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CAPACITY OF DRY-STACK MASONRY WALL WITH BUNDLED BARS

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ABSTRACT

Bundling bars is used in reinforced concrete when limited space or bar size is a concern. However, little to no research has been done on bundling of bars in masonry. In the United States, this has led to an inconsistency when using the Allowable Stress Design (ASD) method compared to the Strength Design (SD) method. Currently, the ASD method allows for bundled bars but the SD method does not. Research was done on reinforced dry-stack masonry to examine the capacity of bundled bars. A total of 6 walls were constructed and tested. Two unreinforced walls were built to determine the simple shear capacity of the walls while the other four walls were built with 2 #4 horizontal bars at 1.22 m on center and various amounts of vertical reinforcement. The average shear capacity of the unreinforced walls was 12.45 kN, that of the walls with 2 #4 vertical and 2 #4 horizontal bars at 1.22 m on center was 68.83 kN, and that of the walls with 2 #4 vertical bars at 0.61 m on center and 2 #4 horizontal bars at 1.22 m on center was 89.18 kN. The data from these tests were compared to expectant results using the TMS 402 design equation following the SD method to assess the validity of the method when bundled bars are used. Since a dry-stack system was utilized, the TMS shear equation only accounts for the capacity of the reinforcement, and since the bars were bundled only one reinforcement bar is considered to resist shear. When the contribution of the dry-stack system and grout are considered, the calculated capacity is between 42.17 kN to 71.09 kN depending on the number of bars considered and the number of grouted cells. The results show that the walls reinforced with bundled bars were able to resist 54 and 140% greater load than that predicted by the equation when ignoring the contribution of the masonry and grout. When considering the contribution of the grout, masonry, and bundled bars the capacity of the walls were 4-42% greater than the calculated capacity. The results are preliminary since the design equation include built in factors of safety that are necessary for design.

KEYWORDS: *bundled bars, dry-stack, masonry*

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INTRODUCTION

Bundling of bars has been a construction practice in reinforced concrete for several years. Bundling bars adds flexibility when detailing the placement of bars. It is advantageous as it can minimize congestion, can be used when larger bars are restricted, and can be easier to maneuver on the construction site. Significant amount of research has been done on bundled bars in concrete and some of the conclusion of those studies are that bundling bars is a safe detailing practice as long as each bar is individually well anchored [1], that the bars act as a unit and an effective perimeter of the bundle can be calculated to account for bond strength [2], and entire bundles can be lapped spliced [3].

Despite the amount of research involving bundled bars that has been done on reinforced concrete little to no research has been done on bundled bars in masonry [4]. Due to this lack of research a discrepancy in the current design code exists. Per chapter 6 of the code, up to 2 bars can be bundled together. This indicated that when the allowable stress design method (ASD) is used, bundled bars are permitted. However, chapter 9 of the code prohibits the use of bundled for design using the strength design method. It is believed that the reason for this discrepancy is that during compilation of masonry design standards in the 80's, the committees used requirements from the American Concrete Institute (ACI) 318 code that seemed suitable for masonry [5] and this was applied to the ASD method. However, when the strength design method was being developed, no literature existed on bundled bars in masonry and so such provision was excluded in the SD method.

DRY-STACK MASONRY

Dry-stack masonry provides a system that is uniquely fitting for evaluating the in-plane shear capacity of bundled bars. Dry-stack systems have been evaluated for their compressive capacity and has been shown that it can carry loads similar to traditional masonry [6] [7]. However, little research has been done on the shear capacity of dry-stack systems, and in general is considered to provide little shear resistance. Currently, the International Building Code (IBC) sets a limit to the shear capacity of dry-stack systems to 69 kPa (10 psi) over the gross area of the wall [8]. For the dry-stack system that was utilized in this research, only the reinforcement is considered to resist shear loads [9].

The dry-stack system used for this research has a unique set of concrete masonry blocks (CMUs) that uses expanded polystyrene (EPS) insulation inserts. A cementitous surface bonding compound is used to provide a link between the blocks while adding lateral strength. In general, the exterior faces of the block are similar to a traditional 20.3 cm x 20.3 cm x 40.6 cm (8 in. x 8 in. x 16 in.) concrete masonry block, but the interior configuration of the block has two rows of openings with offset webs as shown in Figure 1. These openings allow room for grout, reinforcement, insulation inserts, or electrical and plumbing ducts.

