PARTIALLY-GROUTED MASONRY SHEAR WALL PERFORMANCE WITH A BASALT-REINFORCED REPAIR OVERLAY

C. Johnson¹ and A. E. Schultz²

ABSTRACT

Partially-grouted reinforced masonry is a common form of construction in institutional, government, and commercial buildings throughout the eastern, southern, and central United States. It is important for this lateral-load resisting system to remain intact during a seismic event, as many of these buildings require quick accessibility after an earthquake for community resilience. Previous experimental work has shown partially-grouted masonry shear walls (PGMSW) to underperform under lateral loading relative to code expectations for fully-grouted masonry shear walls. The non-conservative nature of the design of these walls presents a need for retrofit and repair methods should a large seismic event occur where these buildings exist. Numerous experimental studies have focused on fiber-reinforced polymer (FRP) retrofit of unreinforced masonry structures through the use of carbon or fiberglass bars, sheets, strips and meshes. While these experiments show the overall benefit of FRP retrofits for unreinforced masonry walls, issues with delamination and under-utilization of the FRP materials has been reported. Additionally, there exists a gap in the literature for the retrofit of reinforced and partially-grouted masonry using FRP materials other than the commonly used carbon and glass. To address these issues, as well as to better understand the behavior of repaired PGMSWs, an experimental repair investigation was completed at the University of Minnesota. The study focused on a PGMSW sub-assemblage (shear wall with cross-walls and window opening) that was heavily damaged, intentionally, during initial testing. The sub-assemblage was subsequently repaired using a cementitious overlay reinforced with basalt miniature reinforcing bars (i.e. Minibars). The design, construction, and data from cyclic testing of these specimens are used to evaluate the feasibility of the basalt-reinforced cementitious overlay as a repair method for PGMSW.

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Partially-grouted reinforced masonry is a common form of construction in institutional, government, and commercial buildings throughout the eastern, southern, and central United States. It is important for this lateral-load resisting system to remain intact during a seismic event, as many of these buildings require quick accessibility after an earthquake for community resilience. Previous experimental work has shown partially-grouted masonry shear walls (PGMSW) to underperform under lateral loading relative to code expectations for fully-grouted masonry shear walls. The non-conservative nature of the design of these walls presents a need for retrofit and repair methods should a large seismic event occur where these buildings exist. Numerous experimental studies have focused on fiber-reinforced polymer (FRP) retrofit of unreinforced masonry structures through the use of carbon or fiberglass bars, sheets, strips and meshes. While these experiments show the overall benefit of FRP retrofits for unreinforced masonry walls, issues with delamination and under-utilization of the FRP materials has been reported. Additionally, there exists a gap in the literature for the retrofit of reinforced and partially-grouted masonry using FRP materials other than the commonly used carbon and glass. To address these issues, as well as to better understand the behavior of repaired PGMSWs, an experimental repair investigation was completed at the University of Minnesota. The study focused on a PGMSW sub-assemblage (shear wall with cross-walls and window opening) that was heavily damaged, intentionally, during initial testing. The sub-assemblage was subsequently repaired using a cementitious overlay reinforced with basalt miniature reinforcing bars (i.e. Minibars). The design, construction, and data from cyclic testing of these specimens are used to evaluate the feasibility of the basalt-reinforced cementitious overlay as a repair method for PGMSW.

Introduction
Partially-grouted masonry is the preferred construction method for masonry construction in the Midwestern and Eastern United States due to its decreased costs from ease of construction and lower use of materials. While partially grouted masonry shear walls (PGMSW) are commonly used in construction as a primary lateral load resisting system, their behavior is not as well understood as their fully grouted counterparts due to the complex interactions between the grouted cells surrounding hollow panels. Experimental studies that have been completed on PGMSWs have shown the walls to underperform when compared to the design equations given in TMS 402, the design standard for design of masonry structures in the USA [1,2]. It is

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important to note that the TMS 402 design equations for PGMSW’s were empirically developed from data on a collection of experiments mostly on fully grouted shear walls, and to differentiate between fully and partially grouted walls the net instead of gross masonry area is used for partially grouted walls [3]. Recently, a grouting coefficient, equal to 1.00 for fully grouted walls and 0.75 for partially grouted walls, was adopted by TMS 402 to address the concern with the capacity of PGMSW [4]. Because the literature shows that many existing PGMSW are likely to have less shear capacity than designed for, evaluation of repair options for these walls becomes important.

