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Lateral response of 0.6m pipe piles near a Mechanically Stabilized Earth (MSE) wall

Réponse latérale des pieux de tuyau de 0,6 m près d'un mur de terre mécaniquement stabilisé

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ABSTRACT: Pile foundations near bridge abutments surrounded by Mechanically Stabilized Earth (MSE) walls must often resist lateral loads produced by earthquakes and thermal variation. Previous full-scale testing with 0.3-m diameter piles has indicated that reduced lateral pile resistance near an MSE wall can be reasonably predicted using p-multipliers for piles within four pile diameters of the wall. In this study full-scale lateral load tests were performed on 0.6-m diameter piles at 2, 3, 4, and 5 pile diameters behind a 6-m tall MSE wall with steel reinforcements to determine the effect of pile diameter on lateral resistance. The larger diameter piles carried three to four times the lateral load and induced three to four times more displacement on the MSE wall relative to the 0.3-m diameter piles. Despite the larger load, the p-multipliers obtained for the 0.6-m diameter piles were similar to those predicted using p-multiplier vs. normalized distance relationships developed for the 0.3-m diameter piles. This simple p-multiplier approach was able to produce reasonable agreement between the measured and computed pile head lateral load-displacement curves.

RÉSUMÉ :
Les fondations sur pieux près des culées de ponts entourées de murs en terre mécaniquement statabilisés (MSE) doivent souvent résister aux charges latérales produites par les tremblements de terre et les variations thermiques. Des essais à grande échelle antérieurs avec des pieux de 0,3 m de diamètre ont indiqué que la réduction de la résistance des pieux latéraux près d'un mur MSE peut être raisonnablement prédite en utilisant des multiplicateurs p dans des pieux à moins de quatre pieux de diamètre du mur. Dans cette étude, des tests de charge latérale à grande échelle ont été effectués sur des pieux de 0,6 m de diamètre à 2, 3, 4 et 5 diamètres de pieux derrière un mur MSE de 6 m de haut pour déterminer l'effet du diamètre du pieu sur la résistance latérale. Les pieux de plus grand diamètre supportaient trois à quatre fois la charge latérale et induisaient trois à quatre fois plus de déplacement sur le mur du MSE par rapport aux pieux de 0,3 m de diamètre. Malgré la charge plus importante, les p-multiplicateurs obtenus pour les pieux de 0,6 m de diamètre étaient similaires à ceux prédits en utilisant le p-multiplicateur par rapport à normaliser les relations de distance développées pour les pieux de 0,3 m de diamètre. Ce concept simple de multiplicateur p a permis de produire une concordance raisonnable entre les courbes de déplacement de charge-déplacement latéral de la tête de pieu mesurées.

KEYWORDS: Bridge abutments, Lateral Load, Lateral Pile Resistance, Mechanically Stabilized Earth Walls, Piles

1 INTRODUCTION.

Pile foundations for bridge abutments with wrap-around Mechanically Stabilized Earth (MSE) walls must often resist lateral loads produced by earthquakes and thermal variations as shown in Figure 1.

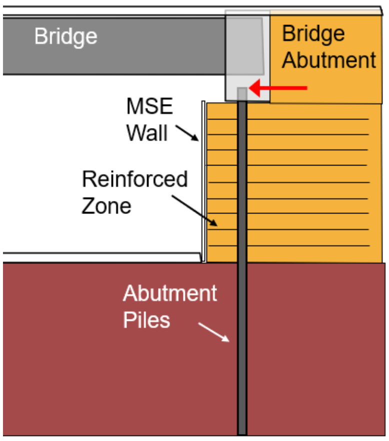

Figure 1 Schematic drawing of abutment piles near MSE wall.

Because of the lack of large-scale tests, there has been little guidance for engineers in assessing the lateral resistance of piles located close to these MSE walls. As a result, some engineers neglect any soil resistance, which increases the required number of piles or pile diameter, increasing the foundation cost. Others drive the piles further away from the wall to reduce interaction effects, which increases the bridge span and the bridge cost. Still others assume some reduction in lateral resistance owing to the proximity of the pile to the MSE wall. Unfortunately, there has been little guidance available to determine what the reduction factor should be. The lateral pile resistance would be expected to be more significantly reduced as the pile is located closer to the MSE wall as observed by Pierson et al. (2009).

