A Critical Assessment of Rammed Aggregate Pier Load Test Results

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Abstract

Within the confines of this paper, field performance of rammed aggregate pier (RAP) is compared with the performance predictions estimated by available empirical models. Site investigations revealed that the underlying soil profile is composed normally consolidated, low-high plasticity, soft to stiff clay (CL-CH) extending to a depth ~ 15m from existing ground level. The results of impact pier (IP) load tests were summarized in the form of mobilized capacity/subgrade modulus and stiffness versus representative SPT N value curves. Also a normalization scheme was introduced, which provided a dimensionless and more precise prediction of the field response. Consistent with theoretical background, normalization of the mobilized resistance by the length of the pier produces better predictions with the observed response. Hence, impact piers are concluded to resist to applied load mostly through skin friction/adhesion mechanism. At relatively smaller settlements/strains, mobilized resistance is less sensitive to the shear strength characteristics of native soil. With increasing strain levels, an increase in representative SPT N values produces a higher capacity for impact piers. For the soil sites studied, modulus of subgrade varies in the range of 30 to 80 MN/m^3 corresponding to a settlement range of 0.6 cm to 5.0 cm (1/4 to 2 inches). Even though some of the conclusions are judged to be trivial, with increasing number of field performance test data, as introduced in this manuscript, a performance based design framework could be developed, which is the intent of the authors in their future studies.

Keywords: *Rammed aggregate pier, load tests, stiffness, settlement*

1 Introduction

Due to increasing demand in urban residential development along with civil structures in the form of industrial plants, transportation and hydraulic structures, ground improvement applications have been significantly used for the rehabilitation of poorly performing soil sites over the past two decades. Among existing alternatives, the rammed aggregate pier solution has been listed and served as an alternative to deep foundations and over excavation replacement of compressible soils. Within the confines of this manuscript, rammed aggregate or impact pier terminologies will be used interchangeably to refer to 50 cm diameter piers constructed by displacement method. It involves densely compacting successive thin lifts of high quality crushed stone in a 2 to 3 foot cavity of varying length by using patented ramming equipment. This paper presents the field response of rammed aggregate piers compiled as part of field load tests. For this purpose full-scale "quick" field load tests were performed on 13 rammed aggregate piers constructed by Impact System. Before the discussion of pier load tests and their results, installation methodology as well as a brief presentation of soil site profiles will be introduced.

2 Construction of Columns

In the field 13 rammed aggregate piers were installed closely following the Impact System construction procedures. Impact Rammed Aggregate Piers (RAP) are constructed following these steps:

(1) a closed ended mandrel with a diameter of 36 cm is hydraulically pushed into the design depth (Figure 1a).
(2) the mandrel and hopper are filled with aggregate (Figure 1b).
(3) the ramming action is applied with 3 feet up / 2 feet down compaction efforts, during which vertical vibration is also introduced (Figure 1c). The ramming actions expand the diameter from 36 cm to 50 cm if 3 feet up and 2 feet down compaction procedure is chosen.

Figure 1. The construction of Impact Rammed Aggregate Piers.

3 Site Investigation

As part of the site investigation program series of conventional boreholes were drilled extending to 25m – 40m depths. At various depths, standard penetration tests were performed along with disturbed and undisturbed soil sampling. Figure 2 presents generic soil profiles documented after the site investigation program at Afyon site. On the acquired disturbed and undisturbed samples classification, and/or shear strength tests were performed. The sites studied mostly consist of normally consolidated, low to high plasticity, soft to stiff clay (CL-CH) extending to depths of 15m from existing ground level. Below this layer, medium dense to dense gravelly, clayey, silty sand / hard greywacke / very stiff to hard clay are located. Groundwater table is reported to be at approximately 1.2m – 5.0m depth range.

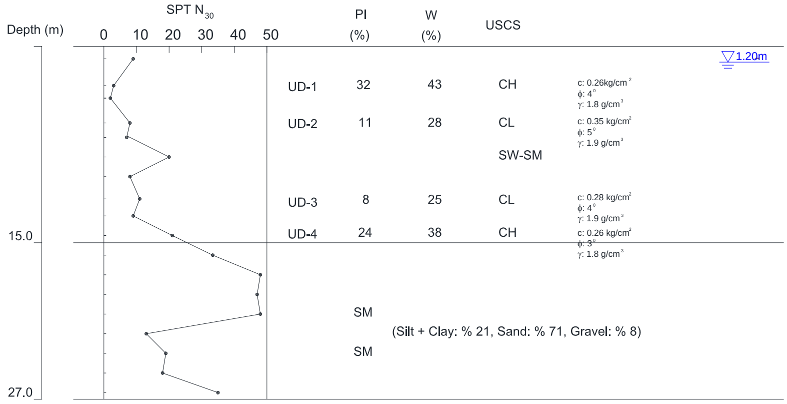

Figure 2. Generic soil profile from Afyon site.

