AXIAL CAPACITY OF DRY-STACKED ENDURA MASONRY WALLS

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Testing was conducted on ten dry-stacked surface-bonded masonry walls, ENDURA walls, at Brigham Young University to determine their axial capacity. ENDURA walls are dry-stacked concrete masonry units with eccentrically placed reinforcement and a surface bonding cement applied to both faces of the walls. The surface coat provides a physical bond between the blocks, and is also applied for aesthetic purposes. Polystyrene insulation inserts are placed in all ungrouted hollow block cells. ENDURA system relies on shims and the surface bonding coat to ensure that the wall is level and plumb. The walls tested were 3.00 meters high by 2.44 meters wide. Each of the walls was built using a different configuration of block type, reinforcement spacing, and number and spacing of grouted cavities. Also, some walls were constructed with a thin mortar layer on the bed joints. A steel frame with two hydraulic jacks was used to load the walls. All walls were tested to failure by applying a uniformly distributed axial load at the top of the walls. The research is in its infancy but preliminary results indicate that the number of grouted cells influences the capacity of the walls more than the vertical spacing of the reinforcement. Preliminary results are mixed as far as the use of a thin mortar layer on the bed joints is concerned: some walls experienced an increase in strength while others experienced a decrease in strength; these results simply indicate that further research is needed.

Keywords: Masonry Walls, Dry-Stack Masonry Construction, Axial Strength

DRY-STACK MASONRY SYSTEMS

Generally speaking dry stack systems can be divided into two groups: traditional and interlocking. Traditional systems use customary masonry blocks while interlocking systems use interlocking masonry blocks. Customary masonry blocks are hollow while interlocking blocks can be solid but most often are also hollow. The traditional system usually depends on surface bonding to provide stability and lateral strength; surface bonding can also be used in the interlocking system. Both systems can be grouted, ungrouted, reinforced, or unreinforced if hollow blocks are used, and the decision to use any one of these variables is based on load demand.

Ramamurthy and Kunhanandan (2004) provide an excellent review of dry stacked systems including block details and structural performance studies. Since that review, there have been several technical articles reporting different research projects including but not limited to the work by Ferozkhan et al (2004) describing the behavior of dry stack masonry under axial compression; Lourenco and Ramos (2004) explaining the cyclic behavior of dry masonry joints; Lourenco et al (2005) characterizing the response of dry joint stone masonry walls subjected to in-plane combined loading; Uzoegbo et al (2007) reporting the load capacity of dry stack masonry walls developed in South Africa; and Thanoon et al (2004) and Nasly and
Yassin (2009) describing innovative interlocking block building systems developed in Malaysia.

In North America there have been several systems developed and patented. Non-technical literature on these systems abounds (VanderWerf 1999, Farnsworth 2004, Palmer 2001, and Sauve 2007) but, because of the proprietary nature of many of them, technical literature is scarce. The Azar system is discussed by Drysdale and Gazzola (1991) while the sparlock system has been studied by Hatzinikolas et al (1986) and Drysdale and Guo (1995). The block introduced by Whelan (1985) is further discussed by Harris et al (1992), who also studied the modified H block or WHD block.

Fundamentally all these systems attempt to reduce labor cost, material usage, variability of construction, dependence on skill labor, and dependence on good weather during construction while improving performance (Beall 2000). Dry stack systems, however, are not without disadvantages. One of the chief issues is that without the mortar between block courses there is no easy way to deal with irregularities of the individual blocks. Also, there is no economical method of producing concrete masonry blocks with little or no variation in height (Anand and Ramamurthy 2000, VanderWerf 1999).

THE ENDURA BLOCK SYSTEM
The ENDURA block system has a set of blocks and a set of expanded polystyrene (EPS) insulation inserts. A surface bonding compound provides the physical bond between the block while adding lateral strength. The exterior shape of the individual block is similar to a conventional 200 mm x 200 mm x 400 mm block, but the interior configuration of the block is significantly different. The two rows of openings allow room for grout, reinforcement, insulation inserts, or electrical and plumbing ducts. Figure 1 shows the five different blocks.

