

Rammed Aggregate Pier Load Test Results

FEATURE

ARTICLE



Pier test site

The Middle East consists of vast deserts that reach temperatures up to 50°C (122°F), with some areas characterized by endless sand dunes. When project team members selected the location for a large new transportation project, little did they know that the site would be characterized by soft, nearly normally consolidated silt and clay extending below a thin sand crust.

Searching for solutions, the design team elected to reinforce the sub-surface with rammed aggregate pier ground improvement elements and wanted verification that the method would stand the test of prolonged static loads. Because the concrete pavements would serve to spread the load, a “zone load test” was performed, consisting of applying a uniform bearing pressure of 150 kPa (22 Psi) over a 7.5 m (25 ft) square concrete

footing. Unlike individual pier load tests, the zone test allowed the project engineers the opportunity to confirm design parameter values under large-loading area conditions, a situation not often available to most designers.

This paper presents the results of the zone load test, which may be interpreted to better understand design parameter values, and is of particular significance because it adds a valuable contribution to the relatively few numbers of “group load tests” available in the literature for ground improvement applications.

Introduction

The new transportation project will be located midway between two cities with large, anticipated growth in the next few years. The nearby metropolitan areas are projected to become

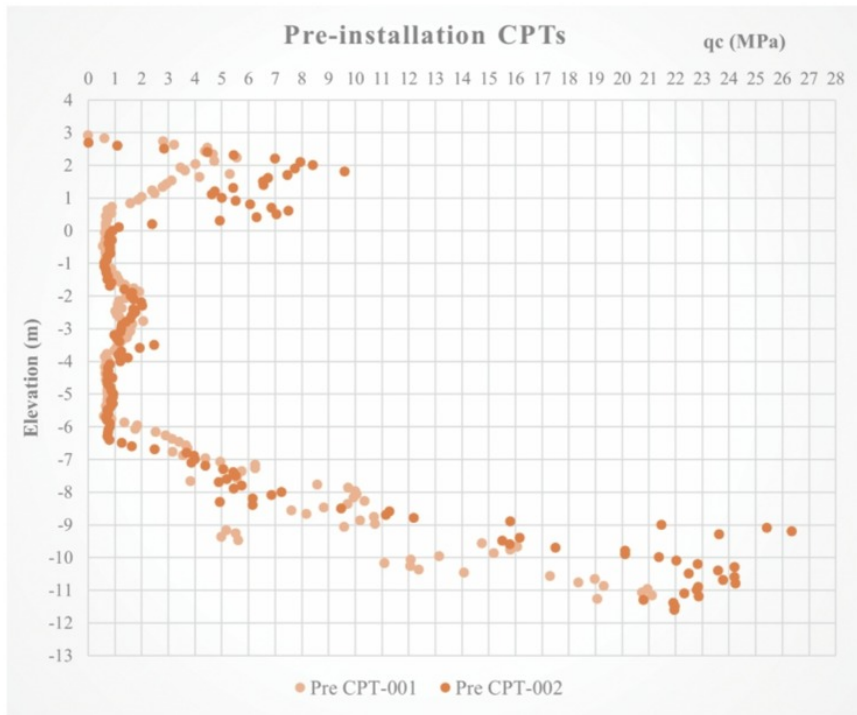
manufacturing and logistics hubs and the new transportation project needed to be designed to serve the rapidly growing population in the area.

Such an important and ambitious project did not present ideal soil conditions for the project team. Beneath the ground surface elevation of about EL +3 m (10 ft), was a layer of eolian deposits consisting of loose to medium dense sands and very soft to soft sandy silts with pockets of clay to about 10 m (33 ft) depth, followed by medium dense to dense Quaternary deposits. Groundwater was generally encountered between 1–5 m (3–16 ft) depth.

Static loads of up to 1,066 kN (240 kip) were specified in the contract documents, resulting in an equivalent uniform pressure of 30 kPa (4 Psi) at the bottom of grade raise fill elevation. Site

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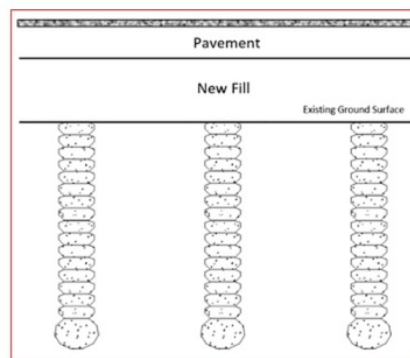
CPT results of the unimproved in-situ soils

grades needed to be raised by 2.1 m (7 ft), resulting in an estimated 50 mm (2 in) settlement that would need to have been completed prior to placement of the pavement. Further, the design loads would have resulted in up to 150 mm (6 in) of static settlement, which would have been excessive.

