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LATERAL RESISTANCE OF ABUTMENT PILES BEHIND MECHANICALLY STABILIZED EARTH (MSE) WALLS: FINAL SUMMARY REPORT #1 FOR STUDY TPF-5(272)

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Final Summary Report #1
August 2018

DISCLAIMER

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16. Abstract
A full-scale mechanically stabilized earth (MSE) wall was constructed and 24 lateral load tests were conducted on piles spaced at distances of about 2 to 5 pile diameters (D) from the back face of the wall to the center of the pile. Sixteen of the tests involved pipe piles, four tests involved H-piles, and four tests involved square piles. Lateral resistance decreased significantly when piles were located closer than about 4D from the wall. P-multipliers were developed to account for the reduction in lateral soil resistance as a function of normalized distance behind the wall using results from these tests and eight previous full-scale tests. P-multipliers were approximately one for piles located further than about 4D from the wall, while p-multipliers decreased linearly as distance to the wall decreased. P-multipliers were not significantly affected by differences in reinforcement length to height (L/H) ratio, reinforcing type, or pile shape, but provided a reasonable means for estimating reduced lateral pile resistance. Measured tensile force in the reinforcements tended to reach a peak near the location of the pile rather than at the wall face, indicating that the reinforcements were anchoring the wall against lateral movement. Multi-linear regression equations were developed to predict maximum tensile force in the reinforcements. Tensile force tended to increase as pile head load and vertical stress increased, and it tended to decrease with increases in transverse distance from the load point and normalized distance from the wall.

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Laterally loaded piles, piles, p-multiplier, MSE wall, ribbed strip reinforcement, welded wire reinforcement, bridge abutments, p-y curve, tensile force, reinforcement load

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UNIT CONVERSION FACTORS

SI* (MODERN METRIC) CONVERSION FACTORS				
APPROXIMATE CONVERSIONS TO SI UNITS				
Symbol	**When You Know**	**Multiply By**	**To Find**	**Symbol**
		LENGTH		
in	inches	25.4	millimeters	mm
ft	feet	0.305	meters	m
yd	yards	0.914	meters	m
mi	miles	1.61	kilometers	km
		AREA		
in^2	square inches	645.2	square millimeters	mm^2
ft^2	square feet	0.093	square meters	m^2
yd^2	square yard	0.836	square meters	m^2
ac	acres	0.405	hectares	ha
mi^2	square miles	2.59	square kilometers	km^2
		VOLUME		
fl oz	fluid ounces	29.57	milliliters	mL
gal	gallons	3.785	liters	L
ft^3	cubic feet	0.028	cubic meters	m^3
yd^3	cubic yards	0.765	cubic meters	m^3
		NOTE: volumes greater than 1000 L shall be shown in m^3		
		MASS		
oz	ounces	28.35	grams	g
lb	pounds	0.454	kilograms	kg
T	short tons (2000 lb)	0.907	megagrams (or "metric ton")	Mg (or "t")
		TEMPERATURE (exact degrees)		
°F	Fahrenheit	5 (F-32)/9 or (F-32)/1.8	Celsius	°C
		ILLUMINATION		
fc	foot-candles	10.76	lux	lx
fl	foot-Lamberts	3.426	candela/m^2	cd/m^2
		FORCE and PRESSURE or STRESS		
lbf	poundforce	4.45	newtons	N
lbf/in^2	poundforce per square inch	6.89	kilopascals	kPa
APPROXIMATE CONVERSIONS FROM SI UNITS				
Symbol	**When You Know**	**Multiply By**	**To Find**	**Symbol**
		LENGTH		
mm	millimeters	0.039	inches	in
m	meters	3.28	feet	ft
m	meters	1.09	yards	yd
km	kilometers	0.621	miles	mi
		AREA		
mm^2	square millimeters	0.0016	square inches	in^2
m^2	square meters	10.764	square feet	ft^2
m^2	square meters	1.195	square yards	yd^2
ha	hectares	2.47	acres	ac
km^2	square kilometers	0.386	square miles	mi^2
		VOLUME		
mL	milliliters	0.034	fluid ounces	fl oz
L	liters	0.264	gallons	gal
m^3	cubic meters	35.314	cubic feet	ft^3
m^3	cubic meters	1.307	cubic yards	yd^3
		MASS		
g	grams	0.035	ounces	oz
kg	kilograms	2.202	pounds	lb
Mg (or "t")	megagrams (or "metric ton")	1.103	short tons (2000 lb)	T
		TEMPERATURE (exact degrees)		
°C	Celsius	1.8C+32	Fahrenheit	°F
		ILLUMINATION		
lx	lux	0.0929	foot-candles	fc
cd/m^2	candela/m^2	0.2919	foot-Lamberts	fl
		FORCE and PRESSURE or STRESS		
N	newtons	0.225	poundforce	lbf
kPa	kilopascals	0.145	poundforce per square inch	lbf/in^2

*SI is the symbol for the International System of Units. (Adapted from FHWA report template, Revised March 2003)

EXECUTIVE SUMMARY

A full-scale mechanically stabilized earth (MSE) wall was constructed and 24 lateral load tests were conducted on piles spaced at distances of about 2 to 5 pile diameters (D) from the back face of the wall to the center of the pile. Sixteen of the tests involved pipe piles, four tests involved H-piles, and four tests involved square piles. Lateral resistance decreased significantly when piles were located closer than about 4D from the wall. P-multipliers were developed to account for the reduction in lateral soil resistance as a function of normalized distance behind the wall using results from these tests and eight previous full-scale tests. P-multipliers were approximately one for piles located further than about 4D from the wall, while p-multipliers decreased linearly as distance to the wall decreased. P-multipliers were not significantly affected by differences in reinforcement length to height (L/H) ratio, reinforcing type, or pile shape, but provided a reasonable means for estimating reduced lateral pile resistance. Measured tensile force in the reinforcements tended to reach a peak near the location of the pile rather than at the wall face, indicating that the reinforcements were anchoring the wall against lateral movement. Multi-linear regression equations were developed to predict maximum tensile force in the reinforcements. Tensile force tended to increase as pile head load and vertical stress increased, and it tended to decrease with increases in transverse distance from the load point and normalized distance from the wall.

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1.0 INTRODUCTION

Increasing right-of-way constraints have led to the increased use of Mechanically Stabilized Earth (MSE) walls near bridge abutments rather than slopes. Piles located within the reinforced zone of MSE walls that are used to support bridge abutments must resist both vertical loads from the bridge superstructure as well as lateral loads produced by earthquakes and thermal expansion and contraction for integral abutments. Currently, there is little guidance for determining the lateral resistance of piles behind MSE walls. Common methods employed at this time are to place the piles far enough behind the wall (perhaps four to eight pile diameters) to negate the wall's influence; to assume that there is no lateral resistance from the wall; or to place the pile close to the wall and assume a lateral resistance reduction factor based on engineering judgment. These methods are inefficient for the following reasons: (1) increasing the distance between the wall and the pile increases the bridge cost by increasing the bridge span; (2) assuming no lateral resistance increases foundation costs because the pile size and/or the number of piles required will increase; and (3) using engineering judgment could yield widely varying results in the absence of full-scale field test data.

Lateral load tests performed by Pierson et al. (2009) on 0.9-m (3-ft) diameter drilled shafts behind a 6.1-m (20-ft) high masonry block wall reinforced with extensible geosynthetic geogrids indicate that lateral resistance decreases as piles are placed closer to the wall. In addition, significant wall distortion occurred in the masonry block wall face during loading. Additional lateral load tests conducted by Rollins et al. (2013) on 324 to 406 mm (12.75 to 16-inch) diameter steel pipe piles with inextensible metallic reinforcement also found a decrease in lateral resistance for piles located closer than about four pile diameters behind the wall. However, wall panel movement was generally quite low relative to that observed with the geogrid reinforcements.

Furthermore, Rollins et al. (2013) concluded that p-multipliers based on pile spacing behind the wall could be used to account for the decreased lateral soil resistance. Nevertheless, the p-multipliers developed by Rollins et al. (2013) were significantly higher than back-calculated for the geogrid reinforcement (Pierson et al. 2009). Rollins et al. (2013) concluded that tensile forces induced in the reinforcements by lateral pile loading could be estimated using variables such as pile load, pile spacing behind the wall, and transverse distance between the loaded pile and the reinforcing element.

Although the previous research is clearly valuable in developing a framework of understanding, results are limited to a handful of tests having a significant number of variations with respect to soil density, reinforcement type, and reinforcement length to height ratios. Clearly additional test results and analysis are necessary to develop design recommendations.

This report describes the results of additional full-scale testing where the objectives are: (1) to measure lateral pile resistance vs. displacement curves for piles placed at different distances behind an MSE wall, (2) to measure the distribution of tensile force in the reinforcement induced by lateral pile loading, (3) to develop design rules (e.g. p-multipliers) to account for reduced pile resistance as a function of pile position behind the MSE wall, and (4) to develop equations to predict maximum reinforcement force induced by lateral pile loading.

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2.0 TEST LAYOUT AND TESTING APPROACH

To accomplish the objectives of the study, a full-scale MSE abutment wall was constructed to conduct research on laterally loaded piles. The wall was constructed in two phases using welded-wire grid on one half of the wall and steel strip reinforcements on the other, as shown in Fig. 1, so that the performance of the two reinforcement systems could be evaluated separately but with comparable backfill conditions. Because each reinforcement system develops resistance in different ways, this allows separate design approaches to be developed, if necessary. The MSE wall consisted of nominally 1.5-m by 3-m (5-ft by 10-ft) reinforced concrete facing panels with 5.49 m (18-ft) long reinforcements.

