Lateral Resistance of Grouped Piles Near 20-ft Tall MSE Abutment Wall with Strip Reinforcements

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Lateral Resistance of Grouped Piles Near 20-ft Tall MSE Abutment Wall with Strip Reinforcements

Zachary Farnsworth

A thesis submitted to the faculty of Brigham Young University in partial fulfillment of the requirements for the degree of Master of Science

Kyle M. Rollins, Chair
Fernando S. Fonseca
Gus Williams

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Brigham Young University

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ABSTRACT

Lateral Resistance of Grouped Piles Near 20-ft Tall MSE Abutment Wall with Strip Reinforcements

Zachary Farnsworth
Department of Civil and Environmental Engineering, BYU
Master of Science

A team from Brigham Young University and I performed full-scale lateral load tests on individual and grouped 12.75x0.375 inch pipe piles spaced at varying distances behind an MSE wall. The individually loaded pile which acted as a control was spaced at 4.0 pile diameters from the wall face, and the three grouped piles which were loaded in unison were spaced at 3.0, 2.8, and 1.8 pile diameters from the wall face and transversely spaced at 4.7 pile diameters center-to-center. The purpose of these tests was to determine the extent of group effects on lateral pile resistance, induced loads in soil reinforcements, and MSE wall panel deflections compared to those previously observed in individually laterally loaded piles behind MSE walls.

The computer model LPILE was used in my analysis of the measured test data. The p-multipliers back-calculated with LPILE for the grouped piles were 0.25, 0.60, and 0.25 for the grouped piles spaced at 3.0, 2.8, and 1.8 pile diameters from the wall, respectively. These values are lower than that predicted for piles at the same pile-to-wall spacings using the most recent equation for computing p-multipliers. I propose the use of an additional p-multiplier for grouped piles near an MSE wall, a group-effect p-multiplier, to account for discrepancies between individual and grouped pile behaviors. The group effect p-multipliers were 0.35, 0.91, and 0.74 for the grouped piles spaced at 3.0, 2.8, and 1.8 pile diameters from the wall, respectively. The average group-effect p-multiplier was 0.66.

Additionally, I used LPILE to analyze test data from Pierson et al. (2009), who had previously performed full-scale lateral load tests of individual and grouped shafts. In said analysis, the group of three 3-foot diameter concrete shafts spaced at 2.0 shaft diameters from the wall face and transversely spaced at 5.0 shaft diameters center-to-center had an average group effect p-multiplier of 0.78.

As in previous studies, the induced forces in soil reinforcements in this study were greatest either near the locations of the test piles or at the MSE wall face. The most recent equation for calculating the maximum induced force in a soil reinforcement strip was reasonably effective in predicting the measured maximum loads when superimposed between the test piles, with 65% and 85% of the data points falling within the one and two standard deviation boundaries, respectively, of the original data used to develop the equation. Deflection of the MSE wall panels was greater during the grouped pile test than was previously observed for individually loaded piles under similar pile head deflections—with a maximum wall deflection of 0.31 inch compared to the previous average of 0.10 inch for pile head deflections of about 1.25 inches.

Keywords: laterally loaded piles, MSE wall, grouped piles, p-multiplier, soil reinforcement
ACKNOWLEDGEMENTS

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I would also like to thank my research professor, Dr. Kyle Rollins; other members on my committee, Dr. Fernando Fonseca and Dr. Gus Williams; various colleagues, including but not limited to Addison Wilson and Estephania Flores; my many friends in the civil engineering structures lab at Brigham Young University; and my family—all of whom made this research project and thesis possible. I additionally gratefully recognize the diligent effort of previous researchers who have paved the way in the study of lateral resistance of deep foundations, for which I add some small part in this thesis.
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1 INTRODUCTION

Mechanically stabilized earth (MSE) walls are often used in conjunction with deep foundations in overpass designs to maximize the usable space below the bridge. Figure 1-1 shows one such case of a group of steel piles close to the face of an MSE wall. Several studies have previously been performed to better understand the complex soil-structure interactions between a single pile loaded laterally towards an MSE wall from various distances behind the wall face (Pierson et al., 2009; Price, 2012; Nelson, 2013; Hatch, 2014; Han, 2014; Besendorfer, 2015; Budd, 2016; Luna, 2016; Rollins et al., 2016). However, as Figure 1-1 depicts, deep foundations used in a bridge abutment system are typically loaded as a group rather than singly. Grouped piles in normal soil conditions beyond the presence of a wall have been shown to have reduced lateral soil resistance compared to singly loaded piles (Rollins, 2006). However, this reduction has normally been quite small for the first row of piles, which is typical of bridge abutments. Nevertheless, the lateral resistance of piles near an MSE wall are strongly dependent on the resistance provided by the reinforcement strips. If more than one pile is relying on the resistance provided by a single reinforcement, the resistance of the grouped piles could be reduced.

Currently only one full-scale test has been performed on grouped piles near an MSE wall (Pierson et al., 2009). Pile head load-deflection curves from these tests show some reduction in lateral resistance relative to a comparison single pile, but Pierson et al. (2009) provide no
qualifications of the reduction or procedure to account for the effect. To further explore the behavior of grouped piles near MSE walls, this thesis will present and analyze data from a full-scale test on a group of three laterally loaded 12.75-inch diameter steel pipe piles at distances of 1.8, 2.8, and 3.0 pile diameters behind an MSE wall. These tests were performed at a dedicated MSE wall test site near the Point of the Mountain, in Utah.

![Figure 1-1: Group of pipe piles behind an MSE wall prior to backfill placement.](image)
2 LITERATURE REVIEW AND ANALYSIS

Though several studies have explored the lateral resistance of grouped piles with no MSE wall and several other studies have explored the lateral resistance of individual piles with an MSE wall, there is currently very little data on the lateral behavior of grouped piles located behind an MSE wall. A review of lateral load testing on piles both without and with an MSE wall and the effects of lateral pile deflection on MSE wall soil reinforcement and wall displacement is presented in this chapter.

2.1 Grouped Pile Lateral Load Resistance

Several full-scale tests have been conducted on the lateral resistance of pile groups in normal ground conditions, without the presence of a retaining wall or soil reinforcement. A comprehensive review will not be made of these tests due to their limited applicability to the focus of this literature review. However, Table 2-1 shows results from the following studies: Brown (1988), Ruesta and Townsend (1997), Huang et al. (2001), Rollins (2005), and Rollins (2006). The pile groups tested usually consist of an array with 3 or 4 piles per row, with 3 or 4 rows. Only the p-multiplier for the front row of piles is included in the table.
Table 2-1: Full-Scale Tests of Lateral Resistance of Piles Without an MSE Wall

<table>
<thead>
<tr>
<th>Author</th>
<th>Array Shape (Row by Width)</th>
<th>Normalized Row Spacing</th>
<th>Front Row p-Multiplier</th>
</tr>
</thead>
<tbody>
<tr>
<td>Brown (1988)</td>
<td>3x3</td>
<td>3 Diameters</td>
<td>0.8</td>
</tr>
<tr>
<td>Ruesta and Townsend (1997)</td>
<td>4x4</td>
<td>3 Diameters</td>
<td>0.8</td>
</tr>
<tr>
<td>Huang et al. (2001)</td>
<td>4x3</td>
<td>3 Diameters</td>
<td>0.89</td>
</tr>
<tr>
<td>Rollins (2005)</td>
<td>3x3</td>
<td>3.29 Diameters</td>
<td>0.8</td>
</tr>
<tr>
<td>Rollins (2006)</td>
<td>3x3</td>
<td>3.3 Diameters</td>
<td>0.82</td>
</tr>
<tr>
<td></td>
<td></td>
<td>4.4 Diameters</td>
<td>0.90</td>
</tr>
<tr>
<td></td>
<td></td>
<td>5.65 Diameters</td>
<td>0.95</td>
</tr>
</tbody>
</table>

In the study by Rollins (2006), the effect of the front-to-back row spacing of piles was examined. Three cases were tested—row spacings of 3.3, 4.4, and 5.65 pile diameters—each with a transverse pile-to-pile spacing of 3.3 pile diameters. The front or leading row of piles for these cases had p-multipliers of 0.82, 0.90, and 0.95, respectively. This result led to the assumption that for sufficiently large row spacing, there would be no reduction in the lateral resistance of the front row of piles (Rollins, 2006). However, no mention is made of transverse spacing. If one were to extrapolate these findings to a bridge abutment pile group which has no second or third rows of piles, then there ought not be a reduction in lateral resistance between a pile loaded individually or a group of piles loaded together. Model tests performed on a single row of piles suggest that group interactions would become unimportant when piles are spaced more than 3.5 diameters in the transverse direction (Van Impe and Reese, 2010). However, an MSE wall presents much more complex soil-structure interactions than natural or compacted soil without the presence of a wall or soil reinforcements, so extrapolation of results from these studies may be unwise.
2.2 Lateral Load Resistance of Piles Behind MSE Walls

The full-scale tests on laterally loaded piles near MSE walls have been conducted to study the complex soil-structure behaviors of these systems. Of particular focus in these studies is the relationship between pile head load and pile head deflection of piles at varying distances from the wall. Table 2-2, Table 2-3, Table 2-4, and Table 2-5 show the pile, soil reinforcement, wall, and soil parameters used in these tests. With the exception of Pierson et al. (2009), all the studies explored here have been conducted by Brigham Young University graduate students under the direction of Dr. Kyle Rollins (Price, 2012; Nelson, 2013; Han, 2013; Hatch, 2014; Besendorfer, 2015; Budd, 2016; Luna, 2016). With the exception of one group of three shafts at 2 shaft diameters from the wall in the study by Pierson et al., all previous tests loaded piles individually.

Table 2-2: Pile/Shaft Parameters of Full-Scale Tests

<table>
<thead>
<tr>
<th>Author</th>
<th>Test Site</th>
<th>Number of Piles</th>
<th>Material</th>
<th>Cross-Section</th>
<th>Depth of Embedment (ft)</th>
<th>Diameter (in)</th>
<th>Spacing from Wall (Pile Diameters)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Pierson et al. (2009)</td>
<td>I-435/Leavenworth Road Interchange</td>
<td>8</td>
<td>Concrete</td>
<td>Circular</td>
<td>20</td>
<td>36</td>
<td>1.0D, 2.0D, 3.0D, 4.0D</td>
</tr>
<tr>
<td>Price (2012)</td>
<td>US HWY 89</td>
<td>2</td>
<td>Steel</td>
<td>Pipe</td>
<td>70.5</td>
<td>12.75</td>
<td>3.8D, 7.2D</td>
</tr>
<tr>
<td></td>
<td>Pioneer Crossing</td>
<td>3</td>
<td>Steel</td>
<td>Pipe</td>
<td>90, 95, 98</td>
<td>16</td>
<td>2.2D, 2.9D, 5.2D</td>
</tr>
<tr>
<td>Nelson (2013)</td>
<td>Provo Center Street</td>
<td>3</td>
<td>Steel</td>
<td>Pipe</td>
<td>42.25</td>
<td>12.75</td>
<td>1.3D, 2.7D, 6.3D</td>
</tr>
<tr>
<td>Hatch (2014)</td>
<td>Point of the Mountain (Phase 1)</td>
<td>4</td>
<td>Steel</td>
<td>Pipe</td>
<td>33</td>
<td>12.75</td>
<td>1.9D, 3.2D, 4.3D, 5.3D</td>
</tr>
<tr>
<td>Han (2014)</td>
<td>Point of the Mountain (Phase 1)</td>
<td>4</td>
<td>Steel</td>
<td>Pipe</td>
<td>33</td>
<td>12.75</td>
<td>1.7D, 2.8D, 3.1D, 3.9D</td>
</tr>
<tr>
<td>Besendorfer (2015)</td>
<td>Point of the Mountain (Phase 2)</td>
<td>4</td>
<td>Steel</td>
<td>Pipe</td>
<td>38</td>
<td>12.75</td>
<td>1.7D, 2.8D, 2.9D, 3.9D</td>
</tr>
<tr>
<td>Budd (2016)</td>
<td>Point of the Mountain (Phase 2)</td>
<td>4</td>
<td>Steel</td>
<td>Pipe</td>
<td>38</td>
<td>12.75</td>
<td>1.8D, 3.4D, 4.3D, 5.2D</td>
</tr>
<tr>
<td>Luna (2016)</td>
<td>Point of the Mountain (Phase 1)</td>
<td>4</td>
<td>Steel</td>
<td>H</td>
<td>33</td>
<td>12.2</td>
<td>2.2D, 2.5D, 3.2D, 4.5D</td>
</tr>
<tr>
<td></td>
<td>Point of the Mountain (Phase 2)</td>
<td>4</td>
<td>Steel</td>
<td>Square</td>
<td>38</td>
<td>12</td>
<td>2.1D, 3.1D, 4.2D, 5.7D</td>
</tr>
</tbody>
</table>
### Table 2-3: Soil Reinforcement Parameters of Full-Scale Tests

<table>
<thead>
<tr>
<th>Author</th>
<th>Test Site</th>
<th>Type</th>
<th>Material</th>
<th>Vertical Spacing (ft)</th>
<th>Transverse Spacing (ft)</th>
<th>Length (ft)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Pierson et al. (2009)</td>
<td>I-435/Leavenworth Road Interchange</td>
<td>Geogrid (UX1400 and UX1500)</td>
<td>HDPE</td>
<td>2</td>
<td>2</td>
<td>14</td>
</tr>
<tr>
<td>Price (2012)</td>
<td>US HWY 89</td>
<td>Welded Wire Grid</td>
<td>Steel</td>
<td>2.5</td>
<td>1 - 2.5</td>
<td>33</td>
</tr>
<tr>
<td>Pioneer Crossing</td>
<td></td>
<td>Welded Wire Grid</td>
<td>Steel</td>
<td>2.5</td>
<td>5</td>
<td>50, 42, 39</td>
</tr>
<tr>
<td>Nelson (2013)</td>
<td>Provo Center Street</td>
<td>Ribbed Strips</td>
<td>Steel</td>
<td>2</td>
<td>2</td>
<td>28</td>
</tr>
<tr>
<td>Hatch (2014)</td>
<td>Point of the Mountain (Phase 1)</td>
<td>Welded Wire Grid</td>
<td>Steel</td>
<td>2.5</td>
<td>5</td>
<td>18</td>
</tr>
<tr>
<td>Han (2014)</td>
<td>Point of the Mountain (Phase 1)</td>
<td>Ribbed Strips</td>
<td>Steel</td>
<td>2.5</td>
<td>2.25</td>
<td>18</td>
</tr>
<tr>
<td>Besendorfer (2015)</td>
<td>Point of the Mountain (Phase 2)</td>
<td>Ribbed Strips</td>
<td>Steel</td>
<td>2.5</td>
<td>2.5</td>
<td>18</td>
</tr>
<tr>
<td>Budd (2016)</td>
<td>Point of the Mountain (Phase 2)</td>
<td>Welded Wire Grid</td>
<td>Steel</td>
<td>2.5</td>
<td>5</td>
<td>18</td>
</tr>
<tr>
<td>Luna (2016)</td>
<td>Point of the Mountain (Phase 2)</td>
<td>Ribbed Strips</td>
<td>Steel</td>
<td>2.5</td>
<td>2.5</td>
<td>18</td>
</tr>
<tr>
<td></td>
<td>Point of the Mountain (Phase 1)</td>
<td>Welded Wire Grid</td>
<td>Steel</td>
<td>2.5</td>
<td>5</td>
<td>18</td>
</tr>
</tbody>
</table>