4 Field Load Tests

Field load tests were performed by closely following the loading scheme summarized in Table 1. Stage loading starting with 5 % has been continued until the pier is tested under 150 % of its service load. Then an unloading procedure was followed.

Table 1. Test procedure.

No	Time (min.) (min. / max.)	Load (%)	No	Time (min.) (min. / max.)	Load (%)
0	15 / 60	5	8	15 / 60	133
1	15 / 60	16	9	15 / 60	150
2	15 / 60	33	10	N/A	100
3	15 / 60	50	11	N/A	66
4	15 / 60	66	12	N/A	33
5	15 / 60	83	13	N/A	0
6	15 / 60	100	14	N/A	100
7	60 / 240	116 *	15	N/A	0

* The load increment that represents approximately 115% of the design maximum stress on the Rammed Aggregate Pier shall be held for a minimum of 60 minutes and until the rate of deflection is less than 0.254mm per hour or less, or for a maximum duration of 4 hours.

This test is widely referred to as "quick" tests due to relatively rapid application of the loading scheme and the test procedure is very similar to pile load tests defined by ASTM D 1143. As part of the test load is directly applied on the pier, as opposed to alternative distributed application of the load on both the site soil and pier. Figure 3 shows modulus test sections of the test set up.

Figure 3. Modulus load test.

The modulus load tests of Impact Pier elements often incorporate tell-tales at different elevations within the pier (Brian et al., 2006). The tell-tale consists of a horizontal steel plate that is attached to two sleeved vertical bars extending to the top of the pier. During the load test, displacements at top of the pier and at the tell-tale plate were recorded which enable relative displacement (straining) of the pier element.

The loading tests were performed to obtain the bearing capacity and stiffness of RAP elements. Although the purpose of the modulus load test is to verify the RAP stiffness modulus used for design calculations, the tests may also be used to add insight into how the RAP behaves in the matrix soil. This is done by observing the deflection of tell-tales installed into the bottoms of the rammed aggregate piers. Different modes of deformation can be observed in RAPs subjected to compressive loads depending on the soil conditions and pier length. Typical modes of deformation for RAPs can be listed as bulging, shear failures and to a lesser extent punching (Wissmann et al. 2011). When a RAP is equipped with a tell-tale reference plate, the dominant deformation mode of the pier can be recognized from the shape of tell-tale load settlement curve in comparison with the top of pier settlement. Despite its contributions to quality control and to understand the interactive response of the pier with native soil profile, direct use of capacity and stiffness parameters obtained from load tests may be erroneous if a group of piers are to be planned. Additionally, if design loading scheme (e.g.: embankment loads) is different than the concentrated loading scheme applied during single pier load tests, deviations from monitored single pile response may be observed. However, differences in pier load tests and design loading schemes can be corrected if the behavior of piers under various loading schemes are understood, which will establish the basis of this manuscript.

5 Experimental Results

As previously discussed, 13 loading tests were performed on rammed aggregate piers (RAP) installed in soft to stiff clay soils to assess the bearing capacity and stiffness response of individual piers. The rammed aggregate piers were constructed at four different soil sites, and all piers were constructed to a final diameter of 50 cm with varying lengths of 8.0 m to 16.0 m. As will be discussed later in the manuscript, for the purpose of eliminating the effects of pier length on capacity and stiffness values, a normalization scheme will be introduced. Two distinct representative SPT N estimation procedure (weighted and simple arithmetic) was introduced to model soil strength and stiffness in cohesive soils. N_{mean} value is obtained by simply taking the average of SPT N values obtained at depths corresponding to pier length. Additionally, a linear weighting scheme (a weight of 1 at the ground surface linearly decreasing to 0 at the tip of the pier) is also used to overweight the shallower soils shear contribution as compared to deeper ones, since mobilized pier capacity is mostly due to skin friction between the piers and the soil. This is also supported by the tell-tales installed at various depths. The SPT N values estimated by the weighting scheme is referred to as N_{rep}. SPT N values, length of RAPs, the project sites, and loads under 2.5 and 5.0 cm mobilized surface settlements are summarized in Table 2. All load-settlement curves are shown in Figure 4 and Figure 5.