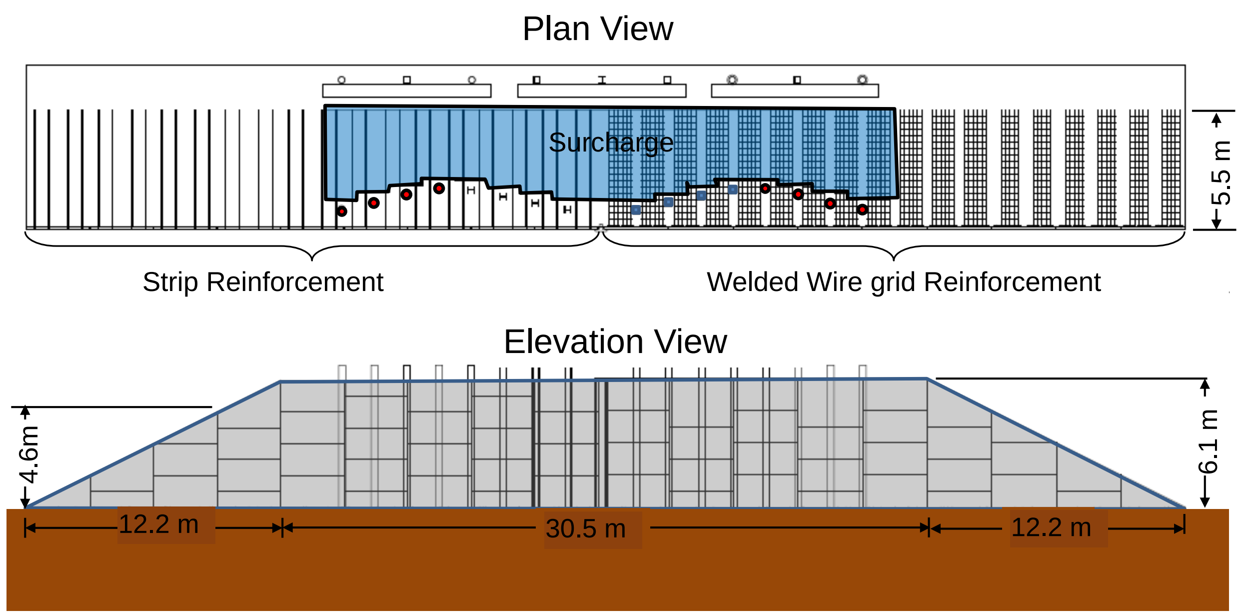


Fig. 1. Plan and profile views of mechanically stabilized earth (MSE) wall, test piles, and reinforcement for Phase 1 (4.6-m or 15-ft) and Phase 2 (6.1-m or 20-ft) high wall.

During Phase 1, lateral pile load tests were performed at a wall height of 4.6 m (15 feet) with a 28.7 kPa (600 psf) surcharge to simulate the weight of the soil behind the abutment above the reinforced zone as illustrated in Fig. 2. Including the equivalent height of the surcharge [about 1.5 m (5 ft)] produced a reinforcement length to height (L/H) ratio of about 0.9, which might be common for seismic design. Lateral load tests on pipe piles within welded wire and ribbed strip reinforcement and H-piles within ribbed strip reinforcement were performed during Phase 1. During Phase 2, tests were conducted at a wall height of 6.1 m (20 feet) along with the surcharge to give an L/H ratio of about 0.7, which is more typical for static loading. After fill placement, lateral load tests were again performed on the same pipe piles, along with square piles within welded wire reinforcement for Phase 2. The difference in reinforcement ratios (L/H) makes it possible to determine if reinforcement length has any significant effect on lateral pile resistance or induced force in the reinforcements.

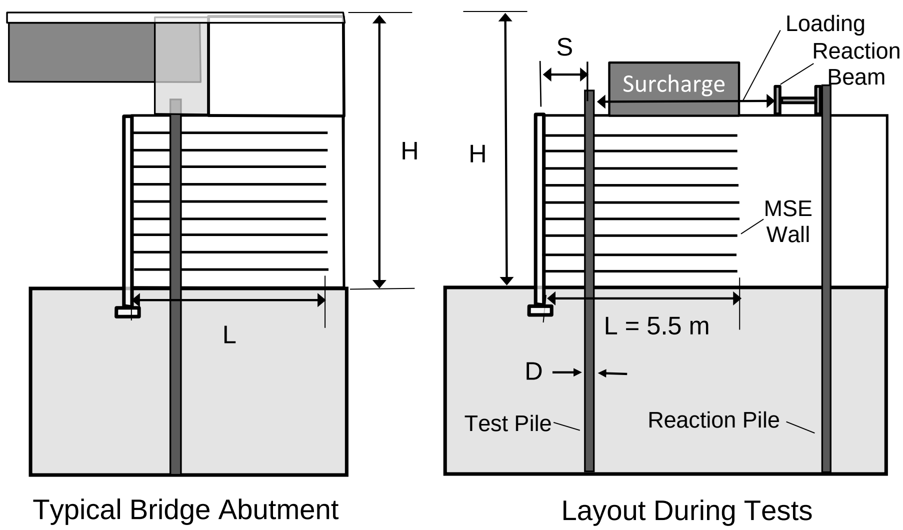


Fig. 2. Typical layout of bridge abutment with MSE wall relative to test layout.

The reinforcements were designed according to AASHTO 2012 LRFD standards. The galvanized ribbed strip reinforcements provided by the Reinforced Earth Co. were 4 mm thick and 50 mm wide (0.16 x 1.97 inch). Galvanized ribbed strip reinforcements were spaced 0.76 m (30 inches) vertically and horizontally. Each galvanized welded wire grid provided by SSL, LLC was typically composed of 5 or 6 W11 longitudinal bars spaced at 20.3 cm (8 in.) with W11 transverse bars spaced at 30 cm (12 in.) (15 cm or 6 in. at the top layer). Each grid was spaced 0.76 m (30 in.) vertically and 1.52 m (60 in.) horizontally, center-to-center. Yield strength for both reinforcements was 448 MPa (65 ksi).

The backfill soil consisted of gravelly sand with 10 to 14% fines and about 20% gravel. The backfill classified as A-1-a material according to the AASHTO system and SM to SP-SM according to the USCS system. The modified Proctor maximum density was 20.9 kN/m^3 (133 lbs/ft^3) with an optimum moisture content of 7%. After the initial row of wall panels were leveled and installed, backfill was placed in 30-cm (12-in.) lifts behind the test piles and in 15-cm (6-in.) lifts between the test piles and the back of the wall. Soil behind the test piles was densified using a vibratory roller compactor, while soil between the wall and the tests piles was compacted with a plate compactor. Behind the test piles, relative compaction was uniformly greater than 95% with an average of 97%; however, between the piles and the wall, the relative compaction varied from 88% to 94% with an average of 92%. Although this lower and variable density state near the wall face is not particularly desirable it is considered representative of typical construction practice.

Pile types consisting of pipe, square and H-piles were located behind the wall in the reinforced zone at nominal distances of approximately 2, 3, 4 and 5 pile diameters measured from the back face of the wall to the center of the pile. To facilitate comparisons, all piles were made of steel with a yield strength of 393 MPa (57 ksi) and had a width/diameter of about 30 cm. The pipe piles were 32.4 cm (12.75 in.) in diameter with a nominal wall thickness of 9.5 mm (0.375 in.). The moment of inertia of each pipe pile was 11,613 cm^4 (279 in^4); however, angle sections were welded to the sides to protect strain gauges which increased the moment of inertia to 13,070 cm^4 (314 in^4). The square piles were 30 cm (12 in.) square with a wall thickness of 7.4 mm (0.29) inch. The moment of inertia of each square pile was 12,653 cm^4 (304 in^4); however, with the angle sections added, the moment of inertia increased to 13,944 cm^4 (335 in^4). The H-piles were

HP12x74 sections loaded about the weak axis with a width of 31 cm (12.2 inch) in the direction of loading. Measured flange and web thicknesses were (0.61 in.) and (0.605 in.). The moment of inertia of each H-pile was 7,742 cm^4 (186 in^4); however, with the angle sections added, the moment of inertia increased to 7,784 cm^4 (187 in^4). Prior to fill placement, each 12.2-m (40-ft) test pile was driven 5.5 m (18 ft.) into the native silty sand below the base of the MSE wall and extended 6.7 m (22 ft.) above the base.

3.0 TESTING PROCEDURE AND INSTRUMENTATION

Initially, reaction piles behind the reinforced zone (See Fig. 1 and 2) were loaded laterally parallel to the MSE wall face. Subsequently, test piles near the wall face were loaded laterally perpendicular to the MSE wall face with a hydraulic jack. Steel struts were used to transfer the load to a reaction beam and reaction piles located beyond the reinforced soil zone. This was done to avoid affecting the load distribution in the reinforcements. Load was applied at a height of 30 cm (12 in.) above the ground surface with a pinned-head connection so that the boundary condition would be well-defined. Load was applied to produce displacement increments of approximately 6.4 mm (0.25 in.) up to a maximum deflection of about 76 mm (3.0 in.). At each 6.4-mm (0.25-in.) deflection increment, the hydraulic fluid was locked off and the load was allowed to equilibrate with the applied deflection for a period of 5 minutes. After about one minute, the load typically decreased by about 6-10% from the peak and remained relatively stable. Therefore, load-defection plots are based on the load and deflection at the one-minute hold.