Research on repair and retrofit methods for masonry is limited to mostly unreinforced masonry and utilizes fiber-reinforced polymer bars, sheets, meshes, and strips to complete the repairs/retrofits. There particularly have been many studies completed using FRP sheets of carbon or fiberglass to retrofit unreinforced masonry. Many of these studies have shown issues with delamination of the FRP sheets, therefore under-utilizing the retrofit material [5, 6]. The use of a cementitious matrix reinforced with an FRP mesh has also been evaluated as a retrofit method; however a mechanical bond to the wall through the use of anchors was necessary, which could be time consuming in practical construction [7]. To address the drawbacks of the methods described above, as well as understand the feasibility and effectiveness of repairing a heavily damaged reinforced masonry wall rather than retrofitting an unreinforced wall, an experimental repair investigation was completed at the University of Minnesota using a basalt reinforced cementitious overlay (BRCO) to repair a heavily damaged PGMSW sub-assemblage. The primary objective of the study was to introduce and develop a new repair technique, evaluate the efficiency of the repair, understand how the repair affects the behavior of PGMSWs, and finally provide any design recommendations or conclusions about this repair method based on the test results and construction experience.

**Specimen Configurations**

The repairs were completed on a damaged wall will be referred to as specimen (DR), which was a C-shaped wall built with 8” concrete masonry units (CMU). The wall was 21 ¼’ long by 14’ tall with an 8’ by 3 ½’ window opening. Cross-walls with 4’ lengths were located on either end of the shear wall. The elements of wall on either side of the window opening are referred to as piers. The portions of the wall above and below the window opening are referred to as spandrels. Specimen DR was built on a 2’ deep reinforced concrete foundation block. And a 14” roof system was installed at the top of the wall, consisting of an 8” hollow core plank with a 6” reinforced concrete cast-in-place topping. Shear reinforcement was provided by four bond beams placed at the top and bottom of the wall, as well as above and below the window opening, and by ladder-style bed joint reinforcement placed in placed at every course in the piers and every other course in the spandrels. Double-reinforced vertical cells were placed at both ends of the wall, on either side of the window opening, and at the end of each cross wall. Double-reinforced vertical cells are constructed by grouting two adjacent vertical cells and placing a reinforcing bar in each cell. Figure 1 illustrates the layout of this wall.
Damage was extensive to Specimen DR during initial testing, with cracks up to 1” wide and loss of face-shells in several areas. An example of this damage is also in Figure 1. Specimen DR subsequently went through two phases of repair and testing. The first will be referred to as specimen (DR-R1). In specimen DR-R1 a 1.5” thick layer of the BRCO was placed on the outside face of the shear wall up to a height of 12 1/3’. During testing of specimen DR, little damage occurred in the top three courses of masonry; therefore this top portion of the wall was not repaired. During testing of specimen DR-R1, a majority of the damage in the wall occurred in these three unrepaired masonry courses at the top of the wall, with little damage in the portion of the wall with the BRCO repair. Therefore, in order to fully utilize the BRCO an additional 2’ of BRCO was added above the previous repair, covering both the damaged masonry in the top three courses and a portion of the roof system. This 4” overlap with the roof system was provided to avoid a plane of weakness at the masonry-roof interface at the top of the wall and provide a load path from the roof to the rest of the masonry wall and the entire BRCO. This second step in the repair is referred to as specimen (DR-R2). The layout of DR-R1 and DR-R2 can be seen in Figure 2.