In a typical ENDURA wall, the stretcher is the most commonly used block. The purpose of the stretcher is to span between corners. The right and left corner blocks are used at corners depending on which direction the wall is turning. The half stretcher block is used in situations which may arise on a job. The half square blocks are typically used where the wall ends without continuing around a corner.
The EPS inserts provide insulation for the building. In addition, because they are slightly taller than the blocks, they help, along with metal shims, alleviate the problems created by block surface irregularities. There are two sizes of inserts: long and short. Long inserts fit in the rectangular exterior cavity of the stretcher. Short inserts fit in all other rectangular cavities: mid-interior cavity of the stretcher, exterior cavity of a corner block, and interior cavity formed when blocks butt together. The short EPS insert can also be trimmed to fit the exterior cavity of the half stretcher. The mid-interior rectangular cavity in a corner block can be either grouted together with the square cavity of that block or be filled with half of a short EPS insert. Reinforcement is typically placed in the square cavity of a corner block or in the cavity of a half square block. When needed, reinforcement can also be placed in the mid-interior cavity of a stretcher or in the interior cavity formed when blocks butt together. Reinforcement placed in interior cavities is eccentric in the wall.

OBJECTIVES
The fundamental goal of this research was to determine the ultimate axial capacity of ENDURA walls and determine the difference, if any, in capacity of walls typically built and those built with a thin layer of mortar on all bed joints. The tested walls were 3.00 meters (15 courses) high by 2.44 meters (6 blocks) wide. All walls were tested to failure by applying a uniformly distributed axial load at the top of the walls.

TEST SPECIMENS
Ten walls, shown in Figure 2, were tested. The variables were block type, number of grouted cavities, reinforcement spacing, and the use of a thin layer of mortar on the bed joints. All walls, except wall #10, had a horizontal reinforced grouted bond beam at the sixth, twelfth, and fifteenth block courses. Wall #10 had a horizontal reinforced grouted bond beam at the seventh and fifteenth block courses. The steel reinforcement used had a 12.5 mm diameter.

Wall #1 was built using corner, half square, and half stretcher blocks. All cells were grouted and vertical reinforcement was placed at approximately 0.41 m. on center in the square cells. Wall #2 was built exactly as wall #1 except that exterior rectangular cavities were filled with EPS inserts. Wall #3 was built with the same reinforcement as that of wall #1 and #2 but the only grouted cells were the square cell with reinforcement; all other cavities were filled with EPS inserts. Wall #4 was built exactly as wall #3 except that a thin layer of mortar was placed
on all bed joints. The layout of walls #1, #2, #3, and #4, including placement of vertical and longitudinal reinforcement, is shown in figure 3.

Wall #5 was built using alternating corner and stretcher (full and half) blocks. Vertical reinforcement was placed at approximately 0.61 m. on center in grouted square cells of corner blocks. All interior cavities were also grouted while all exterior cavities were filled with EPS inserts. Wall #6 was built with the same block layout as that of wall #5. Vertical reinforcement was also placed at approximately 0.61 m. on center in the grouted square cells of corner blocks. All other interior and exterior cavities were filled with EPS inserts. Wall #7 was built exactly as wall #6 except that a thin layer of mortar was placed on all bed joints. The layout of walls #5, #6, and #7, including placement of vertical and longitudinal reinforcement, is shown in figure 4.

Wall #8 was built using only stretcher (full and half) blocks and had a thin layer of mortar placed on all bed joints. Vertical reinforcement was placed at approximately 0.61 m. on center in a grouted interior rectangular cell. All exterior and other interior cavities were filled with EPS inserts. When reinforcement is placed in an interior cavity, as it was in this case, the reinforcement is placed approximately 40 mm from the center of the wall. Wall #9 was built exactly as wall #8 except that no thin layer of mortar was used. Wall #10 was built using the same block layout as those of walls #8 and #9. Vertical reinforcement, however, was placed at 1.22 m on center in a grouted interior rectangular cavity. All other exterior and interior cavities were filled with EPS inserts. Similar to walls #8 and #9, the reinforcement was not centered in the wall. The layout of walls #8, #9, and #10, including placement of vertical and longitudinal reinforcement, is shown in figure 5.

