Static & Dynamic Passive Force-Deflection Behavior of Abutments with MSE Confined Approach Fills

Interim Report on Load Tests Performed for
Work Task 6

by

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ABSTRACT

Approach fills behind bridge abutments are commonly supported by wrap-around mechanically stabilized earth (MSE) walls. In contrast to a sloped fill, the vertical MSE wall face would tend to reduce the passive resistance provided by the abutment wall. However, the reinforcing strips provide confinement to the approach fill which would increase the passive resistance. This paper describes the first large-scale tests to evaluate passive force-deflection curves for abutments with MSE side walls. A test was also performed with fill extending beyond the edge of the abutment wall for comparison. The abutment wall was simulated with a pile supported cap 5.58 ft high, 11 ft wide, and 15 ft long in the direction of loading backfilled to 5.5 ft. The backfill behind the pile cap consisted of clean sand compacted to 96% of the modified Proctor maximum density. During lateral loading, the MSE wall panels moved outward 1.4 inches when the ultimate passive force developed. However, the passive force was still 76% of the resistance provided by the cap with fill extending beyond the edges. The normalized passive force-deflection curves for the tests with and without the MSE walls were similar and reached an ultimate at deflections of 3.8% and 4.2% of the wall height for the MSE wall confined and unconfined backfills, respectively. The measured ultimate passive force is compared to the computed ultimate passive force for the Rankine, Coulomb, log spiral and Caltrans methods. The log spiral method underestimated the ultimate passive resistance by 36% while the Caltrans method provided an excellent estimate. Damping ratios estimated using the half-power band width approach indicate damping ratios of 0.40 to 0.45 which remained relatively constant with initial static displacement and for both backfill conditions.

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INTRODUCTION

Passive force-deflection curves play a significant role in seismic design of bridge abutments. Numerical analyses conducted by El-Gamal and Siddharthan (1998), Faraji et al. (2001), and Shamsabadi et al. (2007) indicate significant influences of abutment stiffness on bridge response. Several large-scale tests have been conducted to gather data to quantify this relationship so that the results may be used in improved design models (Rollins and Cole 2006, Rollins and Sparks 2002, Mokwa and Duncan 2001, Romstad et al. 1996). These tests have typically used a backfill which extends beyond the edges of the pile cap or abutment. Commonly, however, space constraints necessitate that wrap-around mechanically stabilized earth (MSE) walls be used which would truncate the approach fill soil to the width of the bridge or abutment wall. This truncation reduces the effective width of the approach fill, which in turn tends to decrease the passive resistance on the abutment. In contrast, the reinforcing grids would tend to provide increased confinement to the soil backfill and increase the resistance.

This paper describes the first large-scale test to assess the effects of MSE side walls on the passive resistance provided by soils in approach fills. As a comparison, a test with backfill extending beyond the edges of the pile cap abutment wall, subsequently referred to as an unconfined backfill, was also performed. Passive force-deflection curves were developed under static loading using two hydraulic actuators with capacities of 600 kips in compression and 450 kips in tension. The passive force-deflection curves were obtained by subtracting the response of the pile cap without backfill from the total force-deflection curves with backfill in place. The backfill for both tests consisted of clean sand compacted to 96% of the modified Proctor maximum density. The heave and cracking patterns in the backfill during the progression of each test was documented as was the transverse outward movement of the MSE wall.

The ultimate passive force obtained from the testing was compared with various methods for computing passive force, namely the Rankine, Coulomb, log spiral, and Caltrans methods. In addition, the measured passive force-deflection curves were compared with curves computed using the hyperbolic method and the Caltrans method.

TEST LAYOUT AND PROCEDURE

Backfill Confined by MSE Walls Test

Figures 1a and 1b show a plan and elevation view of the MSE wall confined backfill test layout and data instrumentation. The abutment wall was simulated with a pile supported concrete cap 5.58 ft high, 11 ft wide and 15 ft long in the direction of loading backfilled to a height of 5.5 ft. The top of the pile cap was level with the surrounding ground elevation; therefore, an area behind the pile cap was excavated to permit the MSE walls to be constructed adjacent to the cap. The MSE wall panels were 12-ft x 5-ft x 6-in reinforced concrete. To match the height of the cap, pieces of 2x8 sawn lumber were used to extend the height of the wall panels by one-half foot. The MSE walls on each side of the cap and backfill consisted of two MSE wall panels oriented in the direction of loading. As constructed, the MSE walls on either side were 5.5 ft tall and 24 ft long. The bottoms of the panels were placed on timber 4x4s that laid flat on the web of a steel I-beam that was turned on its side. This structure acted as a leveling pad for the wall panels and maintained alignment on the soft bottom of the excavation. The top of the 4x4s were level with the bottom of the pile cap. During backfilling, the panels were held plumb with bracing while the clean sand backfill was placed and compacted in 8 to 12 inch lifts. During this process, two rows of five bar steel reinforcing grid panels (16 grids total) were placed at heights of 1.33 and 3.83 ft from the bottom of the MSE wall. The edge of the first grid was approximately 12 inches from the pile cap face. The grid reinforcement was designed according to FHWA standards and was embedded into the backfill 5.5 ft., midway of the pile cap width. Deflection data during testing was gathered using string potentiometers. Longitudinal movement of the pile cap was measured from an independent reference frame while measurements of the compression movement in the soil backfill was referenced from the pile cap face to metal stakes placed near the centerline of the sand backfill at distances of 2, 6, 12, and 16 ft behind the cap. Transverse movement of the MSE walls was also measured. Three string potentiometers were located on the East MSE wall panel closest to the pile cap at distances of 1.33, 5.25, and 10.92 ft from the pile cap face. Load cells within each hydraulic actuator were used to measure the force applied to the pile cap and soil backfill system and then summed to obtain the total applied force. The hydraulic actuators reacted against a system consisting of two 4 ft diameter concrete drilled shafts and a sheet pile wall driven just in front of the concrete shafts. A 5-ft x 28-ft I-beam was placed on either side of the shaft-sheet pile system to distribute the load through the system. The hydraulic actuators were post-tensioned to the I-beams with 2 1/2 inch all-thread-bar.

The procedure for the test was to load the pile cap with the hydraulic actuators in approximately 1/4 inch increments. The actual deflection increments used were: 0.20, 0.61, 0.68, 1.07, 1.30, 1.55, 1.75, 1.98, 2.27, 2.51, 2.76, 3.05, 3.30, and 3.51 inches. At increments 0.20, 0.61, 1.30, 1.98, 2.76, and 3.51, fifteen cycles corresponding to a typical cap displacement of 1/8 inch were applied by the actuators at a frequency of 0.75 Hz. Immediately before or after cycling, an eccentric mass shaker applied a dynamic load of up to 100 kips with frequencies increasing from 1 to 10 Hz at these specific increments. Since passive force-displacement relationships can be improved by considering the effects of damping of the system these dynamic loads were applied to quantify this effect.

Backfill Without MSE Walls Test

Figure 2 provides an elevation view of the test setup for the unconfined backfill extending beyond the edges of the pile cap. The layout for the unconfined soil backfill test consisted of the same pile cap and reacting system. An area behind the pile cap was excavated below the bottom of the cap approximately one and a half feet for a distance of eight feet and then sloped at 1V:3.5H at an elevation equal to the bottom of the pile cap. This geometry was considered large enough to enclose the passive failure surface. The excavation extended five feet beyond the width of the pile cap to allow for 3D end effects of the failure wedge.

The testing procedure for this backfill condition followed a similar pattern as the MSE wall confined soil backfill test. The actual deflection increments were: 0.12, 0.26, 0.46, 0.64, 0.88, 1.19, 1.50, 1.80, 2.07, 2.27, and 2.50 inches. Cyclic and dynamic loadings were applied at every increment during this test in the same manner as described above for the MSE wall test. Again, the analysis and results of these cyclic/dynamic effects are beyond the scope of this paper.

