Lateral Resistance of 24-inch Statically Loaded and 12.75-Inch Cyclically Loaded Pipe Piles Near a 20-ft Mechanically Stabilized Earth (MSE) Wall

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Lateral Resistance of 24-Inch Statically Loaded and 12.75-Inch Cyclically Loaded Pipe Piles Near a 20-ft Mechanically Stabilized Earth (MSE) Wall

Addison Joseph Wilson

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

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ABSTRACT

Lateral Resistance of 24-Inch Statically Loaded and 12.75-Inch Cyclically Loaded Pipe Piles Near a 20-ft Tall Mechanically Stabilized Earth (MSE) Wall

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Installing load bearing piles within the reinforcement zone of mechanically stabilized earth (MSE) retaining walls is common practice in the construction industry. Bridge abutments are often constructed in this manner to adapt to increasing right-of-way constraints, and must be capable of supporting horizontal loads imposed by, traffic, earthquakes, and thermal expansion and contraction. Previous researchers have concluded that lateral pile resistance is reduced when pile are placed next to MSE walls but no design codes have been established to address this issue. Full –scale testing of statically applied lateral loads to four 24”x0.5” pipe piles, and cyclically applied lateral load to four 12.75”x0.375” pipe piles placed 1.5-5.3 pile diameters behind a 20-foot MSE wall was performed. The MSE wall was constructed using 5’x10’ concrete panels and was supported with ribbed strip and welded wire steel reinforcements.

The computer software LPILE was used to back-calculate P-multipliers for the 24” piles. P-multipliers are used to indicate the amount of reduction in lateral resistance the piles experience due to their placement near the MSE wall. Previous researchers have proposed that any pile spaced 3.9 pile diameters (D) or more away from the MSE wall will have a P-multiplier of 1; meaning the pile experiences no reduction in lateral resistance due to its proximity to the wall. P-multipliers for piles spaced closer than 3.9D away from the wall decrease linearly as distance from the wall decreases. P-multipliers for the 24” piles spaced 5.1D, 4.1D, 3.0D, and 2.0D were 1, 0.84, 0.55, and 0.44 respectively. Lateral resistance of the 12.75” cyclically loaded piles decreased as the number of loading cycles increased. Lateral resistance of the piles when loads were applied in the direction of the wall was less than the lateral resistance of the piles when loads were applied away from the wall at larger pile head loads.

The maximum tensile force experienced by the soil reinforcements generally occurred near the wall side of the pile face when the lateral loads were applied in the direction of the wall. Behind the pile, the tensile force decreased as the distance from the wall increased. Equation 5-4, modified from Rollins (2018) was found to be adequate for predicting the maximum tensile force experienced by the ribbed strip reinforcements during the static loading of the 24” pipe piles, particularly for lower loads. About 65% of the measured forces measured in this study fell within the one standard deviation boundary of the proposed equation.

Keywords: laterally loaded pile, MSE wall, p-y curve, p-multiplier, cyclic piles
ACKNOWLEDGEMENTS

This study was funded through FHWA Transportation Pooled Fund TPF-F(381) Evaluation of Lateral Pile Resistance Near MSE Walls at a Dedicated Wall Site – supported by Departments of Transportation from California, Florida, Kansas, Minnesota, Montana, New York, Utah, and Wisconsin. Utah served as the lead agency with David Stevens as the project manager. This support is gratefully acknowledged; however, the opinion, conclusions, and recommendations contained herein do not necessarily represent those of the sponsoring organizations.

We are also grateful to Atlas Tube, Inc. for donating the piles; Desert Deep foundations, Inc. for providing pile driving services at cost; Geneva Rock, Inc. for providing site grading services and the test site location; and the Reinforced Earth Company for donating the steel reinforcing strips and wall panels. We would also like to express our gratitude to David Anderson and Rodney Mayo for their assistance throughout the testing process.

I would like to thank Dr. Kyle Rollins for the opportunity to assist in meaningful research while pursuing my master’s degree at BYU, and for his mentorship during my studies. I would also like to acknowledge Dr. Norman Jones and Dr. Kyle Franke for participating in my graduate committee, and for the breadth of knowledge that I have taken away from their areas of expertise. I would also like to express my appreciation for my peers, Zachary Farnsworth and Jaide Bosen for assisting me in my work.

Finally, I would like to thank my parents Dr. Brooks M. and Sarah Wilson for their support throughout and my education. Most of all, I would like to thank my wife Karlee for her unwavering loyalty and love.
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1 INTRODUCTION

Installing load bearing piles within the reinforcement zone of mechanically stabilized earth (MSE) retaining walls is common practice in the construction industry. Bridge abutments are often constructed in this manner to adapt to increasing right-of-way constraints. These pile-MSE wall systems must be capable of supporting vertical loads imposed by the weight of the bridge, and horizontal loads imposed by, traffic, earthquakes, and thermal expansion and contraction. Previous research has shown that abutment piles located close to the back of an MSE wall will develop less lateral resistance than a pile far from the wall. Designers have typically designed these systems conservatively, because current design guidance is lacking. Current methods of dealing with wall-pile interaction include: (1) installing the piles far enough back from the wall to eliminate wall-pile interaction, (2) increasing the number, or diameter of the piles assuming the wall contributes no lateral resistance, or (3) installing the piles near the MSE wall while using engineering judgement to approximate reduction factors for lateral resistance. Undesirable consequences of these methods are respectively: (1) increasing the cost of the bridge super structure by increasing the length of the bridge span, (2) increasing material cost by using more piles than may be necessary, and (3) inducing uncertainty and complicating the design process by not conforming to a specified design standard.

Pierson et al (2009), investigated the performance of laterally loaded drilled concrete shafts behind a block MSE wall. Pierson concluded that the lateral resistance of the shaft-wall
system decreased as the spacing between the wall and the shaft decreased. Full-scale testing of steel pipe piles behind metallic strip reinforced MSE wall, executed by Rollins et al (2013), bridge abutments under construction, confirmed the research performed by Pierson. Rollins’ research also established preliminary reduction factors (p-multipliers) for lateral resistance of soil around piles near MSE walls and generalized the behavior of soil reinforcements during lateral loading. As part of FHWA pooled-fund study TPF-5(272), a series of 24 lateral pile load test were performed near a dedicated 20-ft high MSE wall site in Lehi, Utah under the direction of Prof. Rollins. Reinforcements consisted of ribbed strips and welded wire grids. As part of this study, full-scale testing was performed by Han (2014) and Hatch (2014) to validate the preliminary reduction factors established by Rollins. Han and Hatch considered the effects of lateral pile loading on 12.75-inch diameter pipe piles behind an MSE wall with reinforcement lengths typical of designs for seismic loading situations. Besendorfer (2015), and Budd (2016) conducted lateral pile head loading on 12.75-inch diameter pipe piles behind an MSE wall with reinforcements lengths typical of designs tailored for static loading situations. Finally, Luna (2016) reported on lateral load tests conducted on H and square pile about 12-inches wide.

These researchers created a foundational approach for understanding and predicting the behavior of laterally loaded piles behind MSE walls. However, due to time and monetary constraints, several variables were left unexplored including: backfill soil density, cyclic loading, larger diameter piles, and fixed-head piles. This thesis summarized the lateral load testing and analysis associated with 24-inch diameter steel pipe piles subjected to monotonic static lateral loads, and 12-inch diameter pipe piles subjected to cyclic lateral loads.
1.1 Objectives

The primary objectives of this thesis are as follows:

1. Measure lateral resistance of cyclically and statically loaded pipe piles at varying installation distances from an MSE wall.

2. Measure tensile force experienced by welded wire steel reinforcements induced by cyclic lateral loads applied at the pile head, and tensile force experienced by ribbed strip steel reinforcements induced by static lateral loads also applied at the pile head.

3. Measure displacement and deformation experienced by the MSE wall during cyclic and static lateral loads.

4. Develop p-multipliers to account for reduced soil resistance as a function of proximity of the pile to the MSE wall for both the static load tests and the cyclic load tests.

5. Examine the validity of previously developed equations that aim to assist designers in predicting the maximum tensile force in the soil reinforcements during lateral pile head loading.

1.2 Scope

This report will focus on the behavior of two sets of steel pipe piles installed within the reinforcement zone of a 20-foot tall MSE wall. The first set of piles consist of four 24-inch diameter steel pipe pile, driven at nominal distances of 5.1, 4.1, 3.0, and 2.0 pile diameters behind the MSE wall reinforced by galvanized steel ribbed strips. These piles were loaded statically with a monotonic loading towards the MSE wall. The second set of piles consisted of 12.75-inch diameter steel pipe piles driven at nominal distances of 5.3, 4.2, 3.1, 1.5 pile
diameters behind the MSE wall reinforced by galvanized welded wire reinforcement panels. The piles were loaded cyclically with a sinusoidal loading toward and way from the MSE wall. The MSE wall was supported by reinforcements extending 18 feet into the backfill behind the wall. Considering the surcharge loading applied to simulate the approach fill, the effective length to height ratio (L/H) of the reinforcements was about 0.70, which is a typical length for static loadings.

The development of p-multipliers was performed using the computer program LPILE. The deflection of the pile head and the load applied to the pile head was measured during the full-scale testing. This data was then plotted as pile head deflection versus pile head load curves. The loading scenarios applied to the pile during full-scale testing were inputted as loading scenarios into LPILE. For each loading scenario, LPILE predicted the pile head deflection expected to take place. The soil parameter inputs in LPILE were adjusted until the LPILE predicted load versus deflection curves matched the load versus deflection curve plotted using measured data from the test pile located furthest away from the MSE wall. Once the measured load versus deflection curve from the pile located furthest away from the wall matched a predicted load versus deflection curve produced using LPILE, p-multipliers were applied to the predicted curve until predicted load versus deflection curve with the applied p-multipliers matched the load versus deflection curves of the remaining pile located at their respective distances behind the MSE wall.

Previous researchers in Rollins’ research group have developed equations aimed to assist designers in predicting the maximum tensile force in the soil reinforcements during lateral pile head loading. The validity of these equations was examined in this thesis. This was done by comparing the measured tensile force experienced by the soil reinforcements, at a given pile
head load, to the predicted tensile force the reinforcements were expected to experience. This predicted force was produced by the equations proposed by the researchers in Rollins’ group. A sample of 55 strain gauge readings was used in the analysis.
2 BACKGROUND

2.1 Laterally Loaded Analysis of Piles

The p-y method was used in the analysis of the laterally loaded piles for this study. The p-y method has been commonly used to predict the behavior of laterally loaded piles since it was first proposed by McClelland and Focht (1956). The p-y method models the pile-soil interaction as a nonlinear beam on an elastic foundation. The soil is modeled as a series of discrete springs placed incrementally along the length of the pile. A rendering of the p-y model is displayed in Figure 2-1.

![Figure 2-1 Rendering of the p-y model (Isenhower et al. 2019).](image)

The soil resistance immediately surrounding the pile is represented by the variable p, and is a function of the pile deflection represented by the variable y. As the pile experiences
displacement $y$, the pile is met by soil resistance $p$ in the opposite direction. A depiction of the relationship of the variables $p$ and $y$ is shown in Figure 2-2. Figure 2-2 is not realistically scaled but is exaggerated for clarification purposes.

![Figure 2-2 Distribution of soil resistance acting on the pile, (a) before lateral pile loading and (b) after pile displacement $y$ (Isenhower et al 2019).](image)

The $p$-$y$ method is programmed into the computer software LPILE. LPILE is the commercial version of the computer program COM624 which was originally developed by Reese and Matlock at the University of Texas. LPILE is a finite difference computer program that assists engineers in analyzing the effects of lateral loading on piles and drilled shafts. Some of the computational capabilities of LPILE include: pile bending moments, shear forces within the pile, pile deflection, pile rotation, pile curvature, and the behavior of adjacent soils in response to lateral loading of piles.
appropriate for the model, a dialog will appear on the screen prompting the user to insert the soil parameters needed to perform the analysis.

Some of the soil types available in LPILE include: soft clays, stiff clays with and without water, sands, liquefied sands, weak rock, and strong rock. The API Sand (O’Neill) soil type was used for this study. The soil properties required to perform the analysis for laterally loaded piles in the API Sand (O’Neill & Dunnivant) soil type are the effective unit weight ($\gamma'$), friction angle ($\phi$), and subgrade reaction coefficient, $k$.

The friction angle ($\phi$) and subgrade reaction coefficient ($k$) both effect the computed load-displacement curves. The parameter $k$ causes a greater effect at the smaller pile head deflections, while $\phi$ has a greater effect at larger deflections near the ultimate soil resistance.

A linear analysis model was used in LPILE for this research. Additionally, the test piles were considered to be hollow steel pipe piles with a consistent cross-section. The models assumed a static loading scenario and homogenous layers of backfill. While these assumptions are not realistically accurate, they are adequate for the purposes of this research.

Calculations of the ultimate soil resistance at depth $x$ can be performed using Equation (2-1) for soil failure near the surface of the soil profile, and Equation (2-2) for soil failure at deeper depths. The equation that produces the smaller ultimate soil resistance value controls.

$$P_{us}=(C_1 x+C_2 B)\gamma' x$$

(2-1)

$$P_{ud}=C_3 B \gamma' x$$

(2-2)

Where:

$P_u$ = the ultimate soil resistance ($s$ = shallow and $d$ = deeper depths) (force/unit length),
C1, C2, and C3 = coefficients determined by Equations 2-3 through 2-5

x = depth,

\( \gamma' \) = effective unit weight, and

\( \phi \) = angle of friction of the soil.

Equations (2-3) through (2-5) can be used to estimate coefficients C1, C2, and C3 found in Equations (2-1) and (2-2).

\[
C_1 = \tan(\beta) \left\{ K_p \tan(\alpha) + K_0 \left[ \tan(\phi) \sin(\beta) \left( \frac{1}{\cos(\alpha)} + 1 \right) - \tan(\alpha) \right] \right\}
\]

(2-3)

\[
C_2 = K_p - K_A
\]

(2-4)

\[
C_3 = K_p^2 \left( K_p + K_0 \tan(\phi) \right) - K_A
\]

(2-5)

Where:

\[
K_p = \tan^2(45^\circ + \frac{\phi}{2})
\]

\[
K_A = \tan^2(45^\circ - \frac{\phi}{2})
\]

\[
K_0 = 0.4
\]

\[
\alpha = \frac{\phi}{2}
\]

\[
\beta = 45^\circ + \frac{\phi}{2}
\]

As was previously mentioned, the API sand soil type, developed by API (1982) was used to model the backfill in this study. In this model, \( p \) represents the soil resistance per length of pile.
and y represents the lateral soil displacement. The p-y curve for the API model is produced by using Equation (2-6).

\[ p = A P_u \tanh\left( \frac{kx}{(AP_u)} y \right) \]  

(2 − 6)

Where:

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

\( y = \) the lateral soil displacement

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

\( A = 0.9 \) for cyclic loading

\( k = \) the subgrade modulus of the soil

\( x = \) the depth below the ground surface

\( b = \) pile width/diameter

\( P_u = \) the ultimate lateral soil resistance

2.2 Full-Scale Testing

Researchers have performed full-scale tests to analyze the behavior of laterally loaded piles behind earth retaining structures. Each of the previously conducted full-scale tests considered how different configurations of the system affected the overall behavior of the system. These configurations include varying pile cross-sections, pile spacing from the wall, wall types, wall heights, and soil reinforcement types. The following authors were a part of Dr. Kyle Rollins’ research group at Brigham Young University: Nelson, Price, Hatch, Han, Besendorfer,
Luna, and Budd. Each of these authors analyzed the behaviors of different systems, but their data has been compiled by the research group to compare varying system configurations. This paper is also a part of Dr. Rollins’ research group.

Several pile types of various cross-section, diameters/widths, and spacing behind the retaining structure have been considered in full-scale testing of laterally loaded piles behind a MSE wall. Table 2-1 contains the pile configurations for several previously performed studies.

<table>
<thead>
<tr>
<th>Author</th>
<th>Number of Piles</th>
<th>Material</th>
<th>Installation Method</th>
<th>Cross-Section</th>
<th>Depth (ft)</th>
<th>Width/Diameter (in)</th>
<th>Spacing From Wall (Pile Diameters)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Pierson (2009)</td>
<td>8</td>
<td>Concrete</td>
<td>Drilled Shaft</td>
<td>Circular</td>
<td>20</td>
<td>36</td>
<td>1D, 2D, 3D, 4D</td>
</tr>
<tr>
<td>Nelson (2013)*</td>
<td>4</td>
<td>Steel</td>
<td>Driven Pipe</td>
<td></td>
<td>42.25</td>
<td>12.75</td>
<td>1.3D, 2.8D, 5.25D, 6.7D</td>
</tr>
<tr>
<td>Price (2013)*</td>
<td>2</td>
<td>Steel</td>
<td>Driven Pipe</td>
<td></td>
<td>70.5</td>
<td>12.75</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>90, 95, 98</td>
<td>16</td>
<td>2.9D, 5.2D, 2.2D</td>
</tr>
<tr>
<td>Hatch (2014)*</td>
<td>4</td>
<td>Steel</td>
<td>Driven Pipe</td>
<td></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>4</td>
<td>Steel</td>
<td>Driven Pipe</td>
<td></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>4</td>
<td>Steel</td>
<td>Driven Pipe</td>
<td></td>
<td>38</td>
<td>12.75</td>
<td>1.7D, 2.8D, 2.9D, 3.9D</td>
</tr>
<tr>
<td>Luna (2016)*</td>
<td>4</td>
<td>Steel</td>
<td>Driven H</td>
<td></td>
<td>33</td>
<td>12.2</td>
<td>2.2D, 2.5D, 3.2D, 4.5D</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td>H</td>
<td>38</td>
<td>12</td>
<td>2.1D, 3.1D, 4.2D, 5.7D</td>
</tr>
<tr>
<td>Budd (2016)*</td>
<td>4</td>
<td>Steel</td>
<td>Driven Pipe</td>
<td></td>
<td>38</td>
<td>12.75</td>
<td>1.8D, 3.4D, 4.3D, 5.2D</td>
</tr>
</tbody>
</table>

* Indicates authors that participated in Dr. Kyle Rollins’ research group at Brigham Young University
Multiple variations of MSE wall configurations have been considered in past full-scale testing of laterally loaded piles behind MSE walls. Table 2-2 contains the wall configurations for several previously performed studies.

The length of soil reinforcement to effective MSE wall height ratio (L/H) is also included Table 2-2. This ratio represents the ratio of the length of the soil reinforcement (L) and the effective wall height (H). The effective MSE wall height is comprised of the height of the MSE wall in combination with the height of the surcharge blocks placed on top of the backfill. Section 3.4 contains a detailed explanation of the surcharge purpose and configuration. Reinforcement length to wall height ratios near 1.0 are typical for seismic loadings. Reinforcement length to wall height ratios near 0.7 are more representative of static loading conditions.

Various sizes and types of soil reinforcement have been considered in past full-scale testing of laterally loaded piles behind MSE walls. Table 2-3 summarizes the parameters of soil reinforcements used in previous studies.

Additionally, multiple backfill conditions were present at the previously conducted full-scale testing of laterally loaded piles behind MSE walls. Table 2-4 summarizes the backfill conditions at each of the previously performed studies.

The achieved compaction percentages and water content percentages are averaged values for the entirety of each respective site. These values were measured using nuclear density gauges.
Table 2-2 Wall types and configurations of several previously performed studies.

<table>
<thead>
<tr>
<th>Author</th>
<th>Type of MSE Wall</th>
<th>Height (ft)</th>
<th>Length (ft)</th>
<th>L/H Ratio</th>
<th>Block/Panel Area (ft²)</th>
<th>Wall Thickness (in)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Pierson (2009)</td>
<td>Concrete Blocks</td>
<td>20</td>
<td>140</td>
<td>0.7</td>
<td>Not Specified</td>
<td>Not Specified</td>
</tr>
<tr>
<td>Nelson (2013)*</td>
<td>Welded Wire Panels with Geo-Fabric</td>
<td>22.25</td>
<td>Not Specified</td>
<td>0.78</td>
<td>46.8</td>
<td>Not Specified</td>
</tr>
<tr>
<td>Price (2013)*</td>
<td>Concrete Panels</td>
<td>20.5</td>
<td>Not Specified</td>
<td>1.0</td>
<td>60</td>
<td>6</td>
</tr>
<tr>
<td></td>
<td>Concrete Panels</td>
<td>37.7, 34.7, 29.8</td>
<td>Not Specified</td>
<td>1.1, 1.1, 1.7</td>
<td>50</td>
<td>6</td>
</tr>
<tr>
<td>Hatch (2014)*</td>
<td>Concrete Panels</td>
<td>15</td>
<td>90</td>
<td>0.9</td>
<td>50</td>
<td>6</td>
</tr>
<tr>
<td>Han (2014)*</td>
<td>Concrete Panels</td>
<td>15</td>
<td>90</td>
<td>0.9</td>
<td>50</td>
<td>6</td>
</tr>
<tr>
<td>Besendorfer (2015)*</td>
<td>Concrete Panels</td>
<td>20</td>
<td>90</td>
<td>0.72</td>
<td>50</td>
<td>6</td>
</tr>
<tr>
<td>Luna (2016)*</td>
<td>Concrete Panels</td>
<td>15</td>
<td>90</td>
<td>0.9</td>
<td>50</td>
<td>6</td>
</tr>
<tr>
<td></td>
<td>Concrete Panels</td>
<td>20</td>
<td>90</td>
<td>0.72</td>
<td>50</td>
<td>6</td>
</tr>
<tr>
<td>Budd (2016)*</td>
<td>Concrete Panels</td>
<td>20</td>
<td>90</td>
<td>0.72</td>
<td>50</td>
<td>6</td>
</tr>
</tbody>
</table>

* Indicates authors that participated in Dr. Kyle Rollins’ research group at Brigham Young University

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Table 2-3 Soil reinforcement configurations for several previously performed studies.