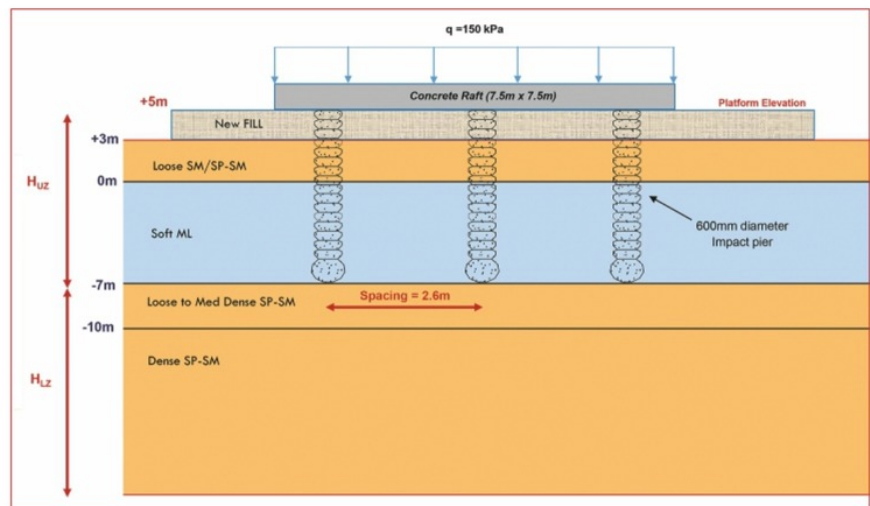
The site is also situated in a seismically active area; and with a design magnitude of Mw 7.5 and peak ground accelerations on the order of 0.2g, the site was determined to be highly susceptible to liquefaction. Significant post-liquefaction settlements of up to 200 mm (8 in) were anticipated in the loose sands and soft silts. The high risk of post-seismic displacements rendered the pavement particularly vulnerable.

Proposed Ground Improvement Solution

The project specifications required an improved allowable bearing capacity of 150 kPa (22 Psi). Total static settlements were not to exceed 50 mm (2 in) and differential settlements were not to exceed 1/500 from service load



Typical pavement and fill support detail



Zone load test

conditions for a raft. Post-liquefaction settlements needed to be controlled to 25 mm (1 in) or less.

It became very clear to the design team that an effective ground improvement system was required to achieve adequate performance of the pavements. The system needed to reduce the delay time from placement of grade raise fill and provide control of static and post-seismic settlements.

The ground improvement elements selected provided superior stiffness over traditional stone columns and are able to greatly densify granular soils between the elements. They have been increasingly implemented over the years as an effective ground improvement system alternative for support of a variety of structures and in soil conditions ranging from very soft or loose to very stiff or medium dense. Recent studies in New Zealand and Ecuador had also evidenced that the elements are very effective at densifying in situ coarse grained soils and provide a stiffening shear response in silts.

The team chose the Impact pier system with a constructed diameter of 600 mm (24 in). The system features a vibro-displacement technique for driving the mandrel in the ground. The mounted mast system, vibro hammer and patented compaction chamber aid to impact very high vertical ramming

energy during pier construction to create highly densified aggregate piers. The displacement technique is particularly effective for maximizing densification on the in-situ soils. Open-graded stone was to be used to construct the piers.

Recent studies in New Zealand and Ecuador had also evidenced that the elements are very effective at densifying in situ coarse grained soils and provide a stiffening shear response in silts.

The piers were designed to fully penetrate the soft silts and loose sands and tag into the underlying loose to medium dense sands, with installed shaft lengths ranging from 9–12 m (30–40 ft) throughout the site. The pier spacing was selected as 2.6 m (8.5 ft) on-center, which would control static settlements and provide sufficient densification and reinforcement to control liquefaction.

