Performance-based Assessment of Rammed Aggregate Piers

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Abstract

Within the confines of this paper, a normalized displacement-based capacity mobilization scheme is presented for Rammed Aggregate Piers (RAP). For this purpose, field load tests, performed on 76 RAPs, in Turkey, were assessed. Site investigations at these sites revealed that generalized soil profiles are mostly composed of normally consolidated clay layers extending to a depth of 18 m. Below this depth, usually medium dense to dense sand / hard greywacke / very stiff to hard clay layers are present. A weighted mean SPT N_{60} assessment procedure was utilized to estimate representative soil strength and stiffness parameter in cohesive soils. The results of RAP load tests were summarized in the form of normalized mobilized capacity versus settlement curves as a function of representative SPT N_{60} values. The normalized field load test database revealed that: i) the shaft resistance is observed to be fully mobilized at normalized displacements of 40 % of RAP diameter for very soft clays and at 10 % of RAP diameter for firm clays, ii) up to normalized displacements of 2-5% of RAP diameter, 30-50% of the shaft resistance capacity is mobilized in a rather linear elastic manner, iii) normalized capacity mobilization response of RAPs is more flexible than the ones of bored concrete piles, iv) under compressive loads, RAPs exhibit a strain hardening response, as a result of which, the design-basis capacity is dominated by allowable settlement criterion. The proposed normalized capacity and displacement response curves, presented herein enable displacement (performance)-based assessment and design of RAPs.

Keywords: *rammed aggregate piers, load tests, bearing capacity, normalized capacity, normalized displacement*

1 Introduction

Ground improvement engineering solutions in the form of rigid column intrusions are frequently used to eliminate i) static bearing capacity and excessive settlement problems, ii) seismic soil liquefaction-induced failures and deformations. Stiff Rammed Aggregate Pier (RAP) elements serve as an alternative to existing solutions (e.g.: bored piles, sand drains, vacuum consolidations, stone columns or over excavation and replacement of compressible soils). Within the confines of this manuscript, the deformation performance of 50 cm diameter Impact Pier® elements, constructed by bottom-fed dry method (Impact® System), is evaluated. The diameter of the RAP elements are monitored in real time through sensor monitored quality control system with volume measurements. Hence, column diameter to be 50 cm is confirmed within ±2 cm accuracy. The resulting response is summarized as normalized load-settlement curves obtained from full scale load tests. For this purpose, 76 RAP field load tests were used, which were constructed at sixteen different soil sites in Turkey. The field load test results and the proposed capacity mobilization curves will be presented after a brief review of the existing literature.

2 An Overview of Literature

Barksdale and Bachus (1983) defined three distinct failure modes for stone columns subjected to vertical loading: bulging, shearing, and punching failures. A schematic illustration of these modes is presented in Figure 1. For the assessment of the failure load triggering uniquely bulging-induced failures Datye and Nagaraju (1975), Hughes and Withers (1974) and Madhav and Vitkar (1978) presented analytical and / or numerical solutions, as presented in Equations 1-4, respectively. On the other hand, Wong (1975), Barksdale and Bachus (1983) solutions are widely used for the assessment of shearing-induced failures. For stone columns contructed in very soft clayey deposits, punching-induced failure mechanism can be assessed by Aboshi et al., 1979.

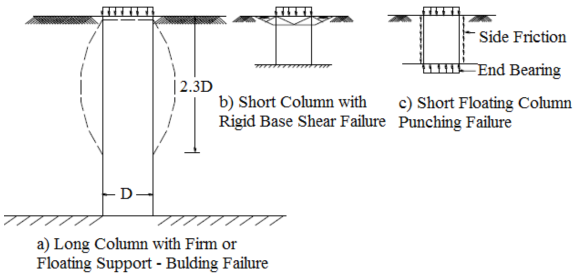

Figure 1. Failure mechanisms of a single stone column in a homogeneous soft layer (Barksdale and Bachus, 1983)

$$q_{ult}=\left(\gamma_c z k_{pc} + 2c_o\sqrt{k_{pc}}\ \right)\frac{1+\sin\varphi_s}{1-\sin\varphi_s} \tag{1}$$

$$q_{ult}=\left(F'_c C_o + F'_q Q_o\ \right)\frac{1+\sin\varphi_s}{1-\sin\varphi_s} \tag{2}$$

$$q_{ult}=(\sigma_{ro} + 4C_o)\frac{1+\sin\varphi_s}{1-\sin\varphi_s} \tag{3}$$

$$q_{ult}=\ C_o\,N_c + \left(\frac{1}{2}\,\gamma_c\,BN_\gamma\right) + \gamma_c\,D_f\,N_q \tag{4}$$

where;

