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CAPACITY OF ANCHOR BOLTS IN CONCRETE MASONRY

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Abstract

Past research has indicated that the interaction of the shear and tensile strength of anchors embedded in concrete masonry is not linear, but is instead elliptical. Further investigation of this issue is currently underway aimed at determining whether the current code defined capacity equations are reasonably accurate and to determine the form of the interaction equation for a representative sample of masonry and anchor bolt configurations. Phase 2 of this investigation focused on the tension and shear capacity of a variety of wall and anchor configurations and tested a total of 185 anchor specimens in both shear and tension loading. Phase 3 tested 225 anchor specimens in a variety of load combinations. The results of Phase 2 and Phase 3 testing are complete and this report summarizes these results.

Key Words

Anchors, Bolts, Shear, Tension

1 Introduction

Past research indicated that the interaction of the shear strength and tensile strength of anchors embedded in concrete masonry is not linear, as is generally assumed by code defined design equations but is instead elliptical. (Fabrello-Streufert, 2003) Further investigation of this issue is currently underway at the National Concrete Masonry Association lab with assistance from North Carolina A & T State University and is aimed at determining whether the current code defined capacity equations are reasonably accurate and to determine the value of the exponent (the value of the

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power ‘N’ shown in Figure 1) for the anchor bolt shear and tension interaction equation for a representative sample of masonry and anchor bolt configurations.

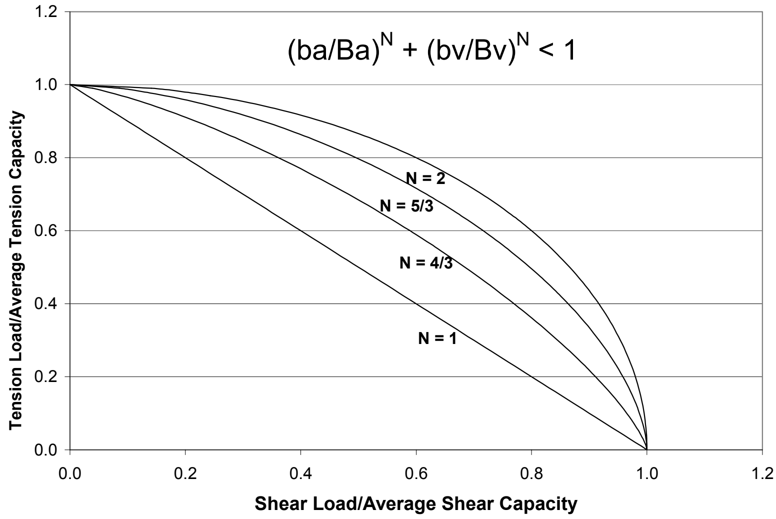

Figure 1 – Interaction of anchor shear strength to tensile strength

Phase 2 of this investigation tested a total of 185 anchor specimens (both headed and L bolt configurations) in shear or tension loading. This testing is complete and found that the anchor bolt strength level design provisions listed in the MSJC Chapter 3 (MSJC, 2002) appeared to be generally adequate for tension loading but can be quite unconservative for larger bolt diameters where masonry shear failures dominated the behaviour of the system (McGinley, 2003). The third phase of the program has also been completed. This paper describes the Phase 3 results, compares them to the results of Phase 2 and the MSJC code predicted behaviour. A regression analysis was also conducted to determine what the “best fit” value of N for the interaction equation for each of the combined loading configurations tested.

2 Testing Program

In Phase 3 of this investigation, a total of 225 masonry wallet test specimens were constructed and tested using one of the configurations described in Table 1 and Figure 2. The bolts were all cast in-place in solidly grouted 194 mm to 295 mm (8-inch or 12-inch) CMU prisms, no confinement was provided and coarse grout with a specified strength of approximately 17.2 MPa (2,500 psi) was used with normal weight concrete masonry units. The specified masonry strength, f'_m was expected to be between 10.3 MPa and 13.8 MPa (1,500 – 2,000 psi) and each specimen configuration had five replicates.