In addition to pile head load and deflection, the movement of the ground and wall face directly in front of each test pile was measured using string potentiometers. Furthermore, a digital image correlation (DIC) camera system was used to measure the movement of thousands of points across each wall panel as load was applied to the pile head. Lastly, tensile force was computed using strain gauge measurements in reinforcing elements at two to four levels below the compacted fill surface and at two distances transverse to the loading. Strain gauge pairs were located at distances of 0.15, 0.61, 0.91, 1.52, 2.44, 3.35, and 4.27 m (0.5, 2, 3, 5, 8, 11, and 14 feet) from the back of the wall.

4.0 LATERAL LOAD VS. DEFLECTION CURVES

Plots of pile head lateral load vs. deflection for four test piles loaded at various distances from the MSE wall face are shown in Fig. 3 and Fig. 4. Fig. 3 involves tests with a wall height of 6.1-m (20-ft.) with the welded wire grid reinforcement, while Fig. 4 shows tests with a wall height of 4.57-m (15-ft.) with ribbed strip reinforcements. Results for companion reaction piles loaded parallel to the wall are shown in each figure for comparison. Fig. 5 and Fig. 6 show similar comparisons, but for the H-piles and square piles, respectively. As noted previously, the H-piles were tested at the 4.57-m (15-ft.) wall height with ribbed strip reinforcement, while the square piles were tested at the 6.1 m (20-ft.) wall height with welded wire reinforcement.

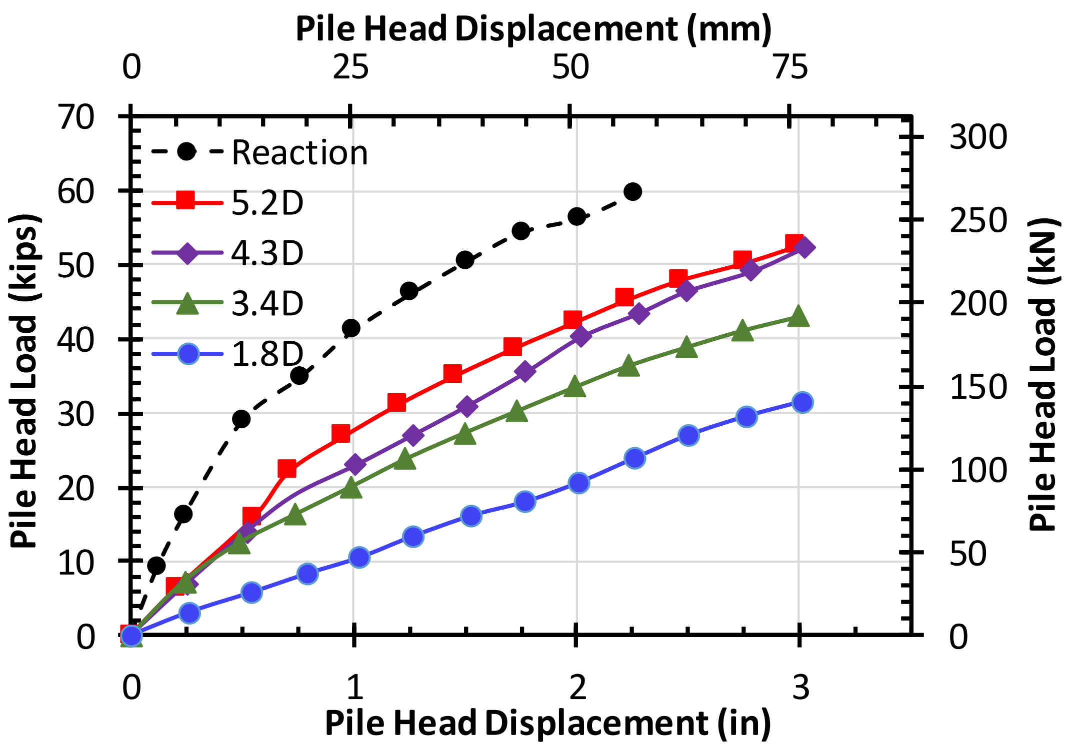


Fig. 3. Pile head load vs. deflection curves for test and reaction piles consisting of pipe piles at 6.1-m (20-ft) wall height with welded-wire reinforcement.

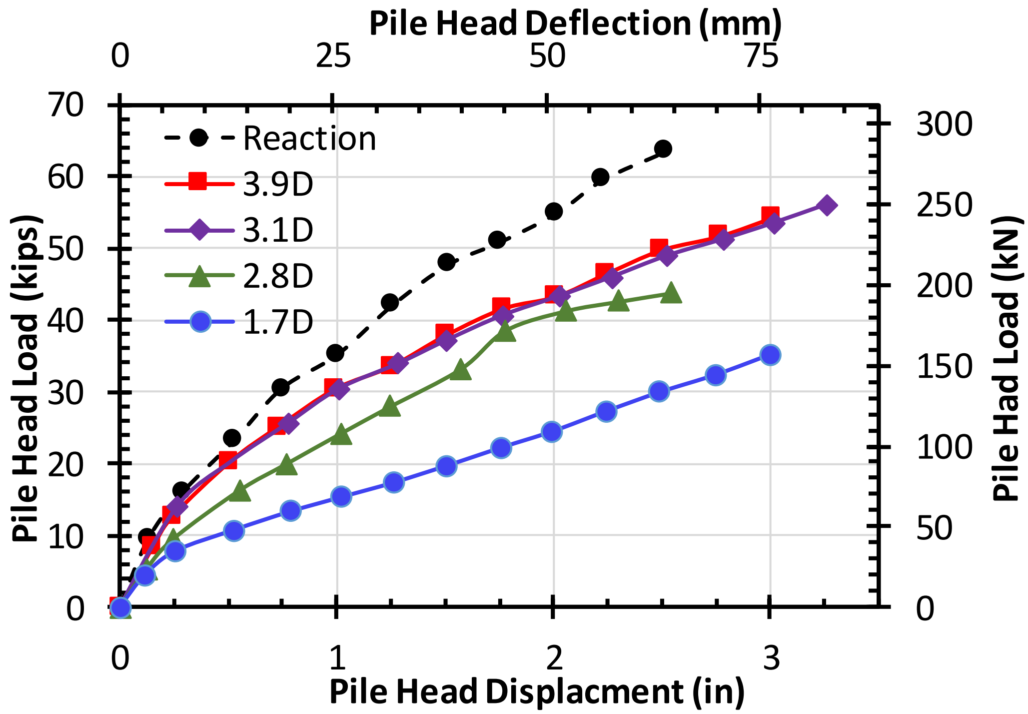


Fig. 4. Pile head load vs. deflection curves for test and reaction piles consisting of pipe piles at 4.57-m (15-ft) wall height with ribbed strip reinforcement.

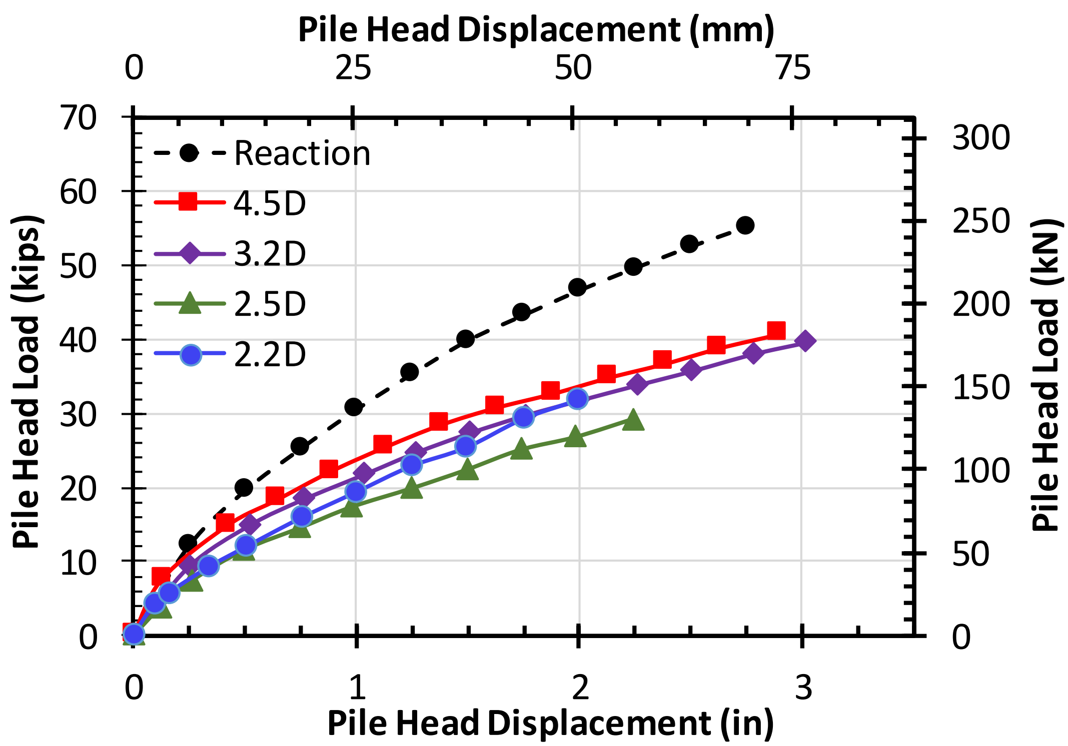


Fig. 5. Pile head load vs. deflection curves for test and reaction piles consisting of H-piles at 4.57-m (15-ft) wall height with ribbed strip reinforcement.