γ_c : unit weight of soil

z : total depth of the limit of bulge of the column

k_{pc} : coefficient of passive earth pressure of soil

c_o : cohesion

φ_s : angle of internal friction of stone column

F'_c, F'_q : cavity expansion factors

Q_o : mean stress within the zone of failure

σ_{ro} : initial radial effective stress

B : foundation width

N_c, N_γ, N_q : dimensionless bearing capacity factors

D_f : depth of foundation

On the basis of these existing studies, it can be concluded that the ultimate bearing capacity of a stone column is a function of the column diameter, strength and stiffness responses of stone column and native soil materials. Usually, the column length is judged to have a negligible effect on the "long" column ultimate bearing capacity. This conclusion is also supported by the results of model tests. It was shown that the load transfer mechanism is simply due to skin friction or adhesion along the shaft (Hughes and Withers, 1974).

Test results also indicated that the ultimate capacity of stone column is governed primarily by the maximum radial reaction (confinement) of the soil which is limited by bulging failure, and extend of vertical movement in the stone column was limited to about 4 times the column diameter. In the literature there exist a number of alternative approaches to assess the bearing capacity of a single column and group of columns (e.g. Etezad et al., 2006).

Effect of column diameter on bearing capacity has also been investigated on the basis of laboratory tests, which were performed on 40, 50 and 70 mm diameter stone columns with constant length to diameter ratio of six (Ali et al., 2010). Results of these studies suggested that relatively small diameter stone columns mobilize larger capacities at the same level of deformations. In simpler terms, smaller diameter stone columns mobilize their capacity much faster with increasing vertical displacements. Bae et al. (2002) studied the factors affecting the failure mechanism of stone columns with laboratory model tests, and compared their findings with finite element model solutions. They concluded that bulging failure for a single stone column is usually observed at a depth of 1.6 to 2.8 columns diameter.

3 Constructions of Columns

As discussed in previous section, one of the main parameters affecting both capacity and deformation behavior of stone columns is the construction process. In the field 76 Rammed Aggregate Piers were installed by Impact® System construction procedures. RAP elements are constructed by following steps:

(1) a closed ended mandrel with a diameter of 36 cm is pushed into the design depth by applying static driving forces assisted with vertical dynamic energy (Figure 2(a)).
(2) the mandrel and hopper are continuously fed with aggregate (Figure 2(b)).
(3) the ramming action is applied with 100 cm up / 67 cm down compaction effort, during which vertical vibration is also introduced (Figure 2(c)). The vertical ramming actions expand the diameter from 36 cm to 50 cm, if 100 cm up and 67 cm down compaction procedure is selected. The significant increase in lateral stress combined with the high density of the stone created by the installation process provides the unique strength and stiffness of the RAP system (Handy 2001, Wissmann et al., 2001).