After each prism specimen was allowed to cure in laboratory conditions, it was placed in the testing frame and subjected to a static tension loading, a static shear loading, or a combined shear and tension loading. For each loading type, the load and deformation was measured until failure using the procedures described in ASTM Standard Test Method Strength of Anchors in Concrete and Masonry – E 488.

In addition to the prism tests described previously, a number of component tests were also conducted. These tests included three hollow concrete masonry unit compression tests on each unit size as described in ASTM Standard Test Methods for Sampling and Testing Concrete Masonry Units and Related Units - C 140, six grout compression tests as described in ASTM Standard Test Method for Sampling and Testing Grout - C

1019 (after moist curing for least 63 days), and a total of six compression tests of two high, fully grouted prisms as per ASTM Standard Test Method for Compressive Strength of Masonry Prisms - C 1314. These prism tests were conducted at an age of 63 days and 108 days after curing in laboratory air.

Table 1 Prism specimen configurations

Units (mm)	Anchor Type	Anchor Dia. (mm)	Embed. Depth (mm)	Edge Cover (mm)	Load Config.
Group 1 – 100% Tension Loading					
194	Headed	13, 16, 19, 22	108	300	D
194	"L" Bolt	13, 16, 19, 22	108	300	D
Group 2 – 100% Shear Loading					
194	Headed	13, 16, 19, 22	108	300	E
Group 3 – 25% Tension Capacity, Shear to Failure					
194	Headed	13, 16, 19, 22	108	300	25D/E
295	Headed	22, 25	203	300	25D/E
Group 4 – 50% Tension Capacity, Shear to Failure					
194	Headed	13, 16, 19, 22	108	300	50D/E
295	Headed	22, 25	203	300	50D/E
Group 5 – 75% Tension Capacity, Shear to Failure					
194	Headed	13, 16, 19, 22	108	300	75D/E
295	Headed	22, 25	203	300	75D/E
Group 6 – 50% Shear Capacity, Tension to Failure					
194	Headed	13,16,19	108	300	50E/D
Group 7 – 100% Tension Loading; Large Anchors					
295	Headed	22, 25	203	300	D
295	"L" Bolt	22, 25	203	300	D
Group 8 – 100% Shear Loading; Large Anchors					
295	Headed	22, 25	203	300	E
295	"L" Bolt	22, 25	203	300	E
Group 9 – Bent Bar Anchors; 50% Tension Capacity, Shear to Failure					
194	"L" Bolt	13, 16, 19, 22	108	300	50D/E

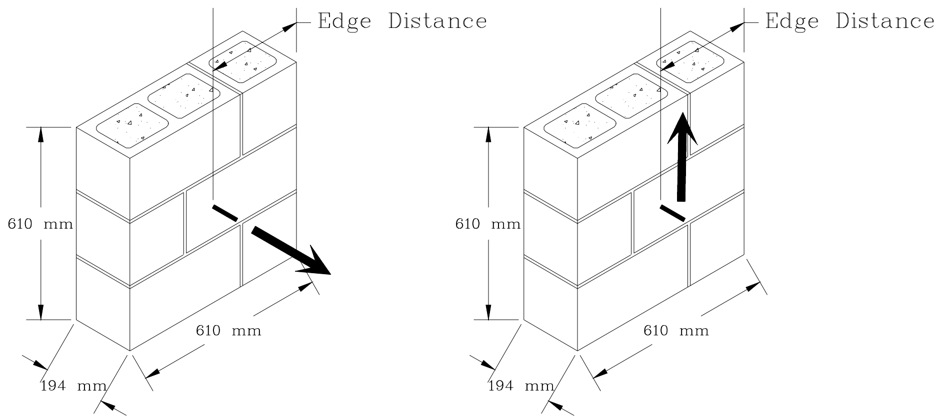

Figure 2 Specimens and loading configurations

Finally, three bolts of each type and size were tested for tension capacity by a contracted laboratory.

