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Reinforcing Bar Splice Performance in Masonry with Self-Consolidating Grout

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ABSTRACT

Reinforcing Bar Splice Performance in Masonry with Self-Consolidating Grout

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The use of self-consolidating grout in reinforced masonry construction provides various advantages such as reduced labor, faster construction, decreased noise pollution and better structural response. This is a relatively new building material however, and little research on self-consolidating grout's structural properties has been conducted. The purpose of this study was to analyze the performance or bond capacity of steel reinforcing bar splices in masonry with self-consolidating grout.

Twelve masonry panels approximately 40 in. wide and 32 in. tall consisting of Type S mortar and concrete masonry units grouted with self-consolidating grout and No. 5 steel reinforcing bars were constructed with splice lengths as prescribed by the current design equation and splices that were slightly shorter. Test Group 1 consisted of six reinforced masonry panels with the code required lap length while Test Groups 2 and 3 had splices two and four inches shorter, respectively. The lap-splices were tested in pure tension to determine if they would fully develop the code mandated stress of 125% of the specified yield strength of the reinforcing bars. More samples were tested with the code required development length to verify if the current provision is adequate for design and the other two groups were used to explore if the required capacity could be achieved with shorter splices.

All lap-splices developed the minimum required stress, even those with splices shorter than required by the design equation. For masonry with self-consolidating grout containing No. 5 bars in the specific configurations tested, the current design equation was shown to be adequate for calculating development length. Testing indicates that a reduction in required splice length for masonry with self-consolidating grout is possible.

Keywords: self-consolidating grout, development length, masonry

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1 INTRODUCTION

1.1 Masonry Construction

Over the last century, building construction and structural design have benefitted from advancements made through research and experience. This progress has come through improved building materials and better understanding of structural response. Structural steel and reinforced concrete have become the preferred building materials for high rise construction and many modern structural wonders would not be possible without these materials (Bjorhovde, 2004). For many projects however, there are more economical options that are easier to work with such as timber or masonry. Until the 20th century, masonry was the predominant building material and is still widely used as infill or the load bearing wall system for buildings of low to medium height in residential, industrial, commercial and educational applications. Ease of construction, versatility, aesthetics, fire protection, thermal and sound insulation and durability are some of the characteristics that make masonry an attractive option when compared to the alternatives of steel, concrete or timber (Hendry 2001).

Structural masonry is a composite material that utilizes masonry units, reinforcing steel, mortar and grout. The strength and ductility of the masonry are dependent on the interaction between these components. Masonry units can be made from clay, concrete, glass and stone but structural construction typically uses concrete masonry units (CMUs). Most masonry units will have hollow sections or cells in them for the placement of both reinforcement and grout. The addition of grout and reinforcement provide masonry with additional shear, axial and flexural capacity to resist gravity and lateral loading induced by wind, seismic or building and occupant weight. Mortar is used to bond the units together and also accounts for any dimensional variance from block to block.

Grout is a low compressive strength concrete that is highly flowable and has a smaller nominal aggregate size. Because grout has almost no tensile capacity, the bond between the grout, masonry units and reinforcing steel is vital to the overall strength and performance of the masonry system. Under extreme loading conditions leading to structural failure, it is desirable that the masonry behaves in a ductile manner to provide building occupants with adequate time to reach safety. To prevent the premature brittle failure condition of rebar pullout from the grout, it is necessary that the chemical and physical bond between the grout and reinforcement is sufficient to resist these pulling forces. To ensure the requisite capacity for pullout, the reinforcement is embedded a code specified distance, or development length, into the grout. This length must provide a bonding surface between the grout and reinforcement such that a minimum stress of 1.25 times the yield stress of the reinforcement is developed (The Masonry Society, TMS 402 2016). The equation to calculate the development length of reinforcement embedded in grout is given in the *Building Code Requirements for Masonry Structures* (TMS 402 2016). During construction, the use of one continuous length of rebar for the required height of the structure is not feasible and necessitates overlap of the reinforcement. The required overlap or splice length is the same as the development length prescribed in the building code. The terms development length, lap-splice, lap and splice will be used synonymously throughout this thesis.

1.2 Self-Consolidating Concrete

Self-consolidating grout (SCG) is a type of self-consolidating concrete (SCC) which is a specially proportioned concrete that flows under its self-weight and fills in the formwork without the need of any internal or external vibration. While being highly fluid, SCC needs to be sufficiently cohesive to prevent segregation or blockage of aggregates during flowing. The enhanced cohesiveness can ensure better suspension of solid particles in the fresh concrete and, therefore, good deformability and filling capability during the spread of fresh concrete through various obstacles (Okamura and Ozawa 1995; Ozawa et al. 1995). SCC was initially developed in Japan during the 1980s in response to concrete durability concerns associated with the lack of laborers skilled in proper consolidation technique (Ozawa et al. 1989). However, SCC has become widely used throughout the world with special applications such as precast, prestressed, bridge decks, high congestion reinforcement and exposed architectural surface (Daczko 2012).

Some of the advantages of SCC include reduced labor, faster construction, decreased noise pollution, increased finish quality and better structural response. SCC is much more sensitive to variations than conventional concrete however, and even slight differences in aggregate gradation, water-cement ratio, mixing procedure, and admixture usage can produce large variances in the stability of the end product. The stability of fresh SCC refers to its ability to resist bleeding, sedimentation, and segregation which depend on the cohesiveness and viscosity of the mixture (Khayat 1998).

1.3 Research Motivation

Self-consolidating grout (SCG) is a special subset of SCC, whose maximum nominal aggregate size is limited to less than ½ in., containing a special class of high-range water-reducing admixtures called polycarboxylates and may include a viscosity-modifying admixture (NCMA 2007). As availability from ready-mix producers increases, more masonry contractors are choosing SCG over conventional grout because it does not require mechanical vibration to fully consolidate. While the material is costlier due to the additional admixtures, the reduced time of construction can provide an overall economic advantage. As the use of SCG in masonry construction increases, governing building authorities may allow larger lift heights thereby reducing labor significantly. Furthermore, the structural performance of masonry will increase as areas of high reinforcement congestion will be completely encased in grout.

While there is an abundance of research on SCC, the available information on the fresh and hardened properties of SCG is limited because of its more specialized nature. However, the narrow openings and large grout volume in masonry walls provide an ideal application for SCC. The equation for reinforcement development length in masonry was derived using conventional grout that underwent mechanical consolidation. Several studies have been performed on the bond strength of steel reinforcement in SCC and initial results indicate an increased pullout capacity compared to conventional concrete (Khayat 1998; Sonebi et al. 2000; Chan et al. 2003; Castel et al. 2006; Hossain and Lachemi 2008; Hassan et al. 2009). Similar research has not yet been conducted on SCG however, and a modification factor, reducing the required splice length, could be adopted into the design equation.

Because of the limited research performed on the properties of self-consolidating grout, an experimental program was designed and conducted. The objective of this study was to investigate how the use of SCG affects the bond performance of steel reinforcement splices in masonry. Fully grouted reinforced masonry panels were constructed using development lengths prescribed by the current design equation as well as reduced lengths. The splice capacities were then determined by tension testing. Although the lap-splices were tested in tension, previous research has determined that the compressive strength of the masonry factors in the performance of lap-splices in tension. As such, grouted masonry prisms were also constructed and tested concurrently with the reinforced masonry panels to determine the compressive strength of masonry at the time of testing. Masonry panel test results from different days were normalized using the compressive strength of masonry which typically increases as the cure time is extended. The measured loads at failure were utilized to calculate the internal stress of the masonry to determine if a minimum of 1.25 times the yield stress of the reinforcement was developed.

1.4 Scope of Research

This research program was limited to the testing of lap-splices for No. 5 Grade 60 reinforcing bars in 8-inch concrete masonry units grouted with self-consolidating grout. Bars were placed at the center of the masonry cell with lap splices positioned at the mid-height of the panel. Specimens were tested to failure in direct tension under a monotonic load at a displacement controlled rate.

Preliminary masonry prisms were constructed to determine the compressive strength of the masonry using the SCG mix design provided by the ready-mix supplier. Coarse and fine aggregate that was previously supplied from the same producer were utilized in conjunction with the specified admixtures to produce the grout. The compression strength obtained from the masonry prisms was utilized to design lap-splices according to the building code requirements (TMS 2016).

Twelve masonry panels that were five cells wide and three courses tall (40 in. x 32 in.) were constructed with No. 5 reinforcement with the bars placed at 16 inches on center. The required tension splice length was calculated utilizing the compressive strength of masonry obtained from the preliminary testing. The lap-splices for six of the panels were installed per the prescribed length to verify conformance to the existing requirements. Two other test groups of three panels each were assembled with decreased splice lengths to determine if the same capacity would be obtained. Twelve masonry prisms were constructed and grouted concurrently with the panels. After grouting, the panels and prisms were allowed to cure for at least 28 days before testing. Not all panels could be tested at 28 days and some cured longer before testing. The lap-splices within the panels were tested in pure tension with all results being normalized by the compressive strength of the masonry prisms tested in the same day. Results were analyzed to determine the bond strength developed using SCG.

1.5 Outline

This thesis is organized in six chapters. A literature review of previous research and corresponding code requirements and standards are presented in Chapter 2. Chapter 3 describes the material selection, specimen construction, testing methods and preliminary work. The results obtained from testing are presented in Chapter 4. Chapter 5 presents the analysis of the test results as well as an examination of current code requirements. Conclusions from this research and future research recommendations are given in Chapter 6.

2 LITERATURE REVIEW

The following sections present a brief literature review of background information such as SCG testing, lap length design equation development and applicable test standards. The material presented includes work from various organizations such as research reports, building codes, test standards, and textbooks. This chapter is not an exhaustive representation of all pertinent knowledge but rather contains a summary of the subject matter most vital to our current research.

2.1 Self-Consolidating Grout

SCG is a relatively new material specific to masonry construction which, is the probable cause for the limited data available. Despite provisions being included in past and current building codes, the properties and performance of SCG are not as well understood as conventional grout. The following section provides an overview of the applicable research.

2.1.1 National Concrete Masonry Association

In 2006, the National Concrete Masonry Association (NCMA) began a two-phase research program on SCG (NCMA 2006). The first phase of the research examined the behavior and performance of SCG. Five masonry walls were constructed using 8 in. CMUs to a height of 12.67 ft. and grout columns were poured with either normal grout or SCG from a ready-mix supplier. Individual grout columns were designated for mortar fins removed or not removed, mechanical consolidation or no mechanical consolidation and reinforcement or no reinforcement. Grout specimens were cut from the walls and evaluated using visual inspection or physical testing.

Visual observation indicated that the SCG exhibited no noticeable aggregate segregation and filled all voids, even in reinforcement congested areas. Voids however, were present beneath the reinforcement in the unconsolidated conventional grout. Grout compressive strength was determined for specimens extracted from the walls and from cast prisms. Results showed that the required compressive strength in SCG was achieved without mechanical consolidation even with multiple layers of horizontal reinforcement or mortar fins. Shear bond between the grout and CMU was also determined for removed wall samples and met the minimum requirement of the California State Building Code. The first phase of the research showed that SCG without mechanical consolidation performed as well or better than normal grout for masonry wall construction.

The second phase of the SCG research was conducted to develop some expertise in the development of SCG mix design (NCMA 2007). The existing targets and procedures for developing SCC were found to be applicable to SCG as a special subset of SCC. Prototype mix designs for coarse and fine SCG were developed through a trial and error procedure while analyzing the effects of aggregate gradation, cementitious material ratios, water content, and various admixtures. Observations indicated that SCG, like SCC, is very sensitive to these parameters and an extra measure of control over processes and materials was required to produce a high-quality repeatable mix. The second phase of the research indicated that the available raw material currently used for conventional grout was feasible to also produce SCG. Proper quality control however, should be exercised to ensure that SCG from ready-mix suppliers used in masonry construction is consistent and achieves the design requirements.

2.2 Development Length

The development length design equation has undergone significant changes and the current form is relatively new (TMS 402 2016). Various organizations performed independent test programs to model the bond between steel reinforcement and grout. The accessible data compiled from various test iterations were eventually fit using a linear regression to form the present design model. The following sections present a chronological synopsis of these test programs that generated the present-day specification.

2.2.1 Masonry Limit States Design Standard

Prior to the 1980's limited data was available on the splice length for masonry construction. Early requirements in masonry codes likely originated from studies done on reinforced concrete. However, many differences exist between reinforced masonry and reinforced concrete, such as water drawn out of the grout by the CMUs, bridging within cores, and weak planes at the CMUs-mortar interface. Recognizing these differences, the Masonry Limit-States Design Standard (MLSDS) (Hammons et al. 1994) began a program to develop a requirement for lap splices. The standard initially adopted an equation developed by The U.S. – Japan Coordinated Program for Masonry Building Research (Soric and Tulin 1987). During this research program, 90 specimens with 6 in. CMUs using #4 and #7 reinforcing bars were tested and models were developed representing the CMUs as thick-walled pressure vessels and the bond stress in the grout as hydraulic pressure. Figure 2-1 shows the typical masonry specimens.

The resulting equation for development length from that research program is given in Equation 2-1.

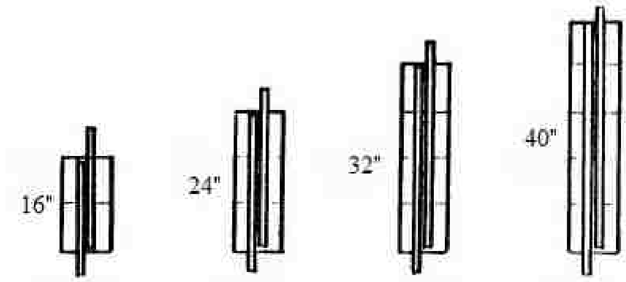

Figure 2-1: Soric & Tulin Reinforced Masonry Specimens

$$l_d = \frac{C\, d_b^{\,2}\, f_y}{(t - d_b)\, f_{gt}} \tag{2-1}$$

where: t = masonry thickness, in.;
f_{gt} = grout tensile strength, psi;
d_b = reinforcing bar diameter, in.;
f_y = reinforcing bar yield strength, psi; and
C = empirical constant.

The empirical constant C accounts for the nonuniformity of the bond stresses along the length of the splice. A mean value of 1.75 for the constant C was obtained by Soric and Tulin (1987), based on the requirement for the lap splice to develop at least 125 percent of the reinforcing bar yield strength. The MLSDS used this value for C and assumed a grout tensile strength of 400 psi (2.75 MPa). With these values, the proposed expression was modified to Equation 2-2.

$$\varphi\, l_d = \frac{0.0045\, d_b^{\,2}\, f_{ye}}{(t - d_b)} \geq 12 \text{ inches} \tag{2-2}$$

where: φ = 0.8 (capacity reduction factor)
f_{ye} = expected yield strength of the reinforcing bar.

Equation 2-2, adopted in the draft MLSDS, resulted in significantly smaller development lengths than those included in other codes and standards. The equation also differed from the

Uniform Building Code (UBC) and Masonry Standards Joint Committee (MSJC) requirements in that it considered a splitting masonry failure mode in addition to a bond stress or rebar pull-out failure.

2.2.2 Construction Productivity Advancement Research

Under the Construction Productivity Advancement Research (CPAR) Program, the U.S. Army Corps of Engineers and Atkinson-Noland and Associates (Hammons et al. 1994) participated in a cooperative effort to study reinforced masonry focusing on lap-splices, tension-stiffening behavior and in-plane biaxial loading. The research on development length analyzed the validity of the MLSDS proposed equation as well as requirements from the Uniform Building Code (International Conference of Building Officials 1992) and MSJC masonry code (MSJC 1992). Researchers investigated parameters believed to contribute to the strength and ductility of lap splices such as masonry unit width, masonry unit type, reinforcement bar diameter and lap length. A total of 124 specimens, in 62 combinations of these parameters, were constructed using single cell masonry units in stack bond to create a vertical cell. The range of lap splice lengths and specimen sizes for concrete masonry units is presented in Figure 2-2.

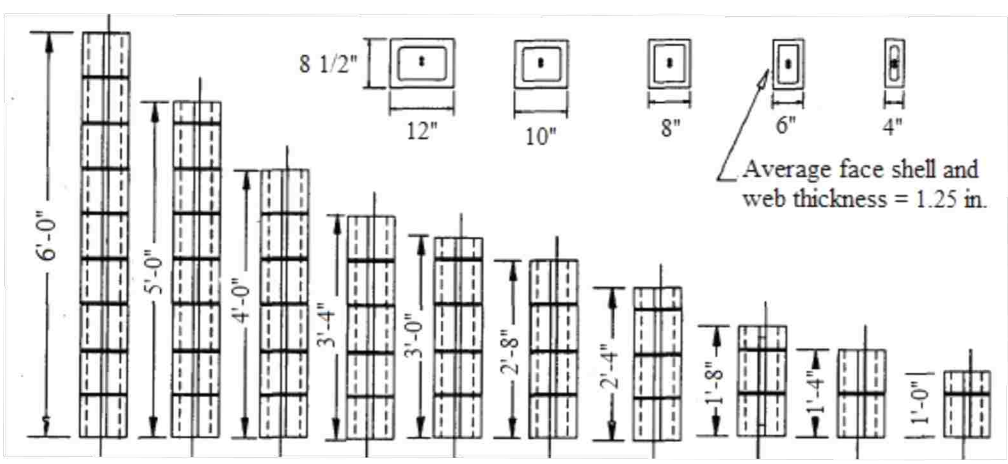

Figure 2-2: CPAR Lap Lengths and Specimen Sizes

The testing apparatus used was designed to test the specimens in pure tension, but an unintended eccentricity was created by the adjacently placed reinforcement that formed the lap splice. A schematic of the testing apparatus is presented in Figure 2-3.

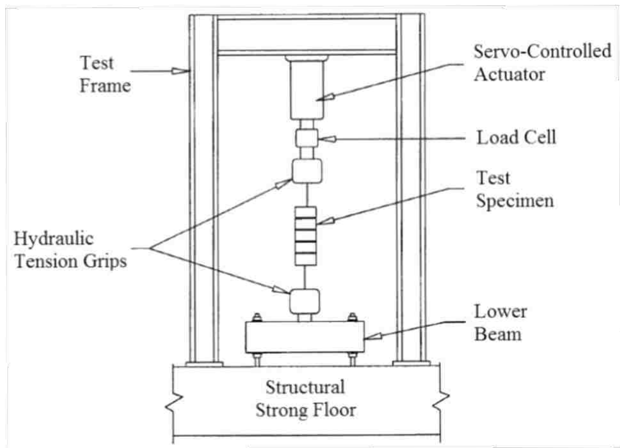

Figure 2-3: CPAR Lap-Splice Test Apparatus

Researchers observed that the minimum cover of the rebar had a significant effect on the capacity of lap splices and that samples with larger bar sizes tended to fail earlier than those with smaller bar diameters. The researchers concluded that the proposed equation (Equation 2-2) generally underestimated the required splice length, especially for larger bar sizes. However, if a different value for the coefficient C, which accounts for the uneven distribution of bond stresses, was used for each bar size, the equation would accurately predict the required splice length.

2.2.3 National Concrete Masonry Association

In 1994, the Uniform Building Code (UBC) introduced a new equation for development length in masonry (International Conference of Building Officials 1994). The splice length strength design expressions are given in Equations 2-3 and 2-4.

$$l_{de} = \frac{0.15\, {d_b}^2\, f_y}{K\sqrt{f'_m}} \leq 52 d_b \tag{2-3}$$

and:

$$l_d = \frac{l_{de}}{\varphi} \geq 12\ inches \tag{2-4}$$

where: l_d = development length of reinforcing bar, in.;

φ = strength reduction factor; equal to 0.80;

l_{de} = basic development length, in.;

d_b = bar diameter, 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 = 28-day compressive strength of masonry, psi.

Also, in 1994, the NCMA began a test program to evaluate the various available design methods such as the UBC requirement and the proposed MLSDS equation (Thomas et al. 1999). The research program investigated the effect of different combinations of masonry material strength, splice length, cover depth, and bar diameter. Masonry panels were constructed in running bond using both 8-inch and 12-inch CMUs with No. 4 through No. 9 reinforcing bars. Test groups of three specimens per set were constructed with various combinations of splice length and cover. Each panel contained two sets of spliced bars to avoid eccentric moments and were pulled in direct tension. A steel frame, laid horizontally, with hydraulic jacks coupled to the reinforcement, was used to test the splices. Figure 2-4 shows the testing apparatus.