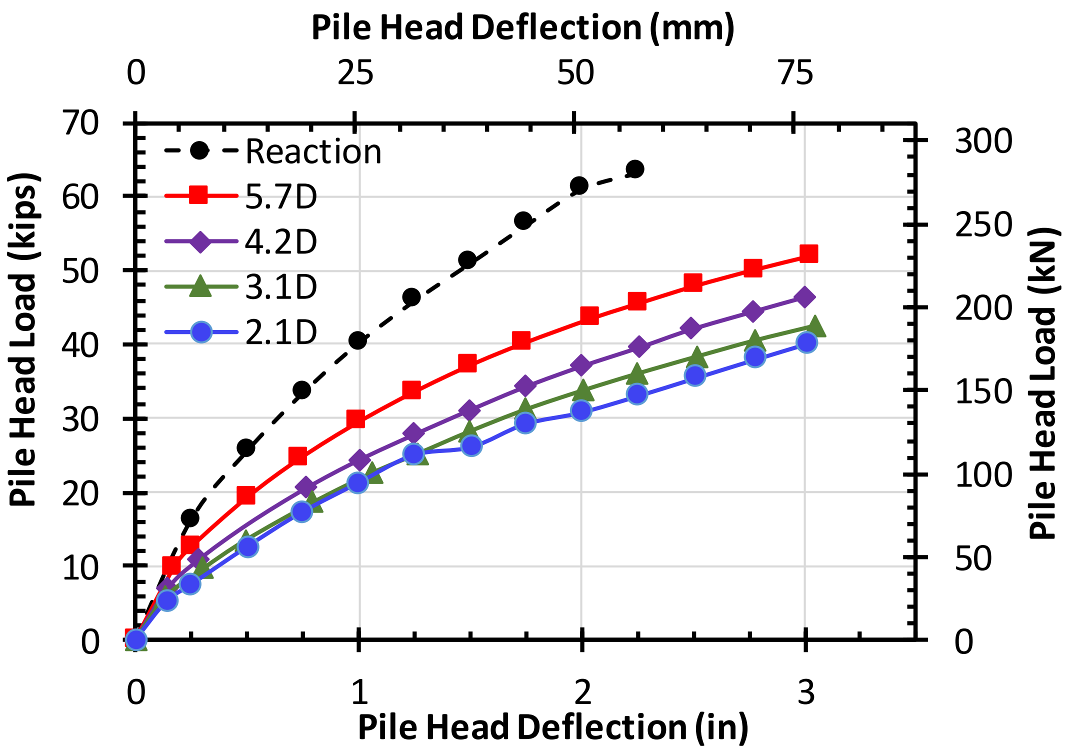


Fig. 6. Pile head load vs. deflection curves for test and reaction piles for square piles at 6.1-m (20-ft) wall height with welded-wire reinforcement.

In all four figures, the load-deflection curve for the reaction pile is stiffer than all those for the piles near the MSE wall. The higher resistance is likely a result of the higher relative compaction for the soil near the reaction pile (97%) than for the piles near the wall (92%). For the piles near the wall there is a general trend for the lateral pile resistance to decrease as the pile is placed closer to the wall as noted by Pierson et al (2009). Similar results were also reported by Rollins et al. (2013) based on lateral load tests at bridge sites in Utah and the reduction in resistance became more pronounced for pile closer than about four pile diameters behind the wall. However, in this study the decrease was not always uniform and in the case of the H piles, resistance was higher for one pile located closer to the wall. The variations in the decrease are likely attributable to variations in the relative compaction in front of the piles noted previously.

5.0 HORIZONTAL GROUND DISPLACEMENT AND WALL DISPLACEMENT

Fig. 7 shows the ground surface displacement in front of the pile normalized by pile deflection at the ground surface plotted versus the distance from the pile face divided by the pile diameter for the pipe piles within the welded wire reinforcement for the 6.1 m (20 ft) wall height. Data points in this figure are for the 76 mm (3 in.) pile deflection loading. The results indicate that the ground displacement decreases very rapidly with distance from the pile face, likely due to the resistance provided by the reinforcements. Fig. 8 presents a similar plot but with 366 data points collected for all of the piles tested along with a best fit curve given by the equation:

$$\frac{\delta}{\delta_P} = 1 - 0.92\tanh\left(\frac{0.8L}{D}\right) \tag{1}$$

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, and $D =$ pile diameter. The data points were obtained from the 6.35, 12.7, 25.4, 50.8, and 74 mm (0.25, 0.5, 1, 2, and 3in.) pile deflections. The coefficient of determination for the best fit curve is 0.97. Once again, the ground displacements decrease rapidly within 2D from the pile face such that horizontal ground displacement is only about 10-20% of the pile displacement beyond this boundary.

Fig. 9 shows a DIC image of wall panel displacement for the square pile located 3.1D behind the wall after a pile loading of 191 kN (43 kips) corresponding to a pile deflection of 76.2 mm (3 in.). This image provides contours of wall displacements over an area of about 3 m x 3.5 m. The maximum displacement [12.5 mm (0.5 in.)] occurs within a rectangular zone centered on the pile and extending to about 1.5 m (5 ft) below the top of the wall. Wall displacements decrease at greater depths and with transverse distance from the load point.

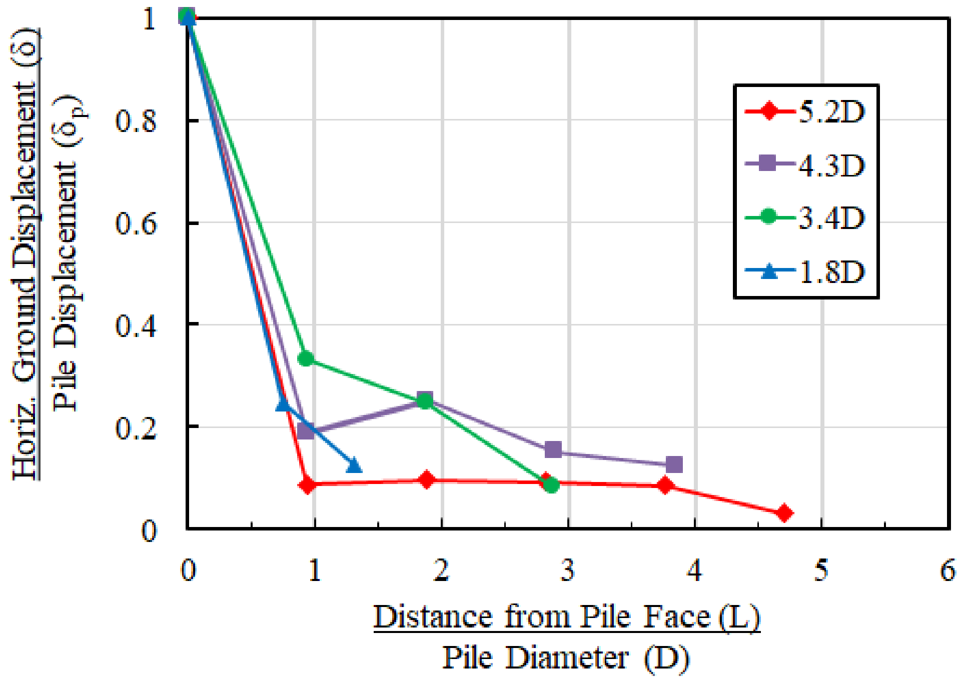


Fig. 7. Normalized ground displacement vs. normalized distance from the pile face for the pipe piles within the welded wire reinforcement for the 6.1-m (20-ft) wall height at the 76.2 mm (3 in.) pile head deflection increment.

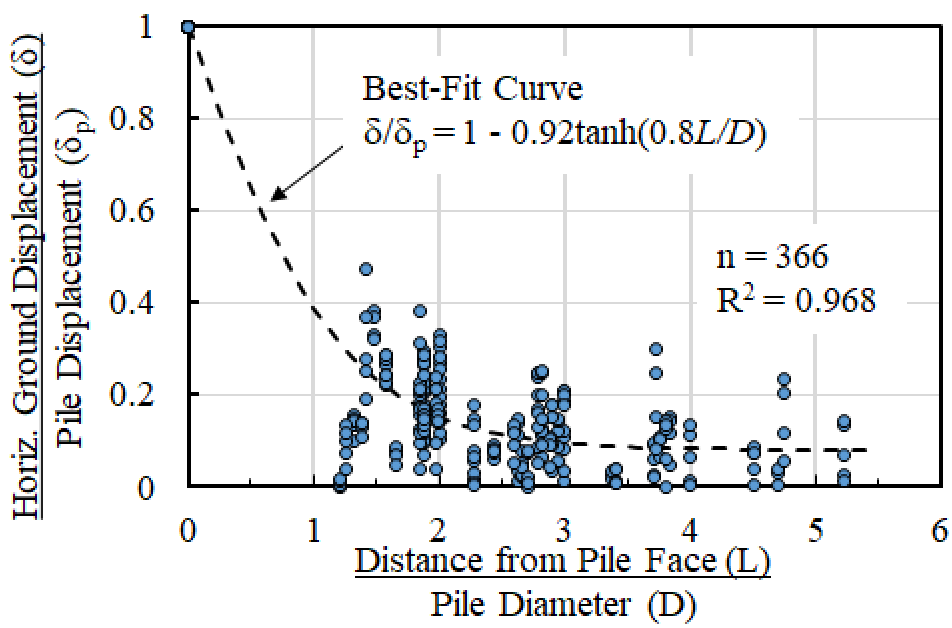


Fig. 8. Horizontal ground displacement divided by displacement of pile at ground level vs. distance from the pile face divided by the pile diameter for all piles with the best fit line.

