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Rammed aggregate pier installation effect on soil properties

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This paper describes the changes in in situ index properties following installation of rammed aggregate piers in a loose sand deposit in New Madrid, Missouri, with a particular focus on the time-dependent strength gain, commonly called sand ageing, as indicated by increased cone penetration test (CPT) tip resistance and dilatometer test (DMT) indices. The method of disturbing the soil and the magnitude of disturbance significantly influence the magnitude of sand-ageing effects. This paper provides the first data on sand ageing following the installation of displacement rammed aggregate piers. In this study, CPT and DMT were performed both immediately before and after the installation of piers. Additionally, CPTs were performed 1 month following the installation of piers. The DMT horizontal stress index increased roughly 100% and the dilatometer modulus increased ~200% in a loose clean sand layer immediately following the installation of the piers. The CPT tip resistance increased roughly 100% in this layer immediately following the pier installation and a further 33% in the month following the installation.

1. Introduction

This paper describes the changes in field testing properties following installation of displacement rammed aggregate piers, commercially known as Impact Piers in a loose sand deposit in New Madrid, Missouri, as determined by cone penetration and dilatometer testing (CPT and DMT, respectively). The particular focus of this study is on the time-dependent changes in CPT tip resistance (q_c) following the installation of the piers, one of several phenomena often referred to as sand 'ageing' in geotechnical engineering. Similarly, time-dependent increases in other in situ test results, such as the standard penetration test (SPT) blow count (N), small strain shear wave velocity (V_s) and DMT indices following disturbance or deposition of a sand layer are reflections of sand 'ageing'. Since in situ test results are commonly the basis for quality assurance in remedial densification projects, failure to account for sand ageing can delay construction schedules.

Mitchell and Solymar (1984) brought attention to the sand-ageing phenomenon, documenting time-dependent increases in the engineering properties of soil following deposition of fill, vibro-compaction and explosive compaction at the Jebba Dam site in Nigeria. Following Mitchell and Solymar's (1984) work, other case histories documented sand ageing following fill deposition, explosive compaction and vibro-compaction, as well as deep dynamic compaction (DDC), vibroseis shaking and earthquakes (e.g. Holzer and Youd, 2007; Saftner, 2011; Schmertmann *et al.*, 1986). However, no case histories have documented sand ageing following installation of piers. The methods of disturbance and disturbance energy significantly influence the magnitude of ageing effects (Dumas and Beaton, 1988; Saftner *et al.*, 2014; Thomann and Hryciw, 1992). Therefore, documenting sand ageing following pier installation fills a gap in current knowledge.

First, this paper presents a review of case histories documenting sand ageing. A description of the pier installation and the in situ testing plan in New Madrid, Missouri follows. In situ test results prior to the pier installation are compared to results from tests conducted immediately following and 1 month following the installation. Finally, the paper addresses the magnitude of the sand ageing in the first month after improvement.

2. Background

The authors found 24 documented cases of sand ageing following vibro-compaction, explosive compaction, DDC, fill placement, and earthquake and vibroseis shaking. The following section briefly summaries these cases.

Two vibro-compaction case histories documented the sand-ageing effects. Mitchell and Solymar (1984) described the construction of the Jebba Dam near Jebba, Nigeria where vibro-compaction was used to densify the upper 30 m of a loose sand deposit. Debats and Sims (1997) reported the site investigation results following vibro-compaction designed to improve the hydraulic fill used in the expansion of the Chek Lap Kok airport in Hong Kong. Details of the vibro-compaction projects are presented in Table 1.

The majority of published sand-ageing case histories describe explosive compaction projects. Fifteen such projects are

Table 1. Summary of data from the vibro-compaction projects summarised in this paper

	Location	Initial tip resistance, q_c: MPa	Ageing time: d	Aged tip resistance, $q_c(t)$: MPa
Mitchell and Solymar (1984)	Jebba, Nigeria	20	15	26
Debats and Sims (1997)	Chek Lap Kok airport, Hong Kong	10	42	13·5

Table 2. Summary of data from the explosive compaction projects summarised in this paper

	Location	Initial tip resistance, q_c: MPa	Ageing time: d	Aged tip resistance, $q_c(t)$: MPa
Mitchell and Solymar (1984)	Jebba, Nigeria	10	21	14
Hryciw and Dowding (1988)	Harriet's Bluff, Georgia	5	30	5
Handford (1988)	Fort McMurray, Canada	9·6	45	12·7
Rogers *et al.* (1990)	Beaufort Sea	11	17	18
Fordham *et al.* (1991)	Fort McMurray, Canada	5·5	30	6·5
Thomann and Hryciw (1992)	Douglas Lake, Michigan	6·5	42	8
Charlie *et al.* (1992)	Greely, Colorado	2·6	126	3
Wheeler (1995)	Sept Iles, Canada	5	12	9
Ashford *et al.* (2004)	Treasure Island, California	4	42	10
Liao and Mayne (2004)	Marked Tree, Arkansas	20	229	20
Liao and Mayne (2004)	Tiptonville, Tennessee	22	229	22
Narsilio (2006)	Charleston, South Carolina	2	484	3
Camp *et al.* (2008)	Charleston, South Carolina	2	30	4
Rollins and Anderson (2008)	Vancouver, Canada	9	27	12
Saftner (2011)	Griffin, Indiana	4·4	75	6·5