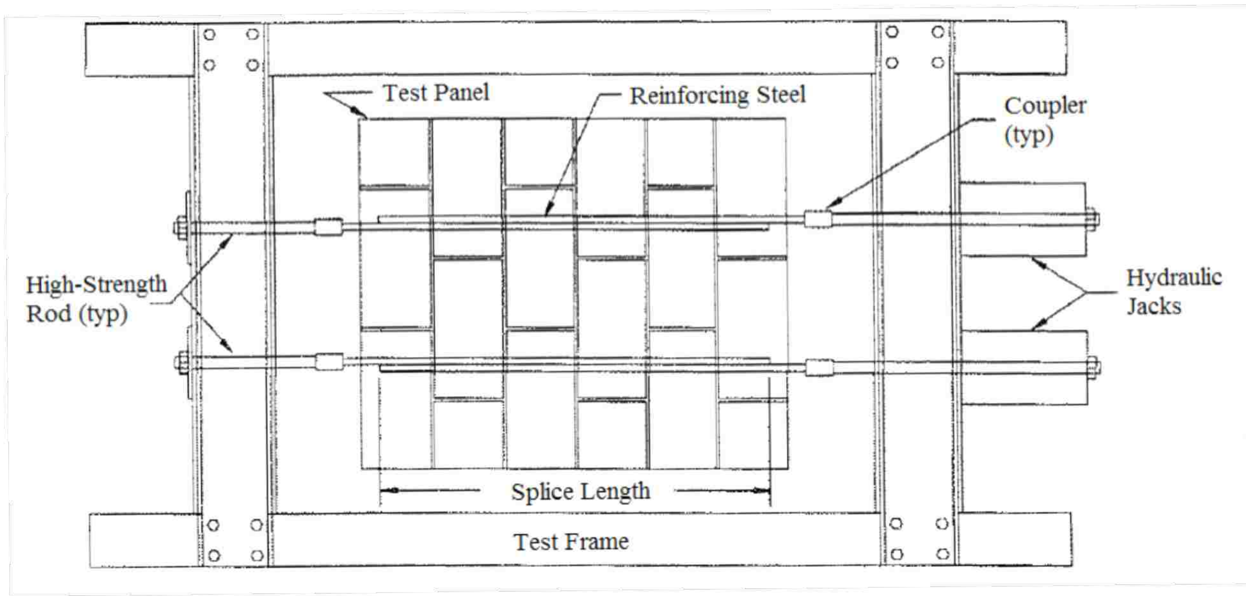

Figure 2-4: NCMA Test Configuration

The results showed that the masonry compressive strength, cover depth, bar diameter and lap length significantly increased the capacity of splices. Also, the 1994 UBC provisions overestimated lap lengths for small reinforcement and underestimated the required splice length for larger bars. As such, a reinforcement size factor was proposed to account for various bar diameters while maintaining the general form of the UBC equation. The new expressions are given in Equations 2-5 and 2-6.

$$l_{de} = \frac{0.13\, {d_b}^2\, f_y\, \gamma}{K\, \sqrt{f'_m}} \qquad (2\text{-}5)$$

and:

$$l_d = \frac{l_{de}}{\varphi} \qquad (2\text{-}6)$$

where: l_{de} = basic development length, in., not to be taken less than 12 inches;
d_b = diameter of reinforcing bar, in.;
f_y = specified yield strength of reinforcing bar, psi;
γ = reinforcement size factor;
= 1.0 for No. 3 through No. 5 reinforcing bars;

	=	1.4 for No. 6 through No. 7 reinforcing bars;
	=	1.5 for No. 8 through No. 11 reinforcing bars;
K	=	minimum clear cover to reinforcing bar, in., not more than $7d_b$;
f'_m	=	specified compressive strength of masonry, psi;
l_d	=	minimum lap splice length of reinforcing bar, in.; and
φ	=	strength reduction factor; equal to 0.80.

2.2.4 Washington State University

Concurrent to the lap-splice testing performed by the NCMA on development length, research was conducted at Washington State University (WSU) (Thompson 1997). The purpose of the research was to verify and complement the testing done by the NCMA and develop a more accurate equation for lap length. Specimens were constructed using nominal 8-inch CMUs in running bond with either No. 5 or No. 7 Grade 60 reinforcing bars. Panels were constructed with two sets of spliced bars to avoid any eccentricities and achieve direct tension during testing. Some specimens also included bed or spiral reinforcement in addition to the lapped bars. Nine different specimen sets were constructed with three identical panels for each set. The splice lengths were selected based on the code requirements at the time as well as the performance of similar specimens in previous research. The test specimens are shown in Figure 2-5.

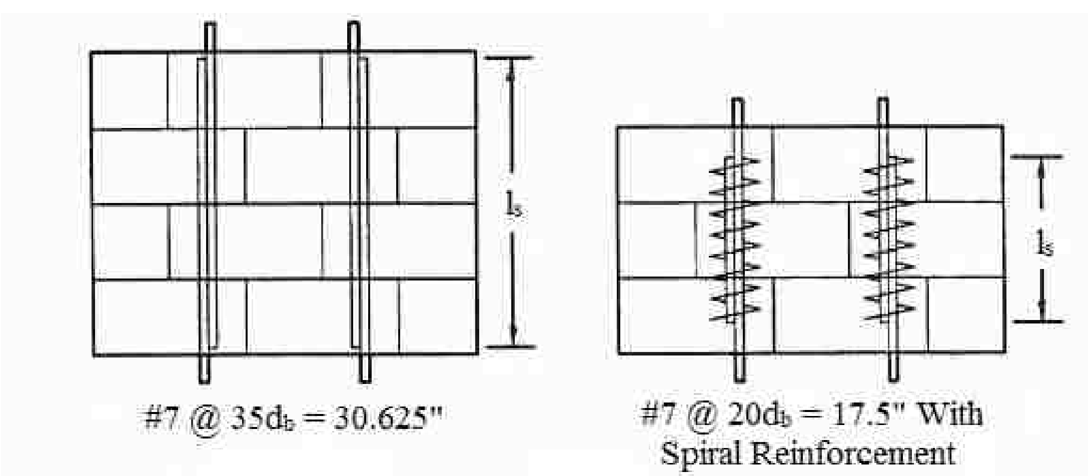

Figure 2-5: WSU Test Specimens

The panels were monotonically loaded within a loading frame with hydraulic jacks in parallel. Figure 2-6 shows the testing apparatus. For analysis purposes, testing results from WSU were combined with the data obtained from NCMA (Thomas et al. 1999), CPAR (Hammons et al. 1994) and that of Soric and Tulin (1987). Data from specimens with transverse or spiral reinforcement or that failed in the reinforcing bar were excluded. The data set resulted in 135 specimens reinforced with Grade 60 lapped reinforcing bar with sizes from No. 4 to No. 11 and a large variety of splice length and clear cover. Linear and multiple linear regression analyses were performed that resulted in Equation 2-7. This model was simplified to a form more consistent with the UBC expression, as shown in Equation 2-8.

$$l_s = \frac{1.25A_b f_y + 23103.54 - 18472.85{d_b}^2 - 319.68\sqrt{f'_m} - 3658.41c_{cl}}{554.81} \quad (2\text{-}7)$$

$$\varphi\, l_s = \frac{0.15\, d_b\, f_y\, \gamma}{K\sqrt{f'_m}} \geq 12\ inches \quad (2\text{-}8)$$

where:

l_s	=	length of lap splice;
f_y	=	specified yield strength of reinforcing bar, psi;
f'_m	=	specified compressive strength of masonry, psi;
γ	=	reinforcement size factor;
		= 1.0 for No. 3 through No. 6 reinforcing bars;
		= 1.4 for No. 7 through No. 11 reinforcing bars;
K	=	$c_{cl}/d_b \leq 5.0$;
c_{cl}	=	minimum clear cover, in.;
d_b	=	diameter of reinforcing bar, in.;
φ	=	strength reduction factor; equal to 0.80.

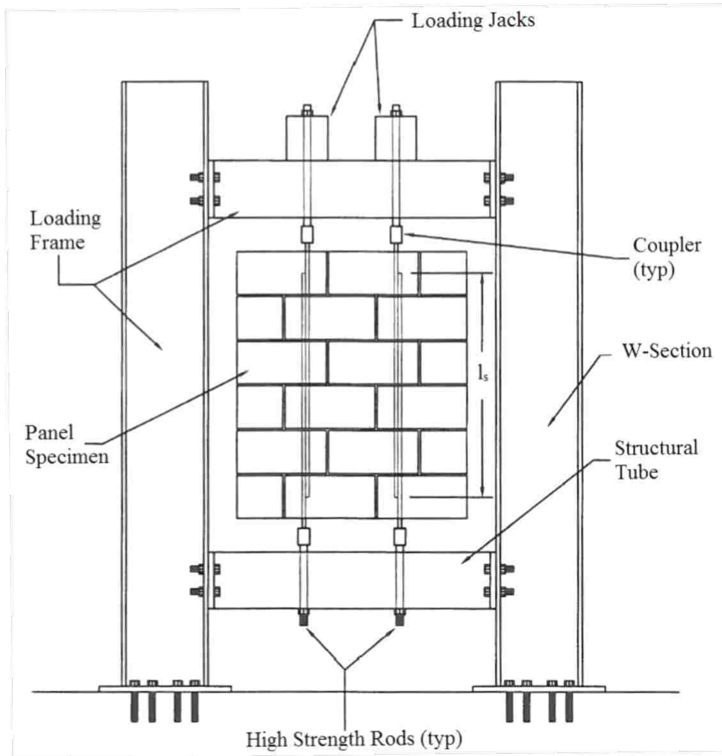

Figure 2-6: WSU Testing Apparatus

2.2.5 Masonry Standards Joint Committee

In 2002, the MSJC adopted the design equation proposed from the research conducted by the NCMA (MSJC 2002). The equation utilized results from the WSU (Thompson 1997) and CPAR (Hammons et al. 1994) research programs in conjunction with the four phases performed by the NCMA (Thomas et al. 1999). The data were fit with linear and multiple linear regression models. The resulting model for predicting the load capacity of splices is presented in Equation 2-9.

$$T_r = -17624 + 305l_s + 25204{d_b}^2 + 322\sqrt{f_{mt}} + 3332c_{cl} \tag{2-9}$$

where:

where:	T_r	=	predicted load capacity of the splice, lb.;
	l_s	=	tested lap length of splice, in.;
	d_b	=	diameter of reinforcing bar, in.;

f_{mt}	=	tested compressive strength of masonry, psi; and
c_{cl}	=	clear cover of reinforcement, in.

This expression was rearranged to isolate the lap length, and the predicted load capacity of the splice, T_r, was replaced with 1.25 times the reinforcing bar yield strength, as a redundancy measure. The splice length design equation is presented in Equation 2-10 and was then adjusted to match the form of the UBC design equation.

$$l_r = \frac{1.25A_b f_y + 17624 - 25204{d_b}^2 - 322\sqrt{f'_m} - 3332c_{cl}}{305} \tag{2-10}$$

where:

l_r	=	basic development length based on regression analysis, in.;
A_b	=	area of reinforcing bar, in^2;
f_y	=	yield strength of reinforcing steel, psi;
d_b	=	diameter of the reinforcing bar, in.;
f'_m	=	specified compressive strength of the masonry, psi; and
c_{cl}	=	clear cover to reinforcement, in.

In 2005, the reinforcement size factor was changed slightly for No. 6 and No. 7 bars resulting in a slightly less conservative value without decreasing the accuracy of the linear fit (MSJC 2005). The current design standard from *Building Code Requirements for Masonry Structures* (TMS 402 2016) remains the same as that in 2005. The equation for development length of uncoated bars is given in Equation 2-11.

$$l_d = \frac{0.13\,{d_b}^2\,f_y\,\gamma}{K\sqrt{f'_m}} \geq 12\ inches \tag{2-11}$$

where:

d_b	=	diameter of reinforcing bar, in.;
f_y	=	specified yield strength of reinforcing bar, psi;
γ	=	reinforcement size factor;
		= 1.0 for No. 3 through No. 5 reinforcing bars;
		= 1.3 for No. 6 through No. 7 reinforcing bars;
		= 1.5 for No. 8 through No. 11 reinforcing bars;
K	=	minimum clear cover to reinforcing bar, in., not more than $7d_b$;
f'_m	=	specified compressive strength of masonry, psi;

l_d = minimum lap splice length of reinforcing bar, in.; and
φ = strength reduction factor; equal to 0.80.

2.3 Standard Specifications for Masonry

The following sections present the material and testing requirements for SCG, mortar and masonry assemblages from applicable ASTM standards.

2.3.1 Self-Consolidating Grout

The standard specification for SCG is given in ASTM C476 (Standard Specification for Grout for Masonry) and ASTM C404 (Standard Specification for Aggregates for Masonry Grout). Fine aggregates are defined as those that pass a 3/8-in. (4.75-mm) sieve whereas coarse aggregates must pass a 1/2-in. (12.5-mm) sieve. Fine grout is produced with only fine aggregate and coarse grout uses both fine and coarse aggregate. SCG must be specified by strength with a minimum required compressive strength of 2,000 psi (13.79 MPa). The compressive strength is determined according to ASTM C1019 (Standard Test Method for Sampling and Testing Grout) and ASTM C39 (Standard Test Method for Compressive Strength of Cylindrical Concrete Specimens) with grout prisms being tested at 28-days. High-range water-reducing admixtures used must conform to ASTM C494/C494M (Standard Specification for Chemical Admixtures for Concrete), Type F or G, and should meet the requirements of ASTM C1017 (Standard Specification for Chemical Admixtures for Use in Producing Flowing Concrete). Viscosity-modifying admixtures must meet the requirements of ASTM C494/C494M, Type S standard. The slump flow should be within the range of 24 to 30 in. (610 to 760 mm) as tested by ASTM C1611/C1611M (Standard Test Method for Slump Flow of Self-Consolidating Concrete) with a Visual Stability Index (VSI) less than or equal to 1. According to ASTM C476, SCG transported to a job-site in a ready-mixed condition may have water added in accordance with recommendations from the producer.

2.3.2 Mortar

The standard specification for mortar is found in ASTM C270 (Standard Specification for Mortar for Unit Masonry). Mortar can be specified by proportion or by property and can be further classified as Type M, S, N or O. Type S and M are most commonly used for modern construction (Masonry Standards Joint Committee 2016). According to ASTM C270, Type S mortar must have a minimum average 28-day compressive strength of 1800 psi and a flow of 110 ± 5%. Test procedures to obtain mortar compressive strength and flow are given in ASTM C109 (Standard Test Method for Compressive Strength of Hydraulic Cement Mortars (Using 2-in. or [50-mm] Cube Specimens) and ASTM C1437 (Standard Test Method for Flow of Hydraulic Cement Mortar), respectively.

2.3.3 Masonry Prisms

The standard specification for constructing and testing masonry prisms is outlined in ASTM C1314 (Standard Test Method for Compressive Strength of Masonry Prisms). Prisms should use materials representative of the corresponding construction and must be at least two units high. Full mortar beds are required and mortar fins should be removed if specimens are to be grouted. Prisms should be grouted at the same time as the masonry but, when not made for field quality control, prisms should be grouted between 4 to 48 hours of initial assemblage. Prior to compressive testing, all grout and masonry prisms should be capped in accordance with ASTM C1552 (Standard Practice for Capping Concrete Masonry Units, Related Units and

Masonry Prisms for Compression Testing). Samples can be capped using high strength gypsum cement or sulfur, and caps should not have an average thickness greater than 1/8 in.

2.4 Summary

SCG is a special type of SCC with smaller nominal aggregate size prepared for use in masonry construction. As this special type of concrete is quite new, there is relatively little research that has been performed on its mechanical properties such as reinforcement development length. This project was undertaken to contribute to the research available and provide data for the advancement of masonry construction using SCG.

3 TEST PROCEDURE

The sections that follow present the project overview, material selection, construction and test procedures for the grout, masonry units, steel reinforcement, mortar and reinforced masonry panels. All materials used conformed to ASTM standards and were selected based on availability.

3.1 Testing Program Overview

Research began with the development of a SCG mix for use in later testing. After an appropriate mix had been developed, grout volume calculations were performed for the reinforced masonry panels and corresponding prisms. The available concrete mixer however, was not large enough to produce the grout required to construct a single masonry panel and corresponding masonry and grout prisms. To decrease construction time and increase the uniformity of specimens, researchers instead decided to use SCG from a local ready-mix supplier to grout the samples in the final phase of testing. This was in large part because the compressive strength of the masonry, f'_m, would need to be determined for each batch of grout to correlate the results from individual tests.

The supplier's mix design was obtained and preliminary masonry and grout prisms were assembled in the laboratory. These were tested to determine f'_m and the results were used to design the required splice length. Masonry panels and prisms were then constructed by professional masons and reinforcement with the desired splice length was inserted. The panels and prisms were grouted with ready-mix SCG and allowed to cure for 28-days. The lap splices were then tested in pure tension to determine if the requisite bond strength had been developed using SCG.

3.2 Grout Material Selection

The coarse and fine aggregates utilized in all laboratory-produced grout were #8 stone and concrete sand, respectively. The coarse aggregate contained a significant amount of fines and was washed over a No. 16 sieve to meet the gradation requirements of ASTM C404. Type I/II portland cement and Class F fly ash constituted the cementitious materials for the grout. The SCG mixes utilized chemical admixtures: a water reducer conforming to ASTM C494 Type A and D and two high-range water reducers conforming to ASTM C494 Type A and F.

3.3 SCG Mix Development

An SCG mix was developed to substantiate and expand upon the research performed by the NCMA (NCMA 2007). The best mix design from their research program was used as the starting point, and iterative SCG batches were produced using locally available material. The primary goal for this portion of the research was to produce a mix that contained the desired rheological properties of stable SCG as outlined in ASTM C1611/C1611M. An appropriate mix was developed but, SCG from a ready-mix supplier was used due to constraints previously mentioned. Thus, the SCG mix design is not provided in the main body of this thesis but a more complete summary of this phase of the research is given in Appendix A.

3.4 Ready-Mix SCG Testing

At the time of this research, the supplier had two SCG ready-mix options. The SCG used was the less expensive variety and was selected because it had been used consistently by a local masonry contractor. The grout used for final specimens was obtained from the ready-mix supplier but, initial testing was performed in the laboratory. Grout and masonry prisms were constructed to determine the compressive strength of masonry, f'_m, needed to calculate the required splice length of the reinforcement. The mix design for the ready-mix SCG was proportioned using fine and coarse aggregate, portland cement, Class F fly ash, water-reducer and high-range water reducer.

A test batch of SCG from the provided mix design was made to observe plastic qualities and make any needed adjustments. The second SCG batch was used to cast grout and masonry prisms. The prescribed mix water produced a grout with a slump of 8 inches with additional water being added to achieve the desired slump flow. Mixing procedures included homogenizing the aggregates and adding 80% of the mix water before introducing the cementitious material. The admixtures were combined with the remaining water and the solution was mixed into the grout. More water was incrementally injected with slump flow tests being performed between each addition until the desired slump flow was obtained. A VSI value was then assigned and grout prisms were cast.

3.4.1 Grout Prisms

All grout prisms were cast in accordance with ASTM C1019. The faces of single core masonry units that would be adjacent to the grout were covered with paper towels and placed to form a square mold. This allowed water to be drawn out of the grout into the CMU while preventing a complete bond between the grout specimen and mold to form, which facilitated the removal of the prisms. A plexiglass plate was located at the base of the mold with form release oil applied. Figure 3-1 shows the grout prism molds. SCG was poured into the molds in a single lift and allowed to consolidate under its own weight without any tamping or vibration. The surface of the prisms was struck off and subsequently refinished within an additional 15 minutes to account for any shrinkage that had occurred. The grout prisms were removed from the molds between 24 and 48 hours after being cast and placed in a fog room to cure.

Figure 3-1: Grout Prism Molds

3.4.2 Masonry Prisms

Seven masonry prisms were constructed according to ASTM C1314 with Type S mortar from ready-mix bags and nominal 8 in. single core masonry units. The hollow prisms were placed in watertight bags, grouted, and the bags were sealed. Nine grout prisms were also cast with the same SCG used for the masonry assemblages. After 14-days three of the grout prisms and one masonry prism were tested in compression. All other prisms were tested at 28-days.

Masonry and grout prisms were cast and tested according to ASTM C1019 and C1314, respectively. Throughout the project, samples were measured and then capped with high-strength gypsum according to ASTM C1552. The caps cured for at least two hours before testing commenced. Specimens were tested in compression under monotonic loading at displacement controlled rate of 0.05 in./min.

3.5 Steel Reinforcement Material Selection

Fifty pieces of rebar were supplied in 4’ lengths for the research. No. 5 bars were chosen to avoid development length being governed by the 12-in. minimum requirement. This in turn allowed for an appropriate analysis of the design equation when using SCG. The headed bars conform to ASTM A970 (Standard Specification for Headed Steel Bars for Concrete Reinforcement), class A and B, and were selected as the means to apply tensile loading to the reinforcement with the available equipment. While still not approved for use in masonry to reduce the required development length, the head-to-bar connection capacity was designed to exceed that of the bar. A picture of the headed reinforcement is provided in Figure 3-2. The reinforcement was tested according to ASTM A370 (Standard Test Methods and Definitions for Mechanical Testing of Steel Products) Method A9, by the supplier and was certified as Grade 60. This requirement is contained in ASTM A615 (Standard Specification for Deformed and Plain Carbon-Steel Bars for Concrete Reinforcement) and states that Grade 60 bars must have minimum yield and ultimate strengths of 60 and 90 ksi, respectively.

Figure 3-2: Headed Steel Reinforcement

3.6 Masonry Panel Construction

As the masonry panels needed to be elevated to allow the reinforcement to extend past the bottom for testing, masonry panel construction began with the preparation of 2x12 DF-L#2 wooden bases. Dimension lines were marked on the boards for correct placement of the masonry units and holes were cut out for the headed reinforcement to pass through. The cutouts were retained to plug the holes prior to grouting. The bases were then placed on top of 8-in. half-blocks to allow the bars to protrude from the bottom. The elevated wooden bases are shown in Figure 3-3.