Table 1 provides the construction parameters for the walls. Column 1 gives the specimen number, column 2 specifies the main type of block used, column 3 specifies the location
(within the block cell) of the vertical reinforcement, column 4 specifies the size of reinforcement, column 5 specifies the vertical reinforcement spacing, column 6 specifies the spacing of the horizontal reinforcement, column 7 specifies the use of thin mortar layer, and columns 8, 9, and 10 specifies which cells are grouted.

![Figure 5: Walls #8, #9, and #10 Layout](image)

**Table 1: Construction Parameters for the Walls**

<table>
<thead>
<tr>
<th>Wall #</th>
<th>Block Type</th>
<th>Vertical Reinforcement Location</th>
<th>Reinforcement size</th>
<th>Vertical Reinforcement Spacing</th>
<th>Horizontal Reinforcement Spacing</th>
<th>Thin Set</th>
<th>Square Cells Grouted</th>
<th>Exterior Rectangular Cells Grouted</th>
<th>Interior Rectangular Cells Grouted</th>
</tr>
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<tbody>
<tr>
<td>1</td>
<td>Corner</td>
<td>Center</td>
<td>#4</td>
<td>0.41 m</td>
<td>1.00 m</td>
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<td>Yes</td>
<td>Yes</td>
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<td>Center</td>
<td>#4</td>
<td>0.41 m</td>
<td>1.00 m</td>
<td>—</td>
<td>-</td>
<td>Yes</td>
<td>—</td>
</tr>
<tr>
<td>3</td>
<td>Corner</td>
<td>Center</td>
<td>#4</td>
<td>0.41 m</td>
<td>1.00 m</td>
<td>—</td>
<td>—</td>
<td>—</td>
<td>—</td>
</tr>
<tr>
<td>4</td>
<td>Corner</td>
<td>Center</td>
<td>#4</td>
<td>0.41 m</td>
<td>1.00 m</td>
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<td>Yes</td>
<td>—</td>
<td>—</td>
</tr>
<tr>
<td>5</td>
<td>Corner &amp; Strecher</td>
<td>Center</td>
<td>#4</td>
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<td>1.00 m</td>
<td>—</td>
<td>Yes</td>
<td>—</td>
<td>Yes</td>
</tr>
<tr>
<td>6</td>
<td>Corner &amp; Strecher</td>
<td>Center</td>
<td>#4</td>
<td>0.61 m</td>
<td>1.00 m</td>
<td>—</td>
<td>-</td>
<td>—</td>
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<td>—</td>
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<tr>
<td>8</td>
<td>Strecher</td>
<td>Offset</td>
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<td>1.00 m</td>
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<td>—</td>
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</tr>
<tr>
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<td>Strecher</td>
<td>Offset</td>
<td>#4</td>
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<td>—</td>
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</tr>
<tr>
<td>10</td>
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<td>1.52 m</td>
<td>—</td>
<td>—</td>
<td>Only Those Reinforced</td>
<td>—</td>
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</tbody>
</table>

**TESTING FRAME AND DATA COLLECTION**

A steel reaction frame was assembled on the strong floor of the Brigham Young University 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. The reaction frame and wall #1 are shown in Figure 6.