Table 3. Summary of data from the DDC projects summarised in this paper

	Location	Initial tip resistance, q_c: MPa	Ageing time: d	Aged tip resistance, $q_c(t)$: MPa
Schmertmann *et al.* (1986)	Jacksonville, Florida	12	10	13·5
Dumas and Beaton (1988)	Sept Iles, Canada	12	8	16

summarised in Table 2. Four explosive compaction projects densified sand layers as part of dam construction (Fordham *et al.*, 1991; Handford, 1988; Mitchell and Solymar, 1984; Wheeler, 1995), one for fill improvement in offshore islands (Rogers *et al.*, 1990), one for improvement of soil-supporting bridge foundations (Camp *et al.*, 2008) and nine as part of research studies (Ashford *et al.*, 2004; Charlie *et al.*, 1992; Hryciw, 1986; Hryciw and Dowding, 1988; Liao and Mayne, 2004; Narsilio, 2006; Rollins and Anderson, 2008; Rollins *et al.*, 2000; Saftner, 2011; Thomann and Hryciw, 1992). Four projects were conducted in Pleistocene-aged or dense soils (Charlie *et al.*, 1992; Hryciw and Dowding, 1988; Liao and Mayne, 2004; Thomann and Hryciw, 1992). In these cases, explosive compaction reduced q_c due to disruption of a strong soil skeleton (Saftner *et al.*, 2015). While sand-ageing effects were observed following explosive compaction in Pleistocene-aged or dense deposits, q_c did not recover to pre-blast values.

Schmertmann *et al.* (1986) and Dumas and Beaton (1988) showed time-dependent changes in q_c following DDC. In both projects, ageing effects were greater closer to the improvement point, demonstrating that ageing effects increase with greater initial disturbance. DDC projects documenting ageing effects are summarised in Table 3.

As shown in Table 4, four projects documented sand ageing following placement of granular fill. Denisov *et al.* (1963) used SPT to characterise the profiles rather than CPT, which was used by the other projects summarised herein. For consistency of presentation and to allow comparisons among the study results, the SPT N values reported by Denisov *et al.* (1963) were converted to equivalent CPT q_c values using the correlation proposed by Kulhawy and Mayne (1990).

Additionally, several projects studied the ageing phenomenon following earthquake or vibroseis shaking. Holzer and Youd (2007) described a site investigation conducted decades earlier prior to installing an instrumentation array in the Imperial Valley, California. Following the 1987 Superstition Hills earthquake, three CPTs were conducted, each within 1 m of

Table 4. Summary of data from the fill placement projects summarised in this paper

	Location	Initial tip resistance, q_c: MPa	Ageing time: d	Aged tip resistance, $q_c(t)$: MPa
Denisov *et al.* (1963)	Volga River, Russia	10·5	30	11
Mitchell and Solymar (1984)	Jebba, Nigeria	2·7	60	4·2
Jefferies *et al.* (1988)	Beaufort Sea	5	154	5
Covil *et al.* (1997)	Chek Lap Kok airport, Hong Kong	8	75	9

pre-earthquake CPT locations. There was no obvious change in q_c following the earthquake.

In an experiment at Griffin, Indiana, Saftner (2011) utilised the Network for Earthquake Engineering Simulation (NEES) vibroseis to simulate earthquake shaking. Changes in the index properties of the sand deposit were not observed following vibroseis shaking, possibly due to ~2 m surficial clay layer damping the motions before reaching the underlying sand layer.

Additional evidence of increased liquefaction resistance in aged deposits came from the 2011 Great East Japan earthquake (i.e. Tohoku earthquake). Ishihara and Sasaki (2012) summarised the evidence of liquefaction near Tokyo Bay. Recently reclaimed land, defined as reclaimed since 1926, showed more occurrences of liquefaction than natural deposits or old reclaimed land. Similarly, Towhata *et al.* (2012) reported an investigation of Urayasu City, where evidence of liquefaction was more common in fill placed after 1960 than in natural deposits.

Maurer *et al.* (2014) investigated short timescale ageing effects (i.e. geotechnically very young deposits) by analysing sites that experienced multiple episodes of liquefaction during the 2010–2011 Canterbury, New Zealand, earthquake sequence. Some of the sites that were moderately to severely liquefied during the 2010 Darfield earthquake were more prone to liquefy during the subsequent 2011 Christchurch earthquake that impacted the same region. Therefore, the geotechnical age of sites experiencing liquefaction was reset. The loss of the ageing benefits made the sites more likely to liquefy in future earthquake events.

3. Displacement rammed aggregate pier study

The Geopier Foundation Company developed the displacement rammed aggregate pier, or Impact Pier, system in 2003. The mandrel, shown in Figure 1, is driven or vibrated to the depth of interest, then filled with the stone with which the pier will be built. When the mandrel is raised a predetermined distance corresponding to lift height, the stone fills the void. The mandrel is then pushed or vibrated down, compacting the

Figure 1. Proprietary 'Impact Pier' mandrel used in installation of displacement rammed aggregate piers

