Proceedings of the XVI ECSMGE
Geotechnical Engineering for Infrastructure and Development
ISBN 978-0-7277-6067-8

doi:10.1680/ecsmge.60678

Performance-based assessment of rammed aggregate piers

Une evaluation sur le comportement des colonnes ballastees pilonnees

K.O. Cetin[*1], E. Kurt Bal[2] and L. Oner[2]

[1] *Department of Civil Engineering, METU, Ankara, Turkey*
[2] *Sentez Insaat, Istanbul, Turkey*
[*] *Corresponding Author*

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 63 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.

RÉSUMÉ Cet article a pour objet l'étude de la capacité de mobilisation de déplacement d'une colonne ballastée pilonées (RAP). A cet effet il a été effectué en Turquie 63 essais de chargement sur les RAPs. Les études de terrain sur ces sites ont révélé que les profils de sols généralisés sont principalement composés de couches d'argile normalement consolidées s'étendant à une profondeur de 18 m. En dessous de cette profondeur, habituellement un sable moyennement dense allant à un sable dense / disque grauwacke / très rigide des couches d'argile durs sont présents. Un SPT N_{60} procédure d'évaluation moyenne pondérée a été utilisée pour estimer la résistance du sol représentatif et le paramètre de rigidité dans les sols cohérents. Les résultats de tests de chargement de (RAP) ont été résumées sous forme de courbe de la capacité mobilisée normalisée en fonction de valeurs de tassement SPT N_{60} représentatives. La base de données de essais de chargement sur le terrain normalisé a révélé que: i) la résistance cylindrique est observée pour être pleinement mobilisé pour des déplacements normalisées de 40% du diamètre de RAP pour les argiles très doux et à 10% du diamètre de RAP pour les argiles fermes, ii) jusqu'à déplacements normalisées de 2-5% du diamètre de RAP, 30-50% de la capacité de résistance du cylindre est mobilisée d'une manière élastique plutôt linéaire, iii) la réponse de la mobilisation des capacités normalisée des RAPs est plus flexible que celles de pieux forés en béton , iv) sous charges de compression, RAPs présentent une réponse de durcissement, à la suite de laquelle, la capacité de dimensionnement est dominée par le critère de tassement admissible. Les courbes normalisées proposées permettent de définir la capacité de déplacement évalué selon la conception des RAPs.

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 conventional solutions (e.g.: deep foundations or over excavation and replacement of compressible soils). Within the confines of this manuscript, the deformation performance of 50 cm diameter RAP elements, constructed by bottom-fed dry method (Geopier-Impact System), is evaluated. The resulting response is summarized as normalized load-settlement curves obtained from full scale load tests. For this purpose, 63 RAP field load tests were used, which were constructed at thirteen 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).

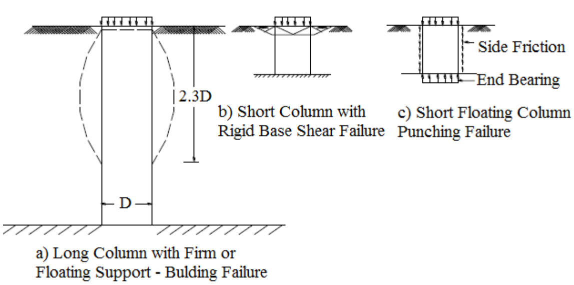

Figure 1. Failure mechanisms of a single stone column in a homogeneous soft layer (Barksdale and Bachus, 1983).

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

$$q_{ult}=\left(F_c^1 C_o + F_q^1 Q_o\right)\frac{1+\sin\varphi_s}{1-\sin\varphi_s} \tag{2}$$

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

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

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
$\emptyset_s$: 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_q, N_γ : 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 CONSTRUCTION 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 63 rammed aggregate piers were installed by Geopier-Impact construction procedures. Impact 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 vibration (Figure 2a).

(2) the mandrel and hopper are continuously fed with aggregate (Figure 2b).

(3) the ramming action is applied with 100 cm up / 67 cm down compaction effort, during which vertical vibration is also introduced (Figure 2c). 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.

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 following the Geopier-Impact construction procedure. Use of them for columns produces via other techniques will be misleading.

4 SITE INVESTIGATION

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 documented after the site investigation program performed at one of the select sites (Yalova site). It mostly consist of normally consolidated, low to high plasticity, soft to stiff clay (CL-CH), where scattered silty and sandy layers extending to depths of 4m - 18m 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 0.0m – 5.0m depth range.