The frame had a capacity of approximately 8900 kN. Two hydraulic jacks, each with a capacity of 4450 kN were attached to the horizontal beam of the frame. A W310x117.0 steel beam was attached to the bottom of the hydraulic jacks. The beam was reinforced with web stiffeners and side plates essentially making it a “box” beam. The modification was done to prevent web buckling, localized flange failure, and distribute the load as uniformly as possible along the length of the walls. A neoprene pad was placed between the steel beam and the top of the wall to accommodate minor irregularities on the top of the wall.
Three types of data were collected during the testing: applied load, vertical consolidation or deflection, and out-of-plane deflection at mid-height. The applied load was obtained from loads cells that were mounted between the hydraulic jacks and the steel spreader beam. The total load was computed as the sum of the two load cells readings. The vertical deflection was measured using string pots. Three string pots were mounted on a frame which was independent from the testing steel frame and walls. The strings were then attached to the top of the steel spreader beam. The vertical displacement was computed as the average of the three string pots readings. The out-of-plane deflection at mid-height of a wall was measured using three Linear Variable Displacement Transformers (LVDTs). The LVDTs were also mounted on a frame which was independent from the testing steel frame and wall. The out-of-plane deflection was computed as the average of the readings from the three LVDTs.

RESULTS AND GENERAL OBSERVATIONS

The manufacturers of the ENDURA block were responsible to conduct the block, mortar, grout, and prism tests but the results of these tests are not yet available. The effects of the block, mortar, grout, and surface bonding on the capacity and behavior of the walls are important elements to the overall research, and without those results, predictions of wall capacity cannot be made. Once the results of the components tests are available researchers plan to disseminate that information through an article discussing the effects of these components on the capacity and behavior of the system. The results presented herein are only of the wall tests.

Figure 7a shows the vertical displacements for walls #1 through #4 and Figure 7b shows the out-of-plane displacements for the same walls; similar plots were obtained for all walls.
The typical initial low stiffness observed in Figure 7a is due to the deformation experienced by the neoprene pad which was placed between the steel spreader beam and the walls. No spalling of the surface bonding compound at the top of the walls due to the lateral expansion of the neoprene pad was observed.

Out-of-place displacements of approximately 15 mm were measured at the end of the tests. The magnitude of the measured displacement may be deemed large since the walls were loaded only axially. Out-of-plane displacements were measured only on one of the faces of the walls and had measurements been made on both faces of the walls they would have allowed researchers to better understand the behavior of the walls. The bottom of the wall was braced against translation but the top of the walls was not (see Figure 6); therefore, it is plausible that the hydraulic jacks rotated slightly causing the entire wall to move out-of-plane; the rotation of the jacks would have been caused by a small rotation of the top reaction frame beam. The jacks were connected eccentrically to the top beam causing a torsional moment on that beam. In retrospective, out-of-plane displacements of both faces as well as of the top of the walls should have been measured to assist researchers determine if the walls were bowing (a buckling issue), if the surface coating was delaminating, or if crushing (and consequent lateral expansion) was occurring. Unfortunately, these additional displacements were not measured.

The measured maximum load (ultimate capacity) and applied stress ratio for each wall are shown in Figure 8. Stresses were normalized using the stress on wall #1 and were calculated using the following net areas for the walls: 0.49 m$^2$ for wall #1; 0.44 m$^2$ for walls #2 and 5; 0.38 m$^2$ for walls #3 and #4; 0.34 m$^2$ for walls #6 and #7; 0.31 m$^2$ for walls #8 and #9; and 0.29 m$^2$ for wall #10.

Stresses comparisons presented are based on the simple but realistic assumption that all blocks used have the same strength, all grout used have the same strength, and all mortar used have the same strength. Such an assumption is realistic because all blocks came from the same batch and grout and mortar were made from pre-mixed bags. Furthermore, two workers, who were experienced, mixed all grout and mortar and constructed all walls as prescribed by the manufacturers. It is possible that, for example, two batches of grout may have different compressive strengths; researchers, however, expect that if differences exist they will be very small or even negligible.