### Table 2-4: MSE Wall Parameters of Full-Scale Tests

<table>
<thead>
<tr>
<th>Author</th>
<th>Test Site</th>
<th>Wall Facing Type</th>
<th>Height (ft)</th>
<th>L/H Ratio</th>
<th>Wall Panel Dimension (ft)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Pierson et al. (2009)</td>
<td>I-435/Leavenworth Road Interchange</td>
<td>Concrete Blocks</td>
<td>20</td>
<td>0.7</td>
<td>0.67x1.5</td>
</tr>
<tr>
<td>Price (2012)</td>
<td>US HWY 89</td>
<td>Concrete Panels</td>
<td>20.5</td>
<td>1</td>
<td>5x12</td>
</tr>
<tr>
<td>Pioneer Crossing</td>
<td></td>
<td>Concrete Panels</td>
<td>37.7, 34.7, 29.8</td>
<td>1.1, 1.1, 1.7</td>
<td>5x10</td>
</tr>
<tr>
<td>Nelson (2013)</td>
<td>Provo Center Street</td>
<td>Welded Wire Panels with Geo-Fabric</td>
<td>22.25</td>
<td>0.78</td>
<td>4.8x9.75</td>
</tr>
<tr>
<td>Hatch (2014)</td>
<td>Point of the Mountain (Phase 1)</td>
<td>Concrete Panels</td>
<td>15</td>
<td>0.9</td>
<td>10x5 and 10x7.5</td>
</tr>
<tr>
<td>Han (2014)</td>
<td>Point of the Mountain (Phase 1)</td>
<td>Concrete Panels</td>
<td>15</td>
<td>0.9</td>
<td>10x5 and 10x2.5</td>
</tr>
<tr>
<td>Besendorfer (2015)</td>
<td>Point of the Mountain (Phase 2)</td>
<td>Concrete Panels</td>
<td>20</td>
<td>0.72</td>
<td>10x5 and 10x2.5</td>
</tr>
<tr>
<td>Budd (2016)</td>
<td>Point of the Mountain (Phase 2)</td>
<td>Concrete Panels</td>
<td>20</td>
<td>0.72</td>
<td>10x5 and 10x7.5</td>
</tr>
<tr>
<td>Luna (2016)</td>
<td>Point of the Mountain (Phase 2)</td>
<td>Concrete Panels</td>
<td>15</td>
<td>0.9</td>
<td>10x5 and 10x2.5</td>
</tr>
<tr>
<td></td>
<td>Point of the Mountain (Phase 1)</td>
<td>Concrete Panels</td>
<td>20</td>
<td>0.72</td>
<td>10x5 and 10x2.5</td>
</tr>
</tbody>
</table>
### Table 2-5: Soil Properties of Full-Scale Tests

<table>
<thead>
<tr>
<th>Author</th>
<th>Test Site</th>
<th>Soil Type</th>
<th>Percentage of Proctor Between Test Piles and Wall</th>
<th>Percentage of Proctor Between Test and Reaction Piles</th>
<th>Surcharge (psf)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Pierson et al. (2009)</td>
<td>I-435/Leavenworth Road Interchange</td>
<td>Crushed Stone (CA-5)</td>
<td>Unknown</td>
<td>Unknown</td>
<td>None</td>
</tr>
<tr>
<td>Price (2012)</td>
<td>US HWY 89</td>
<td>A-1-a</td>
<td>Unknown</td>
<td>97.0%</td>
<td>383, 708</td>
</tr>
<tr>
<td></td>
<td>Pioneer Crossing</td>
<td>Sandy Gravel</td>
<td>Unknown</td>
<td>97.0%</td>
<td>808, 1363, 735</td>
</tr>
<tr>
<td>Nelson (2013)</td>
<td>Provo Center Street</td>
<td>Free Draining and Sandy Gravel (two-stage wall)</td>
<td>Unknown</td>
<td>97.4%</td>
<td>657, 135, 657</td>
</tr>
<tr>
<td>Hatch (2014)</td>
<td>Point of the Mountain (Phase 1)</td>
<td>A-1-a, SM</td>
<td>89.0%</td>
<td>96.0%</td>
<td>600</td>
</tr>
<tr>
<td>Han (2014)</td>
<td>Point of the Mountain (Phase 1)</td>
<td>A-1-a, SM</td>
<td>90.0%</td>
<td>96.0%</td>
<td>600</td>
</tr>
<tr>
<td>Besendorfer (2015)</td>
<td>Point of the Mountain (Phase 2)</td>
<td>A-1-a, SP-SM</td>
<td>92.0%</td>
<td>96.5%</td>
<td>600</td>
</tr>
<tr>
<td>Budd (2016)</td>
<td>Point of the Mountain (Phase 2)</td>
<td>A-1-a, SP-SM</td>
<td>91.8%</td>
<td>96.4%</td>
<td>600</td>
</tr>
<tr>
<td>Luna (2016)</td>
<td>Point of the Mountain (Phase 1)</td>
<td>A-1-a, SM</td>
<td>89.5%</td>
<td>95.7%</td>
<td>600</td>
</tr>
<tr>
<td></td>
<td>Point of the Mountain (Phase 2)</td>
<td>A-1-a, SP-SM</td>
<td>92.9%</td>
<td>92.9%</td>
<td>600</td>
</tr>
</tbody>
</table>

#### 2.2.1 Normalized Pile-to-Wall Distance

Previous full-scale tests were designed primarily to quantify the degree to which pile lateral resistance is reduced at varying distances from a wall (Rollins et al., 2016). In general, it was found that piles nearer to a wall than about four pile diameters had lower lateral resistance than piles at distances further than this (Pierson et al., 2009; Rollins et al., 2016). In one of the earliest studies, a pile at 3.9 pile diameters had an equal lateral resistance to a pile at 7.2 pile diameters, leading researchers to speculate that 3.9 pile diameters represented a distance far enough from the wall to not be affected by the presence of the wall (Nelson, 2013). Studies conducted after Nelson (2013) continued to use 3.9 pile diameters in regression equations as the
upper limit above which lateral resistance was not expected to further increase (Rollins et al., 2016).

Additional test results varied in terms of their agreement with 3.9 pile diameters as the distance from the wall needed to maximize lateral pile resistance. Though one study supported a distance of about four pile diameters as the upper limit (Budd, 2016), other studies were split between closer or further normalized distances. Out of the four total studies that include both piles at about 4 pile diameters and some greater spacing, half reported less resistance of piles at about 4 pile diameters compared to piles at about 5 pile diameters (Hatch, 2014; Luna, 2016). On the other hand, Han (2014) and Besendorfer (2015) observed piles at about 3 pile diameters which had nearly identical lateral resistances of piles at about 4 pile diameters. Considering the spread of data above and below the previously assumed upper limit of 3.9 pile diameters, it appears reasonable at the moment to continue to use 3.9 pile diameters as the rough average for minimum pile-to-wall distance above which lateral pile resistance is not expected to increase.

For all full-scale studies except that by Pierson et al. (2009), LPILE was used to back calculate a p-multiplier for each pile tested. For each group of piles, the pile farthest away from the wall—at about four pile diameters or more—was used for calibration purposes with a p-multiplier of 1.0. The API method for sand was used in the analysis, with the unit weight taken from measured data in the field, and friction angle and modulus of subgrade incrementally adjusted until agreement was obtained between the calculated and observed pile head load versus deflection curve. P-multipliers less than 1.0 were then applied to the calibrated model in an iterative process to match the calculated load-deflection curve with the observed curve for each pile closer to the wall. Figure 2-1 shows the back-calculated p-multipliers versus the distance from the center of the pile to the back face of the wall normalized by the pile diameter from the
various studies. In these analyses, a linear trend was identified between normalized pile-to-wall distance and p-multiplier. The most recent regression equation by Rollins et al. (2016) is plotted along with the results, which calculates the p-multiplier for a pile using the equation:

\[
\text{If } \frac{S}{D} \leq 3.9, \text{ then } P_{MSE} = 0.31 \frac{S}{D} - 0.20 \leq 1.0, \text{ or } (2-1a)
\]

\[
\text{If } \frac{S}{D} > 3.9, \text{ then } P_{MSE} = 1, \text{ } (2-1b)
\]

where \( P_{MSE} \) = the p-multiplier for the pile behind an MSE wall,

\( S \) = the distance from the center of the pile to the MSE wall face, and

\( D \) = the diameter of the pile (same units as \( S \)).

![Figure 2-1: P-multiplier versus normalized distance from center of pile to back face of MSE wall of previous tests.](image-url)
Despite the relatively clear trends in the data, some outlying results were recorded. As discussed earlier, while most studies found a p-multiplier of 1.0 to be suitable for four pile diameters or greater distance behind the wall, Hatch (2014) and Luna (2016) recorded instances when a pile at 4.3 and 4.2 diameters had p-multipliers of 0.57 and 0.77, respectively. Conversely, Besendorfer (2015) found that piles at 3.9, 2.9, and 2.8 diameters all had nearly equal lateral resistances. In addition—unlike every other study—Luna (2016) observed an instance when a pile at a closer distance to the wall had a higher lateral resistance than a pile further out. In that test, the H-Pile at 2.1 pile diameters had a significantly larger p-multiplier than the H-Pile pile at 3.1 pile diameters—0.73 compared to 0.60.

2.2.2 Pile Type

Most piles used in these large-scale tests were about 12-inches in diameter, hollow, and made of steel. Pile cross section shapes varied between circular pipe piles, hollow square piles, and H-Piles (Rollins et al., 2016). However, larger steel piles were tested by Price (2012), with 16-inch pipe piles and Pierson et al. (2009) tested cast-in-place 36-inch diameter concrete shafts. The shafts in the study by Pierson et al. (2009) are further unique in that they do not penetrate below the height of the MSE wall, in contrast to other studies where piles are driven between 18 and 60 feet deeper than the base of the wall (Rollins et al., 2016). Although hollow steel piles are often filled with concrete in the field, piles were tested hollow to avoid uncertainty with non-linear effects associated with concrete. A full description of the piles tested is given in Table 2-2.

A comparison between p-multipliers vs. normalized pile-to-wall distance of different pile shapes and pile diameters is made in Figure 2-2 and Figure 2-3, respectively. From the data currently available, it does not appear that either pile type or pile diameter affect the relationship
between normalized distance from the wall and p-multiplier. However, data are sparse for pile
types other than circular or for pile sizes much different than about 12 inches, so further research
is needed in these areas.

![Figure 2-2: Comparison of p-multipliers versus normalized distance from the MSE wall segregated by pile shape.](image)

![Figure 2-3: Comparison of p-multipliers versus normalized distance from the MSE wall segregated by pile diameter.](image)
2.2.3 Wall and Soil Characteristics

Lateral displacement of piles near MSE walls involve complex interactions between the pile, soil, soil reinforcement, and wall facing. Soil reinforcement, wall, and soil parameters from the full-scale tests are shown in Table 2-3, Table 2-4, and Table 2-5. The effects of these parameters on lateral pile resistance reduction will be considered in this section.

Two types of inextensible soil reinforcement, welded wire and ribbed strips, as well as one type of extensible soil reinforcement, geogrid, were used in the MSE walls previously tested. However, comparison of lateral resistance reduction is possible only between the tests with inextensible reinforcements, as the one test with extensible reinforcement did not analyze the piles in the same way as in the other studies (Pierson et al., 2009). A comparison of p-multipliers between piles placed between welded wire and ribbed strip soil reinforcement is given in Figure 2-4. From current data, there is not a clear difference in back-calculated p-multipliers for piles placed behind MSE walls using either welded-wire or ribbed strip reinforcement.

![Figure 2-4: Comparison of p-multipliers versus normalized distance from the MSE wall segregated by reinforcement type.](image)

Figure 2-4: Comparison of p-multipliers versus normalized distance from the MSE wall segregated by reinforcement type.
Backfill properties for the full-scale tests are given in Table 2-3. Most walls used soil classified as A-1-a and were compacted behind the piles to an average of about 95% of standard proctor density. Relative compaction between the piles and the wall tended to be about 5% lower than behind the piles in the cases where these regions were measured separately. In addition, there was more scatter in the relative compaction near the wall. The difference in compaction near the wall is due to the inability of rolling compactors to be used close to the wall and piles for safety reasons, size constraints, and concerns over damaging the wall. These difficult to access areas were instead compacted with smaller and lighter equipment, such as walk-behind vibratory compactors.

In cases where discrepancies were observed in test data from the expected values, a commonly cited potential cause was inconsistent compaction between the test piles and the wall (Besendorfer, 2015; Luna, 2016). Unlike the relatively wide-open area behind the test piles that can be compacted in a straightforward back-and-forth pattern, the piles act as obstacles that necessitate less conventional weaving patterns, resulting in some soil being crossed more often than other soil with the compacting equipment. With more passes in some areas than others, inconsistency in compaction is inevitable and difficult to quantify.

Soil type and compaction are shown to be a major factor in lateral pile resistance in the two-stage wall studied by Nelson (2013). As Figure 2-5 depicts, piles TP1, TP2, and TP3 are surrounded by a granular draining material which was only loosely compacted, while pile TP4 was surrounded by A-1-a backfill compacted to 97% of standard proctor. Figure 2-6 shows the pile head versus deflection for the piles in the study by Nelson (2013). With just the small increase in distance from the wall that places the pile within the higher compacted A-1-a backfill, pile TP4 gained about twice the lateral resistance as pile TP3. As was discussed, lateral
resistance is not normally expected to increase between piles about four pile diameters from the wall and further (Price, 2012; Rollins et al., 2016) with relatively homogenous backfill. Thus, the increase in lateral resistance for pile TP4 at 7.7 pile diameters compared to pile TP3 at 6.3 pile diameters is likely mostly due to the change in surrounding soil type and compaction.