<table>
<thead>
<tr>
<th>Author</th>
<th>Type</th>
<th>Material</th>
<th>Vertical Spacing (ft)</th>
<th>Cross-Sectional Area (in²)</th>
<th>Transverse Spacing (ft)</th>
<th>Length (ft)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Pierson (2009)</td>
<td>Geogrid</td>
<td>HDPE</td>
<td>2</td>
<td>N/A</td>
<td>2</td>
<td>14</td>
</tr>
<tr>
<td>Nelson (2013)*</td>
<td>Ribbed Strips</td>
<td>Steel</td>
<td>2</td>
<td>0.25</td>
<td>2</td>
<td>28</td>
</tr>
<tr>
<td>Price (2013)*</td>
<td>Welded Wire Grid</td>
<td>Steel</td>
<td>2.5</td>
<td>0.2, 0.11</td>
<td>1 - 2.5</td>
<td>33</td>
</tr>
<tr>
<td></td>
<td>Welded Wire Grid</td>
<td>Steel</td>
<td>2.5</td>
<td>0.2, 0.15, 0.11</td>
<td>5</td>
<td>50, 42, 39</td>
</tr>
<tr>
<td>Hatch (2014)*</td>
<td>Welded Wire Grid</td>
<td>Steel</td>
<td>2.5</td>
<td>0.11</td>
<td>5</td>
<td>18</td>
</tr>
<tr>
<td>Han (2014)*</td>
<td>Ribbed Strips</td>
<td>Steel</td>
<td>2.5</td>
<td>0.32</td>
<td>2.25</td>
<td>18</td>
</tr>
<tr>
<td>Besendorfer (2015)*</td>
<td>Ribbed Strips</td>
<td>Steel</td>
<td>2.5</td>
<td>0.32</td>
<td>2.5</td>
<td>18</td>
</tr>
<tr>
<td>Luna (2016)*</td>
<td>Ribbed Strips</td>
<td>Steel</td>
<td>2.5</td>
<td>0.32</td>
<td>2.5</td>
<td>18</td>
</tr>
<tr>
<td></td>
<td>Welded Wire Grid</td>
<td>Steel</td>
<td>2.5</td>
<td>0.11</td>
<td>5</td>
<td>18</td>
</tr>
<tr>
<td>Budd (2016)*</td>
<td>Welded Wire Grid</td>
<td>Steel</td>
<td>2.5</td>
<td>0.11</td>
<td>5</td>
<td>18</td>
</tr>
</tbody>
</table>

* Indicates authors that participated in Dr. Kyle Rollins’ research group at Brigham Young University
Table 2-4: Backfill conditions of several previously performed studies.

<table>
<thead>
<tr>
<th>Author</th>
<th>Fill Type</th>
<th>Standard Proctor Max Density (pcf)</th>
<th>Achieved Compaction (% of Proctor)</th>
<th>Water Content (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Nelson (2013)*</td>
<td>Sandy Gravel (AASHTO A-1-a)</td>
<td>132.2</td>
<td>97.4</td>
<td>7</td>
</tr>
<tr>
<td>Price (2013)*</td>
<td>Sandy Gravel (AASHTO A-1-a)</td>
<td>139.2</td>
<td>97.0</td>
<td>5</td>
</tr>
<tr>
<td></td>
<td>Sandy Gravel (AASHTO A-1-a)</td>
<td>139.4</td>
<td>97.0</td>
<td>5</td>
</tr>
<tr>
<td>Hatch (2014)*</td>
<td>Silty Sand with Gravel (AASHTO A-1-a)</td>
<td>128</td>
<td>95.0</td>
<td>6</td>
</tr>
<tr>
<td>Han (2014)*</td>
<td>Silty Sand with Gravel (AASHTO A-1-a)</td>
<td>128</td>
<td>95.0</td>
<td>7</td>
</tr>
<tr>
<td>Besendorfer (2015)*</td>
<td>Poorly Graded Sand with Silt and Gravel (AASHTO A-1-a)</td>
<td>126.7</td>
<td>96.4</td>
<td>6</td>
</tr>
<tr>
<td>Luna (2016)*</td>
<td>Silty Sand with Gravel (AASHTO A-1-a)</td>
<td>128</td>
<td>95.0</td>
<td>7</td>
</tr>
<tr>
<td></td>
<td>Poorly Graded Sand with Silt and Gravel (AASHTO A-1-a)</td>
<td>126.7</td>
<td>96</td>
<td>6</td>
</tr>
<tr>
<td>Budd (2016)*</td>
<td>Poorly Graded Sand with Silt and Gravel (AASHTO A-1-a)</td>
<td>126.7</td>
<td>96</td>
<td>6</td>
</tr>
</tbody>
</table>

* Indicates authors that participated in Dr. Kyle Rollins’ research group at Brigham Young University

All of the previously performed full-scale tests of laterally loaded piles behind MSE walls have been loaded under static, monotonic loading-conditions. These loads were typically produced by a hydraulic jack situated between the test pile and a reaction pile or beam. The piles were typically loaded to a target deflection and held at that target for several minutes. This was done to avoid the effects of dynamic loading. After the pile was held at the target deflection for several minutes, the piles were then loaded to an increased deflection target. However, in many cases lateral pile loads may be cyclic in nature causing the pile to be loaded in two directions with increasing and decreasing loads.
2.3 Pile Deflections and P-multipliers

Pierson et al. (2009) performed full-scale lateral load testing on drilled shafts located behind a 20-foot tall MSE block wall. All of the drilled shafts were 20-feet long, except for one shaft that was 75% the height of the MSE wall. Each of the drilled shafts were installed at varying locations behind the wall. In this study, the distance between a pile and the MSE wall was normalized by dividing the distance between the pile and the wall by the pile width or diameter. For example, if a normalized pile distance is noted as 4D, that would represent a pile whose center is located 4 pile diameters away from the MSE wall. If the pile distance is noted as 2D, that would represent a pile whose center is located 2 pile diameters away from the MSE wall. Although this notation was not used by Pierson, it has been added to Pierson’s data for consistency purposes.

Pierson et al. (2009) measured the displacement of the head of the drilled shaft as the lateral load applied to the drilled shaft was increased. Figure 2-3 illustrates the displacement experienced by the drilled shaft as the lateral load applied to the head of the drilled shaft is increased.

Pierson et al. (2009) noticed the following general trends: (1) as the laterally applied load increased, the displacement of the head of the drilled shaft increased, (2) the shorter drilled shaft required smaller loads to be displaced the same distance as the longer drilled shafts, and (3) the drilled shafts located closer to the MSE wall required smaller loads to be displaced the same distance as the drilled shafts located further away from the MSE wall. Pierson et al (2009) concluded that resistance opposing laterally loaded piles diminished as spacing between the wall and the pile was reduced.
Figure 2-3 Depiction of diminished resistance of lateral loads when spacing between the wall and shafts are decreased (Pierson et al 2009).

The researchers from the Rollins’ research group observed similar behavior of their respective systems. Furthermore, the Rollins’ research group proposed preliminary methods of determining p-multipliers based on normalized pile spacing behind the wall. As was previously mentioned, spacing was defined as the distance from the back of the wall to the center of the pile and was normalized by the diameter of the pile. The p-multipliers were determined using the computer software, LPILE (see section 2.1). P-multipliers were used to represent how much reduction in lateral soil resistance the system experienced as the spacing between the piles and the MSE wall was varied. A p-multiplier of one indicates no reduction of lateral soil resistance while a p-multiplier of 0.5 would represent a 50% reduction of lateral soil resistance. This does not mean that the lateral pile resistance would decrease by 50% because the lateral resistance of the pile itself is assumed to remain the same regardless of pile location behind the wall. Figure 2-4 illustrates the relationship between p-multipliers and normalized distance away from the MSE
wall as reported by Rollins et al. (2018) and a point added by Farnsworth (2020) originating from Pierson (2009). Figure 2.4 also includes the proposed equation to compute a p-multiplier based on normalized distance from a MSE wall. Rollins et al. (2018) typically observed a p-multiplier of one was appropriate for piles located four or more pile diameters away from the wall. The preliminary equation to predict a p-multiplier based on pile spacing from the wall that the same authors produced was found to have an R² value of 73.5%.

Figure 2-4: Comparison of p-multipliers vs. normalized distance from wall (distance from wall/pile diameter) Pierson (2009), Price (2012), Nelson (2013), Hatch (2014), Han (2014), Besendorfer (2015), Budd (2016), Luna (2016).

\[ y = 0.3061x - 0.1865 \]

\[ R^2 = 0.7654 \]
2.4 Lateral Soil and Wall Displacement

Researchers from Rollins research group measured wall displacement using digital image correlation (DIC), shape arrays, and string potentiometers. The researchers generally observed increased wall displacement at the joints of the wall panels. They also observed that the highest amounts of deflection occurred at the top of the wall, and gradually decreased towards the bottom of the wall. Wall deflections varied on average between 0.1 to 0.4 inches for a pile head displacement of 1 inch. They also observed that most of the wall deflections were no greater than 0.63 inches while most were less than 0.5 inches. It was also observed that there was no significant difference in the maximum wall displacements when the piles were located either directly behind the center of a wall panel, or behind the joint of multiple wall panels. Finally, it was observed that the normalized distance of the pile behind the wall had little effect on the displacement of the wall panel.

Rollins et al (2018) also measured the lateral displacement of the soil as the pile heads were loaded laterally. Figure 2-5 illustrates the normalized ground displacement as versus the normalized distance of the pile behind the wall. The distance was normalized in the same manner as has been previously explained, and the lateral ground displacement was normalized by dividing the lateral displacement of the soil by the lateral displacement of the pile head during pile head loading. The 366 data points were measured at pile deflections of 0.25, 0.5, 1, 2, and 3 inches. A best fit curve, and corresponding equation are included in Figure 2.5. The equation for the best fit curve is also given in Equation (2-7). The results indicated that the ground displacement decreased rapidly as the distance from the pile face increases. Rollins concluded that this behavior is likely caused by the resistance provided by the soil reinforcements.
\[ \frac{\delta - \delta_p}{\delta_p} = 1 - 0.92\tanh\left(\frac{0.8L}{D}\right) \]  \quad (2-7)

Where:

\( \delta \) = horizontal ground displacement

\( \delta_p \) = horizontal displacement of the pile face at the ground surface

\( L \) = distance from a point in front of the pile to the pile face

\( D \) = pile diameter.

Figure 2-5: Normalized ground displacement as a function of normalized distance of the piles behind the MSE wall. (Rollins et al. 2018)
2.5 Tensile Force in Soil Reinforcements

Price (2013) and Nelson (2013) observed an increase in the maximum tensile force experienced by the reinforcement as the lateral load applied to the pile was increased. They also observed an increase in maximum force experienced by the soil reinforcements as the spacing between the piles and the wall was decreased. An explanation for this behavior is as follows: as the spacing between the wall and pile decreases, the volume of soil resisting lateral pile deflection also decreases, resulting in a system with less lateral load capacity. The lateral resistance of the soil reinforcements required to restrict lateral displacement of the piles then increases, resulting in higher forces experienced by the soil reinforcement. As the piles are spaced further away from the wall the volume of soil resisting the lateral pile deflection increases, resulting in a less required resistance from the soil reinforcements. Price (2013) and Nelson (2013) also observed that induced forces in the soil reinforcement decreased significantly as transverse spacing between the reinforcements and the pile increased.

Hatch (2014) and Han (2014) performed an analysis similar to that of Nelson (2013) and Price (2013) and made similar observations. To conceptually illustrate the behavior of the reinforcements during lateral pile head loading Han (2014) created Figure 2-6.

The black dashed line in the figure represents the tensile force experienced by the soil reinforcements. At any given lateral load, the section of the reinforcement experiencing the greatest amount of tensile force is located approximately the same distance away from the wall as the test pile. The tensile force decreases linearly as distance increases away from the test pile increases in either direction.

The red arrows labeled as “Friction on Reinforcement” represent the friction that is induced in the reinforcements by the movement of the soil that is displaced by the lateral loading
of the test pile. As the test pile is loaded, the soil in between the pile and the wall is displaced in the direction of the wall. The movement of the soil in the direction of the wall applies a lateral earth pressure on the wall. The lateral load on the wall is then transferred through the reinforcements in the form of tension. The portion of the reinforcements between the test pile and the wall experience a friction force in the direction of the wall. Behind the test pile, the soil is not displaced, but the reinforcements are displaced in the direction of the wall. This causes the skin friction experienced by the reinforcements to act in the opposite direction of the wall.

![Figure 2-6 Interaction between soil and soil reinforcements when the pile is laterally loaded (Han 2014)](image)

Rollins (2018) compiled soil reinforcement data from all of the previously conducted full-scale tests performed by researchers in the Rollins’ research group, and performed a
statistical analysis with the assistance of Dr. Dennis L. Eggett of the Department of Statistics at Brigham Young University. Separate statistical analysis was performed for each of the soil reinforcement types. Equation (2-8) represents the maximum predicted tensile force for the ribbed strip reinforcement based on 942 data points. The \( R^2 \) value for the equation was approximately 71%.

\[
F = 10^{0.13 + 0.028P - 2.2 \times 10^{-4}P^2 - 0.01 \frac{T}{D} - 0.0021P \frac{T}{D} - 0.031 \frac{S}{D}} - 1
\]  

(2-8)

Where:

\( F \) = maximum predicted tensile force (kip)

\( P \) = pile head load (kip)

\( T \) = transverse distance from reinforcement to pile center (in.)

\( D \) = pile diameter (in.)

\( \sigma_v \) = vertical stress (psf)

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

In addition to the maximum predicted tensile force for the ribbed strip soil reinforcements, Rollins also produced Equation (2-9) to predict the maximum tensile force experienced by welded wire soil reinforcements based on 1,058 data points. The \( R^2 \) value for this equation was approximately 72%.

\[
F = 10^{-0.04 + 0.027P - 2.7 \times 10^{-4}P^2 + 5.7 \times 10^{-4} \sigma_v - 0.08 \frac{T}{D} - 2.6 \times 10^{-7} \sigma_v^2 - 0.08 \frac{T}{D}} - 1
\]  

(2-9)

Where:
F = maximum predicted tensile force (kip)

P = pile head load (kip)

T = transverse distance from reinforcement to pile center (in.)

D = pile diameter (in.)

$\sigma_v$ = vertical stress (psf)

### 2.6 Limitations of Existing Research and Need for Additional Research

While the research performed to date on the behavior of laterally loaded piles behind MSE walls is helpful, additional parameters that could have significant effects on the behavior of the system need to be explored. One of those parameters is the pile diameter. Previous full-scale tests with driven metallic piles have all had diameters between 12 and 16 inches. Larger piles would be expected to produce much larger lateral loads that could exceed the capacity of the reinforcement to restrain the MSE wall panel displacements. In this report the effects of laterally loading a 24 in pipe pile on the MSE wall system will be explored. Additionally, all of the previously performed full-scale tests analyzed the effects on the pile-wall system while the pile is being loaded statically in one direction. As noted previously, many pile head loads might involve cyclic loading that could induce pile movement towards and away from the MSE wall. This report explores the behavior of the pile wall system when a 12.75-inch pipe pile is loaded in a cyclic fashion (repeatedly towards and away from the wall). Furthermore, all of the previous tests have involved a free-head or pinned connection loading at the top of the pile, whereas most bridge abutment piles are placed within a large concrete abutment wall that likely simulates a fixed-head condition. The influence of a fixed-head load is being investigated by Estephania Flores as
another part of phase 3 work at this test site. Finally, abutment piles are often loaded as a group rather than individually. The lateral load produced by a group might also overwhelm the lateral resistance of the reinforcing elements leading to less lateral resistance for the pile and greater wall displacement. This influence of group interaction on the lateral resistance of piles behind an MSE wall is being investigate by Zachary Farnsworth as part of this study.

The results from additional tests discussed in this report, and in the reports produced by Flores and Farnsworth will be compared to the results produced by other researchers in the Rollins research group. The data from these three reports will increase the sample size of previously performed analyses and will aim to provided further substantiation and insights to correlations and conclusions that have been previously formed.
3 TEST LAYOUT

In this section the construction and configurations of the system will be described in detail. The section is divided into subsections highlighting the major stages of construction. The construction was divided into three phases. Phase one was comprised of loading pile laterally behind a 15-foot wall with a L/H ratio of 0.9. After phase one testing was complete, phase two was constructed by adding 5 feet to the wall height resulting in a L/H ratio of 0.72. After phase two testing was complete, the construction of phase three commenced. This study is a part of phase three. For phase three, the top 6.25 feet of backfill was excavated and re-compacted. In addition, strain gauges on the reinforcements were replaced. While overall wall configurations remained the same as phase two, several piles from phase two were removed and new piles were installed in their place. The land for the test was provided by Geneva Rock, Inc. and was located near Lehi, Utah in Utah County as shown in Figure 3-1. The latitude and longitude coordinates of the test site are 40.45312, -111.8994.

The test piles are located at varying distances behind the 20-foot MSE wall. The MSE wall retains select granular backfill that extends 25 feet behind the MSE wall and extends across the entire length of the wall. Welded wire and ribbed strip soil reinforcements provide support for the MSE wall and extend 18 feet from the back of the MSE wall. Steel I-beams were connected to piles located about 22 feet behind the MSE wall to provide a reaction force for the hydraulic jack used during lateral loading.
Finally, concrete blocks were placed on top of the selected granular backfill to provide a vertical force, meant to mimic the loads produced by a bridge abutment. Figure 3-2 provides an elevation view and plan view for this configuration, while Figure 3-3 provides a profile view.

Figure 3-1 MSE wall test site location (Google Earth 2018).
Figure 3-2: Plan and elevation view of the basic configuration of the analyzed system.

Figure 3-3: Profile view of the basic configuration of the analyzed system.
3.1 Piles

*24-inch Statically Loaded Pipe Piles*

During the construction of phase three, the soil was excavated six feet below the top of the MSE wall, and four H-piles on the western side of the MSE wall from phase two were removed using a vibratory hammer. The vibration during extraction cause the sandy backfill to collapse and fill the void left by the piles. These removed piles were replaced by four round A252-Grade 3 24x0.5 pipe piles which were installed at various distances behind the existing 20-foot MSE wall. These distances included: 2.0 pile diameters (4 feet), 3.0 pile diameters (6 feet), 4.1 pile diameters (8.2 feet), and 5.1 pile diameters (10.2 feet). The piles are center-to-center horizontally spaced approximately five feet from each other. A schematic plan view drawing of the piles is detailed in Figure 3-4. The yield strength of the piles was approximately 57,000 psi. The 24-inch piles were donated by Atlas Steel. Figure 3-5 provides a photograph of the installation process of the 24-inch piles.

![Figure 3-4: Plan view of the 24-inch statically loaded pipe pile configuration.](image)
The piles were approximately 40 feet in length and were driven by Desert Deep Foundations using an ICE I-30V2 diesel hammer. The 24-inch piles were driven open ended and were plugged with soil during the driving process. For the purposes of this research, the test piles were considered to be hollow.

Piles were also driven behind the test piles during phase one to serve as a reaction force for the loading apparatus. Additional, 24-inch pipe piles and 20-inch square piles were driven behind the reaction beam to provide additional resistance for the loading of the larger diameter test piles.

To avoid deformation of the heads of the 24-inch piles, a steel member was installed inside of the pile to provide additional structural stiffness. The steel member was installed in-line
with the load point to provide the maximum reinforcement. A curved steel plate was also added between the reinforcing member and the pile wall to provide a uniform loading surface for the reinforcing member, and additional steel plates were placed between the curved plate to fill any gaps between the reinforcing member and the pile wall. Figure 3-6 provides a photograph of this configuration.