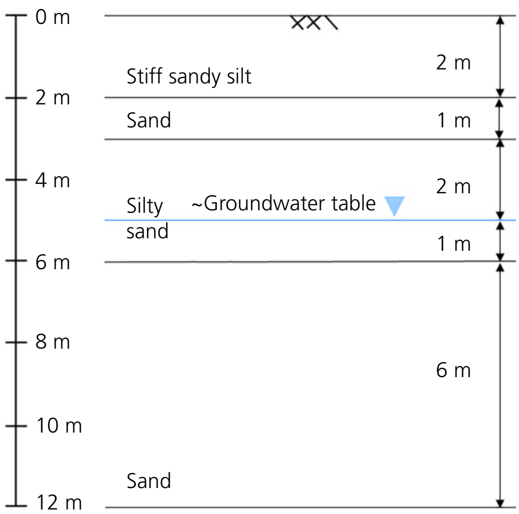

Figure 2. Typical soil profile at the New Madrid, Missouri testing site

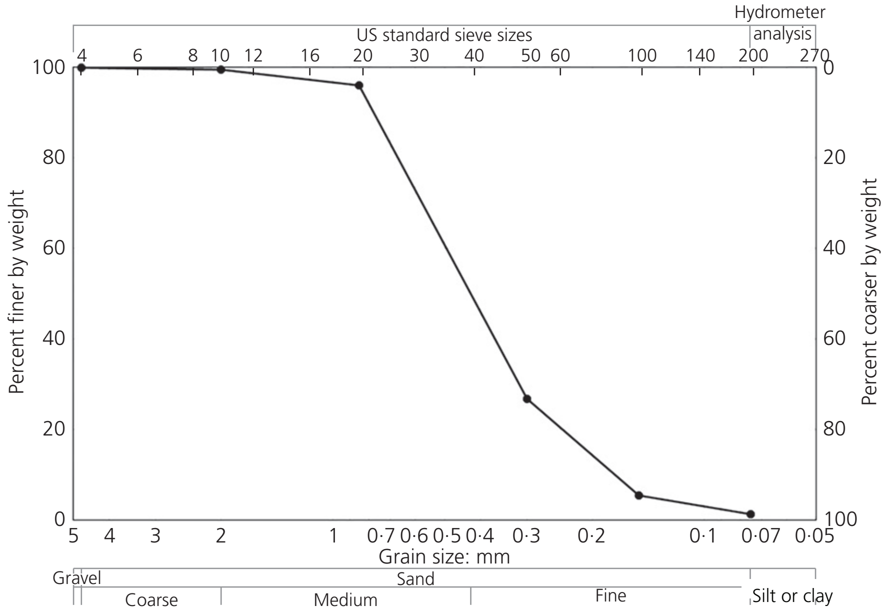

Figure 3. Grain size distribution curve for the sand layer (6–12 m) at New Madrid, Missouri

stone. This process is repeated until installation of an ~0·5 m (~1·6 ft) diameter pier is completed.

A construction site in New Madrid, Missouri, is the focus of the study presented herein. The site contained clean, saturated sand deposits with historical evidence of liquefaction from the 1811 to 1812 New Madrid earthquakes. Liquefaction evaluations for the design earthquake motions showed that the risk was high, and piers were selected to remediate the upper 12 m of the site. A portion of the site was used as a test area to validate the effectiveness of the piers in reducing liquefaction hazard and to study the time-dependent changes in the in situ index properties of the loose sand. The piers in the test area were installed on 15 July 2010.

There are four principal soil layers at the site. As shown in Figure 2, they include a stiff sandy silt layer (~2 m thick), a clean sand layer (~1 m thick), a silty sand layer (~3 m thick) and another clean sand layer (>6 m thick). From a liquefaction hazard perspective, the clean sand layer from a depth of roughly 6–12 m was of most concern. The grain size distribution of this layer is shown in Figure 3. The mean particle size D_{50} is 0·45 mm and coefficient of uniformity C_u is 2·7. The sand is classified as SP by the Unified Soil Classification System (Saftner, 2011).

Blum *et al.* (2000) described New Madrid's geologic history. The soil layers are Pleistocene sand and silt fluvial deposits, placed by the Mississippi River during the Wisconsin ice age. However, the region has been impacted by clusters of large earthquakes that have occurred on average every

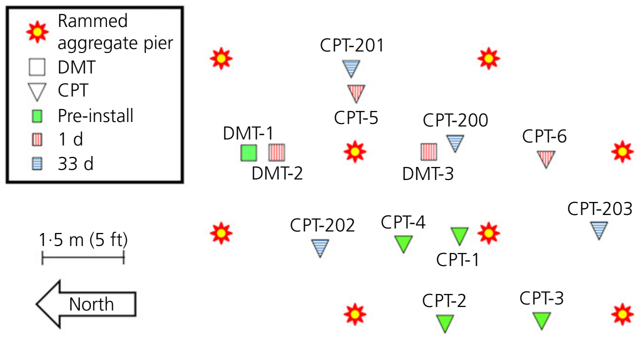

Figure 4. Testing locations at New Madrid, Missouri

500 years over the past 1200 years (Tuttle *et al.*, 2002). The last such cluster occurred in 1811–1812 and caused widespread liquefaction and related phenomena, effectively resetting the soil's engineering age (Andrus *et al.*, 2009). Therefore, the site's geotechnical age was ~200 years at the time of pier installation.

As shown in Figure 4, the piers were installed in a triangular pattern with ~3 m (10 ft) centre-to-centre spacing and extended to a depth of 12 m. Construction workers reported that it was much more difficult to install a new pier in the middle of a pre-existing group than to install a new pier outside the group's perimeter. This demonstrates that horizontal stresses and soil density increase due to pier installation.