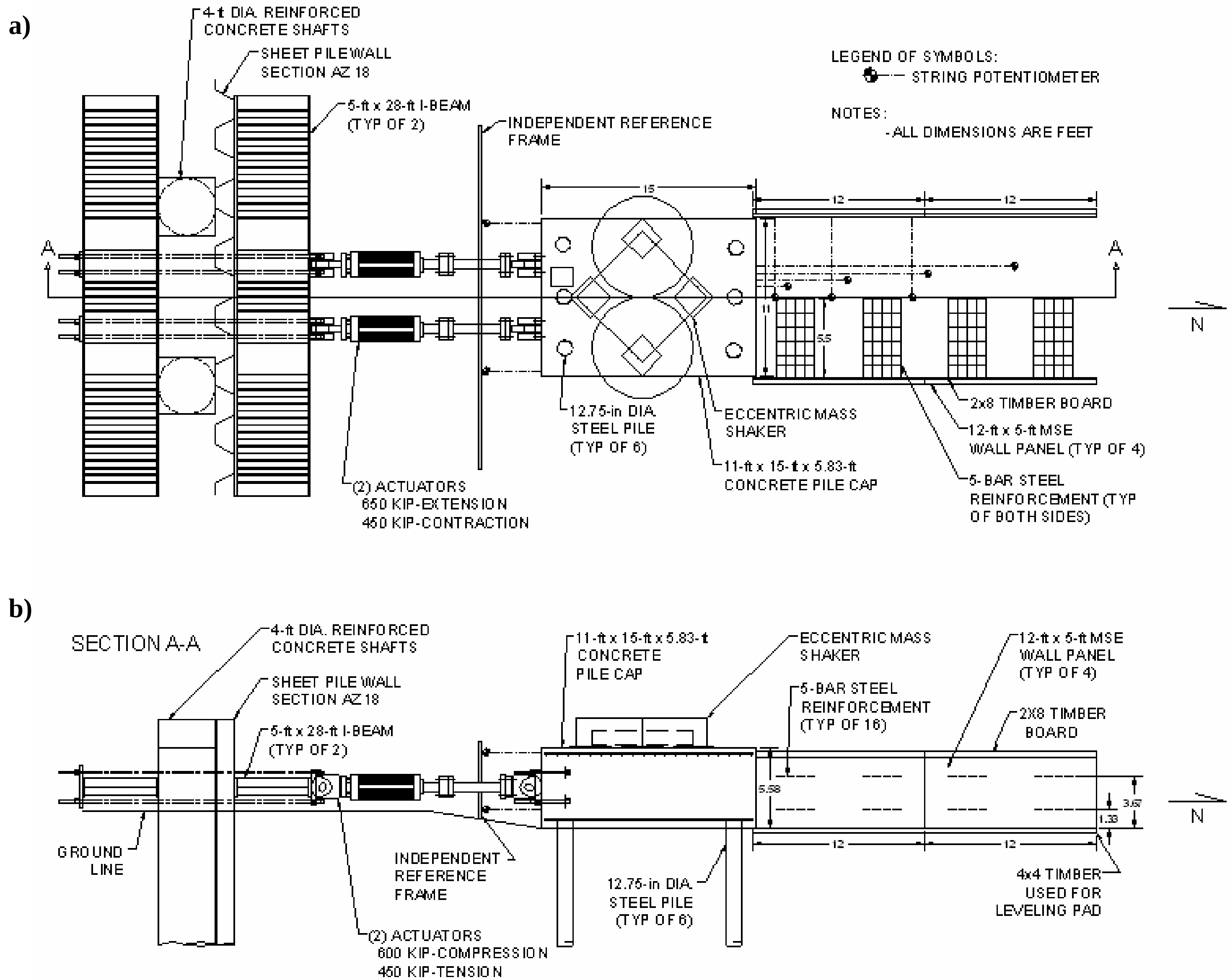

Figure 1. (a) Plan view and (b) elevation view of MSE wall confined soil backfill test layout and instrumentation

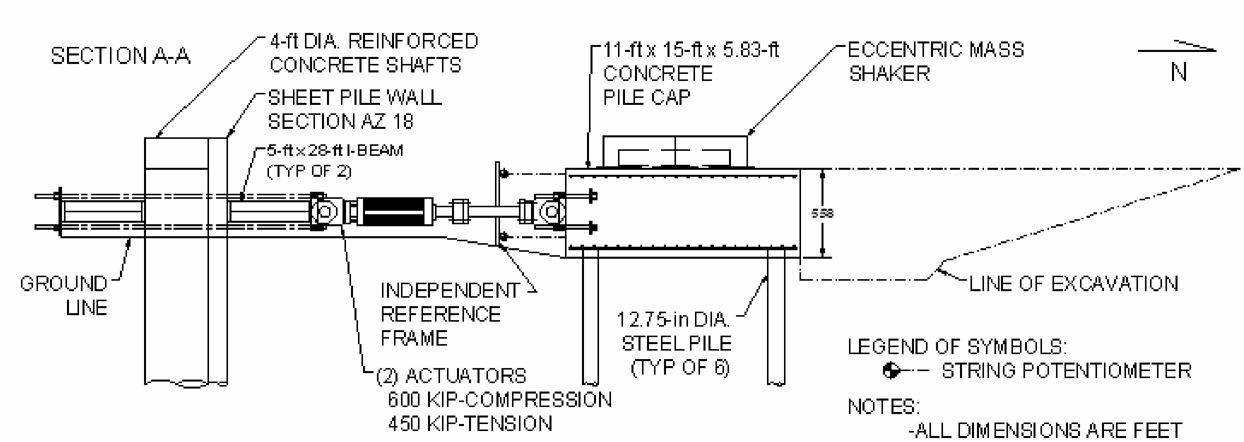

Figure 2. Elevation view of unconfined soil backfill test layout and instrumentation.

TEST RESULTS

Test results presented in this paper include plots of the static total and passive force-displacement curves for the backfill confined by MSE walls as well as for the unconfined backfill. In addition, plots of the observed backfill cracking and vertical heaving patterns for each test are provided. Finally, the longitudinal backfill displacement and the transverse or outward movements of the MSE wall panels are presented as a function of distance behind the wall for a number of longitudinal pile cap displacements.

Backfill Behavior

Surface cracking of the soil backfill was observed during the progression of each test. Figure 3 shows the observed cracking, shown as dashed lines, and vertical heave contours, shown as solid lines, at the maximum displacement associated with both tests. A 2-ft square grid was painted on the backfill to identify and document the cracking. The amount of vertical heave was measured with surveying equipment at the beginning and end of each test. The clean sand grain-characteristics and the cyclic and dynamic loading made it difficult to observe smaller cracks in the surface of the backfill and contributed to some error in the heave measurements due to loosened sand on the surface being shifted by the dynamic loading. However, the larger, well defined cracks were noted and documented. For the MSE wall confined backfill, it was observed that cracking occurred both parallel and perpendicular to the pile cap face. The parallel cracking occurred within 4 feet of the face of the cap while the perpendicular cracking occurred from 6 to 22 ft from the cap face. This observation is made noting that the walls moved outward engaging the grid reinforcement allowing cracks to form parallel to the MSE panels. For the unconfined backfill without MSE wingwalls, radial cracking beginning near the corners of the pile cap developed out to a distance of about 4 feet beyond the edges of the pile cap. This pattern is indicative of 3D end effects and resulted in an effective failure surface width of approximately 18 feet for the unconfined fill. Although there is a significant difference in the widths of the effective soil wedges, the maximum vertical heave for both tests was approximately 1.30 inches and occurred 6 feet from the pile cap face near the center of the width of the backfill.