Table 2. SPT N values, length of RAPs and the project sites.

Test No	Project Site	$N_{rep.}$	$N_{mean.}$	RAP Length (m)
1	Afyon	6	5	16
2	Afyon	5	5	8
3	Yozgat	5	6	10
4	Afyon	3	7	8
5	Afyon	4	7	11
6	Afyon	3	7	16
7	Yozgat	8	8	15
8	Yozgat	11	10	10
9	Istanbul	12	12	10
10	Yozgat	13	14	9
11	Yozgat	13	14	9
12	Yozgat	14	14	9
13	Sivas	9	23	10

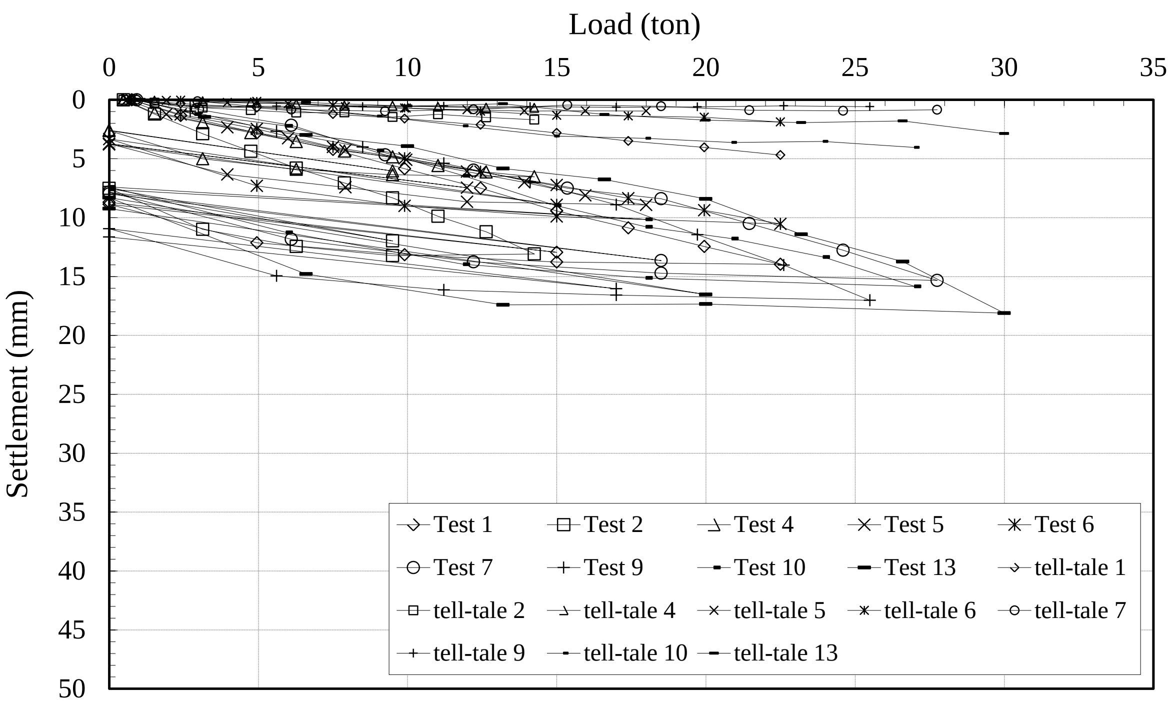

Figure 4. Load-settlement curves - 1.

