The Influence of Dynamic Replacement Method on the Adjacent Soil

Jerzy Sękowski¹ · Sławomir Kwiecień¹ · Piotr Kanty¹

Received: 20 October 2016/Revised: 21 May 2017/Accepted: 3 June 2017/Published online: 24 July 2017
© The Author(s) 2017. This article is an open access publication

Abstract The purpose of this paper is to report on the field tests for the formation of a single DR (dynamic replacement) column and its influence on the surrounding weak soil deposit. The influence of the column formation has been assessed with piezocone and dilatometer measurements, as well as by changes in the strength and deformation parameters obtained from the field tests. These measurements were carried out during and after the column formation and at varying distances from the column. The tests carried out have shown that soil close to the column became weaker during the column formation. As a result the soil stiffness and strength were found to increase over time. The weaker the soil was in natural state, the more significant the strengthening effect became. That indicates that changes occurring around a DR column are complex. The measurements suggest that the changes in soil structure have a tendency to be dependent on the distance from the column, elapsed time and the type and the initial condition of the soil.

Keywords Dynamic replacement · Field tests · Soil strengthening · Parameter changes · Adjacent soil

1 Introduction

In the dynamic replacement method, stiff granular columns are rammed into natural soil to improve its strength and stiffness characteristics (Fig. 1). Previous studies on DR method have been based on the results from different types of field tests (Table 1). They were sometimes additionally accompanied by laboratory tests [1, 2]. All tests focused on the influence of a group of columns on the adjacent soil. The procedures were performed before and after soil strengthening. In few cases they were performed during column formation [3, 4].

The findings show that the stiffness and strength of the column is greater than in the surrounding soil. The values of the cone resistance q_c from CPTs which penetrated columns have been about 20–30 times [3], 13–30 times [5], 12–57 times [4] and up to 150 times [6] higher than those measured in the natural weak soil. Similarly, pressuremeter tests have recorded maximum pressure increasing approximately 10–13 times [7] and 12 times [6]. Blow counts recorded by dynamic probing are found to typically increase by at least 3 times [8].

The strengthening effect of the soil adjacent to the formed columns is subtle. It appears to be dependent upon the type of soil and the time period between the column formation and testing. The improvement effect occurred in soils which were compacted during column formation, i.e., in the peat deposits and was found both under the columns and around them. Lo et al. [6] indicated that the cone resistance in the CPT increased 7 times, the maximum pressure in the pressuremeter test (PMT)—3 times. Gunaratne et al. [4] and Stinnette et al. [9] obtained similar results.

Cohesive soils have experienced various trends of changes during and after column formation. Certain studies

✉ Jerzy Sękowski
jerzysekowski@poczta.onet.pl; rb7@polsl.pl
Sławomir Kwiecień
slawomir.kwiecien@polsl.pl; rb7@polsl.pl
Piotr Kanty
pkanty@menard.pl; rb7@polsl.pl

¹ Department of Geotechnics and Roads, Faculty of Civil Engineering, Silesian University of Technology, Akademicka 5, 44-100 Gliwice, Poland

Fig. 1 Process of dynamic replacement pillar formation

Table 1 Previous studies summary

Authors	Bates and Merifield [3]	Gunaratne et al. [4]	Stinnette et al. [9]	Han [1]	Lo et al. [6]	Yee and Chua [5]	Dumas et al. [10]	Godlewski and Saloni [8]	Hamidi et al. [7]	Kwiecień and Sękowski [2]
Test type	CPT, DMT	CPT, DMT	CPT, DMT	CPT, DPL	CPT, PMT	CPT	DPL, DMT, PMT	DPL	SPT, PMT	SVT
Days after columns formation	–	0	0	28	38, 40	–	–	3, 21	–	14

detected a decrease in soil strength and deformation parameters [5], although there is no change observed both under [10] and with adjacent columns [8]. In contrast, some authors observed soil strengthening. Hamidi et al. [7] recorded an increase in limit pressure (by about 100%) and an increase in Menard modulus (over 350%). Both increases were recorded inside a DR column, while only small changes were noticed between blows. Dumas et al. [10] observed that (PMT), standard penetration test (SPT) and deep cone penetration test (DCPT) parameters measured after strengthening were 1.4–2.0 higher than initial. Han [1] has presented an increase of q_c and number of blows (from SPT) which vary from 1.5 to 4 times.