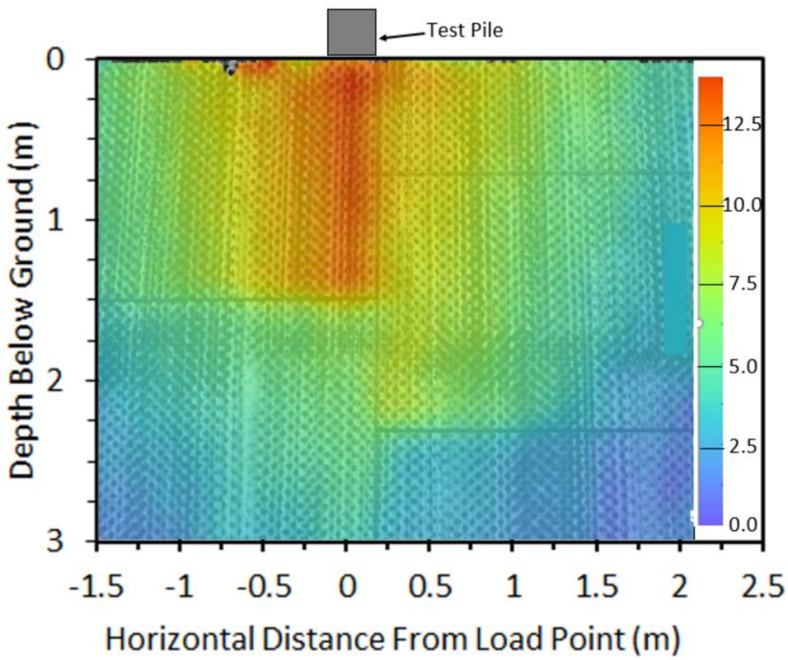
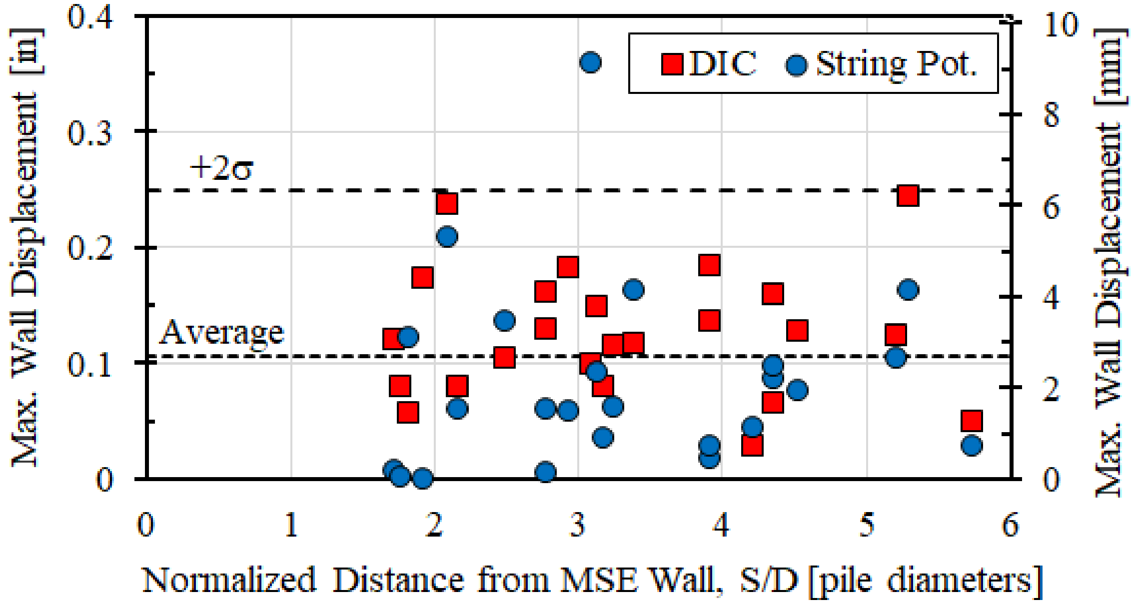


Fig. 9. DIC image showing contours of wall panel displacement (mm) for the square pile at 3.1D from the wall for the 76.2 mm (3 in.) pile head deflection.

Fig. 10 shows plots of the maximum wall displacement versus the normalized distance (S/D) for the pile from the MSE wall for pile head deflections of 25.4 mm and 76.2 mm (1 and 3 inch). Maximum wall displacement was obtained anywhere on the wall from the DIC and at the top of the MSE wall behind the pile for string potentiometers for all of the piles tests. For the 25.4 mm (1 in.) pile head deflection, the average of the maximum wall displacements was about 2.5 and 0.76 mm (0.1 and 0.3 in.) for the 25.4 and 76.2 mm (1 and 3 in.) pile deflections, respectively. The mean plus two standard deviation maximum displacements were 6.35 mm (0.25 in.) for the 25.4 mm (1 in.) pile head deflection and just over 16 mm (0.63 in.) for the 76.2 mm (3 in) pile head deflection. Thus, most of the wall deflections were no greater than 16 mm (0.63 in.), and were typically less than 12.7 mm (0.5 in.). There was no significant difference in maximum wall displacements when piles were located either behind the center of a wall panel or behind a wall panel joint. In addition, there was very little effect of S/D on wall deflection.

(a)



(b)

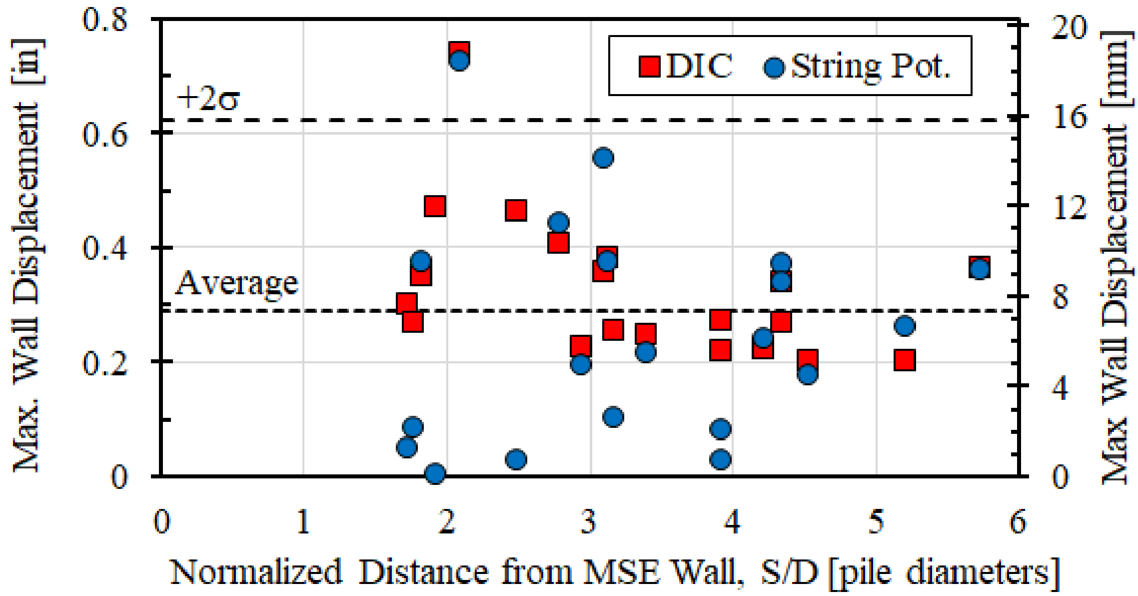


Fig. 10. Maximum wall displacement from DIC or string potentiometer versus normalized distance from the MSE wall for pile head deflection of (a) 25.4 mm (1 in.) and (b) 76.2 mm (3 in.) for all pile tests.

6.0 P-MULTIPLIERS TO ACCOUNT FOR MSE WALL

The computer program LPILE was used to back-calculate appropriate p-multipliers for each group of pile load tests. P-multipliers are factors that are multiplied by the normal lateral soil resistance to account for reduced lateral soil resistance. 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 (Brandenberg et al. 2007). In this case, the p-multipliers (P_{MSE}) account for reduced lateral resistance for a pile near an MSE wall relative to a pile far enough away to be unaffected. Based on previous testing (Rollins et al. 2013), the pile farthest from the MSE wall (typically about 5D) 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. Initially, the pile farthest from the wall was analyzed and the soil properties necessary to produce agreement with the measured load-deflection curve were determined. In calibrating the soil model, both ϕ and k affect the computed load-deflection curve; however, k has more effect on the curve at small deflection levels while ϕ has a greater effect at larger deflections as the soil layers begin to reach failure. Generally, the k value was selected based on the correlation with friction angle for soil above the water table as specified by API (1982). However, some adjustment was allowed to improve agreement with the measured curve. Typically, the back-calculated friction angle was between 38º and 39º.

For piles located closer to the wall, these back-calculated soil parameters were then held constant for each pile type, and a single p-multiplier was back-calculated to produce agreement with the measured load-deflection curve for that pile. Fig. 11 shows the computed pile head load vs. displacement curves relative to their measured data for tests on the square piles after applying the back-calculated p-multipliers. Generally, agreement between measured and computed response was quite good despite the simplicity of the approach. Table 1 provides a summary of the back-calculated p-multipliers for each of the tests.