Figure 3-3: Wooden Bases

Two professional masons constructed 12 panels with 8-in. CMUs and Type S mortar in running bond. The panels were three courses tall with a mortar joint beneath the first course to achieve a level plane. All mortar was prepared in a concrete floor mixer by combining bagged Type S mortar and water. The mortar was mixed for sufficient time to ensure that false set did not occur from the rehydration of the gypsum within the mixture. Each batch was prepared by the tender or the masons and was then transported to the construction area in a wheelbarrow. The mortar was placed on stands and supervised by the tender to maintain the proper consistency.

Panels were checked for level throughout the construction process and all mortar joints were finished with a concave tool. The construction can be seen in Figure 3-4. Twelve masonry prisms were also constructed according to ASTM C1314. These were tested concurrently with the panels to obtain the actual compressive strength of the masonry at the time of testing. Five mortar cubes were cast in accordance with ASTM C109 using the mortar prepared by the tender.

Figure 3-4: Masonry Panel Construction

After construction, mortar fins and droppings were removed from the interior of the cells. Specimens were divided into three test groups with splice length being the only variable. Test Group 1 consisted of six panels with the code mandated development length. Test Groups 2 and

3 each contained three specimens with smaller splices to determine if the same capacity could be achieved with smaller lengths. All panels were nominally identical with the height of the extending reinforcement being approximately equal. The reinforcing bar was assumed to have a yield stress of 60 ksi and the compressive strength of masonry, f'_m, was obtained from the preliminary tests of the masonry prisms. Lap-splice parameters and lengths for each test group are summarized in Table 3-1.

Table 3-1: Development Length Parameters

Test Group	d_b (in.)	f_y (ksi)	γ	K (in)	f'_m (psi)	$l_{d,req}$ (in.)	$l_{d, used}$ (in.)
1	0.625	60	1	3.5	2875	16.25	16.25
2	0.625	60	1	3.5	2875	16.25	14.00
3	0.625	60	1	3.5	2875	16.25	12.00

Lap splices were fabricated by cutting the bars to their proper lengths and tying them together with bailing wire. The bars were placed inside the panels through the hole of the wooden base, which was then patched using the wood cutout and industrial tape. Figure 3-5 shows the panels prior to grouting. A schematic for the panels is shown in Figure 3-6. Appendix C contains more information and drawings.

Figure 3-5: Panels Before Grouting

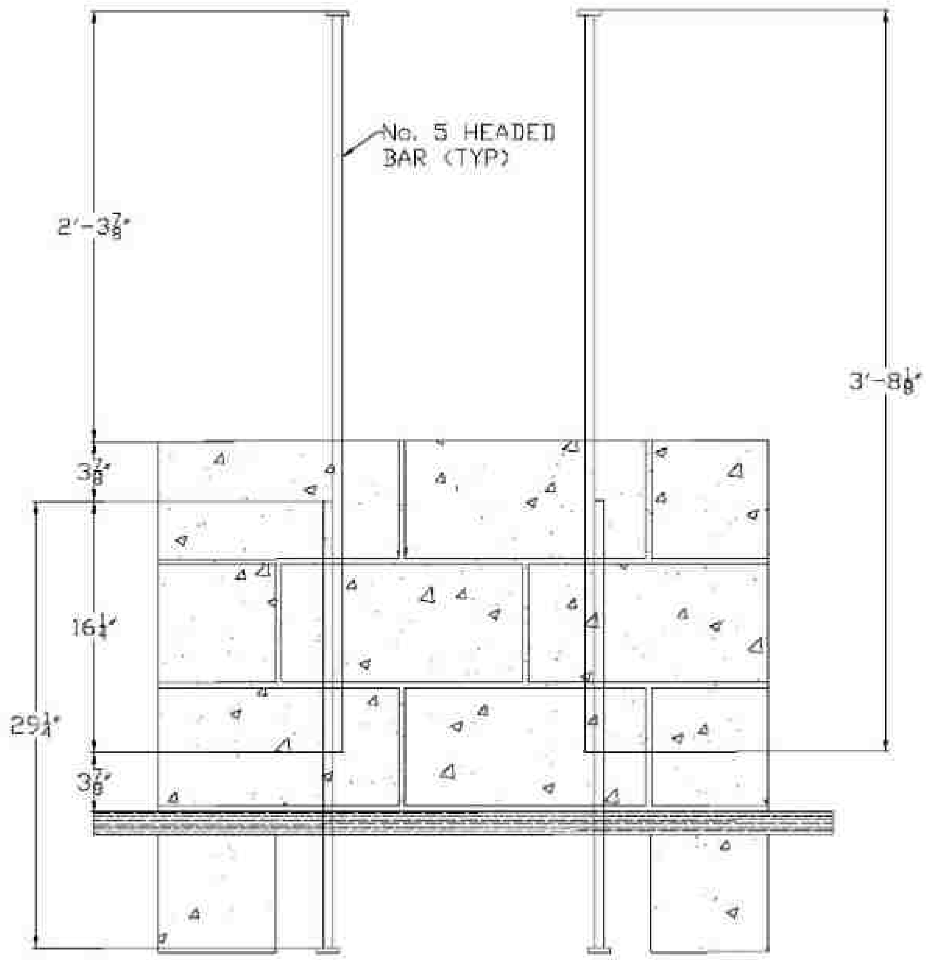

Figure 3-6: Test Group 1 Specimen Schematic

Grouting occurred eight days after initial construction of the panels and prisms. The SCG was delivered in a ready-mix truck and initially had an 8-in slump. Water was added incrementally with slump flow tests performed between each addition until a slump flow of 22 in. was achieved. A VSI value of 0 was observed for each subsequent test. Figure 3-7 shows one of the slump flow test being conducted. When the required workability was achieved, the grout was poured from the ready-mix truck into a large bin and transported closer to the panels by forklift. Grout was then poured into the masonry cells using buckets in a single lift with no mechanical consolidation. After placement the reinforcement was centered and checked for level and the grout surface was finished.

Figure 3-7: Slump Flow Test

The masonry prisms were also grouted and six SCG prisms were cast in accordance with ASTM C1019. The completed grout and masonry prisms are shown in Figure 3-8. After 24 hours, the grout prisms were removed from their molds but were not placed in the fog room. All prisms were allowed to cure in the same ambient temperature and humidity as that of the panels. This was done so that all specimens would cure at the same rate such that the strengths attained in compressive testing of the prisms was as identical as possible to that of the reinforced panels.

Figure 3-8: Masonry and Grout Prisms

All specimens were allowed to cure for 28 days prior to testing. The panels were labeled with the test group number, sample name and the splice length. Some completed panels are shown in Figure 3-9.

Figure 3-9: Grouted Panels Before Testing

3.7 Steel Reinforcement Testing

Although mill tests performed by the supplier gave satisfactory results for Grade 60 steel, the yield and ultimate tensile strengths of the reinforcement were verified. One-foot sections were cut from the longer reinforcing bar lengths and were tested according to ASTM E8 (Standard Test Method for Tension Testing of Metallic Materials) at a strain rate of 0.3 in./min. Three of the samples consisted of only the bar and the final sample included the connection between the head and the shaft. Figure 3-10 and Figure 3-11 show the apparatus for each test method. One of the specimens was loaded in tension past its yield strength but not to failure; the other three specimens were tested to failure. Stress-strain curves were developed for each specimen and the yield strength of the reinforcement was determined using the 0.2% offset method.

Figure 3-10: Rebar Testing Apparatus

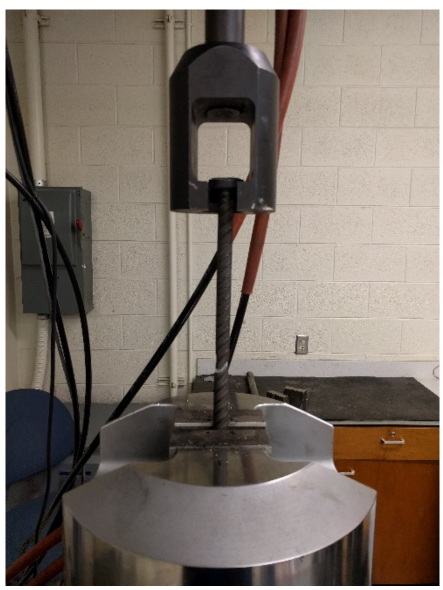

Figure 3-11: Headed Rebar Testing Apparatus

3.8 Specimen Testing

The subsequent sections present the procedures used to test the masonry and grout prisms, mortar cubes and reinforced masonry panels. There is no standard method for the testing of masonry reinforcement splices in tension but, an approach consistent to that of other research programs was maintained.

3.8.1 Mortar Testing

The mortar cubes made at the time of the construction of the panels were removed from their molds five days after being cast and placed in the fog room to cure. The compressive strength was determined according to ASTM C109 at 36 days, coinciding with the first day of masonry panel testing, using a Fourney compression machine. Samples were tested at a displacement controlled rate of 0.05 in./min. Figure 3-12 shows a typical mortar cube after testing.

Figure 3-12: Mortar Cube After Compression Testing

3.8.2 Grout and Masonry Prism Testing

One day before testing panels, all grout and masonry prisms were measured and capped in general accordance with ASTM C1552. Figure 3-13 shows the freshly capped masonry and grout prisms.

Figure 3-13: Masonry and Grout Prism Capping

The grout and masonry prisms were tested according to ASTM C1019 and C1314, respectively, using a Fourney compression testing machine and Baldwin Universal Testing Machine (UTM), respectively. Before testing the masonry prisms, an aluminum plate was placed on top of the specimen to uniformly distribute the load from the circular bearing block to the specimen. The testing apparatus for both sample types is shown in Figure 3-14. Load was applied at a constant rate of 0.05 in./min. and specimens were tested until failure. The maximum applied load was recorded and the specimen's failure mode was noted according to Figure 3-15 and Figure 3-16.

Figure 3-14: Grout and Masonry Prism Testing Apparatus

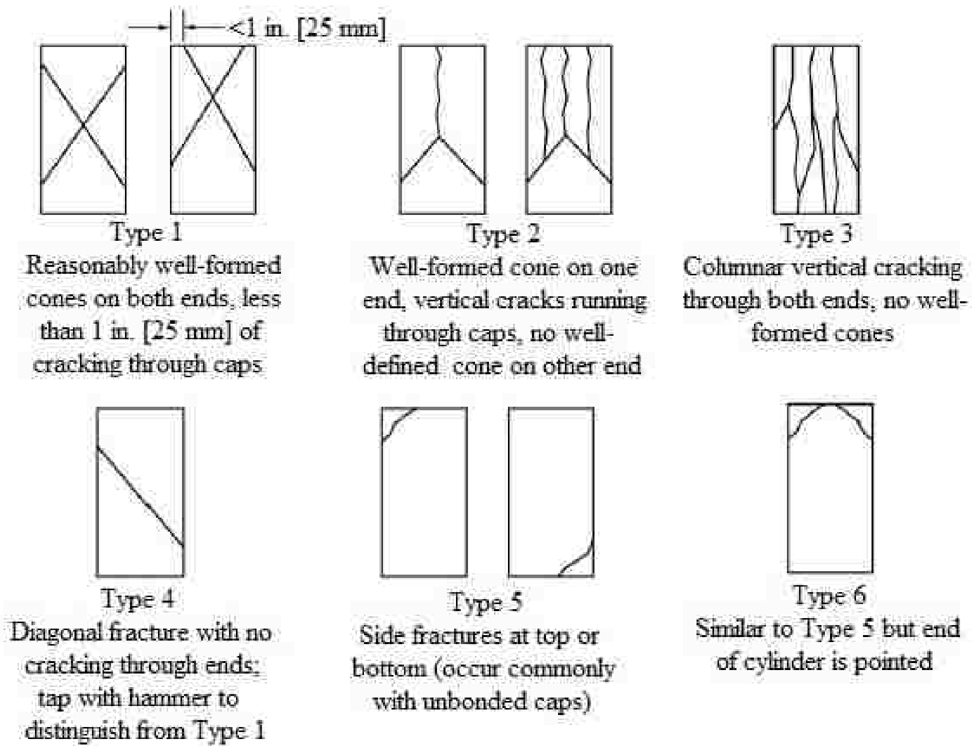

Figure 3-15: Grout Prism Typical Fracture Patterns

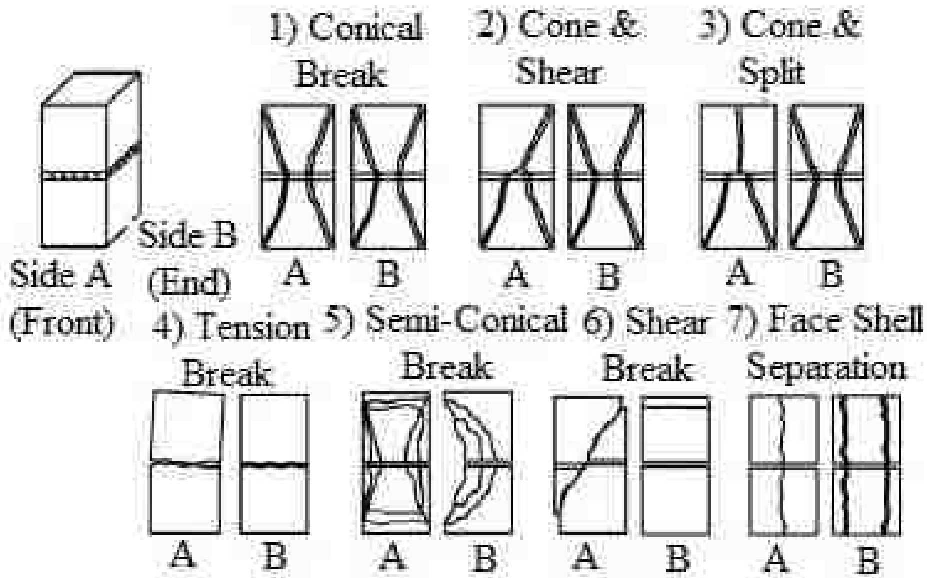

Figure 3-16: Masonry Prism Mode of Failure

3.8.3 Reinforced Masonry Panel Testing

The performance of masonry splices has typically been evaluated using pull-pull and flexural testing methods. Flexural testing typically uses a third-point transverse loading with the splice located in the region of constant moment to induce flexural tension on the splice. This load state is believed to be the most accurate representation of the actual conditions for masonry shear walls under out-of-plane lateral load; however, it can influence the mode of failure causing masonry crushing at the compression face before splice failure (Ahmed and Feldman 2012, Sanchez and Feldman 2015). Nonetheless, the current design provision code was derived from testing performed by only the pull-pull scenario (Hammons et al. 1994; Thompson 1997; Thomas et al. 1999). Masonry shear walls under in-plane lateral loading will induce direct tension on the reinforcing bars as a couple that resists the overturning moment. For these reasons a pull-pull test was selected. A Baldwin UTM was utilized to apply the tensile loads. The specimens were constructed to be as symmetric as possible to negate any eccentricities contributing to the performance of the splices. While a monotonic tensile loading of test specimens represents an extreme loading condition for splices, it allows observation of the failure mode and an evaluation of the performance.

The reinforced masonry panels were loaded at a displacement controlled rate until failure was noticeable or and the load significantly decreased. Before testing began, critical loading was calculated for the minimum required capacity of the splice which was 1.25 times the 60 ksi design yield stress of the reinforcing bar. This was equivalent to an applied tensile load for each splice of approximately 23 kips. The yield stress of 72 ksi from mills tests by the supplier was also multiplied by 125% resulting in an applied tensile force of 28 kips for each splice. These values were used as thresholds at which the displacement was increased to facilitate faster test times. The first specimen was loaded at a rate of 0.03 in./min. until the load reached 23 kips and the rate was increased to 0.15 in./min. When the applied load reached 28 kips, the displacement rate was increased to 0.3 in./min. until failure occurred. The duration for this test exceeded an hour and it was decided to use an increased displacement rate for all other tests. Displacement rates of 0.1 in./min., 0.15 in./min. and 0.3 in./min. were selected with the thresholds aforementioned.

The tensile force was applied to the reinforced masonry panels via steel W8x31 sections. These wide flange sections were attached to the crossheads of the testing machine. Prior to testing, an anticipated loading equal to 1.5 times the design yield stress of the rebar was used to determine the adequacy of the W shape. After the first test, researchers observed that loading caused the web of the W shape to yield with significant deformation. The testing method was then revised with the top beam being loaded through the flanges and the web of the bottom member was reinforced with steel plates on each side. The bottom steel member was inverted for the subsequent test and the deformation was reversed. Figure 3-17 shows a schematic of the final testing apparatus.

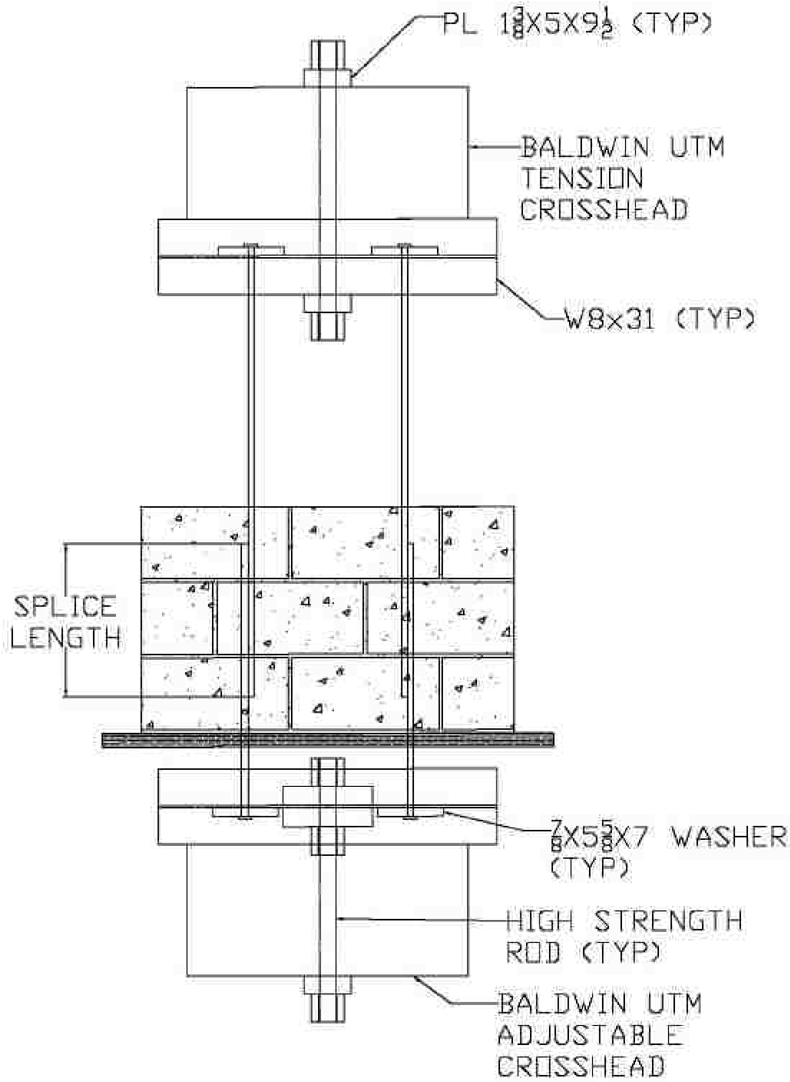

Figure 3-17: Reinforced Masonry Panel Testing Apparatus

Each panel was loaded into the testing apparatus with the following procedure. The sample was placed by a forklift with industrial lifting straps into the center of the testing machine. The panel was raised until the headed reinforcement passed through the oversized holes in the web of the W section. Washers were placed between the rebar heads and the steel member and the sample was lowered until the panel essentially hung from the upper crosshead. The adjustable crosshead at the base was raised and a washer was inserted between the steel member and the rebar heads and then lowered until tight. The two steel W shape members were not tightened down completely against the crossheads to allow some movement and for slight elevation and angle variances between the reinforcement. In cases where there was more than an eighth of an inch of elevation difference between the two lower heads, one or two 1/8” circular washers were placed between the 7/8” washer and the web of the W section. Figure 3-18 shows the connections in greater detail.

Figure 3-18: Headed Rebar to Test Frame Connections

4 RESULTS

The subsequent sections present the results from the testing done according to the procedures described in the previous chapter. Appendices B and C contain additional tables, figures and photos.

4.1 Ready-Mix SCG

The preliminary compressive strength test results from the preliminary masonry and grout prisms are presented in Figure 4-1 and Table 4-1. More detailed results for the tests can be found in Appendix B.

