8</td>
<td>79.2</td>
<td>1.16</td>
<td>1.32</td>
</tr>
<tr>
<td></td>
<td>Average</td>
<td></td>
<td>37.9</td>
<td>86.1</td>
<td>1.26</td>
<td>1.43</td>
</tr>
<tr>
<td>21</td>
<td>1</td>
<td>Masonry Splitting</td>
<td>34.7</td>
<td>78.9</td>
<td>1.15</td>
<td>1.32</td>
</tr>
<tr>
<td></td>
<td>2</td>
<td>Masonry Splitting</td>
<td>34.8</td>
<td>79.1</td>
<td>1.15</td>
<td>1.32</td>
</tr>
<tr>
<td></td>
<td>3</td>
<td>Masonry Splitting</td>
<td>35.2</td>
<td>80.1</td>
<td>1.17</td>
<td>1.33</td>
</tr>
<tr>
<td></td>
<td>Average</td>
<td></td>
<td>34.9</td>
<td>79.3</td>
<td>1.16</td>
<td>1.32</td>
</tr>
</tbody>
</table>

The predicted splice capacity for each masonry panel was calculated using Equation 1.7. The two changing variables were splice length and masonry compressive strength, \( f'_m \). The masonry compressive strength was the average compressive strength of the masonry prisms that were tested on that day. In the case of the first 31-in. panel, however, the compressive strength of the masonry prism tested the next morning was used. This was permitted because the panel was tested at the end of day 46, and the prisms were tested at the beginning of day 47, making the time gap less than 14 hours which was considered insignificant. The actual splice capacity was generally larger than predicted by Equation 1.7. Figure 3.7 shows the measured capacity of each masonry
Figure 3.7: Measured vs. predicted splice capacity.

Panels generally failed by cracking through the mortar and through one or two blocks, as shown in Figure 3.8. A photograph of each panel after failure is included in Appendix B.

The stress vs. displacement curves for the 31-in., 26-in., and 21-in. test groups are shown in Figures 3.9, 3.10, and 3.11, respectively. A combined graph of all the stress vs. displacement curves is included in Figure 3.12, and average curves from each splice length are shown in Figure 3.13. Figures 3.12 and 3.13 are valuable for comparing the behavior of the different splice lengths. All the curves were shifted and modified at the beginning such that an extension of the linear segment crossed the x-axis at a displacement of zero. This helped alleviate any differences between panel curves due to slightly different snugness at the start of testing. The curves were
modified at the end to drop to zero after failure to make it easy to observe ultimate displacement by looking at the plot. The average curve uses the average of all the curves present until it reaches the average ultimate stress. Average ultimate stress was reached before average ultimate displacement in all three cases. A second vertical line represents the average ultimate displacement for each splice length. The regions between the vertical drop at average ultimate stress and average displacement are hatched to represent a range of displacements that could define the end of the curve, depending on whether the average curve terminates at average ultimate stress or average ultimate displacement.

General inspection of the grout-masonry and grout-rebar interface after testing confirmed that self-consolidation of the grout was achieved. In addition, no slip was observed and no voids were present adjacent to the reinforcement or masonry. Figure 3.14 is a photograph of the SCG-rebar interface, showing a clear imprint of the bar with no slippage apparent. Figure 3.15 shows one of the panels which failed in a manner to expose the grout-CMU interface, which was smooth and without voids.
Figure 3.9: Stress vs. displacement for the six panels with 31-in. splices.

Figure 3.10: Stress vs. displacement for the three panels with 26-in. splices.
Reinforced Masonry Panels Analysis

Each panel was designed and carefully constructed to be as symmetric as possible to avoid any eccentricities. All panels were constructed and tested with the same procedures. Tensile strength of masonry panels was relatively uniform within test groups. Minor differences in specimen splice lengths, material properties, rebar positioning, and panel placement in the testing apparatus resulted in the small variance among the data. As expected, the average capacity of the panels decreased with reduced lap length.