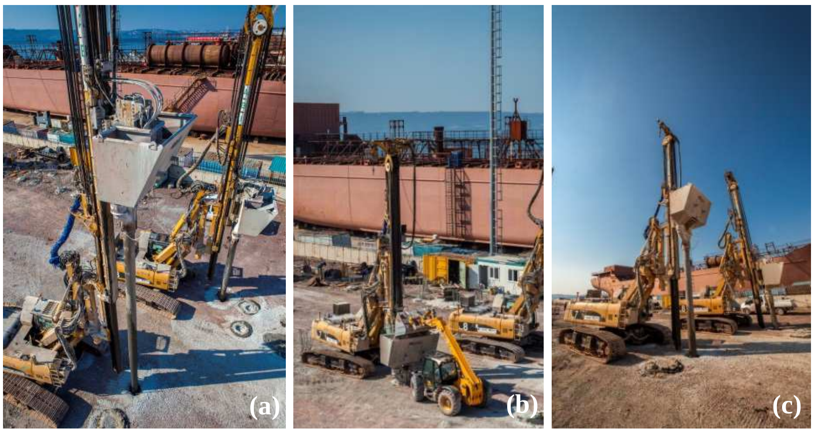

Figure 2. The construction of Impact RAPs

Considering the significant influence of construction method, findings presented herein are only applicable to stone columns installed by Impact System® construction procedure. Use of them for columns produced by using other construction techniques may be misleading.

4 Site Investigations and Soil Profiles

As part of the site investigation program, series of conventional boreholes were drilled extending to 23m – 40m depths. At various depths, standard penetration tests were performed. As a part of investigation program, both disturbed and undisturbed soil samples were retrieved. Figure 3 presents representative soil profile at Afyon city site. It mostly consist of normally consolidated, low to high plasticity, soft to stiff sandy, silty clay (CL-CH) layers extending to depths of 19,5m from existing ground level. Below this layer, stiff to very stiff sandy, silty clay layer is located. Groundwater table is reported to be at approximately 1.0m – 3.5m depth range.

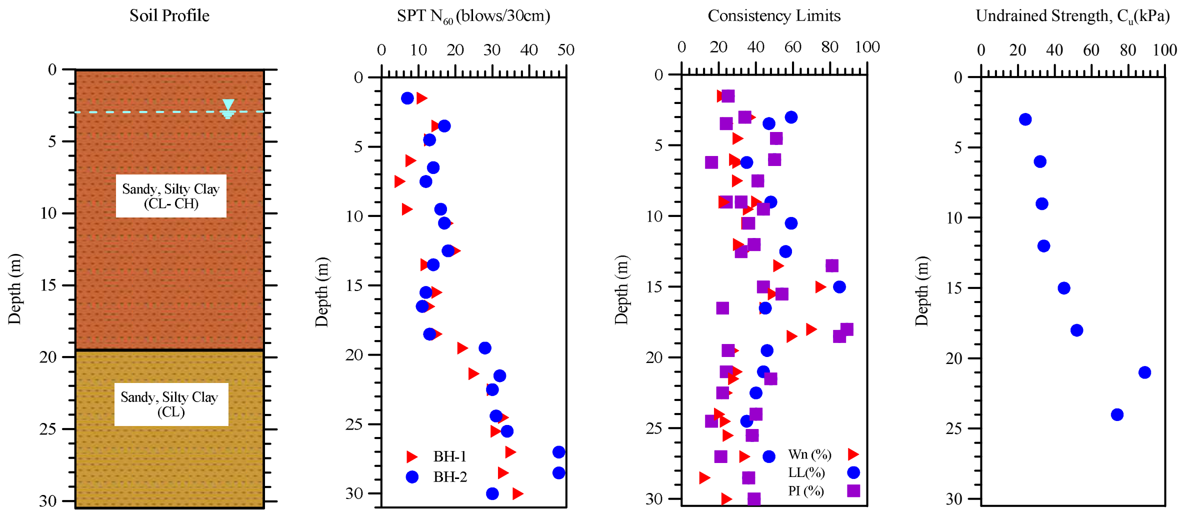

Figure 3. A representative soil profile at Afyon city site

5 Field Load Tests

In the filed after a minimum of two weeks, RAP elements were loaded. This field load test is widely referred to as "quick" tests due to relatively rapid application of the loading scheme. The test procedure is very similar to pile load tests defined by ASTM D 1143. Table 1 summarizes the loading and unloading scheme followed as part of the loading test.

Table 1. Typical 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.

As part of the test, loading is directly applied on the pier, as opposed to alternative distributed application of the load on both the site soil and pier which is widely referred to cell loading. Staged loading starting with 5% of the service load has been continued until the pier is tested under 150% of its service load. Then an unloading procedure was followed.