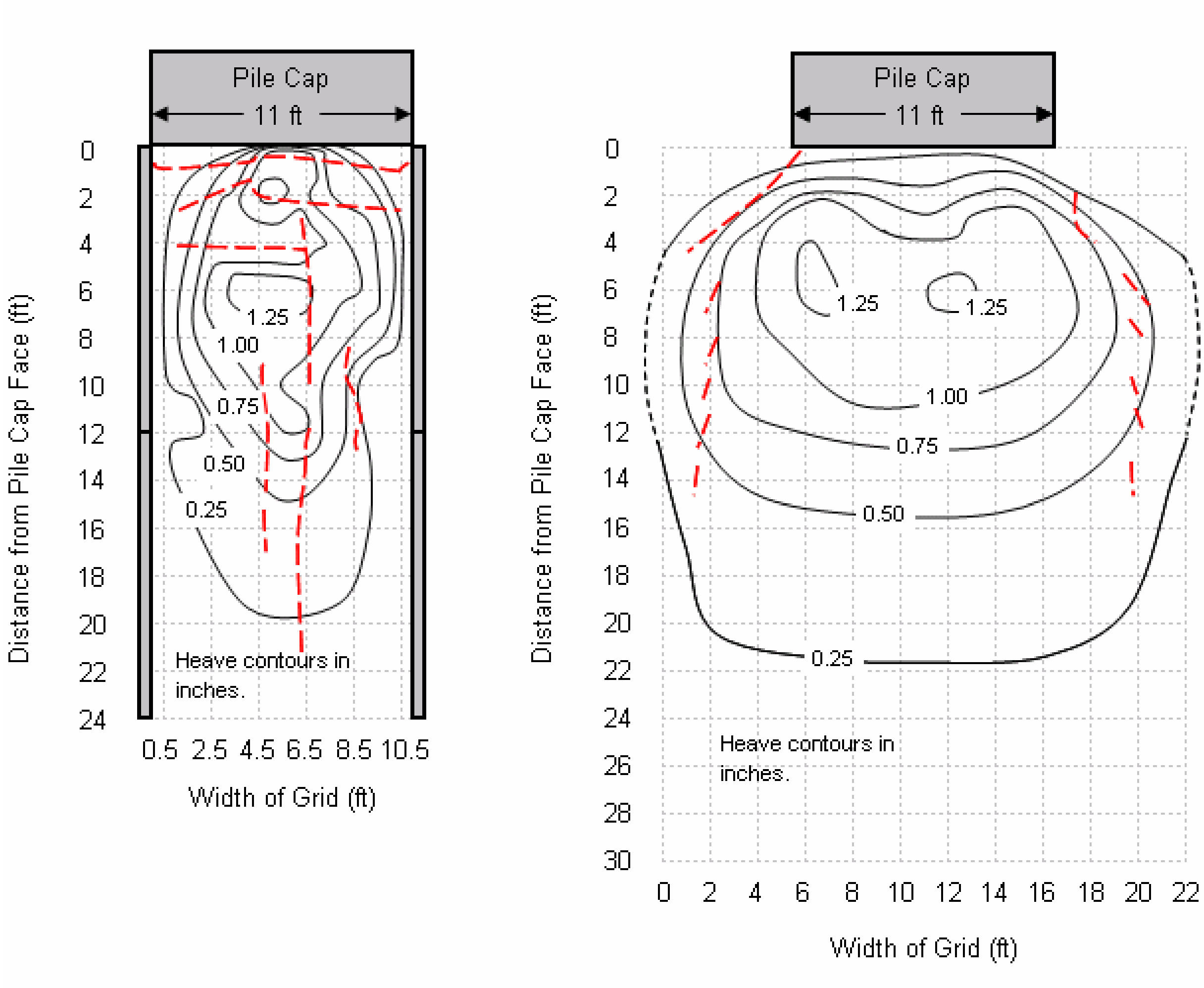

Figure 3. Observed cracking and vertical heave patterns at the maximum displacement. Cracking shown as dashed lines and vertical heave contours shown as solid lines.

Compressive Movement of the Backfill and Transverse Movement of the MSE Wall

String potentiometers were attached to the face of the pile cap and then to steel stakes placed within the sand backfill for each test. The stakes were placed at distances of 2, 6, 12, and 18 ft from the face of the pile cap. Figure 4 shows the relative movement, or change in length, between the pile cap and the stake as they got closer during the progression of testing due to the deflection of the pile cap and compression of the backfill. The dashed lines represent compression for the test with the MSE backfill while the solid lines represent compression during the unconfined soil backfill test. For easier comparison, pile cap deflections at 0.5, 1.0, 1.5, 2.0, and 2.5 inches for each test were used instead of the incremental deflections mentioned earlier for each test. The plots in Figure 4 indicate that there was significantly less compressive movement in the sand backfill for the MSE wall confined test than for the test where the fill extended beyond the pile cap edges, particularly at distances of 12 and 18 ft behind the pile cap. This is most likely due to the transverse outward movement of the MSE walls themselves. Figure 5 shows the transverse movement of the MSE walls versus distance from the pile cap face as the

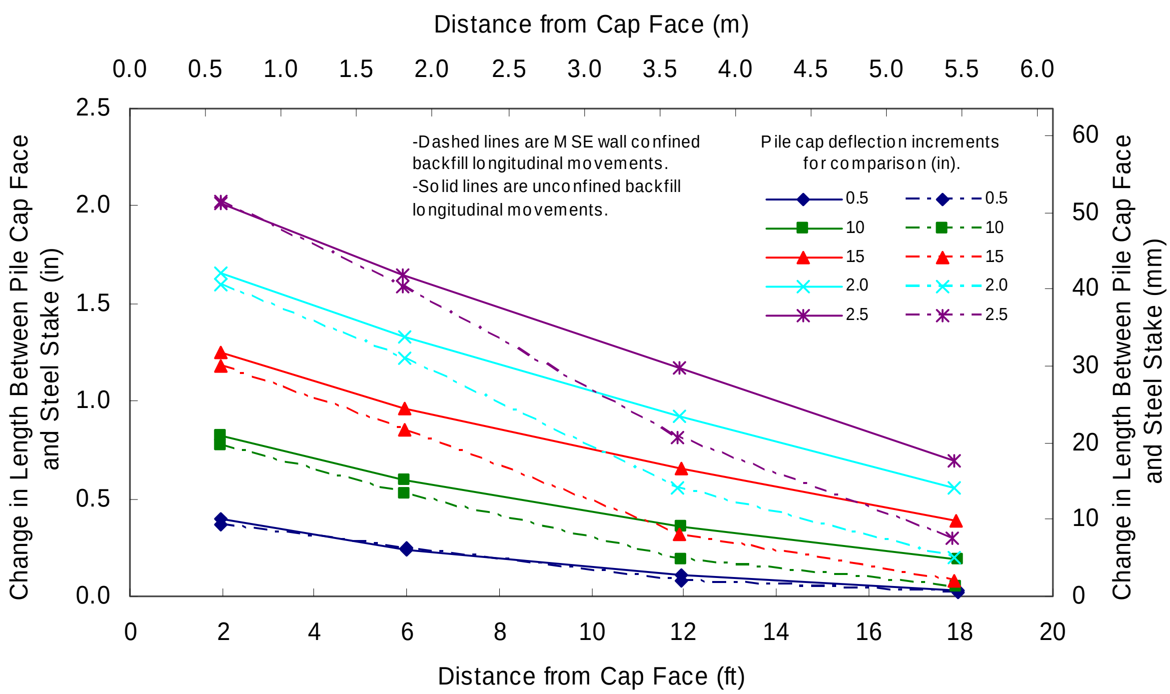

Figure 4. Compressive behavior of soil backfill as a function of distance from the pile cap face as the pile cap deflected longitudinally.