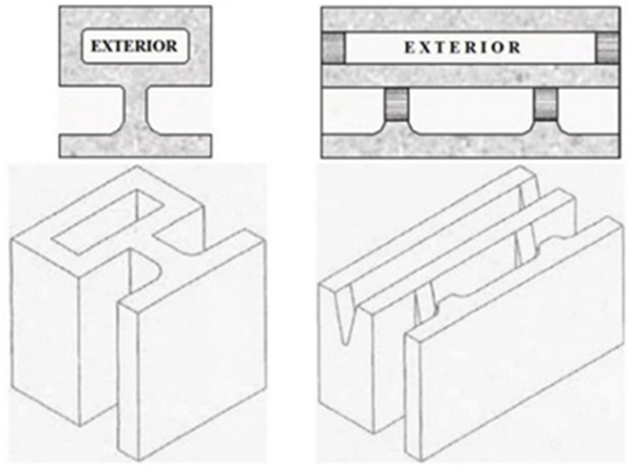


Figure 1: Dry-stack system blocks.

The main reason for the EPS inserts is to increase the R-value of the system; i.e., provide insulation for the building. As these inserts are slightly taller than the blocks, they help align the blocks in place and with the help of metal shims help prevent stress concentrations due to any irregularity on the block. There are two sizes of inserts: large and small. Large inserts fit in the exterior cell of the stretcher. Small inserts fit in the interior cells of the stretcher, and interior cavity formed when blocks are placed together. When needed, reinforcement and grout is placed in the interior cell of a stretcher or in the interior cell created between two stretcher blocks.

OBJECTIVE

The research evaluated the in-plane shear capacity of dry-stack masonry with bundled bars compared to current design standards in the United States. To accomplish this objective 6 dry-stack masonry walls were constructed and tested. A cementitious surface coating was applied to both sides of all walls. Though only the reinforcement is considered to resist the shear loads some resistance will be provided by the wall system. To account for the capacity of the wall system two unreinforced walls were built to determine their shear capacity. The average capacity of these walls was used to account for the wall resistance from the other tests. The other four walls were built with 2 #4 horizontal bars at 1.22 m (4 ft.) on center and varying amount of vertical reinforcement. The walls were tested by applying a lateral load at the top of the walls. The results of these tests were then compared to the predicted results of the TMS 402 shear equation for reinforcement. Two comparisons were made, first while ignoring the contribution of the wall system and second accounting for the capacity of the wall system and grout.

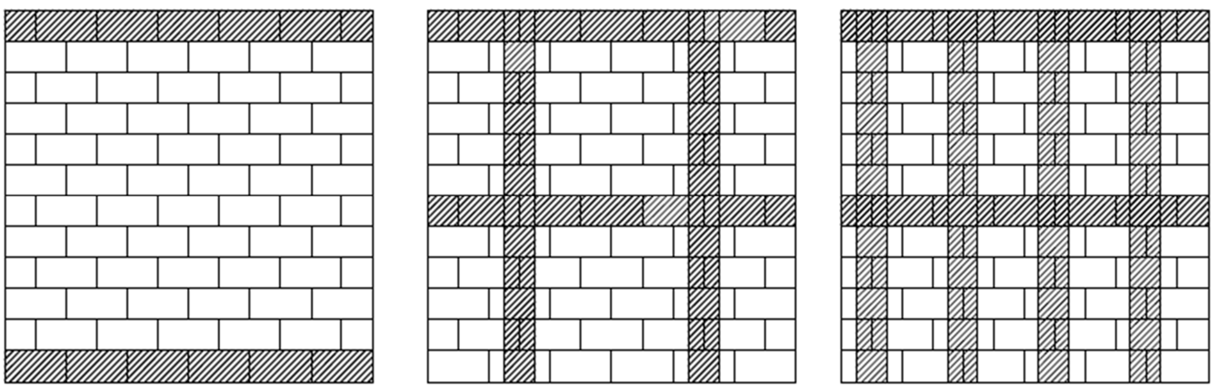
SPECIMENS

A total of 6 walls were built and tested to determine their in-plane shear capacity. Each wall was built to be 2.44 m. (8 ft.) by 2.44 m (8 ft.). The walls were constructed on top of a reinforced concrete foundation. Two walls were built without reinforcement but were grouted on the top and bottom courses to provide a way to attach to the wall footing and testing frame, two walls were built with both vertical and horizontal reinforcement and grout spaced at 1.22 m. (4 ft.) on center, and two walls were built with horizontal reinforcement and grout spaced at 1.22 m. (4 ft.) on center and flexural reinforcement and grout at 0.61 m. (2 ft.) on center. Reinforcement was 2 #4 grade 40 bars in all locations. Table 1 summarizes the wall layouts, and Figure 2 shows a visual representation of the wall layouts.