Performance Verification

A zone load test was specified by the project team to verify the performance of the pier design under static loads. The test consisted of a 7.5 m (25 ft) square concrete footing with a thickness of 0.9 m (3 ft) per the design team's specification. The footing was to rest directly on top of the piers and be loaded to a uniform pressure of 150 kPa (22 Psi) per the required allowable bearing pressure. To comply with project performance specifications, the footing settlements needed to be controlled to 50 mm (2 in) after a 24-hour period at full test load. The test piers were to be installed from the test platform elevation, which corresponded to the top of the new fill elevation.

Poured concrete footing with settlement points



Zone Load Test Setup

The zone load test was setup in the following manner:

1. The site was levelled, and proof rolled. The natural ground level was at approximately EL +3 m (11 ft) and the dimensions of the test site were on the order of 50x50 m (164x164 ft).
2. Two preinstallation cone penetration test (CPT) soundings were performed inside the test site.
3. New fill was placed to reach the test platform elevation of approximately EL +5 m (16 ft). The select fill consisted of American Association of State Highway and Transportation Officials' Soil Classification System Groups A-3 and A-2/4 placed in loose layers without compaction.
4. 600 mm (24 in) diameter piers were installed throughout the entire test site (50x50 m [164x164 ft] area). The piers were installed at a design spacing of 2.6 m (8.5 ft) on-center in a square array and to depths ranging from 10.5–12.5 m (35–41 ft) below platform elevation to terminate in medium dense sand.
5. Rapid impact compaction (RIC) points were performed by the general contractor to densify the new fill. Each RIC point was performed between a group of four piers.
6. The platform was levelled using a static roller compactor.



Concrete loading blocks applied to footing

7. Three post-installation CPT soundings were performed 12 days after pier installations were completed. The post-installation CPTs were performed at distances of 0.45, 0.85 and 1.85 m (1.5, 3 and 6 ft) from the center of installed piers, pertaining to CPTs 4, 5 and 3, respectively. The post-installation CPTs indicate significant densification of the upper 5 m (16 ft) crust containing the sandy fill and native sands. An improvement factor of at least 2 was observed in the native sands. It can also be seen that minor densification is indicated in the soft silts, which is to be expected in fine grained soils.

8. The concrete footing was poured, being centered above nine piers. A total of five settlement points were established on top of the concrete footing. One point was set in each corner and one in the center of the footing. The settlement from the weight of the concrete footing was measured to be 7.5 mm (0.3 in). The concrete blocks were stacked on top of the footing to apply the prescribed design load to the footing.

The footing was loaded in 25% load increments using 5-ton concrete blocks until the target load of 150 kPa (22 Psi) was obtained, which pertained to 100% of the design load. Settlement was recorded at every stage before loading was resumed. The 100% design load was achieved 16 hours after commencement of loading.

The movement of the five settlement points was monitored using two precision survey levels. The levels were calibrated with an established reference point outside the test zone prior to each reading. Settlement readings were taken for a total period of 24 hours after the 100% load was achieved, in



Concrete loading blocks placed on top of test footing

accordance with the approved method statement for the test. Additional readings were taken up to five days after commencement of loading.

Zone Load Test Results

A total average settlement on the order of 30 mm (1 in) was recorded from the design load of 150 kPa (22 Psi), and the largest differential settlement recorded between two settlement points was 1 cm (12 in) over a 10.4 m (34 ft) distance. The pier design successfully controlled settlement of the zone load test to meet project specifications.

Consolidation occurred rapidly during loading of the footing. Using Boussinesq plots of stress with depth as reference, most of the footing loads

were likely concentrated in the densified sands to 5 m (16 ft) depth below the footing. The settlements on the silt layer were, therefore, much lower than on the densified upper sands. The authors believe that the high permeability of the piers also contributed to rapid consolidation of the silts. The high permeability of the gravel likely facilitated radial drainage into the piers and provided a rapid drainage pathway to alleviate excess pore pressures.

The 100% design load was achieved 16 hours after commencement of loading.

Elastic Modulus Back-Calculation

Given the relative scarcity of large-scale zone load tests on ground improvement elements, the team set out to back calculate the elastic modulus values used for design. Settlements were estimated using a composite elastic modulus approach. This approach consists of summing the estimated settlement in the reinforced zone (the "upper zone") and the estimated settlement in the zone of soil below the bottoms of the elements (the "lower zone").