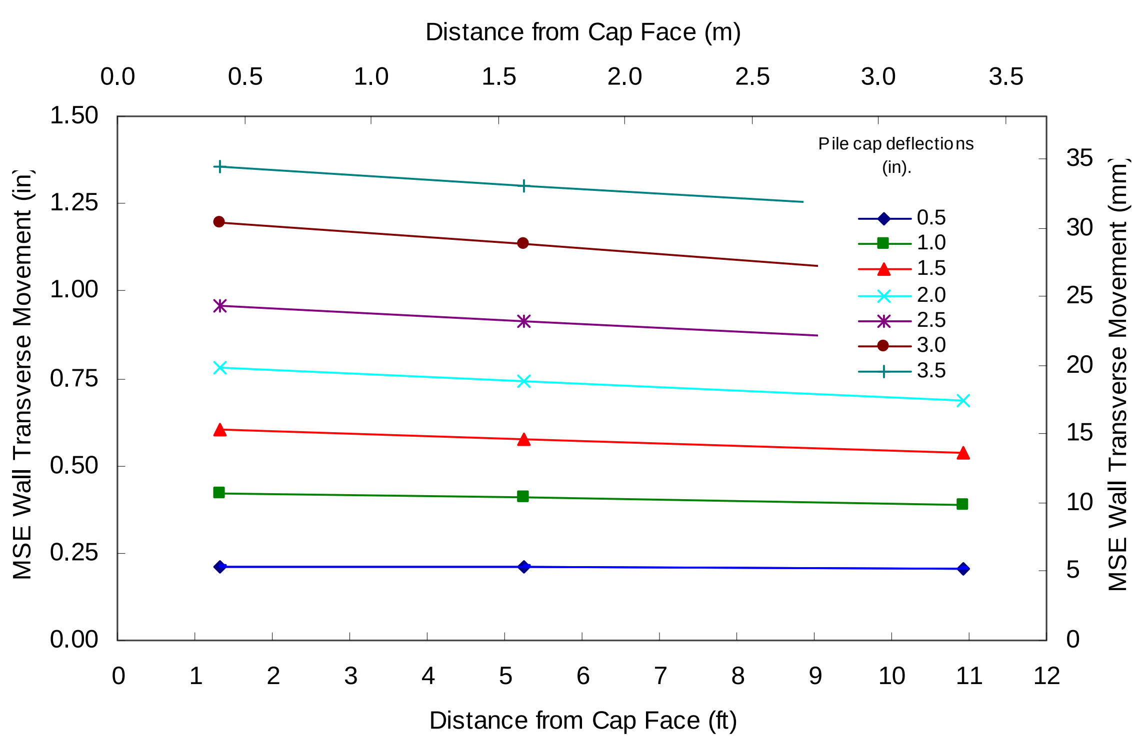

Figure 5. Transverse outward movement of MSE wall as a function of distance from the cap face as the pile cap deflected longitudinally.

test progressed. Again, curves are plotted for each half inch longitudinal displacement of the pile cap to facilitate comparison with data in Figure 4. String potentiometers were attached to the pile cap and to steel stakes located at the center of the width in the sand backfill. Figure 5 shows that the entire instrumented MSE wall panel translated outward with the largest movement near the pile cap face. This behavior indicates that at large strains or displacements of the pile cap, the reinforcement strips began to pull through the sand backfill as the walls translated outward. The transverse wall displacement was typically about 40% of the longitudinal cap displacement. The maximum outward transverse movement of the MSE walls was 1.4 inches and occurred at a pile cap deflection of 3.50 inches. Wall movement was primarily translational, with little rotation.

Earth Pressure Cell Measurements

Earth pressures were measured using six 230 mm-diameter stainless steel earth pressure cells. The cells were designed with a reinforced backplate to reduce point loading effects when directly mounting the cell to a concrete or steel structure, and the cells utilize a semi-conductor pressure transducer rather than a vibrating wire transducer to more accurately measure rapidly changing pressures. The cells were embedded flush with the face of the pile cap and centered at depths of 0.21, 0.49, 0.77, 1.05, 1.33, and 1.61 m below the ground surface.

The pressure distribution curves as a function of depth are plotted for each deflection increment in Figure 6. The curves generally show a linear trend line consistent with expectations based on theory. This is particularly true for the case of the test involving the MSE wingwalls. However, for the test without wingwalls, the pressure tends to increase near the top of the pile cap suggesting either some cohesive component of resistance or rotation of the pilecap. In both tests, the bottom pressure cell shows a decrease in pressure at the higher deflection levels. It is still unclear at this stage of the analysis if this is a bad reading or is related to some real physical phenomenon. At higher displacement increments, the curves tends to converge suggesting a failure state. This is particularly true for the test with the MSE wingwalls. The earth pressures are higher for the test without wingwalls because the total applied force is also higher as discussed subsequently. The total force on the pile cap has been computed based on the pressure distributions measured by the pressure cells and the computed passive force is always less than the measured force based on the difference in load applied by the actuators with and without backfill in place. This could be due to non-linear pressure distributions along the width of the cap. Elastic solutions of soil-structure interaction problems typically indicate stress concentrations at the edges of foundations or walls. Since the pressure cells are located near the center of the pile cap, they might be expected to measure somewhat lower earth pressures than the average over the entire width of the cap if higher pressures concentrations develop at the edges.

Measured Total and Passive Force-Deflection Curves

Figure 7 shows the total and passive force versus displacement curves for the backfill with and without MSE walls. Data points correspond to the deflection increments at which the cyclic and dynamic loadings also occurred as mentioned in the Test Layout and Procedure section. Since the static loads dropped after cycling and shaking, the corresponding loads

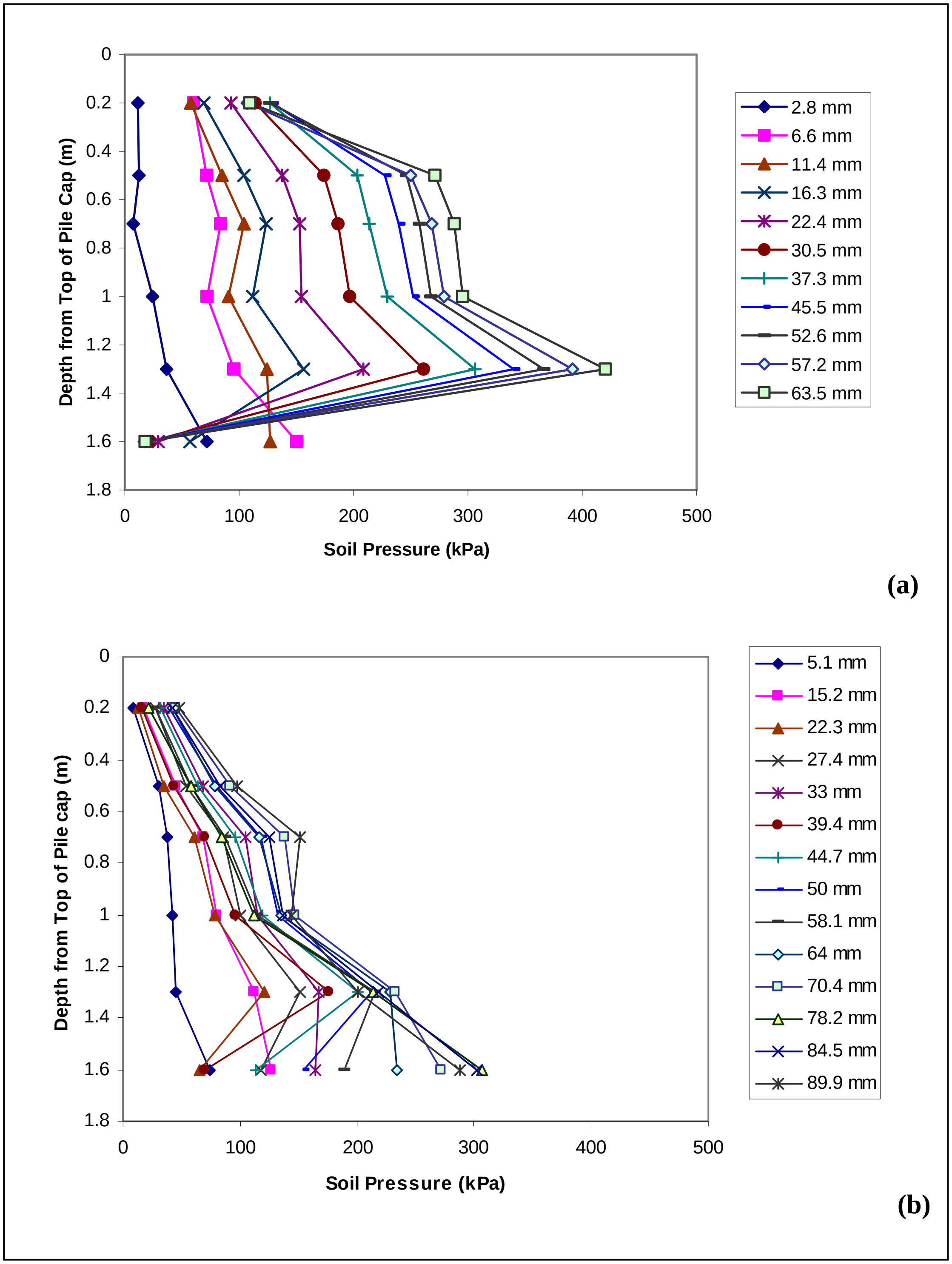

Figure 6. Earth pressure distributions as a function of pile cap displacement for (a) dense clean sand backfill without wingwalls and (b) dense sand backfill with MSE wingwalls.