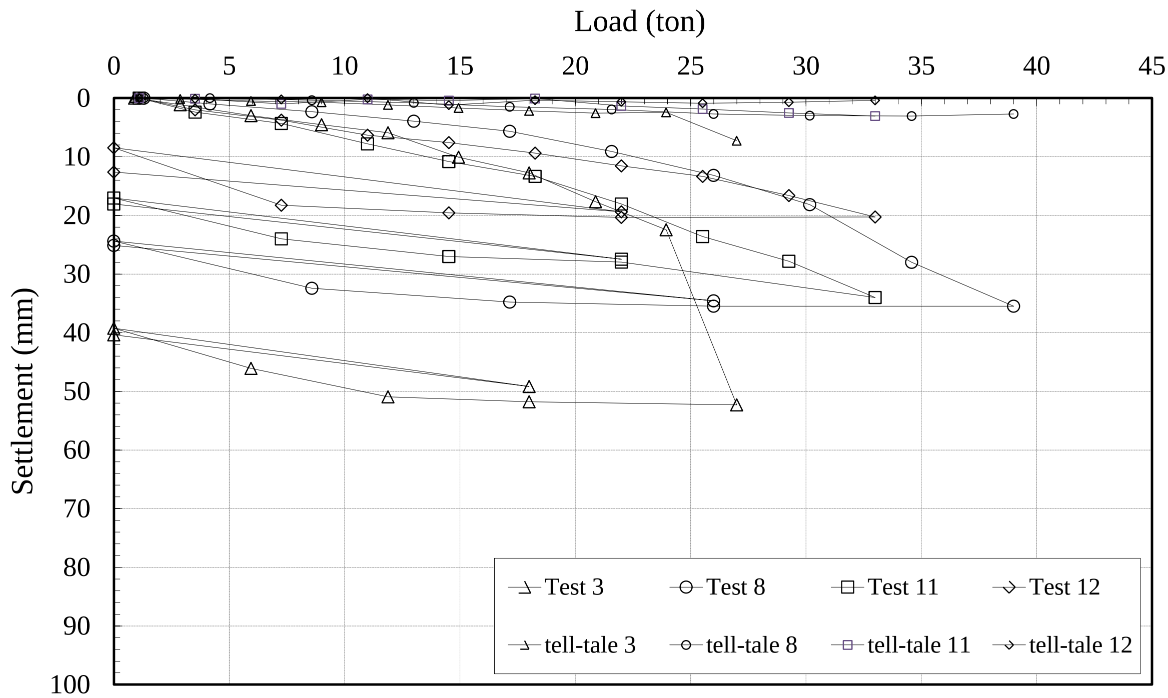

Figure 5. Load-settlement curves - 2.

5.1 Mobilized Capacity

For all tests, cap pier stresses ($q_{IMPACT,}$ kPa) corresponding to the settlements of 0.635 cm (1/4 inches), 1.27 cm (1/2 inches), 2.54 cm (1 inch) and 5.08 cm (2 inches) were assessed from load vs. displacement responses. To convert the stresses on the piers to pier load (Q_{IMPACT}, kN), simply, stress values were multiplied by the plan area of piers (i.e.: area of 50 cm diameter Impact Piers). Note that due to concentric application of the load on the pier element, such simplification is possible. Cap pier load versus SPT $N_{mean.}$ - SPT N_{rep}. values for various mobilized settlements were estimated, which are presented Figures 6 and 7. For comparison purposes, the recommendations by Geopier technical manuals are also shown (Fox and Cowell, 1998) as solid lines.