3 Test Results

3.1 Grout and Prism Compression Test Results

The average net area grout compression strength for the six units tested was 15 MPa (2,185 psi). This overall value meets the minimum compression stress 14 MPa (2000 psi) required for grout in the Specification for Grout for Masonry, ASTM C 476, although individually, Mix 2 did not meet this minimum compression strength requirement. The grout Mix #2 was used for all the 194 mm (8") specimens and Mix # 1 was used for all the 295 mm (12") specimens.

The average net area prism compression strength, f'_m, for all six units is 21 MPa (3,048 psi), with a COV of 4.8%. This f'_m value greatly exceeds the 14 MPa expected, even with the very low grout strength and is probably due to the relatively high unit strength (greater than 22 MPa). It should also be noted that the average strength value of 21 MPa is only 5% less than that reported for the Phase 2 results, even through the grout strengths in Phase 3 were reduced by approximately 50% compared to those in Phase 2.

3.2 Anchor Bolt Tension Test Results

The results of the bolt tests showed a range of average yield stresses that ranged from 329 MPa to 644 MPa for the headed bolts and a 308 MPa to 342 for the L Bolts. The average ultimate stress values ranged from 480 to 659 MPa for the headed bolts and 470 to 524 MPa for the L Bolts. The headed bolts greatly exceed the minimum 414 MPa (60 ksi) tensile strength requirements of ASTM A 307 steel. All the headed bolt values were generally much higher that were found for the Phase 2 bolts whose average yield stress values for headed bolts ranged from 322 to 350 MPa and average ultimate stress values ranged from 354 to 547 MPa.

The L bolts also greatly exceeded the minimum 248 MPa (36 ksi) yield and 400 MPa (58 ksi) ultimate strength requirements for ASTM A 36 steel and, for the most part, had slightly greater capacity than similar bolts tested in Phase 2.

3.3 Anchor Prism Test Results

When anchor bolts embedded in the masonry prisms were tested in tension, the bolts generally deformed little and approximately linearly up to yielding of the bolts. They then deformed plastically followed by either tension rupture of the bolts, or a vertical cracking of the masonry prisms specimens (Masonry Splitting - See Figure 3) that initiated in the head joints of the top and bottom courses. In some of the specimens there were additional radial cracks emanating from the bolts towards the specimen edges with a portion of the bock face shell attaching to the bolt. Generally all the bolts in Phase 3 and all bolts larger than 16 mm in Phase 2 failed in the masonry. In Phase 2, the 13 mm diameter L Bolts also exhibited bolt pullout failure, but the rest of the larger L bolts exhibited a masonry splitting failure. A complete summary of the results of these tests are not reported in this paper due to paper length restrictions but are available in the Phase 2 and Phase 3 reports published by NCMA.(McGinley, 2003), (McGinley, 2004).

Figure 3 Tension (load configuration D) masonry splitting failure of 13 mm (0.5 in) headed bolts with 108 mm (4.25 in) embedment

When anchor bolts embedded in masonry prisms were loaded in shear parallel to the side of the prism, there is generally little deflection until the bolts yielded. Ultimately failure occurred either as a shear rupture of the bolts in the minimum cross-sectional area (all 13 mm and 16 mm bolts in Phase 2 and 13 mm bolts in Phase 3) or a masonry failure that typically resulted from a cracking of the masonry prism. This cracking either mimicked the cracking of the 13 mm pullout (Splitting Failure) specimens as shown in Figure 3, or exhibited a vertical cracking in the lower head joint and then a crack across the middle horizontal bed joint as shown in Figure 4 (Shear Breakout)

Figure 4 - 22 mm ($^7/_8$ in.) diameter headed bolts loaded in shear typical masonry shear - break out

Some specimens also showed both types of cracking. This behaviour is typical for both L bolt and headed bolt prism specimens loaded to failure in shear and that failed in the masonry (even for combined loadings that also included appreciable tension loads).