![Figure 8: Wall Response](image-url)
Wall #1: Wall #1 was solid grouted and heavily reinforced. It was, therefore, expected that this wall would resist the greatest axial load; however this was not the case. At failure, wall #1 experienced crushing of the top two to three courses of blocks while the rest of the wall was still intact (see Figure 9). This wall had a high ratio of grout strength to overall wall strength. As has been suggested by previous research (Uzoegbo 2007) such high ratio is often accompanied by crushing at the top of the wall. The out-of-plane response of the wall is somewhat interesting—the wall started going in one direction and then changed direction. It is possible that the surface coating delaminated and caused the apparent out-of-plane (negative) deflection at the early loading stage. A visual inspection during testing did not reveal any delamination, but since the measured displacements were very small at that early loading stage, it is possible that such delamination was missed by the research team.

Figure 9: Crushing at the Top Courses of Wall #1

Wall #2: The overall in-plane response of wall #2 was similar to that of wall #1, except that it resisted slightly more load and experienced higher stress level even though it was slightly less stiff. Normalized stresses indicate that wall #2 had approximately 15% more capacity than wall #1. The failure of the wall was crushing of the top two to three courses of blocks accompanied by significant delamination of the surface coating along the bottom course. Unlike wall #1, however, crushing of the top blocks was not along the entire length of the wall but happened from the middle toward the left side (facing the wall as shown in Figure 6) of the wall. The wall appears to be failing at the same load as that of wall #1 (Figure 7a) but an unexplained small jump in load resistance is observed. The “extra” capacity may be due to the “intact” right side of the wall, which was able to carry a little more load before the entire wall crushed. After testing, the coating at the bottom of the wall was removed revealing that the bottom course had experienced extensive cracking and crushing, which was not observed in wall #1. The out-of-plane response of wall #2 is similar to that of wall #1 except for “the anomaly” of changing deflection direction that wall #1 experienced.

Wall #3: As wall #3 was loaded, vertical cracks developed at approximately 0.41 m on center along ungrouted blocks (between reinforced cavities, which were also at 0.41 m on center). When failure occurred, the top right corner of the wall failed first (when facing the wall as shown in Figure 6). One possible explanation is that the hydraulic jack may have loaded the wall unevenly, applying slightly more load on the right side. The setup, however, was checked and rechecked and no issues were discovered. No other wall had similar problems. Normalized stresses indicate that wall #3 had 20% less capacity than wall #1.

Wall #4: Wall #4 differed from wall #3 in that a thin mortar layer was used on the bed joints. The wall resisted slightly more load, experienced higher stress level, and experienced slightly
more deflection than wall #3. The additional deflection may be a result of the more compressible mortar. An autopsy of the wall after the test showed that some reinforcing bars buckled. The behavior of the wall was therefore controlled by crushing of the blocks and by buckling of the reinforcement. Normalized stresses indicate that wall #4 had 11% less capacity than wall #1 but approximately 10% more capacity than wall #3, its counterpart wall. This result suggests that the use of a thin layer of mortar on the bed joints increases the capacity of the walls.

Wall #5: Wall #5 resisted approximately 84% of load resisted by wall #1; this may be explained by the fact that this wall along with walls #1 and #2 had both square cells and interior rectangular cells grouted. The failure of the wall was similar to that of walls #1 and #2: crushing of the top two to three block courses. These three walls (#1, #2, and #5) were the “top performers” as far as load carrying capacity is concerned, suggesting that the amount of the grout in the walls has a more significant effect on the axial load capacity of the walls than the spacing of reinforcement. Normalized stresses indicate that wall #5 had only 4% less capacity than wall #1.

Wall #6: Wall #6 experienced fairly uniform crushing across the length and height of the wall. The left top side of the wall appeared to fail first but the right top side was very close to failure as well. At the top half of the wall the blocks split and the face of the block separated and fell forward as shown in Figure 10. Normalized stresses indicate that the capacity of wall #6 was similar to those of walls #1 and #5.