represent a peak load at the given deflection. The curve connecting the data points is defined as the "backbone" curve. It may be observed that the force deflection curves are not as smooth as might be expected. This "roughness" is likely due to the cyclic and dynamic loading during the test sequence. In some cases, it appears that the static displacement for the increment following the cyclic was insufficient to bring the reload curve back to the virgin load-displacement curve. Figure 7 shows that at the maximum deflection of the unconfined backfill of 2.5 inches, the total resistance provided by the pile cap and backfill with MSE walls is 79% of the resistance provided by the pile cap with backfill extending beyond the edges, a 21% reduction in total resistance.

The passive resistance for each test was computed by subtracting the resistance provided by the pile cap system without backfill from the total resistance provided by the pile cap and backfill with and without MSE walls. It should be noted that this test setup was used to test a variety of different backfill materials and configurations including the dense sand backfill with and without MSE walls during the summer of 2007. The unconfined dense sand backfill test was conducted on May 25th while the MSE wall confined dense sand backfill test was performed on June 18th due to logistical constraints. Consequently, a June 1st baseline test without backfill was used to calculate the passive resistance for the backfill without MSE walls while a June 21st baseline test was used to calculate the passive resistance for the backfill with MSE walls.

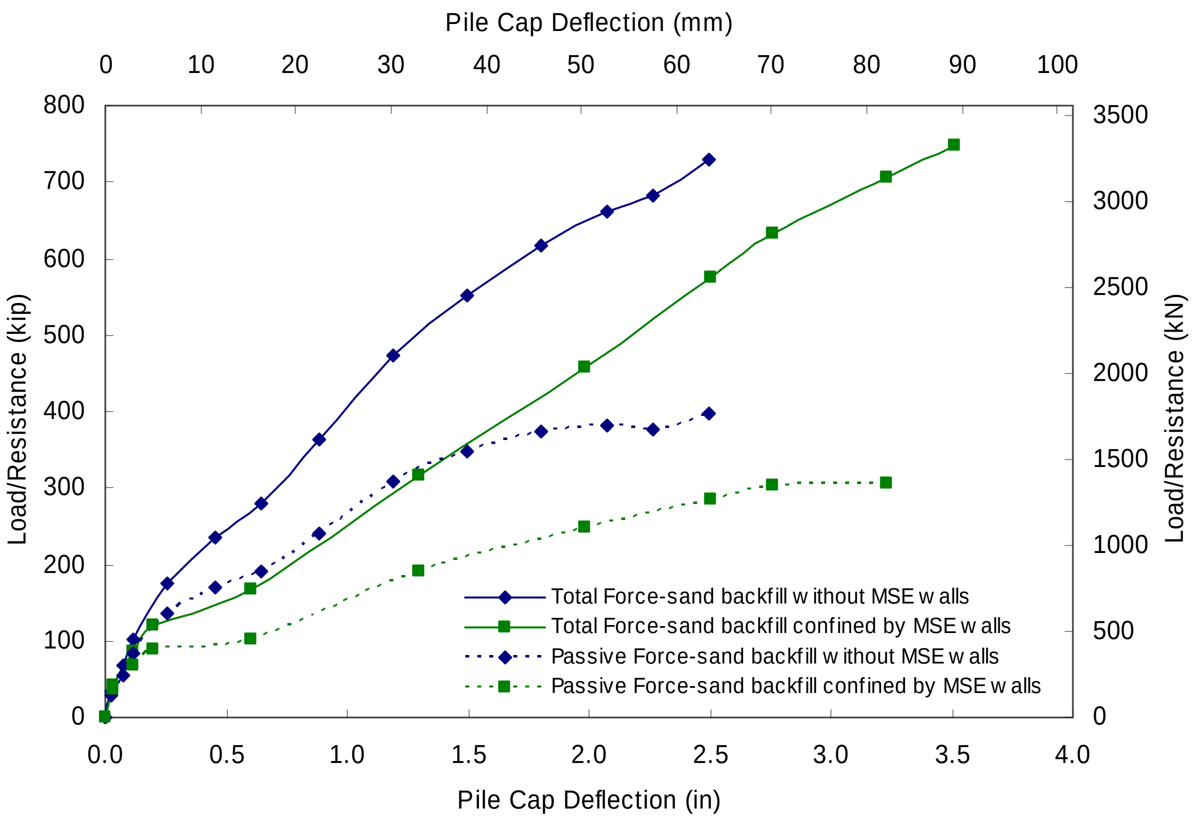

Figure 7. Total and passive force versus deflection curves

At the ultimate state, the MSE wall confined backfill developed 76% of the passive resistance provided by the backfill which extended beyond the edges of the pile cap. The increased passive force for the unconfined backfill appears to be largely attributable to the increased effective width of the pile cap. As noted previously, surface shear zones extended 3 to 4 ft beyond the edge of each side of the cap, effectively increasing the pile cap width from 11 ft to about 18 ft. The ultimate passive resistance occurred at pile cap deflections equal to 4.2% and 3.8% of the wall height for the MSE-confined and unconfined backfill tests, respectively. This deflection level is somewhat greater than the 3.0 to 3.5% range for full-scale lateral pile cap tests in dense sands and gravels reported by Rollins and Cole (2006).

Comparison of Measured and Computed Response

The ultimate passive force was computed for both tests using the Rankine, Coloumb, log spiral and Caltrans methods for comparison with the measured ultimate force. For the MSE wallconfined backfill the cap width was 11 ft, while the effective width of the unconfined backfill was computed as 19.6 ft using the Brinch-Hansen (1966) equation which accounts for 3D shear effects at the ends of the cap. Based on previous studies, the friction angle of the backfill was 39º and wall friction was 0.77 times the friction angle (Rollins and Cole, 2006). The moist unit weight was 117 lbs/ft^3. The Caltrans method was developed based on field tests of large-scale abutments 5.5 ft high with concrete wingwalls confining the soil backfill (Romstad et al 1996). For this wall height the Caltrans method computes the passive force as 5.5 kips/ft^2 times the area of the cap (Caltrans 2001). A summary of the computed passive force using the various methods is provided in Table 1 along with the measured force. The log spiral method, with allowance for shearing beyond the edge of the cap, provided an excellent estimate of the ultimate passive force for the backfill without MSE wingwalls. However, it underestimated the ultimate passive resistance for the backfill with MSE walls by 36% because it does not account for any confinement effects provided by the MSE wall. The higher passive force could be adequately accounted for by using a plane strain friction angle (42.6º) rather than the triaxial friction angle (39º) which accounts for the failure geometry constrained by the MSE wall.

The Caltrans method provided an excellent estimate of the ultimate passive resistance for the MSE confined backfill, but underestimated the ultimate passive resistance for the unconfined backfill because it does not account for 3D shearing effects at the edges of the cap. After accounting for 3D shearing effects, the Rankine method still underestimated the ultimate passive force by a factor of 2.5 to 3, while the Coloumb method overestimated the ultimate passive resistance by 25% to 75%.