**Repair Design and Process**

The BRCO consisted of a 1.5” thick layer of cementitious matrix reinforced with basalt Minibars overlaid on the exterior face of the damaged PGMSW. The cementitious matrix for the repair consisted of a calcium sulfo-aluminate (CSA) cement. CSA cement was chosen due to its high-strength, low-shrink, and fast-setting properties, as well as its high bond strength to existing concrete, which was crucial to avoid the time intensive use of mechanical anchors into the
existing masonry. Con-Korite X’tra was an easily available commercial CSA product, and was therefore used. The basalt Minibar mixed into the cementitious matrix were 1.7” long basalt fiber reinforced polymer Minibars manufactured by ReforceTech. These fibers were chosen due to their high flexural toughness and strong bond capacity to concrete.

A thickness of 1.5” was chosen for the overlay in order to provide enough new material to provide an alternate stress path for the extensively damaged face shells in the masonry, as well as allow reasonable placement of the basalt Minibars. Once the thickness was determined, products provided by ReforceTech [8] were used to estimate the required dosage of minibars for the overlay mix. The amount of fibers required for the wall was determined using the volume fraction given in Eq 1.

\[ V_f \geq \frac{1}{4} \left( \frac{f_t}{0.8} - 1 \right) \]  

(1)

Where \( V_f \) is the volume fraction, and \( f_t \) is the principal tensile stress in the overlay. There are two limiting cases on the locations with maximum tensile stresses. One is at mid-length of the overlay where the idealized flexural stresses vanish and the section is in pure shear. The other is at the tension edge of the shear wall where the bending tensile strength is maximized. For both cases of maximum tensile stress, it was assumed that the overlay would resist 50% of the peak load from the initial testing of specimen DR, and that the stresses are uniformly distributed across the width. This was done in order to bring the sub-assemblage back to full strength. In the case of specimen DR-R1 a bending stress of 2.48 MPa was targeted as the limiting case for \( f_t \) which required a volume fraction of 0.525% minibars in the overlay. The total volume of required BFRP Minibars was determined from the volume fraction \((V_f)\) using the overlay thickness \((t_o)\) of 1.5”, and the wall height \((H)\) and length \((L)\) of 14’ and 21 \(\frac{1}{3}\)’ respectively. The volume fraction was also used to calculate the required weight of 0.33 lbs. of minibars per 55 lb. bag of CSA cement, which yielded 0.5 cubic feet of mix.

The CSA cement was mixed in using a fin concrete mixer in small batches (3 to 5 bags at a time) due to its quick set time. The manufacturer recommended the use of KB25 Acrylic Resin as an admixture to increase bond strength and improve flexural strength of the cement. One-third of the total liquid mixed with the Con-Korite Xtra was the KB25 Acrylic Resin, and the rest was water. A packet of set control per bag, provided by the manufacturer, was used in order to extend the working time from 15 to 35 min. The order of mixing was important based on instructions from both the manufacturers of the Con-Korite Xtra and the Minibars. The liquid was added to the mixer first and then the set control was poured into the liquid. The CSA mortar mix was then added and mixed for 2.5 minutes. Finally the Minibars were added to the fully mixed CSA cement, passing through a 2” sieve to ensure even distribution, and then mixed for an additional minute.