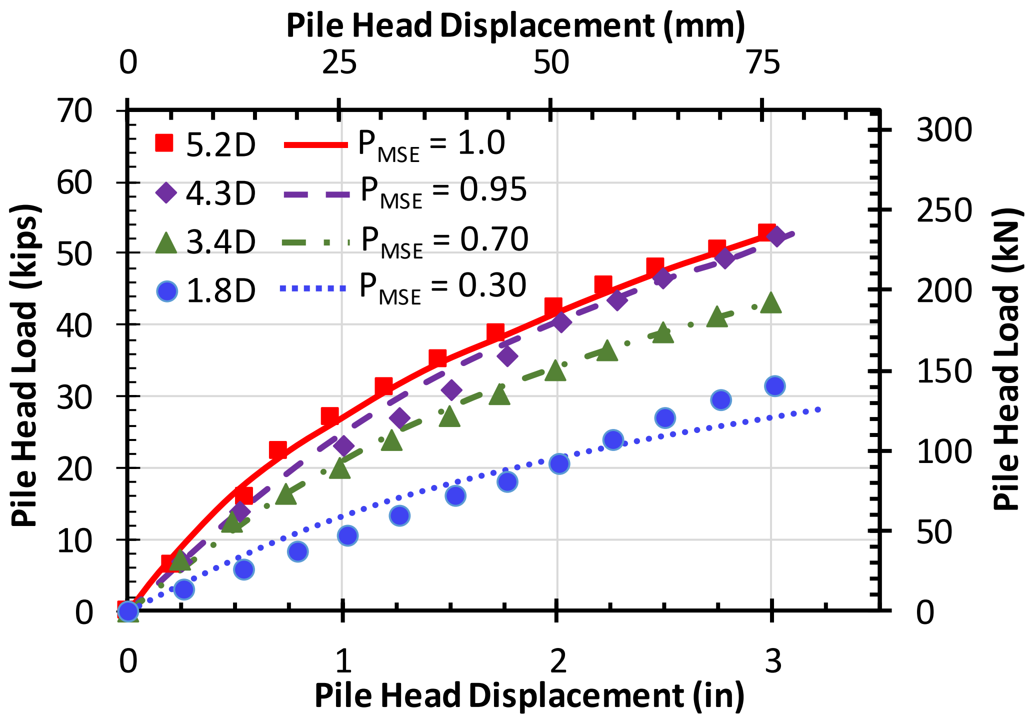


Fig. 11. Comparison of measured load-displacement curves for pipe piles near 6.1 m (20 ft) wall with welded-wire reinforcement computed curve using LPILE using a p-multiplier (P_{MSE}) to account for the presence of the wall.

Back-calculated p-multipliers are plotted versus normalized distance (S/D) from the wall [distance behind back of the wall to the center of the pile (S) divided by pile diameter(D)] in Fig. 12. This figure includes a total of 30 data points including tests conducted in this study along with tests performed previously by Rollins et al. (2013) on piles at bridge sites under construction in Utah. These piles had diameters ranging from 32.4 to 40.6 cm (12.75 to 16 in.).

Table 1 Summary of Lateral Pile Load Test Characteristics and P-multipliers

Pile Shape	Normalized Distance from wall	P-multiplier (P_{MSE})	L/H Ratio	Reinforcement Type	Reference
Pipe (0.324m) Pipe (0.406m) Pipe (0.324m) Pipe (0.406m) Pipe (0.406m)	7.2 5.2 3.8 2.9 1.6	1.0 1.0 1.0 0.8 0.25	1.11 0.97 1.25 1.21 0.96	Welded Wire	Rollins et al (2013)
Pipe (0.324m)	6.3 2.7 1.3	1.0 0.51 0.16	1.03 1.20 1.03	Ribbed Strip	Rollins et al (2013)
Pipe (0.324m)	5.3 4.3 3.2 1.9	1.0 0.7 0.7 0.25	0.90 0.90 0.90 0.90	Welded Wire	Hatch (2014)
Pipe (0.324m)	3.9 3.1 2.8 1.8	1.0 0.95 0.70 0.33	0.90 0.90 0.90 0.90	Ribbed Strip	Han (2014)
Pipe (0.324m)	3.9 2.9 1.7	1.0 1.0 0.45	0.72 0.72 0.72	Ribbed Strip	Besendorfer (2015)
Pipe (0.324m)	5.2 4.3 3.4 1.8	1.0 0.95 0.68 0.30	0.72 0.72 0.72 0.72	Welded Wire	Budd (2016)
H-Pile (HP12x74)	4.5 3.2 2.5 2.2	1.0 0.85 0.6 0.73	0.90 0.90 0.90 0.90	Ribbed Strip	Luna (2016)
Square (12.2x12.2)	5.7 4.2 3.1 2.1	1.0 0.77 0.63 0.57	0.72 0.72 0.72 0.72	Welded Wire	Luna (2016)

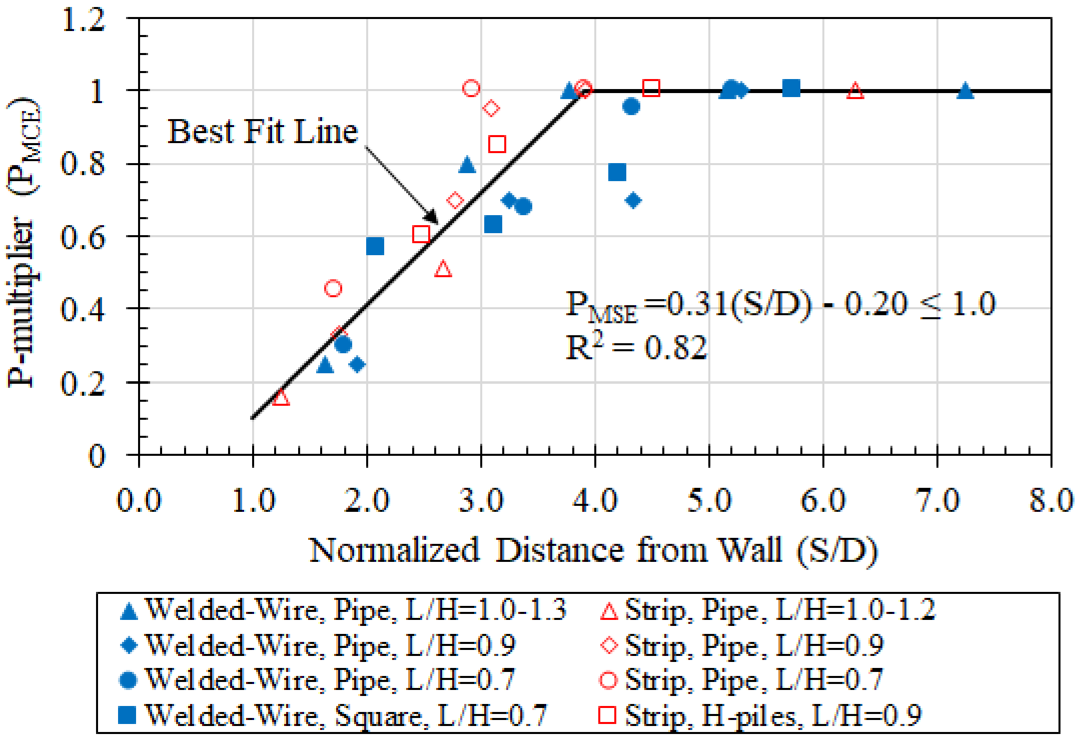


Fig. 12. P-multiplier (P_{MSE}) versus normalized pile spacing (S/D) using pipe, square, and H-pile data from this study and previous study by Rollins et al. (2013).

The hollow red data points are test results using galvanized ribbed strip reinforcement while the solid blue data points are test results using the galvanized welded wire grid reinforcement. All tests have an equivalent L/H ratio ranging between 0.72 and 1.25. In general, neither the L/H ratio, the pile shape, nor the type of steel reinforcement produced any significant or consistent effect on the p-multipliers for steel piles behind the MSE walls for the pile diameters (32.4 to 40.6 cm) involved.

A review of the data in Fig. 12 indicates that nearly all of the back-calculated data points with S/D greater than four have a p-multiplier near 1.0 with the exception of two tests. A linear regression analysis was performed to evaluate the p-multiplier as a function of normalized pile distance using data that was within a distance of 4D from the wall. The best fit relationship for p-multiplier, P_{MSE}, is given by the equation:

$$\text{If } \frac{S}{D} \leq 3.9\text{, then } P_{MSE} = 0.31\frac{S}{D} - 0.20 \leq 1.0 \tag{2a}$$

$$\text{If } \frac{S}{D} > 3.9\text{, then } P_{MSE} = 1.0 \tag{2b}$$

In this regression analysis, the R^2 value is 0.82 for S/D less than 4.0. This regression excludes the data point at 2.2D for the H-pile which has a higher lateral resistance than the 2.5D H-pile (see Fig. 5), in contrast to all other tests, and skews the best-fit line upwards in an unconservative direction. The scatter in the data points is considered to result from the variability associated with lower compactive effort that is typically associated with soil close to the wall face as described previously.

The linear regression analysis of Equation (2a) indicates that a p-multiplier of 1.0 will result from a normalized distance greater than 3.9. For normalized distances less than 3.9, 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. A normalized distance of 3.9 is also generally consistent with the measured ground displacements (see Fig. 8) that are relatively minor beyond this distance. With the aid of Equation 2, a design engineer should be able to determine if it would be more economical to locate piles closer to the wall and allow for reduced pile resistance or place the piles farther back and increase the bridge span.

7.0 FORCE INDUCED IN REINFORCEMENTS FROM LATERAL PILE LOADING

Tensile force induced in the reinforcements owing to lateral pile loading was measured using strain gauges along the length of the reinforcements. Generally, strain was measured at one to four reinforcement levels below the ground surface and in reinforcements at different distances transverse to the loaded pile. The goal of the reinforcement force measurements was to determine where the peak force would develop and how the peak force would be related to a number of variables influencing the induced force.