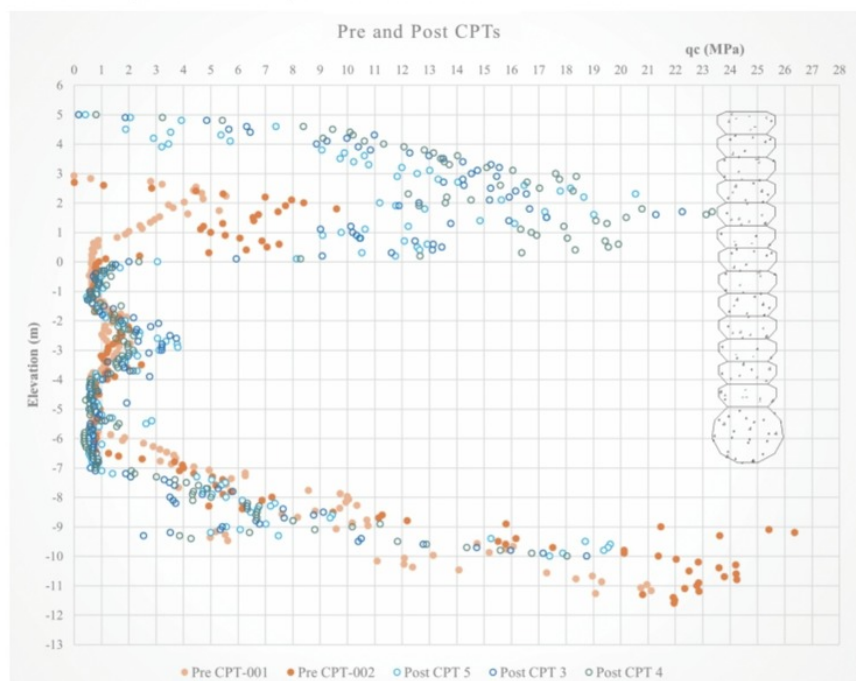
The compression of the upper zone is estimated using conventional settlement calculations as shown in the following equation:

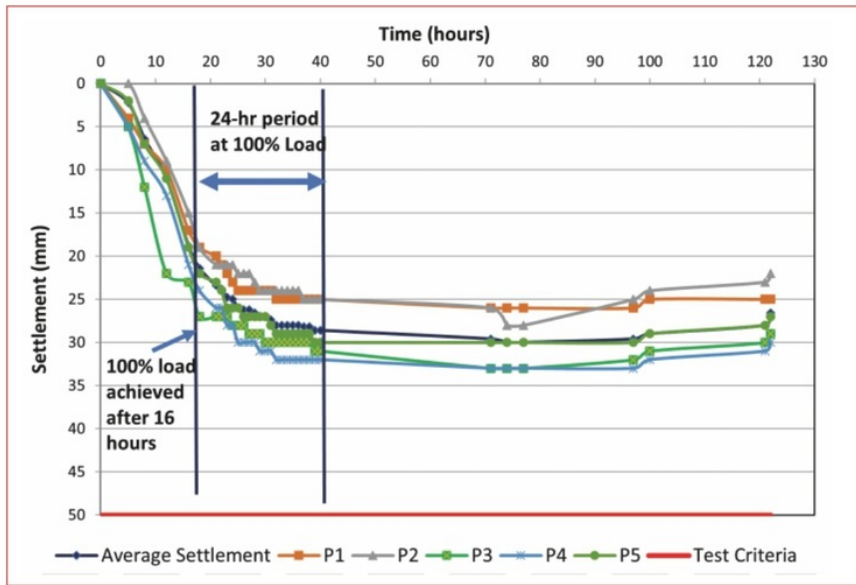
$$S = \frac{q_1 \sigma H}{E_{comp}}$$

Where q is the average bearing pressure, I_σ is the influence factor at depths within the compressible layer, H is the thickness of the compressible layer, and E_{comp} is the composite elastic modulus value within the aggregate pier-reinforced zone.

In cohesive soils, the equivalent constrained modulus (or $E_{equivalent}$) can be back calculated from either compression ratios (C_{ec}) or recompression

Plot of overlain pre-installation and post-installation CPT results at the test site





Settlement during the loading stage and after the full load was achieved.

ratios ($C_{\epsilon r}$), which pertain to normally consolidated and over consolidated behavior from oedometer tests, respectively. The following expression can be adopted for this purpose: Equivalent =

$$\frac{\Delta P}{C_{\epsilon} \log\left(\frac{P_{o'} + \Delta P}{P_{o'}}\right)}$$

Where $P_{o'}$ is the initial vertical effective stress at the center of the layer and ΔP is the change in pressure at the center of the layer.