In situ testing was performed near the centre of the triangles formed by the piers before the installation of the piers, within

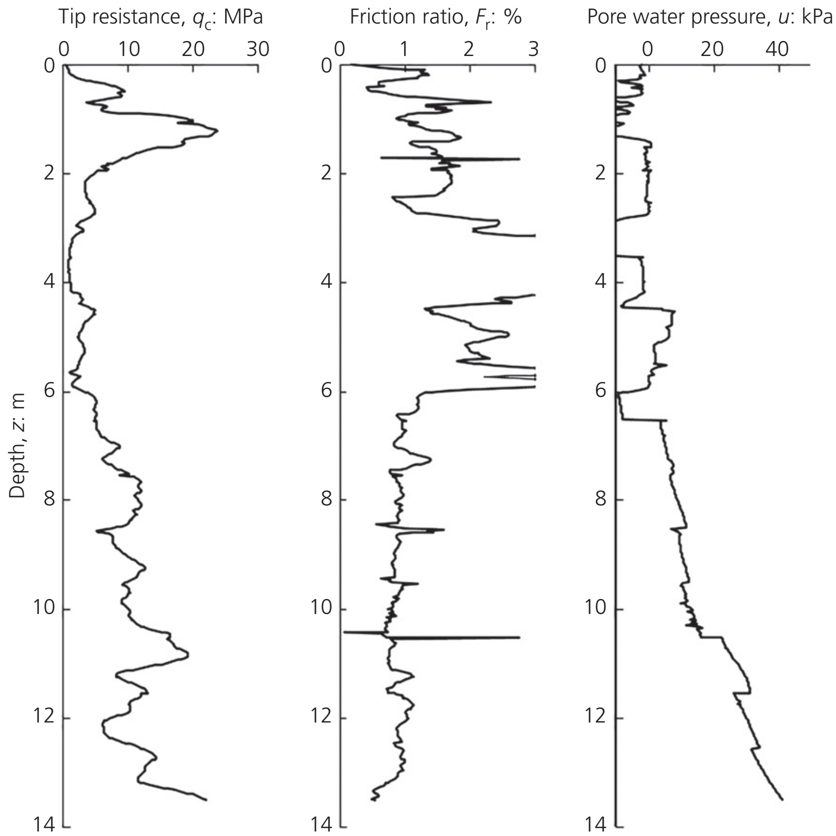

Figure 5. Results from CPT-4, typical of pre-installation testing. F_r greater than 3% is not shown for clarity

24 h after installation, and 1 month after installation. The construction schedule was such that DMT could not be performed 1 month after installation. Laboratory testing, including direct shear tests, odometer tests and grain size and shape analysis, was conducted on samples collected from auger flights during pier installation.

4. Results and discussion

The following section describes the results of in situ and laboratory testing. Testing conducted prior to installation of piers is described first, followed by testing conducted within 24 h of installation and, finally, testing conducted 1 month following installation. In situ test results are compared to laboratory results.

4.1 Initial site characterisation

Four CPT soundings and one DMT sounding were performed prior to the installation of the piers. Figure 5 shows representative pre-installation CPT data. Throughout the site, pore water pressure measurements generally followed the hydrostatic line below the groundwater table. Additionally, friction ratio (F_r) was similar in all tests. Consequently, focus is henceforth given only to q_c data. Figures 6 and 7 display all pre-installation CPT and DMT results, respectively.