The influence of various factors affecting reinforcement force is illustrated by plots of induced reinforcement force vs. distance behind the wall in Fig. 13. This figure shows results for the pipe piles within welded wire reinforcement at the 6.1-m (20-ft) wall height, but results are similar for other pile types and reinforcements. As expected, these plots clearly show that the peak force induced in the reinforcement increases as the pile head load increases. In addition, the peak induced force does not occur at the wall face, but rather near the location of the pile because the pile is pushing a wedge of soil towards the wall, while the reinforcements behind the wedge are serving as "anchors". The peak induced force decreases rapidly with transverse distance from the loaded pile. For example, in Fig. 13(b) the induced force in the reinforcements located 1.17 m (46 in.) transverse to the loaded pile is significantly lower than the induced force in the reinforcement located 0.55 m (21.5 in.) transverse to the pile in Fig. 13(a).

These figures also show that peak loads occur closer to the pile face as transverse distance to the pile decreases, and farther in front of the pile face as the transverse distance increases. This likely occurs because the soil failure wedge in front of the pile fans out during loading so that that peak loads occur farther in front of the pile with increased transverse spacing of the reinforcement. Lastly, the induced reinforcement forces are much less than the nominal pullout resistance of the reinforcement specified by AASHTO (2012) procedures as shown in the figure.

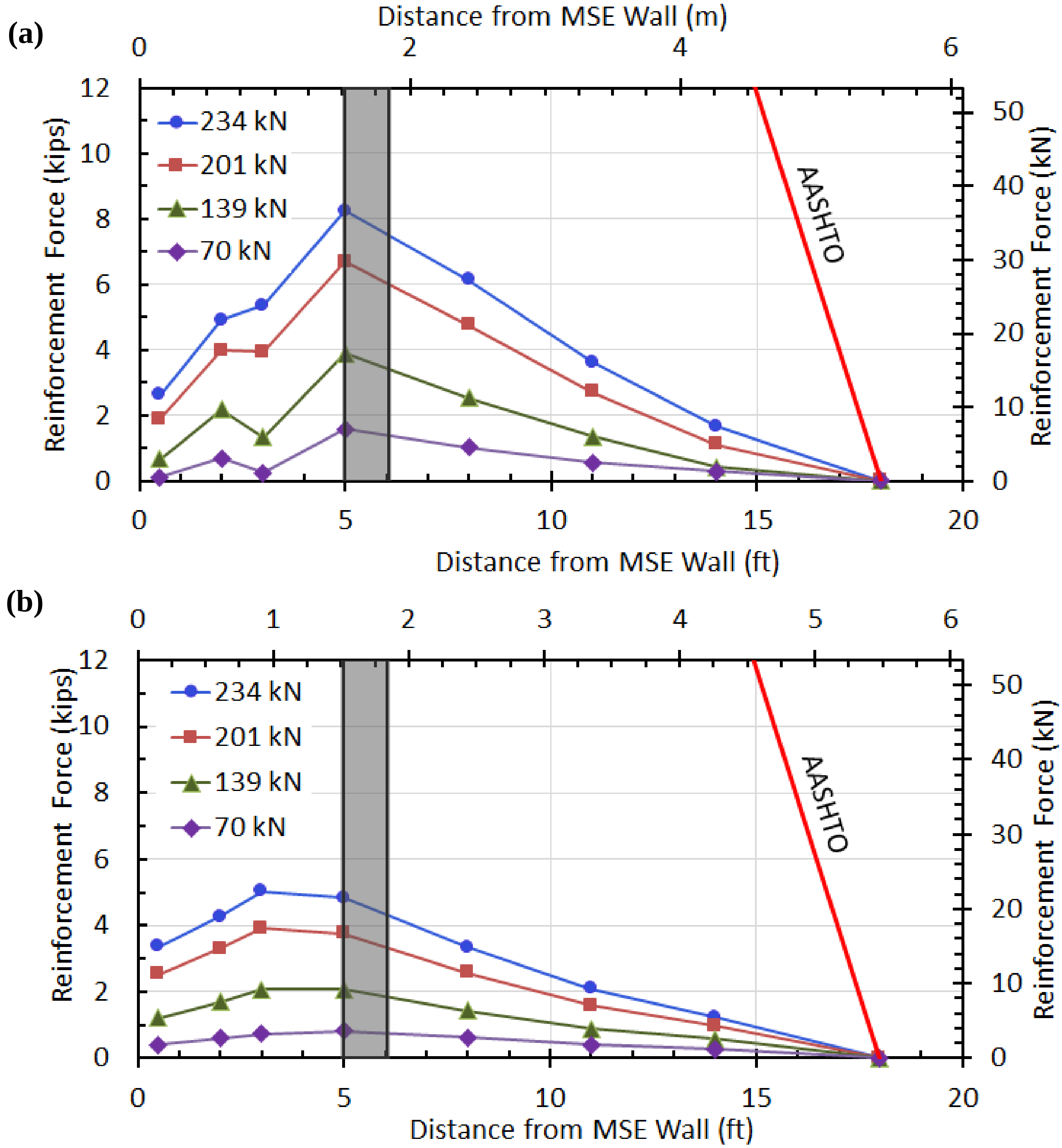


Fig. 13. Induced reinforcement force vs. distance behind the wall for reinforcements at four pile load levels for reinforcements located at (a) 0.55 m (21.5 in) and (b) 1.17 m (46 in.) transverse to the loaded pile. Ultimate pullout resistance from AASHTO (2012) is also shown for comparison.

A schematic drawing illustrating the likely behavior of the pile-soil-reinforcement interaction is presented in Fig. 14. The force distribution in the reinforcement suggests that soil in front of the pile is being pushed forward as the pile is loaded while soil behind the pile is resisting movement of the reinforcement. In front of the pile, the soil is moving toward the wall relative to the reinforcement. This leads to an increase in tension in the reinforcement as load is transferred from the soil to the reinforcement by skin friction. Any positive tensile force in the reinforcement at the wall face is likely a result of the increased earth pressure on the wall. Behind the pile, the reinforcement is moving towards the wall relative to the soil. This leads to a decrease in tension in the reinforcement behind the pile as load is transferred to the surrounding soil by skin friction.

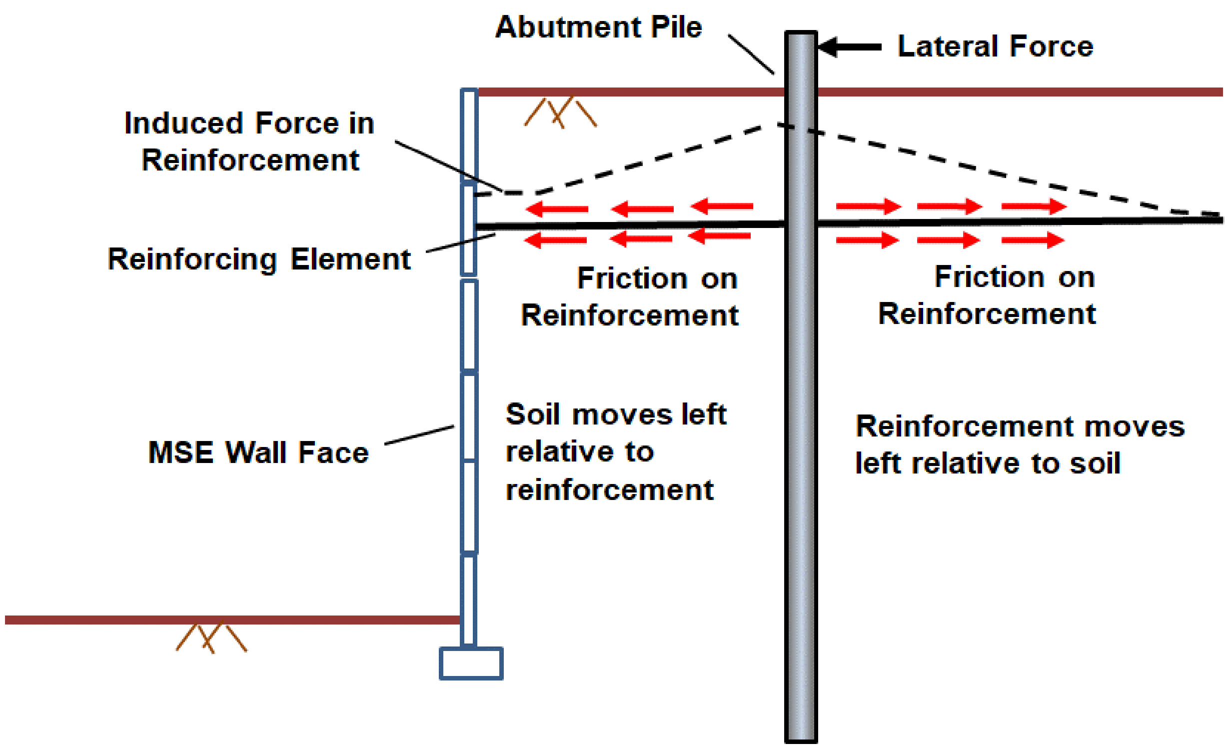


Fig. 14. Conceptual illustration of the mechanisms inducing tensile force in the reinforcements from lateral pile loading behind the wall.