4 Discussion

Figure 5 shows the variation of pure tension wallet test results from configurations that used headed bolts, a 108 mm (4.25 in) embedment and 194 mm (8") CMU prisms, in both Phase 2 and Phase 3. As can be seen from the graph, the higher bolt strengths and lower grout strengths of Phase 3 (with the exception for the two low strength bolts)

shifted the mode of failure in Phase 3 to a masonry failure for most of the tension loading configurations. There is also a reduction in average strengths for comparable configurations in Phase 3 versus Phase 2 for 16 mm (5/8 in) diameter bolts and higher, with this reduction becoming more pronounced as the bolt diameters get larger.

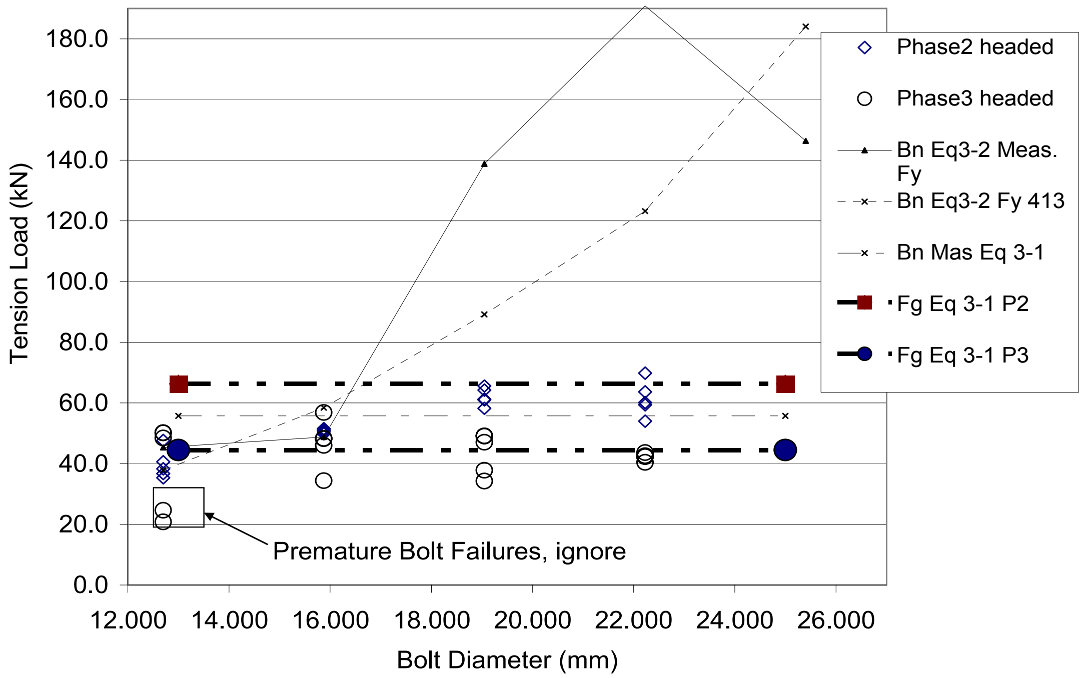

Figure 5—Tension capacities of headed bolts in 194 mm (8 in.) CMU prisms with a 108 mm ($4^1/_4$ in.) embedment

It has been concluded that, the 2002 MSJC Code strength design provisions appear to give the best prediction of anchor strength (Klingner and Weigel, 2003) In this code the tensile capacity of headed fasteners is governed by the following equations (MSJC, 2002): (Note that these equations were used in their US standard unit form and converted to SI after)

MSJC Equation 3-2, is used to define the steel capacity of the anchor bolts.

$$\phi B_{an} = \phi A_b f_y \qquad (1)$$

Tensile breakout capacity of the masonry is given by MSJC Equation 3-1:

$$\phi B_{an} = 4\phi A_{pt}\sqrt{f'_m} \qquad (2)$$

For L Bolts, In addition to the two equations above, bar pullout is addressed by (MSJC Equation 3-6)

$$\phi B_{an} = \phi\left[1.5 f'_m e_b d_b + 300\pi(l_b + e + d_b)d_b\right] \qquad (3)$$

The variables listed in the above equations are defined in the MSJC Code (MSJC, 2000) and are not included here to save space.