Wall #7: Wall #7 performed quite similarly to wall #6, experiencing fairly uniform crushing throughout the wall and an ultimate load capacity similar to that of wall #6. Such response was expected since the only difference between them was that wall #7 had a thin mortar layer along the bed joints. The load capacity of the wall was slightly less than that of wall #6. Normalized stresses indicate that wall #7 had only 8% less capacity than wall #1 and approximately 6% less capacity than wall #6, its counterpart wall. This result appears to negate the previous observation that the use of a thin layer of mortar on the bed joints increases the capacity of the walls.

Wall #8: Wall #8 was built with the reinforcement in the interior cavity of the stretcher blocks. The reinforcement was placed eccentrically in the wall and most likely because of that wall #8 buckled. As shown in figure 11a, cracks developed horizontally directly above and below the bond beam in the center of the wall, which suggests that the wall was experiencing significant out-of-plane bending. At failure, the ungrouted face of the wall separated completely from the grouted face as shown in Figure 11b. Normalized stresses indicate that
the capacity of wall #8 was slightly more than half of that of wall #1.

**Wall #9:** Wall #9 performed similarly to wall #8 as was expected since the only construction difference between the two walls was that wall #9 did not have the thin mortar layer along the bed joints. At failure the ungrouted face of the block separated and fell forward in a similar manner as that of wall #8. Vertical cracks, separating the grouted and ungrouted faces of the blocks developed just prior to failure as shown in Figure 12. The load capacity of the wall was slightly less than that of wall #8. Normalized stresses indicate that the capacity of wall #9 was slightly less than half of that of wall #1 and approximately 14% less than that of wall #8, its counterpart wall. Similar to the results from wall #4, this result suggests that the use of a thin layer of mortar on the bed joints increases the capacity of the walls.

![Cracks Separating Grouted and Ungrouted Faces of the Blocks](image)

**Wall #10:** Wall #10 was the lightest reinforced wall. The only cells grouted were the cells which contained reinforcement. As expected, the wall had the smallest load capacity. The wall was eccentrically reinforced and as the load was applied, the wall experienced significant out-of-plane bending. Wall #10 behaved in a similar manner as walls #8 and #9. These three walls had significantly lower axial capacities than the others suggesting that the eccentricity of grouted cells has a large effect on the axial performance of the walls. Normalized stresses indicate that the capacity of wall #10 was slightly more than half of that of wall #1.

**CONCLUSIONS**

Load and stress results are preliminary and can be used for comparative but not for deterministic conclusions. The comparisons presented herein assume that walls had constituents with the same material properties, a fact that is unknown at this time. The comparison between the load carrying capacity of walls #1, #2, and #3, which had the same block configuration but different grouted area, shows some inconsistency. The fully grouted
wall #1 was expected to have higher carrying load capacity than wall #2, but it didn’t. Researchers are puzzled by this result and are examining the data to determine the reason(s) for such an erratic result. Wall #3, as expected, had lower load carrying capacity than walls #1 and #2. The comparison between the load carrying capacity of walls #5 and #6, which had the same block configuration but again different grouted area, yielded reasonable results—wall #5, as expected, had higher load carrying capacity than wall #6. The conclusion from these simple comparisons is that there are other variables besides grouted area that influences the load carrying capacity of the walls.

A comparison between the load carrying capacity of walls with and without a thin mortar layer along the bed joints also shows some inconsistency. Wall #7, which was similar to wall #6 except for the thin mortar layer, resisted slightly less load than wall #6. Wall #4 and wall #8, which were similar to wall #3 and wall #9, respectively, except for the thin mortar layer, however, had higher load carrying capacity than their counterpart walls. Researchers expected that the full bearing contact between blocks due to the thin mortar layer would indeed increase the load carrying capacity of the walls. Expectations were not met and not all the walls with the thin mortar layer along the bed joints had a higher load capacity than their counterpart wall. Researchers are examining the data to determine the reason for the inconsistent results. It is plausible that the eccentrically placed reinforcement in walls #7 and #9 played a greater role in the capacity of the wall than it was expected.

A single specimen for each wall configuration is not sufficient to allow researchers to make any significant conclusion. The results are encouraging but it is clearly apparent that more research is required.

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