Each panel displayed similar load-displacement behavior. A linear-elastic region was followed by a plastic region due to stretching and cracking, and finally a drastic decrease in carrying capacity or rupture. The cracking was identifiable well before failure on most specimens, and usually started with a stair-step pattern along the mortar joints, as shown in Figure 3.16. It was also common for the crack to propagate through one or two of the blocks, especially in longer splice length specimens.
Panels with longer splice lengths were generally more ductile than panels with shorter splice lengths. This can be observed from a comparison of curves from the three test groups in Figures 3.12 and 3.13. The 31-in. and 26-in. test groups displaced further on average after yielding than did the 21-in. test group. Figure 3.16 shows failed 21-in. splice length panel #2, which lost its load-carrying capacity relatively quickly after yielding and cracking were observed. More ductile specimens with longer splice lengths took longer to fail after cracking began, and often ruptured much more catastrophically, as shown in 3.17.

The failure mode of the masonry panels was usually masonry splitting, but one specimen in the 31-in. test group and one specimen in the 26-in. test group failed by the fracturing of the bar. Both specimens also had accompanying masonry splitting, but slow motion video was used to confirm that the bar was the first to break. The stress vs. displacement curves for these two specimens reached approximately 92 ksi per bar, exceeding the specified tensile ultimate strength (90 ksi) and approaching the average measured ultimate strength of the bar (94.2 ksi). Testing
Figure 3.13: Average stress vs. displacement for each splice length. The two vertical lines for each group are located at average ultimate stress and average ultimate displacement.

Figure 3.14: SCG-rebar interface. No slippage is apparent.
Figure 3.15: SCG-CMU interface. The smooth rounded edge of the grout within the masonry cavity indicates complete consolidation of the SCG.

Figure 3.16: Stair-step patterned cracking in 21-in. test group masonry panel #2.
was designed so that the two reinforcing bars in the panels shared the tensile load equally during testing, but even a slight difference between the load on the two bars would mean that the bar that fractured reached ultimate capacity. This indicates that the masonry bond in these two specimens was stronger than the reinforcement. As previously mentioned, the NCMA analysis excluded data from samples with fractured reinforcement from their linear regression analysis because the actual capacity of the SCG-reinforcement bond was unknown, the measured bond capacity being limited by the ultimate strength of the bar (Thomas et al., 1999). This failure type in the lab is not representative of real conditions because the rebar is always completely enclosed in grout in actual structures. In this thesis, however, the fractured rebar specimens are included in the analysis because even if the reported bond strength was not accurate it was conservative. Dropping these two data points would unnecessarily decrease the average bond strength of the splices.

All panels developed at least 125% of the specified yield stress of the bar. The 31-in., 26-in., and 21-in. test groups averaged a tensile stress of 144%, 143%, and 132% of the specified yield stress of the bar, respectively, at failure.

Figure 3.18 shows the data from this research using #6 bars in SCG combined with data from the similar research program conducted by Roper (2018) using #5 bars in SCG. Figure 3.19 is the same plot with the axes beginning at zero for perspective. The black line has a slope of one
and shows where the data points would fall if the measured bond strength of the splices matched the capacities predicted by the regression analysis represented by Equation 1.7. The gray line has a slope of 1.19, which was the average ratio of measured to predicted strength among the #6 bar panels. Each set of data points also has its own linear trendline. It appears that splices in self-consolidating grout perform slightly better than splices in conventional grout. The group of panels reinforced with #6 bars performed even better than the panels with #5 bars. The two groups of data points do not line up in a parallel manner with the predicted capacity according to Equation 1.7. One difference between the two experiments was curing time, which increased $f'_m$, the strength of the masonry assemblage. This variable may affect the splice capacity more than is accounted for in the regression equation. Another difference between the two groups was the number of courses in the masonry panels. The #5 bar panels were three courses tall and the #6 bar panels were four courses tall, but only the splice length is considered in the regression equation. Equation 1.7 was developed using conventional grout, and the differences between that regression and one using SCG could be more substantial than a simple multiplicative factor. A third set of data using #4 or #7 bars would be helpful to establish a broader relationship and fit a more accurate curve.
Figure 3.18: Measured vs. predicted splice capacity combined data with Roper (2018). Splices grouted with SCG tend to perform better than expected according to the linear regression (Equation 1.7), although the relationship is not linear between different bar sizes.
Figure 3.19: Measured vs. predicted splice capacity combined data with Roper (2018). This is the same as the previous plot with the axes beginning at zero.
CHAPTER 4. CONCLUSIONS

4.1 Summary

A total of twelve masonry panels were constructed and tested to analyze the tensile strength of splices in masonry grouted with SCG and reinforced with #6 reinforcing bars. Preliminary testing of the grout yielded the parameters used to calculate the required development length according to existing design requirements. Six panels were constructed to meet these requirements, and two sets of three panels were constructed with shorter splice lengths than required. The panels were tested in direct tension at a monotonic controlled displacement rate until failure. The measured ultimate splice capacities were compared to the predicted capacities according to a multiple linear regression model from previous research (Thomas et al., 1999). The ultimate strengths of the splices were compared with the design requirement for a splice to develop 125% of the yield strength of the reinforcing bars.