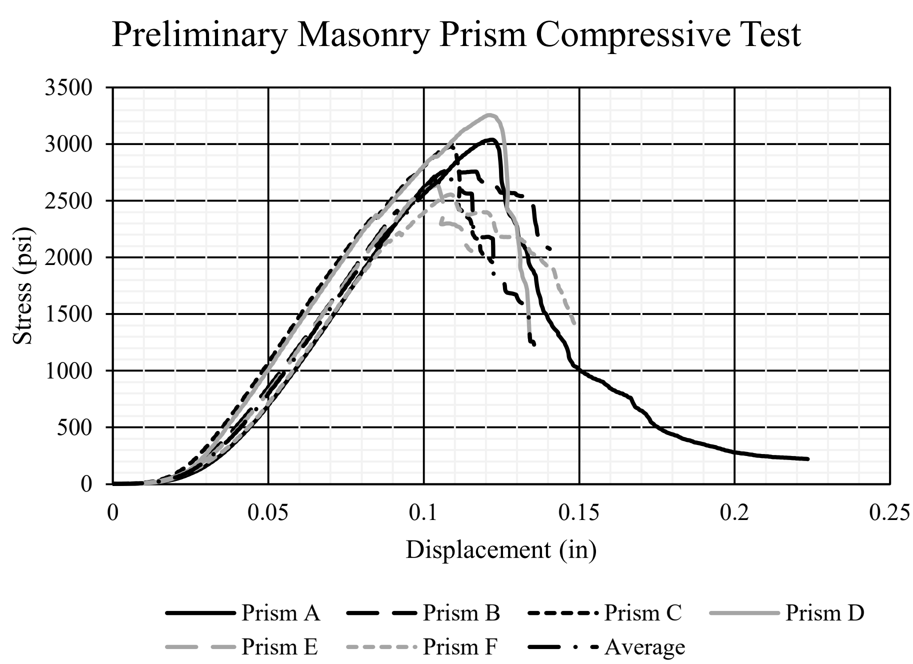

Figure 4-1: Stress vs. Displacement for Preliminary Masonry Prisms

Table 4-1: Preliminary Grout Compressive Strength Testing Results

Prism Type	Average Area (in^2)	Average Load (lbs)	Average Compressive Strength (psi)	Coefficient of Variation
Masonry	57.80	166151	2875	9.2%
Grout	16.17	75549	4675	8.2%

4.2 Steel Reinforcement

The steel reinforcement tension test results are presented in Table 4-2. The yield strength calculation did not include headed reinforcing bar results because the first portion of the curve was very non-linear because of the seating of the head. A sample stress-strain curve is presented in Figure 4-2. Stress-strain curves for other samples are provided in Appendix B.

Table 4-2: Steel Reinforcement Test Results

Sample	Head	Tensile Yield Strength, f_y (ksi)	Tensile Ultimate Strength, f_u (ksi)
2	No	65.0	95.6
3	No	64.0	95.5
4	No	65.2	95.6
5	Yes	-	95.3

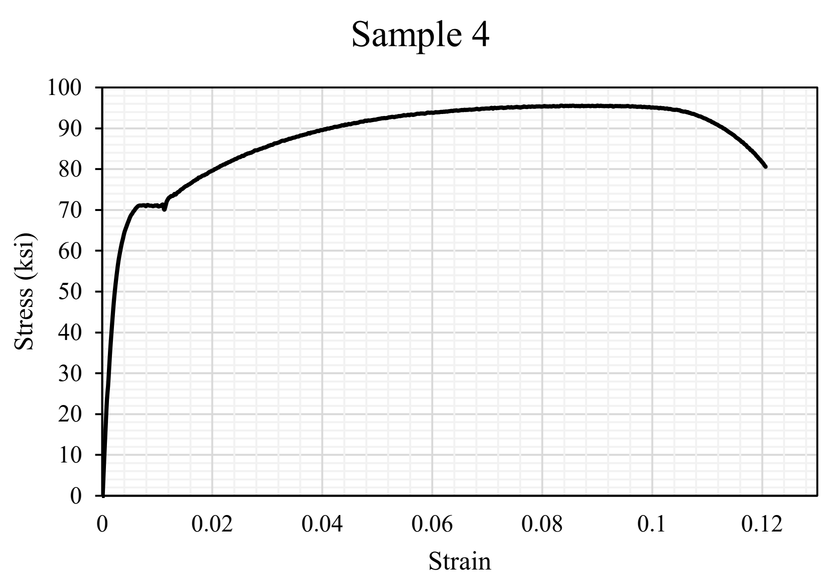

Figure 4-2: Steel Reinforcement Testing Stress-Strain Curve

4.3 Masonry Specimens

The following sections contain the test results for the mortar, grout, masonry prisms and reinforced masonry panels associated with the final testing phase. Appendix C contains photographs from the testing.

4.3.1 Mortar

Table 4-3 presents the results from the mortar cube compressive testing. This includes the cross-sectional area, maximum load at failure and the calculated compressive strength.

Table 4-3: Mortar Cube Test Results

Sample	Area (in^2)	Maximum Load (lb)	Compressive Strength (psi)
1	4	10025	2506
2	4	9455	2364
3	4	9990	2498
4	4	9905	2476
5	4	8855	2214
Average Compressive Strength			2412
Coefficient of Variation			5.2%

4.3.2 Grout

Testing of the ready-mix SCG before grouting yielded a slump flow of 22-in. and a VSI value of 0. Table 4-4 presents the compression test results for the grout prisms cast in conjunction with the masonry panels and prisms. The table includes curing time before testing, cross-sectional area and the failure mode according with Figure 3-15.

Table 4-4: Grout Prism Compression Test Results

Sample	Cure Time (days)	Area (in^2)	Maximum Load (lb)	Failure Mode	Compressive Strength, f'_g (psi)	Coefficient of Variation
A	28	13.21	48965	Type 4	3706	
B	28	13.78	47755	Type 1	3464	
C	28	13.60	46275	Type 1	3403	
		Average Compressive Strength			3524	4.5%
D	33	13.73	43520	Type 1	3170	
E	33	13.49	47680	Type 4	3534	
F	33	13.52	42780	Type 1	3163	
		Average Compressive Strength			3289	6.4%

4.3.3 Masonry Prisms

The masonry prism compression testing results are tabulated in Table 4-5. This includes the curing time before testing, cross-sectional area, load at failure, compressive strength and failure mode per Table 4-5. The data measured were utilized to generate graphs for each testing day, which are presented in Figure 4-3 through Figure 4-5.

Table 4-5: Masonry Prism Compression Test Results

Sample	Curing Time (days)	Area (in^2)	Maximum Load (lb)	Failure Mode	Compressive Strength, f'_m (psi)	Coefficient of Variation
1	28	58.18	146289	Mode 7	2514	
2	28	57.96	161388	Mode 1	2784	
3	28	58.00	160307	Mode 1	2764	
4	28	58.07	160558	Mode 1	2765	
5	28	57.91	165384	Mode 3	2856	
6	28	58.08	156930	Mode 2	2702	
	Average Compressive Strength				2731	4.3%
7	32	58.23	171258	Mode 2	2941	
8	32	58.00	159766	Mode 1	2755	
9	32	58.19	167368	Mode 3	2876	
	Average Compressive Strength				2857	3.3%
10	33	57.99	171030	Mode 2	2949	
11	33	58.01	161998	Mode 7	2792	
12	33	57.62	163022	Mode 1	2829	
	Average Compressive Strength				2857	2.9%

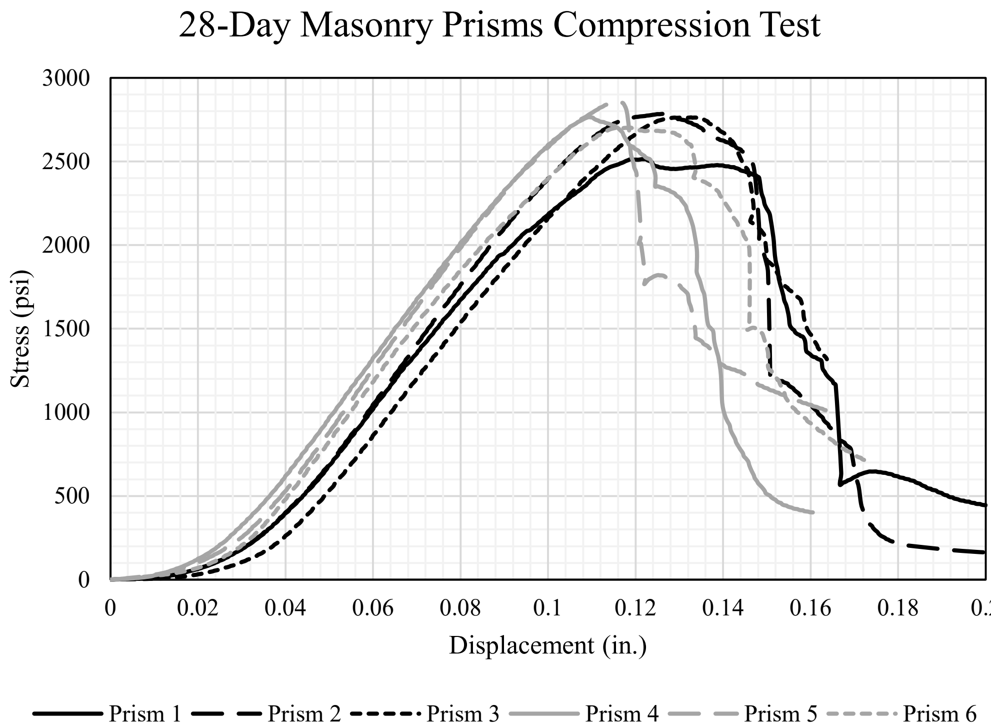

Figure 4-3: Stress vs. Displacement Plot for Masonry Prisms at 28-Days

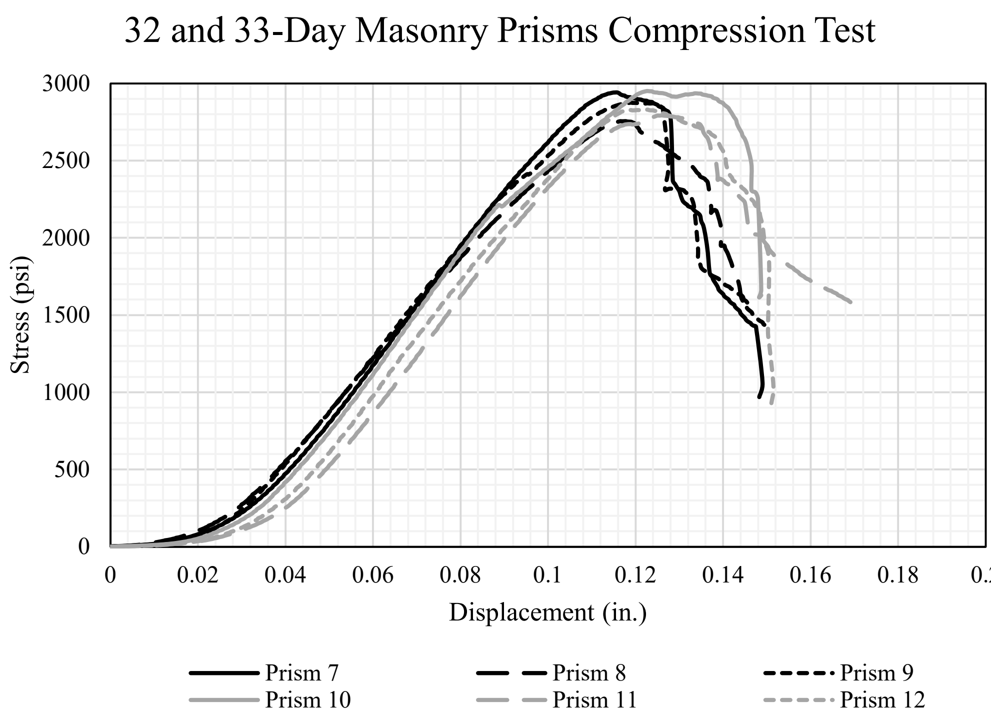

Figure 4-4: Stress vs. Displacement Plot for Masonry Prisms at 32 and 33-Days

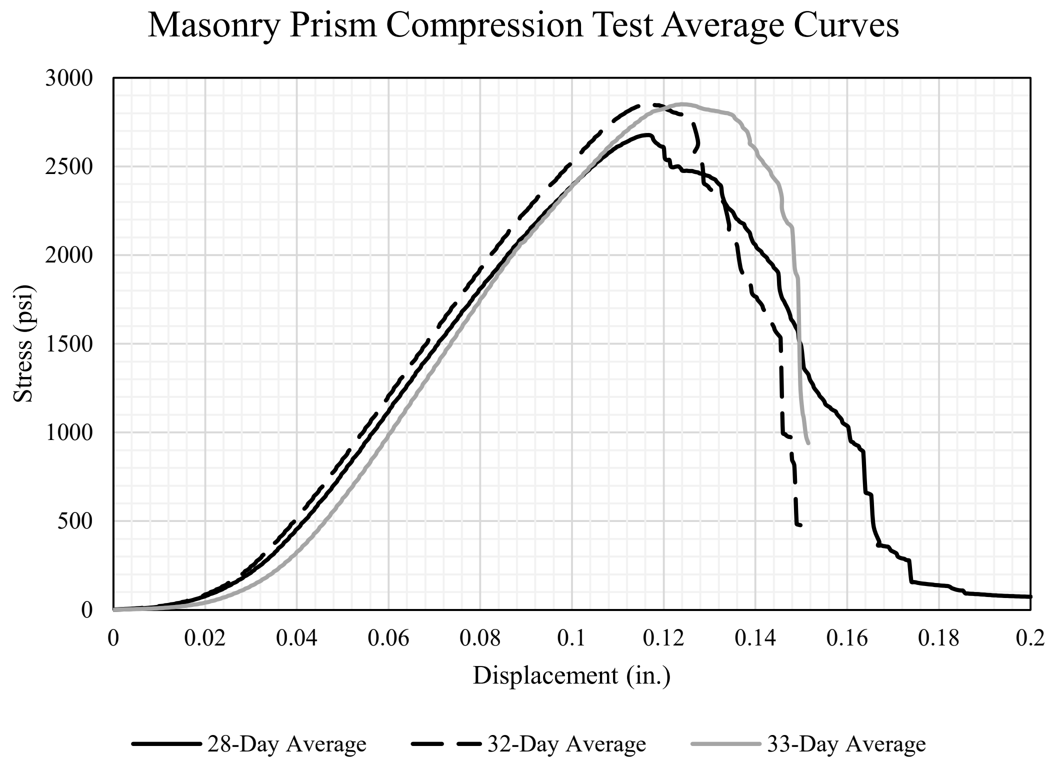

Figure 4-5: Average Stress vs. Displacement Plot for Masonry Prisms

The required development length was calculated for each sample based on the 28-day compressive strength of masonry. This results in an increased required development length as summarized in Table 4-6.

Table 4-6: Adjusted Development Length Parameters

Test Group	d_b in.	f_y ksi	γ	K	f'_m psi	$l_{d,calc}$ in.	$l_{d,\ used}$ in.
1	0.625	60	1	3.5	2774	16.53	16.25
2	0.625	60	1	3.5	2774	16.53	14.00
3	0.625	60	1	3.5	2774	16.53	12.00

4.3.4 Reinforced Masonry Panel Testing

During each masonry panel test, the applied loading and displacement were recorded. After testing, the maximum stress in the reinforcement was determined and compared to the specified yield stress. The results are summarized in Table 4-7. General failure modes are also reported, which were either reinforcement fracture, splitting of the masonry, or failure of the head to bar connection.

Table 4-7: Reinforced Masonry Panel Testing Results

Test Group	Sample	Splice Length (in)	Failure Mode	Maximum Load per Splice (lb)	Ultimate Reinforcement Stress (ksi)	Ratio of Ultimate Stress to Specified Yield Stress
1	A	16.25	Masonry Splitting	28508	92.9	1.55
	B		Masonry Splitting	26160	85.3	1.42
	C		Rebar Fracture	28962	94.4	1.57
	D		Masonry Splitting	27231	88.8	1.48
	E		Masonry Splitting	26905	87.7	1.46
	F		Masonry Splitting	27489	89.6	1.49
2	A	14.00	Head Failure	26252	85.6	1.43
	B		Masonry Splitting	24564	80.1	1.33
	C		Masonry Splitting	25250	82.3	1.37
3	A	12.00	Masonry Splitting	23649	77.1	1.28
	B		Masonry Splitting	25366	82.7	1.38
	C		Masonry Splitting	24983	81.4	1.36

The splice capacity predicted by Equation 2-8 was calculated using the average strength of masonry, f'_m, from masonry prisms tested that day. The actual and predicted splice capacities are summarized in Table 4-8. The maximum load at failure for each splice was plotted against the predicted capacity, as is shown in Figure 4-6, and includes the one-to-one curve and a linear regression curve fit to the measured data for comparison.

A photograph of a typical specimen after failure is shown in Figure 4-7. Photos of each specimen after failure are included in Appendix C. Stress vs. displacement plots for each test group were generated and plots are presented in Figure 4-8 to Figure 4-10.

Table 4-8: Measured and Predicted Splice Capacity Comparison

Test Group	Sample	Splice Length (in)	Average f'$_m$ at Testing	Maximum Load per Splice (lb)	Predicted Splice Capacity (lb)	Ratio of Measured and Predicted Capacity
1	A	16.25	2731	28508	25667	1.11
	B		2857	26160	26052	1.00
	C		2857	28962	26052	1.11
	D		2857	27231	26052	1.05
	E		2857	26905	26052	1.03
	F		2731	27489	25667	1.07
2	A	14.00	2857	26252	25366	1.03
	B		2857	24564	25366	0.97
	C		2857	25250	25366	1.00
3	A	12.00	2857	23649	24756	0.96
	B		2857	25366	24755	1.02
	C		2857	24983	24755	1.01