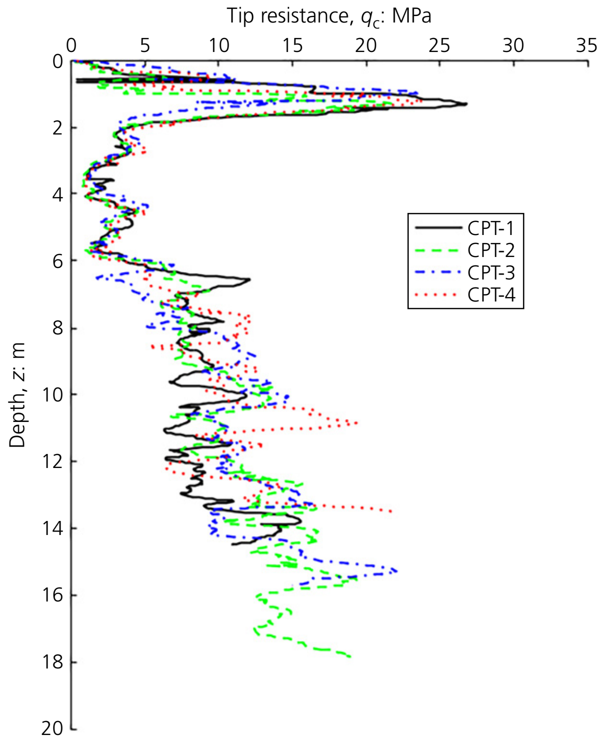

Figure 6. Results of pre-installation CPTs

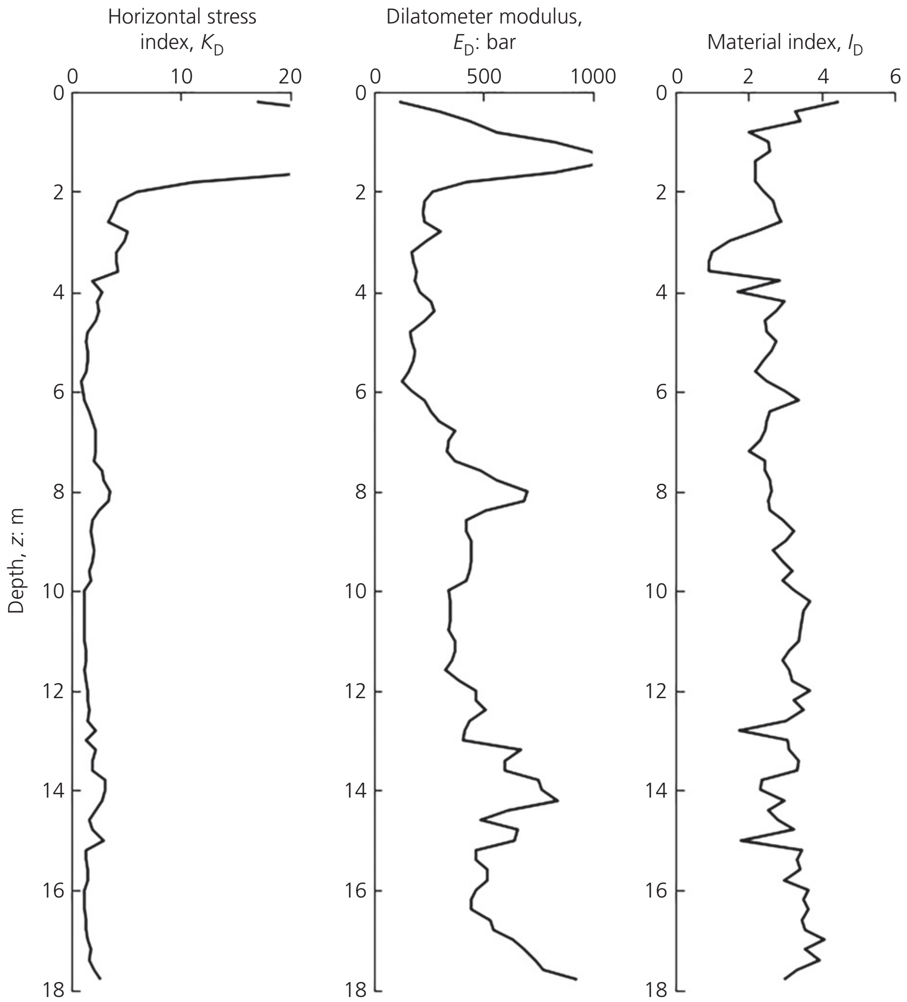

Figure 7. Results of DMT-1, conducted prior to pier installation. K_D and E_D greater than 20 and 1000 bars, respectively, are not shown for clarity

Table 5. Comparison of in situ and laboratory test data

	Prior to soil improvement	One day following soil improvement
Estimated field D_r: %	45	70
ϕ' from CPT: degrees	37	40
ϕ' from DMT: degrees	31	37
ϕ' from direct shear tests: degrees	34	35
M from DMT: MPa	40	100
M from odometer tests: MPa	20	110
K_0 from DMT	0·5	0·8

Comparing in situ and laboratory results prior to improvement demonstrates the similarity of soil properties. Table 5 shows the results of CPT and DMT correlations to drained friction angle (ϕ') (Kulhawy and Mayne, 1990; Marchetti *et al.*, 2001, respectively) and a DMT correlation to constrained modulus (M) (Marchetti, 1980). Additionally, Baldi *et al.*'s (1986) correlation between q_c and relative density (D_r) indicated a representative pre-improvement D_r of 45% in the loose sand layer between 6 and 12 m. Accordingly, direct shear testing was conducted with a D_r of 43%, similar to field conditions. Marchetti *et al.* (2001) emphasised that the $K_D - \phi'$ correlation is conservative and underestimates ϕ' by 2–4°. Therefore, there is good agreement between laboratory and DMT results, with the CPT-based ϕ' being slightly higher but similar. The M determined from DMT is 40 MPa. The laboratory odometer test was performed at the same relative density and predicted M to be 20 kPa, demonstrating the difficulty of predicting compressibility.

Through odometer testing, the compression index (C_c) and recompression index (C_r) were determined, and are listed in Table 6. The computational geometry algorithm developed by Zheng and Hryciw (2015) was used to determine particle roundness and particle sphericity of the collected sand. The roundness is 0·57 while the sphericity is 0·73. The roundness, sphericity and coefficient of uniformity were used to predict e_{max} and e_{min} following the empirical equation developed by

Table 6. Comparison of predicted to measured soil compressibility

	Compression index, C_c	Recompression index, C_r	Minimum void ratio, e_{min}	Maximum void ratio, e_{max}
Predicted	0·0232	0·0105	0·52	0·81
Measured	0·0250	0·0100	0·52	0·84

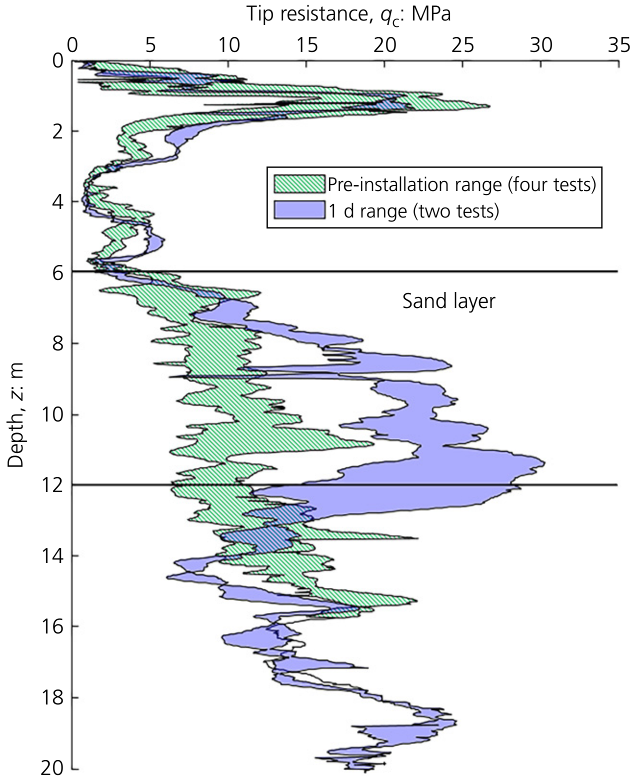

Figure 8. Range of pre-installation CPTs (CPT-1, -2, -3 and -4) compared to the range of CPTs conducted 1 d after installation (CPT-5 and -6)