Figure 2-5: Backfill and pile layout in the two-stage wall tested by Nelson (2013).

Figure 2-6: Pile head load versus deflection for piles in the study by Nelson (2013).
The pile used for LPILE soil parameter calibration for Nelson’s (2013) study was pile TP3 rather than pile TP4. Therefore, this group of piles represent the only data points for piles in loosely compacted backfill—though it is noted that part of the soil reinforcements for these piles penetrated the higher compacted backfill. Figure 2-7 shows the p-multipliers versus normalized pile-to-wall distance from the various studies with the test results from Nelson (2013) emphasized. As these piles were observed to have p-multipliers in general agreement with other piles which had significantly more compacted and stiffer backfill, it is tentatively conjectured that the level of compaction does not significantly affect the relationship between normalized distance from the wall and p-multiplier—as long as compaction is relatively homogenous across the varying distance from the wall. However, as Nelson (2013) is the only test so far with compaction significantly different from the average, more studies are needed to explore the possible effects of soil type and soil compaction on lateral pile resistance reduction.

Figure 2-7: P-multiplier versus normalized pile-to-wall distances.
2.2.4 Pile to Panel Placement

Due to the geometry of the wall panels and soil reinforcement, piles were placed to be either behind joints in wall panels or behind the center of wall panels in every study except for that by Pierson et al. (2009). Figure 2-8 indicates which data points were from piles behind the center of the panel or behind a panel joint (Luna, 2016). There appears to be no clear connection between placement of a pile in relation to joints in a wall and the relationship between normalized distance from the wall and lateral pile resistance reduction. This observation seems reasonable considering that the reinforcing strips provide most of the lateral resistance to lateral pile movement, not the wall panels, and the reinforcing strip spacing is consistent across the wall.

Figure 2-8: Comparison of p-multipliers versus normalized distance from the MSE wall segregated by pile to panel placement (Luna, 2016).
2.2.5 Length to Wall Height Ratio

The reinforcement length to wall height (L/H) ratio was compared across studies to determine if it had an effect in lateral resistance reduction rates (Rollins et al., 2016). Using his own results and that of Price (2012), Nelson (2013) proposed three best-fit lines depending on different L/H ratios. However, after several other studies were completed, Budd (2016) and Luna (2016) found no discernable difference between the L/H ratios and p-multipliers. Figure 2-9 compares different L/H ratios and the p-multipliers of piles at varying normalized distances from MSE walls. One caveat, however, is that when multiple piles were pushed in unison in the study by Pierson et al. (2009), a tension crack developed above the end of the reinforcements, as shown in Figure 2-10. While this phenomenon did not appear in any of the single pile tests, it may be that a larger L/H ratio is needed to develop full lateral resistance in some situations, such as for multiple piles deflecting in unison. Further testing with grouped piles at various distances from the wall and various L/H ratios may yet find a relationship between L/H ratios and lateral pile resistance reduction.

![Figure 2-9: Comparison of p-multipliers versus normalized distance from the MSE wall segregated by length to height wall ratio (Luna, 2016).](image-url)

Figure 2-9: Comparison of p-multipliers versus normalized distance from the MSE wall segregated by length to height wall ratio (Luna, 2016).
2.2.6 Grouped Piles

Prior to this study, only one full-scale test had been conducted to investigate group effects on the lateral resistance of piles near MSE walls. Pierson et al. (2009) compared the lateral resistance of a single drilled shaft spaced two shaft diameters behind the wall with a group of three shafts with a transverse center-to-center spacing of five shaft diameters and a shaft-to-wall spacing of 2 shaft diameters. Figure 2-11 shows the load versus displacement curves for these shafts. Shaft B is the shaft tested individually and shafts BG1, BG2, and BG3 are the grouped shafts, with BG2 in the center. The grouped shafts each have a lower lateral resistance compared to the single shaft. Also, the shaft in the center generally had a somewhat higher resistance compared to the shafts on the edges for larger displacements. This trend is contrary to findings from studies of grouped piles embedded in the ground without soil reinforcements or a
nearby wall, where the piles on the edges had consistently higher resistances than those in the center (Rollins, 2005). Further testing is needed to quantify the extent that group effects reduce lateral resistance.

![Figure 2-11: Shaft head load versus displacement for shafts spaced at two shaft diameters from the wall (Pierson et al., 2009).](image)

### 2.3 Induced Forces in MSE Wall Reinforcements

An MSE wall allows cohesionless soil to withstand tensile force by transferring forces to soil reinforcements through skin friction. MSE walls are designed to consider the tensile forces expected of the active earth pressure of the soil on the wall face. Laterally loaded piles behind the MSE wall add to the tensile force induced on the soil reinforcements, with a positive relationship between pile head load and induced load on reinforcements (Besendorfer, 2015; Budd, 2016; Han, 2014; Hatch, 2014; Luna, 2016; Nelson, 2013; Price, 2012). It is important for an engineer have some idea of the expected induced force on soil reinforcements due to lateral displacement of piles to maintain adequate factors of safety in design. This section will cover the
factors that have been shown to affect the magnitude of induced loads in soil reinforcement, possible mechanics that generate the loads observed along soil reinforcements, and the latest regression equations for predicting maximum induced loads.

2.3.1 Transverse Pile to Reinforcement Distance

A clear connection was found between the parallel-to-the-wall distance between the test pile and the reinforcement and the relative magnitude of the induced load on the reinforcement. Figure 2-12 shows strain in extensible reinforcement in relation to distance from the edge of a test pile parallel to the wall face (Pierson et al., 2009). As the transverse distance from the pile increases, the strain in the reinforcement decreases. Both extensible and inextensible reinforcement were found to have decreased induced tensile loads in soil reinforcement as the transverse distance between the reinforcement and the pile increased (Nelson, 2013; Hatch, 2014; Budd, 2016; Luna, 2016).

Figure 2-12: Strain of geogrid layer at 18.7 feet elevation between grouped shafts (Pierson et al., 2009).
2.3.2 Longitudinal Distance Behind Wall Face

As strain gauges were placed at varying distances along the soil reinforcements, researchers were able to identify general trends in the induced loads versus distance from the MSE wall. In general, induced loads were found to be highest roughly the same distance behind the MSE wall as the displaced pile, with decreased induced loads with increasing distance from the pile (Price, 2012; Nelson, 2013; Han, 2014; Hatch, 2014; Budd, 2016; Luna, 2016). Of the studies that measured induced reinforcement load, only Besendorfer (2015) failed to find a clear connection between the pile-to-wall spacing and the location of the maximum reinforcement induced load. However, the study by Besendorfer (2015) had other anomalies—such as an equal lateral resistance for piles spaced at 3.9, 2.9, and 2.8 pile diameters from the wall—suggesting that there might be some complicating factor such as inconsistent soil compaction.

Figure 2-13 shows an idealized model that several researchers suggest may occur concerning the soil reinforcement interaction with the pile, soil, and wall (Nelson, 2013; Han, 2014; Hatch, 2014; Besendorfer, 2015; Budd, 2016; Luna, 2016). In this model, as the pile is loaded laterally toward the wall, the soil between the pile and the wall moves left relative to the reinforcement—with the most soil movement occurring near the pile—resulting in friction induced tension in the reinforcement. As the reinforcement is pulled left relative to the soil behind the loaded pile, skin friction transfers tensile force from the reinforcement to the soil. Thus, tensile load is largest near the pile, and gradually decreases both behind and in front of the pile. Induced tensile force in the reinforcement at the wall face is thought to be due to increased earth pressure from the displaced soil.
2.3.3 Reinforcement Depth

Studies agree that significant wall and soil mobilization is confined to the top 10–12 feet of the MSE wall (Besendorfer, 2015; Luna, 2016). With little or no soil mobilization, there should be minimal induced reinforcement load below these depths. However, within the top several feet of the wall, trends concerning the relationship between reinforcement depth and the relative magnitude of induced force on the reinforcement from lateral pile movement have not been consistent throughout the various tests. The test performed by Pierson et al. (2009), though not equipped to monitor soil reinforcement loads directly, found that the maximum wall movement was generally a few feet below the top of the wall—which would likely correlate to maximum induced load in the soil reinforcement. This finding was later supported by Han (2014) and Hatch (2014); the inextensible reinforcement from these two studies were found to have somewhat higher maximum induced tensile force in the second layer from the top than the first, a depth of 3.75 feet compared to 1.25 feet. However, in the H-Pile tests by Luna (2016),
half of the laterally loaded piles induced the greatest tensile force in the uppermost layer of reinforcements. It is noted that the most recent regression equation for maximum induced load for ribbed strip reinforcement has no parameter for reinforcement depth (Rollins et al., 2016).

In studies by Budd (2016) and Luna (2016), they suggested that the depth for which maximum induced reinforcement load is found is affected by the pile-to-wall spacing—though the two studies report opposite results. Specifically, Budd (2016) found that as pile-to-wall spacing increases, so too does the depth increase at which maximum tensile force is imparted on the reinforcement. However, in the H-Pile tests by Luna (2016), the opposite was found, with piles at greater distances from the wall inducing maximum reinforcement loads at higher depths than piles at closer distances. More data are needed to find a possible connection between pile-to-wall spacing and depth of maximum soil reinforcement induced load.

2.3.4 Pile-to-Wall Distance

Researchers have failed to reach a consensus on the effect of pile-to-wall distance and the magnitude of induced load on soil reinforcements at any depth. Price (2012) tested at two locations and found contrary results between the sites. In the U.S. Highway 89 site study, though the piles spaced at 7.3 and 3.8 pile diameters from the wall had nearly identical pile head load versus deflection curves, the soil reinforcements near the pile spaced at 3.8 pile diameters had nearly twice the magnitude of induced tensile loads as the other. This suggested that decreased pile-to-wall spacing increases reinforcement loads. On the other hand, in the Pioneer Crossing site study, the piles closer to the wall caused smaller induced soil reinforcement loads compared to piles further away. However, unlike at the U.S. Highway 89 site, the piles further from the wall received larger pile head loads than those closer to the wall due to their stiffer response, and
it may be that pile head load is a more significant factor in induced soil reinforcement load than pile-to-wall distance. Hatch (2014) and Besendorfer (2015) found—similar to the U.S. Highway 89 site from Price (2012)—that decreased pile-to-wall distance led to increased induced loads in soil reinforcements. Budd (2016) found the opposite to be true. Due to the lack of consensus, it may be that other factors play a larger role in affecting induced loads in soil reinforcement than the distance of the pile to the wall.

2.3.5 Most Recent Regression Equations

Due to the relatively complicated soil structure interaction between the pile, backfill, wall, and soil reinforcement, researchers were unable to develop a simple equation to predict forces induced in the soil reinforcements. Instead, regression equations were produced using the Statistical Analysis System (SAS) software program and the Data Analysis pack for Microsoft® Excel. Effort was made to reduce as many parameters as possible without significantly decreasing the R² value for each model (Rollins et al., 2016). Separate equations were developed for the ribbed strip and welded-wire reinforcement types because of the difference in geometry of the two reinforcements.

The most recent regression analysis of ribbed strip soil reinforcements used 942 data observations from previously performed studies, resulting in an R² value of 0.71. In this equation, the maximum induced force in a ribbed strip reinforcement due to a laterally loaded pile is calculated in the following way:

\[
\Delta F \text{(kips)} = 10^\left(0.13 + 0.028P - 2.2 \times 10^{-4}P^2 - 0.01 \frac{T}{D}ight)
- 0.0021P \frac{T}{D} - 0.031 \frac{s}{D}) - 1, \quad (2-2)
\]
where \( F \) = the maximum predicted tensile force (kip),

\[ P \] = the pile head load (kip),

\[ T \] = the transverse distance from reinforcement to pile center (in.),

\[ D \] = the pile diameter (in.), and

\[ S \] = spacing from pile center to back face of MSE wall (in.).

Figure 2-14 shows the predicted induced load on the reinforcement using Equation 2-2 compared with the observed loads, (Rollins et al., 2016). The solid line represents the instances where the measured load equals the predicted load, while the dashed lines represent the mean plus and minus one and two standard deviations. It is noted that unlike the regression equation for welded wire soil reinforcement, Equation 2-3, the equation for ribbed strip reinforcement does not consider the depth of the soil reinforcement.

Figure 2-14: Predicted versus measured maximum ribbed strip reinforcement tensile force.
As my study used ribbed strip reinforcements, I plotted Equation 2-15 in Microsoft Excel to investigate potential limitations in the applicability of the equation. Figure 2-15 depicts the calculated maximum induced force in a hypothetical reinforcement strip for pile head loads between 0 and 80 kip—transversely spaced at varying distances from a 12.75” pile spaced 3 pile diameters from an MSE wall face. In each instance, Equation 2-2 predicts decreased induced force for increased pile head load after a certain point. This inflection point occurs at a decreased pile head load with an increase in transverse pile to reinforcement distance. With sufficient transverse spacing and pile head loads, Equation 2-2 will predict negative induced loads, which approach -1 kip. Care must be taken when applying Equation 2-2 for pile and wall designs differing from the tests from which the data was taken to create the equation. Particular care should be observed when attempting to predict maximum induced tensile forces in reinforcements transversely spaced several feet away from the loaded pile, as even small pile head loads can extend past the inflection point, as seen in Figure 2-15.

Figure 2-15: Calculated reinforcement induced loads with various transverse distances from a 12.75-inch pile spaced at 3 pile diameters from the wall.
The most recent regression analysis of welded wire soil reinforcements used 1,058 data observations from previously performed studies, resulting in an $R^2$ value of 0.72. In this equation, the maximum induced force in a ribbed strip reinforcement due to a laterally loaded pile is calculated in the following way:

\[ \Delta F(kip) = 10^{(-0.04 + 0.027P - 2.7 \times 10^{-4}P^2 + 5.7 \times 10^{-4}\sigma_V \nonumber \}
\]

\[ -2.6 \times 10^{-7}\sigma_V^2 - 0.08 \frac{T}{D}) - 1, \quad (2-3) \]

where $F = \text{the maximum predicted tensile force (kip)},$

$P = \text{the pile head load (kip)},$

$\sigma_V = \text{the vertical stress (psf)},$

$T = \text{the transverse distance from reinforcement to pile center (in.), and}$

$D = \text{the pile diameter (in.).}$

Figure 2-16 shows the predicted induced load on the reinforcement using Equation 2-3 compared with the observed loads, (Rollins et al., 2016). The solid line represents the instances where the measured load equals the predicted load, while the dashed lines represent the mean plus and minus one and two standard deviations. It is noted that unlike the regression equation for ribbed strip soil reinforcement, Equation 2-2, the equation for welded wire reinforcement does not consider the pile-to-wall spacing.