Equation 1 (3-2) was used to calculate the nominal tension capacities of all the anchor bolts in Phase 3 and Phase 2 using average measured yield strengths and were plotted in Figure 5. Also shown in Figure 5 are the nominal capacities assuming that the A 307 bolts had an F_y = 413 MPa (60 Ksi) (since A 307 bolts have no minimum yield strength) and the A 36 L bolts had an F_y = 248 MPa (36 ksi).

Examination of bolt test results showed that the Phase 3 headed bolts are significantly stronger than those tested in Phase 2 (both headed and L bolts). This suggests Phase 3 prism test should exhibit higher strengths than Phase 2 results and Figure 5 confirms this result, but the increase in strength is much smaller than the 40% suggested by the bolt strength comparisons since the masonry capacity appeared to govern in Phase 3. Previous testing programs also evaluated the tension capacities of masonry anchors an the results of Phase 2 and 3 appear to be consistent with these earlier results for comparable configurations although only 16 mm and 19 mm anchors were tested. (Tubbs et-al, 2000) , (Weigle et-al, 2002)

Since masonry strength appeared to dominate the behaviour of most tests, Equation 2 (MSJC 3-1) was used to calculate the capacity of the masonry prisms based on average measured f'_m. This calculation results in 57 kN (12,840 lb) for the masonry tensile breakout capacity of the Phase 2 prisms and 56 kN (12,535 lb) masonry tensile breakout capacity for the Phase 3 prisms (108 mm embedment). This value was plotted as a single dashed horizontal line in Figure 5 and shows that masonry failure would start to govern in bolts above 16 mm in diameter (although the actual yield stress of A 307 bolts is will affect this shift in failure mode).

Figure 5 also shows that the capacity of the masonry defined by Code Equation 3-1 (masonry tensile break out) overestimates the tensile strength of the Phase 3 and under estimates the tensile strength of the Phase 2 specimens. The type of masonry cracking pattern at failure suggests that the lower strength grout may have had a significant effect on the strength of the Phase 3 specimens beyond the small 5% reduction in f'_m. There may be a flexural failure in the specimens occurring initially in head joint of the specimens and lower strength grout will have a greater impact on the flexural failure of the central grouted course. It should be noted that this “flexural failure” may be a result of the testing protocol and not a type of failure normally expected in the field. Further investigation of this issue is needed.

A possible modification to the masonry tensile break out equation might be the use of the grout strength instead of f’m. The results from Equation 2 (3-1) obtained from using the Phase 2 and 3 grout strengths are also plotted in Figure 5 with bolded dashed lines. These values appear to give a better prediction of strength but since there was little evidence of the typical cone breakout modelled by Equation 2 (3-1) observed in any of these tests, further investigation of this issue needs be conducted.

Similar results were obtained for the L bolts, although for the larger embedment lengths, bolt pullout or masonry breakout governed. Further, the code equations generally overestimated the tension capacities of the larger bolt diameters (22 mm and above) but these lower strengths may be a result of the testing configuration and should be investigated further.

A comparison of measured shear capacities for both Phase 2 and 3 are shown in Figure 6. and are generally lower that those found by Tubbs et-la (Tubb et-al, 2000) for similar configurations but higher f’m values.