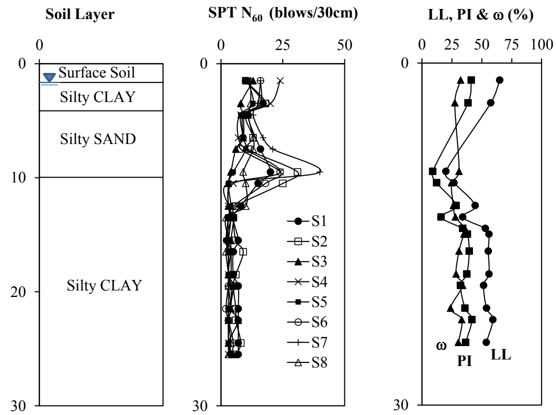

Figure 3. A representative soil profile at Yalova site

5 FIELD LOAD TEST

This 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. As part of the test, test load 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. Field load tests were performed by closely following the loading scheme summarized in Table 1. 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 Geopier-Impact 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.

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.

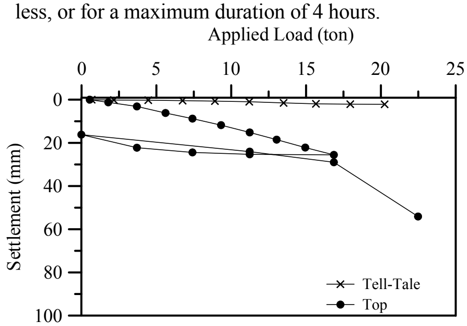

Figure 4. A representative load test results from Yalova site.

6 FIELD LOAD TEST RESULTS

As previously discussed, 63 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 thirteen different soil sites, and all piers were constructed to a final diameter of 50 cm with varying lengths of 8.0 m to 17.0 m. Within the scope of this paper, the ultimate bearing capacity was assessed using load vs. displacement responses. The hyperbola fitting approach was used for the load tests, at which ultimate capacity could not be reached during loading. 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.

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

Project Sites	$N_{60,rep.}$	RAP Length (m)	No
Afyon-1	3-5	8/11/16	1-5
Afyon-2	9	14/17	6-9
Aydın	7-8	13/18	10-13
Bursa	11-14	16/17	14-26
Gaziantep-1	12	7/8	27-28
Gaziantep-2	13	9	29-30
İstanbul-1	13	10	31
İstanbul-2	2-3	8/14	32-33
Kayseri	22	17	34-35
Sivas	5-15	7/9/10/12	36-43
Yalova	6-12	12/14/16	44-51
Yozgat-1	8-12	8/10/12/15/17	52-57
Yozgat-2	4-10	9/10/12/15	58-63

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. A representative load-settlement curve is shown in Figure 4 for illustration purposes.

6.1 *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, 63 load test results were similarly evaluated in order to assess capacity mobilization responses of Impact piers 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 test, hyperbola fitting method was used to assess the ultimate capacity. 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. Field load tests were grouped on the basis of representative SPT N_{60} values, which vary in the range of 2 to 22 blows/30 cm.

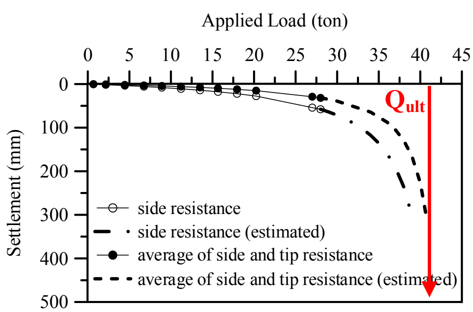

Figure 5. Estimation of Q_{ult} using load-settlement responses Yalova site.

For illustration purposes, the bearing capacity mobilization response is shown in Figure 5 for a cohesive soil layer with SPT N_{60} = 6 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.

The curves of the load normalized by estimated Q_{ult} versus settlement normalized by diameter (D=50cm) was plotted for each field load test. The resulting responses obtained from Yalova site, which was represented by an SPTN_{60} = 6 blows/30 cm was shown in Figure 6. Figure 7a presents the normalized curves for 63 field load tests. The soil sites have a representation SPT N_{60} value ranging from a minimum of 2 to a maximum of 22 blows /30 cm.

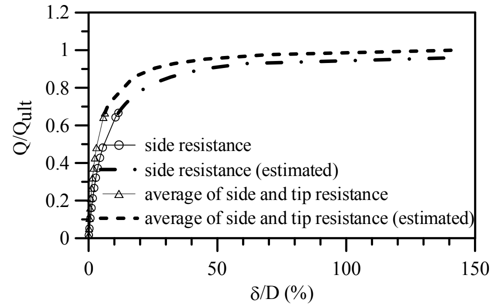

Figure 6. The graphs of normalized load-settlement (SPT N_{60} =6).