The composite elastic modulus value is computed based on a weighted average of the elastic modulus values of the rammed aggregate pier elements (E_g) and matrix soil (E_m).

Settlements within the lower zone are computed using conventional geotechnical settlement methods.

The settlement analysis was subdivided into five representative layers for greater accuracy. The area replacement ratio (R_a) was 4.2% based on the pier diameter of 0.6 m (2 ft) and square spacing of 2.6 m (8.5 ft) on-center. The CPT tip resistances were used to obtain the soil elastic modulus E_m in granular soils, whereby the post-installation q_c values were used for the upper zone. The soft silts were estimated to be lightly over consolidated between Elevation 0 and -5 m (0 and -16 ft) and normally consolidated below elevation -5 m (16 ft) after placement of 2.1 m (7 ft) of new fill. A compression ratio of $C_{\epsilon c}=0.2$ and recompression ratio of $C_{\epsilon r}=0.03$ were used in the normally consolidated and over consolidated portions of loading, respectively. A 2:1 (V:H) stress distribution with depth was adopted.

The parameters and settlement for each layer are summarized in the table below.

Conclusion

The following conclusions can be drawn after conclusion of the load test and pier modulus back calculation:

The high permeability of the gravel likely facilitated radial drainage into the piers and provided a rapid drainage pathway to alleviate excess pore pressures.

Elastic Modulus Values and Settlement for Each Layer

Layer Soil Type	Elevation (m)	H (m)	ΔP (kPa)	CPT q_c (MPa)	E_m (MPa)	E_g (MPa)	E_{comp} (MPa)	Settlement (mm)
New Fill	+5 to +3	2	117	15	60	235	67	3.5
Sand Crust	+3 to 0	3	70	15	60	235	67	3.1
Soft Silt	0 to -7	7	33	0.7	6 (average)	110	10	22.3
Sand	-7 to -10	3	19	7	28	--	--	2.0
Sand	Below -10	0	0	>20	80	--	--	0.0
Total								30.9

- The pier design successfully controlled settlement of the zone load test to within project specifications.
- The total settlements obtained from the load test and the back calculation of E_g performed suggests that the Impact pier elastic moduli in the soft silts were higher than what we had assumed prior to the test. The initially assumed pier modulus in the silts was $E_g=75\text{MPa}$ (10,900 Psi). The pier modulus in the sands appears to have been similar to what was initially estimated.
- Settlements occurred quickly during the test. Two likely explanations are that most of the footing load was concentrated in the first 5 m (16 ft) of the soil profile (densified sands), and that the permeable Impact piers provided a quick drainage pathway to speed consolidation in the soft silts.

- It is not clear how much of the densification of the upper sands was due to execution of RIC points, as the post-installation CPTs were performed after the RICs points were performed. As expected, good densification was achieved in the upper sands as these deposits had trace to little fines content, whereas little to no improvement was observed in the silts due to their high fines content.

Gianni Martinez, P.E., has more than 9 years of experience in the geotechnical engineering industry and serves as a senior project engineer at Geopier, where he is responsible for designing ground improvement solutions for projects in the U.S. West Coast, Europe, Middle East and Asia-Pacific regions. He holds a bachelor's degree from Universidad Rafael Landivar in Guatemala and a master's degree from Virginia Tech.

Kord Wissmann, Ph.D., P.E., BC.GE, is the president of Geopier and has more than 25 years of geotechnical-engineering experience. Wissmann holds 30 U.S. patents, has authored more than 35 papers and is a past president of the Geo-Institute. He earned bachelor's and doctorate degrees from Virginia Tech and master's degree from the University of California, Berkeley.

Brian Metcalfe, P.E., has more than 20 years of experience in the geotechnical engineering industry and is the vice president of engineering at Geopier. Metcalfe holds bachelor's and master's degrees from Virginia Tech, and has been an invited speaker nationally and internationally at geotechnical engineering conferences and technical workshops related to the application of ground improvement for foundation support and seismic liquefaction control.



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