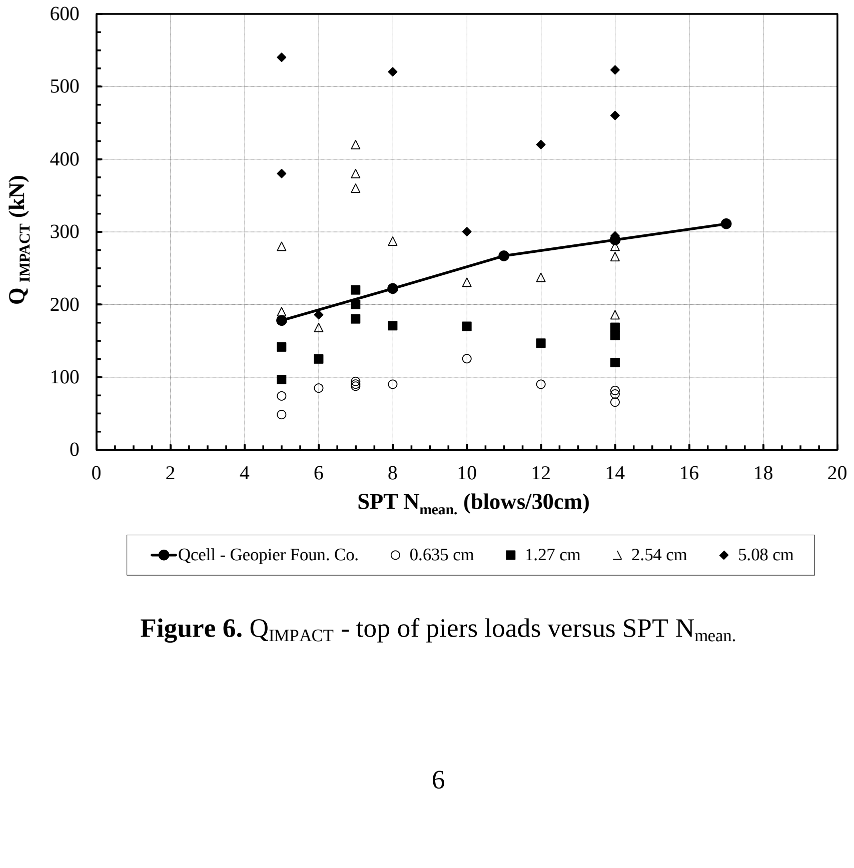

Figure 6. Q_{IMPACT} - top of piers loads versus SPT $N_{mean.}$

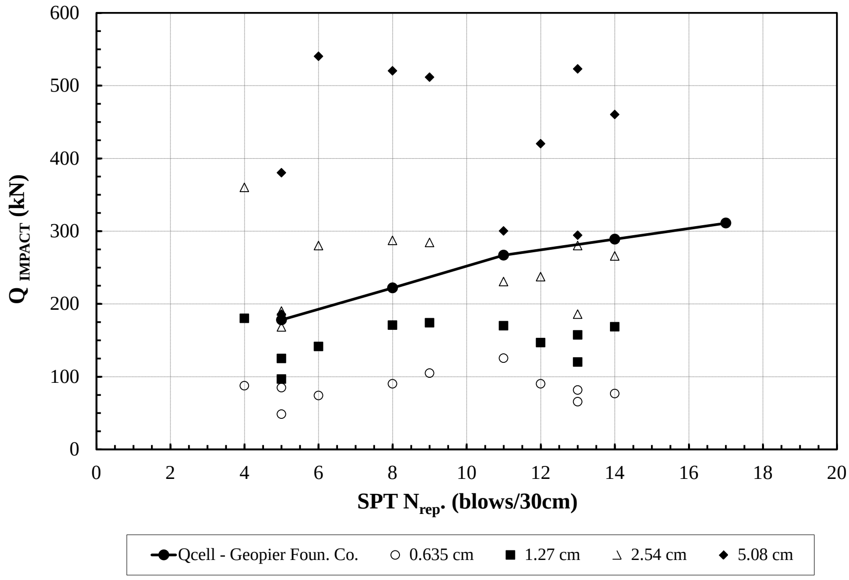

Figure 7. Q_{IMPACT} - top of piers loads versus SPT N_{rep}.