To provide design guidance relative to lateral pile resistance near MSE walls with steel reinforcements, Rollins et al. (2018) performed a series of large-scale lateral load tests on test piles located 2, 3, 4, and 5 pile diameters (D) from the center for the pile to the back of the MSE wall. Tests were performed on round, square and H-piles located behind a 4.6-m and 6-m tall wall with a gravelly sand backfill compacted to between 88 and 92% of the standard Proctor density between the test piles and the MSE wall.

Rollins et al. (2018) proposed a p-multiplier (P_{MSE}) to reduce the p value in a p-y curve for a pile located away from the wall (P_{AW}) to that for a pile located near the wall (P_{NW}) as illustrated in Figure 2. P-multipliers have been used in the past to consider reduction in lateral pile resistance due to pile group interaction (Brown et al. 1988) and for reduced resistance in liquefied sand (Brandenberg et al. 2007). In this case, P_{MSE} accounts for reduced lateral resistance for a pile near an MSE wall relative to a pile far enough away to be unaffected.

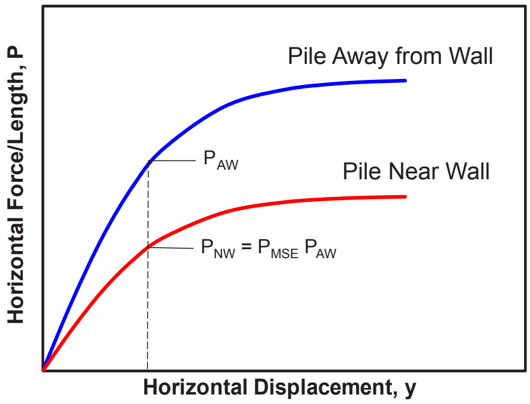

Figure 2. Illustration of the use of p-multiplier (P_{MSE}) to reduce the p value for the p-y curve away from the wall (P_{AW}) to obtain the p value for the p-y curve near the wall (P_{NW}).

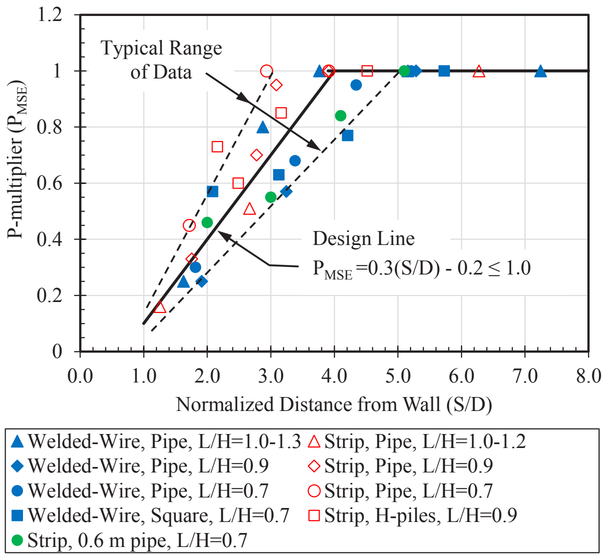

Figure 3. P-multiplier (P_{MSE}) vs. normalized pile spacing (S/D) using 0.3-m pipe, square, and H-pile data (Rollins et al. 2018) along with data for 0.6-m pipe pile from this study. (L=reinforcement length and H is effective wall height.)

Based on lateral pile analyses with the computer model LPILE (Isenhower et al. 2019), Rollins et al. (2018) back-calculated P_{MSE} values to produce agreement with the measured pile head load-deflection curves obtained from field tests. The p-multipliers are plotted against the normalized distance from the MSE wall as shown in Figure 3. The distance from the back of the wall to the center of the pile (S) was divided by the pile width/diameter (D) to obtain the normalized distance (S/D).

The best fit linear relationship for the p-multiplier, P_{MSE}, is given by the equations:

$$P_{MSE} = 0.30(S/D) - 0.20 \leq 1.0 \quad \text{For } S/D < 4.0,\ P_{MSE} > 0 \quad (1a)$$

$$P_{MSE} = 1.0 \quad \text{For } S/D \geq 4.0 \quad (1b)$$

where S is the distance from the center of the pile to the back of the MSE wall and D is the pile diameter. P is greater than 0.