In the 2002 MSJC Code, the nominal shear strength of anchors, $B_{vn,}$ far from a free edge is calculated by the lesser of the values given by Eq. 5 (MSJC Equation 3-8) and Eq. 6 (MSJC Equation 3-9):

$$\phi B_{vn} = \phi 4 \ A_{pv} \sqrt{f'_m} \tag{5}$$

$$\phi B_{vn} = \phi 0.6 \ A_b f_y \tag{6}$$

Using Eq. 6 above, and the average measured yield stress values in a similar manner as the tension calculation, nominal shear anchor capacities were calculated for each bolt configuration. Since all the specimens were loaded in shear with the bolt in the middle of specimens, the distance from the centre of the bolt to the free edge of the masonry in the direction of the load L_{be} was the same and equal to 300 mm (11.81 in). This results in the same masonry breakout capacity being calculated for all the specimens using Eq. 5. The B_{vn} given by Eq. 5 would be 215 kN (48400 lb) and the ϕB_{vn} would be 108 kN (24200 lb).

The nominal capacity calculations would suggest that the anchors (Headed and L bolts) and should all fail in shear rupture, even with the slightly lower f'_m for Phase 3. This did not happen, above the 13 mm (½ in) bolt diameter all the specimens loaded to failure in shear failed by cracking of the masonry prism at loads well below those predicted by the breakout shear formula. Even if the lowest grout strength of 1940 psi was used in Equation 5, the nominal shear break out capacity would still be well above any of the strengths measured. As was concluded in Phase 2, (McGinley, 2003) the MSJC code equations do not appear to be very accurate at the larger bolt diameters and there appears to be at least one other masonry controlled failure mode not being accounted for by the shear design equations.

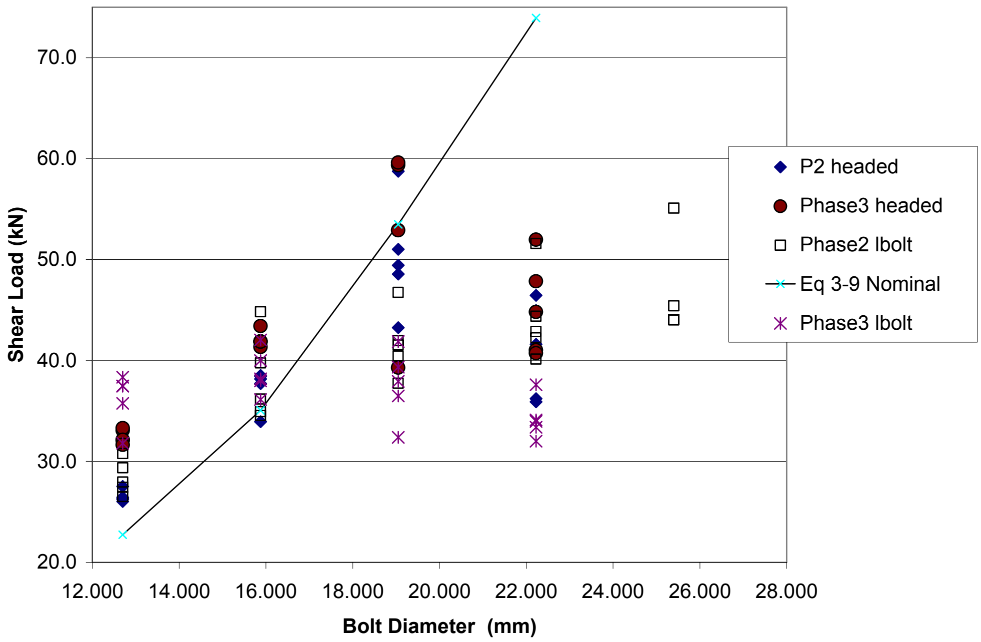

Figure 6 Measured shear strength of bolts in Phase 2 and Phase 3

4.1 Shear and Tension Force Interaction

As was stated in the introduction of this report, the interaction of shear and tension forces on anchors in masonry can be dealt with using an interaction equation in the form of:

$$\left(\frac{b_a}{B_a}\right)^N + \left(\frac{b_v}{B_v}\right)^N = 1 \tag{7}$$

The current MSJC code equation has this form but conservatively uses N =1. The value of N used in most steel structural design steel codes is 2 and recent data for anchors in masonry suggests that the value of N should be greater than 1 for masonry as well.