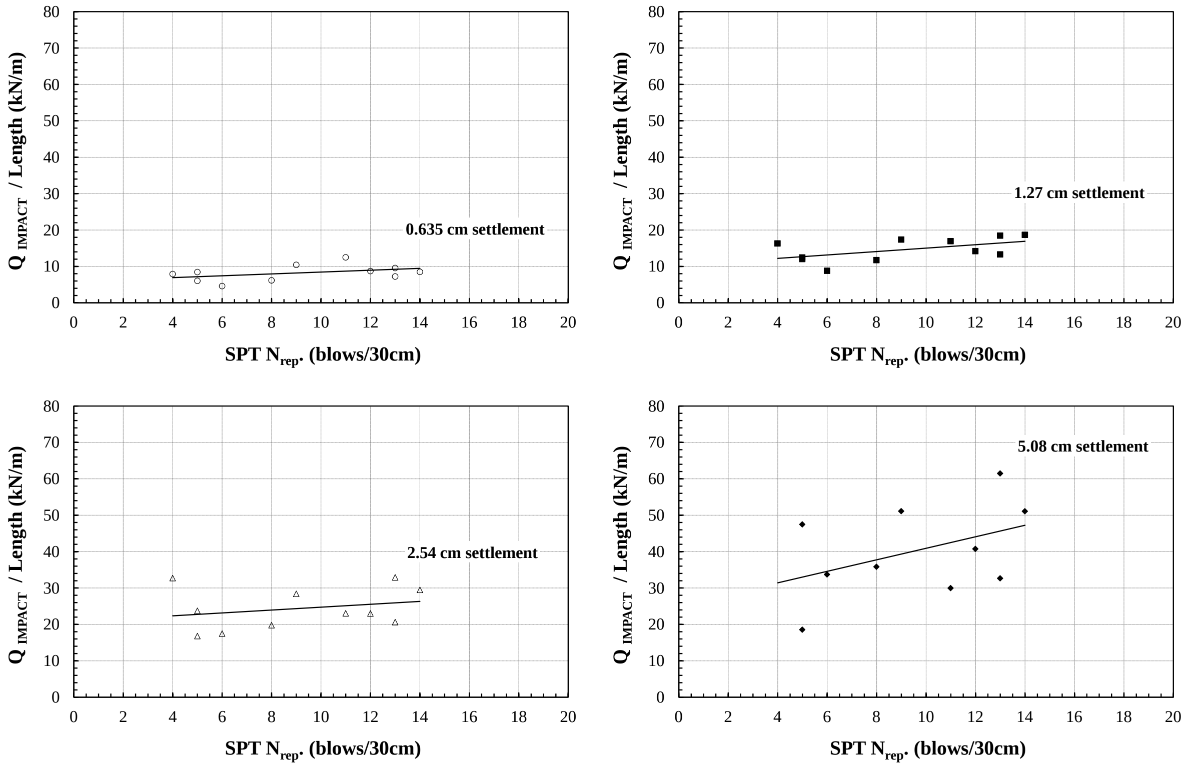

Figure 8. Q_{IMPACT} / Length versus SPT N_{rep}.

It should be noted that the capacities presented by Geopier technical manuals are recommended for 1.27 cm to 2.54 cm (1/2 to 1 inch) mobilized settlement values (Fox and Cowell, 1998) which are shown to be consistent with the observed response in the field. As clearly illustrated by the figures, consistent with theoretical expectation, normalization of the mobilized resistance by length of the pier produce better match with the observed response. Hence, it can be concluded that impact piers are concluded to resist to the applied load mostly through skin friction/adhesion as opposed to tip resistance. Additionally, at relatively smaller settlements/strains, mobilized resistance is less sensitive to the shear strength characteristics of the native soil. However with increased strain levels, an increase in representative SPT N values produce a higher load capacity for impact piers. This argument is supported by increasing slopes of Q vs N lines with increasing mobilized settlements, as clearly shown in Figure 8.

5.2 Mobilized Subgrade Modulus and Stiffness

Mobilized subgrade modulus, expressed as the slope of vertical stress vs. settlement curve is estimated corresponding to settlements of 0.635 cm, 1.27 cm, 2.54 cm and 5.08 cm, for all test results. The mobilized subgrade modulus vs. SPT $N_{mean.}$ or SPT $N_{rep.}$ responses for different settlement thresholds are presented in Figure 9 and 10. The recommendations of Geopier technical manual are also shown in these figures, for comparison purposes (Fox and Cowell, 1998). As clearly illustrated by the figures, deformation (stiffness) responses of IPs are less predictable than their capacity, which is illustrated by significantly wider scatter. Even though there exists a significant level of uncertainty in the predictions, it can be concluded that for the soils studied, modulus of subgrade varies in the range of 30 to 80 MN/m^3 for settlement range of 0.6 cm to 5.0 cm (1/4 to 2 inches). Last but not least, even though SPT N values are judged to be a poor parameter to predict deformation response of piers, compared to SPT $N_{mean.}$, SPT N_{rep} values produce a relatively smaller scatter. Hence, it is judged to be a relatively better parameter to assess the capacity and the stiffness response of piers.