Figure 3-6: Photograph of steel reinforcing member inside of the 24-inch pipe piles.
### 12.75-inch Cyclically Loaded Pipe Piles

Four 12.75-inch steel pipe piles were previously installed at various distances behind the existing 20-foot MSE wall. These distances included: 1.5 pile diameters (1.6 feet), 3.1 pile diameters (3.3 feet), 4.2 pile diameters (4.5 feet), and 5.3 pile diameters (5.6 feet). From east to west, the piles are center-to-center horizontally spaced 54 inches, 55.5 inches and 66.5 inches from each other. A schematic plan view drawing of the 12-inch cyclically loaded piles is detailed in Figure 3-7.

![Figure 3-7: Plan view of the 12.75" cyclically loaded pipe pile configuration.](image)

### 3.2 MSE Wall

The MSE wall was constructed in two phases. The first phase of construction called for a wall height of 15 feet, producing an L/H ratio (reinforcement length/effective wall height) of 0.9.
After the construction of phase one was complete, the piles behind the wall were tested. When phase one testing was complete, phase two of the construction process began. Phase two called for an addition five feet of wall height producing a total wall height of 20 feet and a L/H ratio of 0.72. Phase two wall conditions were present during the testing for this study. Figure 3-8 illustrates the elevation of the wall during each phase.

![Figure 3-8: Elevation view of MSE wall with depiction of phases (Budd 2016)](image)

The MSE wall was divided into two halves. The western half of the wall was constructed in accordance with the AASHTO 2012 LRFD code by Hadco using the Reinforced Earth Company (RECo) wall system. RECo supplied the wall panels and soil reinforcements needed to construct the wall. The soil reinforcements supplied by RECo were 18-foot-long galvanized steel ribbed strip reinforcements. These reinforcements were connected to the wall panels using ASTM A449 0.5-inch bolts. Figure 3-9 provides a depiction of the connection between the strip reinforcement and the wall panels. The 90-foot-long wall was comprised of a 50-foot main body and a 40-foot wing wall. The panels for this portion of the wall were 5’x10’ RECO panels with a smooth texture on the panel face as shown in Figure 3-10.
Figure 3-9: Connection of ribbed strip soil reinforcements to RECo wall panels (Budd 2016).

The eastern half of the wall was also constructed by Hadco. For this portion, the SSL, LLC wall system was used. SSL, LLC contributed the wall panels and soil reinforcements needed to construct the eastern portion of the wall. The soil reinforcements used were 18-foot-long welded wire grids consisting of 4, 5, and 6 longitudinal wires. The reinforcements were connected to 0.75-inch loops in the wall by two W30 connector pins. Figure 3-11 provides a depiction of the connection between the welded wire grids and the wall panels. Table 3-1 provides a detailed summary of the welded wire reinforcement specifications at each depth. The
wall panels for this portion of the wall were also 5’x10’, but consisted of an aesthetic texture on the panel face (see Figure 3-10). The SSL, LLC portion of the wall was also 90-feet-long with a 50-foot main body and a 40-foot wing wall.

Table 3-1: Welded wire reinforcement design specifications (modified from Budd 2016)

<table>
<thead>
<tr>
<th>Grid Layer (From Top of Wall)</th>
<th>Depth From Top of Wall (in)</th>
<th>Longitudinal Wire</th>
<th>Horizontal Wire</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>Number</td>
<td>Size</td>
</tr>
<tr>
<td>1</td>
<td>15</td>
<td>6</td>
<td>W11</td>
</tr>
<tr>
<td>2</td>
<td>45</td>
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</tr>
<tr>
<td>8</td>
<td>225</td>
<td>6</td>
<td>W11</td>
</tr>
</tbody>
</table>

Figure 3-10: The two different wall systems used for the MSE wall (Budd 2016)
3.3 Backfill

The phase one backfill classified as AASHTO A-1-a material and as silty sand with gravel (SM) for the Unified Soil Classification System (USCS). Grain size distribution curves for phase one backfill is displayed in Figure 3-12. The phase one backfill had a maximum standard Proctor density of 128.0 pcf and optimum moisture content of 9.7%. The phase one backfill had a target density of 95% of the standard proctor and was compacted in 12 inch lifts behind the piles and in 6 inch lifts between the piles and the wall. The target density was achieved for the backfill behind the piles, but the resulting density for the backfill between the piles and walls was slightly below the target. This is likely due to the difference in compaction methods for the
backfill behind the piles and the backfill between the piles and the wall. The backfill behind the piles was compacted using a roller compactor, while the backfill between the piles and the wall was compacted using a vibratory plate to minimize distress to the MSE wall panels. The use of different compaction methods on this project is justified because a similar approach is common in the industry. Figures 3-13 and 3-14 provide measured relative compaction and moisture content versus depth plots for the phase one backfill conditions.

Figure 3-12: Grain size distribution for phases one and two (Besendorfer 2015).
Figure 3-13: Relative density vs. depth for phase one backfill at various locations away from the wall (Han 2014)

Figure 3-14: Moisture content vs. depth for phase one backfill (Han 2014).
The phase two backfill was installed on top of the existing phase one backfill, after the additional 5 feet of wall was installed (see section 3.3). The phase two backfill also classified as AASHTO A-1-a but classified as a poorly graded sand with silt and gravel (SP-SM) for the USCS. A grain size distribution chart for phase two is also presented in Figure 3-12. The maximum standard Proctor density for the phase two backfill was 126.7 pcf, and the calculated optimum moisture was 9.7%. Once again, the target compaction was achieved for the backfill behind the piles, but the backfill between the piles and the wall reached a density slightly below the target. Figures 3-15 and 3-16 display the relative compaction of the backfill after the completion of phase two.

Figure 3-15: Relative compaction vs. depth for phase two backfill (Besendorfer 2015).
The construction of phase three began after the completion of phase two testing. The phase three construction process began by excavating the top 6.25 feet of the phase two backfill. This excavation was executed to produce virgin compacted soil around the tests piles and to replace the instrumentation on the soil reinforcements. Due to the impracticality of excavating the fill while preventing damage to the existing soil reinforcements, the top three levels of soil reinforcement within the top 6.25 feet of fill were replaced with new soil reinforcements. A backhoe was used to excavate the fill between the test piles and the reaction piles, and picks and shovels were used to excavate the fill between the test piles and the wall to avoid damaging the wall or the piles. The fill behind the reaction piles was not excavated and replaced, because the soil reinforcements did not extend pass the reaction piles. Figures 3-17 and 3-18 illustrate some of the phase three backfill excavation process and the placement of a welded wire reinforcement panel.
Figure 3-17: Excavation between the test piles and the reaction piles using a backhoe during phase three construction.

Figure 3-18: Re-installment of soil reinforcements after backfill excavation during phase three construction.
The phase three backfill was placed and compacted in 8-inch lifts after the excavation of the top 6.25 feet was complete. The new soil reinforcements were installed as the phase three backfill was distributed evenly using a backhoe, walk-behind compact front end loader, and shovels. A roller compactor was used to compact the backfill between the test and reaction piles, whereas a jumping jack and vibratory plate compactor was used near the test piles and between the test piles and the wall. The standard Proctor density of the phase 3 backfill was 126.7 pcf. The target density of the backfill was 95% of the standard proctor.

The construction crew performing the compaction work was comprised of BYU students that were unfamiliar with standard compaction techniques. The crew received instruction on improving their techniques after the compaction of the backfill on the eastern portion of the wall was complete. These improved techniques include: only using a vibratory plate compactor within three feet of the wall, using a jumping jack compactor at locations that were between the test piles and the wall and further than three feet away from the wall, and using a roller compactor only behind the test piles. The crew then compacted the backfill on the western portion of the wall, with the improved techniques.

The non-standard technique used on the eastern portion of the backfill could explain the discrepancies between the density patterns between the western and eastern portions of the wall. The backfill for the western portion of the wall had an average dry unit weight of 121.0 pcf for the backfill within 3 feet of the wall, and a dry unit weight of 126.7 pcf for the backfill further than 3 feet from the wall. The eastern and western portions of the wall had optimum water contents of 7%.
The standard Proctor density of the phase 3 backfill was 126.7 pcf. The target densification was 95% of the standard Proctor. Figures 3-19 and 3-20 depict the placement and compaction process of the phase three backfill. Figures 3-21 and 3-22 provide plots of the measured dry unit weight versus depth and moisture content versus depth, respectively for the eastern half of the backfill during phase 3. Similarly, Figures 3-23 and 3-24 provide plots of the measured dry unit weight versus depth and moisture content versus depth, respectively for the eastern half of the backfill during phase 3.

The compaction target for the backfill behind the eastern half of the wall was met with just a few exceptions as is shown in Figure 3-21. The successful compaction of the eastern backfill between the pile and the wall is likely caused by the non-standard compaction technique of applying a jumping jack compactor as opposed to a vibratory plate compactor within three feet of the wall. A lower value than the target optimum moisture content was generally achieved for backfill behind the eastern half of the MSE wall.

The compaction target for the backfill behind the western half of the wall was generally met as well as shown in Figure 3-23. For the western backfill, the soil located within 3 feet of the wall was compacted less than soil located further than three feet from the wall. This is likely caused by the improved compaction techniques used on this portion of the backfill. Instead of using a jumping jack to compact soil within three feet of the wall, a vibratory plate compactor was used to compact the backfill within three feet of the wall.

Similar to the eastern portion of the backfill, the moisture content achieved for the western backfill was generally slightly less than the optimum moisture content.
Figure 3-19: Placement of backfill using a front end loader

Figure 3-20: The backfill between the test and reaction piles were compacted using a roller compactor, whereas the backfill between the test piles and wall was compacted using either a jumping jack or vibratory plate compactor.
Figure 3-21: Dry unit weight vs. depth for phase three backfill on the eastern half of the wall.

Figure 3-22: Moisture content vs. depth for the phase three backfill on the eastern half of the wall.
Figure 3-23: Dry unit weight vs. depth for the phase three backfill on the western half of the wall.

Figure 3-24: Moisture content vs. depth for the phase three backfill on the western half of the wall.
3.4 Surcharge

A surcharge load was placed on the fill between the test piles and the reaction piles. The surcharge load was meant to mimic the load produced by a bridge abutment and its accompanying fill as illustrated in Figure 3-25. The mimicked surcharge was produced with 2’x2’x6’ concrete blocks stacked on top of each other. The overall unit weight of the surcharge was 600 psf assuming a unit weight of 150 pcf for the reinforced concrete pre-cast blocks. The concrete plots typically occupied an area of about 8 feet x 8 feet as illustrated by the photo of the blocks in Figure 3-26.

![Figure 3-25: Basic schematic illustrating how the surcharge blocks were used to mimic the vertical load produced by a bridge abutment and its accompanying fill.](image)
3.5 Loading Apparatus

24-Inch Statically Loaded Pipe Piles

The 24-inch piles were loaded statically using a hydraulic jack placed between the test piles and a reaction beam. The hydraulic jack was set to apply the load to the test pile 12 inches above the ground surface. A channel section was welded onto the side of each test pile to provide a flat surface for the hydraulic jack to apply its load. A load cell was placed between the hydraulic jack and the channel to measure the applied load. A steel strut and steel plates were placed between the hydraulic jack and reaction beam to transfer the load to the reaction beam. A hydraulic pump was connected to the hydraulic jack to apply load to the test pile. Pressure applied by the jack was used as a check that the load cell was providing reasonable load values.
during the test. Figure 3-27 illustrates the typical configuration of the loading apparatus near the 24-inch test piles, and near the reaction piles and reaction beam.

Figure 3-27: Loading apparatus configuration (a) near the 24-inch test piles and (b) near the reaction piles and reaction beam.

12.75-Inch Cyclically Loaded Pipe Piles

The 12.75-inch pipe piles were loaded cyclically using a 120-kip MTS load actuator and 60-gallon per minute pump unit. The model number and serial number of the actuator are 66.131F-01 and 10476604 respectively. The actuator was set to apply the load to the test pile 12 inches above the ground surface. The actuator was connected to the test piles similar to the 24-inch statically loaded piles. A channel was welded onto the test piles, and a steel extension beam
was bolted onto the channel. The extension beam was connected with bolts to a steel plate, and the steel plate was connected to the load actuator also using bolts. The actuator was bolt-connected using DYWIDAG bars to steel plates placed on either side of the reaction beam from the actuator to provide the mechanism to apply loads in the opposite direction of the wall. The reaction beam was bolt-connected to the reaction piles, similar to the 24-inch statically loaded pile testing. Additionally, the reaction beam was welded to the reaction piles to provide a mechanism for the pile to be loaded in the opposite direction of the wall. Figures 3-28 and 3-29 illustrate the cyclic load apparatus.

Figure 3-28: Loading apparatus configuration for the cyclically loaded piles near the reaction beam and reaction piles.
Figure 3-29: Loading apparatus configuration for the cyclically loaded piles near the test piles.
4 INSTRUMENTATION

4.1 Load Cell and Pressure Transducers

24-Inch Statically Loaded Pipe Piles

As mentioned previously, a load cell was installed between the hydraulic jack and the test pile to measure the lateral load produced by the loading apparatus. Spherical end-platens were used to minimize the eccentric loading and provide uniform pressure between the load cell and the channel attached to the pile. A pressure transducer was also installed between the hydraulic pump and the hydraulic jack to measure the lateral load transferred into the test pile. Readings of the lateral load were taken every two-tenths of a second. The data provided by the load cell was used in the analysis, because the data produced by the pressure transducer was less consistent between readings.

12.75-Inch Cyclically Loaded Pipe Piles

The load applied by the MTS actuator was measured using pressure transducers built into the actuator. The load from the pressure transducer in the actuator was calibrated against the load from the Baldwin load frame in the BYU Structures Laboratory.

4.2 String Potentiometers

Lateral displacement of the piles and the surrounding soil was collected using string potentiometers. The string potentiometers were attached to various points and independent
reference frame. Two of the string potentiometers were attached to the head of the pile, one
directly opposite of the location of the pile loading, and another three feet above the location of
the pile load point. These pile-string potentiometers were used to measure the lateral
displacement and rotation of the pile head. Several other string potentiometers were attached to
stakes placed between the pile and the wall to measure the lateral displacement of the soil. All of
the string potentiometer were connected to an independent reference frame supported by pre-cast
blocks about 10 feet on either side of the test pile so that they would not be affected by the soil
movement due to the lateral pile loading. Having the reference frame supports outside of the
affected area of lateral loading ensured that the data collected would only represent lateral
displacement of elements within the system. Figure 4-1 provides a photograph that illustrates the
instrumentation configuration relative to a test pile.

Figure 4-1: String potentiometer and reference frame configuration relative to a test pile.
4.3 Strain Gauges

Waterproof electrical resistance-type strain gauges were attached along the length of the test piles and along the length of the soil reinforcements. The strain gauges were used to determine the bending moment experienced by the piles and the tensile force developed in the soil reinforcements during lateral loading of the test pile.

4.3.1 Soil Reinforcement Strain Gauges

Strain gauges were bonded to the soil reinforcement using epoxy. At each installation point, a strain gauge was installed on both sides of the reinforcement to mitigate the effects of bending, and to provide redundancy in the event that one of the strain gauges malfunctioned. To protect the strain gauges from damage due to transportation, installation, and water, the strain gauge wires were wrapped with electrical tape. The lead wires were run through a PVC pipe, attached vertically to the back of the MSE wall, and then to the terminal strips located at the ground surface. The terminal strips were then connected to the MEGADAC data collector. The MEGDAC data collector acquired two readings per second during pile loading.

Some of the strain gauges were damaged either during transportation, during pile driving, or following soil compaction after the installation process. In this event, the redundant strain gauge was used as the only data source for that particular location. In cases where the primary and redundant strain gauges were damaged at a particular location, the data was omitted from the analysis at that location. Figure 4-2 provides a conceptual schematic of the soil reinforcements.
and the strain gauges installed to them. Each black dot along the length of the reinforcements represents a strain gauge.

Figure 4-2: Conceptual schematic of the strain gauge locations on the soil reinforcements.

Statically Loaded 24-Inch Pipe Piles

Galvanized steel ribbed strip reinforcements were used to reinforce the wall and the soil located near the statically loaded 24-inch pipe piles. The reinforcements were installed at three different depths below the ground surface and were installed at varying horizontal distances from the test piles in the direction transverse to the direction of loading. Strain gauges were installed along the reinforcements at distances of 0.5, 2, 3, 5, 8, 11, and 14 feet from the wall. Table 4-1 summarizes the location of each soil reinforcement strain gauge within the system and Figure 4-3
provides an illustration of the location of each strain gauge and its corresponding strain gauge ID number.

Table 4-1: Horizontal distance from the soil reinforcements to the center of the piles (in inches) for instrumented soil reinforcements near the 24-inch piles. The soil reinforcement ID numbers are provided in parenthesis.

<table>
<thead>
<tr>
<th>Test Pile</th>
<th>1.25 Feet</th>
<th>3.75 Feet</th>
<th>6.25 Feet</th>
</tr>
</thead>
<tbody>
<tr>
<td>2.0D</td>
<td>15.25 (1-2)</td>
<td>41.25 (2-2)</td>
<td>15.75 (5-4)</td>
</tr>
<tr>
<td>3.0D</td>
<td>18 (2-2)</td>
<td>44 (1-2)</td>
<td>15 (8-4)</td>
</tr>
<tr>
<td>4.1D</td>
<td>16.25 (3-2)</td>
<td>41.5 (4-2)</td>
<td>15.5 (1-4)</td>
</tr>
<tr>
<td>5.1D</td>
<td>12.5 (4-2)</td>
<td>41 (3-2)</td>
<td>5.75 (6-4)</td>
</tr>
</tbody>
</table>

Figure 4-3: Map of the soil reinforcements near the 24-inch piles. The soil reinforcement identification numbers are provided next to the corresponding soil reinforcements.

Cyclically Loaded 12-Inch Pipe Piles

Welded-wire grid soil reinforcements were used to reinforce the soil behind the wall around the 24-inch pipe piles. The strain gauges installed on the welded-wire soil
reinforcements using epoxy after creating a flat spot on the wire with a grinder. The strain
gauges were placed on the second longitudinal wire from the right, at increments of 0.5, 2, 3, 5,
8, 11, and 14 feet away from the wall. The strain gauge wires were wrapped with electrical tape
to protect the wires from water damage and were secured to the side of the wire reinforcement.
Table 4-2 summarizes the location of each soil reinforcement strain gauge within the system, and
Figure 4-4 provides an illustration of the location of each strain gauge and its corresponding
strain gauge ID number.

<table>
<thead>
<tr>
<th>Test Pile</th>
<th>1.25 Feet</th>
<th>3.75 Feet</th>
<th>6.25 Feet</th>
</tr>
</thead>
<tbody>
<tr>
<td>1.5D</td>
<td>23 (9-2)</td>
<td>48 (8-2)</td>
<td>23 (3-4)</td>
</tr>
<tr>
<td>3.1D</td>
<td>24.4 (8-2)</td>
<td>36.9 (7-2)</td>
<td>24.4 (8-4)</td>
</tr>
<tr>
<td>4.2D</td>
<td>15 (7-2)</td>
<td>46.5 (6-2)</td>
<td>15 (7-4)</td>
</tr>
<tr>
<td>5.3D</td>
<td>21 (6-2)</td>
<td>52.5 (7-2)</td>
<td>21 (6-4)</td>
</tr>
</tbody>
</table>

Figure 4-4: Map of the soil reinforcements near the 12.75-inch piles. The soil reinforcement ID numbers are provided next to the corresponding soil reinforcements.
4.3.2 Pile Strain Gauges

The strain gauges attached to the test piles were used to compute the bending moment experienced by the pile during loading. The strain gauges were installed using an epoxy at depths of 4, 6, 8, 11, 14, 17, and 20 feet below the top of the pile. The strain gauges were protected from weathering effects with foam insulation. The foam insulation and strain gauges were protected from mechanical damage caused by pile driving by covering the strain gauges and foam insulation with an angle iron that was tack welded on to the test piles (see Figure 4-5 and Figure 4-6). The strain gauges were installed in pairs on the tension and compression sides of the test piles as a redundant measure and the lead wires were wrapped in plastic bags at the ground surface, where they were eventually connected to the MEGADAC data collector. In the event that one of the strain gauges at a certain depth malfunctioned, the measurement of the redundant strain gauge was used. In the even that both strain gauges malfunctioned at a given depth, the measurements taken at that depth were omitted from the analysis.