The modulus load tests of RAP elements often incorporate tell-tales at different elevations within the pier. 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. Figure 4 presents the results of field load tests again for Afyon city site.

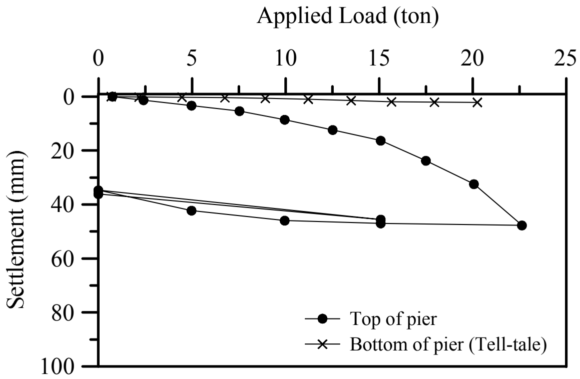

Figure 4. A representative load test results from Afyon city site

5 Field Load Test Results

76 loading tests were performed on Rammed Aggregate Piers (RAP) installed in soft to stiff clayey soils to assess the bearing capacity and stiffness responses of individual piers. The 50 cm diameter RAPs were constructed at sixteen different soil sites with varying lengths of 6.0 m to 18.0 m. The ultimate bearing capacity of RAP elements was assessed by using these field loads vs. displacement responses. The hyperbola fitting approach was used to estimate the ultimate capacity for the test at which ultimate capacity could not be reached during field load tests. Similar to Reese and O'Neill (1988), the field load test results were presented by normalized responses (i.e.: graphs of load normalized by ultimate bearing capacity, versus settlement normalized by pier diameter. Then these normalized responses were grouped as functions of representative N_{60} values, which represent confining soils strength and stiffness characteristics.

The corrected SPT N_{60} values from hammer energy efficiency were used to calculate the representative N_{60} values. In the estimation of representative N_{60} values, a linear weighting scheme, linearly decreasing from 1 at the ground surface to 0 at the tip of the pier, was used to overweight the shallower soils shear contribution as compared to deeper ones. This weighting is preferred due to the fact that mobilized pier capacity is mostly due to skin friction between the piers and the soil and it mobilizes first at shallower depths first. The representative SPT N_{60} values (SPT $N_{60,rep.}$) obtained from weighted arithmetic were summarized in Table 2.

Table 2. A summary of input parameters of the compiled database

Number	**Project Sites**	**$N_{60,rep.}$**	**RAP Length (m)**	**Number**	**Project Sites**	**$N_{60,rep.}$**	**RAP Length (m)**
1-5	Afyon-1	3-5	8/11/16	36-43	Sivas	5-15	7/9/10/12
6-9	Afyon-2	9	14/17	44-51	Yalova-1	6-12	12/14/16
10-13	Aydın	7-8	13/18	52-57	Yozgat-1	8-12	8/10/12/15/17
14-26	Bursa	11-14	16/17	58-63	Yozgat-2	4-10	9/10/12/15
27-28	Gaziantep-1	12	7/8	64-67	Yalova-2	12-14	15
29-30	Gaziantep-2	13	9	68-69	Kırklareli-1	12	7
31	İstanbul-1	13	10	70-73	Kırklareli-2	16	6
32-33	İstanbul-2	2-3	8/14	74-76	Gaziantep-3	12	8
34-35	Kayseri	22	17				

5.1 Mobilization of the skin friction

Reese and O'Neill (1988) assessed a number of compression pile load test data obtained from full-size drilled piers constructed in cohesive and cohesionless soils. On the basis of test results, they developed normalized load-transfer curves for isolated drilled piles. These curves express the mobilized capacity of piles as a function of normalized settlement. Inspired by this, 76 load test results were similarly evaluated in order to assess capacity mobilization responses of RAPs constructed in the cohesive soils. The ultimate bearing capacity (Q_{ult}) was obtained from load vs. displacement response. For the tests where Q_{ult} is not achieved during the field load tests, hyperbola fitting method was used to assess the ultimate capacity as illustrated in Figure 5.