In this regression analysis, the R^2 value is 0.73 for S/D less than 4. Equation (1a) indicates that a p-multiplier of 1.0 will result for a normalized distance greater than 4.0. For normalized distances less than 4.0, the p-multipliers decrease nearly linearly with normalized distance. A p-multiplier of 1.0 indicates that the presence of the wall has no significant effect on the lateral resistance or, alternatively, that the MSE reinforcement is sufficient to provide as much lateral restraint as if the wall were not present.

Although this p-multiplier vs. normalized distance curve provides important guidance related to pile-wall interaction near MSE walls, it is based on single piles where the diameter is typically 0.30 m. The curve is not validated for larger diameter piles. Conceivably, larger diameter piles could induce greater lateral force causing the MSE wall to deflect more and produce less lateral restraint. This could lead to lower p-multipliers.

To investigate the effect of pile diameter on p-multipliers for lateral pile resistance near MSE walls, an additional set of lateral load tests were performed on 0.6-m diameter pipe piles. This paper describes the testing and analysis procedures, then compares the p-multipliers with those for 0.3-m diameter piles.

2 TEST LAYOUT AND TESTING PROCEDURES

The lateral load testing was conducted behind an MSE wall specifically constructed for this purpose at a gravel pit near Lehi, Utah, USA. Figure 4 provides a photograph of the MSE test wall which is about 6 m tall and about 30 m long at the top. The wall itself is composed of 15-cm thick concrete wall panels that are nominally 3 m long by 1.5 m tall.

Figure 4. Photograph of full-scale MSE test wall 6 m tall and 30 m long constructed for lateral pile load tests near Lehi, Utah, USA.

Figure 5 provides a drawing showing the location of the test piles behind the MSE wall along with the locations of ribbed strip reinforcements. The reinforcements were 50 mm wide and 5 mm thick and extended to a distance of 5.5 m behind the wall. The reinforcements were spaced at 0.76 m both vertically and horizontally.

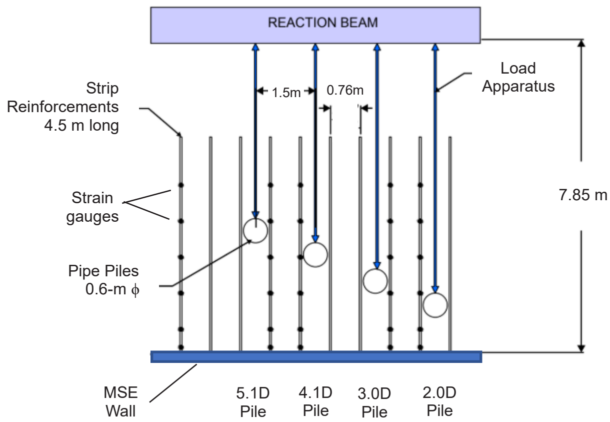

Figure 5 Layout of test piles relative to MSE wall and reinforcements.

Test piles were once again installed at normalized distances of approximately 2, 3, 4, and 5 pile diameters normal to the back face of the wall and were spaced at about 2.5 pile diameters parallel to the wall. Space constraints prevented the piles from being spaced at 5 diameters parallel to the wall as was the case for the 0.3-m diameter test piles.

The steel pipe piles had an outside diameter of 0.6 m, a wall thickness of 12.75 mm, and a yield strength of 393 MPa. The test piles were driven to a depth of 12.2 m below the top of the compacted backfill which was 6.2 m into the native silty sand. The test piles were driven open-ended but plugged at a depth of about 6 m. With a length to diameter ratio of 20, the piles would be classified as long piles.

The backfill soil consisted of silty sand with gravel classifying as SM material according to the Unified Soil Classification System (USCS). Two particle size distribution curves for the backfill are plotted in Figure 6. The silty sand backfill was compacted to an average of 95% of the standard Proctor maximum unit weight in the bottom 4.1 m of the backfill and close to 100% in the upper 1.9 m. The standard Proctor maximum density was 19.9 kN/m^3 at an optimum moisture content of 9%. The native soil below the MSE wall was silty sand to sandy silt.