Preparation of the damaged shear wall was crucial to ensure good bond between the masonry and BRCO. First, to remove any loose particles and dust from the wall, the entire wall was brushed with stiff wire brushes and then cleaned with compressed air. Any of the face shells that were completely damaged during the first repair were removed. If they were at a hollow portion of the wall, plywood was placed to block the cement from pouring into the hollow cells. If they were at grouted portions, the face shell was simply removed and the cement for the overlay was allowed to replace the missing face shell. After surface preparation, groundwork for pouring began. The BRCO was placed in 2’ lifts. This height was chosen to ensure adequate
compaction of each layer, since the mix contained Minibars that were 1.7” long, which is close to the width of the overlay and due to the quick set time of 35 min of the CSA cement. Once the formwork panels were in place and secured, a 1:1 mixture of KB25 Acrylic Resin and water was carefully sprayed on the wall to help increase the bond between the CSA cement and the masonry. A wet mix of the CSA cement and minibars was poured into the formwork from the top of the form along the length of the wall. Once all of the mix was in place, the forms were vibrated by placing a pencil vibrator against the exterior of the forms to ensure compaction of the overlay. This process could not take longer than 35 minutes to prevent any of the CSA cement from setting without being compacted. Once one lift was complete, the next layer of formwork was placed and the new lift was poured. This was repeated until the desired height was met.

![Figure 3. Photographs of formwork and minibars used for DR-R1 and DR-R2 repairs.](image)

**Test Set-Up**

The repaired PGMSW assemblage underwent cyclic, quasi-static displacement histories. Each cycle of loading underwent a target displacement twice in order to capture strength and stiffness degradation. These target displacements were determined by drift levels defined by the performance limits defined by the Applied Technology Council [9]. The test was run until the total load on the system at any given cycle peak dropped to 50% of the maximum load previously measured for the system. This was done in order to provide understanding of the repair up through significant strength deterioration. Two actuators were used to apply the lateral loads preventing torsion due to the asymmetrical layout of the assemblage. The two actuators were attached to loading beams that were anchored into and through the roof of the sub-assemblage. The actuators were supported by a steel reaction frame that was anchored to the strong-floor below. One actuator was operated in stroke control, and the stroke in the second actuator was slaved to the first one. Axial load was applied to the wall by means of the weight of a reinforced concrete beam, in order to simulate dead loads in a full building. The loading protocol and test set-up is shown Figure 4.

LVDTs were used to measure any relative sliding between the foundation and the wall, and between the wall and the roof system. Actuators contained internal LVDTs and load cells to measure their own displacements and loads. String pots were placed at the top of the wall on both sides to measure wall drift. Another was placed on the loading beam of the frame to subtract frame deflections. Horizontal string pots were placed at the top and bottom of the outside of both piers to measure pier drift. Two string-pots were used to measure the vertical displacement from the top of the piers to the foundation, and two string-pots were placed diagonally on each pier to measure shear deformation in the pier.
Figure 4. Test Set-Up, Instrumentation Layout, and Loading Protocol.

Observations

Global Behavior

The load-displacement curves for all 3 specimens, is shown in Figure 5, as well as the cyclic envelope. Table 1 shows the peak shear strength from each test, as well as the yield drift, peak drift, ultimate drift, as well as ductility. The yield drift was determined from the cyclic envelope. A secant was placed passing through the origin and 70% of the peak load, and extended to intersect a horizontal line at the peak load. The yield displacement was the displacement value at which these two lines intersect, and the yield drift was the ratio of the yield displacement to the height of the wall. The ultimate drift is the ratio of the displacement on the cyclic envelope once the wall has lost 20% of its capacity (80% of peak load) to the wall’s height. Ductility is defined as the ratio of the ultimate drift to the yield drift [10].

<table>
<thead>
<tr>
<th>Specimen</th>
<th>Peak Load (kips)</th>
<th>Peak Drift (%)</th>
<th>Yield Drift (%)</th>
<th>Ultimate Drift (%)</th>
<th>Ductility</th>
</tr>
</thead>
<tbody>
<tr>
<td>DR</td>
<td>67.4</td>
<td>0.24%</td>
<td>0.07%</td>
<td>0.60%</td>
<td>8.08</td>
</tr>
<tr>
<td>DR-R1</td>
<td>67.8</td>
<td>0.30%</td>
<td>0.12%</td>
<td>0.46%</td>
<td>3.90</td>
</tr>
<tr>
<td>DR-R2</td>
<td>82.5</td>
<td>0.56%</td>
<td>0.12%</td>
<td>0.76%</td>
<td>6.40</td>
</tr>
</tbody>
</table>