Figure 4-5: Installation of Strain Gauges onto the Outside of the Test Piles
4.4 Digital Imagery Correlation (DIC) System

A Digital Image Correlation (DIC) device was used to measure the displacement of the MSE wall panels in front of the test piles during loading. DIC is a 3D, full-field optical system that can measure deformation and displacement by capturing images of hundreds of thousands of points across a surface of interest. The DIC system used was the Dantec Dynamics Q-400 DIC Standard 3D. This configuration is comprised of two cameras spaced horizontally at a known distance, with overlapping fields-of-view that capture images of a surface with a black and white grid painted onto it. The cameras monitor the movement of this grid digitally and input the data into a computer program called Istra-4D. Using the images imported into Istra-4D, contours of the wall’s displacement and deformation were produced (see section 5.6).
The system was calibrated by capturing images of a black and white checkered board. The cameras’ shutter speed, angle of orientation, and focus were adjusted to meet the calibration standards. The cameras were located approximately 25 feet in front of the wall and produced an image about 20 feet wide by 15 feet tall. Figure 4-7 illustrates the DIC configuration for the test.

Figure 4-7: Configuration of DIC testing apparatus.
The testing of the 24-inch statically loaded pipe piles took place from August 29, 2018 to August 31, 2018. Displacement control criteria was the governing loading procedure. The piles were considered to be hollow during the analysis of the data.

5.1 Load Deflection Curves

The lateral load tests were performed in the following order: 2.0D, 3.0D, 5.1D, and 4.1D. Deflection of the pile head was measured as the pile head load was increased until a target pile head displacement was achieved. Once a target displacement increment was reached, the actuator displacement was held constant for three minutes to allow the pile head load and deflection to come to equilibrium and avoid the effects of dynamic loading. Measurements of pile head displacement after one minute of holding and three minutes of holding were recorded prior to increasing the load. The target displacement increments were typically 0.25 inches for the 24-inch statically loaded piles with an initial increment of 0.125 to better capture the initial stiffness of the curve. Maximum pile head displacements ranged from 0.75 to 1.25 inches.

The pile head load vs. pile head deflection curves are presented in Figures 5-1 – 5-3. The pile head deflection was measured with a string potentiometer attached to the wall side of the pile and at the same vertical elevation as the load application point. Pile head load was measured using a load cell as described previously. Potted points in Figures 5-1 – 5-3 represent the average
of 30 points that occur at, and immediately following the start of the one-minute and three-minute holds to minimize the effect off noise on the test results. Although there is some adjustment in the load and deflection during the first minute after, there is relatively little adjustment between the one-minute and three-minute holds. Because of this fact, and to be consistent with plotting procedures used in previous tests, curves for the one-minute hold will be used in subsequent analyses of the test results. The relatively small variation between the load vs. deflection curves for the peak, one-minute hold, and three-minute hold curves suggest that a static loading condition was achieved.

Figure 5-1: Pile head load vs. pile head deflection during the peak load.
Figure 5-2 Pile head load vs. pile head deflection during the 1-minute hold.

Figure 5-3: Pile head load vs. pile head deflection during the 3-minute hold.

Evidence of decreased spacing behind the wall resulting in diminished lateral pile resistance can be seen in Figures 5-1 through 5-3. The 5.1D pile is assumed to experience negligible effect from the proximity of the wall and the 4.1D would also be expected to
experience little reduction in resistance as was the case. In contrast, the 3.0D and 2.0D piles experienced a substantial in lateral resistance as the spacing behind the wall also decreased. The percent decrease in pile head lateral resistance of the 4.1D, 3.0D, and 2.0D piles compared to the 5.1D pile, measured after the completion of the one-minute hold, are tabulated in Table 5-1. Each $\frac{1}{4}$” displacement increment for the 4.1D, 3.0D, and 2.0D piles was compared to the corresponding $\frac{1}{4}$” displacement increment of the 5.1D pile. An additional displacement increment of 0.15-inches was included to increase the sample size.

### Table 5-1: Percent decrease of pile head resistance for measurements taken during the one-minute hold.

<table>
<thead>
<tr>
<th>Percent Decrease by Pile Head Displacement Increment</th>
<th>5.1D</th>
<th>4.1D</th>
<th>3.0D</th>
<th>2.0D</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.15-Inch</td>
<td>0</td>
<td>26.0</td>
<td>36.3</td>
<td>38.9</td>
</tr>
<tr>
<td>0.25-Inch</td>
<td>0</td>
<td>11.1</td>
<td>36.5</td>
<td>39.2</td>
</tr>
<tr>
<td>0.50-Inch</td>
<td>0</td>
<td>7.2</td>
<td>35.7</td>
<td>36.9</td>
</tr>
<tr>
<td>0.75-Inch</td>
<td>0</td>
<td>0.0</td>
<td>28.2</td>
<td>32.7</td>
</tr>
<tr>
<td>Average Percent Decrease</td>
<td>0</td>
<td>11.1</td>
<td>34.2</td>
<td>37.0</td>
</tr>
</tbody>
</table>

It was expected that any pile with a spacing of 3.9 pile diameters or more should experience negligible effects from wall interaction. A difference at the first increment step between the 4.1D and the 5.1D could be a result of differing compaction levels of the soil immediately surrounding the respective piles. It appears that after the pile is loaded past this zone, the difference between the 5.1D and the 4.1D piles approaches zero. The percent decrease in the lateral pile head resistance of the 3.0D pile is more than what is typically expected, while the percent decrease in the lateral pile head resistance of the 2.0D pile is a little less than would be expected.
The unexpectedly low stiffness seen in the backfill surrounding the 4.1D and 3.0D piles is likely caused by overlapping backfill failure planes produced by loading the other two piles. After the lateral loads were applied, the cracks that developed in the backfill near the pile were highlighted using spray paint. These cracks represent the location of the failure planes of the backfill caused by the lateral loading of a pile head. Figure 5-4 provides a photograph of the highlighted cracks observed after the lateral loading of the 5.1D pile head.

![Figure 5-4: Photograph of cracking observed in the backfill after pile head loading of the 5.1D pile.](image)

The lateral loading of each pipe pile produced a failure plane in the backfill located between the test pile and the MSE wall. Figure 5-5 provides a schematic that illustrates how these planes overlap one another. Because there is a clear overlapping of the failure planes in the
backfill, the order in which the piles were tested significantly impacted the stiffness of the backfill. The piles were tested in the following order: 2.0D, 3.0D, 5.1D, and 4.1D.

For example: The lateral loading of the 2.0D pipe pile produced a failure plane in the backfill that overlapped the failure plane that would eventually occur due to the loading of the 3.0D pile. This caused the backfill to be less stiff during the testing of the 3.0D pile resulting in higher pile head deflection than what was expected.

Figure 5-5: Illustration of the overlapping backfill failure planes observed after lateral pile head loading.
5.2 Pile Head Rotation

Pile head rotation vs. pile head load was recorded for each pile spacing behind the wall. The rotation was measured by placing string potentiometers on either side of the pile. The string potentiometer on the load side of the pile was placed three feet above the location of the applied load. The string potentiometer on the wall side of the pile was placed directly level with the applied load. Pile head rotation, $\theta$, was calculated using the equation (5-1):

$$\theta = \sin^{-1} \left( \frac{D_L - D_W}{36 \text{ in}} \right)$$  \hspace{1cm} (5-1)

Where:

$\theta = $ pile head rotation

$D_L = $ deflection on the load side of the pile (approximately three feet above load), and

$D_W = $ deflection on the wall side of the pile (in-line with load).

For each pile type and spacing, the string potentiometer readings for the one-minute holds were used to calculate the rotation. Typically, it is expected that a decrease in spacing between the pile and wall will increase the pile head rotation. As the lateral load is met with less lateral resistance, due to decreased spacing between the pile and the wall, the pile head is less restrained, and can more freely rotate.

Pile head rotation vs. pile head load curves for the four 24" pipe piles at various normalized spacings are presented in Figure 5-6. The 5.1D pile rotated the most followed by 2.0D, 3.0D and finally, the 4.1D piles.
The 5.1D pile experienced the most pile head rotation, when it was expected to experience the least. Apart from the 5.1D pile, the magnitude of rotation did decrease with increasing spacing. For example, the 4.1D pile experienced much less pile head rotation than the 2.0D and 3.0D piles until the final load increment. Oval-shaped deformation of the pile head could be an explanation for the poor rotation results. Efforts to mitigate the effect of structural deformation of the pile head due to the lateral load were explained in section 3.1. While these efforts provided some support, structural deformation of the pile head was still observed. Figure 5-7 provides a photograph of the observed the structural deformation of the pile head.
Despite the placement of the reinforcing steel member, it was observed that the round pile head began to deform into an oval shape as the pile head load became significant. Because the pile-wall connected to the loading apparatus deformed, but the pile-wall on the far side of the loading apparatus remained motionless, the data suggested significantly more rotation was occurring than what was taking place in reality. Figure 5-8 demonstrates the effect of the oval-shaped deformation in artificially increasing the measured rotation.
Pile Bending Moment Performance

As noted previously, strain gauges were placed on the compression and tension side of the pile at depths of 2, 4, 6, 8, 9, 12, 15, and 18 feet below the ground surface. These strain measurements were used to compute bending moment in the pile as a function of depth throughout the lateral load test. Several of the strain gauge labels were lost or destroyed during the pile installation process. To deal with this issue, when an unlabeled strain gauge recorded a strain measurement, the depth of that strain gauge was assigned based on a fit with the surrounding strain gauge measurements.

The bending moments developed in the piles was calculated using the following equation:

\[ M = \frac{EI}{2x} \left( (\mu \varepsilon_{it} - \mu \varepsilon_{ot}) - (\mu \varepsilon_{ic} - \mu \varepsilon_{oc}) \right) \times 10^{-6} \]  

(5-2)
Where:

\[ M = \text{the bending moment in inch-kips} \]

\[ E = \text{the pile’s modulus of elasticity (29,000ksi)} \]

\[ I = \text{the moment of inertia of the pile not including the protective angle iron (2549.3 inches}^4)\]

\[ \mu\varepsilon_{it} = \text{the micro strain on the tension side of the pile at the } i^{\text{th}} \text{ depth} \]

\[ \mu\varepsilon_{ot} = \text{the initial micro strain on the tension side of the pile at the } i^{\text{th}} \text{ depth} \]

\[ \mu\varepsilon_{ic} = \text{the micro strain on the compression side of the pile at the } i^{\text{th}} \text{ depth} \]

\[ \mu\varepsilon_{oc} = \text{the initial micro strain on the compression side of the pile at the } i^{\text{th}} \text{ depth} \]

\[ x = \text{corrected distance between strain gauges and the neutral axis (see Figure 5.9).} \]

The piles were intended to be installed with the strain gauges directly in-line with the lateral load, but during installation several of the piles rotated causing the strain gauges to not be in line with the lateral load as shown in Figure 5-9. This rotation was accounted for in Equation 5-2. In the event that only one of the strain gauges at a certain depth was functioning correctly, the value recorded by the functioning strain gauge was doubled. The strain data for each pile type was taken at the one-minute load hold.
Pile moment vs. depth curves for each of the four 24-inch statically loaded piles are displayed in Figures 5-10 – 5-13. In each figure, several curves are provided for selected lateral pile head load values as noted in the legend. As expected, the pile moment increased as the pile head load increased. Generally, the bending moment experienced by the pile increased until a depth between 4 to 8 feet, then decreased towards zero with increasing depth. Researchers in the Rollins research group observed maximum pile moments at depths of 4 to 8 feet for 12.75-inch diameter pipe piles, which is consistent with patterns observed in this study. While the pile moment increased as spacing away from the wall increased, it is more likely that the increases in pile moment are due to larger loads applied to the piles that were spaced further away from the wall.
Figure 5-10: Pile moment vs. depth curves at selected pile head loads for pile spaced at 5.1D from the wall

Figure 5-11: Pile moment vs. depth curves at selected pile head loads for pile spaced at 4.1D from the wall
Figure 5-12: Pile moment vs. depth curves at selected pile head loads for pile spaced at 3.0D from the wall

Figure 5-13: Pile moment vs. depth curves at selected pile head loads for pile spaced at 2.0D from the wall
5.4 Induced Tensile Force in the MSE Reinforcements

Strain gauges were installed on the soil reinforcements to measure the strain the soil reinforcements experienced during lateral pile head loading. The measured strain was then used to calculate the tensile force the soil reinforcements experienced. Steel ribbed strip reinforcements were used near the 24-inch pipe piles. Strain gauges were placed at distances of 0.5, 2, 3, 5, 8, 11, and 14 feet away from the wall along the 18-foot long reinforcements and on either side of the reinforcement. If one of the strain gauges malfunctioned, the measured strain gathered by the strain gauge on the opposite side of the reinforcement was doubled. When the strain gauges on either side of the reinforcement malfunctioned, the data point was eliminated.

The equation used to calculate the tensile force experienced by these reinforcements was

\[ T = EA(\mu \varepsilon_{AVG})(10^{-6}) \]  \hspace{1cm} (5-3)

Where:

- \( T \) = the tensile force experienced by ribbed strip soil reinforcements
- \( E \) = the modulus of elasticity of the ribbed strip soil reinforcement (29,000 ksi)
- \( A \) = the cross-sectional area of the 5 mm x 40 mm ribbed strip reinforcement (0.31 in\(^2\))
- \( \mu \varepsilon_{AVG} \) = the average micro strain between the strain gauge on the top and the strain gauge on the bottom of the ribbed strip soil reinforcement.

The calculated tensile force in the ribbed strip reinforcements as a function of distance away from the wall for selected pile head loads is displayed in Figures 5-15 – 5-18. These figures provide results from two reinforcements during load testing of the 3.0D and 5.1D piles. Figure 5-14 provides an illustrated location of each strain gauge. Additional plots of the tensile force
along the reinforcements length can be found in Appendix A for other instrument strip
reinforcements. A map of each instrument soil reinforcement provided by Figure 5-14 previously
displayed in Figure 4-3 has been included in this section for the reader’s convenience. Additional
information on the location of each soil reinforcement is summarized in Table 5-1 which was
also previously given in Table 4-1.

![Figure 5-14: Map of the soil reinforcements near the 24-inch piles. The soil reinforcement
identification numbers are provided next to the corresponding soil reinforcements.]

---

**Table 5-2: Horizontal distance from the soil reinforcements to the center of the piles (in inches) for
instrumented soil reinforcements near the 24-inch piles. The soil reinforcement ID numbers are
provided in parenthesis.**

<table>
<thead>
<tr>
<th>Test Pile</th>
<th>1.25 Feet</th>
<th>3.75 Feet</th>
<th>6.25 Feet</th>
</tr>
</thead>
<tbody>
<tr>
<td>2.0D</td>
<td>15.25 (1-2)</td>
<td>41.25 (2-2)</td>
<td>15.75 (5-4)</td>
</tr>
<tr>
<td>3.0D</td>
<td>18 (2-2)</td>
<td>44 (1-2)</td>
<td>15 (8-4)</td>
</tr>
<tr>
<td>4.1D</td>
<td>16.25 (3-2)</td>
<td>41.5 (4-2)</td>
<td>15.5 (1-4)</td>
</tr>
<tr>
<td>5.1D</td>
<td>12.5 (4-2)</td>
<td>41 (3-2)</td>
<td>5.75 (6-4)</td>
</tr>
</tbody>
</table>
Figure 5-15: Tensile force experienced by soil reinforcement #3-2 for selected pile head loads during lateral load test of the 5.1D pile

Figure 5-16: Tensile force experienced by soil reinforcement #6-4 for selected pile head loads during lateral load test of the 5.1D pile
Figure 5-17: Tensile force experienced by soil reinforcement #2-2 for selected pile head loads during lateral load test of the 3.0D pile.

Figure 5-18: Tensile force experienced by soil reinforcement #5-4 for selected pile head loads during lateral load test of the 3.0D pile.
Researchers in the Rollins research group observed that the maximum tensile force experienced by the reinforcements generally occurred near the soil-pile interface on the wall side of the pile rather than at the wall face. This phenomenon was also observed in this study as shown in Figures 5-15 – 5-18. These figures also show that as in previous studies, the maximum tensile force experienced by the reinforcements increased as the pile head load increased.

The relationship between the maximum tensile force experienced by the reinforcements and the load applied to the pile is illustrated in Figures 5-19 through 5-22. It is evident through these figures that as the pile head load increased, the maximum tensile force in the reinforcements also increased. A map of the locations of each soil reinforcement can be found in Figure 5-14.

![Graph showing the relationship between pile head load and maximum reinforcement load](image)

**Figure 5-19: Maximum reinforcement tensile force vs. pile head load for 5.1D pile. See Figure 4-3 for reinforcement ID locations.**
Figure 5-20: Maximum reinforcement tensile force vs. pile head load for 4.1D pile. See Figure 4-3 for reinforcement ID locations.

Figure 5-21: Maximum reinforcement tensile force vs. pile head load for 3.0D pile. See Figure 4-3 for reinforcement ID locations.
In Figures 5-15 and 5-16, it can be seen that the maximum tensile load experienced by the reinforcements increased with depth until a distance of 4 feet away from the wall, after which it again decreased.

Generally, the observations of the reinforcement data agree with Hatch (2014), Han (2014), Besendorfer (2015), and Budd (2016) in the following ways: the reinforcements located at a depth of 4 feet consistently experienced the greatest tensile force, and that the maximum tensile force in the reinforcement increased as the spacing between the pile and the wall increases.

Figure 5-22: Maximum reinforcement tensile force vs. pile head load for 2.0D pile. See Figure 4-3 for reinforcement ID locations.
However, the correlation between the tensile force and the spacing between the pile is most likely caused by the larger loads applied to the piles that were spaced further away from the pile.

Figures 5-23 – 5-24 illustrate the maximum tensile force experienced by the soil reinforcements as a function of transverse distance from the reinforcement to the center pile and depth below the ground surface respectively.

Previous researchers in Dr. Rollin’s research group noticed that as transverse distance between the soil reinforcement and the center of the pile increased, the maximum tensile loaded experienced by the reinforcements decreased. This trend is not obvious in Figure 5-23. A major difference between this test and the test performed by Budd (2016) and Besnedorfer (2015) is the width of the test pile and its associated wedge of failure. The backfill failure wedge produced by the 24-Inch piles is much wider than the wedge produced by the 12.75-inch piles used in the previous studies. It is likely that the larger backfill failure wedge produced by the lateral loading of the 24-inch more fully encompassed surrounding reinforcements in comparison to the backfill failure wedge produced by the lateral loading of the 12.75-inch piles. It is likely if reinforcements were installed at larger transverse distances from the center of the 24-inch piles, the trend seen by Budd (2016) and Besendorfer (2016) would also be observed in this study.

In contrast, a fairly significant trend is evident in Figure 5-24. The maximum tensile force experienced by the soil reinforcements increased with depth until a depth of 3.75 feet. From that depth the maximum tensile load experienced by the reinforcements decreased as depth increased. The trend from this figure would suggest that the maximum tensile force experienced by soil reinforcements generally occurs at a depth of 3.75 feet.
Researchers in Dr. Rollins’ research group developed equations to predict the tensile force experienced by ribbed strip soil reinforcements during lateral loading of 12.75-inch
diameter/width piles behind MSE walls. The most up-to-date equation from this group for ribbed-strip reinforcements is presented in Equation (2-3). Because the piles in this study were 24-inches in diameter and induced much greater loads in the reinforcements, Equation (2-3) significantly under predicted the maximum tensile forces measured in this study. To deal with this variation, I proposed that the maximum induced tensile force, \( F \), in kips be computed using the equation below.

\[
F = 10^{\left(0.13 + 0.028P \frac{D}{12.75} - 2.2 \times 10^{-4}P^2 \frac{D}{12.75} - 0.01 \frac{T}{D} - 0.0021P \frac{T}{D} - 0.031 \frac{S}{D}\right) - 1} \quad (5-4)
\]

Where:

\( D \) = the outside pile diameter in inches,

\( T \) = the transverse distance from the reinforcement to the pile center in inches,

\( S \) = the distance from the back of the wall to the center of the pile in inches, and

\( P \) = the pile head load in kips.

Equation (5-4) differs from the equation proposed by Rollins (2018) in that the load variable is multiplied by the ratio of pile diameter in the test (24 inches) divided by the pile diameter used to develop the original equation (12.75 inches). This ratio is designed to account for the use of a larger pile diameter than has been used in previous testing.

The predicted maximum tensile force experienced by each instrumented reinforcement within the 24-inch statically loaded pile system was calculated using Equation 5-4 for each load increment for each of the four test piles. The predicted maximum tensile force was then compared to the maximum observed tensile forces in the various reinforcements during lateral
load testing of each pile. Figure 5-25 provides a comparison of the log of the measured maximum tensile force + 1.0 relative to the log of the computed maximum tensile force plus one. A value of 1.0 was added to the maximum tensile force on both axes to prevent negative numbers.