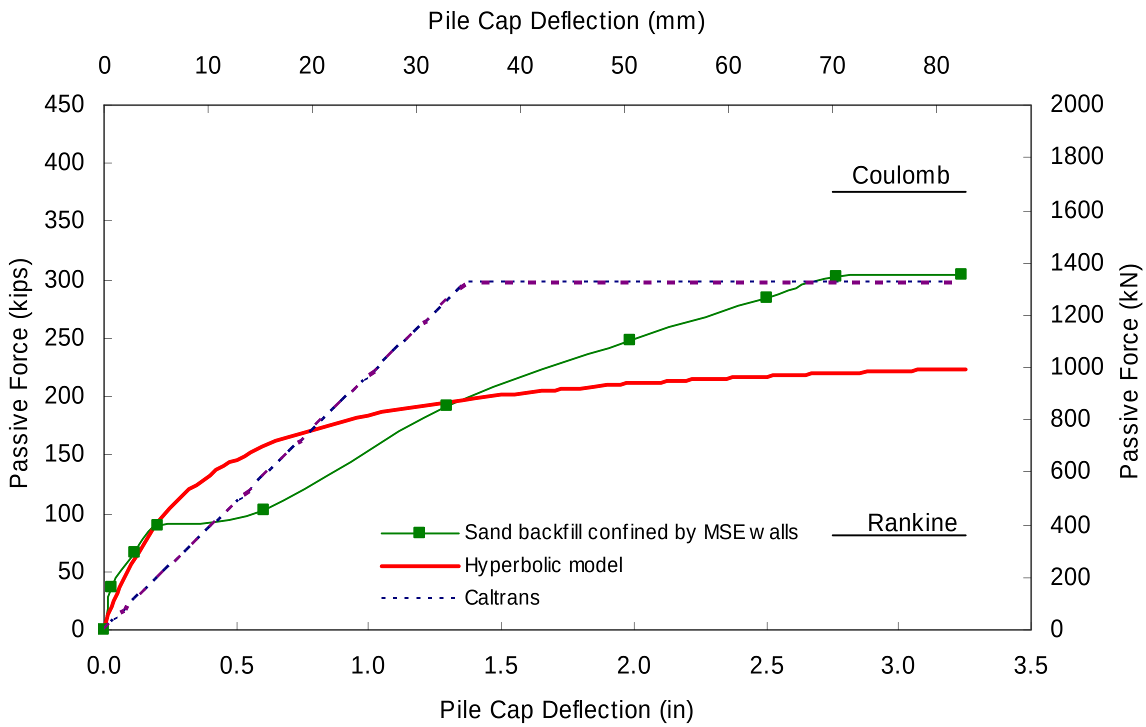

Figure 8. Comparison of measured MSE wall confined backfill passive force and computed passive force versus deflection relationships.

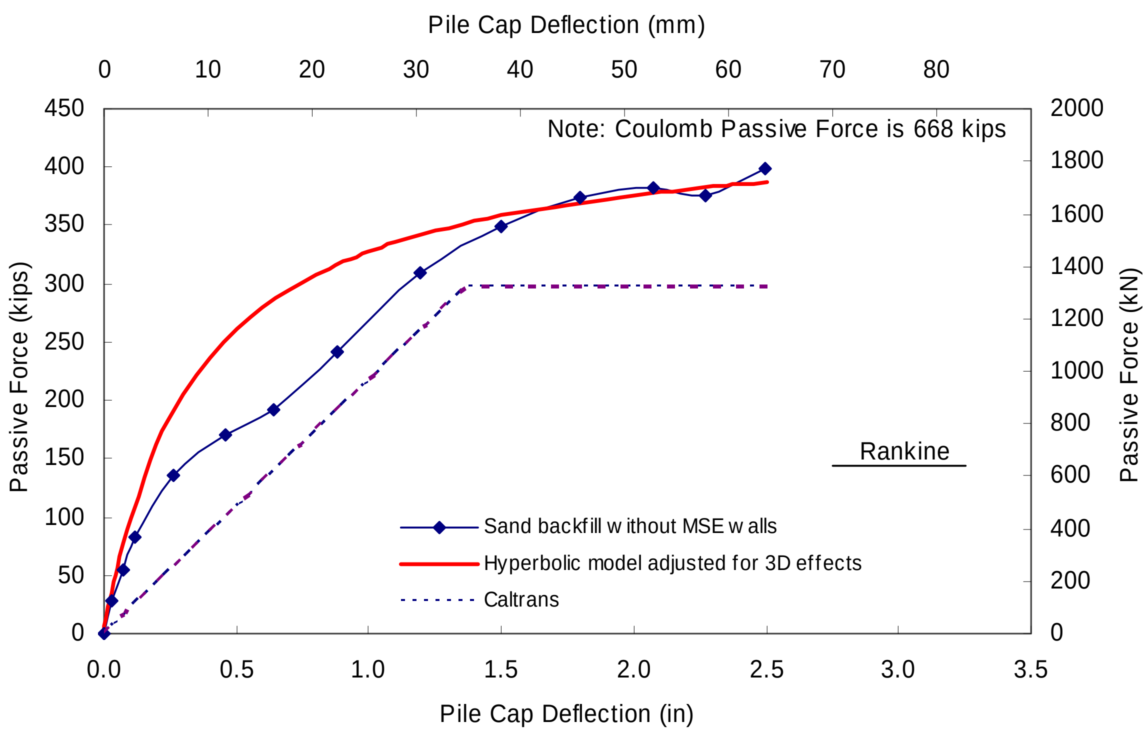

Figure 9. Comparison of measured and computed passive force versus deflection relationships for backfill without MSE walls.

TABLE 1. Comparison of measured and computed peak horizontal passive force.

Peak horizontal passive force (kips)					
		Computed			
Backfill Type	Measured	Rankine	Coulomb	Log spiral	Caltrans
MSE wall confined fill	305	81	375	213	298
Fill without MSE walls	398	144	668	380	298

Passive force-displacement curves were computed using the hyperbolic method (Duncan and Mokwa 2001) and the Caltrans approach (Caltrans 2001) and are compared with the measured curves for the backfills with and without MSE walls in Figures 8 and 9, respectively. The Caltrans method uses a stiffness of 20 kips/in per ft of pile cap width. The hyperbolic method uses an initial stiffness based on the elastic modulus which was taken as 775 kip/ft^2 based on previous studies (Cole and Rollins 2006). The hyperbolic method provided a reasonable estimate of the initial stiffness of the backfill in both cases as well as a reasonable fit to the curve for the unconfined backfill, but provided a poor match for the curve for the MSE wall confined backfill. The Caltrans method underestimated the initial stiffness by a factor of two. Neither, method matched the measured curve throughout the entire loading cycle. This may be partly due to softening of the curve due to cyclic loading during the testing process as discussed previously.

Measured Damping and Dynamic Stiffness

The cyclic/dynamic response of the pile cap was assessed by examining load-displacement loops from both cycling of the load actuator and operation of the eccentric mass shaker. Figures 10 and 11 show results obtained from cyclic loading by the actuator. These figures show the magnitude of the cyclic displacement (peak-to-peak amplitudes can be found by doubling the values shown) as well as the associated stiffness and damping at a low frequency (~ 0.75 Hz). Figure 10 shows results for the dense sand backfill without wingwalls while Figure 11 shows results for the dense swand with the MSE wingwalls. In each test the cyclic displacement remained relatively constant with values of 1 to 1.5 mm. Dynamic stiffness tended to increase substantially as the initial static displacement level increased in both cases; however, the stiffness was higher for the dense sand without wingwalls than the dense sand MSE wingwalls because the total force was larger as discussed previously. The damping ratio for both backfills was reasonably similar with values typically ranging between 0.25 and 0.15. The damping ratios tended to decrease as the initial static displacement increased.