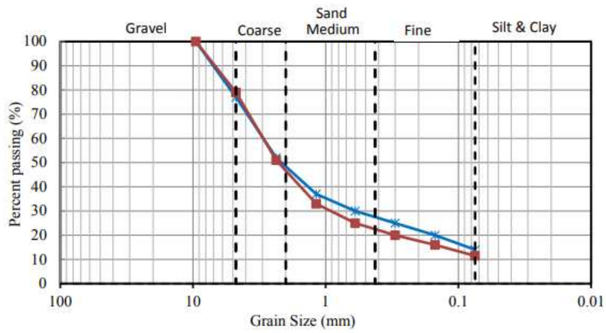

Figure 6 Particle size distribution curves for backfill material behind MSE wall.

Prior to lateral load testing, a surcharge load was placed on the fill between the test piles and the reaction piles. The surcharge load was meant to mimic the pressure produced by the approach fill behind the test pile and abutment as illustrated in Figure 1. A surcharge of about 29 kPa was produced by stacking pre-cast concrete blocks on top of each other. This is equivalent to a fill height behind the abutment of about 1.5 m. Based on the effective wall height of 7.5 m with the surcharge, the reinforcement length to wall height ratio (L/H) is 0.73, which is common for static loadings where L/H is the minimum requirement based on AASHTO standards (AASHTO, 2012). The concrete blocks typically occupied a width of about 5 m parallel to the wall and extended back to the reaction beam shown in Figure 5.

Load was applied to the test piles at heights ranging from 200 to 300 mm above the ground surface using a hydraulic jack. The hydraulic jack was attached to a strut that reacted against the reaction beam that was located beyond the reinforced zone so that it would not affect the load in the reinforcements. The reaction beam was in turn restrained against excessive lateral deflection by four large diameter reaction piles. Hemispherical end platens were positioned between the jack and the test pile to prevent eccentric loading. The piles at 5.1D and 2.0D behind the wall were loaded first and did not have interference with shear planes from lateral loading of adjacent test piles. In contrast, the test piles at 3.0D and 4.1D were loaded after loading adjacent test piles. This likely led to some reduction in lateral resistance which is difficult to quantify.

Pile head load was measured using a load cell, while pile head deflection was measured with a string potentiometer located at the height of the applied load and attached to an independent reference frame. We performed the test using a deflection-controlled approach. Load was applied incrementally to produce pile head deflection increments of about 6.35 mm. When the target deflection was reached, the fluid from the jack was locked off for three minutes while the pile head load and deflection came to equilibrium. There was very little change in deflection between 1 and 3 minutes so the load and deflection are based on the readings at the one-minute hold.

3 LATERAL LOAD TEST RESULTS

A plot showing the pile head load vs. deflection curves for the four test piles at 5.1, 4.1, 3.0 and 2.0 pile diameters (D) are provided in Figure 7. There was relatively little reduction in the lateral load-deflection curves for the pile at 4.1D relative to the pile at 5.1D. This behavior is consistent with previous research (Rollins et al. 2018) which indicates that piles located further than about 4.1D experience little reduction in resistance owing to the presence of the MSE wall, as noted previously. However, there is a substantial decrease in lateral resistance for the test piles located at 3.0D and 2.0D behind the wall relative to the pile at 5.1D, as expected. This result indicates that the presence of the wall is significantly reducing the lateral resistance for these piles located closer than 4D from the wall. However, the reduction in resistance for the pile at 3D behind the wall is greater than would be expected and is close to the curve for the pile at 2D spacing. This may be attributable to overlapping shear planes with the test pile at 2D that was previously loaded. Because of the larger diameter and somewhat denser backfill soil, the 0.6-m diameter piles carried three to four times the load of the 0.3-m diameter piles.

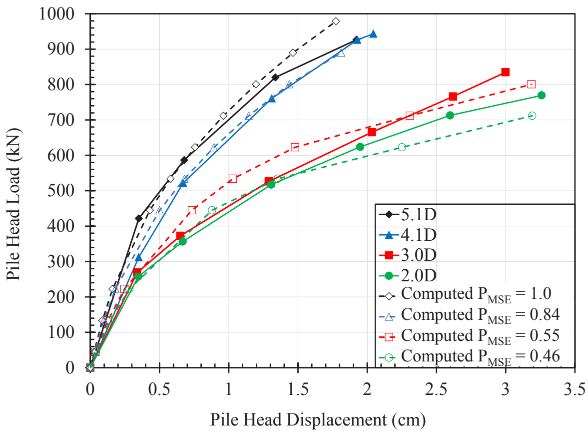

Figure 7 Pile head lateral load vs. deflection curves for the 0.60-m diameter test piles located at 5.1, 4.1, 3.0, and 2.0 pile diameters (D) behind the MSE wall.