Because the tensile force in the reinforcements increases as the lateral pile head load increases, there are multiple data points, each representing tensile forces during different pile head load increments.

Any tensile force measurements associated with pile head loads that would exceed the maximum bending capacity of the pile were omitted from this analysis. The maximum load that could be applied to the pile without exceeding the pile’s bending moment capacity ranged from 140 kips to 170 kips depending on the spacing of the pile behind the MSE wall.

Figure 5-25: Statistical comparison of previously suggested equation for prediction of tensile force experienced by the soil reinforcements and the measured tensile force experienced by soil reinforcements as a part of this study.
Any point falling on the 1:1 line represents a perfect match between the predicted maximum tensile force in the reinforcement and the maximum field measured tensile force. The other boundary lines represent the envelopes for the first and second standard deviations scatter about the best-fit line. Of the 55 data points included in this analysis, 64% fell within the one standard deviation boundary, which is quite similar to the 68% that would be expected within this range based on a normal distribution of error.

There are several explanations for why the correlation between the predicted and measured tensile forces were not stronger. Figure 5-26 compares the continuous results of Equation 5-4 (with inputs being arbitrary load values) and measured tensile forces experienced by a couple of the soil reinforcements. Additional figures illustrating this comparison for other soil reinforcements can be found in Appendix B.

Two continuous curves of the equation are given in the figure. The curve with the dashed line represents the results of Equation 5-4. The curve with the solid line represents the results of Equation 5-4 if the pile diameter ratio (D/12.75) was not included. The square data points represent 1 plus the measured maximum tensile force experienced by a given soil reinforcements during various pile head loads. As was expected, as the pile head load increased, the maximum tensile force in the reinforcements increased.

In part (a) of Figure 5-26, the measured maximum tensile force fit more closely to the curve with the pile diameter ratio in comparison to the curve without it. This would suggest that equation (5-4) more closely predicts the maximum tensile force experienced by the soil reinforcements than the equation produced by Rollins et al. (2018). It can also be seen through the dashed-curve in part (a) of Figure 5-26 that any loads greater than 125 kips inputted into equation (5-4) will result in a decrease in maximum tensile force due to the quadratic nature of
equation (5-4). A quadratic function was used by Rollins et al. (2018), because the maximum tensile load data gathered by those researchers trended in a pattern similar to that of a positive slope of a quadratic function. Because the lateral loads applied to piles studied by Rollins et al. (2018) did not exceed 75 kips, all of the measured tensile force data points shared a domain with the positive slope of the quadratic function. In this study, the 24-inch piles had applied loads of over 200 kips exceeding the domain of the of the positive slope of equation (5-4).

Figure 5-26: Evaluation of the suitability of the equation developed by Rollins (2018) to predict the maximum tensile force experienced by a soil reinforcement. Part (a) represents measurements taken from soil reinforcement #8-6 near the 2.0D pile. Part (b) represents the measurements taken from soil reinforcement #4-4 near the 5.1D pile.
In part (b) of Figure 5-26, it can be seen that the measured reinforcements loads do not correlate to the dashed curve well in comparison to part (a) of Figure 5-26. The difference between part (a) and (b) is the spacing of the laterally load pile behind the MSE wall. The measurements shown in part (a) were taken during the loading of the 2.0D pile, while the measurements shown in part (b) were taken during the loading of the 5.1D pile. Most of the piles used to develop the equation produced by Rollins et al. (2018) had pile widths/diameters of approximately 1 foot. The spacing between the MSE wall and the test piles from these studies was 5.7D or less, making the maximum distance between the piles and the wall no more than 6 feet. In this study, the diameter of the piles was 24-inches, with a maximum pile spacing of 5.1D or 10.2 feet. It is likely if reinforcement tensile data gathered during lateral pile head testing of piles spaced at distances similar to the piles in this study were used in the regression analysis to create Equation (5-4), that the data points seen in part (b) of Figure 5-26 would fit more closely to the dashed curve representing Equation (5-4).

The data provided in this study would enhance the sample variability and could potentially lend to an improved equation for predicting the maximum tensile force experienced by ribbed strip reinforcements. Such an investigation is beyond the scope of this research but is being pursued by other students in current investigations.

### 5.5 Vertical and Lateral Soil Displacement

Vertical and lateral ground displacement occurred during the lateral loading of each pipe pile. The following sections will detail the findings of the observed soil behaviors.
5.5.1 Lateral Ground Displacement

The lateral displacement of the ground was measured using string potentiometers. String potentiometers were placed at varying locations between the pile and the wall depending on the pile spacing. Generally, it is expected that the greatest lateral soil displacement would take place near the face of the pile and that soil displacement would decrease as the distance between the pile and the wall decreased.

The lateral ground displacement of the soil in front of the 24-inch pipe piles is represented in Figures 5-27 through 5-30. Each curve represents the peak the lateral ground displacement in front of the pile for a given lateral pile head load.

Figure 5-27: Lateral ground displacement vs. distance in front of the pile for the pile at 5.1D behind the MSE wall.
Figure 5-28: Lateral ground displacement vs. distance in front of the pile for the pile at 4.1D behind the MSE wall.

Figure 5-29: Lateral ground displacement vs. distance in front of the pile for the pile at 3.0D behind the MSE wall.
Figure 5-30: Lateral ground displacement vs. distance in front of the pile for the pile at 2.0D behind the MSE wall.

As expected, the greatest lateral ground displacement for each of the piles occurred at the pile face, and lateral ground displacement increased as the load increased. It also appears that as spacing between the pile and the wall decreases, the lateral ground displacement increases.

Figure 5-31 provides plots of the normalized ground displacement vs. the normalized distance from the pile face after each pile was loaded 0.75in. The distance in front of the piles was normalized by dividing by the pile diameter, while the lateral ground displacement was normalized by dividing by the lateral displacement of the pile head.
With the exception of the 4.0D pile, the curves generally trend downward from 1.0 to an asymptote of 0.5 at a normalized distance of about 4.0D. These curves differ from the curves reported by Besendorfer (2015) and Budd (2016) for 12.75-inch diameter piles. The piles studied by those authors experienced a decreasing normalized lateral displacement until the ground displacement reached nearly 0% of the pile head displacement at a normalized distance of about 4.0D. In contrast, the ground displacement in this study never fell below 30% of the pile head displacement. This could be due to the higher compaction level that was achieved immediately surrounding the piles as opposed to the compaction achieved in the space between the piles and the wall. Another explanation of the discrepancy is the effect of a larger pile diameter in this study. The piles studied by Besendorfer (2015) and Budd (2016) were approximately 12-inch
diameter pipe piles and may have had a smaller displacement “footprint” in comparison with the 24-inch diameter piles in this study.

5.5.2 Ground Heave

Measurements of vertical ground displacements were also gathered using an optical surveying level and rod. Measurements were taken before the lateral load was applied and taken again at the end of testing. The difference between the two measurements is the vertical ground displacement.

Measurements of vertical ground displacement near the piles were difficult to record. Placing the surveying rod at the pile face was awkward and impractical. Due to this complication, the vertical ground displacement at the pile face was interpolated. The vertical ground displacement at the maximum pile head load is plotted as a function of distance in front of the pile for the four test piles in Figure 5-32. The greatest ground heave occurs near the pile face, and gradually decreases approaching the wall. The vertical ground displacement also dramatically decreased one pile diameter away from the pile face. It is expected that the maximum vertical displacement would increases as pile spacing behind the wall increases. That is the case for the 2.0D, 3.0D, and 4.1D piles. However, the 5.1D pile experienced similar maximum ground displacements as the 2.0D and 3.0D piles. This could be due to higher soil compaction achieved immediately around the 5.1D pile that was not achieved immediately around the 2.0D, 3.0D, or 4.1D piles.
Figure 5-32 Vertical ground displacement at the end of the statically-lateral loading of each 24-inch pipe pile. The location of each test pile is shown by a colored rectangle corresponding to the color of the line indicating the ground heave.

5.6 MSE Wall Panel Displacement

Displacement of the MSE wall panels was measured using digital imagery correlation (DIC) as noted previously. In addition, a string potentiometer was also placed at the top of the wall, directly in line with the pile and applied load, to verify the results of the DIC. The computer program, ISTRA-4D was used to analyze the images produced by the DIC cameras. An explanation of the DIC apparatus is given in section 4.4.

Figure 5-33 provide color contour images of the wall panel displacement from the DIC analysis of the 24-inch laterally loaded tests. Each subfigure represents the displacement of the wall panel during the last displacement interval for each test pile.
A correlation is evident between the spacing between the test pile and the wall, and the displacement experienced by the wall. As the spacing between the pile and the wall increases, the wall panel displacement decreases. It also appears that wall panel displacement occurs to greater depths when the pile is loaded directly behind a wall panel joint than in the center of a wall panel.

Figures 5-35 through 5-38 compare the results of the DIC testing to the measurements gathered by the string potentiometer attached to the top of the wall. A strong correlation can be seen between the DIC and string potentiometers for the 2.0D and the 5.1D tests. The poor correlation seen in the 3.0D and the 4.1D could be a result of the DIC cameras being disturbed by wind during testing.
The average wall deflection measured by a string potentiometer attached to the top of the wall panel for the 5.1D, 4.1D, 3.0D, and 2.0D was 0.41, 0.33, 0.52, and 0.59 respectively while each pile head was experiencing 0.75 inches of deflection. Previous researchers in the Rollins (2018) research group observed an average pile head deflection of approximately 0.1 inches during a pile head deflection of 1.0 inch. Figure 5-34 compares the maximum wall panel displacement during a pile head deflection of one inch as a function of normalized pile spacing between this studies and previous studies in the Rollins (2018) research group. It is likely that the wall panel displacement seen in this study was significantly larger due to the larger applied pile head loads, and the accompanying shear planes.

![Figure 5-34: Comparison between maximum wall panel displacement during a one-inch pile head deflection as a function of normalized distance of the pile form the MSE wall between studies performed in the Rollins (2018) research group, and this study.](image_url)
Figure 5-35: Wall displacement vs. depth for wall panels near the 5.1D pipe pile.

Figure 5-36: Wall displacement vs. depth for wall panels near the 4.1D pipe pile.
Figure 5-37: Wall displacement vs. depth for wall panels near the 3.0D pipe pile.

Figure 5-38: Wall displacement vs. depth for wall panels near the 2.0D pipe pile.
6 CYCLIC LATERAL LOAD TESTING OF 12.75-INCH PIPE PILES

The testing of the 12.75-inch cyclically loaded pipe piles took place from July 31, 2018 to August 13, 2018. Displacement control criteria was the governing loading procedure.

6.1 Load Deflection Curves

Pile head load vs. deflection was also measured for the 12-inch cyclically loaded pipe piles. In contrast to the 24-inch statically loaded piles where piles were pushed monotonically towards the wall with a hydraulic jack, the cyclically loaded piles were loaded using an MTS actuator placed between the test pile and a reaction beam. This system allowed the test piles to be pushed towards and pulled away from the MSE wall with a sinusoidal load application. A detailed description of the loading system and instrumentation is provided in section 3.6. The actuator provided the lateral load until a target displacement was reached. Once the actuator displaced to the target displacement, the load was applied in the opposite direction. The piles were loaded at a rate of .01 inches/second in both directions. This process was repeated for 15 loading cycles in each direction. After the 15 loading cycles were complete, then lateral loads were applied until the actuator reached the next displacement increment value. Then, another 15 loading cycles were applied to complete the loading cycles for that displacement increment. This process was repeated until the desired number of displacement increments had been applied to the test pile.
A loading cycle represents the increase in load, until the target actuator displacement is reached, the decrease of load, until the same target actuator displacement is reached in the opposite loading direction, and finally, the increase in load, until the actuator displacement returns to its initial starting point. A displacement increment represents a compilation of load cycles that share a common actuator displacement target. Figures 6-1 through 6-4 display the continuous measured applied load vs. displacement for each of the four cyclically loaded piles. The piles were tested in the following order: 1.5D, 3.1D, 4.2D, and 5.3D.

For each of the load-deflection charts, a positive load indicates applied loading towards the wall and negative load indicates applied loading away from the wall. While a positive displacement value represents a location between the initial zero point of the test pile and the wall, while a negative displacement value represents a location between the zero point of the test pile and the reaction beam. Displacement in the positive direction, or towards the wall, was typically achieved at smaller loads compared to displacements in the negative direction. This suggests that the soil-wall system is less stiff than the in-situ soil located behind the test piles.

In Figure 6-4 irregularities can be seen during the first displacement increment of the 1.5D pile. These irregularities were caused by failures that occurred with the connections to the reaction pile/reaction beam during tension loading (loading away from the wall) allowing for the actuator to freely displace. Once the failure was observed, the testing was paused until repairs were made. With the repaired system, the piles were pushed back into the zero-displacement location, and testing was resumed.

At small deflections, the load-deflection loops are relatively steep and linear with little reduction in load for multiple cycles. As the deflection level increases the slope of the curves flatten and the loops increase in widths as a result of hysteresis. In addition, at larger deflections
there is greater evidence of reduced lateral resistance with increasing numbers of cycles and the deflection also tends to increase with cycling. For each set of load-deflection cycles, the first load cycle exhibits a concave downward shape when the pile is deflecting into the soil under virgin loading. For the second and subsequent loading, the load-deflection curve exhibits a concave upward shape as the pile moves through the gap with reduced lateral soil resistance but picks up increasing load as the pile face moves back into the soil that has relaxed into the gap.

Figure 6-1: Continuous pile head load vs. pile head displacement for the 5.2D pipe pile.
Figure 6.2: Continuous pile head load vs. pile head displacement for the 4.2D pipe pile.

Figure 6-3: Continuous pile head load vs. pile head displacement for the 3.1D pipe pile.
Figure 6-4: Continuous pile head load vs. pile head displacement for the 1.5D pipe pile.

Figure 6-5 displays the lateral displacement of each pile as a function of pile head load. Each point represents the peak pile head load of the first and last loading cycle of each displacement increment. Once again, positive displacement values represent displacement towards the wall, and positive pile head load values represent pile loads being applied in the direction of the wall. It was again observed that as spacing between the test pile and that wall increased, the lateral displacement of the pile head decreased. Results from Figure 6-5 further support the presumption that displacements in the direction of the wall require less lateral load applied to the pile than equivalent displacements in the opposite direction of the wall.
Figures 6-6 – 6-9 compare the first cycle peak load vs. displacement curves towards and away from the wall for the four cyclically loaded piles at 1.5, 3.1, 4.2 and 5.3D, respectively. The first several points on each curve represent the accumulation of loading during the first loading cycle of the first displacement increment, while the remainder of the points represents the maximum displacement of the first loading cycle of each displacement increment. For the piles at 4.2D and 5.3D, the load-deflection curves are nearly identical suggesting that the piles are far enough behind the wall that the presence of the wall has little effect on response. Another explanation is that the reinforcements provide enough lateral resistance to prevent a reduction in lateral pile resistance during loading towards the wall. For the piles at 1.5D and 3.2D behind the wall, the load-deflection curves are similar under loading in either direction for small deflections. However, at greater load levels, these piles are generally more easily displaced towards the wall rather than away from the wall for the same loads. This result suggests that for piles spaced closer than 3.2D, at a minimum, the piles are experiencing reduced lateral resistance when
loaded towards the wall. This is consistent with previous testing which shows reduced lateral resistance for piles located less than 4D behind an MSE wall.

Figure 6-6: Pile head load vs. pile head displacement for displacement increments towards the wall and inverted displacement increments away from the wall for the 5.3D cyclically loaded pip pile.

Figure 6-7: Pile head load vs. pile head displacement for displacement increments towards the wall and inverted displacement increments away from the wall for the 4.2D cyclically loaded pip pile.
Figure 6-8: Pile head load vs. pile head displacement for displacement increments towards the wall and inverted displacement increments away from the wall for the 3.1D cyclically loaded pip pile.

Figure 6-9: Pile head load vs. pile head displacement for displacement increments towards the wall and inverted displacement increments away from the wall for the 1.5D cyclically loaded pip pile.

The 4.2D pile exhibits a very stiff initial response with little to no pile head displacement during the accumulation of load within the first loading cycle of the first displacement increment, but then experiences a sudden positive pile head displacement. This behavior is reflected by
results depicted in Figure 6-7. Because the lack of pile head displacement is present during loading in both loading directions, it is unlikely that this is a malfunction of the loading system or instrumentation. It is more likely that the soil immediately surrounding the pile was more compacted compared to the soils surrounding the other cyclically loaded piles, and after the first displacement increment was complete, the soil had dilated enough for more normal pile head displacements to occur. Similar stiff initial response for piles in compacted soil were observed during the static loading of the 24-inch diameter piles in this study.

Figures 6-10 and 6-11 compare the first cycle pile head load vs. displacement curves for the piles at 1.5, 3.1 and 4.2 and 5.3D from the MSE wall. For statically loaded piles, spacing from the wall at 3.9 pile diameters or greater generally resulted in no loss of stiffness in the soil-wall system. Any pile spacing less than 3.9 pile diameters generally resulted in decreased stiffness of the laterally load piles, and as the spacing decreased, lateral pile stiffness decreased as well. For the cyclically loaded piles, this behavior was not as clearly observed. For pile deflections caused by loads applied in the direction of the wall, there is no correlation between pile spacing from the wall and the lateral stiffness until the pile head was displaced by about a 0.25 inch. From that point onward, the correlation between pile spacing from the wall and lateral pile resistance becomes apparent. For given pile head deflection the lateral pile resistance clearly decreases at the pile spacing decreases.

For loads applied in the opposite direction from the wall, the load-deflection curves are generally similar for the piles at all spacing’s over the range of deflections tested. This result is expected because the soil resistance is essentially uniform in the direction away from the wall. The variations in lateral resistance at smaller deflections can likely be attributed to variations in compactive effort immediately adjacent to each pile.
Figure 6-10: Pile head deflection vs. pile head loads for the 12.75-inch cyclic pile head deflections towards the wall.

Figure 6-11: Pile head deflection vs. pile head loads for the 12.75-inch cyclic pile head deflections away from the wall.

Figures 6-12 through 6-15 depict the reduction of normalized load as a function of the number of loading cycles for each displacement increment for the piles spaced at 1.5, 3.1, 4.2
and 5.3D from the MSE wall, respectively. The fourth and sixth displacement increments for the 3.1D pile were omitted due to the small sample size available. The loads were normalized by dividing the load occurring at the peak of each loading cycle by the peak load of the first loading cycle. This was repeated for each displacement increment. Generally, as the number of loading cycles increased, the normalized load decreased. This suggests that less load was required to achieve the target actuator displacement as the number of loading cycles increased. On average the reduction in lateral load after 15 cycles was about 10 to 15%. A comparison of the average normalized load vs. number of load cycles for each pile is given in Figure 6-16. This is consistent with previous lateral load tests conducted on a single pile and pile groups in clay (Rollins et al. 2005). The variation from the average is likely attributable to variations in the deflection level produced by the actuators during the testing. Small increases in deflection would significantly increase the measured lateral resistance for a cycle, while small decreases in deflection would decrease the measured pile resistance.

![Figure 6-12: Decreasing normalized load with each loading cycle for the 5.3D pile.](image)
Figure 6-13: Decreasing normalized load with each loading cycle for the 4.2D pile.

Figure 6-14: Decreasing normalized load with each loading cycle for the 3.1D pile.
Figure 6-15: Decreasing normalized load with each loading cycle for the 1.5D pile.

Figure 6-16: Decreasing average normalized load of each displacement increment for each pile loaded in the direction of the wall.
6.2 Pile Head Rotation

As was mentioned previously, pile head rotation vs. pile head load was recorded for each pile spacing behind the wall.

The rotation was measured by placing string potentiometers on either side of the pipe pile. The string potentiometer on the load side of the pipe pile was placed three feet above the location of the applied load. The string potentiometer on the wall side of the pipe pile was placed directly level with the applied lateral load. Pile head rotation, \( \theta \), was calculated using Equation (5-1) and can be found in Section 5.

For each pile spacing, the string potentiometer readings recorded during the one-minute lock-off of the hydraulic jack were used to calculate the rotation. Typically, it is expected that the decrease of spacing between the pipe pile and the MSE wall increases the pile head rotation. As the lateral load is met with less lateral resistance, due to decreased spacing between the pile and the MSE wall, the pile head is less restrained, and can more freely rotate.

Figures 6-17 and 6-18 represent the pile head load vs. pile head rotation for the cyclically loaded piles for loads applied in the direction of the wall and away from the wall, respectively. Each data point along each curve represents the peak of the 1st or 15th loading cycle for each displacement increment.