4.2 Conclusions

This was not a comprehensive study on the behavior of spliced reinforcement in SCG; however, the following conclusions are made based on this research. These conclusions are limited to the SCG mix design and configuration investigated in this research using #6 bars.

1. 21-in., 26-in., and 31-in. splices using #6 bars develop the minimum 125% of the specified yield strength of the bar required by code.

2. Splices in SCG perform approximately 19% better than predicted by the regression model (Thomas et al., 1999) used to derive the TMS 2016 code equation for splice length (American Concrete Institute, 2016).
3. The TMS 2016 code equation for splice length predicts conservative reinforcement splice lengths in masonry grouted with SCG and reinforced with #6 bars.

4.3 Recommendations for Future Research

In order to obtain more confidence in the findings of this research program, similar research will need to be conducted and the results combined with this and comparable studies (Roper, 2018). The testing strategy should also align with similar studies so that results will correspond with those of previous research programs (Hammons et al., 1994; Thompson, 1997; Thomas et al., 1999; Roper, 2018). More research could justify a development length reduction factor when SCG is used in masonry.

1. This study used #6 reinforcing bars, adding to the database of SCG panels tested by Roper (2018) that were reinforced with #5 bars. Additional bar sizes should be used to generate a larger data set and better understanding of the effect of SCG on splice performance.

2. This study used SCG from the same supplier as Roper (2018). Examining the effect of different SCG suppliers and mix designs on the performance of the splices would be valuable.

3. More precise instrumentation could lead to a more accurate model of splice performance. For example, strain gauges installed on the reinforcement would enable observation of bond stress distributions along the length of the splices.

4. This study did not account for the bond strength contribution from bar segments above and below the splice. One of two strategies should be implemented to correct this in future research. First, individual bars that are embedded in SCG could be tested in tension to measure this contribution to bond strength. Second, a bond breaker could be installed above and below the splices.
REFERENCES

American Concrete Institute (2016). *Building Code Requirements for Masonry Structures*. TMS 402-16, Longmont, CO. 13, 54


APPENDIX A. RESULTS
Figure A.1: Stress-strain curve for steel reinforcing bar sample 1.

Figure A.2: Stress-strain curve for steel reinforcing bar sample 2.
Figure A.3: Stress-strain curve for Headed steel reinforcing bar sample.

Figure A.4: Stress vs. displacement for preliminary stacked masonry prisms grouted with SCG.
APPENDIX B. SPECIMEN SCHEMATICS AND PHOTOGRAPHS
Figure B.1: 21-in. splice length test group panel schematic.
Figure B.2: 31-in. splice length test group panel schematic.

Figure B.3: 31-in. panel #1.
Figure B.4: 31-in. panel #2.

Figure B.5: 31-in. panel #3.
Figure B.6: 31-in. panel #4 with broken rebar.

Figure B.7: 31-in. panel #5.
Figure B.8: 31-in. panel #6.

Figure B.9: 26-in. panel #1.
Figure B.10: 26-in. panel #2 with broken rebar.
Figure B.11: 26-in. panel #3.
Figure B.12: 21-in. panel #1.

Figure B.13: 21-in. panel #2.
Figure B.14: 21-in. panel #3.

Figure B.15: Stacked masonry prism #1, showing failure mode 2.
Figure B.16: Stacked masonry prism #2, showing failure mode 2.

Figure B.17: Stacked masonry prism #3, showing failure mode 3.
Figure B.18: Stacked masonry prism #4, showing failure mode 3.

Figure B.19: Stacked masonry prism #5, showing failure mode 1.
Figure B.20: Stacked masonry prism #6, showing failure mode 3.

Figure B.21: Grout prism #1, showing failure type 1.
Figure B.22: Grout prism #2, showing failure type 1.

Figure B.23: Grout prism #3, showing failure type 4.
Figure B.24: Grout prism #4, showing failure type 1.

Figure B.25: Grout prism #5, showing failure type 2.
Figure B.26: Grout prism #6, showing failure type 1.