These regression equations may give an MSE wall designer some general idea of likely induced loads in soil reinforcements from lateral pile loads. However, the data are limited to two to three MSE wall locations for either the ribbed strip or welded wire reinforcements. It may be
that these regression equations have limited usability for different wall designs or different pile and soil types than those used in the analysis.

Figure 2-16: Predicted versus measured maximum welded wire reinforcement tensile force.

2.4 MSE Wall Displacement from Laterally Loaded Piles

MSE walls are faced with various materials, such as concrete panels and blocks or wire mesh. These facings are able to deform to some degree before losing functionality. However, excess deformation can lead to failure or negatively affect the aesthetics of the wall. It has been shown that there is a positive correlation between lateral pile head load and induced deformations on the wall face (Rollins et al., 2016). It is important for an engineer to have an estimate of this deformation. Additionally, an understanding of the variables which increase or decrease wall deflection will allow an engineer to design an MSE with small deformations as an objective.
2.4.1 Pile-to-Wall Distance

Pile-to-wall distance has been observed to affect the relative magnitude of MSE wall deflections. In each study where wall deflections were measured, decreased distance between the pile and the wall correlated with increased wall movement (Pierson et al., 2009; Price, 2012; Nelson, 2013; Hatch, 2014; Luna, 2016). Piles tend to gain greater stiffness with increased distance from an MSE wall, meaning that for a given load, a pile further from an MSE wall will generally deflect less than one closer (Nelson, 2013). However, even for similar pile head deflections, piles further from the wall tend to induce smaller wall deflections than closer piles, as seen in Table 2-6 (Pierson et al., 2009). The table shows maximum wall displacements for various shaft head displacements and normalized distances from the wall. The shaft spaced at one shaft diameter deflected the wall by about twice the amount as the shaft spaced at four shaft diameters for a given shaft head displacement.

<table>
<thead>
<tr>
<th>Normalized shaft to wall distance</th>
<th>Maximum wall deflection for a 3-inch pile head deflection</th>
<th>Maximum wall deflection for a 4.5-inch pile head deflection</th>
</tr>
</thead>
<tbody>
<tr>
<td>1D</td>
<td>2.25”</td>
<td>4”</td>
</tr>
<tr>
<td>2D</td>
<td>2.25”</td>
<td>3.25”</td>
</tr>
<tr>
<td>3D</td>
<td>1.75”</td>
<td>3”</td>
</tr>
<tr>
<td>4D</td>
<td>1.1”</td>
<td>2”</td>
</tr>
</tbody>
</table>
2.4.2 **Wall and Reinforcement Type and Soil Characteristics**

Comparison of the magnitude of wall displacement across test sites is complicated by differences in wall type and soil properties and compaction, which make singling out significant factors difficult. However, it is noted that the MSE walls tested by Pierson et al. (2009) and Nelson (2013) exhibited much larger wall deflections than the walls at other sites. For pile or shaft deflections of about 3 inches, Pierson et al. (2009) and Nelson (2013) reported wall deflections up to 2.25 and 1.15 inches, respectively. This is in contrast with the maximum wall deflections between 0.1 and 0.5 inches reported at other sites (Price, 2012; Nelson, 2013; Besendorfer, 2015). See Table 2-2, Table 2-3, Table 2-4, and Table 2-5 for a full description of the differences between these test sites. Possible significant factors for the relatively large wall deflections at the two test sites are the extensible soil reinforcement and block wall facing for Pierson et al. (2009) and the two-stage wall backfill and wire mesh wall facing for Nelson (2013). However, more testing is needed to determine which if any of these factors were the true cause of the large wall displacements.

Of the tests performed at the same site with the same backfill and compaction practices, there was found to be no discernable difference in wall deflection between MSE walls built with either ribbed strips or welded-wire reinforcements (Rollins et al., 2016). These studies did find, however, that half-sized wall panels used in the top layer of a wall would deflect much more than full or extra-large wall panels in the top layer (Besendorfer, 2015; Luna, 2016). The half-sized panels were secured with a single layer of soil reinforcement unlike the larger panels which had
two or three layers of connection. This allowed the half-sized panels to rotate during loading, leading to larger deflections (Luna, 2016).

In walls built with paneled facings, the greatest deflections are typically seen along panel joints (Luna 2016). Also, discontinuities across joints are common, with sudden changes of wall deflection observed between both vertically and horizontally adjoined panels (Besendorfer, 2015; Luna, 2016). Changes in wall deflection in walls built with block or wire mesh wall facings tended to be more gradual than paneled walls, with wall deflections more evenly centered around the location of the pile (Pierson et al., 2009; Nelson, 2013).

2.4.3 Pile Placement

The placement of piles in relation to joints in the wall panels was found to have a significant effect on wall deflection. A typical panel design as tested in many of the studies is shown in Figure 2-17. To avoid soil reinforcements, piles were placed either directly behind joints in wall panels or behind the center of a row of panels. In general, the region of significant wall displacement was found to be less spread horizontally for piles directly behind wall joints (Besendorfer, 2015; Budd, 2016). In addition, there may be a difference in the magnitude of wall displacement based on pile to panel placement, with greater wall deflections generally observed for pile placed behind joints in wall panels (Luna, 2016). One study found that piles behind panel joints induced about twice the amount of wall displacement compared to piles behind panel centers (Budd, 2016).
2.4.4 Distance from Top of Wall

When piles were pushed laterally behind MSE walls, the walls would often have maximum deformation a few feet below the top of the wall (Pierson et al., 2009; Besendorfer, 2015; Budd, 2016). This indicates that rather than simply rotating, the wall moves largely horizontally. However, wall displacement was largely confined to the top 10 to 12 feet from the top of the wall (Besendorfer, 2015). One study, conducted by Budd (2016), found that increased pile-to-wall distance was correlated with increased distance from the top of the wall to the site of maximum wall deflection. This finding has not been noted in any other study, however.

2.4.5 Grouped Piles

Pierson et al. (2009) offers the only example of the effect of grouped piles on MSE wall deflection. Figure 2-18 shows the induced horizontal deflection on the block wall by a single shaft placed at two shaft-diameters from the wall. The data points are all at 17.7 feet elevation, or 2.3 feet from the top of the wall—the approximate location of typical maximum wall movement (Pierson et al., 2009). Wall deflection followed a bell-shaped curve, with less than 0.5 inches of
displacement 9 feet away from centerline of the shaft for a shaft head displacement of 6 inches. As noted previously, another test was performed on a group of three shafts also spaced at two shaft-diameters from the wall and transversely spaced at five shaft-diameters from each other. The induced wall deflections at 17.7 feet elevation from these grouped shafts is shown in Figure 2-19. For similar shaft head deflections, the wall in-line with the center of the shafts displaced nearly equally for both the single and grouped shafts. Likewise, deflections outside of the outer grouped shafts were similar to that measured to either side of the single shaft. However, the regions of the wall between the grouped shafts displaced significantly more than was observed at a similar distance from the shaft for the single shaft test—deflections at six feet from centerline are over twice the magnitude between the grouped shafts as in the single shaft. Thus, deflections between the grouped shafts was greater than the superimposed deflections of single shafts, though maximum wall deflections did not increase for grouped shafts.

Figure 2-18: Single pile incremental deflection of wall at elevation 17.7 feet (Pierson et al., 2009).
2.5 Limitations of Existing Research and Need for Additional Research

While abutment piles placed near MSE walls act as a group, almost all the full-scale tests were performed on individual piles. Testing done on grouped piles in soil without the presence of an MSE wall or other impedance has shown that the interaction of multiple simultaneously loaded piles decreases the lateral soil resistance on the piles (Brown et al., 1988). The one full-scale test on grouped piles near an MSE wall likewise indicates that the soil loses additional lateral resistance for grouped piles above that of individual piles at the same distance from the wall (Pierson et al., 2009). However, the reduction in lateral resistance owing to group effects near a wall have not been adequately quantified. In my research, I will add to the limited data on the subject of grouped piles near MSE walls and I will quantify the loss of lateral soil resistance by back-calculating separate p-multipliers for each pile based on normalized distance from the wall and on grouped pile effects.
3 TEST LAYOUT

The test site is located on Geneva Rock property, just south of the southern face of Point of the Mountain in Utah County, near Lehi, Utah. The coordinates for the site are 40.453194, -111.899304. Figure 3-1 gives a long-distance view of the general location. Figure 3-2 shows two closer perspectives of the site.

Figure 3-1: Long-distance view of test area from Google Maps.
3.1 Site Preparation

Prior to construction, the site had a slope with a run-to-rise ratio between 3:1 and 5:1. Initial leveling was accomplished using a CAT D9T bulldozer with built-in automatic leveling and elevation instrumentation. In order to place the wall and leveling pad at minimum embedment depth, a 2-foot deep cut was made along the planned length of the wall, which extended 25 feet behind the wall. This cut extended between 5 to 7 feet into the existing slope.

3.2 Piles

This research examines circular pipe piles donated by Atlas Steel, with a diameter of 12.75 inches, wall thickness of 0.375 inches, and length of 40 feet. The piles are made of A252-Grade 3 steel, with a yield strength of approximately 57,000 psi. Following the leveling of the site, the piles were driven open-ended 18 feet below grade. Piles plugged during installation. Rather than being filled with concrete, the piles were left hollow to avoid the nonlinear effects of...
concrete cracking during testing. Piles were originally planned to be driven at normalized
distances of 1, 2, 3, and 4 pile diameters from the wall, but final normalized distances were 1.8,
2.8, 3.0, and 4.0 pile diameters, as shown in Figure 3-4. The pile at 4.0 pile diameters from the
wall was tested individually for calibration purposes, and the remaining piles were grouped
together in testing. The piles will hereafter be referred to by both their normalized distance as
well as their placement in relation to the other piles: 1.8D (West) pile, 2.8D (Center) pile, 3D
(East) pile, and 4D single pile, as shown in Figure 3-3. The group of three piles were spaced an
average of 4.7 pile diameters from each other. Additional piles were driven beyond the reach of
the soil reinforcements to support a reaction beam.

![Figure 3-3: Pile and surcharge placement and pile naming.](image-url)
3.3 MSE Wall

The MSE wall was designed in accordance with the AASHTO 2012 LRFD code. Construction took place in three phases. In Phase 1, the wall was built to a height of 15 feet and tested. Phase 2 added five more feet onto the wall, bringing the wall up to 20 feet tall, and further testing was completed. Finally, Phase 3 included 6.25 feet of the existing wall being excavated and then rebuilt with new instrumentation and soil reinforcement, bringing the wall back up to the same 20 feet height of Phase 2. Phase 3 cannot be seen as a repeat of Phase 2, however, as

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**Figure 3-4: Pile placement and spacing.**
the soil was compacted to a higher target density in Phase 3 than in Phase 2. In addition, while Phases 1 and 2 were constructed by Hadco, Inc., BYU employees and students excavated and rebuilt the top 6.25 feet of the wall in Phase 3.

The wall is composed of two sections, each with 50 feet of main wall and 40 feet of wing wall. The west side uses the Reinforced Earth Company (RECO) wall system, with smooth facing 5’x10’ reinforced concrete panels and 18-foot long galvanized steel ribbed strip soil reinforcements connected to the wall panels with bolts, as shown in Figure 3-5. The east side of the wall uses the SSL wall system, with texture facing 5’x10’ reinforced concrete panels and 18-foot long galvanized steel welded-wire grid soil reinforcements, as shown in Figure 3-6. The two wall systems are separated with a wall joint to prevent one wall from affecting the other. This study analyzes piles on the far west side of the main wall section with the RECO wall system, as seen in Figure 3-7.

Figure 3-5: Ribbed-strip soil reinforcement placement as on the site’s west side (RECO).
Figure 3-6: Welded-wire grid soil reinforcement placement as on the site's east side (SSL).

Figure 3-7: Wall face directly in front of grouped piles.
3.3.1 Backfill

Backfill for this project was provided by Geneva Rock. As Phase 3 fully excavated the soil placed in Phase 2, only Phase 1 and 3 will be discussed here. Phase 1 backfill is classified as AASHTO A-1-a material and as silty sand with gravel (SM) by the Unified Soil Classification System (USCS). Phase 3 backfill also classifies as AASHTO A-1-a material, but due to somewhat less fines content is designated as poorly graded sand with silt and gravel (SP-SM) by the USCS. Phase 1 backfill had a maximum standard proctor density of 128.0 pcf and optimum moisture content of 7.8%, while Phase 3 had maximum standard proctor density of 126.7 pcf and optimum moisture content of 9.7%. Figure 3-8 shows the grain size distribution of the two backfill soils used.

![Grain Size Distribution Graph](image)

**Figure 3-8: Backfill grain size distribution.**
Phase 1 compacted the backfill to a target density of 95% of standard proctor, though actual densities were measured to fall somewhat less than that target between the piles and the wall, which was compacted using a vibratory plate compactor rather than the rolling compactor used to compact the soil behind the piles. Figure 3-9 show the measured percentage of standard proctor versus depth of the backfill in Phase 1. Note, however, that Phase 3 later excavates and re-compacts the top 1.25 feet of backfill here shown and then adds 5 additional feet of compacted fill.

![Figure 3-9: Measured backfill density at various depths of Phase 1.](image)

In Phase 3 of construction on the MSE wall, the site was excavated to a depth of 6.25 feet below the final height of the wall in Phase 2. As Phase 2 consisted of the addition of only the top 5 feet, Phase 3 fully excavates the compaction of Phase 2 and the top 1.25 feet of Phase 1. Figure 3-10 shows a view of the east side of the site just prior to excavation, and Figure 3-11 and Figure 3-12 show portions of the site during excavation. A backhoe was used to excavate soil behind the piles, but picks and shovels were used for soil between the piles and the wall to avoid damaging
either the piles or the wall. Rather than excavating the entire site, excavation was limited to the area with soil reinforcement, as the primary purpose for excavating was to place newly instrumented soil reinforcement.

Figure 3-10: Test site before excavation.

Figure 3-11: Excavation between test and reaction piles with backhoe.
Figure 3-12: Excavation near wall face with pick and shovels.