Again, positive values for rotation and pile head load represent rotation and applied loading in the direction of the wall, and negative values represent rotation and applied loading in the opposite direction of the wall.
In addition, the pipe pile head becomes more susceptible to rotational deflection as more loading cycles are applied to the pile head. The behavior described further supports the finding that as spacing between the pile and the wall decreases, the lateral stiffness of the pile also decreases.

Figure 6-17: Pile Head Load vs. Pile Head Rotation for Loads Applied Towards The Wall.
A correlation between pile head rotation and the pile spacing behind the wall is not apparent when loads are applied in the opposite direction from the wall. Although the test results for the 1.5D pile show less rotation, most of the piles have relatively similar load versus rotation curves. Once again, this result is expected because there is not difference in the soil behind the four piles. A comparison between the results in Figure 6-17 and 6-18 indicates that the average pile head load-rotation curve away from the wall is similar to that for the piles head load-rotation curves in the direction of the wall for the piles at 5.3D and 4.1D. In contrast, the load-rotation curves in the direction of the wall are higher for the piles at 3.1D and 1.5D, than the average load-rotation away from the MSE wall. Once again, this result suggests that the presence of the wall is reducing the rotational stiffness (and lateral resistance) for piles spaced closer than about 4.0D from the MSE wall even during cyclic loading.
6.3 Pile Bending Moment Performance

Bending moments developed in the piles were obtained from strain gauges placed on the surface of the piles at varying depths. Strain gauges were placed on the compression and tension sides of the pile at depths of 2, 4, 6, 8, 9, 12, 15, and 18 feet below the ground surface. Several of the strain gauge labels were lost or destroyed during the installation process. To combat this issue, when an unlabeled strain gauge recorded a strain measurement, the depth of that strain gauge was assigned based on the depth that strain measurement would be expected to occur. The bending moments developed in the piles was calculated using Equation (5-2). Bending moment at the ground surface was computed as the pile head load multiplied by the load height above the ground surface.

The piles were intended to be installed with the strain gauges directly in-line with the lateral load, but during installation several of the piles rotated causing the strain gauges to not be in line with the lateral load as shown in Figure 5-9. This rotation was accounted for in Equation 5-2. When only one of the strain gauges at a certain depth was functioning correctly, the value recorded by the functioning strain gauge was doubled. The strain data for each pile type was taken at the one-minute load hold.

Figures 6-19 through 6-22 represent the bending moment experienced by the piles as a function of depth below the ground surface. Two plots were created for each cyclically loaded pile. One plot for loading cycles in the direction of the wall, and another plot for loading cycles in the opposite direction or away from the wall.

It was generally observed that the maximum pile bending moment occurred between a depth of 3 to 5 ft. Pile bending moments produced by laterally applied loads to the pile at 1.5D in
the direction of the wall were the only exceptions. This depth range is slightly shallower than that observed in previous studies performed by Besemdorfer (2015) and Budd (2016).

Additionally, it was observed when the piles were loaded in the direction of the wall, a decrease in pile bending moment was seen from the first to the fifteenth loading cycle. This was especially apparent in larger displacement increments that incorporated larger applied loads. When the pile was loaded away from the wall, an increase in pile bending from the first to the fifteenth loading cycle was generally observed. Earlier in this report, it was observed that as cyclic loading of the pile in the direction of the wall occurred, the system experienced a greater decrease in stiffness compared to when the pile was loaded in the opposite direction of the wall. The correlation between the system’s stiffness and the direction that the load is applied could play a part in the correlation between pile bending moments observed in the first and fifteenth loading cycles.

![Figure 6-19: 5.3D pile bending moment vs. depth curves for selected pile head loads for (a) loads applied towards the wall and (b) loads applied away from the wall. Values given in parentheses in the legend represent the load applied to the 5.3D pile head.](image)
Figure 6-20: 4.2D pile bending moment vs. depth curves for selected pile head loads for (a) loads applied towards the wall and (b) loads applied away from the wall. Values given in parentheses in the legend represent the load applied to the 4.2D pile head.

Figure 6-21: 3.1D pile bending moment vs. depth curves for selected pile head loads for (a) loads applied towards the wall and (b) loads applied away from the wall. Values given in parentheses in the legend represent the load applied to the 3.1D pile head.
Figure 6-22: 1.5 pile bending moment vs. depth curves for selected pile head loads for (a) loads applied towards the wall and (b) loads applied away from the wall. Values given in parentheses in the legend represent the load applied to the 1.5D pile head.

Figure 6-23 shows the maximum bending moment experienced by the pile vs. the load applied to the pile head. Each curve represents the maximum pile bending moment occurring during the first loading cycle of the first displacement increment. In Figure 6-23, it can be seen that the 1.5 and 3.1D piles experienced larger maximum bending moments at smaller pile head loads in comparison to the 4.2 and 5.3D piles. It can also be seen that the 4.2 and 5.3D piles experienced nearly identical maximum pile bending moments at any given pile head load.

Generally speaking, if the presence of the wall is having an effect on the system stiffness, the bending moments will increase as the pile spacing behind the wall decreases. The behavior depicted in Figure 6-23 further supports the presumption, that piles spaced 3.9D behind an MSE wall do not experience a reduction in lateral stiffness due to the presence of a wall.
Figure 6-23: Maximum pile bending moment vs. pile head load for cyclically load piles loaded in the direction of the wall.

6.4 Induced Tensile Force in the MSE Reinforcements

Strain gauges were installed on the soil reinforcements to measure the strain distribution on the reinforcements during cyclic lateral pile head loading. The measured strain was used to calculate the tensile force the soil reinforcements experienced. Steel welded wire soil reinforcements were used near the 12.75-inch pipe piles. Strain gauges were placed at distances of 0.5, 2, 3, 5, 8, 11, and 14 feet away from the wall along the reinforcement and on the top and bottom sides of the reinforcement. If one of the strain gauges malfunctioned, the measured strain from the strain gauge was considered to be the average value in Equation 6-1. When the strain gauges on both sides of the reinforcement malfunctioned, the data point was disregarded. The tensile force, \( T \), experienced in the reinforcements was calculated using the equation

\[
T = EA(\mu \varepsilon_{AVG})(10^{-6})B
\]  

(6 – 1)

Where:
T = the tensile force experienced by ribbed strip soil reinforcements

E = the modulus of elasticity of the ribbed strip soil reinforcement (29,000 ksi)

A = the cross-sectional area of the ribbed strip soil reinforcement (0.11 inches²)

με_{AVG} = the average micro strain between the strain gauge on the top and the strain gauge on the bottom of the ribbed strip soil reinforcement, and

B = n-1 where n is the number of longitudinal bars on the reinforcement grid.

The calculated tensile force in the welded wire steel reinforcements as a function of distance away from the wall is displayed in Figures 6-25 through 6-28 for piles spaced at 1.5, 3.1, 4.1 and 5.3D behind the MSE wall, respectively. Two figures are provided for each test pile. The part (a) of each figure for each pile depicts the results when the load was applied in the direction of the wall, and part (b) of each figure depicts the results when the load was applied in the direction away from the wall. Each displacement increment experienced by the pile is included in each figure, along with the first and fifteenth loading cycles of each displacement increment. Refer to Figure 6-24 for an illustrated location of each soil reinforcements. Figure 6-24 was previously provided by Figure 4-2 but has been re-inserted in this section for the convenience of the reader. Table 4-2 has also been re-inserted into this section and is provided by Table 6-1.

As was seen in previous studies performed by researchers in the Rollins research group, the maximum tensile force experienced by the reinforcements was generally located near the soil-pile interface on the wall side of the pile when loads were applied to the pile in the direction of the wall. This was not the case when loads were applied to the pile in the opposite direction of the wall. For this loading scenario, the maximum magnitude of force was also located near the
pile-soil interface, but the side that the maximum magnitude of force occurred varied between tests but were nearly equal on both sides.

It was also observed that the tensile force experienced by the reinforcements decreased from the first loading cycle at a given displacement to the fifteenth cycle when loads were applied in the direction of the wall. When the load was applied away from the wall, an increase in tensile force was seen from the first loading cycle to the fifteenth cycle for sections of the reinforcement undergoing a tensile load. However, for sections of the soil reinforcements experiencing compression when loads were applied in the direction away from the wall, a decrease in the maximum tensile force was generally observed.

The magnitude of tensile force experienced by the soil reinforcements was significantly less when loads were applied away from the wall in comparison to when loads were applied in the direction of the wall. This would further suggest that the lateral resistance of the soil-wall system is less in the direction of the wall than away from the wall. There is an apparent correlation between pile spacing from the wall and the magnitude of force experienced by the soil reinforcements. However, this correlation is likely a product of the larger loads that were applied to piles that were spaced further away from the wall.

Figure 6-24: Map of the soil reinforcements near the 12.75-inch piles. The soil reinforcement ID numbers are provided next to the corresponding soil reinforcements.
Table 6-1: Horizontal distance from the instrumented bar of the soil reinforcements to the center of the pile (in inches) near the 12.75-inch piles. The soil reinforcement ID numbers are provided in parenthesis.

<table>
<thead>
<tr>
<th>Test Pile</th>
<th>1.25 Feet</th>
<th>3.75 Feet</th>
<th>6.25 Feet</th>
</tr>
</thead>
<tbody>
<tr>
<td>1.5D</td>
<td>23 (9-2)</td>
<td>23 (3-4)</td>
<td>23 (9-6)</td>
</tr>
<tr>
<td></td>
<td>48 (8-2)</td>
<td>48 (8-4)</td>
<td>48 (8-6)</td>
</tr>
<tr>
<td>3.1D</td>
<td>24.4 (8-2)</td>
<td>24.4 (8-4)</td>
<td>24.4 (8-6)</td>
</tr>
<tr>
<td></td>
<td>36.9 (7-2)</td>
<td>36.9 (7-4)</td>
<td>36.9 (7-6)</td>
</tr>
<tr>
<td>4.2D</td>
<td>15 (7-2)</td>
<td>15 (7-4)</td>
<td>15 (7-6)</td>
</tr>
<tr>
<td></td>
<td>46.5 (6-2)</td>
<td>46.5 (6-4)</td>
<td>46.5 (6-6)</td>
</tr>
<tr>
<td>5.3D</td>
<td>21 (6-2)</td>
<td>21 (6-4)</td>
<td>21 (6-6)</td>
</tr>
<tr>
<td></td>
<td>52.5 (7-2)</td>
<td>52.5 (7-4)</td>
<td>52.5 (7-6)</td>
</tr>
</tbody>
</table>

Figure 6-25: Loads Experienced by Soil Reinforcement #5-2 for the 5.3D Cyclically Loaded Pile With Loads Applied (a) Towards The Wall and (b) Away From The Wall.
Figure 6-26: Loads Experienced by Soil Reinforcement #7-4 for the 4.2D Cyclically Loaded Pile With Loads Applied (a) Towards The Wall and (b) Away From The Wall.

Figure 6-27: Loads Experienced by Soil Reinforcement #9-6 for the 1.5D Cyclically Loaded Pile With Loads Applied (a) Towards The Wall and (b) Away From The Wall.
Figures 6-29 – 6-30 illustrate the maximum tensile force induced in the soil reinforcements as a function of depth below the ground surface, and transverse distance away from the center of the pile respectively. The first load cycle of each displacement increments for loads applied towards and away from the wall were used in these figures. In both part (a) and (b) of Figure 6-29, the largest magnitude of tensile forces experienced by the soil reinforcements occur during at a depth of 1.25 feet and decrease as depth increases. This suggests that depth is a governing factor in the force experienced by the soil reinforcements, and that the magnitude of the force is greater at more shallow depths. A similar phenomenon is apparent in Figure 6-30. In both part (a) and (b) of Figure 6-30, the largest magnitude of tensile force experienced by the soil reinforcements occur at smaller transverse distances between the reinforcements and the center of the pile and decreases as the transverse distance increases. This suggests that reinforcements closer to the laterally load pile is more susceptible to higher tensile loads in comparison to reinforcements a larger transverse distance.
Figure 6-29: Maximum tensile force induced in soil reinforcements as a function of depth below the ground surface for (a) loads applied in the direction of the wall and (b) loads applied in the opposite direction of the wall.

Figure 6-30: Maximum tensile force induced in soil reinforcements as a function of transverse distance between the reinforcements and the center of the pile for (a) loads applied in the direction of the wall and (b) loads applied in the opposite direction of the wall.

A statistical analysis to examine the suitability of the equation produced by Rollins (2018) to predict the tensile force experienced by the soil reinforcements was not performed for the
cyclically loaded piles. This is due to the differences in the nature of loading applied to the test piles in this study in comparison to the studies used to develop the equation. It is recommended that future researchers investigate the suitability of the equation proposed by Rollins (2018) (see Equation (2-4), but such a comparison is beyond the scope of this study.

6.5 Soil Performance

Vertical and lateral ground displacement occurred during the loading of each pipe pile. The following sections will detail the findings of the observed soil behaviors.

6.5.1 Lateral Ground Displacement

The lateral displacement of the ground was measured using string potentiometers. String potentiometers were attached to stake in the ground at varying locations between the pile and the wall depending on the pile spacing. For loading in the direction of the wall, it is generally expected for the greatest lateral soil displacement to take place near the face of the pile and then decrease as spacing between the pile and the wall decreases.

The lateral ground displacement of the soil near the 12.75-inch cyclically loaded piles is displayed in Figure 6-31 through 6-34 for test piles at 1.5, 3.1, 4.2 and 5.3D from the MSE wall, respectively. Each curve in the figures represents conditions at the peak of a loading cycle applied to the pile. Each curve identified first noted by the displacement increment, then the cycle number of that displacement increment. For example, the curve representing the lateral ground displacement for the 15th cycle of the 3rd displacement increment would be notated as 3-15. The data was divided into two graphs for each pile, based on the loading direction. A positive value for lateral ground displacement represents movement of the ground in the direction of the
wall, while a negative value for lateral ground displacement represent movement of the ground in the direction away from the wall.

Figure 6-31: Lateral ground displacement as a function of distance from the wall for loads applied to the 5.3D pile (a) in the direction of the wall and (b) in the direction away from the wall. Legend notation is as follows: displacement increment - cycle Number.

Figure 6-32: Lateral Ground Displacement as a Function of Distance from the Wall for Loads Applied to the 4.2D Pile in the Direction Away From the Wall. Legend Notation is as Follows: displacement Increment - cycle Number.
Figure 6-33: Lateral ground displacement as a function of distance from the wall for loads applied to the 3.1D pile in the direction of the wall. Legend notation is as follows: displacement increment - cycle number.

Figure 6-34: Lateral ground displacement as a function of distance from the wall for loads applied to the 1.5D pile (a) in the direction of the wall and (b) away from the wall. Legend notation is as follows: displacement increment - cycle number.
Like the statically-loaded 24-inch pipe piles, it was generally observed, that for loading cycles towards the wall, the lateral ground displacement was highest near the pile and decreased as the distance between the pile and the MSE wall decreased. When the pile was loaded in the opposite direction of the MSE wall, negative displacement values developed in the soil adjacent to the pile as it moved back into the gap left by pile. The lateral soil displacement between the pile and the MSE wall also decreased somewhat in response to the relaxation of the soil near the gap.

The 15th loading cycle of each displacement increment typically resulted in higher lateral ground displacements. This is likely caused by a reduction of soil stiffness caused by the previous loading cycles.

Normalized lateral ground displacements for each pile spacing when loads were applied in the direction and the opposite direction of the wall are presented through Figure 6-35. Similar to the 24-inch statically load piles, the x-axis of the figure was normalized by dividing the distance from the pile face by the pile diameter, while the y-axis was normalized by dividing the lateral ground displacement by the lateral displacement of the pile head during the lateral loading of the pile.

The first and fifteenth loading cycle of the maximum displacement increment applied to each pile was examined. The curves represent measurements taken during the first loading cycle of the final displacement increment for each pile.
Figure 6-35: Normalized lateral ground displacement for 12.75-inch piles loaded cyclically (a) in the direction of the wall and (b) in the opposite direction of the wall.

As was stated previously, the piles studied by Besendorfer (2015) and Budd (2016) experienced a decreasing normalized lateral displacement until the ground displacement reached nearly 0% of the pile head displacement at 3 to 4 pile diameters. In this study, an initial increase of normalized ground displacement was seen from the first to the second string potentiometer, and then a decrease of normalized ground displacement was seen from that point on. This could result from a weakening of the soil immediately in front of the soil due to cyclic loading. The normalized ground displacement typically trended to near zero between 3 to 4 pile diameters from the pile face during loading away from the wall.
6.5.2 Ground Heave

Measurements of vertical ground displacements were also gathered using an optical surveying level and rod. Measurements were taken before the lateral load was applied and then again at the end of testing. The difference between the two measurements is the vertical ground displacement.

Figure 6-36 represents the vertical ground displacement of the soil near the 12.75-inch cyclically-loaded piles. Placing the surveying equipment near the 12.75-inch piles was not an issue like it was for the 24-inch piles. The largest vertical ground displacements were seen near the pile-soil interface, and the vertical ground displacement decreased as the distance between the pile and the wall decreased. There were no obvious trends with pile spacing.

![Figure 6-36: Vertical Ground Displacement at the End of the Cyclically-Lateral Loading of Each 12.75” Pile.](image-url)
6.6 MSE Wall Panel Displacement

Displacement of the wall was measured using digital imagery correlation (DIC). A string potentiometer was also placed at the top of the wall, directly in line with the pile and applied load, to verify the results of the DIC. The computer program, ISTRA-4D was used to analyze the images produced by the DIC cameras. An explanation of the DIC apparatus is given in section 4.4.

Figure 6.37 depict the results of the DIC wall panel displacement analysis for the 12-inch cyclically loaded tests. Each subfigure represents the measurements of the wall panel displacement taken during the final loading cycle of the final displacement increment with loads being applied in the direction of the wall. After testing, it was discovered that the measurements taken during the testing of the 5.3D pile were corrupt, and for this cause, they have been omitted from the analysis.

Figure 6-37: DIC Imagery Results for the 12.75-Inch Cyclically-Loaded Piles
Similar to the 24-inch statically loaded pile testing, a correlation between the pile spacing from the wall and the amount of displacement experienced by the wall panels was observed for wall panels associated with the 12.75-inch cyclically loaded piles. It is also apparent that for test piles located at panel joints, (subfigures 1.5D and 4.2D) the wall panel displacements were concentrated around the wall panel joints. The presence of negative displacement values also indicates that the panels appeared to be rotating about a vertical axis during testing. In contrast, for the 3.1D test pile located in the center of a wall panel, the panel displacements are more uniformly distributed.

Figures 6-38 through 6-40 compare the wall displacement measurements recorded by the DIC imagery and the string potentiometer attached to the top of the wall during the 12.75-inch cyclically loaded tests. The agreement between the DIC and stringpot measurements is quite good for each of the test piles. Wall displacement is typically highest at the top of the wall and tends to increase somewhat at the spacing between the pile and the wall decreases. The wall displacements approach zero at between 6 and 8 feet below the top of the wall. This result suggests that the three levels of reinforcement that were instrumented with strain gauges will be carrying most of the load induced by the laterally loaded test piles.

The average wall deflection measured by a string potentiometer attached to the top of the wall panel for the 5.3D, 4.2D, 3.1D, and 1.5D was 0.27, 0.32, 0.36, and 0.47 respectively while each pile head was experiencing approximately 1.0 inch of deflection towards the wall. Previous researchers in the Rollins (2018) research group observed an average pile head deflection of approximately 0.1 inches during a pile head deflection of 1.0 inch. Figure 6-38 compares the maximum wall panel displacement during a pile head deflection of one inch as a function of normalized pile spacing between this studies and previous studies in the Rollins (2018) research
It is likely that the wall panel displacement seen in this study was significantly larger due to the larger applied pile head loads.

Figure 6-38: Comparison between maximum wall panel displacement during a one-inch pile head deflection as a function of normalized distance of the pile form the MSE wall between studies performed in the Rollins (2018) research group, and this study.
Figure 6-39: Wall Displacement vs. Depth for the 4.2D Pile

Figure 6-40: Wall Displacement vs. Depth for the 3.1D Pile
Figure 6-41: Wall Displacement vs. Depth for the 1.5D Pile
7 LATERAL PILE LOAD ANALYSIS

This chapter reports on the findings from lateral pile load analyses using the computer program LPILE (Isenhower et al. 2019). 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. As discussed in chapter 2, 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 lateral loaded pile by the finite difference method has been researched extensively by Reese and Matlock since the 1960s.