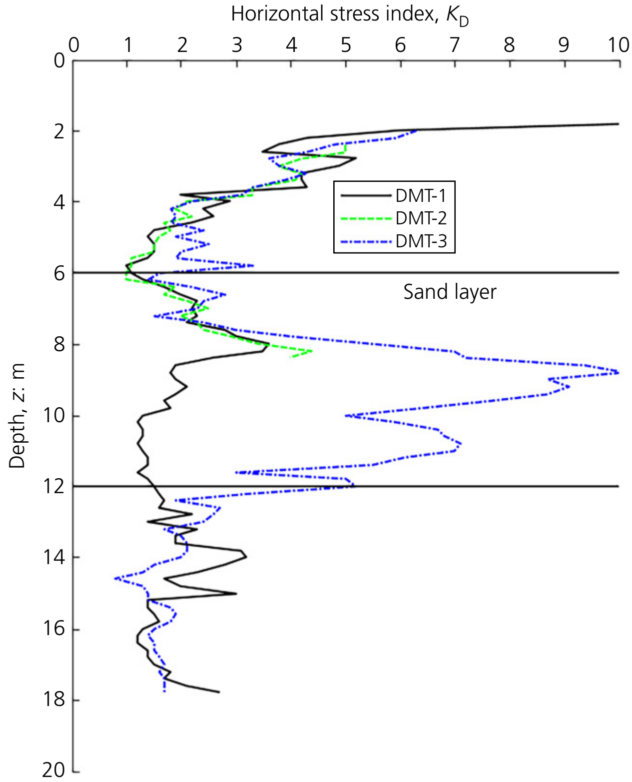

Figure 9. DMT horizontal stress index from DMT-1 conducted before installation and tests conducted 1 d after installation (DMT-2 and -3)

Zheng and Hryciw (2016). The roundness and relative density ($D_r = 45\%$) were used to predict C_c and recompression index C_r following the empirical equation from Zheng *et al.* (2017). The predictions and measurements agree with each other very well as shown in Table 6.

4.2 In situ improvement

The ranges of post-installation q_c, K_D and E_D compared with pre-installation values are shown in Figures 8, 9 and 10, respectively. In situ results above 6 m were generally unchanged from the pre-installation results. Following pier installation, q_c and K_D increased roughly 100%, while E_D increased ~200% in the sand layer from 6–12 m. Below the depth of installation (~12 m), there was no discernible change in the in situ test results. Because I_D is related to soil type, it did not change significantly due to installation of the piers. The authors pushed a pilot cone through the stiff sandy silt layer prior to pushing the DMT blade. Insertion of the pilot cone allowed the DMT blade to penetrate through the stiff sandy silt and into the deeper layers but prevented the collection of useful data in the stiff sandy silt. Because the stiff sandy silt layer was not of interest to this study, the pilot cone was used in both post-installation DMTs.

In situ test results point to a significant increase in horizontal stress as a result of pier installation. Figure 11 shows the increase in the coefficient of lateral earth pressure at rest (K_0) following installation as determined using Baldi *et al.*'s (1986) K_D–q_c–K_0 relationship. Because horizontal stress influences both K_D and q_c, it is not surprising that pier installation caused similar changes to both K_D and q_c. The increase in E_D due to installation of the piers was over 200%, demonstrating that impact pier installation increases soil stiffness.

As shown in Table 5, ϕ' predicted from both CPT and DMT increased significantly following pier installation. Because the

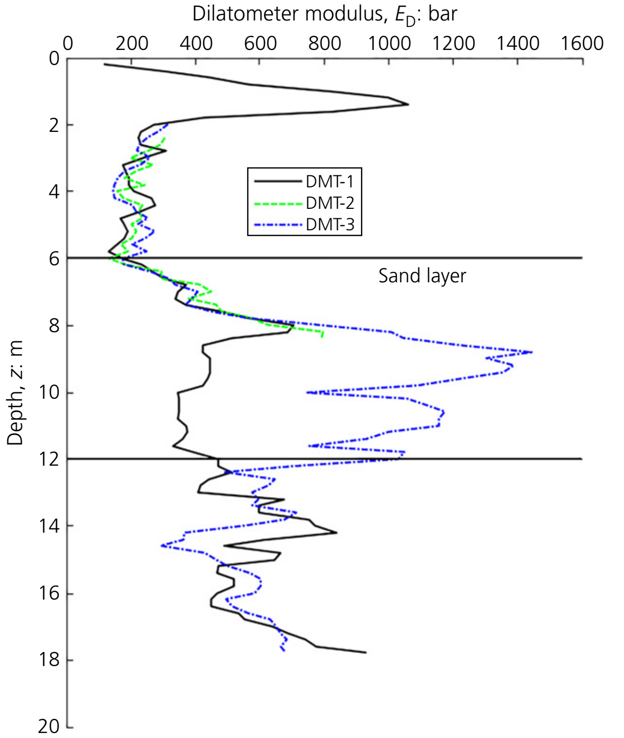

Figure 10. Dilatometer modulus from DMT-1 conducted before installation and tests conducted 1 d after installation (DMT-2 and -3)