To illustrate the variation of normalized responses with representative SPT N_{60} values, responses corresponding to minimum and maximum representative N_{60} values are shown in Figure 7b with a larger scale.

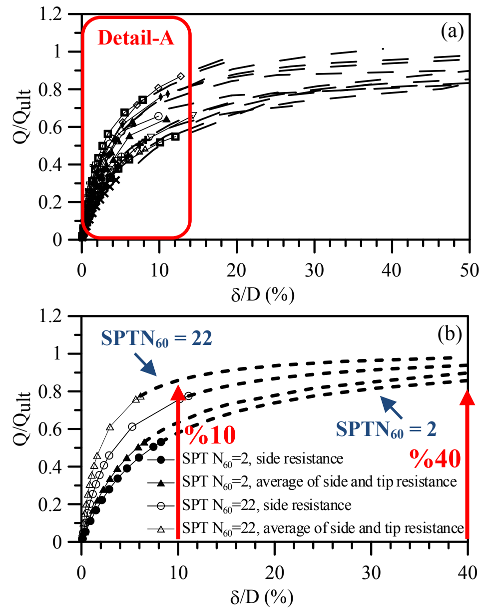

Figure 7. The graphs of normalized load-settlement.

For comparison purposes, the normalized pile capacity curves for skin friction in cohesive soils proposed by Reese and O'Neill (1988), is presented together with the estimated normalized responses for Impact pier elements, as shown in Figure 8a. 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 shown in Figure 8b.

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.

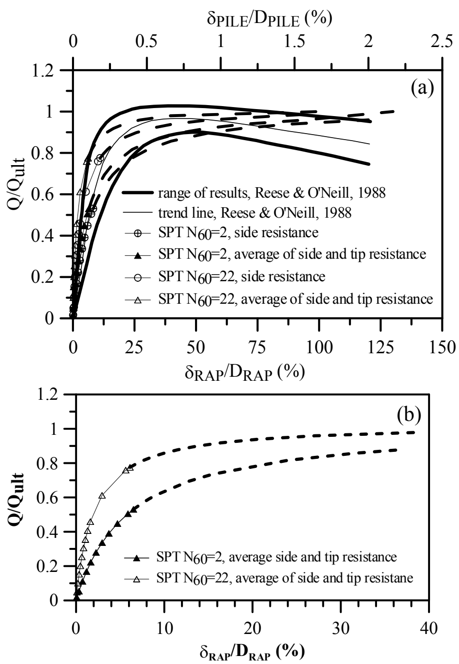

Figure 8. a) Comparision of normalized load-settlement curves for RAP elements and Reese & O'Neill (1988) Method, b) normalized load-settlement for SPT $N_{60} = 2$ and 22.

7 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 63 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.

REFERENCES

Ali, K., Shahu, J.T. & Sharma, K.G. 2010. Behaviour of Reinforced Stone Columns in Soft Soils: An Experimental Study, Indian Geotechnical Conference.

ASTM D1143 – 81. Reapproved 1994. Standard Test Methods for Deep Foundations Under Static Axial Compressive Load.

Bae, W.S., Shin, B.W. & An, B.C. 2002. Behaviors of Foundation System Improved with Stone Columns, Kitakyushu, Japan.

Barksdale, R.D. & Bachus, R.C. 1983. Design and Construction of Stone Column. Report No.FHWA/RD-83/026, Springfield, VA.

Huges, J.M.O. & Withers, N.J. 1974. Reinforcing of Soft Cohesive Soils with Stone Columns. Ground Engineering, 7(3): 42-49.

Datye, K.R. & Nagaraju, S.S. 1975. Installation and Testing of Rammed Stone Columns. Proc. of IGS Specialty Session, Bangalor, India: 101-104.

Madhav, M.R. & P.P. Vitkar (1978) Strip footing on weak clay stabilized with a granular trench or pile. Canadian Geotechnical Journal, 15(4), 605 – 609.

Aboshi, H., Ichimoto, E., Harada, K., Emoki, M. 1979. The Composer- A Method to Improve the Characteristics of Soft Clays by Inclusion of Large Diameter Sand Columns. Proc. of Int. Conf. on Soil Reinforcement, E.N.P.C, 1, Paris: 211–216.

Reese, L. C. & O'Neill, M. W. 1988. Drilled Shafts: Construction Procedures and Design Methods, Pub. No. FHWA-HI-88-042, U.S., Washington, D.C., 564–564.