The primary purpose of the LPILE analyses was to back-calculate p-multipliers to account for the reduction in lateral pile resistance produced by the presence of the wall. Pile moment vs. depth curves computed in LPILE were also compared to field measured pile moment vs. depth curves to verify the correlation of the load deflection curves. A lateral pile load analysis was not performed in LPILE for the 12.75-inch cyclically loaded piles, because of the dynamic nature of their load deflection curves (see section 5.1). A correlation between the lateral stiffness of the pile and the normalized pile spacing was not evident enough to warrant such an analysis.
7.1 LPILE Parameters and Calibrations

LPILE requires several parameters and configurations to be provided before the analysis can be performed. The first set of parameters define the geometry and structural properties of the pile itself. The 24-inch pipe piles conformed to ASTM A500 Grade C standards with a minimum tensile strength of 46 psi and a minimum ultimate strength of 62 ksi. However, the manufacturer of the piles, Trinity Steel, Inc. indicates that the minimum tensile strength of the pile is 50 ksi. Table 7-1 summarizes the structural properties of the 24-inch test piles. The increased moment of inertia produced by the addition of the angle irons (36 ksi yield strength) welded onto the sides of the piles was neglected. During the loading process, several people present observed the pile yielding into an oval shape, despite the strut that was placed to mitigate the ovaling. The decrease in moment of inertia caused by the oval-shaped-yielding is thought to have negated the increased moment of inertia caused by the addition of the angle irons based on the LPILE analyses. In the LPILE analyses, the structural behavior was simulated using the user-defined lineal elastic model as this produced better agreement with the measured curves than the non-linear model.

Table 7-1: Properties of the 24-inch pipe piles for LPILE.

<table>
<thead>
<tr>
<th>Section Type</th>
<th>Steel Pipe Section</th>
</tr>
</thead>
<tbody>
<tr>
<td>Pipe Outside Diameter (in)</td>
<td>24</td>
</tr>
<tr>
<td>Pipe Wall Thickness (in)</td>
<td>0.5</td>
</tr>
<tr>
<td>Pile Length (ft)</td>
<td>40</td>
</tr>
<tr>
<td>Yield Stress of Pipe (psi)</td>
<td>50,000</td>
</tr>
<tr>
<td>Minimum Ultimate Strength (psi)</td>
<td>62,000</td>
</tr>
<tr>
<td>Elastic Modulus of Pipe (psi)</td>
<td>29,000,000</td>
</tr>
</tbody>
</table>
The second set of parameters defines the soil profile and properties of each layer. Table 7-2 summarizes the soil properties of the three layers in the profile. These layers include the native silty and sand below the original ground surface, the backfill soil from the ground surface to 13.75 feet, and the backfill placed during phase 3 from 13.75 feet to 20 feet above the ground surface. The soil properties of the bottom two soil layers in Table 7-2 were previously back-calculated by Luna (2016) using LPILE and the results from the lateral load tests with the H-piles. In this study, I back-calculated the soil properties of the top layer using LPILE, and the results from the lateral load tests on the 24-inch pipe pile at 5.1D behind the MSE wall where the wall should have limited effect on the lateral pile resistance. Specifically, I adjusted the modulus of subgrade reaction (k) and the soil friction angle (ϕ) until agreement was obtained between the measured and LPILE-computed pile head load vs. pile head displacement curves. In all cases, analyses were performed using the generic API Sand model proposed by O’Neill and Dunnivant (1982). The k value had the most influence on the curves at small deflection, while the ϕ value had the most influence at larger deflection. To achieve agreement between the measured and computed curves, a modulus of subgrade reaction of 5,500 lbs/in\(^3\) and a soil friction angle of 57.5° was required. The measured load-displacement curves are plotted against the computed curve in Figure 7-5 and the agreement is very good. Both the back-calculated k and ϕ values are significantly higher than would typically be used for most applications involving piles. There are several possible explanations for why higher soil parameters were necessary.

First, during the soil compaction process in phase 3, the soil was consistently compacted to 95% of the standard Proctor maximum density in the zone between the test piles and the wall as explained in chapter 3. In addition, jumping jack compactors were used immediately around the test piles rather than plate compactors. Although 95% relative compaction is typically
specified for MSE wall backfills, density standards are often relaxed or density test are not performed near the wall to prevent distortion of the wall during construction. For example, the backfill in this zone during phase 1 and 2 testing was typically only compacted to between 88% and 91% of the standard Proctor using plate compactors only. This process created a very stiff load-deflection curve relative to previous load tests, particularly at very small displacements. Similar very high initial stiffness values were observed for lateral load tests on drilled shaft foundations in compacted gravel near MSE walls reported by Pierson et al (2009).

The reasonability of using such a high value for the internal friction angle is supported by Reese et al (1974), who proposed that the internal friction angle ($\phi$) of a highly compacted soil can be estimated by measuring the angle of the failure plane ($\alpha$) that is produced in the soil after the lateral loading of driven piles. Figure 7-1 provides a schematic of this phenomenon. Measurements of the angle displayed in Figure 7-1 were made for each of the 24-inch pipe piles. Figure 5-4 provides a photograph of the failure planes (highlighted with orange spray paint) produced after laterally loading the 5.1D pile. Figure 7-2 provides a photograph of the angle of the failure planes immediately surrounding the pile.

![Figure 7-1: Schematic providing the relationship between the angle of the backfill's failure plane after the lateral loading of a test pile, and the backfill's internal angle of friction.](image-url)
In Figure 7-2 a failure plane angle of 57 degrees and 45 degrees can be seen on either side of the 5.1D test pile. The difference between the two angles is likely caused by the varying stiffness of the soil on either side of the pile. On the side of the pile where the failure plane angle is 45 degrees, the soil along that failure plane was previously failed by lateral load testing of the 2.0D and 3.0D piles. In contrast, on the side of the pile where the failure plane angle is 57 degrees no testing had been performed along the failure plane. As will be discussed, the friction angle used to calibrate the soil model in LPILE was 57.5 degrees. This value is very similar to the 57 degrees of the measured failure plane.
Secondly, LPILE is an empirical finite difference program with p-y curve shapes that are calibrated over a limited range of relative densities. The parameters in LPILE are reasonably well calibrated for looser soils that typically surround piles in natural soil, but rather poorly calibrated for dense compacted soils surrounding piles at approach fills near bridge abutments. For very dense soil conditions, such as those surrounding the 24-inch piles, the curves defining $k$ and $\phi$ are not even defined as illustrated in Figure 7-3. If user-specified values exceed the limited range of values in the LPILE model, then LPILE extrapolates a solution correlating to the higher parameters values from known solutions within its known parameter range. It is possible that the extrapolated solution produced by LPILE does not represent reality, and that using unrealistic parameter values forces LPILE into producing a solution that is more closely aligned with observed measurements. For this reason, the $k$ value was used as a parameter to achieve an optimal correlation between the measured and computer data, rather than as a descriptive parameter of the soil conditions.

Finally, the failure wedge model as shown in Figure 2-3 that was developed by Reese et al (1974) is based on a planar Rankine failure surface rather than a log-spiral failure surface typically observed for passive failure. While the Rankine failure surface might provide reasonable estimates of passive resistance for looser sands with lower friction angles where a general shear failure surface does not develop, the passive resistance would likely become progressively more underestimated for denser soils with higher friction angles. Therefore, the need for a higher friction angle to achieve realistic load-deflection curves could result from using the Rankine failure model that fails to consider the higher passive resistance that would be computed by a log-spiral model for a given friction angle.
Table 7-2: Soil layers and parameters used for 24-inch pipe pile analysis in LPILE.

<table>
<thead>
<tr>
<th>Soil Type</th>
<th>Layer Thickness (ft)</th>
<th>Unit Weight (pcf)</th>
<th>Friction Angle, $\phi$ (Degrees)</th>
<th>Modulus of Subgrade Reaction, $k$ (pci)</th>
</tr>
</thead>
<tbody>
<tr>
<td>API Sand (O'Niell)</td>
<td>0.83-7.08</td>
<td>130</td>
<td>57.5</td>
<td>5500</td>
</tr>
<tr>
<td>API Sand (O'Niell)</td>
<td>7.08-21</td>
<td>127.5</td>
<td>38</td>
<td>205</td>
</tr>
<tr>
<td>API Sand (O’Niell)</td>
<td>21-50</td>
<td>125</td>
<td>34</td>
<td>115</td>
</tr>
</tbody>
</table>

Figure 7-3: Correlation Between Modulus of Subgrade Reaction, Friction Angle, and Relative Density for Soils Modeled Using the API Sand Setting in LPILE.
7.2 Development of P-Multipliers

One primary purpose of this study was to produce reduction factors, or p multipliers dependent on the spacing between the test pile and the wall for laterally loaded piles. P-multipliers have been used to account for reduced lateral resistance from pile group interaction (Brown et al. 1988) and for reduced resistance in liquefied sand (Brandberg et al. 2007). In this case, the p-multipliers ($P_{\text{MULT}}$) account for reduced lateral soil resistance for a pile near an MSE wall relative to a pile far enough away to be unaffected. The P-multiplier would decrease the ordinate for each p-y curve by a constant factor based on spacing behind the wall as illustrated in Figure 7-4.

![Figure 7-4: Illustration of p-multiplier concept for reducing the p-y curve away from the MSE wall (aw) to an appropriate p-y curve near the wall (nw).](image-url)
Based on previous testing (Rollins et al. 2013), the farthest from the MSE wall (typically about 5.0D) was assumed to be relatively unaffected by the presence of the wall, a p-multiplier of 1.0 was assumed for this case indicating no wall interaction. Iterations of the LPILE analysis were performed until the computed force-deflection curve agreed well with the measured force-deflection curve. Between each iteration, soil properties were adjusted to improve the agreement between the computed vs. measured curves. For the LPILE analyses, the 10 pile head loads listed in Table 7-3 were used to obtain agreement with the measure displacements listed in the table.

For each pile located closer to the wall, the back-calculated soil parameters obtained for the pile at 5.1D behind the wall were then kept constant and a single p-multiplier was back-calculated, by trial and error, to produce agreement with measured load-deflection curve for that pile. The pile head loads along with the measured displacements for each pile spacing are summarized in Table 7-3.

Table 7-4 displays the p-multiplier determined for each normalized pile spacing behind the wall. Figure 7-5 also provides plots of measured pile head load vs. deflection for the piles at 4.1D, 3.0D and 2.0D spacing behind the wall compared with the predicted curve using the p-multipliers in Table 7-4. Considering the simplicity of the approach and the use of a constant p-multiplier with depth, the agreement between the two curves is very good. As has been seen in previous research, as the spacing between the pile and the wall increases, the p-multiplier associated with that pile also increases.
Table 7-3: Lateral loads and measured pile head deflections for each pile spacing used in the LPILE assessment of p-multipliers.

<table>
<thead>
<tr>
<th>LPile Loading Scenarios</th>
<th>Pile Head Deflection (in)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>2.0D</td>
</tr>
<tr>
<td>Case 1: 10 kips</td>
<td>0.01892</td>
</tr>
<tr>
<td>Case 2: 30 kips</td>
<td>0.06307</td>
</tr>
<tr>
<td>Case 3: 50 kips</td>
<td>0.1218</td>
</tr>
<tr>
<td>Case 4: 70 kips</td>
<td>0.2016</td>
</tr>
<tr>
<td>Case 5: 100 kips</td>
<td>0.379</td>
</tr>
<tr>
<td>Case 6: 120 kips</td>
<td>0.5433</td>
</tr>
<tr>
<td>Case 7: 140 kips</td>
<td>0.7419</td>
</tr>
<tr>
<td>Case 8: 160 kips</td>
<td>1.1242</td>
</tr>
<tr>
<td>Case 9: 180 kips</td>
<td>No Solution</td>
</tr>
<tr>
<td>Case 10: 200 kips</td>
<td>No Solution</td>
</tr>
</tbody>
</table>

Table 7-4: P-Multipliers Corresponding to Each Laterally Loaded Pile.

<table>
<thead>
<tr>
<th>Pile</th>
<th>P-Multiplier</th>
</tr>
</thead>
<tbody>
<tr>
<td>5.1D</td>
<td>1.0</td>
</tr>
<tr>
<td>4.1D</td>
<td>0.84</td>
</tr>
<tr>
<td>3.0D</td>
<td>0.55</td>
</tr>
<tr>
<td>2.0D</td>
<td>0.44</td>
</tr>
</tbody>
</table>

Figure 7-5: Comparison between computed pile head load vs. pile head deflection curves with a p-multiplier and measured pile head load vs. pile head deflection curves for each pile head spacing.
7.3 P-Multipliers and Pile Spacing Curves

Figure 7-6 provides a plot of the back-calculated p-multipliers vs. normalized pile spacing behind the wall for every full-scale test performed by researchers in the Rollins research group including the p-multipliers obtained from this study. A total of 37 data points are included in Figure 7-6. The p-multipliers ($P_{\text{MSE}}$) based on proximity to the MSE wall obtained from the LPILE analyses in this study generally fall within the scatter of data about the best-fit curve defined by Rollins et al. (2018) and given by the equation displayed in Figure 2-6. The $P_{\text{MSE}}$ values for the piles at 5.1D and 2.0D are very close to the best-fit line; however, the $P_{\text{MSE}}$ values for the piles at 4.1D and 3.0D are somewhat lower than would be predicted by the equation. As indicated in Chapter 5, the pile at 2.0D was tested first when there would have been no overlapping shear planes from previous pile load tests in the vicinity to influence it. Likewise, the pile at 5.1D was loaded when no adjacent overlapping shear planes were present. In contrast, both the piles at 4.1 and 3.0D were tested adjacent to previously tests piles where there were clearly overlapping shear planes as shown in Figure 5-5. It appears likely that these overlapping shear planes led to some reduction in the lateral pile resistance and somewhat lower $P_{\text{MSE}}$ values than would otherwise have been the case.

The potential for overlapping shear planes was greater for the 24-inch test piles relative to the previously tested 12.75-inch test piles for two reasons. First, the 24-inch test piles left less clear space between the test piles which were both spaced at about 5 feet spacing in the direction parallel to the wall. This placed the start of the shear planes fanning out from the sides of the test piles in closer proximity. Secondly, the fan angle of the shear planes was increased owing to the higher relative compaction of the backfill soil around the 24-inch piles in comparison with that for the 12.75-inch piles.
Given the good agreement between computed $P_{\text{MSE}}$ values for the 24-inch piles with the best-fit equation previously developed for the 12.75-inch piles, it seems reasonable to conclude that the pile diameter has relatively little effect on the computed $p$-multipliers after normalizing the pile spacing by pile diameter. Previous studies have concluded that the $p$-multipliers were relatively unaffected by the reinforcing length to height ratio and the reinforcement type if they were designed according to AASHTO code requirements (Luna 2016, Rollins et al. 2018).

Despite the good agreement with the previous equation observed visually, the complete data set was also used to determine a best-fit line for $P_{\text{MSE}}$ vs. normalized pile spacing. The linear equation correlating to the best-fit line is shown in Figure 7-6 is given by Equation 7-1. The $R^2$ value for this equation is 0.75. One data point collected by Besendorfer (2015) was omitted from the linear regression analysis due to its repetitive nature, as two test piles were located nearly the same distance from the wall and produced the same $P_{\text{MSE}}$ value. These points had $(S/D)$ ratios of 2.8 and 2.9 and $p$-multipliers of 1.

\[
P_{\text{MSE}} = 0.30 \left(\frac{S}{D}\right)^{-0.18} \quad \text{for } \frac{S}{D} < 3.97
\]

\[
P_{\text{MSE}} = 1.0 \quad \text{for } \frac{S}{D} > 3.97
\]

Where $P_{\text{MSE}} = p$-multiplier to account for pile interaction with the MSE wall,

$S =$ distance from the center of the pile to the back face of the MSE wall, and

$D =$ outside diameter of the pile.

Again, a $p$-multiplier value of one indicates that the proximity of the wall has no effect on the lateral soil resistance while undergoing lateral pile head loading. A $p$-multiplier of one can also be thought of as a case where the soil reinforcements provide sufficient resistance to
compensate for any loss in lateral soil resistance near the wall as shown in Figure 7-4, the
difference between $P_{\text{MSE}}$ values computed with the new regression equation and the previous
equation are typically less than about 0.01. It is expected that as more data is collected by future
researchers, that the linear correlation between $p$-multipliers and the normalized distance
between the center of the pile and the wall will continue to improve.

Figure 7-6: Correlation between the normalized distance for the wall and $p$-multipliers computed
using LPILE. The data points were taken from studies performed by Pierson (2009), Price (2012),
Nelson (2013), Han (2014), Hatch (2014), Besendorfer (2015), Budd (2016), Luna (2016) and this
study. A few outlier data points were omitted from this figure.

7.4 Pile Bending Moment Curves

As was mentioned in section 5.3, pile bending moment was measured using strain gauges
that were attached to either side of the pile at various depths. Additional information on the
calculated bending moments can be found in that section. Pile bending moments were also computed using LPILE to verify the p-multipliers produced by the program. After the p-multipliers were established, the p-multipliers for each pile head spacing were used as inputs (see Table 7-4) to compute pile bending moments. Figures 7-7 through 7-10 provide a comparison between the field measured pile bending moments, and the pile bending moments computed in LPILE for selected applied loads. The curves for a given pile head load have the same line color, but curves for measured moment are solid while curves for computed moment are dashed.

The agreement between the shapes of the measured and computed bending moment vs. depth curves are very good for the piles at 4.1D and 5.1D. Both the magnitude of the maximum moment and the depth to the maximum moment are well predicted. However, the agreement becomes progressively poorer as the normalized pile spacing decreases to 3.0D and 2.0D. Several strain gauges attached to the piles malfunction or were damaged during the installation in this event, the data collected at that depth was omitted. These omissions made it especially difficult to produce reliable comparisons between the measured and computed bending moments for the 3.0D pile as shown in Figure 7-8. No data from depths between five and sixteen feet were salvageable. However, the few available measured points correlate reasonably well with the points calculated in LPILE. Additional strain gauge malfunctions can be seen in Figure 7-10. The strain gauges located at depths between four and eight feet measured the highest moments for the first three loading scenarios, but suddenly see a decrease in measured bending moment from the third to the fourth loading scenario. These values are included in Figure 7-10 but should be noted as outliers.
Figure 7-7: Comparison of pile bending moments measured during full scale testing and pile bending moments computed using LPILE at selected pile head loads for the 5.1D pile.

Figure 7-8: Comparison of pile bending moments measured during full scale testing and pile bending moments computed using LPILE at selected pile head loads for the 4.1D pile.
Figure 7.9: Comparison of pile bending moments measured during full scale testing and pile bending moments computed using LPILE at selected pile head loads for the 3.0D pile.

Figure 7.10: Comparison of pile bending moments measured during full scale testing and pile bending moments computed using LPILE at selected pile head loads for the 2.0D pile.
Figures 7-11 through 7-14 compare the computed and measured maximum pile bending moment as a function of pile head load. As noted previously, the agreement between these curves for the 2.0D and the 3.0D piles were not as good as the agreement for the piles in the 4.1D and the 5.1D tests. This is partially attributable to the malfunction of the strain gauges on the 2.0D and 3.0D piles as was previously discussed. The agreement between the measured and computed curves for the 4.1D and 5.1D piles was excellent, with difference being less than about 10% in general. For the 3.0D pile, LPILE typically overestimated the maximum moment by 25% or less for pile head loads 150 kips or less with error increase somewhat with pile head load. For the 2.0D pile, LPILE generally over-predicted maximum moments with errors of 25% to 75%.

Figure 7-11: Comparison of the computed and measured maximum bending moment experienced by the 5.1D pile.
Figure 7-12: Comparison of the computed and measured maximum bending moment experienced by the 4.1D pile.

Figure 7-13: Comparison of the computed and measured maximum bending moment experienced by the 3.0D pile.
Figure 7-14: Comparison of the computed and measured maximum bending moment experienced by the 2.0D pile.
8 CONCLUSIONS

Full-scale load tests were performed on four statically loaded 24”x0.5” pipe piles spaced at distances of 2.0, 3.0, 4.1, and 5.1 pile diameters behind a 20-ft high MSE wall. Full-scale load tests were also performed on four cyclically 12.75”x0.375” pipe piles spaced at distances of 1.5, 3.1, 4.2 and 5.3 pile diameters behind the same wall. Ribbed strip soil reinforcements were used in the vicinity of the statically loaded piles, while welded wire soil reinforcements were used in the vicinity of the cyclically loaded piles. The primary objectives of this study were as follows:

1. Measure lateral resistance of cyclically and statically loaded pipe piles at varying installation distances from a MSE wall.

2. Determine forces experienced by welded wire steel reinforcements induced by cyclic lateral loads applied at the pile head, and forces experienced by ribbed strip steel reinforcements induced by static lateral loads also applied at the pile head.