The damping associated with loadings from the eccentric mass shaker was interpreted by based on the area of the loops and with the half-power band width method. The half-power band width approach is the more conventional approach. To this point, the computations using the loop areas have not yielded consistent and reliable results. The damping ratios obtained with the half-power band width approach are presented in Figures 12 and 13 for the backfills without and with the MSE wingwall. The damped natural period of the pile-cap-backfill system, as determined from peak dynamic displacement amplitudes normalized by shaker force, is approximately 7 Hz. As shown in Figures 12 and 13, the amount of system damping is relatively constant for all displacement levels and backfill conditions, being on the order of 40 to 45%. The damping ratio was nearly the same for both backfill conditions.

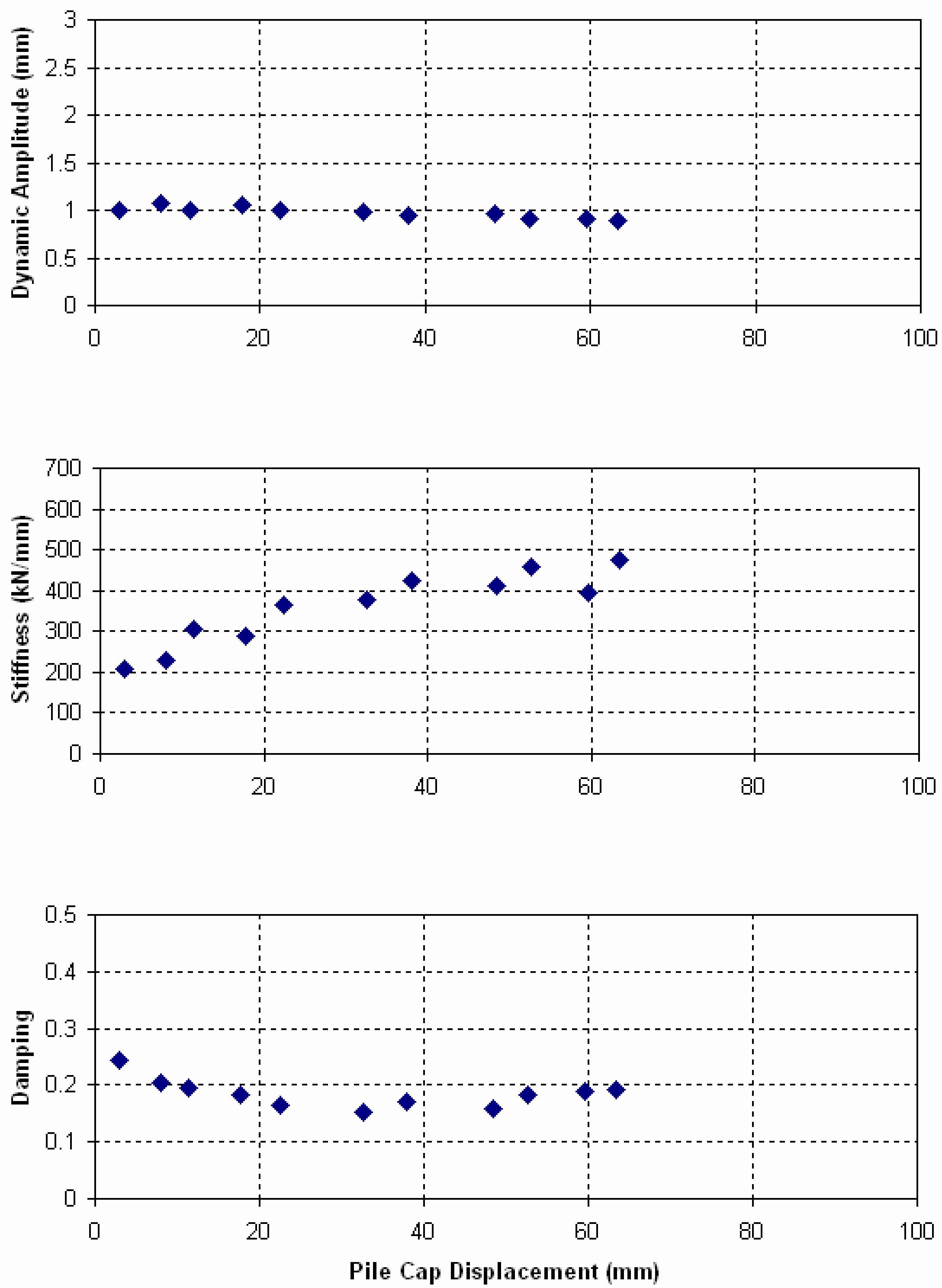

Figure 10. Results for cyclic loading with actuator for Dense Clean Sand backfill conditions

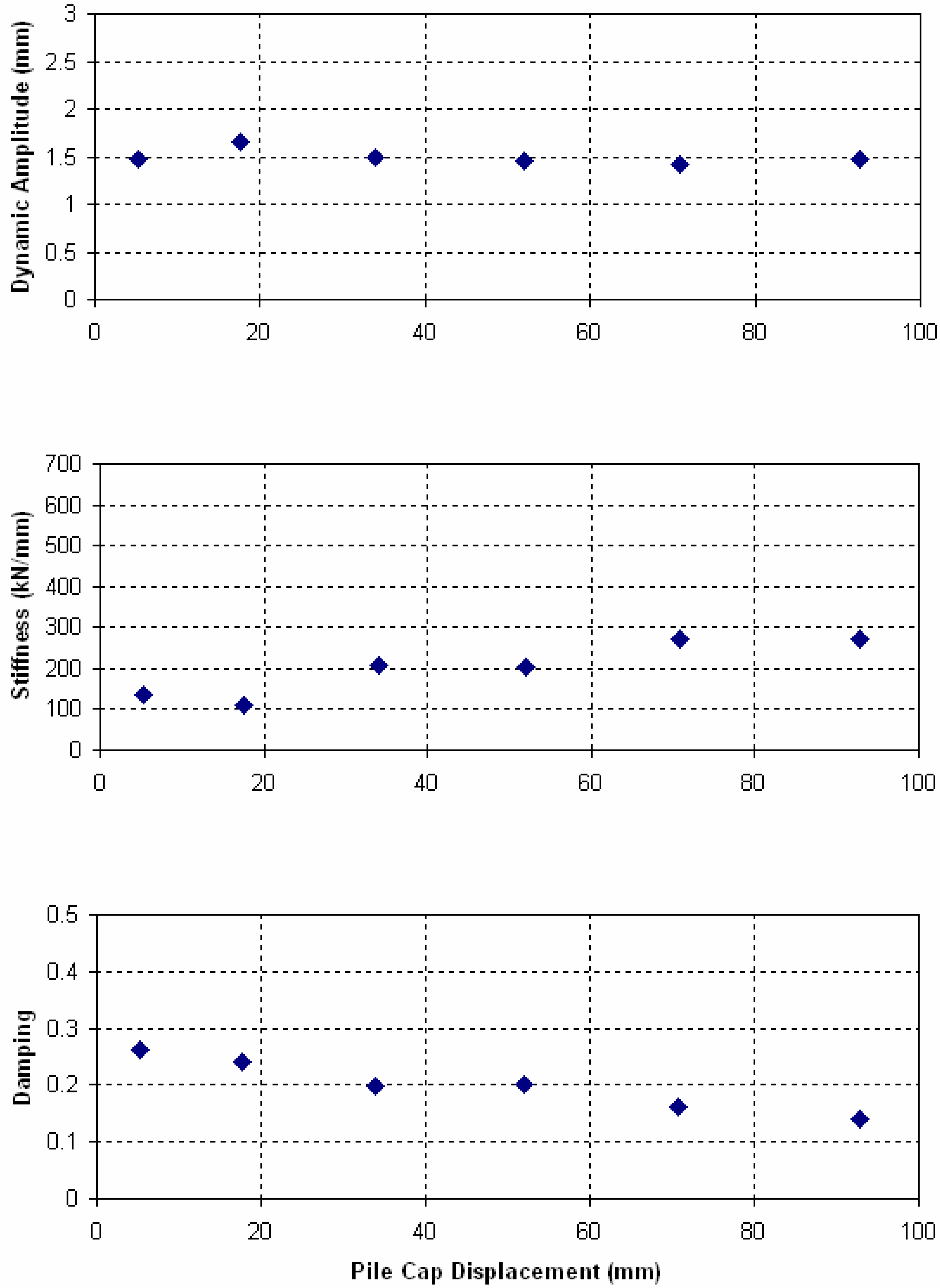

Figure 11. Results for cyclic loading with actuator for Mechanically Stabilized Earth (MSE) Wall with Dense Clean Sand backfill conditions