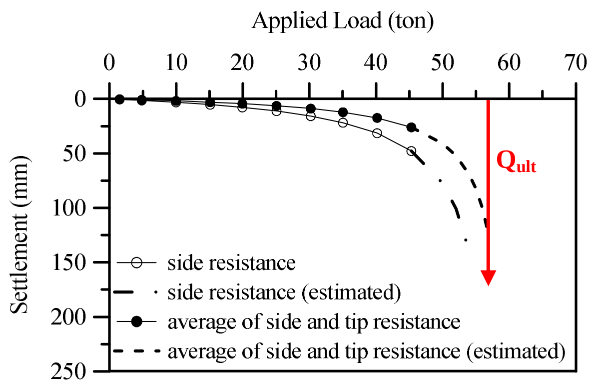

Figure 5. Estimation of Q_{ult} using load-settlement responses Afyon city site

The average values of the displacements at the top of the pier and at the tell-tale plate were calculated in order to estimate the average deformations exerted on the Impact Pier® elements. The curves of the load normalized by estimated Q_{ult} versus settlement normalized by diameter (D=50cm) was plotted for each field load test. These are shown in Figure 6 for the representative SPT N_{60} values, which vary in the range of 2 to 22 blows/30 cm.

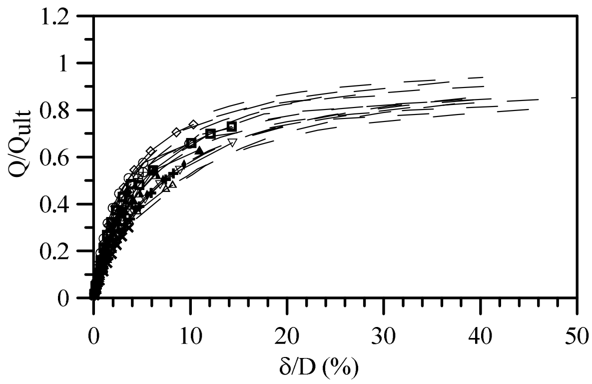

Figure 6. Available normalized field load tests data

For illustration purposes, the bearing capacity mobilization response is shown in Figure 7 for a cohesive soil layer with representative SPT N_{60} = 9 blows/30 cm. On the same figure, the maximum capacity estimation by using a hyperbolic curve is also shown. Monitored values are shown by solid lines, while extrapolated response by using hyperbolic expression is shown by dash lines.

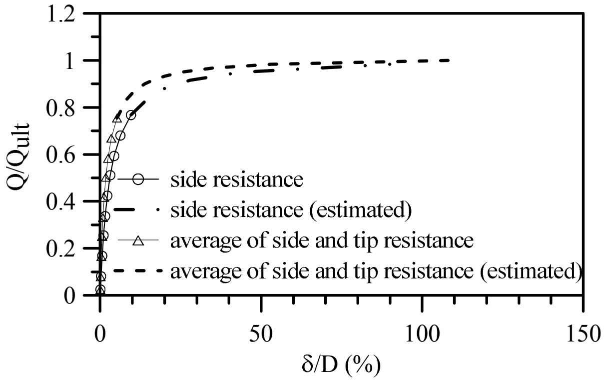

Figure 7. The graphs of normalized load-settlement (SPT N_{60} =9 blows/30 cm)

Figure 8 presents the variation of normalized responses with representative SPT N_{60} values corresponding to minimum and maximum representative N_{60} values of 2 and 22 blows/30 cm.

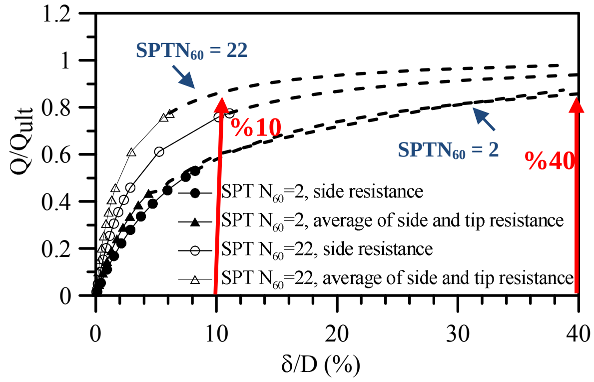

Figure 8. The normalized load-settlement responses for representative N_{60} values of 2 and 22 blows/30 cm