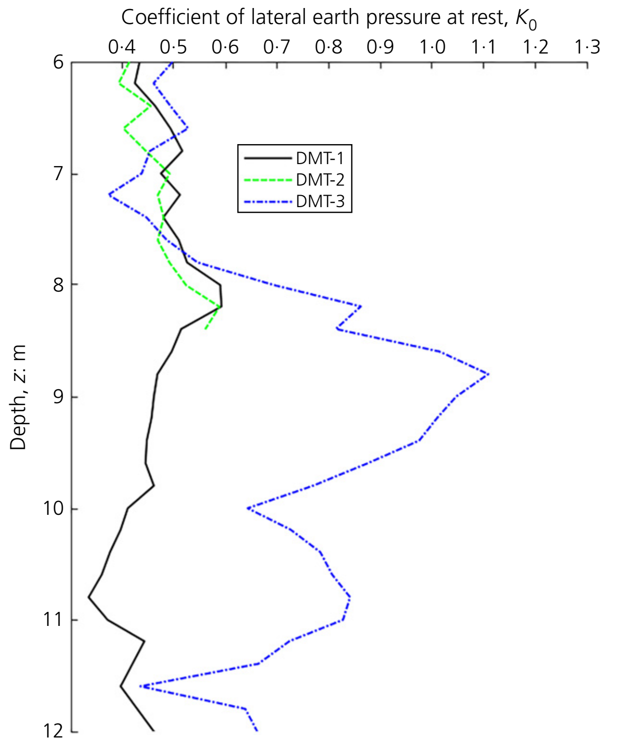

Figure 11. Coefficient of lateral earth pressure at rest as determined from DMT-1 conducted before installation and tests conducted 1 d after installation (DMT-2 and -3)

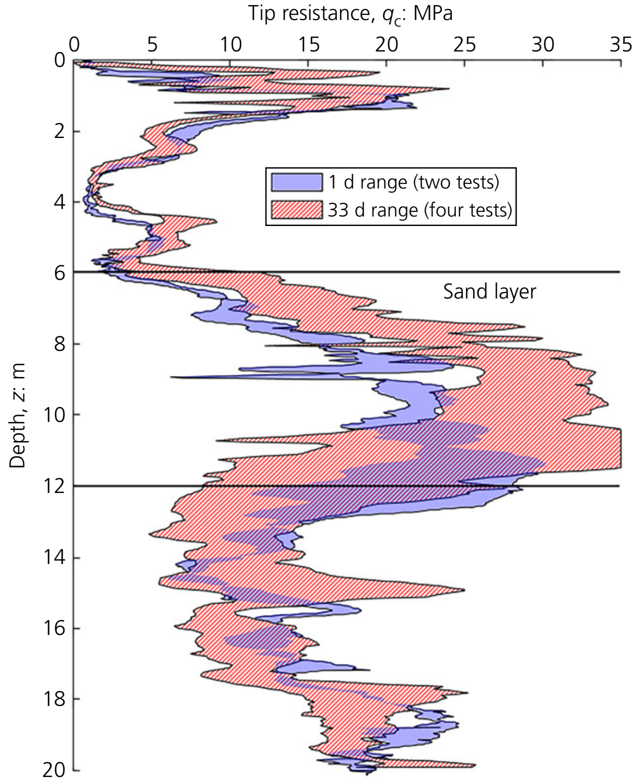

Figure 12. Range of q_c from CPT conducted 1 d after installation (CPT-5 and -6) compared to range of q_c from CPT conducted 1 month after installation (CPT-200, -201, -202 and -203)

direct shear samples are at K_0 conditions and do not account for the increase in horizontal stress following pier installation, a lower laboratory-determined ϕ' was expected. The DMT and laboratory M are similar and substantially greater than those prior to improvement. In summary, pier installation increased horizontal stress, D_r, ϕ' and M as shown through DMT, CPT and laboratory testing.

4.3 Ageing effects

One month after the installation of the piers, a local contractor conducted four CPT soundings (i.e. Figure 4: CPT-200, -201, -202 and -203). As shown in Figure 12, q_c values measured 1 month after installation increased ~33% in the silty sand layer from 3–6 m and in the deeper sand layer to the depth of pier installation (12 m). Tip resistance values remained generally consistent with 1 d testing in the shallow layers, as well as below pier installation.

Figure 13 shows the mean q_c profile for each set of tests. The stiff sandy silt layer (0–2 m) q_c showed little change with time following impact pier installation. In the sand layer from 2–3 m, q_c increased due to soil improvement but remains constant with time following pier installation. In the silty sand layer from 3–6 m, q_c was not initially affected, but increased with time following installation. Finally, q_c in the sand layer from 6–12 m increased initially and with time following installation up to the