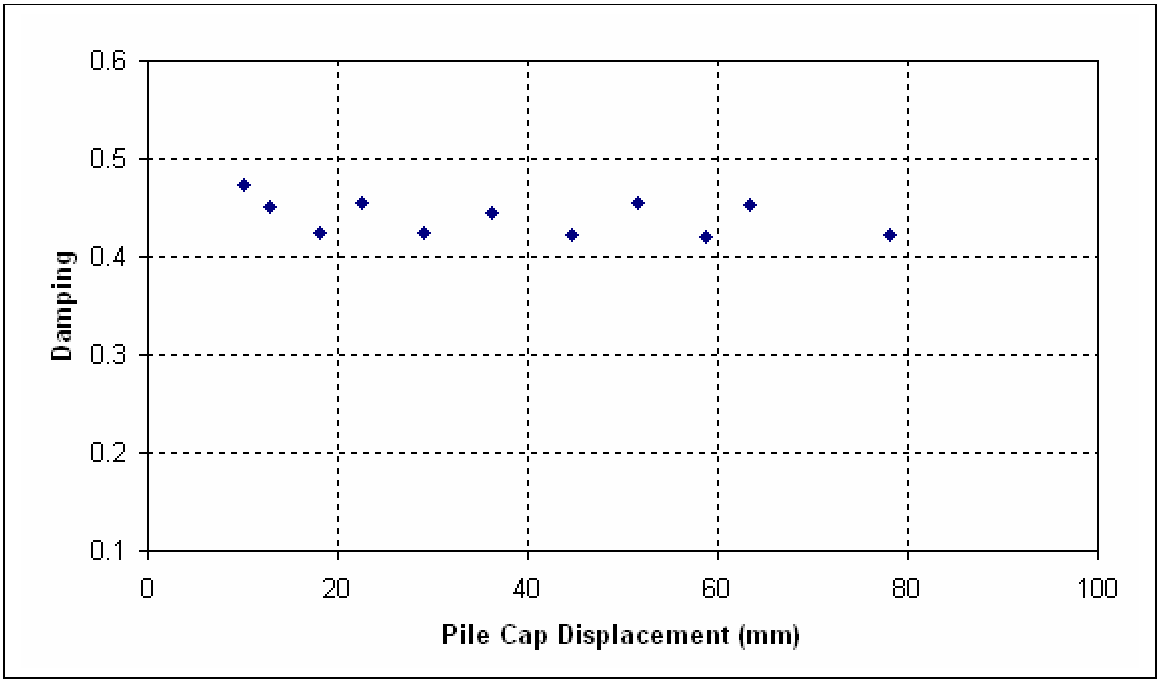

Figure 12 Results for dynamic loading with eccentric mass shaker for Dense Clean Sand backfill conditions

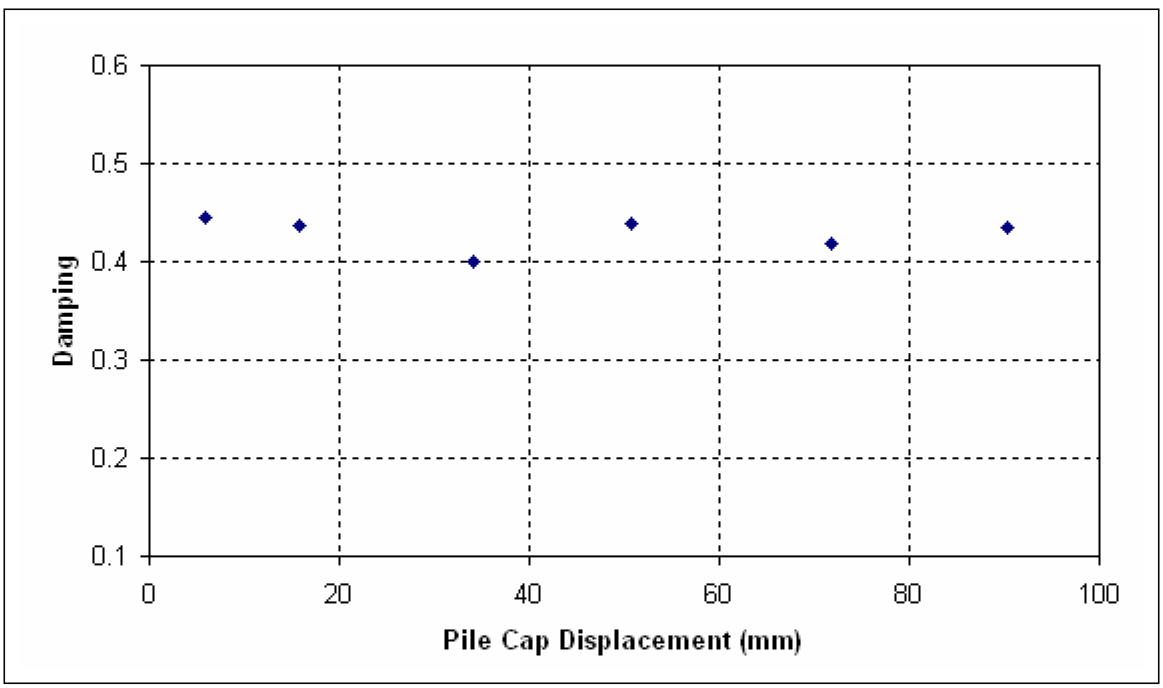

Figure 13 Results for dynamic loading with eccentric mass shaker for Mechanically Stabilized Earth (MSE) Wall with Dense Clean Sand backfill conditions

CONCLUSIONS

Based on the test results and analysis relative to the large-scale field tests on the backfills with and without MSE wingwalls, the following conclusions have been developed:

1. During lateral loading of the unconfined backfill, shear zones extended about 4 ft beyond the edge of the cap increasing the effective cap width from 11 ft to 18 ft.
2. Lateral loading of the pile cap caused outward transverse displacement of the MSE wall which was typically about 40% of the longitudinal displacement of the pile cap. However, the magnitude of heaving behind the pile cap was very similar to that for the cap with unconfined backfill soil.
3. The ultimate passive force for the MSE wall confined backfill was 24% lower than for the unconfined backfill primarily due to the reduced effective width of the cap. However, confinement did increase the passive force above what would be expected based on the reduction in cap width.
4. The log spiral method, with allowance for shearing beyond the edge of the cap, provided an excellent estimate of the ultimate passive resistance for the unconfined backfill. However, it underestimated the ultimate passive resistance for the MSE confined backfill by 36% because it does not account for confinement effects provided by the MSE wall.
5. The Caltrans method, which was developed based on tests involving concrete wingwalls, provided an excellent estimate of the ultimate passive resistance for the MSE wall confined backfill, but underestimated the ultimate passive resistance for the backfill extending beyond the width of the pile cap because it does not account for 3D shearing effects at the edge of the cap.
6. After accounting for 3D shearing effects, the Rankine method still underestimated the ultimate passive force by a factor of 2.5 to 3, while the Coloumb method overestimated the ultimate passive resistance by 25% to 75%.
7. The hyperbolic method provided a reasonable estimate of the initial stiffness of the backfill, while the Caltrans method underestimated the initial stiffness by a factor of two. Neither method matched the measured curve throughout the entire loading cycle. This is likely due to the effects of cyclic loading at the end of static loading cycles.
8. Damping measurements obtained by the half-power band width approach indicate damping ratios of 0.40 to 0.45 which relatively unaffected by initial static displacement or the wall configuration.

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