From Figure 5, we can see that specimens DR and DR-R1 exhibited similar peak shear strengths, while specimen DR-R2 was significantly stronger. The peak capacity of DR-R1 was nearly identical to that of DR, however DR-R1 required a larger drift than DR, as can be observed in Table 1. This shows that repair brought the wall back to its original strength, however not its original stiffness. Once DR-R1 reached its peak shear capacity, it lost its load carrying capacity more quickly than specimen DR, as shown in Figure 5. This is also demonstrated by the lower ultimate drift of DR-R1 as well as the lower ductility. This quicker deterioration is due to the significant amount of damage that began to occur in the top two unrepaired courses of masonry at the top of the PGMSW. Once peak load was met, sliding began to occur between the masonry courses, and faceshells were lost right below where grouted vertical cells met the top course (which was a bond beam). Because cracking was not excessive in the BRCO at this point, it was decided that the masonry at the top of the wall was controlling the behavior of the specimen rather than the overlay, so the test was stopped, and the decision was made to repair the remaining exposed masonry courses.
Specimen DR-R2 appeared to show greater efficiency of the BRCO than DR-R1. The peak capacity of DR-R2 was 22.4% larger than that of the unrepaired DR. This peak capacity was met at a drift over twice as large than in specimen DR. The damage from the previous test again decreased the stiffness of the wall, with specimen DR-R2 being less stiff than both DR and DR-R1. Thus, while specimen DR-R2 did have a larger peak drift capacity than DR, it is had a lower ductility due to a larger yield drift.

**Location of Damage**

Figure 6 shows the crack patterns for specimens DR, DR-R1, and DR-R2. While the cracks were distributed throughout the wall in specimen DR, the largest cracks appeared in the spandrel below the window opening, starting at the corners of the window opening and moving towards the toe of the piers. During testing of specimen DR-R1, horizontal flexural cracking first occurred in the overlay on either end of the wall at the same height of the window opening. This was observed in the cycles before peak load was met. Then diagonal shear cracks, initiating at the bottom of the window opening and moving towards the toe of the wall occurred. It is important to note these cracks corresponded very closely to the largest cracks in the masonry below the BRCO in the test of DR. Finally during the cycle where peak load was met, large vertical cracks formed from the top of the window to the unrepaired masonry. After this, all of the cracking occurred in the unrepaired masonry courses. None of the cracks from the test of specimen DR opened up much larger than 1/8 of an inch during the test of specimen DR-R1. In Specimen DR-R2, the dominant cracks in the system were the inclined cracks running from the bottom of the window opening to the toes, again where the most damage occurred during testing of specimen DR, as well as the mix of inclined cracks and horizontal flexural cracks initiating and opening up in the piers. During the first cycle, before peak load was met, the bottom diagonal shear cracks, which were already present from the testing of DR-R1, opened further. The inclined cracks above the
window opening appeared at peak load, and are likely the reason for the initial loss of capacity. During the unloading cycles existing cracks continued to open, and the horizontal flexural cracks in the piers from testing of specimen DR-R1 branched and extended towards the inclined cracks.

Figure 7. Gross shear strain in south pier, and distribution of displacement in north pier.

Very limited delamination of the overlay from the wall was observed during testing. The only locations of delamination were two small areas at the ends of the walls. At these locations, the ends of the forms were not secured as well as in other areas, and the overlay extended slightly past the ends of the walls. Slight delamination occurred in these two places, but over a length of only one inch, and the BFRP Minibars within cracks prevented collapse. In fact, during demolition of the sub-assemblage, it could be seen that most cracks lying parallel to the overlay occurred through the masonry blocks rather than the block-overlay interface. This strong bond capacity is essential to fully utilize the repair and is a significant benefit over other FRP systems.

Figure 8. Comparison of location of displacements on south side of wall in specimen DR and DR-R2.