8.0 REGRESSION EQUATIONS FOR PREDICTING INDUCED FORCE FROM LATERAL PILE LOADING

The development of tensile force owing to lateral pile loading adjacent to an MSE wall is a relatively complicated soil structure interaction problem. The pile is interacting with the soil and soil is interacting with the reinforcements. As a result, it was not possible to develop any meaningful simple correlations to describe the observed behavior. The Statistical Analysis System (SAS) software program was used to perform regression analyses using the General Linear Model (GLM) procedure. SAS was used to determine the statistically significant parameters in the model, after which the Data Analysis pack for Microsoft® Excel was used to fine tune the model by eliminating parameters and thereby simplifying the model without decreasing the R^2 value significantly.

Statistical analysis of the data indicates that a number of factors influence the induced peak reinforcement force because of the complex interaction between loaded pile-soil-reinforcement and wall panels. These factors include: the applied pile head load, the transverse distance from the loaded pile normalized by the pile diameter, the vertical stress on the reinforcement, and the distance of the pile behind the wall normalized by the pile diameter. In computing the vertical stress, the weight of the surcharge was considered and the surcharge weight was also considered to increase the effective wall height in accordance with AASHTO code requirements. The L/H ratio was also found to be statistically significant, but this factor was eliminated to simplify the resulting design equation with a small decrease in R^2. Data for the measured maximum tensile force were not normally distributed but were log normally distributed. Therefore, a base 10 log transformation was applied to the tensile force before performing the analysis to account better for scatter in the data.

Separate regression analyses were required for the ribbed strip and welded wire reinforcements. A total of 1,058 data observations were used in the regression analysis of the welded-wire grid reinforcements resulting in an R^2 value of 0.72. An R^2 of 0.72 indicates that the equation accounts for approximately 72 percent of the variability in the observed tensile force. The maximum induced force, *ΔF(kips)* (>0), in the welded-wire reinforcement produced by the lateral pile load is given by:

$$\Delta F(kips) = 10\wedge\left(-0.04 + 0.027P - 2.7x10^{-4}P^2 + 5.7x10^{-4}\sigma_V - 2.6x10^{-7}\sigma_V^2 - 0.08\frac{T}{D}\right) - 1 \qquad (3)$$

where $P =$ the pile head load (kips), $T =$ the transverse distance from the reinforcement to the pile center, $D =$ the outside pile diameter (same units as T), and $\sigma_V =$ the vertical stress (psf). When P is given in kN and σ_v is given in kPa, then the regression equation becomes:

$$\Delta F(kN) = 10\wedge\left(0.0986 + 0.0117P - 3.07x10^{-5}P^2 + 0.0146\sigma_v - 1.45x10^{-4}\sigma_V^2 - 0.116\frac{T}{D}\right) - 1 \qquad (4)$$

A total of 942 data observations were used in the regression analysis of the ribbed strip reinforcements resulting in an R^2 value of 0.71. The maximum induced force, *ΔF (kips)* (>0), induced in a ribbed strip reinforcement due to lateral pile load is given by the equation:

$$\Delta F(kips) = 10\wedge\left(0.13 + 0.028P - 2.2x10^{-4}P^2 - 0.01\frac{T}{D} - 0.0021P\frac{T}{D} - 0.031\frac{S}{D}\right) - 1 \qquad (5)$$

where S = the distance from the back of the wall to the center of the pile (same units as D), P is in kips and σ_v is in psf. When P is given in kN and σ_v is given in kPa, then the regression equation becomes:

$$\Delta F(kN) = 10\wedge\left(0.34 + 0.012P - 2.66x10^{-5}P^2 - 0.029\frac{T}{D} - 5.53x10^{-4}P\frac{T}{D} - 0.0535\frac{S}{D}\right) - 1 \qquad (6)$$

A comparison between the predicted and measured maximum log of maximum tensile force plus one kip is shown in Figs. 15 and 16 for the welded-wire grid and ribbed strip reinforcements, respectively. Data on the red line indicates that the measured and predicted values are equal. Considering the complexity of the interactions involved, the agreement is reasonably good. Even in cases where reinforcement resistance is only produced by fill weight, significant variation in pullout resistance has been observed, particularly at shallow depths (Lawson et al. 2013). Dashed lines showing boundaries for plus and minus one and two standard deviations are also shown in each figure which allows a user to select a more conservative estimate of induced reinforcement force, if desired.

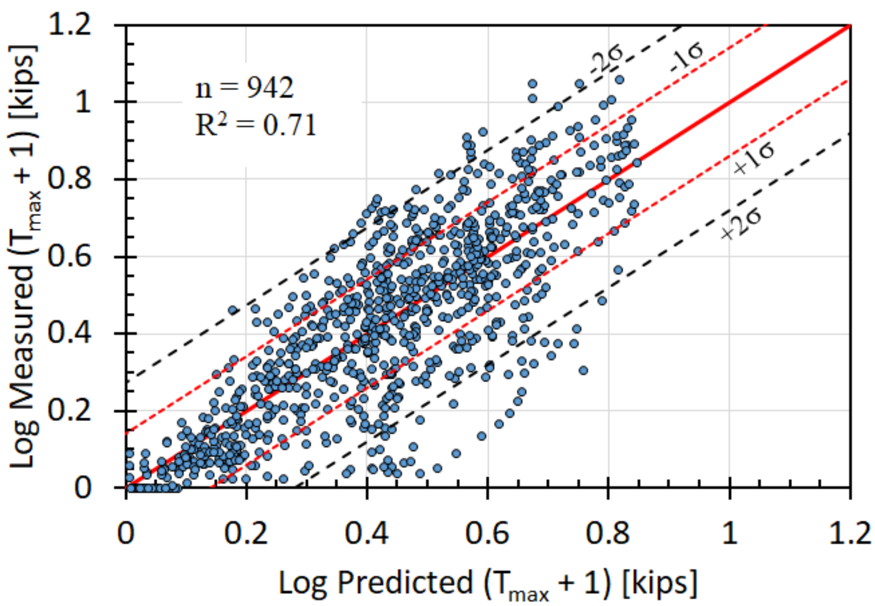


Fig. 15. Comparison of measured and predicted log of tensile force for ribbed strip reinforcements.

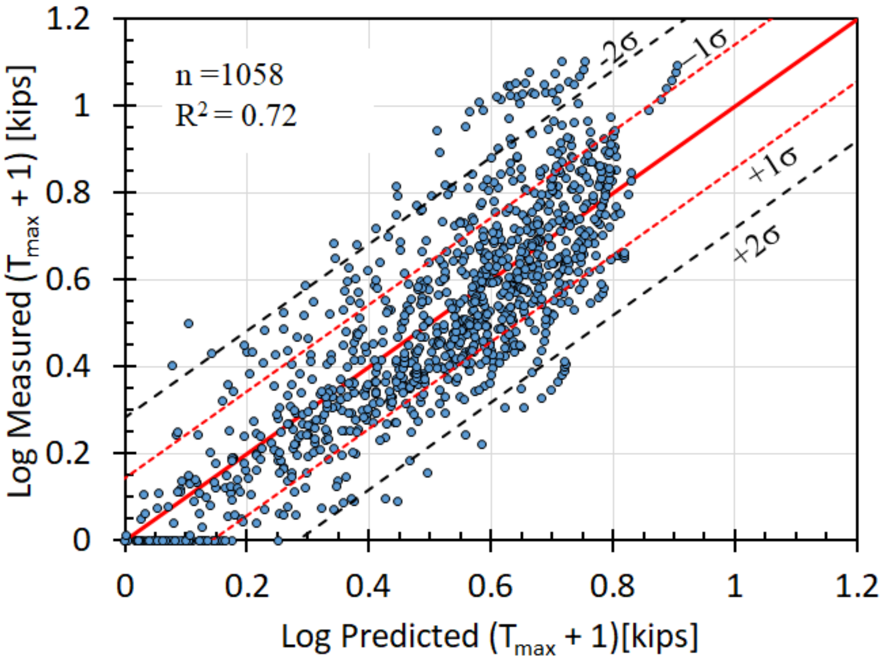


Fig. 16. Comparison of measured and predicted log of tensile force for welded-wire reinforcements.

9.0 CONCLUSIONS

1. Lateral pile resistance is relatively unaffected for small diameter (32.4 to 40.6 cm, or 12.75 to 16 in.) piles located more than about four pile diameters behind an MSE wall but decreases for closer distances.
2. Based on this study and previous test data, a simple p-multiplier approach provides reasonably accurate estimates of lateral load-displacement curves. Lateral soil resistance remains relatively constant (p-multiplier of 1.0) for piles located greater than approximately 3.9 pile diameters (3.9D) behind an MSE wall with inextensible reinforcements. For piles spaced closer than 3.9D, a linear reduction in the p-multiplier was observed.
3. Variations in reinforcement length to height (L/H) ratios, pile shape, and inextensible reinforcement type do not appear to significantly or consistently affect the p-multiplier vs. normalized pile spacing (S/D) relationship.
4. Lateral wall deformations for the concrete MSE wall panels were generally less than 12.7 mm (0.5 in.) with inextensible reinforcements even at large pile head deflections [76.2 mm (3 in.)] and loads of 226 kN (60 kips).
5. Maximum induced tensile force occurs in the soil reinforcement near the pile location rather than at the wall face owing to the development of a failure wedge as the pile deflects towards the MSE wall.
6. Maximum induced tensile force increases as pile head load (and resulting deflection) increases. In contrast, maximum induced tensile force decreases as transverse distance between the reinforcement and the pile center increases and distance behind the wall increases.
7. The statistical regression equations developed for ribbed strip and welded wire reinforcement in this study account for about 70% of the variation in maximum induced tensile force observed in the field testing.

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