After excavation, soil reinforcements were placed and secured to the wall, as in Figure 3-5 and Figure 3-6, and then compacted over in 8-inch lifts of backfill. Soil was placed and evenly spread by backhoe, walk-behind front-end loader, and shovels, as shown in Figure 3-13, Figure 3-14, Figure 3-12, Figure 3-15, and Figure 3-16.
Figure 3-13: Adding soil to excavated site with backhoe.

Figure 3-14: Evenly spreading soil in eight-inch lifts by backhoe.
Figure 3-15: Evenly spreading soil in eight-inch lifts by compact front-end loader.

Figure 3-16: Evenly spreading soil in eight-inch lifts by shovel.
Following soil placement, the soil was wetted to near its optimal moisture content and compacted using a combination of vibratory plate compactor, jumping jack compactor, and roller compactor. As shown in Figure 3-17, the vibratory compactor was used within 3 feet of the wall to prevent damaging the wall—with a target density of 95% of standard proctor compaction. The roller compactor and jumping jack compactor were used for all other areas, with a target density of 100% standard proctor compaction, as shown in Figure 3-18. The jumping jack compactor was used for areas further than 3 feet away from the wall that were difficult or impossible for the roller compactor to reach—such as near or between the piles or at the edge of the excavation, as shown in Figure 3-19.

Figure 3-17: Soil wetting and vibratory plate compactor usage.
Figure 3-18: Roller compactor and jumping jack compactor in use.

Figure 3-19: Jumping jack compaction near soil walls at the edge of excavation.
Soil density and water content measurements were made regularly during compaction with a nuclear density gauge, as shown in Figure 3-20. Dry density results from these measurements are presented in Figure 3-21.

**Figure 3-20**: Nuclear density gauge testing.

**Figure 3-21**: Dry density versus depth of Phase 3 soil.
3.3.2 Surcharge

Concrete blocks were placed behind the test piles during loading to simulate the surcharge induced by the abutment and approach fill typical for a bridge. The blocks measured 2’x2’x6’ and weighed an estimated 150 pcf. A typical block configuration for single pile testing consistent with that for the 4D single pile test can be seen in Figure 3-22. The concrete block surcharge for the group test is shown in Figure 3-3. At two units high, the blocks approximate a 600 psf surcharge, or 5 feet of 120 pcf soil. However, actual surcharge is somewhat less than this amount due to the gaps needed for the loading system and frame.

Figure 3-22: Typical surcharge setup for single-pile tests.
3.4 Frame  

A combination of channel and flange sections provided a connecting frame between the two hydraulic actuators and the three pile heads in the group test, as seen in Figure 3-23 and Figure 3-24. Load cells connected the frame to the piles with pinned connections. Wheels were welded under the longitudinal flange sections to minimize frictional resistance. Flange sections placed on the ground surface acted as runners for the wheels.

Figure 3-23: Group test frame to piles pinned connections.
3.5 Loading System

A reaction beam and a series of reaction piles served as a support for the loading system to push the test piles. Figure 3-25 shows the reaction beam and some of the reaction beams used on site. In the case of the 4D single pile test, a single hydraulic jack provided the loading force on the pile in compression, separated from the pile head with a load cell to measure the applied load and a hemispherical compression platen to reduce the possibility of eccentricity during loading, as shown in Figure 3-27. In the group test, two 110 kip MTS linear hydraulic actuators applied load to the frame in compression, which transferred the load to the three piles through the load cell connections in tension. Figure 3-26 shows the reaction structure and loading system used during the grouped pile test. Load was applied to the pile head about 12-inches above the ground surface in both the single and group test.
Figure 3-25: Placement and leveling of the reaction beam.

Figure 3-26: Loading system for the group test with two 110 kip MTS linear actuators positioned between the load frame and the reaction frame.
Figure 3-27: 4D single pile loading system.
4 INSTRUMENTATION

Instrumentation was installed to measure the performance of the pile, soil, and wall. Data monitored in the group test included the extension and applied force of the two hydraulic actuators; the induced load, deflection, and rotation of each of the three pile heads; the horizontal and vertical deflection of the top of the wall panels; the vertical heave of the soil between the test piles and the wall; the induced strain the top three layers of soil reinforcement; and the deflection of the wall in front of the test piles.

4.1 Load Cells

Pile head load was measured using a combination of load cells. In the case of the single pile test, a load cell was placed between the hydraulic jack and the pile and load was applied in compression, as shown in Figure 3-27. For the grouped piles, load cells connected the steel frame to the individual piles on the opposite side of the pile as the hydraulic actuators, as shown in Figure 4-1. This setup applied load to the pile in tension. Applied moments were avoided with a pinned connection integrated into the load cell. Load cells in the hydraulic actuators were also used on the second day of group testing to measure load for the 3D (East) pile due to a load cell malfunction. Load for the 3D (East) pile on the second day of testing was measured as the difference between the total load measured by the two hydraulic actuators load cells and the combined load measured by the load cells connected to the 2.8D (Center) and 1.8D (West) piles. This approach for measuring the 3D (East) pile load was acceptable due to the close fit between
the total load measured by the two hydraulic actuators load cells and the total load measured by
the load cells connected to the three piles on the first day of group testing.

Figure 4-1: Pinned connection with load cell in the group test.

4.2 String Potentiometers

String potentiometers were used to measure movement and rotation of the piles,
movement of the soil between the test piles and the wall, and movement of the top of the wall.
Placement of some of the string potentiometers in the group test is shown in Figure 4-2. The steel
reference frame was supported on concrete blocks far from the test piles to avoid being affected
by movement of the soil, as shown in Figure 4-3.
Figure 4-2: Reference frame and string potentiometers.

Some of the string potentiometers

Reference Frame

Figure 4-3: Reference frame and supports.

Reference Frame

Reference Frame Supports
4.3 Strain Gauges

Strain gauges adhered to the soil reinforcements provided measurements of micro strain in the reinforcements. The 18-foot long strips were instrumented at 0.5, 2.25, 4.25, 6.25, 9.25, 12.25, and 15.25 feet from the face of the wall on both the top and bottom side. These distances were adjusted by no more than three inches to accommodate the ribbing in the strips when necessary. In preparation for instrumentation, the strips were sanded to a near mirror-finish. The gauges were then adhered to the strips with epoxy and covered with a layer of waterproofing silicone. The wires were wrapped in electrical tape to protect against damage during transportation and placement of the strips and compaction of the backfill. Figure 4-4 shows a reinforcement strip during instrumentation, and Figure 4-5 and Figure 4-6 show finished strips before and after installation on the wall, respectively. PVC piping was used against the face of the wall to protect the wires as they ran vertically up to the surface.

Figure 4-4: Instrumentation of a soil reinforcement strip.
Figure 4-5: Completed instrumented soil reinforcement strip.

Figure 4-6: Installed instrumented soil reinforcement strips.
Reinforcement strips were instrumented at depths of 1.25, 3.75, and 6.25 feet below the top of the wall. Section 5.2 provides additional details regarding the layout of the instrumented soil reinforcement, including the transverse distance between the reinforcement strips and the test piles.

4.4 Digital Imagery Correlation

Digital imagery correlation (DIC) instrumentation was used during the grouped pile tests to monitor the movement of the MSE wall. As seen in Figure 4-7, two cameras were positioned below the wall at a fixed distance apart from each other. Simultaneous photos were taken by each camera before the start of testing and at the peak load and three minutes into the hold period for each push increment of the piles. A high contrast array was painted on the wall surface to aid software in identifying identical points in the images across time steps and between the two cameras. DIC software interpreted the three-dimensional movement of wall using these images, which could then be represented in a heat map overlaying the image of the wall to give a graphic representation of the wall panel deflection induced by the lateral pile movement.

Figure 4-7: Digital imagery correlation instrumentation setup.
5 LATERAL LOAD TESTING

Testing took place in three stages. First, the single pile at four pile diameters from the wall was pushed at 0.25-inch increments to a total deflection of 2.5 inches and then unloaded. Then, the grouped piles were tested over the course of two days. The grouped piles were incrementally pushed to an average deflection of 1.24 inches on the first day, and up to an average deflection of 2.38 inches on the second day.

In both the single and grouped pile tests, the pile was pushed in incremental steps, separated by hold periods lasting approximately 5 minutes. The hold periods allowed the piles to settle slightly into static equilibrium with a stable displacement and load for consistent measurement purposes. Each hold period resulted in an average 7% loss of the peak pile head load and a gradual increase in pile head displacement of about 0.02 inches. In keeping with previous studies, measurements were compared between the moment of peak pile head load as well as one and three minutes after the peak pile head load, or in other words one and three minutes into the hold period. As pile head loads and displacements at the 3-minute hold were very similar to that at the 1-minute hold, data from the 1-minute hold were used as representative of each push interval.
5.1 Load Displacement Curves

Figure 5-1, Figure 5-2, and Figure 5-3 show the pile head load versus displacement curves for the group and single pile tests. As testing took place over two days for the grouped piles, the last three data points for these curves were obtained from the second day of testing. Displacements were taken from measurements recorded from string potentiometers placed on the piles at the same elevation as the load point. Pile head load data was typically measured from the load cells for each pile and corroborated with the total load measured by the two hydraulic actuator load cells. However, due to a malfunction in the load cell for the 3D (East) pile on the second day of testing the load for this pile was calculated as the difference between the loads measured by the two hydraulic actuators and the 2.8D (Center) and 1.8D (West) piles. As stated previously, the 1-minute hold data points will be used for all further purposes.

Figure 5-1: Pile head load versus pile head displacement at the peak load.
In a general comparison between the 1-minute hold curves, the 4D single pile and 2.8D (Center) pile initially increase at a near equal rate, but the 2.8D (Center) pile curve drops below the 4D single pile curve at just under 0.25 inches deflection, and is about 86% of the 4D single pile load near the end of testing with equal displacements. The 3D (East) pile and 1.8D (West)
pile curves are significantly lower than either the 4D single pile or the 2.8D (Center) pile. At the maximum displacement—about 2.4 inches—the 3D (East) pile and 1.8D (West) pile carried about 59% and 50% of the load from the 4D single pile, respectively. Both the east and west pile curves are nearly equal to each other until the last two displacement increments—or at about 2 inches of pile head displacement—after which the 1.8D (West) pile experienced a sudden loss in lateral resistance. This loss in lateral resistance occurred shortly after the peak load was reached, resulting in a greater than average loss in pile head load as well as a larger than typical increase in pile head deflection between the peak load point and the 1-minute hold. During this event, the 1.8D (West) pile experienced a 14% decrease in load and an increase in displacement of 0.22 inches within the space of 1 second. This suggests the occurrence of a partial failure in the soil-structure system around the 1.8D (West) pile at a load of 47.0 kips and 1.93 inches of displacement.

As noted previously, only one other full-scale test has been performed for grouped piles near an MSE wall. In that study by Pierson et al. (2009), each of the three grouped shafts were placed at 2 shaft-diameters from the wall. In both studies the center pile had a higher lateral resistance than either outside pile. However, the difference between the center and outside pile curves in this test is significantly larger than in the study by Pierson et al. (2009). It is noted that lateral grouped pile resistance tests that do not involve walls or soil reinforcement have often shown little variation of piles within a row (Walsh, 2005; Rollins et al., 2006), or with the outside piles exhibiting larger lateral resistances than piles in the center of a row (Rollins, 2005).

An unusual observation from this study was the near equal lateral resistance of the 3D (East) pile and the 1.8D (West) pile for the majority of testing, despite the east pile being spaced significantly further away from the wall than the west pile. In previous single pile tests, piles
placed about 3 pile diameters from the wall have generally reacted more stifferly compared to those placed about 2 pile diameters from the wall (Rollins et al., 2016). However, in one previous study on piles pushed individually, a pile at 2.1 pile diameters had a greater lateral resistance than a pile at 3.1 pile diameters (Luna, 2016). This observation was attributed by Luna (2016) to inconsistent compaction practices—which may also be the case in this study. Another possible contributing factor for this study was the proximity of the 3D (East) pile to the previously tested 4D single pile. As will be discussed later, well-defined shear planes developed during the 4D single pile test, shown in Figure 5-19, part of which extended in front of the 3D (East) pile. It may be that the 3D (East) pile would have had a stiffer lateral response if the 4D single pile test had not been previously performed. More full-scale testing of grouped piles is needed to make a determination whether the load-deflection data from this test is typical, especially concerning the relatively low stiffness of the 3D (East) pile.

5.2 Soil Reinforcement Performance

Strain gauges bonded to the soil reinforcements provided measurements of microstrain in the reinforcements. As noted previously, the 18-foot long strips were instrumented at 0.5, 2.25, 4.25, 6.25, 9.25, 12.25, and 15.25 feet from the face of the wall on both the top and bottom side. These distances were adjusted by no more than three inches to accommodate the ribbing in the strips when necessary. Reinforcement strips were instrumented at depths of 1.25, 3.75, and 6.25 feet below the top of the wall. The location and naming of the instrumented reinforcement strips is shown in Figure 5-4. The strips were instrumented at four sections, A through D. Section A was instrumented only at the 6.25-foot depth, but sections B, C, and D were instrumented at all three depths. The reinforcement strips between Sections A and B and between C and D were not
instrumented. Table 5-1 presents the distance from the centerline of the reinforcing strips to the center of each pile.

Induced load in the soil reinforcements from this study were calculated using the following equation:

\[ T = EA(\mu \varepsilon \times 10^{-6}) \], \hspace{1cm} (5-1)

where \( T \) = the induced tensile force in the strip at the location of the strain gauge,

\( E \) = the modulus of elasticity of the strip, or 29,000 ksi,

\( A \) = the area of the strip, or 0.31 in\(^2\), and

\( \mu \varepsilon \) = the microstrain as measured by the strain gauge.

The average induced load between the top and bottom of the reinforcement for each location was used when possible, to account for any bending of the strips. However, many of the strain gauges were damaged during installation. Table 5-2 presents which strain gauges were functioning at the time of testing. Locations with one or two working strain gauge are presented in the results, but some locations had no working strain gauges and are left blank. Strain gauge data was only omitted when the instrumentation was clearly damaged, so some outlying data may be caused by unknown errors in the instrumentation or by bending in the reinforcement that could not be accounted for when only one side of the reinforcement had a working strain gauge.
Figure 5-4: Location and naming of instrumented soil reinforcing strips.