Table 1: Wall Layout Summary

Wall	Grout Pattern	Vertical Reinforcement	Horizontal Reinforcement
#1	Ungrouted	N.A.	N.A.
#2	Ungrouted	N.A.	N.A.
#3	1.22m x 1.22 m	2#4 A_s=258 mm^2 @ 1.22 m o.c.	2#4 A_s=258 mm^2 @ 1.22 m o.c.
#4	1.22 m x 1.22 m	2#4 A_s=258 mm^2 @ 1.22 m o.c.	2#4 A_s=258 mm^2 @ 1.22 m o.c.
#5	0.61 m x 1.22 m	2#4 A_s=258 mm^2 @ 0.61 m o.c.	2#4 A_s=258 mm^2 @ 1.22 m o.c.
#6	0.61 m x 1.22 m	2#4 A_s=258 mm^2 @ 0.61 m o.c.	2#4 A_s=258 mm^2 @ 1.22 m o.c.

*(1 meter=3.28 ft., 1 mm^2=0.00155 $in.^2$)



(a) Ungrouted **(b) 1.22 m x 1.22 m** **(c) 0.61 m x 1.22 m**

Figure 2: Wall Configurations

In addition to the testing on the walls, testing was accomplished on the various components of the wall system. These include surface bond, grout, blocks, masonry prisms, and reinforcement. At this time, only preliminary results are available and it is assumed that the average values found form these component test apply to all the walls.

Footing

Every wall was built upon a reinforced concrete footing that was 3 m (10 ft.) long, 0.46 m (1.5 ft.) wide, and 0.30 m (1 ft.) high. Each footing was reinforced with 2 #6 bars and 1 #3 bar in the top and bottom of the footing. Stirrups made of #4 bars at 20 cm (8 in.) on center were used as shear reinforcement. Each footing was attached to the laboratory strong floor using DWIDAG bars. The bars were placed through PVC tubes that had been placed vertically and fixed inside the wood formwork before concrete pouring. Figure 3 shows the design of the footing.

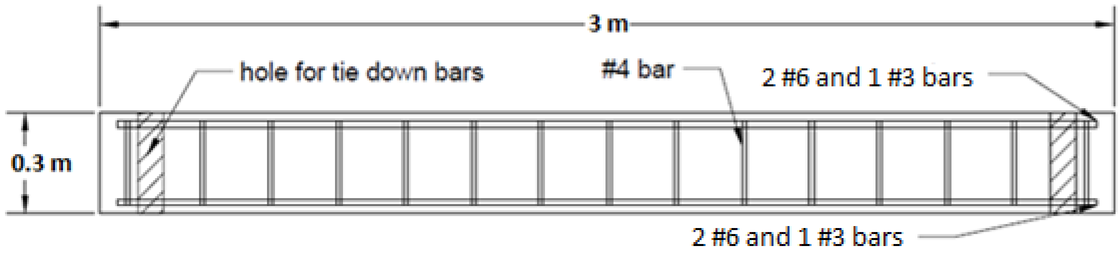


Figure 3: Footing Design

Wall Construction

Wall construction was accomplished by laying the blocks out on the concrete footing, grouting, and applying the surface coating. After each course was laid, EPS inserts were placed in cells before the next course was laid. For the cells that were to be grouted, no inserts were placed. After every 6 courses reinforcement was placed in their respective cells, and the vertical grout cells as well as a horizontal bond beam was poured.

After the blocks were laid and grouted, the walls were wetted using a hose for 10 minutes. The surface coating was then applied in 2 layers. The first layer was applied using a hawk and trowel and then spread using a Darby. Once this layer began to set up a second layer was applied with a hawk and trowel and troweled until the surface coating was smooth. The walls were then allowed to cure for 28 days before testing.

TESTING FRAME

A steel reaction frame was assembled on the strong floor of the structural laboratory. DYWIDAG bars, which connected the steel frame to the strong floor, were post-tensioned to the strong floor in order to minimize frame movement. A steel cap was attached to the frame by 2 steel tubes that were then attached to a 445 kN (100 kip) capacity actuator. The steel cap was attached to the top of the wall with a total of 24 masonry bolts, 12 on each side. Figure 4 shows the test setup.