Deformation of the piers changed throughout each test. In specimen DR shear strain was developed in both piers regardless of loading direction, however more shear strain was present in the north pier where more damage and cracking occurred. This began to change in specimen DR-R1 where the shear strain was much larger when the loading placed the pier in tension rather than compression. When the pier was in tension, shear strain was about the same in DR-R1 as it was in DR. During testing of specimen DR-R2, the trend that began in DR-R1 where the pier had much larger gross shear strains when the pier was in tension continued, but became much more pronounced. The shear strain was also much larger during DR-R2 with a lot of residual deformation. This is all illustrated in Figure 7 for the south pier. The location of the majority of
deformation also changed during testing. In Figure 7 we also compare the relative pier displacement, which is the horizontal displacement at the top of the pier minus the horizontal displacement at the bottom of the pier, for all three specimens. The green line represents all of the movement occurring in the pier. The trend being nearly vertical in DR-R1 implies little movement in the piers, which was clearly observed. The slope of the relative pier displacement of specimen DR is less steep than DR-1, implying more damage occurred in the piers, but not as much as specimen DR-R2 where the displacement in the piers begins to approach the total displacement of the wall.

The trend for increased damage in the piers is further illustrated by Figure 8, where we see a comparison of displacements in various elements of the wall for specimens DR and DR-R2. For specimen DR we can see that most of the damage occurred in the bottom spandrel because the magnitude of the displacement in the bottom spandrel is similar to the total wall displacement, while that in the pier is much less. This matched physical observations during testing. However for specimen DR-R2 this is not the case. The location of the damage depends on the direction of loading. When a pier was on the side of the wall in tension, most of the displacement occurred in the pier, due to the observed mixture of flexural and shear cracks. However, when the pier was in compression, the majority of the displacement is occurring in the bottom spandrel, where significant toe crushing is occurring. This is indicative that specimen DR-R2 experienced more flexural damage, where specimen DR experienced a more shear-dominated failure.

Conclusions

The tests described above demonstrate potential for BRCO as a repair method for heavily damaged, partially grouted masonry shear walls. The design recommendations provided by ReforceTech resulted in a workable mix with adequately dispersed fibers that significantly increased the strength of the damaged specimen DR. The strength increase due to the addition of the BRCO overlay, however was 34% larger than the designed value. Results from specimen DR-R1 indicate the importance of placing the BRCO on the entire wall rather than just the damaged portions of the wall. If the repair is not placed on the entire wall, damage could become localized in portions of the wall that do not receive the overlay, whether or not this region is previously damaged. Such damage localization prevents full utilization of overlay capacity.

The following conclusions can be made about the construction process. For ideal use of the fibers and placement of the overlay, it is recommended to use a grid to disperse the fibers in the mix and ensure even distribution throughout the matrix. Using a bonding agent and ensuring a clean, debris-free masonry surface helps ensure strong bond between the overlay and the wall. In addition, allowing the BRCO to enter cracks and replace missing face-shells of damaged blocks allows for a mechanical interlock between the existing wall and the overlay, increasing its effectiveness. For the CSA cement used in this test, set-time was the most difficult aspect of placing the overlay: if too much time was spent between mixing and placing of the overlay, the finish of the overlay was less uniform, and the risk of gaps in the repair overlay is possible.

The following conclusions can be made about the behavior of the repair. Specimen DR-
R2, showed both strength gain, as well as an increase in ultimate drift capacity. There was a decrease, however, in the ductility of the repaired walls versus the unrepaired wall, however this decrease in ductility was due to the decreased initial stiffness of the repaired walls versus the unrepaired wall, rather than the deformation capacity. It is expected that the ductility would be better in retrofitted walls that are not previously damaged. No delamination of the BRCO was present during the test, allowing for the full utilization of the repair. Based on these conclusions, application of a BRCO is recommended as a repair method of damaged partially-grouted masonry shear walls, and further investigation into its retrofit potential is recommended.

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