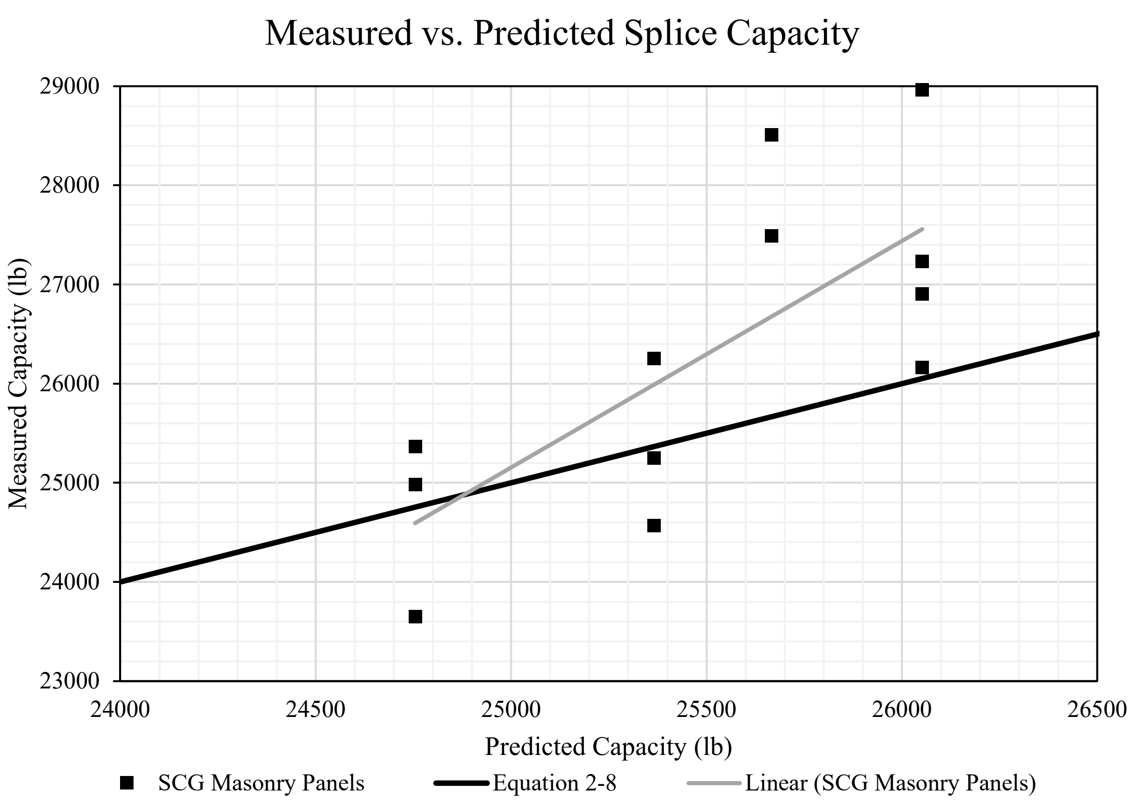

Figure 4-6: Measured vs. Predicted Splice Capacity

Figure 4-7: Typical Masonry Panel After Failure

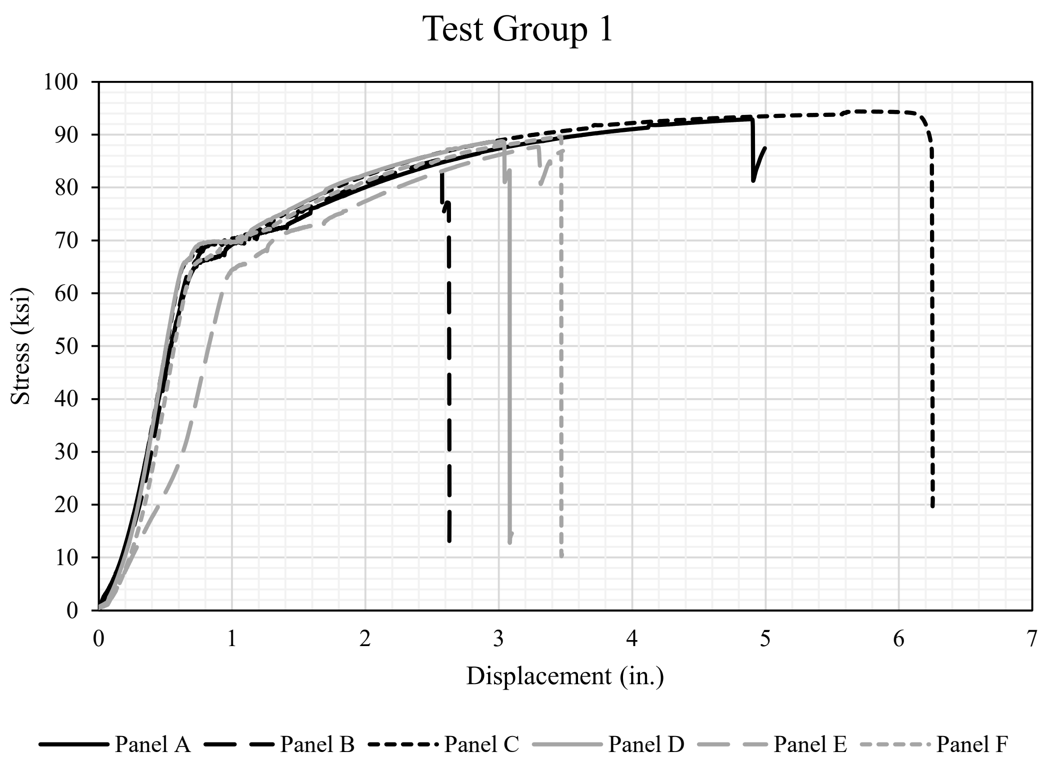

Figure 4-8: Stress vs. Displacement for Test Group 1

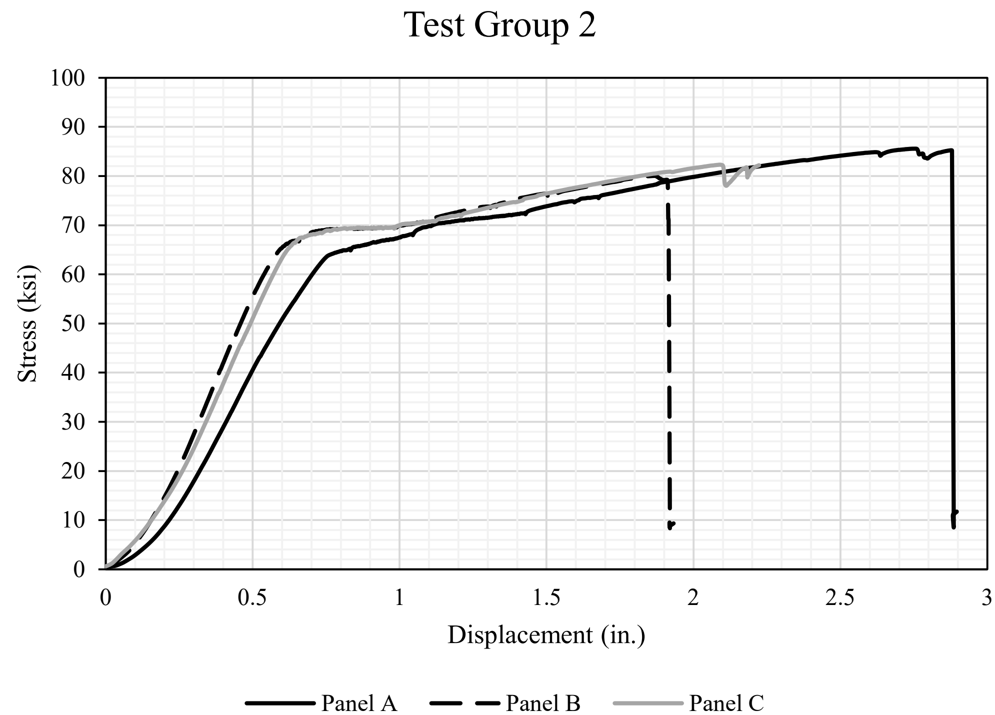

Figure 4-9: Stress vs. Displacement for Test Group 2

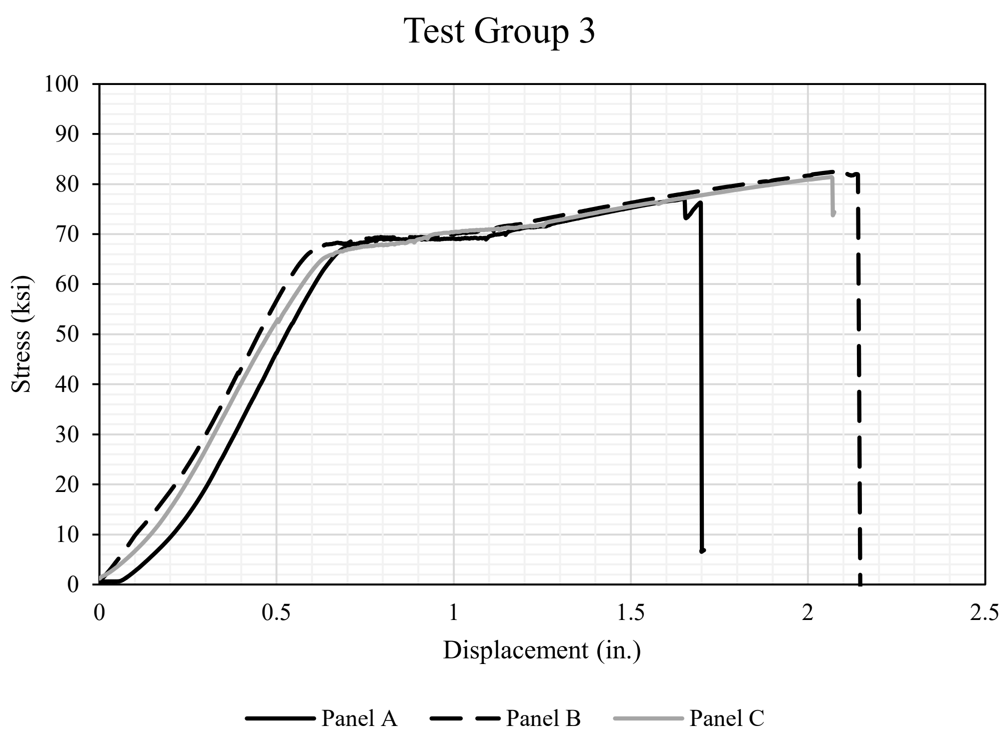

Figure 4-10: Stress vs. Displacement for Test Group 3

Examination of the grout within the masonry panels after failure indicated that self-consolidation of the grout was achieved throughout the samples. Voids were not present adjacent to the reinforcement or masonry units and no slip was observed. A photograph of the grout-reinforcing bar interface is shown in Figure 4-11. Figure 4-12 presents a specimen after failure for which the grout core was exposed demonstrating the masonry-grout interface.

Figure 4-11: SCG-Bar Interface

Figure 4-12: SCG-CMU Interface

5 ANALYSIS

The following sections present a discussion of the results. The results from the mortar and SCG testing are examined against the applicable ASTM standards and a comparison of the measured and predicted splice capacities is performed. The required stress in reinforced masonry splices is also analyzed using the code requirements.

5.1 Mortar

The mortar used in all masonry samples was Type S, mortar cement. The mortar flow test was not performed, but was assumed to meet the consistency requirements because of close monitoring by experienced masons. Testing of mortar cubes determined that the average 36-day compressive strength was 2412 psi. This is well above the minimum requirement of 1800 psi from ASTM C270. While the tests were performed eight days after the mandated testing time, only a small increase in strength is believed to have resulted from the extended curing.

5.2 SCG

Coarse SCG was used for all test specimens which met the gradation requirements of ASTM C476 and ASTM C404. The compressive strength of the grout prisms tested at 28 and 33-days was above the minimum requirement of 2000 psi as required by ASTM C476 with average strengths of 3525 and 3289 psi, respectively. The prisms tested at 33-days were expected to have developed greater strength than those tested at 28-days. Researchers believe that this discrepancy was due to the geometry of the samples as well as the gypsum caps for these specimens. When removed from their molds some of the prisms were observed to not be perfect rectangular prisms. This made capping them difficult as the base and top were not completely level. The testing apparatus was also experiencing issues as the base will typically adjust to account for slight variations. When testing the 33-day samples however, the base was not lubricated sufficiently, possibly causing the load to not be applied normal to the samples. Additionally, the sampling was relatively small for the grout prisms with only six total being tested. While the reason the samples did not behave as expected is unknown, these factors were identified the probable cause. The difference in sample strengths is considered inconsequential to the research outcomes as final results were compared using the compressive strength from the masonry assemblages.

The minimum slump flow requirement of 24-in. from ASTM C476 was not achieved with the ready-mix grout used for the final grout and masonry samples. Researchers were concerned that additional water could result in exceedance of the upper slump flow limit or that the grout would segregate. The slump flow requirement is used primarily to ensure that self-consolidation occurs. Some voids along the corners of the grout prisms were observed when the molds were removed but this is likely a result of the 90-degree corner. The cells of masonry units are typically rounded which decreases the likelihood that the voids will form within the grout. All masonry prism and panel specimens inspected after failure indicated that the grout had consolidated and there were no excessive voids within these samples.

5.3 Masonry

Equation 2-10, the design provision for development length, includes the compressive strength of masonry, f'_m, which is affected by the mortar, grout and masonry units. The masonry prism testing resulted in 28, 32 and 33-day average compressive strengths of 2731, 2857 and 2857 psi, respectively. These strengths are close to the outcome of the preliminary ready-mix SCG testing used to design the splices. As expected, the 32 and 33-day results were slightly higher than those at 28-days at which time the rate of strength gain appears to have decreased dramatically.

The average masonry compressive strength from each day of testing was used in Equation 2-8 to produce the predicted splice capacity. All other variables remained the same as specified in Table 3-1. The required development length using the actual masonry compressive strength was also calculated resulting in a slightly increased lap length for all specimens.

5.4 Reinforced Masonry Panels

The same general procedures were used throughout construction of the masonry panels as researchers sought to create symmetrical, uniform specimens. Slight variations in splice length, rebar placement and material properties however, resulted in small discrepancies during testing. Splice performance of samples corresponded relatively well within test groups; and the average capacity of each set decreased slightly with the reduced lap length, as expected.

The load-displacement behavior of the splices, as shown in Figures 4-6 through 4-8, was similar for all samples. During testing an approximately linear elastic region was initially observed which transitioned into plastic deformation as cracking and stretching occurred. This slower rate of increased load per displacement continued until failure and accompanying drop in capacity. Preliminary cracking in masonry units and mortar joints was typically observed before failure. Figure 5-1 shows typical crack propagation. For many specimens, the in-plane portion of masonry exterior to the splice was catastrophically expelled from the testing apparatus as shown in Figure 5-2. The failure mode was typically masonry splitting, but two panels experienced reinforcement rupture. These had accompanying masonry splitting but, recorded video was used to verify that the rebar had failed prior to the masonry. Shorter lap-splices were not as ductile as those with longer development lengths. This was especially evident in Test Group 3 as the samples failed very shortly after cracking in the mortar joints was observed.

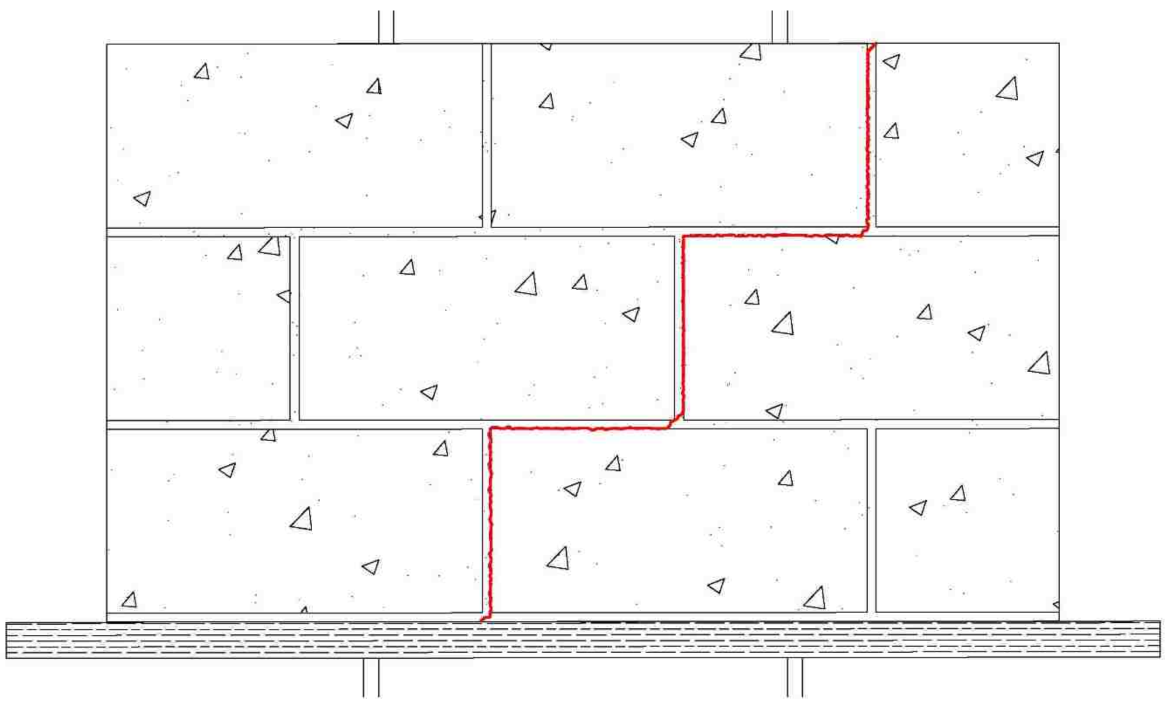

Figure 5-1: Masonry Panel Crack Propagation

Figure 5-2: Catastrophic Masonry Splitting Failure

The head connection in the reinforcement was manufactured to have more capacity than the shaft, which was verified in independent testing. As such, it is uncertain why Test Group 2 Sample A fractured at this interface. Sample C from Test Group 1 was the only panel that experienced reinforcing bar rupture. The stress strain curve for this specimen, shown in Figure 4-6, is useful for comparison with other samples as it shows the load path when the bond strength is greater than the ultimate steel capacity. This failure type however, is not representative of actual conditions because the reinforcement will always be encapsulated in grout. Furthermore, the linear regression analysis performed by the NCMA excluded data from all samples with fractured reinforcement because the capacity beyond that which was required to rupture the steel is unknown (Thomas 1999).

Each of the splices developed a tensile stress greater than 125% of the specified yield stress with the test groups averaging 150%, 138% and 134%, respectively. However, the measured values compared relatively well to Equation 2-8 predicted splice capacities with only Test Group 1 averaging a greater actual capacity. This indicates that while the shorter lap splices developed the required strength, the model from which the current design equation is derived adequately predicted the splice capacity. However, further analysis of the lap design provision of Equation 2-11 in comparison to the Equation 2-10 model from which it is based proves problematic as the strength of masonry increases. This is summarized by Table 5-1 with Equation 2-10 predicting the splice length required to achieve 125% of the rebar yield strength. The required splice lengths are also compared with the WSU linear regression model and proposed design requirement of Equations 2-7 and 2-8, respectively (Thomas 1997). Only the compressive strength of masonry varied in this analysis with Grade 60 #5 bars placed at the center of nominal 8 in. units.

Table 5-1: Recommended Splice Length with Increasing Masonry Strength

Compressive Strength of Masonry, f'_m (psi)	Equation 2-11 Code Required Splice Length, (in)	Equation 2-10 NCMA Modeled Splice Length (in)	Equation 2-7 WSU Modeled Splice Length (in)	Equation 2-8 WSU Suggested Splice Length (in)
1500	22.48	21.84	24.71	25.94
2000	19.47	15.53	21.26	22.46
2500	17.41	9.97	18.22	20.09
2774	16.53	7.15	16.68	19.07
3000	15.89	4.94	15.47	18.34
3500	14.71	0.31	12.94	16.98

Table 5-1 demonstrates that the multiple linear regression model developed by the NCMA does not necessarily correlate to the current design provision with increasing compressive strength of masonry. This is likely due to the hypothesis of bond stress distribution in reinforced masonry being highly non-linear, even before plastic deformation (Thompson 1997). As such, a linear regression analysis may not always capture the actual variation with any one of the factors. This discrepancy becomes even more pronounced as the lap-splice model was simplified to produce the design requirement.

6 CONCLUSIONS

6.1 Summary

A research program was conducted to analyze the performance of lap splices in reinforced masonry using SCG. Material testing on the grout mix design provided by the ready-mix supplier was performed to determine the required development length. Twelve masonry panels with various splice-lengths were designed, constructed and tested to verify development of the minimum 125% of the yield strength of the steel reinforcement. These specimens were subjected to a monotonic controlled displacement in direct tension until failure. The ultimate splice capacities from testing were compared to the predicted strength from a multiple linear regression model.

6.2 Findings

This study is not considered to be an exhaustive evaluation of splice behavior with SCG; however, the following general conclusions can be made based on the results and analysis:

1. All of the splices placed in SCG were able to develop more than 125% of the yield strength of the specified steel reinforcement. The longer lap splices however, developed more strength and performed in a more ductile manner than those that were shorter.

2. The measured splice capacities fit the linear regression model relatively well with actual strengths generally being slightly larger than predicted. These tests are not considered conclusive enough to suggest a reduction in development length when SCG is used.

3. Reinforcement splices in masonry with SCG should perform adequately if designed using current code provisions. With more testing, a development length reduction factor for masonry with SCG could be proposed.

6.3 Recommendations for Future Research

Further testing should be conducted to observe the performance of reinforcement splices in reinforced masonry with SCG. The testing should be done in a similar manner to this program to compare results for this research as well as those of others (Hammons et al 1994, Thompson 1997, Thomas 1999).

1. A single bar size was used in this study; additional bar sizes should be tested to determine the effects that SCG might have. The minimum splice length installed in this study was 12 inches because of the minimum code requirement. If test groups with larger bars were constructed using a larger difference in lap length, the minimum required length to produce 125% of the strength could be identified.
2. Multiple mix designs should be used to verify that lap-splice performance with different SCG is relatively uniform between producers. This will also create various compressive strengths for both grout and masonry allowing the contribution from each to be examined.

3. An analysis of the current design provision should be done to verify correlation to the multiple linear regression model with variance of the design inputs. There are significant differences for both the compressive strength of masonry, f'_m, and specified steel yield strength, f_y.
4. Strain gauges should be installed within the reinforced masonry panels to observe the bond stress distribution of the splices. More advanced instrumentation could result in a better understanding of splice performance and lead to a more accurate model.
5. The total embedment of reinforcing bars in masonry should be evaluated in conjunction to development length. The lap length has been evaluated using direct tension tests to calculate the performance of the splice; however, the bond between the grout and the reinforcing bar outside of the lap-splice is likely to contribute to the capacity.

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APPENDIX A. SELF-CONSOLIDATING GROUT MIX DESIGN

Preliminary self-consolidating grout mix design was performed in an effort to validate and augment the research conducted by the NCMA on SCG. Ultimately, it was determined unfeasible to mix all of the required grout due to the volumetric constraints of available materials and laboratory equipment. Instead it was decided to use SCG provided by a local supplier to perform the final grouting of all test specimens. However, a significant amount of time and effort were invested in SCG mix design and the procedures and findings are presented here.

A.1 Materials Selection

Initial mix design began with an attempt to replicate what was reported as the best batch from Phase II of the NCMA's testing on SCG, which was proportioned by volume. The design utilized 750 lb/yd^3 of cementitious material with 33% of this being Class F fly ash and the remainder Type I/II portland cement. The same concrete sand and #8 stone used to replicate the ready-mix SCG for preliminary masonry compressive strength were utilized as the fine and coarse aggregates, respectively. These were assumed to have the same specific gravities and properties as those reported by the NCMA. Initial tests used the coarse aggregate as delivered by the supplier but further use revealed that the aggregate was contaminated with a significant amount of fines. After this observation, the coarse aggregate was washed over a No. 16 sieve to control the amount of fines within the mix.

After consulting a representative from an admixture supplier, a high-range water reducer or superplasticizer was selected. The chemical admixture was designed for use in the production of self-consolidating concrete with high levels of workability without segregation. It was reported to adhere to ASTM C494 Type A and F as well as ASTM C1017 Type I. The representative indicated that a suitable mix design for SCG could be produced without the addition of a Viscosity Modifying Admixture (VMA).

A.2 Grout Prism Procedures

Mixing procedures were established within the first few trials of the mix design process and were followed for each successive batch. All materials were measured out by weight and placed in close proximity to the mixer. The fly ash and portland cement were combined and stirred well prior to commencing. The mixer was wetted and the aggregates were homogenized for 30 seconds. Eighty percent of the water was added and the contents were allowed to mix for another 30 seconds. The fly ash and cement were then carefully introduced with another minute of mixing. The superplasticizer was combined with the remaining mix water to aid in distribution and the solution was added to the grout. After another three minutes, the mixer was turned off and a two-minute rest period was observed followed by an additional 2 minutes of mixing. Total mixing time for each batch was approximately nine minutes.