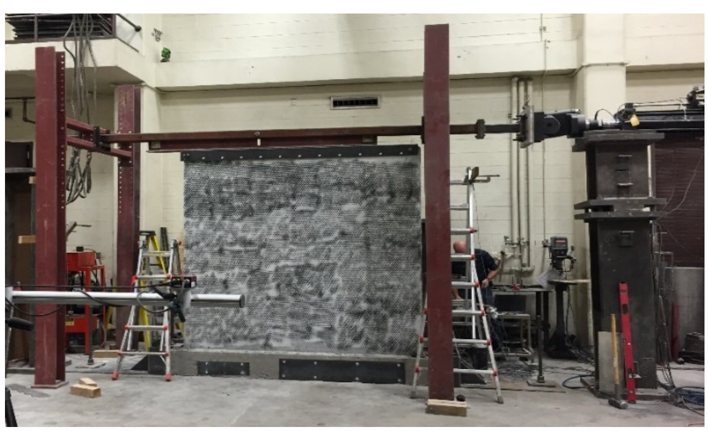


Figure 4: Test Frame Setup

INSTRUMENTATION AND DATA COLLECTION

Two types of data were collected during the testing: applied load and displacement. The applied load was obtained from the actuator. The horizontal deflection was measured using string pots that were mounted on a frame independent from the testing frame and walls. Figure 5 shows the layout of the string pots, where the rectangular section shows where the instrument was placed and the line is the string pot line. String pot 1 was used to measure the displacement at the top of the wall, while string pot 2 and 3 were used to measure the displacement of the bottom of the wall and the footing, respectively. String pot 4 measured the vertical uplift of the footing and string pot 5 measured the uplift of the wall. String pot 6 and 7 measured the diagonal deformation of the walls.

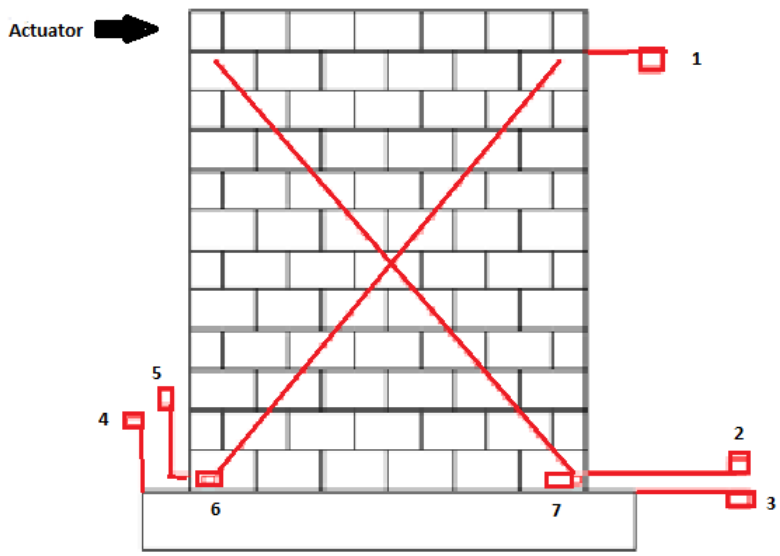


Figure 5: Instrumentation Placement

For the results that are presented, the displacement is the difference in displacement from string pot 1 and 2. The uplift that is mentioned is the difference in uplift between the wall and the footing or string pot 5 and 4.

RESULTS

Figure 6 shows the load-displacement for walls 1 and 2 and Figure 7 shows the load-displacement of walls 3-6. The maximum load and displacement of the tests are summarized in Table 2. The maximum displacement is considered where the wall had a drop of 20% of its maximum capacity.