Table 5-1: Reinforcement Strip to Pile Distance

<table>
<thead>
<tr>
<th>Strip Section</th>
<th>4D Single</th>
<th>3D (East)</th>
<th>2.8D (Center)</th>
<th>1.8D (West)</th>
</tr>
</thead>
<tbody>
<tr>
<td>A</td>
<td>51.7</td>
<td>9.8</td>
<td>68.9</td>
<td>128.2</td>
</tr>
<tr>
<td>B</td>
<td>108.2</td>
<td>46.7</td>
<td>12.1</td>
<td>71.4</td>
</tr>
<tr>
<td>C</td>
<td>NA</td>
<td>80.8</td>
<td>22.0</td>
<td>36.6</td>
</tr>
<tr>
<td>D</td>
<td>NA</td>
<td>138.1</td>
<td>79.3</td>
<td>20.2</td>
</tr>
</tbody>
</table>
Table 5-2: Functioning Strain Gauge Locations.

<table>
<thead>
<tr>
<th>Strip Section</th>
<th>Strip Depth (ft)</th>
<th>Functioning Strain Gauge Distances from Back Face of MSE Wall (ft)</th>
</tr>
</thead>
<tbody>
<tr>
<td>A</td>
<td>6.25</td>
<td>0.50*, 2.25, 4.17, 6.25, 9.25, 15.25</td>
</tr>
<tr>
<td>B</td>
<td>1.25</td>
<td>0.50, 2.25*, 6.25, 9.25, 15.25</td>
</tr>
<tr>
<td></td>
<td>3.75</td>
<td>0.50, 2.25*, 6.38, 12.25, 15.25</td>
</tr>
<tr>
<td></td>
<td>6.25</td>
<td>0.50*, 2.25, 4.17*, 6.25, 9.25*, 12.25*, 15.25</td>
</tr>
<tr>
<td>C</td>
<td>1.25</td>
<td>0.50, 2.25, 4.33, 6.25*, 9.25, 12.25, 15.17</td>
</tr>
<tr>
<td></td>
<td>3.75</td>
<td>2.25, 4.25*, 6.33, 9.25, 12.25, 15.25</td>
</tr>
<tr>
<td></td>
<td>6.25</td>
<td>0.50, 2.25, 4.17, 6.25, 9.25, 12.25, 15.25</td>
</tr>
<tr>
<td>D</td>
<td>1.25</td>
<td>0.50*, 2.25, 4.25, 6.33, 9.33, 12.25, 15.25</td>
</tr>
<tr>
<td></td>
<td>3.75</td>
<td>0.50, 2.25, 4.17, 6.25, 9.25, 12.25, 15.25</td>
</tr>
<tr>
<td></td>
<td>6.25</td>
<td>0.50, 2.25, 4.13, 9.25, 12.25, 15.25</td>
</tr>
</tbody>
</table>

*These locations functioned on the first day of testing only. Red locations have functioning gauges on both the top and bottom, while black locations have one functioning side.

Figure 5-5 and Figure 5-6 show the induced load in the instrumented soil reinforcements measured during the 4D single pile test. Data were measured at the 1-minute hold for each push increment and are labeled with the pile head load for each increment. The pile-to-wall spacing of the 4D single pile is shown in the plots for reference. The general trend of these plots is similar to that from previous studies—with the greatest induced load occurring near the loaded piles—though the maximum observed induced force tended to occur somewhat closer to the MSE wall than was generally seen during Phase 1 and 2 testing at this site. It should also be noted that for both reinforcements the tensile force is significant near the wall face, indicating that pressure on the MSE wall panel is generating force on the wall rather than just friction on the reinforcement.

The measured induced loads for the reinforcement at section B is unique compared to previous studies in that the reinforcement is transversely spaced much further from the pile than is typically measured, at 108 inches or 8.5 pile diameters. Even at this large transverse spacing, the induced forces in the section B reinforcement strip are significantly large.
Figure 5-5: Section A reinforcement at 6.25-foot depth during 4D single pile test.

Figure 5-6: Section B reinforcement at 3.75-foot depth during 4D single pile test.
Figure 5-7, Figure 5-8, Figure 5-9, and Figure 5-10 show the induced load in the soil reinforcements during the group test at various depths for sections A, B, C, and D, respectively. See Appendix X for the remaining instrumented strip data. Data were measured at the 1-minute hold for each load increment and are labeled with the average pile head load for each increment. The pile-to-wall spacing of the pile or piles on either side of the reinforcing strips are shown in the plots for reference. As testing took place over two days, the last three curves in each figure are taken from the second day data and are differentiated with dashed rather than solid lines. In general, induced reinforcement loads are greatest near the location of the closest pile—as in other studies—though the maximum induced force was often also observed at the wall face in the group test (Rollins et al., 2016). A notable exception to this trend was in Section D at the 1.25-foot depth—Figure 5-10—where the peak load was 4 to 5 pile diameters further from the wall than the closest pile.

![Graph showing induced load in soil reinforcements during group test.](image)

**Figure 5-7**: Section A reinforcement strip at 6.25-foot depth during group test.
Figure 5-8: Section B reinforcement strip at 3.75-foot depth during group test.

Figure 5-9: Section C reinforcement strip at 6.25-foot depth during group test.
Figure 5-10: Section D reinforcement strip at 1.25-foot depth during group test.

Figure 5-11, Figure 5-12, Figure 5-13, and Figure 5-14 show the maximum tensile load induced on the reinforcement strips for each loading increment as a function of strip depth. Strip depth is differentiated by color, and the first and second day are differentiated by continuous and dashed lines, respectively. Axes are left constant for ease in comparison across sections. In general, reinforcements at 1.25 and 3.75-foot depths experienced similar magnitudes of maximum induced loads. Maximum induced loads tended to be smaller at the 6.25-foot depth than the other two depths closer to the ground surface. Additionally, the maximum induced loads were generally greater on the first day of testing than the second day for similar pile head loads. However, the strain gauges readings were re-zeroed on the second day of testing, so any possible residual strain from the first day of testing was neglected.
Figure 5-11: Maximum induced load in reinforcement for each pile head load increment at Section A reinforcements.

Figure 5-12: Maximum induced load in reinforcement for each pile head load increment at Section B reinforcements.
Figure 5-13: Maximum induced load in reinforcement for each pile head load increment at Section C reinforcements.

Figure 5-14: Maximum induced load in reinforcement for each pile head load increment at Section D reinforcements.
Section D consistently had lower maximum induced reinforcement loads than other sections. A possible contributing factor to this result is the relatively large distance between the test piles and these soil reinforcements, as shown in Table 5-1. Previous studies have found that increased distance from test piles tend to decrease the induced loads on soil reinforcements (Pierson et al., 2009).

Another possible contributing factor to the small induced loads at section D is the relatively close distance between its nearest test pile and the wall—at 1.8 pile diameters—as one other study found that decreased pile-to-wall spacing tended to decrease the magnitude of induced force on soil reinforcements (Budd, 2016). However, previous research has generally observed larger induced forces in soil reinforcements for piles closer to the wall, and the latest regression equation, Equation 5-2, predicts the same (Hatch, 2014; Besendorfer, 2015; Rollins et al., 2016).

### 5.2.1 Analysis of Maximum Loads in Soil Reinforcements

The most recent regression analysis for predicting the maximum induced tensile load in a strip reinforcement due to lateral pile deflection is given by Rollins et al. (2016). All data used in this analysis were gathered from single piles tests. The maximum induced force in a strip reinforcement is calculated by the equation:

\[
\Delta F (kips) = 10^a \left( 0.13 + 0.028P - 2.2 \times 10^{-4}P^2 - 0.01 \frac{T}{D} - 0.0021P \frac{T}{D} - 0.031 \frac{S}{D} \right) - 1, \tag{5-2}
\]
where \( \Delta F \) = the maximum induced force (kips),

\[
P = \text{the pile head load (kips)},
\]

\[
T = \text{the transverse distance from the reinforcement to the pile center},
\]

\[
D = \text{the outside pile diameter (same units as } T),\text{ and}
\]

\[
S = \text{the distance from the back of the wall to the center of the pile (same units as } D).\]

Despite problems with large transverse distances and pile head loads as discussed in Section 2.3.5, Equation 5-2 predicted the induced reinforcement forces reasonably well when induced forces from the three piles were superimposed. In this superimposition, negative predicted induced forces were omitted from the total. Figure 5-15 shows the maximum measured induced forces in the soil reinforcements for each load increment compared with the predicted values. Data along the solid red line indicates that the measured and predicted forces are equal. Dotted lines denoting plus and minus one and two standard deviations were taken from the statistical analysis that produced the regression equation (Rollins et al., 2016). The percentage of data points from this study within the one and two standard deviation boundaries are 63% and 85%, respectively. The superimposed Equation 5-2 tended to overestimate induced forces for small pile head loads and underestimated induced forces for large pile head loads. However, as was mentioned, the pile head loads near the end of testing were likely large enough to cause yielding in the piles—which loads may be beyond the scope of Equation 5-2.
Figure 5-15: Measured versus predicted maximum tension in grouped pile soil reinforcements.

5.3 Ground Displacement and Cracking

Lateral loads on the test piles caused displacement in the ground between the piles and the wall, both vertically and horizontally. Vertical measurements were made with an automatic level before testing and during the final push increment during testing. A comparison of the vertical soil displacement between each test pile and the wall is shown in Figure 5-16.
Figure 5-16: Vertical ground displacement versus distance from the wall for each test pile.

Vertical ground displacements between the grouped test piles and the MSE wall follow the same general trend as in previous experiments, with the greatest displacement at the pile face. Heave at the pile face was typically between 1.5 and 2 inches, while heave at the MSE wall was typically less than 0.5 inch. There was not a clear connection between the distance between the pile and the wall and the magnitude of vertical displacements. An unexplained outlier in the data was the vertical ground displacement at the face of the 4D single pile, which was recorded as just over a tenth of an inch, much lower than expected. Possible reasons for the apparent small vertical ground displacement could be an error in measurement—such as by the level rod being held off from vertical—or an error in recording of the measurement. Another anomaly was the half-inch decrease in soil height at the face of the wall in front of the 1.8D west pile, although this may be due to rotation of the wall panel.

Horizontal ground displacements were measured using string potentiometers connecting an independent reference frame and the wall or stakes in the ground at known distances directly
between the test pile and the wall, as shown in Figure 5-17. Measurements were taken continuously throughout the tests, and results were selected at the 1-minute hold after each incremental push of the test piles. Due to the low clearance between the frame joining the grouped piles and the ground, horizontal displacements of the ground between the grouped piles and the wall were not measured. Thus, only the horizontal displacement of the soil for the 4D single pile is included here as shown in Figure 5-18.

The horizontal ground displacements follow the same general trend as in previous experiments, with a near linear decrease in displacement with increased distance from the test pile (Rollins et al., 2016). The dip in the trend line at a distance of about one foot from the pile base may likely be due to backward rotation of the survey stake as the soil near the surface fails
under lateral loading. Figure 5-17 shows the survey stake closest to the pile rotated toward the pile during the final load increment of that test.

**Figure 5-18: Horizontal ground displacement versus distance from the wall for the 4D single pile at various pile head loads.**

Extensive cracking occurred during the 4D single pile test as shear planes fanned out from the sides of the pile. Figure 5-19 shows the location of the visible cracks, spray painted orange for ease in viewing. Reese et al. (1974) suggest that fan angle, $\alpha$, as seen in Figure 6-1, is approximately equal to the friction angle of the soil for denser sands. The fan angle for this test was approximately 59º, which is considerably higher than the friction angle for which the MSE wall was designed (34º). Due to the large amount of disturbance to the soil near the 3D (East) pile, the 4D single pile test may have contributed to the relatively small lateral resistance observed from the 3D (East) pile during the grouped pile testing, as noted previously.
Several tension cracks formed in the backfill during testing of the grouped piles. As Figure 5-20 shows, tension cracks tended to form in three locations: between piles, from the front of the pile directly towards the wall, and from the side of the pile between about 25–45 degrees off of parallel from the wall face. Photos of the cracks are shown in Figure 5-21, Figure 5-22, and Figure 5-23. An additional crack formed above the end of the soil reinforcement, as shown in Figure 5-24. This crack is similar to that observed during grouped shaft testing in the study by Pierson et al. (2009) and suggests that the reinforced backfill may have mobilized together as a block.
Figure 5-20: Locations of major visible soil cracking during testing.

Figure 5-21: Tension cracks from the 3D (East) and 2.8D (Center) piles.
Figure 5-22: Tension crack between piles.

Figure 5-23: Tension cracks from the 1.8D (West) pile.
Wall Panel Displacement

Figure 5-25 shows the location of a vertical crack that developed down the center of a wall panel directly in front of the center grouped pile on the second day of testing. The crack spanned the entire height of the panel but was relatively thin. No visible bulging of the wall was evident on either side of the crack.
Digital imagery correlation instrumentation was used by Brigham Young University lab technicians to monitor the wall panel displacement during the grouped pile testing. A heat map of the wall movement during the last push increment on the first day of testing is shown in Figure 5-26. The average pile head displacement and load for this push increment was 1.24-inches and 42.2 kip. The red circles at the top of the wall indicate the positions of string potentiometers attached to the wall panels. Comparisons between the wall displacement as measured by the DIC instrumentation and the string potentiometers are given in Figure 5-27, Figure 5-28, Figure 5-29, and Figure 5-30. The DIC data was generally somewhat higher than
that from the string potentiometers, though the largest discrepancies between the two measuring devices were a little over 0.15-inches in both the positive and negative direction. The largest panel displacement anywhere on the wall during the first day of grouped pile testing was 0.31-inch, as measured by the DIC equipment and 0.11-inch at the top of the wall, as measured by the string potentiometers. These maximum wall displacements are considerably smaller than the maximum pile head deflection of 1.26-inches at the end of the first day. However, the maximum DIC-based panel deflection of 0.31 inch is much higher than the average maximum deflection of 0.10 inch measured in previous single pile testing at similar pile deflections. This result suggests that grouped pile lateral loading increases the MSE wall panel deflection relative to single pile lateral loading.

Large panel displacements were generally confined to the region between the grouped piles. The largest magnitude displacements were observed on the west side of the grouping where the piles were spaced closest to the wall. This is similar to previous research, where decreased pile-to-wall spacing correlated with increased wall displacement (Pierson et al., 2009; Price, 2012; Nelson, 2013; Hatch, 2014; Luna, 2016). Additionally, significant panel displacements on the west side of the grouping were confined to a higher elevation compared to the east side. This is also similar to previous research, where increased pile-to-wall spacing correlated to increased distance from the top of the wall to the maximum panel displacement (Budd, 2016).
Figure 5-26: Wall panel displacement heat map.

Figure 5-27: Comparison between DIC and string potentiometer measured wall displacements at the west panel location.
Figure 5-28: Comparison between DIC and string potentiometer measured wall displacements at the mid-west panel location.