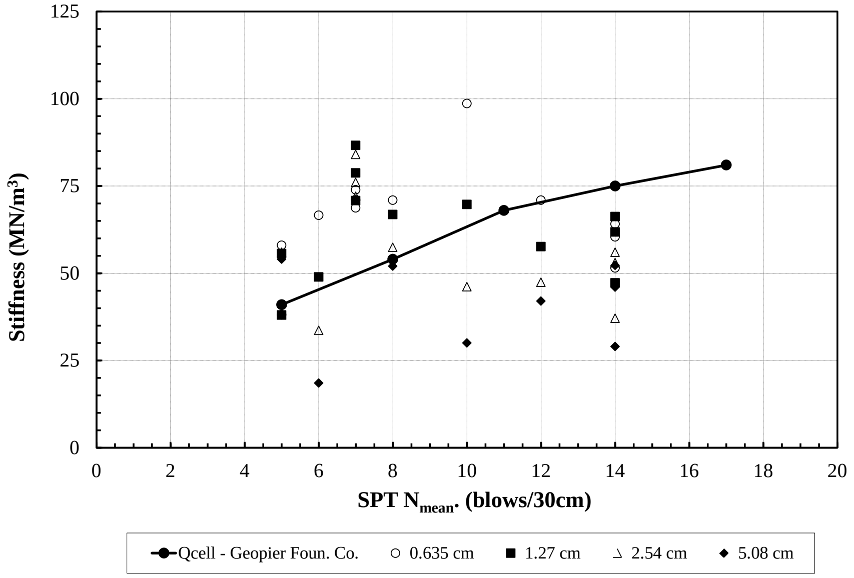

Figure 9. Stiffness versus SPT N_{mean}.

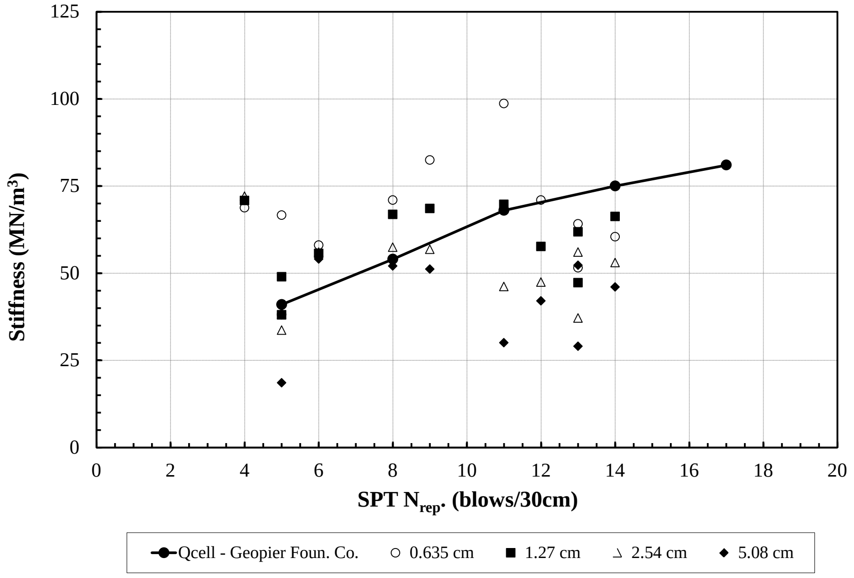

Figure 10. Stiffness versus SPT N_{rep}.

Additionally, by normalizing the stress levels with induced average strains (i.e.: settlement/length of pier), vertical mobilized stiffness, E vs. SPT N_{rep} responses of piers were presented. As illustrated by these figures, E modulus of piers decrease with increasing mobilized settlements and is not very sensitive to variations of SPT N values in the range studied.