3. Measure displacement and deformation experienced by the MSE wall during cyclic and static lateral loads.

4. Develop p-multipliers to account for reduced soil resistance as a function of proximity of the pile to the MSE wall for both the static load tests and the cyclic load tests.
5. Examine the validity of previously developed equations that aim to assist designers in predicting the maximum tensile force in the soil reinforcements during lateral pile head loading.

8.1 Conclusions Regarding Statically Loaded 24-Inch Piles

8.1.1 Conclusions Regarding Lateral Pile Resistance

1. Lateral resistance of the piles decreased as spacing between the wall and the pile decreased. The average percent decrease of lateral resistance in comparison to the 5.1D pile for the 4.1D, 3.0D, and 2.0D piles are 11.1%, 34.2%, and 37.0% respectively.

2. P-multipliers were computed for each of the statically loaded piles. The p-multipliers corresponding to the 2.0D, 3.0D, 4.1D and 5.1D are 0.44, 0.55, 0.84, and 1.0 respectively. The p-multipliers for the 3.0D and 4.1D are lower than what was expected according to the equation developed by previous researchers in the Rollins (2018) research group. This is likely caused by overlapping failure planes of the backfill between the test piles and the MSE wall. Figure 5.5 located in section 5.1 illustrates this phenomenon.

3. The p-multiplier versus normalized distance curves were not significantly impacted by pile diameter, L/H ratio, pile shape, or soil reinforcement type.

4. A p-multiplier value of 1.0 is recommended for piles with normalized spacing’s of 3.9 pile diameters or greater form the wall. The recommended p-multiplier value decreased linearly as a function of normalized pile spacing starting at 3.9 pile diameters. Equation 7-1 provides the recommended design equation for predicting p-multipliers as a
function of the pile’s normalized distance from the wall. Equation 7-1 has an $R^2$ of 0.75.

8.1.2 Conclusions Regarding Forces Induced in The Reinforcements

1. The maximum tensile force induced in the soil reinforcements generally occurred near the wall side of the pile face. Behind the pile, the tensile force decreased with distance from the wall. The tensile force experienced by the soil reinforcements increased as the load applied to the pile increased.

2. For a given load, the maximum tensile force experienced by reinforcements increased as depth increased from 1.25 to 3.75 feet. At greater depths, the maximum tensile force experienced by the reinforcements began to decrease. This could be explained by reduced lateral stress with depth.

3. A clear correlation between the maximum tensile force induced in the soil reinforcements as a function of transverse distance of the reinforcement from the center of the pile was not evident. In previous studies performed by researchers in the Rollins (2018) research group typically noted that the maximum tensile force induced in the soil reinforcements decreased as transverse distance of the soil reinforcements from the center of the pile increased. The differences in these observations is likely caused by the larger failure “foot-prints” caused by the lateral loading of the 24-inch pipe piles in comparison to the smaller failure “foot-prints” induced by the lateral loading of smaller diameter piles.

4. Equation 5-4, modified from Rollins (2018) was found to be adequate for predicting the maximum tensile force experienced by the ribbed strip reinforcements during the
static loading of the 24” pipe piles, particularly for lower loads. About 65% of the measured forces measured in this study fell within the one standard deviation boundary of the proposed equation. The equation produced by Rollins (2018) is not compatible with pile head loads much larger than 115-120 kips. A strong correlation would exist between measured and predicted tensile forces if the positive slope of the quadratic curve were extrapolated to include larger pile head loads. The validity of the equation weakens as the spacing between the pile head and the wall increase.

8.1.3 Conclusions Regarding Wall Deflections

1. The average wall deflection measured by a string potentiometer attached to the top of the wall panel for the 5.1D, 4.1D, 3.0D, and 2.0D was 0.41, 0.33, 0.52, and 0.59 respectively while each pile head was experiencing 1.0 inches of deflection. Previous researchers in the Rollins (2018) research group observed an average pile head deflection of approximately 0.1 inches during a pile head deflection of 1.0 inch. It is likely that the wall panel displacement seen in this study was significantly larger due to the larger applied pile head loads, and the accompanying shear planes.

8.2 Conclusions Regarding Cyclically Loaded 12.75-Inch Piles

8.2.1 Conclusions Regarding Lateral Pile Resistance

1. For deflection levels less than about 0.5 inches, load-deflection curves were similar when loading towards or away from the wall indication little pile-wall interaction.

2. For loading toward the wall, the load-deflection curves for piles at 5.3D and 4.2D were quite similar, while piles at 3.0D and 2.0D exhibited reduced lateral resistance as
deflection exceeded 0.5 inches. These results suggest that piles spaced closer than about 4D experience reduced lateral resistance during cyclic loading due to the presence of the MSE wall as was observed for static, monotonic loading.

3. For loading away from the wall, there was relatively little difference between the piles spaced at different distances from the MSE wall face because the soil in that direction was uniform and consistently compacted.

4. As the number of loading cycles increased, the normalized load decreased. After 15 loading cycles, the lateral pile resistance was 10% to 15% lower than for the first cycle. This reduction is 25% to 33% less than for cyclically loaded pile in clay (Rollins et al. 2005).

8.2.2 Conclusions Regarding Forces Induced in The Reinforcements

1. The maximum tensile force experienced by the soil reinforcements generally occurred near the wall side of the pile face when the lateral loads were applied in the direction of the wall. Behind the pile, the tensile force decreased as the distance from the wall increased. The tensile force experienced by the soil reinforcements increased as the load applied to the pile increased.

2. When loads were applied away from the wall during cyclic loading, the maximum magnitude of tensile force decreased but was still located near the pile-soil interface. In addition, the reinforcements behind the test pile experienced compressive loading.

3. The reinforcements located at a depth of 3.75 feet consistently experienced the greatest tensile force. The magnitude of the maximum tensile force experienced by the
reinforcement decreased as transverse distance between a given pile and the reinforcement increased.

4. The tensile force experienced by soil reinforcements decreased from the first loading cycle at a given displacement to the fifteenth cycle when loads were applied in the direction of the wall. When loads were applied away from the wall, tensile force increased from the first loading cycle to the fifteenth for sections of the reinforcement undergoing a tensile load. But, for sections of the soil reinforcements experiencing compression when loads were applied away from the wall, the compressive force decreased.

5. The magnitude of tensile force experienced by the soil reinforcements is significantly less when loads are applied in the opposite direction of the wall in comparison to when loads were applied in the direction of the wall.

8.2.3 Conclusions Regarding Wall Deflections

1. The average wall deflection measured by a string potentiometer attached to the top of the wall panel for the 5.3D, 4.2D, 3.1D, and 1.5D was 0.27, 0.32, 0.36, and 0.47 respectively while each pile head was experiencing approximately 1.0 inch of deflection towards the wall. Previous researchers in the Rollins (2018) research group observed an average pile head deflection of approximately 0.1 inches during a pile head deflection of 1.0 inch. It is likely that the wall panel displacement seen in this study was significantly larger due to the larger applied pile head loads.
8.3 Recommendations for Further Research

The compaction of the backfill was a governing variable in the behavior of many of the components of the system. It is recommended that future researchers either seek to reduce the variability of compaction of the backfill or perform full-scale testing with various levels of compaction. During a construction process, achieving a homogenously compacted backfill is difficult. A study that could elaborate on the effects of varying compaction levels on lateral pile resistance would be beneficial for practicality purposes. Such a study would perform full-scale testing with piles installed equal distances away from the MSE wall while varying the compaction of the backfill surrounding each pile. The data gathered from these full-scale tests could then be used to calculate p-multipliers and reinforcement loads as a function of relative compaction of a given pile spacing in contrast to those of other pile spacing’s.

During full-scale testing of the 24-Inch pipe piles, shear planes created by the lateral loading of some piles overlapped. This overlapping appeared to reduce the lateral resistance of adjacent piles that were loaded subsequently. Although the piles in this study were constrained to be placed at 5 feet on centers, it is recommended that future research be performed on piles that are spaced far enough apart in the transverse direction (perhaps 2D) to avoid these overlapping failure planes.

Equation (2-8) proposed by Rollins et al. (2018) aims to predict the maximum tensile force experienced by a given soil reinforcement. About 65% of the data points collected during the lateral loading of the 24-inch piles fell within one standard deviation of Equation (2-8). Nevertheless, it was observed that many of the points collected in this study were not predicted well by the equation because the applied loads and distances away from the MSE wall were significantly larger than the applied loads and pile-wall distances used to develop Equation (2-8).
Further investigations into modification of this equation to included larger laterally applied loads, larger distances between the center of the pile and the wall is recommended.

Additionally, Equation (2-8) proposed by Rollins et al. (2018) did not included any data points for piles undergoing cyclic lateral loading. Cycle loading of piles behind MSE walls supporting bridge abutments is a common phenomenon. These cyclic loads are caused by freeze thaw cycles, traffic loads, and earthquake loads. It is recommended that the tensile force measured in the strain gauges from the cyclic loading portion of this study be included as part of the regression analysis to modify Equation (2-8).
REFERENCES


Han, J. J. C. (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.


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Source: “Test Site”. 40°27’11.23” N and 111°53’57.99” W. GOOGLE EARTH. April 9, 2013. February 6, 201
9.1 Appendix A – Soil Reinforcement Load Curves for The 24-Inch Piles

Figure 10-1: Loads experienced by soil reinforcement (2.0D, 4 feet deep, group #5)
Figure 10-2: Loads experienced by soil reinforcement (2.0D, 4 feet deep, group #8)

Figure 10-3: Loads experienced by soil reinforcement (2.0D, 6 feet deep, group #8)
Figure 10-4: Loads experienced by soil reinforcement (2.0D, 6 feet deep, group #7)

Figure 10-5: Loads experienced by soil reinforcement (3.0D, 4 feet deep, group #8)
Figure 10-6: Loads experienced by soil reinforcement (3.0D, 6 feet deep, group #8)

Figure 10-7: Loads experienced by soil reinforcement (3.0D, 6 feet deep, group #7)
Figure 10-8: Loads experienced by soil reinforcement (4.1D, 2 feet deep, group #3)

Figure 10-9: Loads experienced by soil reinforcement (4.1D, 2 feet deep, group #4)
Figure 10-10: Loads experienced by soil reinforcement (4.1D, 4 feet deep, group #1)

Figure 10-11: Loads experienced by soil reinforcement (4.1D, 4 feet deep, group #6)
Figure 10-12: Loads experienced by soil reinforcement (4.1D, 6 feet deep, group #5)

Figure 10-13: Loads experienced by soil reinforcement (4.1D, 6 feet deep, group #6)
Figure 10-14: Loads experienced by soil reinforcement (4.1D, 2 feet deep, group #4)

Figure 10-15: Loads experienced by soil reinforcement (4.1D, 4 feet deep, group #4)
Figure 10-16: Loads experienced by soil reinforcement (4.1D, 6 feet deep, group #6)

Figure 10-17: Loads experienced by soil reinforcement (4.1D, 6 feet deep, group #5)
9.2 Appendix B – Comparison of the Continuous Results of Equation (5-4) and Measured Maximum Tensile Forces Experienced by Soil Reinforcements.

Figure 10-18: Evaluation of the suitability of the equation developed by Rollins (2018) to predict the maximum tensile force experienced by a soil reinforcement. Square data points represent measurements taken from soil reinforcement #5-4 near the 2.0D pile.

Figure 10-19: Evaluation of the suitability of the equation developed by Rollins (2018) to predict the maximum tensile force experienced by a soil reinforcement. Square data points represent measurements taken from soil reinforcement #7-6 near the 2.0D pile.
Figure 10-20: Evaluation of the suitability of the equation developed by Rollins (2018) to predict the maximum tensile force experienced by a soil reinforcement. Square data points represent measurements taken from soil reinforcement #8-4 near the 2.0D pile.

Figure 10-21: Evaluation of the suitability of the equation developed by Rollins (2018) to predict the maximum tensile force experienced by a soil reinforcement. Square data points represent measurements taken from soil reinforcement #1-2 near the 3.0D pile.
Figure 10-22: Evaluation of the suitability of the equation developed by Rollins (2018) to predict the maximum tensile force experienced by a soil reinforcement. Square data points represent measurements taken from soil reinforcement #2-2 near the 3.0D pile.

Figure 10-23: Evaluation of the suitability of the equation developed by Rollins (2018) to predict the maximum tensile force experienced by a soil reinforcement. Square data points represent measurements taken from soil reinforcement #5-4 near the 3.0D pile.
Figure 10-24: Evaluation of the suitability of the equation developed by Rollins (2018) to predict the maximum tensile force experienced by a soil reinforcement. Square data points represent measurements taken from soil reinforcement #7-6 near the 3.0D pile.

Figure 10-25: Evaluation of the suitability of the equation developed by Rollins (2018) to predict the maximum tensile force experienced by a soil reinforcement. Square data points represent measurements taken from soil reinforcement #8-4 near the 3.0D pile.
Figure 10-26: Evaluation of the suitability of the equation developed by Rollins (2018) to predict the maximum tensile force experienced by a soil reinforcement. Square data points represent measurements taken from soil reinforcement #8-6 near the 3.0D pile.

Figure 10-27: Evaluation of the suitability of the equation developed by Rollins (2018) to predict the maximum tensile force experienced by a soil reinforcement. Square data points represent measurements taken from soil reinforcement #1-4 near the 4.1D pile.
Figure 10-28: Evaluation of the suitability of the equation developed by Rollins (2018) to predict the maximum tensile force experienced by a soil reinforcement. Square data points represent measurements taken from soil reinforcement #3-2 near the 4.1D pile.

Figure 10-29: Evaluation of the suitability of the equation developed by Rollins (2018) to predict the maximum tensile force experienced by a soil reinforcement. Square data points represent measurements taken from soil reinforcement #5-6 near the 4.1D pile.
Figure 10-30: Evaluation of the suitability of the equation developed by Rollins (2018) to predict the maximum tensile force experienced by a soil reinforcement. Square data points represent measurements taken from soil reinforcement #6-4 near the 4.1D pile.

Figure 10-31: Evaluation of the suitability of the equation developed by Rollins (2018) to predict the maximum tensile force experienced by a soil reinforcement. Square data points represent measurements taken from soil reinforcement #6-6 near the 4.1D pile.
Figure 10-32: Evaluation of the suitability of the equation developed by Rollins (2018) to predict the maximum tensile force experienced by a soil reinforcement. Square data points represent measurements taken from soil reinforcement #4-2 near the 4.1D pile.

Figure 10-33: Evaluation of the suitability of the equation developed by Rollins (2018) to predict the maximum tensile force experienced by a soil reinforcement. Square data points represent measurements taken from soil reinforcement #3-2 near the 5.1D pile.
Figure 10-34: Evaluation of the suitability of the equation developed by Rollins (2018) to predict the maximum tensile force experienced by a soil reinforcement. Square data points represent measurements taken from soil reinforcement #4-2 near the 5.1D pile.

Figure 10-35: Evaluation of the suitability of the equation developed by Rollins (2018) to predict the maximum tensile force experienced by a soil reinforcement. Square data points represent measurements taken from soil reinforcement #4-4 near the 5.1D pile.
Figure 10-36: Evaluation of the suitability of the equation developed by Rollins (2018) to predict the maximum tensile force experienced by a soil reinforcement. Square data points represent measurements taken from soil reinforcement #5-6 near the 5.1D pile.

Figure 10-37: Evaluation of the suitability of the equation developed by Rollins (2018) to predict the maximum tensile force experienced by a soil reinforcement. Square data points represent measurements taken from soil reinforcement #6-4 near the 5.1D pile.
Figure 10-38: Evaluation of the suitability of the equation developed by Rollins (2018) to predict the maximum tensile force experienced by a soil reinforcement. Square data points represent measurements taken from soil reinforcement #6-6 near the 5.1D pile.

9.3 Appendix C – Soil Reinforcement Load Curves for The 12.75-Inch Piles

Figure 10-39: Loads experienced by soil reinforcement #8-4 for the 1.5D cyclically loaded pile with loads applied towards the wall.
Figure 10-40: Loads experienced by soil reinforcement #8-4 for the 1.5D cyclically loaded pile with loads applied away from the wall.

Figure 10-41: Loads experienced by soil reinforcement #8-6 for the 1.5D cyclically loaded pile with loads applied towards the wall.
Figure 10-42: Loads experienced by soil reinforcement #8-6 for the 1.5D cyclically loaded pile with loads applied away from the wall.

Figure 10-43: Loads experienced by soil reinforcement #9-2 for the 1.5D cyclically loaded pile with loads applied towards the wall.
Figure 10-44: Loads experienced by soil reinforcement #9-2 for the 1.5D cyclically loaded pile with loads applied away from the wall.

Figure 10-45: Loads experienced by soil reinforcement #9-4 for the 1.5D cyclically loaded pile with loads applied towards the wall.
Figure 10-46: Loads experienced by soil reinforcement #9-4 for the 1.5D cyclically loaded pile with loads applied away from the wall.

Figure 10-47: Loads experienced by soil reinforcement #9-6 for the 1.5D cyclically loaded pile with loads applied towards the wall.
Figure 10-48: Loads experienced by soil reinforcement #9-6 for the 1.5D cyclically loaded pile with loads applied away from the wall.

Figure 10-49: Loads experienced by soil reinforcement #7-2 for the 3.1D cyclically loaded pile with loads applied towards the wall.
Figure 10-50: Loads experienced by soil reinforcement #7-2 for the 3.1D cyclically loaded pile with loads applied away from the wall.

Figure 10-51: Loads experienced by soil reinforcement #7-6 for the 3.1D cyclically loaded pile with loads applied towards the wall.
Figure 10-52: Loads experienced by soil reinforcement #7-2 for the 3.1D cyclically loaded pile with loads applied away from the wall.

Figure 10-53: Loads experienced by soil reinforcement #8-4 for the 3.1D cyclically loaded pile with loads applied towards the wall.
Figure 10-54: Loads experienced by soil reinforcement #7-2 for the 3.1D cyclically loaded pile with loads applied away from the wall.

Figure 10-55: Loads experienced by soil reinforcement #8-6 for the 3.1D cyclically loaded pile with loads applied towards the wall.
Figure 10-56: Loads experienced by soil reinforcement #8-6 for the 3.1D cyclically loaded pile with loads applied away from the wall.

Figure 10-57: Loads experienced by soil reinforcement #6-2 for the 4.2D cyclically loaded pile with loads applied towards the wall.
Figure 10-58: Loads experienced by soil reinforcement #6-2 for the 4.2D cyclically loaded pile with loads applied away from the wall.

Figure 10-59: Loads experienced by soil reinforcement #6-4 for the 4.2D cyclically loaded pile with loads applied towards the wall.
Figure 10-60: Loads experienced by soil reinforcement #6-4 for the 4.2D cyclically loaded pile with loads applied towards the wall.

Figure 10-61: Loads experienced by soil reinforcement #6-6 for the 4.2D cyclically loaded pile with loads applied towards the wall.
Figure 10-62: Loads experienced by soil reinforcement #6-6 for the 4.2D cyclically loaded pile with loads applied towards the wall.

Figure 10-63: Loads experienced by soil reinforcement #7-2 for the 4.2D cyclically loaded pile with loads applied towards the wall.
Figure 10-64: Loads experienced by soil reinforcement #7-2 for the 4.2D cyclically loaded pile with loads applied towards the wall.

Figure 10-65: Loads experienced by soil reinforcement #7-6 for the 4.2D cyclically loaded pile with loads applied towards the wall.
Figure 10-66: Loads experienced by soil reinforcement #7-6 for the 4.2D cyclically loaded pile with loads applied towards the wall.

Figure 10-67: Loads experienced by soil reinforcement #5-4 for the 5.3D cyclically loaded pile with loads applied towards the wall.
Figure 10-68:Loads experienced by soil reinforcement #5-4 for the 5.3D cyclically loaded pile with loads applied towards the wall.

Figure 10-69: Loads experienced by soil reinforcement #5-6 for the 5.3D cyclically loaded pile with loads applied towards the wall.
Figure 10-70: Loads experienced by soil reinforcement #5-6 for the 5.3D cyclically loaded pile with loads applied towards the wall.

Figure 10-71: Loads experienced by soil reinforcement #6-2 for the 5.3D cyclically loaded pile with loads applied towards the wall.
Figure 10-72: Loads experienced by soil reinforcement #6-2 for the 5.3D cyclically loaded pile with loads applied towards the wall.

Figure 10-73: Loads experienced by soil reinforcement #6-3 for the 5.3D cyclically loaded pile with loads applied towards the wall.
Figure 10-74: Loads experienced by soil reinforcement #6-3 for the 5.3D cyclically loaded pile with loads applied towards the wall.

Figure 10-75: Loads experienced by soil reinforcement #6-4 for the 5.3D cyclically loaded pile with loads applied towards the wall.
Figure 10-76: Loads experienced by soil reinforcement #6-4 for the 5.3D cyclically loaded pile with loads applied towards the wall.