After the SCG was mixed, the slump flow test was performed according to ASTM C1611. The slump flow was recorded and a VSI value was assigned. Occasionally when the slump was inadequate, an additional amount of water was introduced and the grout was homogenized for an additional minute and the slump flow test was performed again. Prior to successive slump flow testing, the spread plate and slump cone were washed and dried. Grout prisms were then cast in accordance to ASTM C1019 in a single lift with no rodding. After 24 hours, the specimens were removed from their molds and the surface finish was observed to verify that self-consolidation had occurred without segregation. They were then placed in a fog room and allowed to cure. After approximately seven days, the grout prisms were measured and then tested under compressive monotonic loading at a displacement controlled rate of 0.1 in./min. and the strength was determined.

A.3 Results

Results from the exploratory SCG mix design are presented in Table A – 1. This summary includes important values such as preparation and test dates, water-cement ratio, slump flow, VSI rating and compressive strength. Some of the samples did not have the compressive strength tested as there was more concern placed on achieving self-consolidation and stability. More detailed results including the mix design and comments about each batch are included in Tables A – 2 through A – 15.

Table A – 1. SCG Mix Design Summary

Batch ID	Date Prepared	Date Tested	w/cm	Slump Flow (in.)	VSI	Average Compressive Strength, f'_g (psi)
SCG 4	9/28/2017	10/5/2017	0.49	26	1	3087
SCG 6	10/10/2017	10/12/2017	0.50	29	2	2738
SCG 7	10/12/2017	10/20/2017	0.45	24	0	5394
SCG 8	10/12/2017	10/20/2017	0.46	22	0	4242
SCG 9	10/17/2017	10/24/2017	0.49	25	0	4008
SCG 10	10/17/2017	10/24/2017	0.50	26	1	3185
SCG 11	10/20/2017	NA	0.50	26	0	NA
SCG 12	10/24/2017	NA	0.49	25	1	NA
SCG 13	10/26/2017	NA	0.49	25	0	NA
SCG 14	10/26/2017	11/2/2017	0.49	27.5	0	4504
SCG 15	10/26/2017	11/2/2017	0.49	26	0	4908
SCG 16	10/31/2017	NA	0.48	27	1	NA
SCG 18	11/7/2017	NA	0.49	25	1	NA

Table A – 2. SCG Preliminary Batch 4

Batch ID:	SCG 4	Mix	9/28/2017		Test	10/5/2017	
Batch Goal:	Mix preliminary self-consolidating grout to observe properties						
	Weight	Abs Vol		Abs Vol		Admixture	
Design	(lb/yd^3)	(ft^3)		(%)		Dosage	
Cementitious			% wt		***Admixture***	fl oz/cwt	
Cement (SpG = 3.15)	**500**	**2.54**	66.7%	9.4%	ADVA-405 (80% Max)	108	
Fly Ash (SpG = 2.50)	**250**	**1.60**	33.3%	5.9%			
Total Cementitious	**750**			**15.4%**			
Aggregates			% vol				
#8 Stone (SpG = 2.91)	**1413**	**7.78**	47.0%	28.8%			
Conc Sand (SpG = 2.62)	**1435**	**8.78**	53.0%	32.5%			
Water	**368**	**5.89**		21.8%			
Design Air*	**1.5%**	**0.41**		1.5%			
TOTALS	**3966**	**27.00**		100%			

Lab Batch Size (ft^3):	0.5		**Actual**		Vol	Vol
Design	Weight (lb)	Vol (ft^3)	**Wt (lb)**		(fl oz)	(mL)
Cementitions				***Admixture***		
Cementitions	9.3	0.047	**9.3**	ADVA-405	2	59
Fly Ash	4.63	0.030	**4.63**			
Total Cementitious	13.89	0.077	**13.89**			
Aggregates						
#8 Stone (ssd=0.5%)	26.17	0.144	**26.17**			
Conc Sand (ssd=0.76%)	26.57	0.163	**26.57**			
Total Aggregates	52.74	0.307	***52.74***			
Water	6.81	0.109	**6.81**			
Additional Water	0.19	0.003		**Water/Cement (w/cm)**		0.49
Water from add'nl Admix						
Design Air	1.5%	0.015				
TOTALS	73.63	0.511	**73.44**			

Plastic SCG Properties		**Hardened SCG Properties (7-Day)**						
Slump Flow (in)	26.0	Sample	a_1 (in)	a_2 (in)	b_1 (in)	b_2 (in)	c_1 (in)	P (lbs)	σ_g (psi)
VSI (#)	1	A	3.8205	3.7935	3.6825	3.6845	7.2900	38105	2717
VSI (Description)	Stable	B	3.9915	3.9925	3.7125	3.6785	7.3375	45465	3082
Remarks:		C	3.8155	3.7990	3.6115	3.6165	7.2680	47640	3462

The preliminary mix gave us some experience mixing and casting SCG grout specimens. The slump test showed some initial bleeding but not too much. However the mix showed a tendency to segregate which was evident when the prisms were cast. The mix needed to continually be mixed in order to prevent this. The capping was not done correctly as the water to gypsum ratio was too large and not enough time was alotted to cure the cap.

Table A – 3. SCG Preliminary Batch 6

Batch ID:	SCG 6	Mix	10/10/2017		Test	10/12/2017
Batch Goal:	Finalize SCG mix for stability and practice capping					

Design	Weight (lb/yd^3)	Abs Vol (ft^3)		Abs Vol (%)		Admixture Dosage
Cementitious			% wt		***Admixture***	fl oz/cwt
Cement (SpG = 3.15)	**500**	**2.54**	66.7%	9.4%	ADVA-405 (80% Max)	108
Fly Ash (SpG = 2.50)	**250**	**1.60**	33.3%	5.9%		
Total Cementitious	**750**			**15.4%**		
Aggregates			% vol			
#8 Stone (SpG = 2.91)	**1413**	**7.78**	47.0%	28.8%		
Conc Sand (SpG = 2.62)	**1435**	**8.78**	53.0%	32.5%		
Water	**368**	**5.89**		21.8%		
Design Air*	**1.5%**	**0.41**		1.5%		
TOTALS	**3966**	**27.00**		100%		

Lab Batch Size (ft^3):	0.5		**Actual**			Vol	Vol
Design	Weight (lb)	Vol (ft^3)	**Wt (lb)**			(fl oz)	(mL)
Cementitions				***Admixture***			
Cementitions	9.3	0.047	**9.3**	ADVA-405		2	59
Fly Ash	4.63	0.030	**4.63**				
Total Cementitious	13.89	0.077	**13.89**				
Aggregates							
#8 Stone (ssd=0.5%)	26.17	0.144	**26.17**				
Conc Sand (ssd=0.76%)	26.57	0.163	**26.57**				
Total Aggregates	52.74	0.307	***52.74***				
Water	6.81	0.109	**6.81**				
Additional Water	0.19	0.003	7.00		**Water/Cement (w/cm)**		0.50
Water from add'nl Admix							
Design Air	1.5%	0.015					
TOTALS	73.63	0.511	**73.44**				

Plastic SCG Properties		**Hardened SCG Properties (2-Day)**							
Slump Flow (in)	29.0	Sample	a_1 (in)	a_2 (in)	b_1 (in)	b_2 (in)	c_1 (in)	P (lbs)	σ_g (psi)
VSI (#)	2	A	3.8010	3.6055	3.7905	3.8025	7.3230	37350	2657
VSI (Description)	Unstable	B	3.6000	3.6510	3.5920	3.5700	7.3455	36845	2838
Remarks:		C	3.5535	3.5150	3.6140	3.6180	7.2770	34770	2721

This SCG mix was not stable and exhibited bleeding and separation. Discussion and further literature review indicate this is a result of the fines within the coarse aggregate. Another cause could be the extra water that was added to achieve a natural slump comparable to normal grout. In future trial mixes the fines within the coarse aggregate configuration will be washed through a No. 16 sieve to reduce these effects and more closely follow the mix prescribed by the NCMA in Phase II of their research.

Table A – 4. SCG Preliminary Batch 7

Batch ID:	SCG 7		Mix	10/12/2017		Test	10/20/2017
Batch Goal:	Water sensitivity testing for coarse aggregate mix with fines for stability						

Design	Weight (lb/yd^3)	Abs Vol (ft^3)		Abs Vol (%)		Admixture Dosage
Cementitious			% wt		***Admixture***	fl oz/cwt
Cement (SpG = 3.15)	**500**	**2.54**	66.7%	9.4%	ADVA-405 (80% Max)	108
Fly Ash (SpG = 2.50)	**250**	**1.60**	33.3%	5.9%		
Total Cementitious	**750**			**15.4%**		
Aggregates			% vol			
#8 Stone (SpG = 2.91)	**1413**	**7.78**	47.0%	28.8%		
Conc Sand (SpG = 2.62)	**1435**	**8.78**	53.0%	32.5%		
Water	**368**	**5.89**		21.8%		
Design Air*	**1.5%**	**0.41**		1.5%		
TOTALS	**3966**	**27.00**		100%		

Lab Batch Size (ft^3):	0.5		**Actual**			Vol	Vol
Design	Weight (lb)	Vol (ft^3)	**Wt (lb)**			(fl oz)	(mL)
Cementitions				***Admixture***			
Cementitions	9.3	0.047	**9.3**	ADVA-405		2	59
Fly Ash	4.63	0.030	**4.63**				
Total Cementitious	13.89	0.077	**13.89**				
Aggregates							
#8 Stone (ssd=0.5%)	26.17	0.144	**26.17**				
Conc Sand (ssd=0.76%)	26.57	0.163	**26.57**				
Total Aggregates	52.74	0.307	***52.74***				
Water	6.20	0.109	**6.20**				
Additional Water		0.000			**Water/Cement (w/cm)**		0.45
Water from add'nl Admix							
Design Air	1.5%	0.015					
TOTALS	72.83	0.508	**72.83**				

Plastic SCG Properties		**Hardened SCG Properties (8-Day)**							
Slump Flow (in)	24.0	Sample	a_1 (in)	a_2 (in)	b_1 (in)	b_2 (in)	c_1 (in)	P (lbs)	σ_g (psi)
VSI (#)	0	A	3.6680	3.6790	3.7380	3.6600	7.3750	0	0
VSI (Description)	Stable	B	3.6000	3.5940	3.6570	3.6570	3.6590	72095	5481
Remarks:		C	3.5780	3.5810	3.6980	3.6660	7.2520	69945	5307

The mix was really stable cohesive and more viscous but was not robust at all. The initial mix was with 6 lbs of water and yielded a spread of 17 in. The grout was placed back into the mixer and 0.2 lbs of water was added for a total of 6.2 lbs water which then yielded a spread of 24 in. The non-robust nature of the mixture seems to originate from the extra fines in our coarse aggregate and future mixes will have the fines washed out. There was a machine error while testing the first prism and a value for failure was not achieved.

Table A – 5. SCG Preliminary Mix 8

Batch ID:	SCG 8		Mix		10/12/2017	Test	10/20/2017	
Batch Goal:	Fine aggregate and water sensitivity testing for coarse aggregate mix with fines washed							
Design	Weight (lb/yd^3)	Abs Vol (ft^3)		Abs Vol (%)			Admixture Dosage	
Cementitious			% wt		***Admixture***		fl oz/cwt	
Cement (SpG = 3.15)	**500**	**2.54**	66.7%	9.4%	ADVA-405 (80% Max)		108	
Fly Ash (SpG = 2.50)	**250**	**1.60**	33.3%	5.9%				
Total Cementitious	**750**			**15.4%**				
Aggregates			% vol					
#8 Stone (SpG = 2.91)	**1413**	**7.78**	47.0%	28.8%				
Conc Sand (SpG = 2.62)	**1435**	**8.78**	53.0%	32.5%				
Water	**368**	**5.89**		21.8%				
Design Air*	**1.5%**	**0.41**		1.5%				
TOTALS	**3966**	**27.00**		100%				

Lab Batch Size (ft^3):	0.5				**Actual Wt (lb)**		Vol (fl oz)	Vol (mL)
Design	Weight (lb)	Vol (ft^3)						
Cementitions						***Admixture***		
Cementitions	9.3	0.047			**9.3**	ADVA-405	2	59
Fly Ash	4.63	0.030			**4.63**			
Total Cementitious	13.89	0.077			**13.89**			
Aggregates			Total Moist	Amt to Adjust				
#8 Stone (ssd=0.5%)	26.17	0.144			**26.17**			
Conc Sand (ssd=0.76%)	26.57	0.163			**26.57**			
Total Aggregates	52.74	0.307			***52.74***			
Water	6.40	0.109			**6.40**			
Additional Water		0.000				**Water/Cement (w/cm)**		0.46
Water from add'nl Admix								
Design Air	1.5%	0.015						
TOTALS	73.03	0.508			**73.03**			

Plastic SCG Properties		**Hardened SCG Properties (8-Day)**							
Slump Flow (in)	22.0	Sample	a_1 (in)	a_2 (in)	b_1 (in)	b_2 (in)	c_1 (in)	P (lbs)	σ_g (psi)
VSI (#)	0	A	3.7500	3.7200	3.6470	3.6160	7.3680	54290	4003
VSI (Description)	Stable	B	3.7610	3.7390	3.6920	3.6580	7.3480	63020	4573
Remarks:		C	3.7450	3.7630	3.6440	3.6750	7.3320	57035	4152

This was the first mix with the coarse aggregate being washed. Initially 6 lbs of water was mixed and this yielded a spread of about 19.5 in which was outside of the minimum of 22 in. When the tested grout was placed back into the mixer 0.4 lbs of water was added and the spread averaged out to 22 inches. When removed from the mold the samples had not filled all of the space and there were voids along the sides and corners of the prisms. After further literature review we've determined to shoot for a spread from 26 - 28 in. as the NCMA used for their target.

Table A – 6. SCG Preliminary Grout Mix 9

Batch ID:	SCG 9	Mix	10/17/2017		Test	10/24/2017		
Batch Goal:	Water sensitivity testing to achieve stability and hit target spread (26-28 in.)							
Design	Weight (lb/yd^3)	Abs Vol (ft^3)		Abs Vol (%)			Admixture Dosage	
Cementitious			% wt		***Admixture***		fl oz/cwt	
Cement (SpG = 3.15)	**500**	**2.54**	66.7%	9.4%	ADVA-405 (80% Max)		108	
Fly Ash (SpG = 2.50)	**250**	**1.60**	33.3%	5.9%				
Total Cementitious	**750**			**15.4%**				
Aggregates			% vol					
#8 Stone (SpG = 2.91)	**1413**	**7.78**	47.0%	28.8%				
Conc Sand (SpG = 2.62)	**1435**	**8.78**	53.0%	32.5%				
Water	**368**	**5.89**		21.8%				
Design Air*	**1.5%**	**0.41**		1.5%				
TOTALS	**3966**	**27.00**		100%				

Lab Batch Size (ft^3):	0.5					
Design	Weight (lb)	Vol (ft^3)	**Actual Wt (lb)**		Vol (fl oz)	Vol (mL)
Cementitions				***Admixture***		
Cementitions	9.3	0.047	**9.3**	ADVA-405	2	59
Fly Ash	4.63	0.030	**4.63**			
Total Cementitious	13.89	0.077	**13.89**			
Aggregates						
#8 Stone (ssd=0.5%)	26.17	0.144	**26.17**			
Conc Sand (ssd=0.76%)	26.57	0.163	**26.57**			
Total Aggregates	52.74	0.307	***52.74***			
Water	6.80	0.109	**6.80**			
Additional Water		0.000		**Water/Cement (w/cm)**		0.49
Water from add'nl Admix						
Design Air	1.5%	0.015				
TOTALS	73.43	0.508	**73.43**			

Plastic SCG Properties		**Hardened SCG Properties (7-Day)**							
Slump Flow (in)	25	Sample	a_1 (in)	a_2 (in)	b_1 (in)	b_2 (in)	c_1 (in)	P (lbs)	σ_g (psi)
VSI (#)	1	A	3.6510	3.6530	3.7280	3.7590	7.3130	52005	3804
VSI (Description)	Stable	B	3.6330	3.6410	3.6590	3.6490	7.2960	57105	4297
Remarks:		C	3.7990	3.7510	3.6990	3.6860	7.3550	54680	3923

The mix came out very stable with slight segregation and produced a spread just below the lower bound of the target. Unfortunately the prisms are not filling the voids completely and there are still holes along the corners and the sides of the prisms. After further discussion we will attempt to hit the higher bound by adjusting the percentage of the maximum recommended dosage of the superplasticizer. This should enable us to increase our slump without losing the stability we've gained through previous adjustments.

Table A – 7. SCG Preliminary Mix 10

Batch ID:	SCG 10		Mix	10/17/2017		Test		10/24/2017	
Batch Goal:	Adjust water/cement ratio to hit our targeted spread (26-28 in.)								
	Weight	Abs Vol		Abs Vol				Admixture	
Design	(lb/yd^3)	(ft^3)		(%)				Dosage	
Cementitious			% wt			***Admixture***		fl oz/cwt	
Cement (SpG = 3.15)	**500**	**2.54**	66.7%	9.4%		ADVA-405 (80% Max)		108	
Fly Ash (SpG = 2.50)	**250**	**1.60**	33.3%	5.9%					
Total Cementitious	**750**			**15.4%**					
Aggregates			% vol						
#8 Stone (SpG = 2.91)	**1413**	**7.78**	47.0%	28.8%					
Conc Sand (SpG = 2.62)	**1435**	**8.78**	53.0%	32.5%					
Water	**368**	**5.89**		21.8%					
Design Air*	**1.5%**	**0.41**		1.5%					
TOTALS	**3966**	**27.00**		100%					
Lab Batch Size (ft^3):	0.5				**Actual**			Vol	Vol
Design	Weight (lb)	Vol (ft^3)			**Wt (lb)**			(fl oz)	(mL)
Cementitions						***Admixture***			
Cementitions	9.3	0.047			**9.3**	ADVA-405		2	59
Fly Ash	4.63	0.030			**4.63**				
Total Cementitious	13.89	0.077			**13.89**				
			Total	Amt to					
Aggregates			Moist	Adjust					
#8 Stone (ssd=0.5%)	26.17	0.144			**26.17**				
Conc Sand (ssd=0.76%)	26.57	0.163			**26.57**				
Total Aggregates	52.74	0.307			***52.74***				
Water	7.00	0.109			**7.00**				
Additional Water		0.000					**Water/Cement (w/cm)**		0.50
Water from add'nl Admix									
Design Air	1.5%	0.015							
TOTALS	73.63	0.508			**73.63**				

Plastic SCG Properties		**Hardened SCG Properties**							
Slump Flow (in)	26	Sample	a_1 (in)	a_2 (in)	b_1 (in)	b_2 (in)	c_1 (in)	P (lbs)	σ_g (psi)
VSI (#)	1	A	3.6630	3.6750	3.6760	3.6660	7.3290	42320	3142
VSI (Description)	Good	B	3.6970	3.7130	3.6240	3.6500	7.3310	42885	3183
Remarks:		C	3.7660	3.7660	3.6760	3.6940	7.4320	44850	3232

This mix turned out pretty well. Initially we had planned to test with 6.60 lbs water but 7 lbs was used. This was fortuitous because the mix exhibited good stability but had some bleeding and segregation. It was determined to use 6.81 lbs of water for the next mix to try to get as stable a mix as possible. The samples were not fully consolidated and had voids along the corners and sides.

Table A – 8. SCG Preliminary Mix 11

Batch ID:	SCG 11	Mix	10/20/2017				
Batch Goal:	Increase superplasticizer dosage and observe properties						
Design	Weight (lb/yd^3)	Abs Vol (ft^3)		Abs Vol (%)			Admixture Dosage
Cementitious			% wt			***Admixture***	fl oz/cwt
Cement (SpG = 3.15)	**500**	**2.54**	66.7%	9.4%		ADVA-405 (85% Max)	115
Fly Ash (SpG = 2.50)	**250**	**1.60**	33.3%	5.9%			
Total Cementitious	**750**			**15.4%**			
Aggregates			% vol				
#8 Stone (SpG = 2.91)	**1413**	**7.78**	47.0%	28.8%			
Conc Sand (SpG = 2.62)	**1435**	**8.78**	53.0%	32.5%			
Water	**368**	**5.89**		21.8%			
Design Air*	**1.5%**	**0.41**		1.5%			
TOTALS	**3966**	**27.00**		100%			

Lab Batch Size (ft^3):	0.5				**Actual**		Vol	Vol
Design	Weight (lb)	Vol (ft^3)			**Wt (lb)**		(fl oz)	(mL)
Cementitions						***Admixture***		
Cementitions	9.3	0.047			**9.3**	ADVA-405	2	63
Fly Ash	4.63	0.030			**4.63**			
Total Cementitious	13.89	0.077			**13.89**			
Aggregates			Total Moist	Amt to Adjust				
#8 Stone (ssd=0.5%)	26.17	0.144			**26.17**			
Conc Sand (ssd=0.76%)	26.57	0.163			**26.57**			
Total Aggregates	52.74	0.307			***52.74***			
Water	6.81	0.109			**6.81**			
Additional Water		0.000				**Water/Cement (w/cm)**		0.49
Water from add'nl Admix								
Design Air	1.5%	0.015						
TOTALS	73.44	0.508			**73.44**			

Plastic SCG Properties	
Slump Flow (in)	26
VSI (Description)	0

Remarks:

This mixture was very stable and exhibited no signs of segregation or bleeding. The mixture is also quite robust as evidenced by the slight increase in superplasticizer not affecting the spread very much. In future mixes additional care should be taken to adjust for the additional fluid from the admixture being subtracted from the water dosage.