4 LATERAL LOAD ANALYSIS OF PILE AT 5.1D

In this study, we performed 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 Sullivan (1980) at the University of Texas and is one of the most widely used programs for the lateral pile load analysis. LPILE uses the finite difference method (Reese and Matlock, 1960), to iteratively solve for the deflection, shear force, and bending moment of the pile with depth.

In LPILE, the pile is modeled as a beam while the soil is modeled using non-linear springs or p-y curves. The p value is the horizontal force/length along the pile and y is the horizontal pile deflection at a given depth. Because the backfill behind the MSE wall is granular, we used the API (American Petroleum Institute) p-y curve model for sand developed by O'Neill and Murchison (1983). This model requires the effective unit weight (γ'), drained friction angle (ϕ), and horizontal soil stiffness factor (k) for each soil layer.

Based on previous testing reported by Rollins et al. 2018, piles further than about 4D behind the wall were relatively unaffected by the presence of the wall. Therefore, a p-multiplier of 1.0, indicating no wall interaction, was assumed for the test pile located at 5.1D in this case. Iterations of the LPILE analysis were performed until the computed force-deflection curve agreed well with the measured load-deflection curve. Between each iteration, soil properties were adjusted to improve the agreement between the computed and measured curves. Using the API p-y curve model, the k value had the most influence on the computed load-deflection curve at small deflections while the ϕ value had the most effect at larger deflections. The k and ϕ values for the backfill 1.9 to 6 m below the top of the wall had been previously back-calculated using LPILE based on lateral load tests in this layer reported by Rollins and Bustamante (2015). Thus, the adjustments in this study only involved properties of the backfill soil in the top 1.9 m of the backfill. The soil properties used to compute the load-deflection curves are summarized in Table 1.

Table 1. Summary of soil properties for each layer in the LPILE model.

Depth Interval (m)	Moist Unit weight, γ (kN/m^3)	Soil Friction angle, ϕ (degrees)	Horizontal stiffness factor, k (kN/m^3)
0-1.9	20.4	57	948
1.9-6.0	19.8	45	68
6.0-12.5	19.5	34	31

The computed pile head load-deflection curve for the pile located at 5.1D behind the MSE wall is presented in Figure 7 in comparison with the measured curve. Overall, the agreement is very good, indicating that the soil model is reasonable. Although the back-calculated friction angle for the top layer is quite high, it is consistent with the observed fan angle for the shear planes during the lateral load test, which are typically about equal to the friction angle. (Reese et al. 1974). In addition, the friction angle is consistent with the back-calculated value for fixed-head pile tests conducted in the same layer (Rollins and Flores, 2020). The high friction angle may be attributable to dilation of the relatively dense sand as low confining pressure (shallow depths) as reported by Bolton (1986).

The k value is poorly defined for dense compacted sands with high friction angles. Therefore, it was largely determined by curve fitting with the initial segment of the load-deflection curve and is far higher than predicted by the API relationships. A very high k was required in the top layer to match the very steep initial load-deflection curves for the 0.6-m diameter test piles. Similar very high initial load-deflection curves were reported by Pierson et al. (2003) for lateral load tests on rockfill compacted around 0.9-m diameter drilled shafts.

5 DETERMINATION OF P-MULTIPLIERS

For each test pile located closer to the wall, the back-calculated soil parameters obtained for the pile at 5.1D behind the wall were then held constant and a single p-multiplier was back-calculated, by trial and error, to produce agreement with the measured load-deflection curve for that pile. Table 2 summarizes the p-multipliers (P_{MSE}) determined for the test piles at each normalized pile spacing behind the wall. Figure 7 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 curves using the p-multipliers in Table 2.

Considering the simplicity of the approach and the use of a constant p-multiplier with depth, the agreement between the measured and computed curves is generally reasonable (within 10%) except for the pile at 3.0D behind the wall. As has been reported in previous research, as the spacing between the pile and the wall increases, the p-multiplier associated with that pile also increases. The back-calculated p-multipliers derived from the lateral load tests on the 0.6-m diameter test piles are also plotted in Figure 3 in comparison with those obtained for the 0.3-m diameter test piles (Rollins et al. 2018).