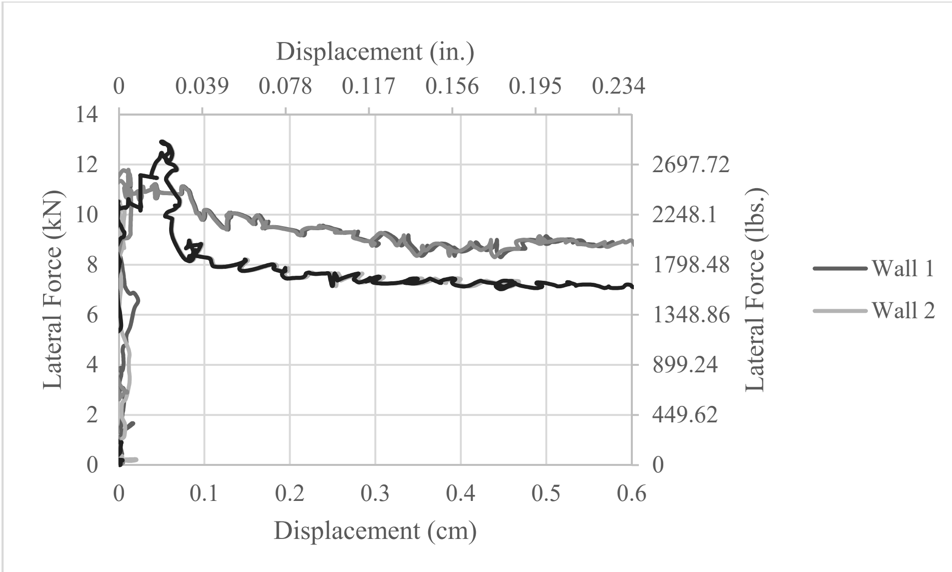


Figure 6: Load-Displacement Curve for Walls 1 and 2

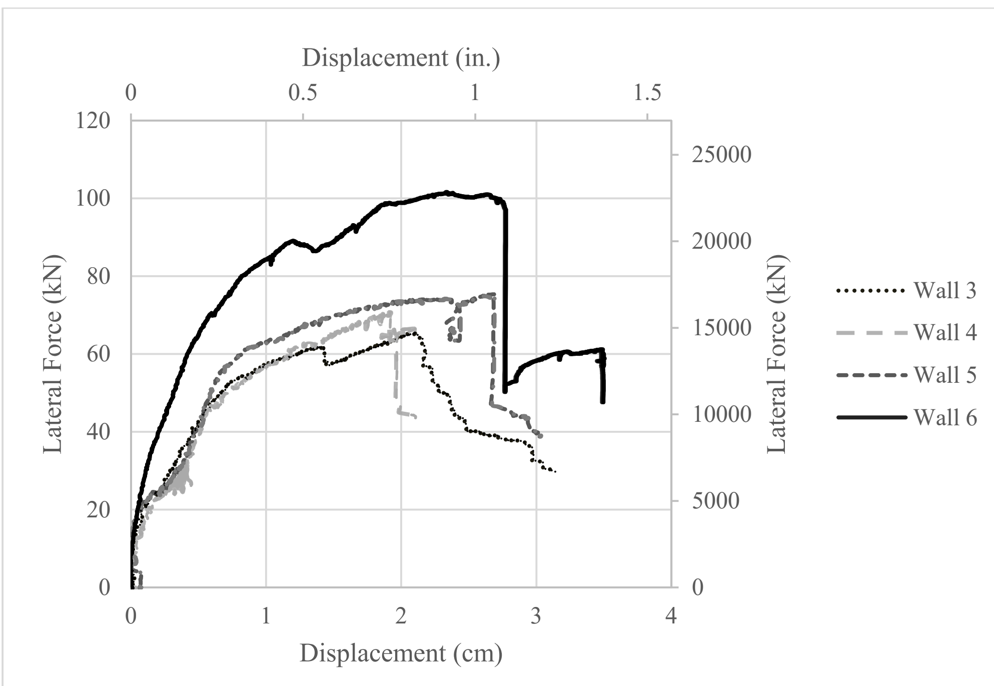


Figure 7: Load-Displacement Curve for Walls 3-6.

Table 2: Wall Results Summary

Wall	Configuration	Max Load (kN)	Max Displacement (cm)
1	ungrouted	11.88	0.12
2	ungrouted	13.01	0.07
3	1.22m x 1.22m	66.02	2.23
4	1.22m x 1.22m	71.63	1.96
5	0.61m x 1.22m	75.95	2.68
6	0.61m x 1.22m	102.41	2.76

*(1 m=3.28 ft, 1 kN=224.8 lbs., 1 cm=0.39 in.)

No axial load was added to any of the walls, though this resulted in some rocking because axial load has been shown to increase the shear capacity of walls [10]. For future testing it is recommended that axial loads be applied to minimize rocking. As expected, as the vertical reinforcement increased so did the maximum displacement and the ductility of the wall.