For comparison purposes, the normalized pile capacity curves for skin friction in cohesive soils proposed by Reese and O'Neill (1988), is presented along with the estimated normalized responses for Impact Pier® elements in Figure 9(a). The normalized load-settlement curves assessed using by the average values of the displacements at top the pier and at the tell-tale plate for SPT N_{60} values of between 2 to 22 are also shown in the same figure. The mobilization of capacity for RAP and reinforced concrete pile elements is surprisingly similar except that reinforced concrete columns exhibit 35-60 times more rigid response. In Figure 9(b), the resulting proposed normalized capacity mobilization responses as functions of representative SPT N_{60} values are shown.

On the basis of these normalized field responses, it is concluded that: i) the shaft resistance is observed to be fully mobilized at normalized displacements of 40 % of RAP diameter for very soft clays to 10 % of RAP diameter for firm clays, ii) up to normalized displacements of 2-5% of RAP diameter, 30-50% of the shaft resistance capacity is mobilized in a rather linear elastic manner, iii) normalized capacity mobilization response of RAPs is more flexible than the ones of bored piles, iv) under compressive loads, RAPs exhibit a strain hardening response, as a result of which the design-basis capacity is dominated by allowable settlements. The proposed normalized capacity and displacement response curves enable performance-based assessment and design of RAPs.

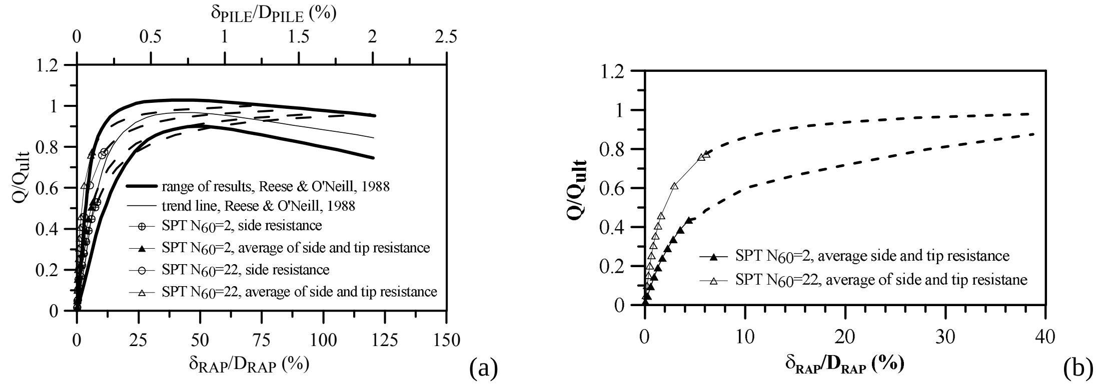

Figure 9. a) Comparision of normalized load-settlement curves for RAP elements and Reese & O'Neill (1988) Method, b) normalized load-settlement for SPT N_{60} = 2 and 22 blows/30 cm

6 Summary and Conclusions

Within the confines of this paper, a normalized deformation (performance) based capacity mobilization assessment scheme is presented for Rammed Aggregate Piers (RAP). For this purpose, field load tests performed on 76 RAPs in Turkey were assessed. 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. 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. The results of RAP load tests were summarized in the form of normalized mobilized capacity versus settlement curves as functions of representative SPT N_{60} values. The normalized field load test database revealed that:

i) the shaft resistance is observed to be fully mobilized at normalized displacements of 40 % of RAP diameter for very soft clays to 10% for firm clays,

ii) up to normalized displacements of 2-5% of RAP diameter, 30-50% of the shaft resistance capacity is mobilized in a rather linear elastic manner,

iii) normalized capacity mobilization response of RAPs is more flexible than the ones of bored concrete piles,

iv) under compressive loads, RAPs exhibit a strain hardening response, as a result of which the design-basis capacity is dominated by allowable settlement criterion.

The proposed normalized capacity and displacement response curves enable performance-based assessment and design of RAPs.

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