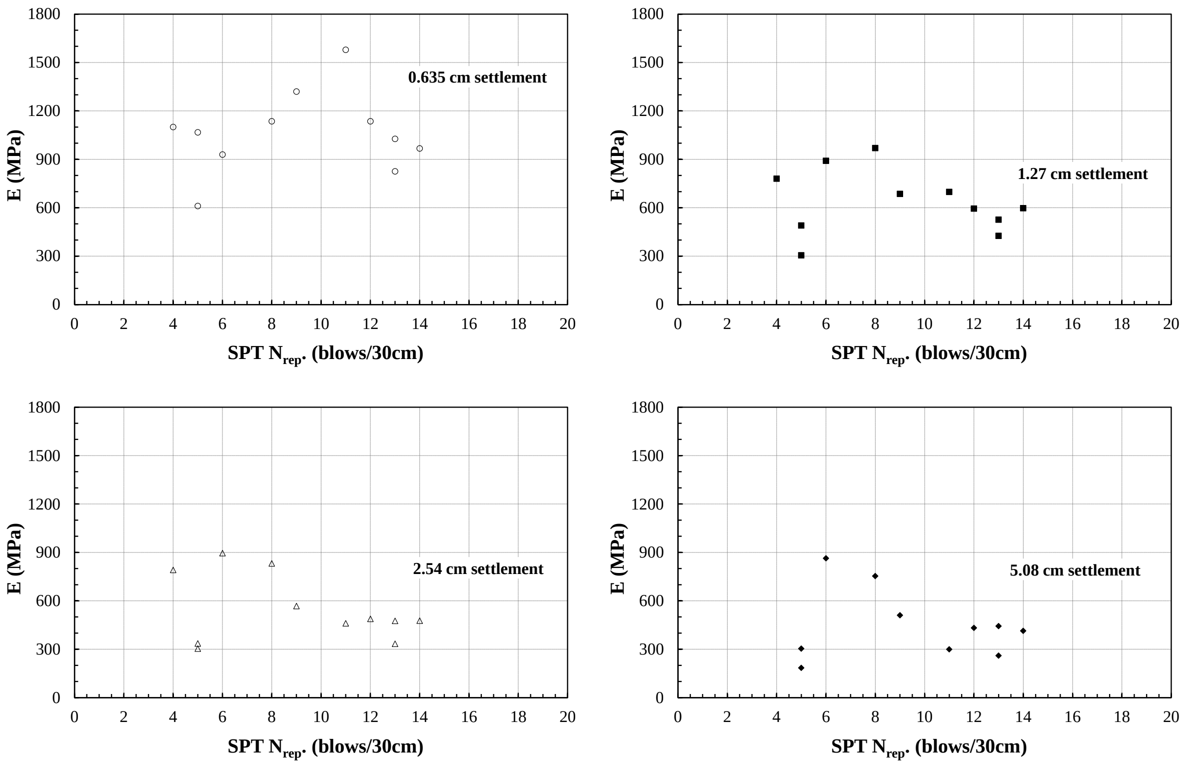

Figure 11. E versus SPT N_{rep}.

6 Summary and Conclusions

Within the confines of this manuscript, 13 full-scale rammed aggregate (impact) pier load tests were critically analyzed from capacity and deformation points of views. These rammed aggregate piers were constructed following these steps:

(1) a closed ended mandrel with a diameter of 36cm is hydraulically pushed into the design depth.
(2) the mandrel and hopper are filled with aggregate.
(3) the ramming is applied with 3 feet up / 2 feet down compaction actions during which vertical vibration is introduced. The ramming actions expand the diameter from 36 cm to 50 cm if 3 feet up and 2 feet down compaction procedure is chosen.

The loading scheme was chosen to be very similar to pile load tests defined by ASTM D 1143. As part of the test, load is directly applied on the pier, as opposed to alternative distributed application of the load on both the site soil and pier. The impact pier elements are loaded to % 150 of the maximum top-of-pier stress. relative deformation response of the pier was monitored through tell-tales installed at different elevations within the pier (Brian et al., 2006). The results of impact pier load tests were summarized in the form of mobilized capacity/subgrade modulus and stiffness versus representative SPT N_{rep} responses. Also a normalization scheme was introduced, which provided a dimensionless and more precise response. Followings are the specific conclusions of the assessments performed:

- Consistent with theoretical expectation, normalization of the mobilized resistance by length of the pier produce better predictions with the observed response.
- Hence, impact piers are concluded to mostly resist to applied load through skin friction/adhesion.
- At relatively smaller settlements/strains, mobilized resistance is less sensitive to the shear strength characteristics of the native soil.
- With increased strain levels, an increase in representative SPT N values produces a higher load capacity for impact piers.
- Deformation (stiffness) responses of IPs are less predictable than their capacity and involves significant level of uncertainty.
- For the soil sites studied, modulus of subgrade varies in the range of 30 to 80 MN/m^3 for settlement range of 0.6 cm to 5.0 cm (1/4 to 2 inches).
- SPT N values are judged to be a relatively poor parameter to predict deformation response of piers. Significant scatter in mobilized capacity and stiffness vs. SPT N values reveals that a single soil parameter (such as N values) is not enough to correlated complex layered soil response. This is due to the fact that there exist multiple modes of pier failures (e.g.: bulging, local shear, punching), one of which will dictate the capacity depending on layering and shear strength characteristic of soil layers. A single parameter (i.e.: SPT N values) can only capture this complex interaction with significant scatter.
- E modulus of piers decreases with increasing mobilized settlements, and contrary to expectation, is judged to not very sensitive to variations in SPT N values of the soil site (in the range studied).

Even though some of the conclusions are judged to be trivial, with increasing number of field performance test data as introduced in this manuscript, a performance based design framework could be developed, which is the intent of the authors in their future studies.

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