Table A – 9. SCG Preliminary Mix 12

Batch ID:	SCG 12	Mix	10/24/2017					
Batch Goal:	Adjust coarse/fine aggregate percentages to get more paste							
Design	Weight (lb/yd^3)	Abs Vol (ft^3)		Abs Vol (%)			Admixture Dosage	
Cementitious			% wt			***Admixture***	oz/cwt	ft^3/cwt
Cement (SpG = 3.15)	**500**	**2.54**	66.7%	9.4%		ADVA-405 (92% Max)	124	0.130
Fly Ash (SpG = 2.50)	**250**	**1.60**	33.3%	5.9%				
Total Cementitious	**750**			**15.4%**				
Aggregates			% vol					
#8 Stone (SpG = 2.91)	**1338**	**7.37**	44.5%	27.3%				
Conc Sand (SpG = 2.62)	**1503**	**9.19**	55.5%	34.0%				
Water	**368**	**5.89**		21.8%				
Design Air*	**1.5%**	**0.41**		1.5%				
TOTALS	**3958**	**27.00**		100%				

Lab Batch Size (ft^3):	0.5				**Actual Wt (lb)**		Vol (fl oz)	Vol (mL)
Design	Weight (lb)	Vol (ft^3)						
Cementitious						***Admixture***		
Cementitions	9.3	0.047			**9.3**	ADVA-405	2	68
Fly Ash	4.63	0.030			**4.63**			
Total Cementitious	13.89	0.077			**13.89**			
Aggregates			Total Moist	Amt to Adjust				
#8 Stone (ssd=0.5%)	24.78	0.136	**0.87%**		**24.78**			
Conc Sand (ssd=0.76%)	27.82	0.170	**0.53%**		**27.82**			
Total Aggregates	52.60	0.307			***52.60***			
Water	6.81	0.109			**6.81**			
Additional Water	0.00	0.000				**Water/Cement (w/cm)**		0.49
Water from add'nl Admix	0	0						
Design Air	1.5%	0.015						
TOTALS	73.30	0.508			**73.30**			

Plastic SCG Properties	
Slump Flow (in)	25
VSI (#)	1
VSI (Description)	Stable

Remarks:

This mix was really stable but did not hit our target slump flow. The initial slump test yielded a slump of 24 in. so the grout was returned to the mixer and approximately 5 mL of superplasticizer was added. This gave us a little bit more slump but was slightly less stable. The grout prisms still exhibited voids which indicates that the grout is not self-consolidating. Further mix variations will include changing the ratio of sand to gravel, superplasticizer dosage, increased volume, and a different sand gradation. Prisms were not tested in compression.

Table A – 10. SCG Preliminary Mix 13

Batch ID:	SCG 13		Mix	10/26/2017				
Batch Goal:	Adjust coarse/fine aggregate percentages to get more paste							
	Weight	Abs Vol		Abs Vol			Admixture	
Design	(lb/yd^3)	(ft^3)		(%)			Dosage	
Cementitious			% wt			***Admixture***	oz/cwt	ft^3/cwt
Cement (SpG = 3.15)	**500**	**2.54**	66.7%	9.4%		ADVA-405 (100% Max)	135	0.141
Fly Ash (SpG = 2.50)	**250**	**1.60**	33.3%	5.9%				
Total Cementitious	**750**			**15.4%**				
Aggregates			% vol					
#8 Stone (SpG = 2.91)	**1263**	**6.95**	42.0%	25.8%				
Conc Sand (SpG = 2.62)	**1570**	**9.60**	58.0%	35.6%				
Water	**368**	**5.89**		21.8%				
Design Air*	**1.5%**	**0.41**		1.5%				
TOTALS	**3951**	**27.00**		100%				

Lab Batch Size (ft^3):	0.5				**Actual**		Vol	Vol
Design	Weight (lb)	Vol (ft^3)			**Wt (lb)**		(fl oz)	(mL)
Cementitions						***Admixture***		
Cementitions	9.3	0.047			**9.3**	ADVA-405	3	74
Fly Ash	4.63	0.030			**4.63**			
Total Cementitious	13.89	0.077			**13.89**			
			Total	Amt to				
Aggregates			Moist	Adjust				
#8 Stone (ssd=0.5%)	23.39	0.129	**0.87%**		**23.39**			
Conc Sand (ssd=0.76%)	29.08	0.178	**0.53%**		**29.08**			
Total Aggregates	52.46	0.307			***52.46***			
Water	6.81	0.109			**6.81**			
Additional Water	0.00	0.000				**Water/Cement (w/cm)**		0.49
Water from add'nl Admix	0	0						
Design Air	1.5%	0.015						
TOTALS	73.16	0.508			**73.16**			

Plastic SCG Properties	
Slump Flow (in)	25
VSI (#)	0
VSI (Description)	Very Stable

Remarks:

This mix design was very stable but seemed to lack the plastic qualities that we are looking for. The self-healing test was better than the other mixes that we've done but still not great. The slump flow was not within the target range that we were looking for. When the samples were removed from their molds there were still many voids present indicating that it did not self-consolidate. Because of this the prisms will not be tested in compression.

Table A – 11. SCG Preliminary Mix 14

Batch ID:	SCG 14	10/26/2017	Test 11/2/2017
Batch Goal:	Adjust batch size to see if self-consolidation is achieved		

Design	Weight (lb/yd^3)	Abs Vol (ft^3)		Abs Vol (%)
Cementitious			% wt	
Cement (SpG = 3.15)	**500**	**2.54**	66.7%	9.4%
Fly Ash (SpG = 2.50)	**250**	**1.60**	33.3%	5.9%
Total Cementitious	**750**			**15.4%**
Aggregates			% vol	
#8 Stone (SpG = 2.91)	**1263**	**6.95**	42.0%	25.8%
Conc Sand (SpG = 2.62)	**1570**	**9.60**	58.0%	35.6%
Water	**368**	**5.89**		21.8%
Design Air*	**1.5%**	**0.41**		1.5%
TOTALS	**3951**	**27.00**		100%

Admixture	Admixture Dosage oz/cwt	ft^3/cwt
ADVA-405 (100% Max)	135	0.141

Lab Batch Size (ft^3): 1

Design	Weight (lb)	Vol (ft^3)	Total Moist	Amt to Adjust	**Actual Wt (lb)**
Cementitions					
Cementitions	18.5	0.094			**18.5**
Fly Ash	9.26	0.059			**9.26**
Total Cementitious	27.78	0.154			**27.78**
Aggregates					
#8 Stone (ssd=0.5%)	46.77	0.258	**0.87%**		**46.77**
Conc Sand (ssd=0.76%)	58.16	0.356	**0.53%**		**58.16**
Total Aggregates	104.93	0.613			***104.93***
Water	13.61	0.218			**13.61**
Additional Water	0.00	0.000			
Water from add'nl Admix	0	0			
Design Air	1.5%	0.015			
TOTALS	146.32	1.000			**146.32**

Admixture	Vol (fl oz)	Vol (mL)
ADVA-405	5	148

Water/Cement (w/cm) 0.49

Plastic SCG Properties	
Slump Flow (in)	27.5
T_{20} (sec)	
VSI (#)	0
VSI (Description)	Very Stable

Hardened SCG Properties

Sample	a_1 (in)	a_2 (in)	b_1 (in)	b_2 (in)	c_1 (in)	P (lbs)	σ_g (psi)
A	4.0450	4.0130	4.0142	4.0110	7.3385	77705	4806
B	4.1405	4.0675	4.0810	4.0610	7.3200	74955	4486
C	4.0870	4.0630	4.0820	4.0500	7.3025	69895	4218

Remarks:

This mix design had the same proportions as SCG 13 but was double in volume. The mix was much more workable than SCG 13 and exhibited very good stability. This was the first time that we achieved our target slump flow range and was on the high end. The self-healing test was really good but could have been slightly better. In further tests we will use this mix design. We'll verify if 1 ft^3 is representative of any larger batch size by doing a 2 ft^3 mix.

Table A – 12. SCG Preliminary Mix 15

Batch ID:	SCG 15	Mix	10/26/2017		Test	11/2/2017		
Batch Goal:	Use NCMA concrete sand gradation to achieve self-consolidation							
Design	Weight (lb/yd^3)	Abs Vol (ft^3)		Abs Vol (%)		Admixture Dosage		
Cementitious			% wt		***Admixture***		oz/cwt	ft^3/cwt
Cement (SpG = 3.15)	**500**	**2.54**	66.7%	9.4%	ADVA-405 (100% Max)		135	0.141
Fly Ash (SpG = 2.50)	**250**	**1.60**	33.3%	5.9%				
Total Cementitious	**750**			**15.4%**				
Aggregates			% vol					
#8 Stone (SpG = 2.91)	**1413**	**7.78**	47.0%	28.8%				
Conc Sand (SpG = 2.62)	**1435**	**8.78**	53.0%	32.5%				
Water	**368**	**5.89**		21.8%				
Design Air*	**1.5%**	**0.41**		1.5%				
TOTALS	**3966**	**27.00**		100%				

Lab Batch Size (ft^3):	0.5		**Actual**		Vol	Vol
Design	Weight (lb)	Vol (ft^3)	**Wt (lb)**		(fl oz)	(mL)
Cementitions				***Admixture***		
Cementitions	9.3	0.047	**9.3**	ADVA-405	3	74
Fly Ash	4.63	0.030	**4.63**			
Total Cementitious	13.89	0.077	**13.89**			
Aggregates						
#8 Stone (ssd=0.5%)	26.17	0.144	**26.17**			
Conc Sand (ssd=0.76%)	26.57	0.163	**26.57**			
Total Aggregates	52.74	0.307	***52.74***			
Water	6.81	0.109	**6.81**			
Additional Water	0.00	0.000		**Water/Cement (w/cm)**		0.49
Water from add'nl Admix	0	0				
Design Air	1.5%	0.015				
TOTALS	73.44	0.508	**73.44**			

Plastic SCG Properties		**Hardened SCG Properties**							
Slump Flow (in)	26	Sample	a_1 (in)	a_2 (in)	b_1 (in)	b_2 (in)	c_1 (in)	P (lbs)	σ_g (psi)
VSI (#)	0	A	4.2010	4.2255	4.0350	4.0715	7.3120	78820	4615
VSI (Description)	Very Stable	B	4.0550	4.0455	4.0675	4.0860	7.2830	86675	5249
Remarks:		C	4.0935	4.1125	4.0490	4.0760	7.3190	81015	4860

This mix was performed using a fine aggregate gradation that was the same as that used by the NCMA in Phase II of their research. This mix exhibited the best plastic properties that we've observed yet. The self-healing test was performed and the mix filled the gap almost immediately. However, in further mix designs it's not feasible for us to sieve out all of our material and combine it with this gradation. The finish from SCG 14 is almost identical to these prisms which indicates that our other mix design should be sufficient for our purposes.

Table A – 13. SCG Preliminary Mix 16

Batch ID: SCG 16 Mix 10/31/2017

Batch Goal: Adjust batch size to see if self-consolidation is achieved

Design	Weight (lb/yd^3)	Abs Vol (ft^3)		Abs Vol (%)		Admixture Dosage	
Cementitious			% wt		***Admixture***	oz/cwt	ft^3/cwt
Cement (SpG = 3.15)	**500**	**2.54**	66.7%	9.4%	ADVA-405 (100% Max)	135	0.141
Fly Ash (SpG = 2.50)	**250**	**1.60**	33.3%	5.9%			
Total Cementitious	**750**			**15.4%**			
Aggregates			% vol				
#8 Stone (SpG = 2.91)	**1263**	**6.95**	42.0%	25.8%			
Conc Sand (SpG = 2.62)	**1570**	**9.60**	58.0%	35.6%			
Water	**368**	**5.89**		21.8%			
Design Air*	**1.5%**	**0.41**		1.5%			
TOTALS	**3951**	**27.00**		100%			

Lab Batch Size (ft^3):	2							
Design	Weight (lb)	Vol (ft^3)	Total Moist	Amt to Adjust	**Actual Wt (lb)**		Vol (fl oz)	Vol (mL)
Cementitions						***Admixture***		
Cementitions	37.0	0.188			**37.0**	ADVA-405	10	296
Fly Ash	18.52	0.119			**18.52**			
Total Cementitious	55.56	0.307			**55.56**			
Aggregates								
#8 Stone (ssd=0.5%)	93.55	0.515	**0.87%**		**93.55**			
Conc Sand (ssd=0.76%)	116.31	0.711	**0.53%**		**116.31**			
Total Aggregates	209.86	1.227			***209.86***			
Water	26.57	0.436			**26.57**			
Additional Water	0.00	0.000				**Water/Cement (w/cm)**		0.48
Water from add'nl Admix	0	0						
Design Air	1.5%	0.015						
TOTALS	291.99	1.985			**291.99**			

Plastic SCG Properties	
Slump Flow (in)	27
VSI (#)	1
VSI (Description)	Stable

Remarks:

The larger volume mix was not as good as the mix done with 1 ft^3 as far as stability. The slump flow was within our target parameters but exhibited a mortar halo and sheen. While this was very good in order to move forward we should probably refine our mix slightly in order to obtain a VSI of 0. Segregation was especially evident when the excess material was discarded as three very distinct portions formed. One factor could be that the increased amount of admixture introduced too much fluid for our mix and at higher volumes this needs to be accounted for.

Table A – 14. SCG Preliminary Mix 18

Batch ID:	SCG 18		11/7/2017					
Batch Goal:	Use large mixer with 1 ft^3 to check the variance							
	Weight	Abs Vol		Abs Vol			Admixture	
Design	(lb/yd^3)	(ft^3)		(%)			Dosage	
Cementitious			% wt			***Admixture***	oz/cwt	ft^3/cwt
Cement (SpG = 3.15)	**500**	**2.54**	66.7%	9.4%		ADVA-405 (100% Max)	135	0.141
Fly Ash (SpG = 2.50)	**250**	**1.60**	33.3%	5.9%				
Total Cementitious	**750**			**15.4%**				
Aggregates			% vol					
#8 Stone (SpG = 2.91)	**1263**	**6.95**	42.0%	25.8%				
Conc Sand (SpG = 2.62)	**1570**	**9.60**	58.0%	35.6%				
Water	**368**	**5.89**		21.8%				
Design Air*	**1.5%**	**0.41**		1.5%				
TOTALS	**3951**	**27.00**		100%				
Lab Batch Size (ft^3):	1				**Actual**		Vol	Vol
Design	Weight (lb)	Vol (ft^3)			**Wt (lb)**		(fl oz)	(mL)
Cementitious						***Admixture***		
Cementitions	18.5	0.094			**18.5**	ADVA-405	5	148
Fly Ash	9.26	0.059			**9.26**			
Total Cementitious	27.78	0.154			**27.78**			
			Total	Amt to				
Aggregates			Moist	Adjust				
#8 Stone (ssd=0.5%)	46.77	0.258	**0.87%**		**46.77**			
Conc Sand (ssd=0.76%)	58.16	0.356	**0.53%**		**58.16**			
Total Aggregates	104.93	0.613			***104.93***			
Water	13.61	0.218			**13.61**			
Additional Water	0.00	0.000				**Water/Cement (w/cm)**		0.49
Water from add'nl Admix	0	0						
Design Air	1.5%	0.015						
TOTALS	146.32	1.000			**146.32**			
Plastic SCG Properties								
Slump Flow (in)	25							
VSI (#)	1							
VSI (Description)	Stable							

Remarks:

Utilizing the large mixer for a lower volume was not ideal as it did not appear that the paddles engaged sufficiently. The grout was pretty stable but did not achieve the target VSI of 0. This helped solidify the notion that the mixer is partially responsible for the variation in plastic properties when the mix is scaled. It is becoming very apparent that SCG is very sensitive to any sort of variation. Because this was only a verification of hypothesis prisms were not cast.

A.4 Discussion

While attempting to develop an adequate mix design for use in later phases of this research, there were various observations made regarding SCG. While it was not feasible to continue the experimentation until all issues were solved, there is value in the findings gained through experience. This portion of the research was performed by individuals who were relatively new to the theory and methodologies of mix design and those more experienced would likely achieve better results. However, some of these observations may prove useful to those desiring to produce SCG for further research or commercial purposes.

SCG is a very sensitive material to work with and slight variations in mixing procedures would produce significantly differing results. While mixing procedures were developed very early on in this process, slight variations in the process could have contributed to the results. When an initial SCG mix design was determined to be adequate for use, the volume was increased and a larger mixer was utilized. This resulted in plastic properties that were much less stable than what was obtained with smaller batch sizes. As such, great care should be observed when scaling mixes to larger sizes for ready-mix applications.

Aggregate gradation also had a significant effect on the plastic properties of the grout. Control is especially important as evidenced by the requirement that the coarse grout be washed to remove the fines contaminating the material. Variation observed farther along in this process was believed to be an effect in the method that the fine aggregate was obtained for mixing. The concrete sand was stored in a large concrete receptacle with a chute for dispensing. Sometimes the sand dropped into a metal pan at the base and was then scooped out, whereas other times the sand fell directly into the container used for weighing. While seemingly insignificant, this small variance in method is believed to have played a large role in the inconsistencies from batch to batch. The amount of small fines is believed to be of special concern as these contribute to the formation and consistency of the paste which is essential for the rheological properties of the grout. The batch with the best plastic characteristics was obtained through a fine aggregate manufactured by combining portions that had been separated by sieving. However, this method was used only once as the labor intensive process was not practical. What typically may be categorized as normal variation in aggregate gradation for conventional grout may produce insufficient results if used for SCG. Researchers and suppliers seeking to produce SCG may have trouble with locally available aggregate sources and may need to blend two or more to achieve a suitable gradation.

The use of different admixtures could also aid in achieving a suitable SCG mix design. For this research program, a single high-range water reducing admixture was used; however, the use of a VMA or an air-entraining admixture could have improved the rheological properties of the mix. While air-entrainers are typically used to improve durability via resistance to freeze-thaw, they also improve workability and consolidation which are desirable qualities within SCG. However, a superplasticizer may adversely affect the ability to entrain air and caution should be used when entrained air is needed. A VMA could potentially provide better grout cohesiveness thereby reducing segregation and making the grout more stable (Mindess, 2003).

Testing of the SCG prisms resulted in relatively high compressive strengths, even though the samples were all tested within 8-days of being cast. High strengths were also obtained by the NCMA with all samples tested at 28-days (NCMA, 2007). The *Building Code Requirements for Masonry Structures* states that specified compressive strength of grout should not exceed 5000 psi. This upper limit is due to a lack of available research with higher material strengths (TMS

402 2016). Typically, it is desirable that the masonry unit and the grout have similar compressive strengths so that the masonry will act as a composite of similar properties. However, it seems that SCG will likely achieve compressive strengths much greater than the CMU leading to the possibility of performance issues.

A.5 Conclusions

Preliminary SCG mix design was performed to develop a mix for use in further testing. This study attempted to follow and expand upon the research performed by the NCMA. Multiple batches of grout were produced and their plastic properties were observed through the slump flow test and casting grout prisms. The effects of variables such as water-cement ratio, aggregate gradation, admixture dosage and batch volume were explored. The compressive strength of grout was also determined for many of the mixes. There were a few mix designs that proved satisfactory for use in continued research, but it was determined infeasible to mix the required volume of grout with the available materials and equipment.

While this phase of the research was not an extensive testing program and the results are considered incomplete, the following conclusions can be made:

1. SCG is a very sensitive material that requires a lot of control in the production process. Simply adding superplasticizer to a conventional grout mix is not likely to produce satisfactory results. An SCG mix design needs to be developed specifically for this application.
2. The aggregate gradation is vital to achieve a stable mix that will self-consolidate within reinforced masonry. The fine aggregate is of special consideration as this

affects the ability of the paste to suspend and transport the coarse aggregate without segregation.

3. Variations in mix procedure can create diverse results in SCG. Mix designs generated using small volume batches may not scale well when produced in larger quantity.

Additional SCG testing should be done to refine and enhance the results of this study. Multiple questions remain which require additional study to resolve. The following items are suggested for future research:

1. The paste volume within the SCG seemed to fluctuate with each mix based on the amount of small fines. An ideal paste volume for cohesive SCG should be explored such that the plastic properties are consistent.

2. Rheological properties of SCG in research have only been quantified by visual observation through the slump flow test, self-healing test and the T20 or T50 tests. Further testing could utilize a rheometer to provide a less subjective measure of the fresh SCG parameters such as thixotropy, shear strength and viscosity.

3. The compressive strengths for SCG appear to be higher than the maximum limit set within the masonry building code. A program to develop lower strength SCG meeting this criterion would be beneficial.