Figure 5-29: Comparison between DIC and string potentiometer measured wall displacements at the mid-east panel location.
5.5 Pile Head Rotation

The rotation of the pile heads was measured by the use of two string potentiometers for each pile. One string potentiometer measured pile movement at the point of loading and the other measured approximately three feet above the point of loading. Figure 5-31 shows the typical configuration of instrumentation for a pile to measure rotation. The following equation was used in calculating the rotation of the test piles, with measurements taken as shown in Figure 5-32:

\[
\theta = \sin^{-1} \left( \frac{\Delta x_2 - \Delta x_1}{h} \right),
\]  

(5-3)

where \( \theta \) = the pile head rotation measured from vertical,

\( \Delta x_2 = \) the change in measured string potentiometer value at distance \( h \) above load point,

\( \Delta x_1 = \) the change in measured string potentiometer value at load point, and

\( h = \) the distance between the upper and lower string potentiometers.
Figure 5-31: Instrumentation set up for pile rotation.

Figure 5-32: Measurements taken for pile rotation.
Figure 5-33 shows a plot of the pile head rotation versus pile head load for each of the grouped piles and the single pile. As in the pile head displacement versus pile head load curves, the 4D single pile exhibited the stiffest response, followed by the 2.8D (Center) pile. Likewise, the 3D (East) pile again slightly trailed behind the 1.8D (West) pile in stiffness until the final three displacement increments, where it maintained its stiffness while the 1.8D (West) pile experienced a sudden loss in stiffness. The 1.8D (West) pile finished with the lowest rotational stiffness.

Figure 5-33: Pile head load versus pile head rotation at the 1-minute hold.
6 LATERAL PILE LOAD ANALYSES

The computer program LPILE (v2019) was used to perform the lateral pile load analyses in this study. LPILE is the commercial version of the computer program COM624 which was originally developed by Reese and Matlock at the University of Texas in the 1970s and is one of the most widely used programs for the lateral pile load analysis. LPILE uses the finite difference method to iteratively solve for the deflection, shear force, and bending moment of the pile with depth by modeling the pile as a beam column. The analysis of the laterally loaded pile by the finite difference method has been researched extensively by Reese and Matlock since the 1960s.

The granular backfill and native soil layers were modeled using the p-y curve shape for sand developed by the API (1982). In this approach, p is the horizontal soil resistance per length of pile and y is the lateral soil displacement. According to API, the p-y curve is given by the equation:

\[ p = A P_u \tanh \left[ \frac{(k_x)}{(A P_u)} y \right], \tag{6-1} \]

where \( p = \) the horizontal soil resistance per length of pile,

\( y = \) the lateral soil displacement,

\( A = 3.0 - 0.8(z/b) > 0.9 \) for static loading,

\( A = 0.9 \) for cyclic loading,
The ultimate lateral resistance for sand has been found to vary from a wedge type failure at the shallow depths determined by Equation 6-2 and a flow-around type failure at greater depths defined by Equation 6-3. The equation giving the smallest value of \( P_u \) should be used as the ultimate resistance. The typical wedge type failure shape is illustrated in Figure 6-1. The angle \( \beta \) is typically assumed to be \( 45^\circ + \phi/2 \) while the fan angle, \( \alpha \), is thought to be between \( \phi/2 \) and \( \phi \) for dense sand and about \( \phi/2 \) for loose sand. The two equations for calculating the ultimate resistance are given as:

\[
P_{us} = (C_1 x + C_2 b)\gamma' x \quad (6-2)
\]

\[
P_{ud} = C_3 b\gamma' x, \quad (6-3)
\]

where \( P_u \) = the ultimate resistance (force/unit length), (s=shallow, d=depth),

\( \gamma' \) = the effective soil unit weight (lb/ft\(^3\)),

\( x \) = the depth (inch),

\( \phi' \) = the angle of internal friction of sand (degrees),

\( C_1, C_2, C_3 \) = coefficients determined from Figure 6-2 as a function of friction angle, and

\( b \) = the average pile diameter from surface to depth (inch).
Figure 6-1: Illustration of wedge type failure adjacent to piles at shallow depths during lateral loading.

Figure 6-2: Coefficients $C_1$, $C_2$, and $C_3$ as functions of friction angle.
To develop the p-y curves, the model requires the user to provide data regarding the soil friction angle, $\phi$; the lateral soil stiffness, $k$; and moist unit weight. The unit weight of the soil used was measured from the density tests performed during construction; however, direct measurements of $\phi$ and $k$ were not available. API suggest that $\phi$ and $k$ can be estimated using the curves shown in Figure 6-3, but experience indicates that there can be significant variations in these values in actual practice (Brown et al 1988, Rollins et al 2005, Rollins et al 2011). Since relatively few lateral load tests have been performed on piles in compacted gravelly sands such as the backfill material used in this research, the soil strength and stiffness parameters are poorly calibrated. Thus, the friction angle, $\phi$; and stiffness, $k$; were back-calculated from LPILE by matching the calculated load-deflection curves with those obtained from the field tests.

![Figure 6-3: Subgrade Reaction Modulus, k Used for API Sand Criteria in p-y Analysis (API, 1982).](image)
Both friction angle and soil stiffness have an effect on the computed load-displacement curves; however, $k$ has a greater effect on the curve at small deflections, while $\phi$ has a greater effect on the curve at large deflection near the ultimate resistance. Altering the soil stiffness for example, can have significant effects on the shape of the load-deflection curve of the pile.

### 6.1 Material Properties

As previous tests have shown that lateral pile resistance does not usually increase beyond a distance of four pile diameters from the wall, the 4D single pile was assumed to be suitable to calibrate the modeled fill-material in LPILE with a $p$-multiplier of 1.0 (Rollins et al., 2016). In keeping with previous studies, the API Sand (1982) model was used in this analysis, which generates $p$-$y$ curves of the soil based on unit weight, friction angle, and subgrade modulus. The soil properties at depths greater than 6.25 feet from the top of the wall were set to match those from previous investigations, as this soil was left undisturbed from previous studies (Han, 2014).

The effective unit weight for the top 6.25 feet was inputted as the average moist unit weight of the backfill between the test piles and the wall as measured by the density gauge. The friction angle and subgrade modulus for the top 6.25 feet of soil were selected with a trial-and-error method so that the computed pile head deflection versus load curve most closely matched the measured curve. Table 6-1 presents the backfill properties used in LPILE for all analyses, and Table 6-2 shows the properties of the pile used in the model.
Table 6-1: Backfill Properties in LPILE.

<table>
<thead>
<tr>
<th>Depth</th>
<th>p-y Curve Type</th>
<th>Effective Unit Weight</th>
<th>Friction Angle</th>
<th>Subgrade Modulus</th>
</tr>
</thead>
<tbody>
<tr>
<td>1–7.25 ft</td>
<td>API Sand (O’Neill)</td>
<td>130 pcf</td>
<td>60°</td>
<td>550 pci</td>
</tr>
<tr>
<td>7.25–21 ft</td>
<td>API Sand (O’Neill)</td>
<td>129 pcf</td>
<td>39°</td>
<td>225 pci</td>
</tr>
<tr>
<td>21–50 ft</td>
<td>API Sand (O’Neill)</td>
<td>125 pcf</td>
<td>34°</td>
<td>100 pci</td>
</tr>
</tbody>
</table>

Table 6-2: Pile Properties in LPILE

<table>
<thead>
<tr>
<th>Section Type</th>
<th>Length</th>
<th>Outside Diameter</th>
<th>Wall Thickness</th>
<th>Yield Stress</th>
<th>Elastic Modulus</th>
<th>Moment of Inertia</th>
</tr>
</thead>
<tbody>
<tr>
<td>Steel Pipe Section</td>
<td>40 ft</td>
<td>12.75 in</td>
<td>0.375 in</td>
<td>57 ksi</td>
<td>29000 ksi</td>
<td>279.3 in(^4)</td>
</tr>
</tbody>
</table>

As discussed previously, the fan angle, \( \alpha \), is thought to be between \( \phi/2 \) and \( \phi \) for dense sand, as in Figure 6-1. The fan angle about the 4D single pile was measured to be about 59°—very close to the 60° friction angle used in LPILE for the top backfill layer. Also, although the effective unit weights for the top two layers were input as near identical, it must be remembered that the backfill used in the second layer from the top was somewhat different than the top layer—with a proctor dry density of 128.0 pcf compared to 126.7 pcf for the top layer. Thus, the difference in relative density between the two layers is somewhat more pronounced than it appears. In addition, the 129 pcf effective unit weight reported by Han (2014) may likely be the average across the length of the soil reinforcements, which would be artificially higher than the backfill only between the test piles and the wall.
6.2 LPILE Analysis Results

The p-y curve from the LPILE model calibrated to the 4D single pile was adjusted with a constant p-multiplier along the length of the pile back-calculated in an iterative process for each of the grouped piles so that the computed load-deflection curve most closely matched the measured curve. P-multipliers have been shown to accurately model the loss of lateral soil resistance for piles at various distances from MSE walls as well as for grouped pile (Rollins, 2006; Rollins et al., 2016). Computed pile head rotations using these p-multipliers were also compared with the measured data.

6.2.1 Load–Deflection Curves

Figure 6-4 shows the pile head load versus displacement curve computed by LPILE in comparison with the measured curve for the single pile at 4D spacing with a p-multiplier of 1.0. Generally, the two curves are in reasonable agreement, although the computed curve is somewhat high for the displacement range from 1 to 2 inches. For each pile in the group, the soil properties calibrated for the single pile test were then kept the same while a constant p-multiplier was back-calculated to produce agreement with the measured curve. Figure 6-5, Figure 6-6, and Figure 6-7 show the pile head displacement versus pile head load curves as computed by LPILE with appropriate p-multipliers to most closely match the measured curves for the west, center, and east piles in the group, respectively. As the 1.8D (West) pile experienced a loss of lateral resistance near the end of testing, two p-multiplier curves are included in Figure 6-7 to bound the beginning and ending behaviors; however, the p-multiplier associated with the initial stiffness of the pile will be used for all further purposes. Figure 6-8 displays all the LPILE curves and measured points in one plot for comparison between the test piles. Relatively close agreement was achieved for each measured load-displacement curve.
Figure 6-4: Comparison of the pile head load versus pile head displacement for the 4D single-pile test with curve computed by LPILE.

Figure 6-5: Comparison of the pile head load versus pile head displacement for the 3D east-pile test with curve computed by LPILE.
Figure 6-6: Comparison of the pile head load versus pile head displacement for the 2.8D center-pile test with curve computed by LPILE.

Figure 6-7: Comparison of the pile head load versus pile head displacement for the 1.8D west-pile test with curve computed by LPILE.
Figure 6-8: Comparison of the measured pile head load versus pile head displacement for all test piles with curve computed by LPile.

6.2.2 P-Multipliers and Pile Spacing Curves

Figure 6-9 compares the p-multipliers for each grouped pile and the group average with the single pile tests from previous studies. The most recent regression equation for p-multipliers based on relative distance from the wall is also plotted, Equation 2-1 (Rollins et al., 2016). Expected p-multipliers from this regression equation are 0.72, 0.66, and 0.34 for the 3D (East), 2.8D (Center), and 1.8D (West) piles, respectively. Actual, back-calculated p-multipliers for these piles were 0.25, 0.60, and 0.25. The actual p-multipliers for each grouped pile is below the regression line, though only the 3D (East) pile p-multiplier falls significantly lower than is typically observed in previous tests.
Figure 6-9: P-multipliers versus normalized pile-to-wall distance.

Figure 6-10, Figure 6-11, and Figure 6-12 show LPILE computed pile head load versus displacement curves using the p-multipliers expected for the three grouped piles using Equation 2-1, compared with the measured data. I propose that in addition to the loss of lateral resistance due to the proximity of the wall, the piles in this test experienced an additional loss of lateral resistance due to group effects, which would account for the discrepancy between the LPILE computed curves using the expected p-multipliers and the measured data. If the pile-to-wall spacing p-multiplier for each pile is assumed to be that calculated by Equation 2-1, then the group effect p-multipliers are 0.35, 0.91, and 0.74 for the 3D, 2.8D, and 1.8D piles, respectively, to satisfy the equation:

\[(P_m)_{\text{total}} = (P_m)_{\text{group}} * (P_m)_{\text{wall}}\]  \hspace{1cm} (6-1)

where \((P_m)_{\text{total}}\) = the total p-multiplier for a grouped pile near an MSE wall,
(P_m)_{\text{group}} = \text{the group effect p-multiplier, and}

(P_m)_{\text{wall}} = \text{the pile-to-wall spacing p-multiplier.}

Figure 6-10: Expected single pile p-multiplier LPILE curve versus observed group 3D (East) pile.

Figure 6-11: Expected single pile p-multiplier LPILE curve versus observed group 2.8D (Center) pile.
The average group effect \( p \)-multiplier from this pile group is 0.66. If one were to extrapolate this finding to a grouped pile design, then the \( p \)-multiplier expected for a single pile at their chosen normalized distance from the wall should be multiplied by 0.66 to account for group effects. However, this average group effect \( p \)-multiplier may not be accurate or valid for all pile designs. There are compelling reasons that this value could be too low for most pile designs. For instance, if piles on the inside of a pile group indeed exhibit significantly stiffer lateral responses than piles on an outside edge, then the average \( p \)-multiplier would increase for each additional pile in a group. Full scale studies involving groups of four or more grouped piles are needed to validate this possibility. Additionally, the abnormally low lateral resistance of the 3D (East) pile in this study decreased the average group effect \( p \)-multiplier from where it may have otherwise been if the resistance were somewhat greater for this pile. Had the group effect \( p \)-multiplier for the 3D (East) pile been the same as the 1.8D (West) pile, at 0.74, then the average group effect for the piles would have been 0.79. Again, more testing would be needed to confirm that the 3D (East) pile lateral response was significantly low.

Figure 6-12: Expected single pile \( p \)-multiplier LPILE curve versus observed group 1.8D (West) pile.
6.2.3 Pile Head Load versus Rotation Curves

The expected pile head rotation for the single pile and grouped piles were also computed in LPILE using the back-calculated p-multiplier associated with each pile. Graphs of the observed and calculated pile head rotation versus pile head load for each pile are found in Figure 6-13, Figure 6-14, Figure 6-15, Figure 6-16 for the single pile as well as the east, center, and west piles in the group, respectively. Figure 6-17 shows the curves for each of the four piles in one plot for comparison. Observed rotation values are shown by points and the calculated rotation values are shown by solid lines. Relatively close agreement is had between the observed and calculated rotation values for the 4D single pile. However, calculated rotation values for the group piles tended to overpredict the amount of rotation for a given load compared to that observed. It is currently unclear whether LPILE consistently overpredicts grouped pile head rotation or if some other factor caused the overprediction.