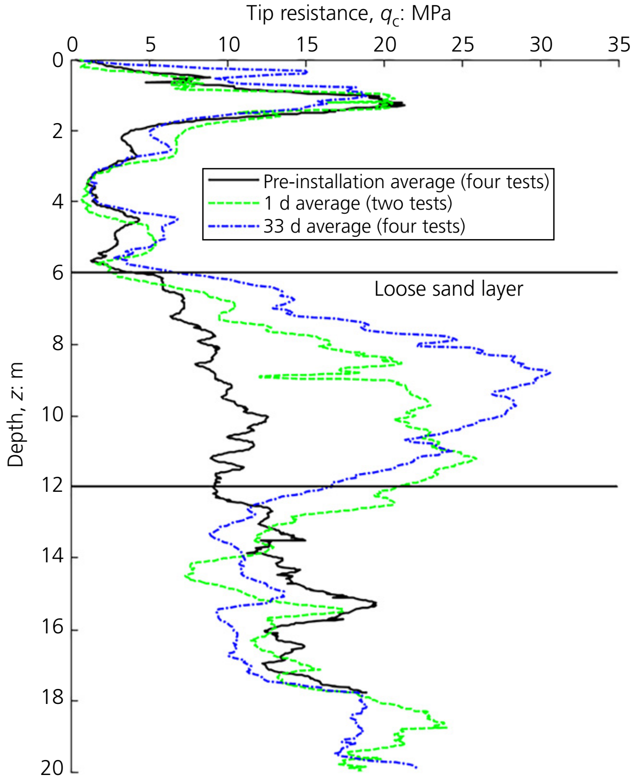

Figure 13. Mean CPT q_c from testing before, within 24 h and 1 month following pier installation

depth of installation. Below the depth of impact pier installation (12 m) in the deeper sand layer, there was little change in q_c with time.

This project is the first documented case of sand ageing following installation of piers. Despite a lack of surface settlement, q_c increased roughly 33%. Laboratory tests conducted on freshly prepared samples do not replicate the field conditions. As shown in Figure 14, these findings are consistent with previous studies documenting sand-ageing effects (e.g. Ishihara and Sasaki, 2012; Maurer *et al.*, 2014; Saftner, 2011; Towhata *et al.*, 2012). Additionally, Saftner *et al.* (2014) demonstrated that disturbance type plays a significant role in sand-ageing effects, breaking disturbances into two general categories: high-disturbance methods (i.e. explosive compaction, fill placement, DDC at shallow depths, vibro-compaction and impact rammed aggregate piers) and low-disturbance methods (i.e. DDC at greater depths and vibroseis shaking). Figure 15 shows high-disturbance methods, including data from this project, displaying great ageing effects.

In summary, Impact Pier installation initially increased the q_c of the shallower and deeper clean sand layers. Sand ageing occurred in the silty sand and deeper sand layers. One month after installation of the piers, the q_c in the deeper sand layer increased roughly 33% from values measured immediately following installation.

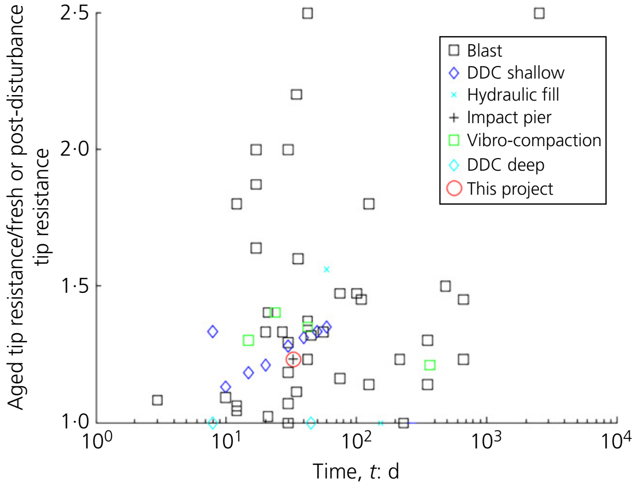

Figure 14. Ageing effects compared to disturbance method highlighting the data from this project, the first documented case of sand ageing following impact rammed aggregate pier installation

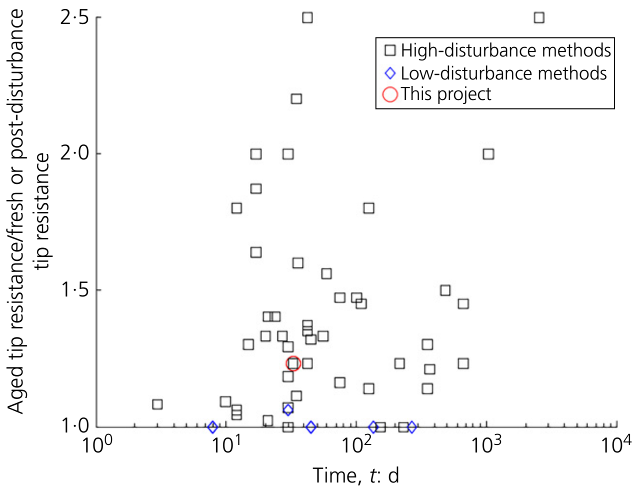

Figure 15. Ageing effects compared to high-disturbance and low-disturbance methods highlighting the data from this project.

5. Conclusion

This paper presented in situ testing data collected before, immediately after and 1 month after installation of displacement rammed aggregate piers in New Madrid, Missouri. Following pier installation, q_c and K_D increased roughly 100% and E_D increased roughly 200%. The increases in ϕ' and M, as determined using CPT and DMT results before and following pier installation, closely match the laboratory results. Cone penetration test results showed a time-dependent strength gain following the disturbance of the loose clean sand layer, increasing roughly 33% in the month after pier installation. DMTs were conducted only before and immediately after pier installation, so sand-ageing effects were not measured with the DMT.

This case history provides the first example of time-dependent increases in the in situ index properties of sand following installation of piers. According to published case histories and this project, research has shown that sand-ageing effects are tied to the method and magnitude of disturbance. This study expands the current body of knowledge in sand ageing and demonstrates the effectiveness of piers as a soil-improvement method.

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