APPENDIX B. RESULTS

Table B-1: Preliminary Masonry Prism Compressive Testing Results

Sample	a_1 (in)	a_2 (in)	b_1 (in)	b_2 (in)	Area (in^2)	Load (lbs)	Compressive Strength, f'_m (psi)
A	7.652	7.641	7.624	7.6595	57.88	175749	3036
B	7.5855	7.5755	7.674	7.6965	57.70	160788	2786
C	7.631	7.6425	7.646	7.642	57.82	172278	2979
D	7.6665	7.6605	7.604	7.596	57.69	187864	3256
E	7.581	7.574	7.6735	7.6615	57.55	151724	2637
F	7.6445	7.635	7.676	7.6875	58.13	148505	2555
Average					57.80	166151	2875

Table B-2: Preliminary Grout Prism Compressive Testing Results

Sample	a_1 (in)	a_2 (in)	b_1 (in)	b_2 (in)	Area (in^2)	Load (lbs)	Compressive Strength, f'_g (psi)
2b A	4.056	4.074	3.97	3.977	16.15	83950	5197
2b B	4.021	4.03	4.028	4.025	16.21	74405	4590
2b C	4.143	4.09	4.025	3.982	16.48	68065	4130
3b A	3.939	3.945	4.102	4.069	16.11	76290	4737
3b B	3.922	3.929	4.085	4.049	15.97	79335	4969
3b C	4.027	3.967	4.037	4.018	16.10	71250	4426
Average					16.17	75549	4675

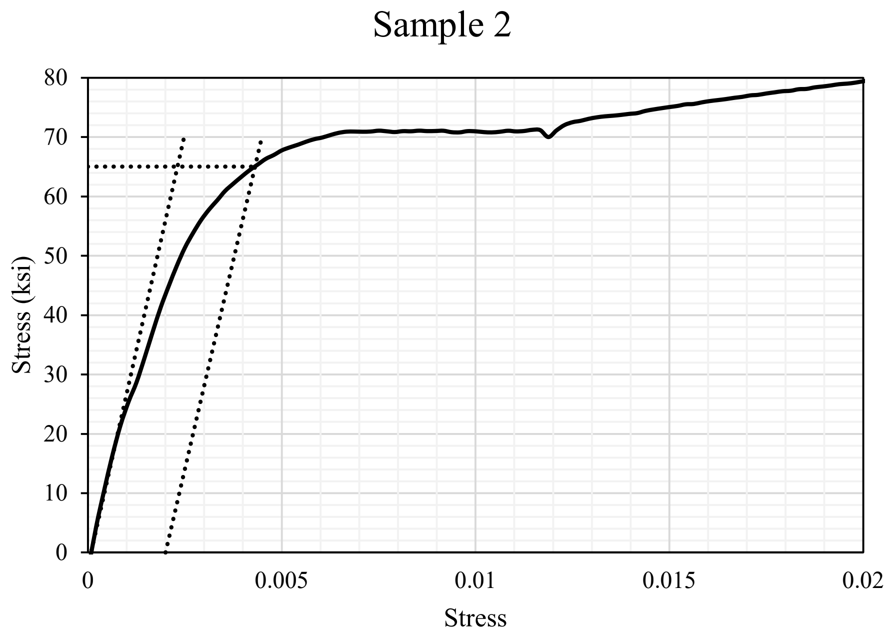


Figure B-1: Yield Stress-Strain Curve for Reinforcement Sample 2

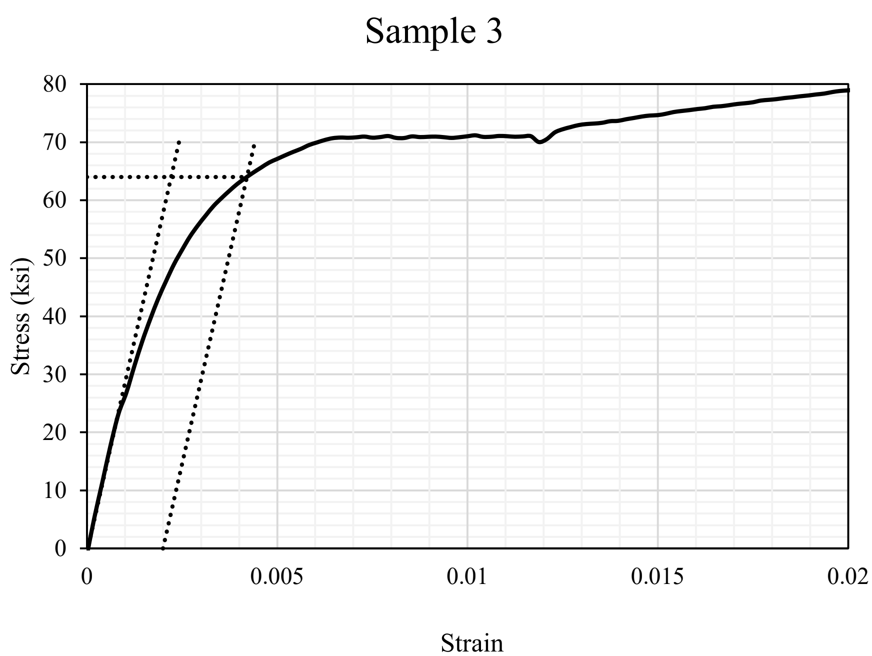


Figure B-2: Yield Stress-Strain Curve for Reinforcement Sample 3

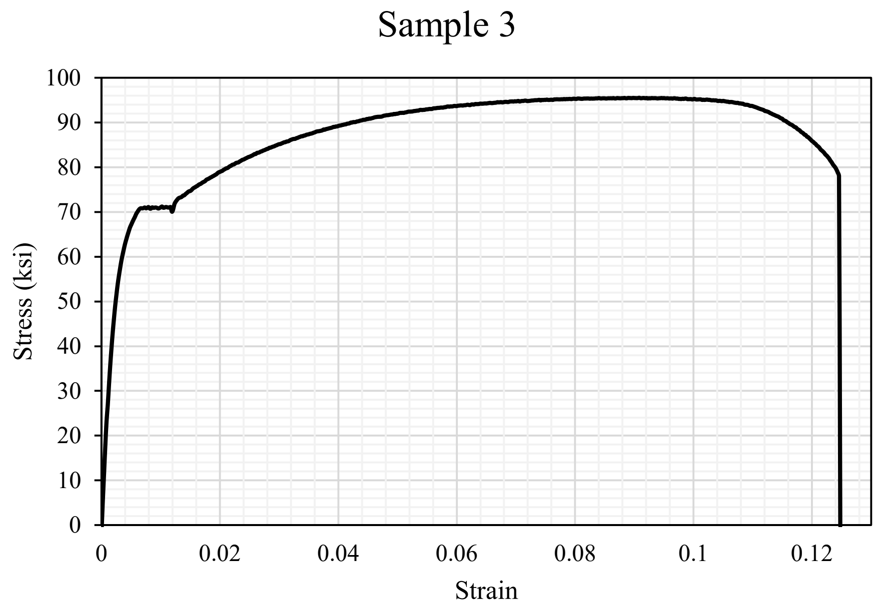


Figure B-3: Stress-Strain Curve for Reinforcement Sample 3

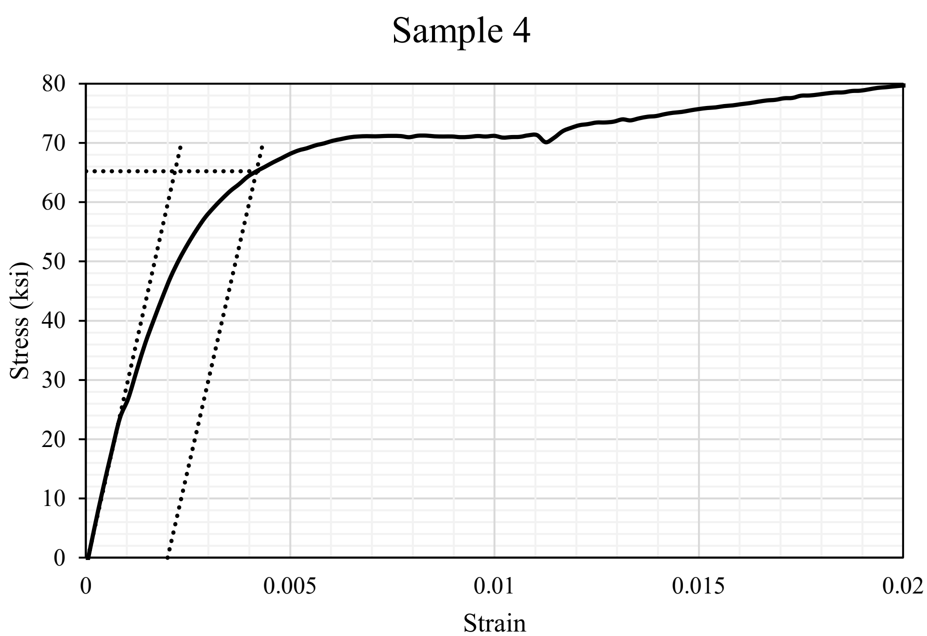


Figure B-4: Yield Stress-Strain Curve for Reinforcement Sample 4

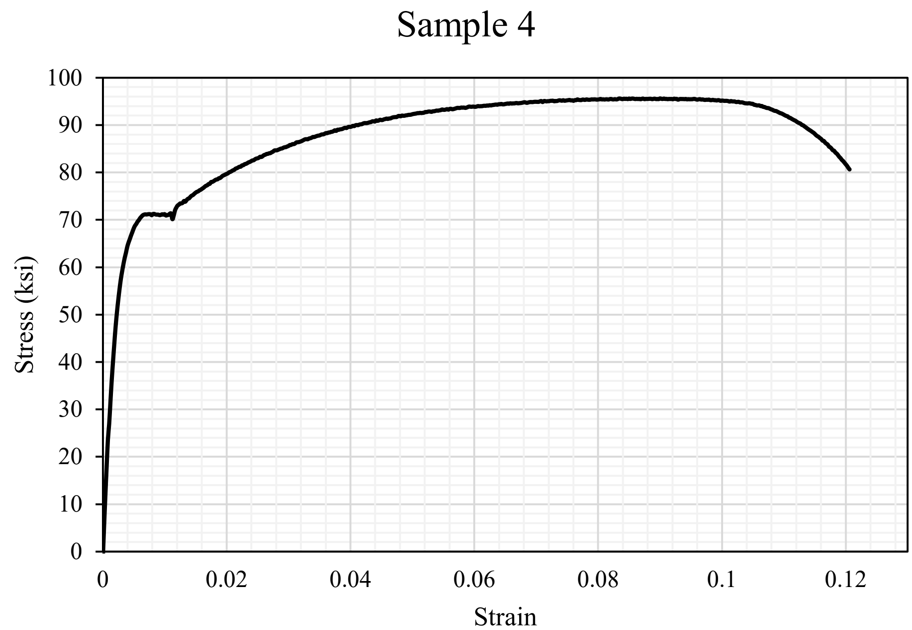


Figure B-5: Stress-Strain Curve for Reinforcement Sample 4

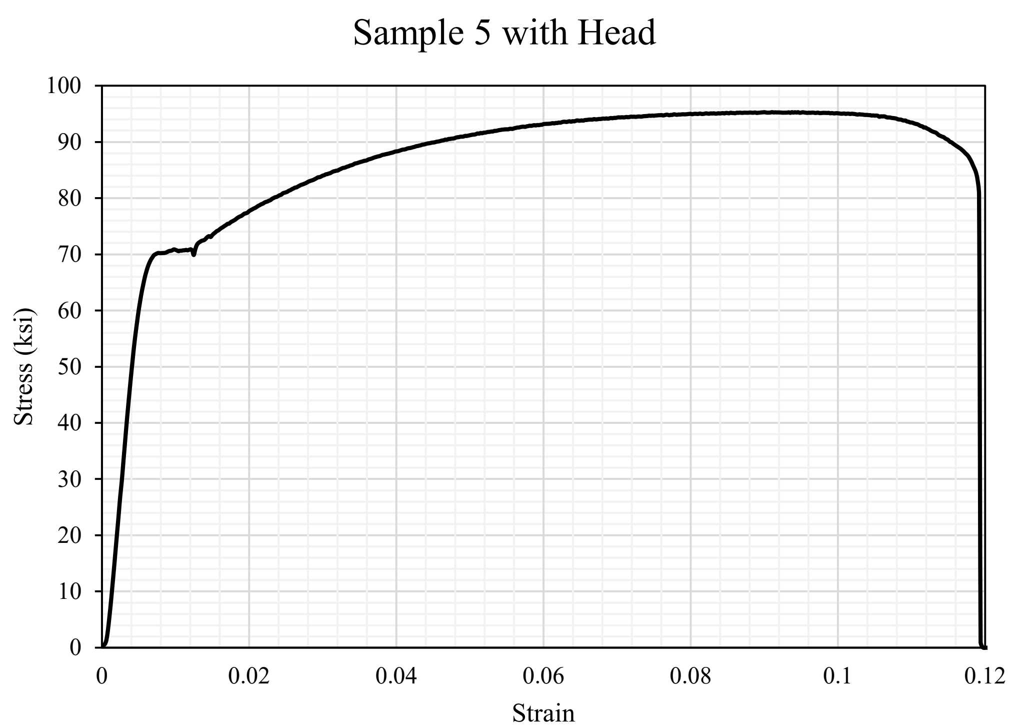


Figure B-6: Stress Strain Curve for Reinforcement Sample 5

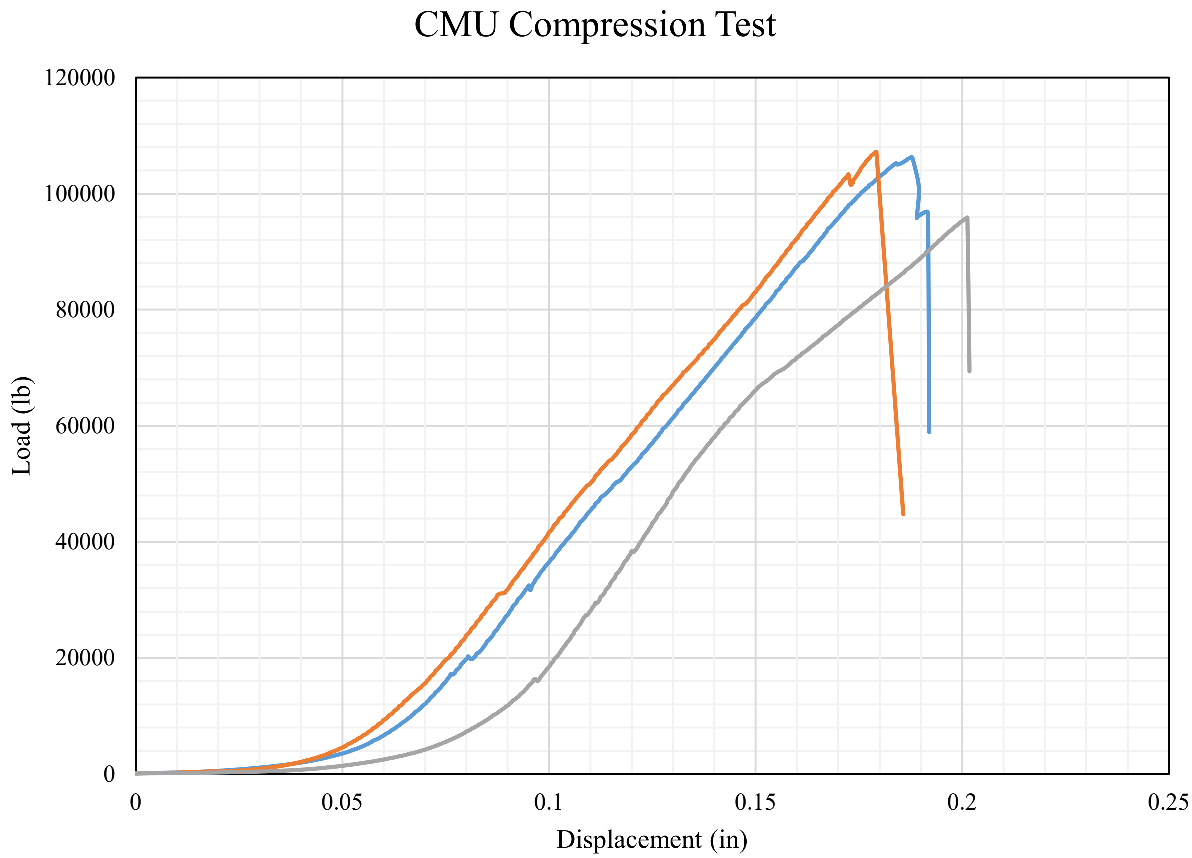


Figure B-7: Load vs. Displacement Plot for Masonry Units

APPENDIX C. SPECIMEN SCHEMATICS AND PHOTOGRAPHS

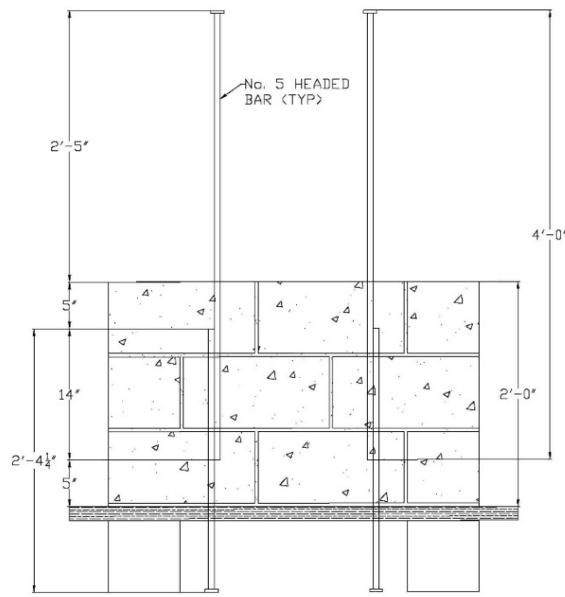


Figure C-1: Test Group 2 Schematic

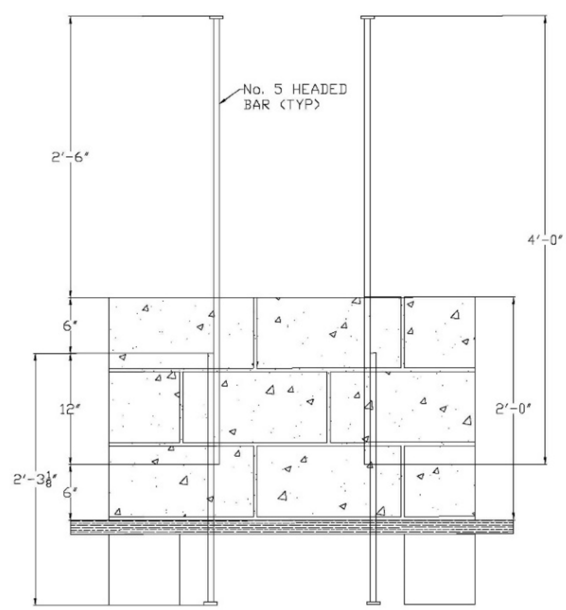


Figure C-2: Test Group 3 Schematic

A B C

Figure C-3: SCG Prisms at 28-Day Failure

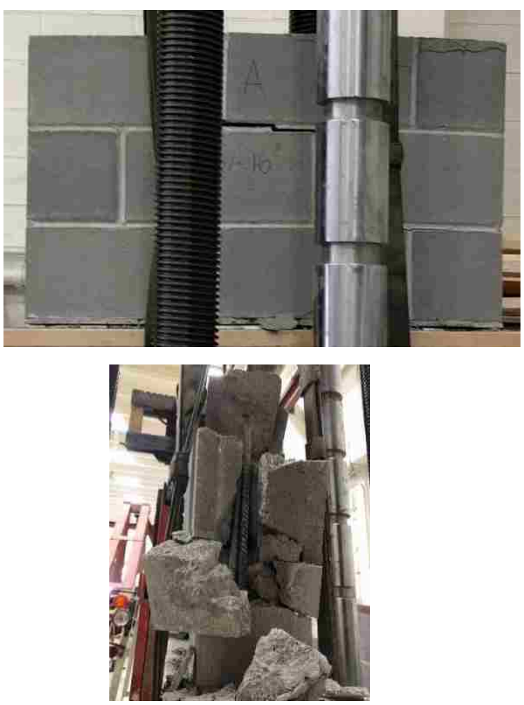
D E F

Figure C-4: SCG Prisms at 32-Day Failure

Prism 1 Prism 2 Prism 3

Figure C-5: Masonry Prisms at 28-Day Failure

Figure C-6: Masonry Prisms at 28-Day Failure



Panel A Panel B

Figure C-7: Test Group 1 Masonry Panels A & B at Failure

Panel C **Panel D**

Figure C-8: Test Group 1 Masonry Panels C & D at Failure

Panel E **Panel F**

Figure C-9: Test Group 1 Masonry Panels E & F at Failure

Panel A **Panel B**

Figure C-10: Test Group 2 Masonry Panels A & B at Failure

Figure C-11: Test Group 2 Masonry Panel C at Failure

Panel A

Panel B

Figure C-12: Test Group 3 Masonry Panels A & B at Failure

Figure C-13: Test Group 3 Masonry Panel C at Failure