COMPARISON WITH DESIGN CODE

The values from experimental results were compared to the calculated shear strength using equations 1 through 3 for walls 3-6 [4]. In these equations M_u is the factored moment acting on the wall, V_u is the factored shear acting on the wall, d_v is the depth in the shear direction, A_{nv} is the net area in the shear direction, f'_m is the compressive strength of the masonry blocks, P_u is the factored axial load acting on the wall, A_v is the area of steel in in.^{2}, s is the spacing between shear reinforcement in in., f_y is the yield strength of the reinforcement in psi, d_v is the depth of the wall in the shear direction in in., V_n is the nominal shear strength, V_{nm} is the nominal shear strength from the masonry, V_{ns} is the nominal shear strength from the reinforcement, and γ_g is a factor to account for partially grouted walls. Equation 1 shows the nominal shear strength for masonry and equation 2 shows the nominal shear strength of the steel, and equation 3 shows the overall nominal shear strength of a wall.

$$V_{nm} = \left[4.0 - 1.75\left(\frac{M_u}{V_u d_v}\right)\right] A_{nv}\sqrt{f'_m} + 0.25P_u \tag{1}$$

$$V_{ns} = 0.5 * \left(\frac{A_v}{s}\right) * f_y * d_v \tag{2}$$

$$V_n = (V_{nm} + V_{ns}) * \gamma_g \tag{3}$$

Currently, when using these equations bars are not allowed to be bundled [4], and for this dry-stack system only the reinforcement is considered to resist shear [9]. Due to these requirements, an initial comparison is made assuming V_{nm}=0 and A_v=129 mm^2 (0.2 in.^{2}) to account for only one #4 bar. In addition, another comparison is made assuming V_{nm}=0 and A_v=258 mm^2 (0.4 in.^{2})

assuming that both #4 bars resist shear. In these comparisons s=122 cm (48 in.), f_y=276 MPa (40,000 psi), d_v= 244 cm (96 in.), and γ_g=0.75. Results of these comparisons are shown in Figure 8. For all the walls, the capacity of the experimental walls exceeded that of the calculated capacity by 209-380% when assuming only one #4 bar. When assuming 2#4 bars the capacity of the experimental walls exceeded that of the calculated capacity by 54-140%.

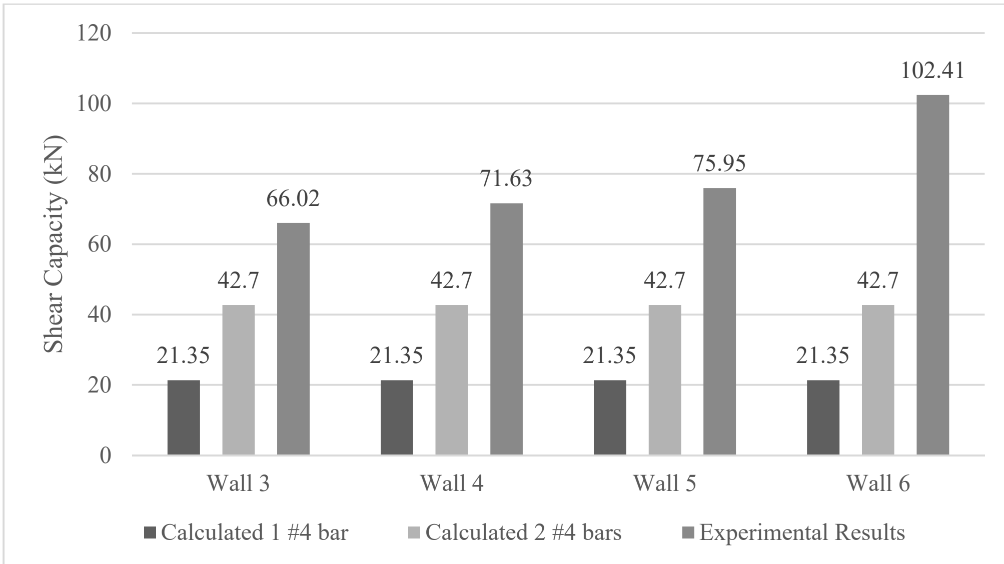


Figure 8: Comparison of Experimental Results to Calculated Capacity Assuming No Contribution by the Dry-Stack Wall System.