Figure 6-13: Comparison of the pile head load versus pile head rotation for the 4D single-pile test and LPILE analysis.
Figure 6-14: Comparison of the pile head load versus pile head rotation for the 3D east-pile test and LPILE analysis.

Figure 6-15: Comparison of the pile head load versus pile head rotation for the 2.8D center-pile test and LPILE analysis.
Figure 6-16: Comparison of the pile head load versus pile head rotation for the 1.8D west-pile test and LPILE analysis.

Figure 6-17: Comparison of the pile head load versus pile head rotation for all test piles and LPILE analyses.
6.3 LPILE Analysis of Pierson et al. (2009) Data

I also performed a separate LPILE analysis on data from Pierson et al. (2009) to compare against the results from this study. Initially in this analysis, the LPILE model parameters were calibrated relative to the lateral shaft head load-deflection curve for the individually loaded Shaft D located 4.0 shaft diameters from the MSE wall. As before noted, previous testing suggests that a deep foundation at this distance behind the wall would be unaffected by the presence of the wall. Parameters used for the backfill and pile in this first analysis are included in Table 6-3 and Table 6-4, respectively. The apparent shaft length was 21.5 feet, as the shafts were loaded about 1.5 feet above the embedded length of 20 feet. No shear resistance was specified on the base of the shaft. The non-linear reinforced concrete model in LPILE was used for all the lateral load analyses in LPILE.

<table>
<thead>
<tr>
<th>Depth</th>
<th>p-y Curve Type</th>
<th>Effective Unit Weight</th>
<th>Friction Angle</th>
<th>Subgrade Modulus</th>
</tr>
</thead>
<tbody>
<tr>
<td>1.5–25 ft</td>
<td>API Sand (O'Neill)</td>
<td>130 pcf</td>
<td>31 Deg.</td>
<td>20 pci</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Section Type</th>
<th>Length</th>
<th>Diameter</th>
<th>Compressive Strength</th>
<th>Bar Size</th>
<th>Number of Bars</th>
<th>Bar Yield Stress</th>
</tr>
</thead>
<tbody>
<tr>
<td>Round Concrete Shaft (Bored Pile)</td>
<td>21.5 ft</td>
<td>36 in</td>
<td>6200 psi</td>
<td>#11</td>
<td>12</td>
<td>60 ksi</td>
</tr>
</tbody>
</table>
Figure 6-18 shows the measured shaft head load versus deflection curves for the individually loaded Shaft D located at 4.0 shaft diameters relative to the calibrated curve computed by LPILE. To obtain this agreement with the measured curve, a friction angle of 31° was necessary, rather than the 51° friction angle reported by Pierson et al. (2009). This result suggests that the compaction effort in the field was much less than that used in the laboratory tests for the crushed rock backfill. The lack of field compaction test results suggests that this may be the case (Prof. Robert Parsons, Personal Communication, 2017).

For analysis of the other shafts, the soil and pile properties calibrated for Shaft D were kept the same and one constant p-multiplier was back-calculated for each pile using a trial and error approach to obtain the best agreement with the measured shaft head load-deflection data. P-multipliers were applied to match the individually loaded Shaft B spaced at 2.0 shaft diameters from the wall as well as the average of three grouped shafts—BG1, BG2, and BG3—also spaced at 2.0 shaft diameters from the wall. Figure 6-18 shows the measured shaft head load versus deflection data points compared against LPILE generated curves with back-calculated p-multipliers. The p-multiplier for Shaft B represents only the reduction in soil resistance owing to the presence of the wall, while the p-multiplier for shafts BG1, BG2, and BG3 represent the reduced lateral soil resistance for both the presence of the wall and pile group interaction effects, as in Equation 6-1.
Figure 6-18: Comparison of the shaft head load versus shaft head displacement for the 4D and 2D single shafts and group shaft tests and LPILE analysis.

Although a reasonable fit was achieved between the LPILE generated curve and measured curve for Shaft D, spaced at four shaft diameters, a simple p-multiplier reduction was not adequate to match the measured data points for the single and grouped shafts spaced at two shaft diameters along the full length of the curve. P-multipliers that most closely matched the individually loaded Shaft B and the average of the grouped shafts were back-calculated to be 0.60 and 0.47, respectively. These p-multipliers achieved a close fit with the measured data for large displacements—above about 2.5 inches—but underestimated load capacity for smaller displacements. The group effect p-multiplier for the grouped shafts is calculated to be 0.78 using Equation 6-1, by dividing the total p-multiplier of 0.47 for shafts in the group by the pile-to-wall spacing p-multiplier of 0.60 for an individual shaft located at the same spacing behind the wall.
In an attempt to better fit the LPILE generated curves to the measured data, I calibrated new soil parameters to match the individually loaded Shaft B spaced at 2.0 shaft diameters. These new parameters are given in Table 6-5. In this case, the reduced friction angle of 15.1 degrees accounts for the reduction of lateral stiffness of the shaft due to proximity to the wall compared to the shaft spaced at 4.0 shaft diameters. Figure 6-19 compares the measured shaft head load versus deflection data for the single and grouped shafts spaced at 2.0 shaft diameters and matching LPILE generated curves with appropriate p-multipliers. As the calibrated soil properties account for the pile-to-wall spacing p-multiplier, the p-multiplier for the grouped shafts in Figure 6-19 represents the group effect p-multipliers directly, as in Equation 6-1. As in the Shaft D calibrated model, the group effect p-multiplier was back-calculated to be 0.78. The fact that both analysis procedures produced the same value adds credence to the accuracy of said group-effect p-multiplier and to the effectiveness of the combined p-multiplier approach used in Equation 6-1.

Table 6-5: LPILE Soil Parameters for Shaft at Two Diameters

<table>
<thead>
<tr>
<th>Depth</th>
<th>p-y Curve Type</th>
<th>Effective Unit Weight</th>
<th>Friction Angle</th>
<th>Subgrade Modulus</th>
</tr>
</thead>
<tbody>
<tr>
<td>1.5–25 ft</td>
<td>API Sand (O'Neill)</td>
<td>130 pcf</td>
<td>15.1 Deg.</td>
<td>40 pci</td>
</tr>
</tbody>
</table>
In comparison with the average group effect p-multiplier from this study—0.66—the group effect p-multiplier for the shafts in the study by Pierson et al. (2009) is somewhat higher, at 0.78. However, the average group effect p-multiplier for this study would have been 0.79 if the group effect p-multiplier for the 3D (East) pile had been the same as the 1.8D (West) pile—0.74 instead of 0.32—which is more in line with expectations based on the location of the pile behind the wall. Given this agreement, it seems likely that previous testing of the single pile at 4D adversely impacted the 3D (East) pile in the group, as discussed previously. In the absence of additional test results, group p-multipliers could be selected within a range of 0.66 to 0.78 for pile groups spaced two to three pile diameters behind an MSE wall.

More data is clearly needed before a general-use group effect p-multiplier can be recommended with confidence. Of particular importance in future studies would be testing groups of 4 or more piles to observe whether the trend continues for interior piles within a group.
to exhibit greater lateral resistance compared to exterior piles. As abutment walls generally use large numbers of piles in tandem, only a small fraction of the piles are on the outside edge of a group; if inside piles truly tend to have higher lateral resistances than outside piles, the average group effect p-multiplier in practice may be significantly greater than what was found in these studies involving only groups of three piles. Given this reasoning, a high-end value for the group p-multiplier might be justifiable at this time.
7 SUMMARY AND CONCLUSIONS

Bridge abutments that utilize MSE walls are often supported on piles placed in close proximity to the wall. The abutment connects the piles which act together as a group during lateral loading from an earthquake or thermal expansion and contraction. Full-scale lateral deflection testing was performed by employees of Brigham Young University and me on a group of three 12.75x0.375 pipe piles spaced at 1.8, 2.8, and 3.0 pile diameters from a 20-ft tall MSE wall reinforced with galvanized ribbed steel strips. Previous full-scale testing has been conducted almost exclusively on individual piles, so an additional test was performed on an adjacent individual pile with the same dimensions spaced at 4.0 pile diameters to act as a control and to calibrate the backfill parameters used in analysis with the computer program LPILE. I also performed an analysis on data from the study by Pierson et al. (2009), which included individually loaded shafts at 4.0 and 2.0 shaft diameters as well as three grouped shafts at 2.0 shaft diameters. Conclusions relative to the lateral pile resistance, induced force in soil reinforcements, and wall panel displacements are summarized in this chapter.

7.1 Conclusions Relative to Lateral Pile Resistance

- Piles on the outside edge of the group exhibited decreased lateral force resistance compared to the pile on the inside of the group.
The 3D (East), 2.8D (Center), and 1.8D (West) piles carried 59%, 86%, and 50% of the load carried by the 4D single pile, respectively, at pile head deflections of about 2.5 inches.

The seemingly low lateral pile resistance in the 3D (East) pile may have been due to the testing order, with soil cracking from the 4D single pile test fanning out in front of the 3D (East) pile, possibly reducing the soil stiffness.

Soil cracking above the end of the soil reinforcements around the grouped piles suggests that the backfill mobilized as a block during grouped pile testing. It is possible that an increase in soil reinforcement length could increase lateral grouped pile resistance.

An LPILE (v2019) analysis was performed for the piles from this study. Using soil parameters calibrated to the 4D single pile, p-multipliers of 0.25, 0.60, and 0.25 were found to match the calculated load-deflection curves to the measured curves for the 3D (East), 2.8D (Center), and 1.8D (West) piles, respectively.

If the pile-to-wall spacing p-multipliers are assumed to be 0.72, 0.66, and 0.34—calculated using Equation 2-1—then the group effect p-multipliers are calculated to be 0.35, 0.91, and 0.74 for the 3D (East), 2.8D (Center), and 1.8D (West) piles, respectively—using Equation 6-1. This gives an average group effect p-multiplier of 0.66 for the three piles.

Two LPILE analyses were performed for a group of three shafts from the study by Pierson et al. (2009), using soil parameters calibrated to either of the individually loaded shafts spaced at 4.0 or 2.0 shaft diameters from the wall. In both analyses, the
average group effect p-multiplier for the grouped shafts spaced at 2.0 shaft diameters
was calculated to be 0.78.

• Based on the analyses of the group test in this study and the group test by Pierson et al.
(2009), a range for group effect p-multipliers of 0.66 and 0.78 is proposed. The
possibly unusually low lateral resistance of the 3D (East) pile and limitations in both
these studies of testing groups of only three piles each might indicate that values near
the upper end of this range are more realistic.

7.2 Conclusions Relative to Forces Induced in Soil Reinforcements

• As has been seen in previous studies, the location in the soil reinforcement about the
same distance from the wall as the pile being pushed tended to correlate to the location
of the greatest induced force in the reinforcement.

• Locations in the soil reinforcement near the MSE wall also often exhibited large
induced loads in the grouped pile test, suggesting an increase in active soil pressure on
the wall due to the displaced piles.

• Induced load in the soil reinforcements in the top two layers—depths of 1.25 and 3.75
feet—were generally larger than in the reinforcement layer at a depth of 6.25 feet.

• By superimposing the regression equation calibrated to previous study results across
each of the three grouped piles, reasonable agreement was found between the
calculated maximum induced reinforcement loads and the measured maximum loads. It
was found that 65% and 85% of the data points from the grouped pile test fell within
the one and two standard deviation boundaries from the previous studies results,
respectively.
7.3 Conclusions Relative to MSE Wall Displacements

- Wall movements were only measured on the first day of grouped pile testing, with a maximum displacement of 0.31 inch. The average maximum pile head displacement for this day was 1.24 inches.

- The average maximum wall displacement for similar pile head loads from previous studies was 0.10 inch, much less than the 0.31 inch displacement measured in this study—suggesting that grouped pile lateral loading increases the MSE wall panel deflection relative to single pile lateral loading.

- Significant panel displacements were generally confined to the region between the grouped piles—and were largest in the top west corner of that region, near the location of the pile spaced closest to the wall.

- Local maximum panel displacements tended to occur deeper below the top of the wall around piles further from the wall.

7.4 Recommendations for Further Research

Including this study, there are currently only two full-scale tests of laterally loaded grouped piles near MSE walls. Any further research in this area would be beneficial in reducing uncertainty due to the small sample size. However, should additional testing take place, it would perhaps be most beneficial to include groups of four or more piles displaced in tandem, to better differentiate between inside and outside pile behavior. If piles on the inside of groups continue to exhibit increased lateral resistance compared to outside piles, then the group effect p-multiplier associated with a large array of grouped piles could potentially be significantly increased compared to a group of only three piles.
In further tests, in addition to an individually loaded pile spaced at least 3.9 pile diameters from the wall, I recommend that another individually loaded pile be tested at the same pile-to-wall spacing as the grouped piles as well—preferably sufficiently far from the grouped piles to not disturb the soil in front of the other test piles. Analysis would be simplified if piles in a group were spaced at the same distance from the wall as each other. It is currently unknown if group effects also affect the pile-to-wall spacing versus p-multiplier trendline from what is observed in individually loaded piles; it would be instructive to test multiple groups of piles spaced at varying distances from the wall at the same site. As soil cracking above the end of the soil reinforcement has been observed in both full-scale grouped pile tests, an analysis of the effect of reinforcement length on the lateral resistance of grouped piles may also prove useful.
REFERENCES


Han, J. (2014). “Lateral Resistance of Piles Near 15 Foot Vertical MSE Abutment Walls Reinforced with Ribbed Steel Strips” MS Thesis, Department of Civil and Environmental Engineering, Brigham Young University, Provo, UT.


APPENDIX A

Figure A-1: Section A reinforcement strip at 6.25-foot depth.

Figure A-2: Section B reinforcement strip at 1.25-foot depth.
Figure A-3: Section B reinforcement strip at 3.75-foot depth.

Figure A-4: Section B reinforcement strip at 6.25-foot depth.
**Figure A-5:** Section C reinforcement strip at 1.25-foot depth.

**Figure A-6:** Section C reinforcement strip at 3.75-foot depth.
Figure A-7: Section C reinforcement strip at 6.25-foot depth.

Figure A-8: Section D reinforcement strip at 1.25-foot depth.
Figure A-9: Section D reinforcement strip at 3.75-foot depth.

Figure A-10: Section D reinforcement strip at 6.25-foot depth.