To determine what the most appropriate value for N should be for masonry, a least square regression was performed on the combined loading data for each test configuration. The base tension and shear capacity values, B_a and B_v, were taken as the average measured pure tension and shear values. These average values were then used to determine the ratio ba/Ba and bv/Bv. It should be noted that the pure shear tests generally showed bolt shear rupture at lower bolt diameters but the remainder of the tests exhibited a masonry failure. A least square fit was then obtained for these ratios and a regression coefficient, R^2 was calculated. Note that the L bolt base shear values, where not directly measured, were assumed to be the same as those for similar headed configurations. The results of the regression analysis (*N* and R^2 values) are summarized in Table 2 and it appears that, with a few exceptions, the N value was near 1.4 to 1.5.

Table 2 also shows the results of the regression analysis of the 22 mm (7/8 in) headed bolt specimens placed in the 194 mm (8 in) CMU prisms produced an N value near 2.5. This value was significantly above the N value obtained for the L bolts for the same configuration and above the N value obtained for the 22 mm (7/8 in) bolts in 295 mm (12 in) CMU prisms. The 2.5 value is also above the N = 2.0 used for steel bolt design. However, since almost all these test configurations exhibited masonry failure, we would assume that a value of N below 2.0 would result. Further examination of the data (McGinley, 2004) indicates that in the region of lower tension loading, a significant number of the tests showed measured shear load maximums well in excess of the average pure shear loading case. This suggests that the shear strength of the section may increase with increased tension loading. This behaviour is not accounted for in the interaction equation which cannot have bv/Bv greater than one and has thus skewed the regression analysis. This high N value is questionable and will require further investigation.

Table 2—Regression Analysis Results

Bolt Diameter (mm) Headed	***N***	***R^2***	**L Bolts**	***N***	***R^2***
13	1.48	0.94	13	1.71	0.89
16	1.46	0.7	16	1.5	0.93
19	1.26	0.79	19	1.16	0.95
22 -194CMU	2.53	0.68	22 -194 CMU	1.58	0.90
22-295 CMU	1.38	0.83			
25 -295 CMU	1.38	0.91			

The N values determined in this investigation are generally lower than the 1.67 value determined by Fabrello-Streufert and that listed for concrete anchor bolts (Fabrello-Streufert,2003). However, Phase 3 but encompassed a much larger range of anchor configurations than this previous work which likely reduced the N values. In addition, the shear failure of anchors in masonry is not well enough understood and is likely affecting the results. Finally, the concrete anchor bolts interact with reinforcing and this has not been addressed yet by testing in masonry. It is likely that steel reinforcing will increase the measured anchor capacities and decrease the variability of the masonry governed failures.

5 Conclusions

Based on the results of Phase 3 of this investigation the following conclusions can be made:

1. The lower strength grout used in Phase 3 did not reduce the prism compression strength of the masonry (f'_m) significantly, but did appear to significantly affect the measured tension and, to some extent, the shear anchor strengths.

2. The nominal tension anchor bolt capacity equations in the MSJC Code appear to give relatively good results at lower anchor diameters. However, the low grout strengths of Phase 3 suggest that use of the grout compression strength instead of f'_m for the prediction of the masonry tensile breakout may be a more accurate method to predict this failure mode. In addition, unaccounted for tension splitting/flexural failure modes appear to be present in larger anchor diameters. These failure modes may be a result of the testing protocol and this issue needs further study.

3. The shear load provisions in the MSJC appear to be adequate when predicting the capacity of lower diameter bolts, but can be very unconservative for shear load capacities for bolts above $^5/_8$ in. (17 mm) in diameter. The masonry break out equation does not appear to accurately predict the splitting/prying masonry failures observed with these large diameter anchors. The code shear capacity equations need to be developed further if they are to be used successfully for larger diameter anchor bolts.

4. The results of the regression analysis on the combined tension and shear interaction equation appears to suggest that an *N* between 1.4 to 1.5 may be the most appropriate power value to use in the interaction equation when the anchor behaviour is dominated by masonry failures.

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