Walls 1 and 2 showed that the dry-stack system did have some shear capacity. In addition to the wall system, grout was used in several cells in walls 3-6 and should also be accounted for. Therefore, another comparison similar to the previous comparison was made accounting for the contribution of the wall system and the grout. To account for the contribution of the wall system the average capacity of walls 1 and 2, 12.45 kN (2.8 kips), was added to the calculated capacity. To account for the grout, equation 1 was used where $M_u/(V_u d_v)$=1, A_{nv}=116 cm^2 (18 in.^{2}) multiplied by the # of grouted vertical cells, f'_m=10.34 MPa (1500 psi), which was the average compressive strength of the grout, and P_u=0. Results of these comparisons can be seen in Figure 9. These results show that when only one #4 bar is considered the experimental capacity was 70-103% higher than the calculated capacity. If both of the bundled bars are considered, then the experimental capacity was 4-42% higher than the calculated capacity. In all cases though, the measured results were higher than the calculated capacity using the Strength Design method.

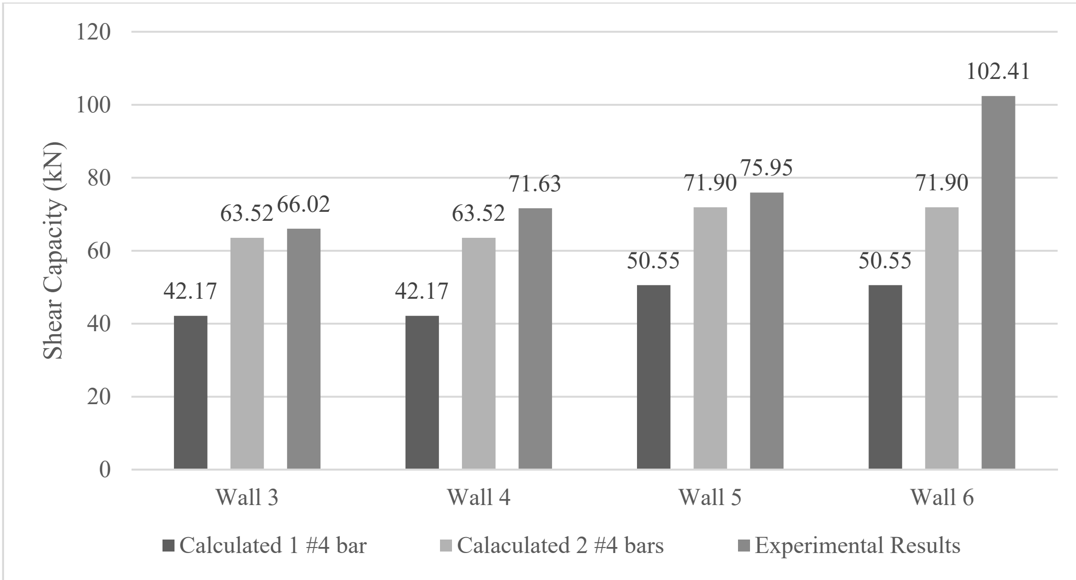


Figure 9: Comparison of Experimental Results to Predicted Capacity from Design Code

CONCLUSIONS

Testing was conducted on 6 dry-stack surface bonded walls to quantify the maximum capacity of the shear reinforcement of bundled bars. These capacities were then compared to the strength design method from the MSJC code in the United States, as currently there is a lack of research in the capacity of bundled bars. The research showed that when ignoring the contribution of the dry-stack masonry system, as is currently done, the capacity of the overall system was 209-380% greater than the calculated capacity when only one reinforcement bar is considered. While ignoring the contribution of the masonry system and both reinforcement bars are considered, the capacity of the overall system was 54-140% greater than the calculated capacity. When considering the contributions of the grout, masonry system, and only one reinforcement bar, the experimental capacity was 70-103% higher than the calculated capacity. When considering the contributions of the grout, masonry system, and both reinforcing bars, the capacity of the overall system were 4-42% higher than the calculated capacity.

Results from this research were compared to calculated values using design equations. Though results showed that the experimental data was conservative compared to that of design equations using bundled bars, they should be used only preliminary since design equations inherently have built in factors of safety. More research is needed before really concluding that bundled bars can be utilized within the SD method. Future testing should be done on various amounts of bundled bars, spacing of reinforcement, aspect ratios, size of reinforcement, various gravity loads, and comparing test results of bundled bars to an equivalent area of a single reinforcing bar.

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