Table 2. Summary of back-calculated p-multipliers versus normalized pile distance behind the MSE wall.

Normalized Distance Behind MSE wall (Pile Diameters, D)	P-multiplier P_{MSE}
5.1D	1.0
4.1D	0.84
3.0D	0.55
2.0D	0.46

Although the p-multipliers for the 0.6-m diameter piles all fall within the range of data for all the tests, two points fall below the best-fit design line. This result suggests the possibility that the additional load applied by the 0.6-m diameter piles may have led to some reduction in the p-multipliers on average. However, a closer examination of the test results suggests that this is not likely the case. The lower p-multipliers were computed for the test piles at 4.1 and 3.0D where an adjacent lateral load test had previously been performed. Because the test piles could only be spaced at 2.5 pile diameters center-to-center parallel to the wall and the compacted fill had a high friction angle, the shear planes fanning out from the pile during lateral loading clearly overlapped. Therefore, the overlapping shear planes likely led to the lower lateral resistance in the subsequent adjacent lateral load test and artificially reduced the p-multiplier. Based on these observations, the p-multipliers for the 0.6-m diameter lateral load tests appear to be consistent with those for the 0.3-m diameter tests.

The agreement in the p-multipliers considering the large difference in lateral resistance between the 0.3 and 0.6 m piles is somewhat surprising given the higher loads. However, it must be recognized that p-multipliers are a measure of the relative lateral soil resistance for a pile close to the wall relative to one far away, rather than a comparison of the absolute lateral pile resistance, which clearly does change with diameter.

6 MSE WALL DEFLECTION FROM LATERAL LOAD

Although the p-multipliers appear to be reasonably consistent for the two pile diameters investigated, the lateral displacement of the MSE wall was not consistent. In previous lateral load tests involving the 0.30-m diameter piles, the maximum wall displacement was about 2.5 mm, with a standard deviation of 2mm, for a pile head deflection of 25 mm. However, the maximum wall deflections for the 0.6-m diameter piles at 5.1D, 4.1D, 3.0D, and 2.0D behind the wall were 6.9, 8.1, 9.1, and 11.9 mm, respectively while each pile head was deflecting 25 mm deflection towards the wall. These wall deflections are 2.75 to 4.75 times larger than those for the 0.3-m diameter test piles. They are also generally higher than the mean plus two standard deviation maximum wall deflections from the 0.3-m diameter pile tests. This increase in maximum wall deflection can likely be attributed to the fact that the lateral loads carried by the 0.6-m diameter piles are three to four times higher than those for the 0.3-m diameter piles as noted previously.

7 CONCLUSIONS

Based on the results of the field testing and analysis work described in this paper, the following conclusions have been made:

1. Lateral pile resistance for the 0.6-m diameter test piles was three to four times higher than for previous tests involving the 0.3-m diameter test piles.
2. As a result of the higher lateral loads, the maximum lateral displacement of the MSE wall was three to four times higher for the 0.6-m diameter piles than for the 0.3-m diameter piles.
3. Lateral pile resistance for the test pile at 4.1D behind the wall was slightly lower than that for the pile at 5.1D, but the lateral pile resistance decreased significantly for the test piles at 3.0D and 2.0D behind the wall. These results are consistent with previous tests indicating that lateral pile resistance decreases for piles driven less than 4D behind an MSE wall (Rollins et al. 2018). Some reduction in the lateral resistance for the pile at 3.0D spacing was observed owing to prior loading of an adjacent pile.
4. The p-multipliers (P_{MSE}) for the 0.6-m diameter piles back-calculated from full-scale lateral load tests on 0.6-m diameter piles in this study are consistent with P_{MSE} vs. normalized distance (S/D) curves developed previously for 0.3-m diameter piles (Rollins et al. 2018). This agreement is possible because p-multipliers are based on relative lateral soil resistance rather than the absolute lateral resistance.
5. The simple p-multiplier approach can produce reasonable agreement between the measured and computed load-deflection curves for piles driven close to an MSE wall. This conclusion presumes that the lateral load-deflection curve away from the wall is well-calibrated to the soil conditions.

8 ACKNOWLEDGEMENTS

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