The available research has not considered the influence of a single DR column on the surrounding soil. This paper seeks to address this aspect of column behavior. It also gives some insights on time and distance influences. The process of DR column formation is complicated and affected by many factors. Therefore, to measure and analyze some of those first, a single DR column formation was chosen to be the topic of the research. Further research is planned to analyze a group of column (which will allow to consider other factors).

Currently researchers aim for numerical modeling of DR column formation; examples of such attempts are described in [11–13]. However, to calibrate numerical models we

first need to perform wide range of well documented in situ tests. Such tests are the focus of the article. Many studies are performed at different sites, but as they mainly are based on a few in situ results [14], they do not form good enough basis for numerical modeling.

2 Test Field and Research Program

CPTU, DMT and boreholes were carried out to determine soil characteristics on the test field. The soil profile consisted of four layers [15]. The first layer (layer I) up to 1.5 m below ground level (b.g.l.) comprised silty sands and sandy silts. The second layer (layer II) between 1.5 and 2.5 m b.g.l. is silt (10% sand, 84% silt, 6% clay). The third layer (layer III), up to 4.8 m b.g.l. was built of silt/silty sand (20% sand, 74% silt, 6% clay). The fourth layer (layer IV) consisted of fine and medium sand (Figs. 2, 3). The water table was found during drilling at a depth of 4.8–5.3 m b.g.l. and rose to 3 m b.g.l.

The DR column was formed using a free dropped, 10 t barrel-like pounder from the height of 15 m. The pounder diameter was 1.0 m in the middle section, whereas at the bottom and the top it was 0.8 m. The mixture of fine gravel with coarse sand and rubble (0–200 mm fraction in 1:1 proportion) was used as backfill material. The uniformity

Fig. 2 Results of visual inspection and CPTU tests (q_c) before (continuous lines) and 30 days after columns formation (dashed lines)

Fig. 3 Results of visual inspection and DMT tests (P_0 continuous lines, P_1 dashed lines) before (black) and 30 days after columns formation (grey)

coefficient of the material was greater than 25 and the coefficient of gradation was less than 1. Approximately 18 m^3 of the mentioned material was used to form the column.

The column was formed by dropping the rammer onto the soil 36 times from different heights (2–15 m). The column formation was divided into three stages (10 + 15 + 11 drops).

During the test, ground heave was measured in points located 2, 3, 4 and 6 m from the column axis. The total volume of heave was roughly 7.5 m^3 . The maximal uplift was noted 2 m from the column (0.15–0.32 m). At the point located 6 m from the column axis, the uplift was between 0 and 0.03 m.

CPTU and DMT tests were carried out at various time intervals and at different distances from the column and depth. The first series of measurements was carried out before the stone column formation and consisted of four CPTU tests conducted at 2, 3, 4 and 6 m from the column axis and of three DMT tests performed at 2, 3 and 6 m from the column axis. A further series of tests were conducted at points located on the circumference of a circle passing through the points from the initial tests. This was designed to ensure that future testing would not be unduly affected by the previous tests. The field tests were conducted after 1/3, 2/3 and a completion of the column to the full depth. Tests were performed 1, 8 and 30 days after construction.

Table 2 Mechanical soil parameters obtained from CPTU, DMT tests (before and after strengthening)

Distance from column (m):	2				3				4				6			
Time since column construction (days):	0	1	8	30	0	1	8	30	0	1	8	30	0	1	8	30
CPTU layer II																
ϕ' (°)	20	20	20	27	21	25	26	25	25	26	26	26	24	24	26	27
c' (kPa)	4	5	7	7	6	6	10	12	6	5	7	8	5	5	5	5
S_u (kPa)	200	225	210	200	80	210	205	153	162	170	190	178	137	120	170	203
M (MPa)	20	22	21	20	10	16	19	18	16	18	18	18	15	15	15	16
CPTU layer III																
ϕ' (°)	28	28	29	29	29	29	29	29	28	27	26	27	28	29	27	29
c' (kPa)	8	8	7	8	8	7	8	7	8	7	7	8	5	5	7	5
S_u (kPa)	215	232	237	232	258	200	220	208	223	195	185	192	132	158	175	175
M (MPa)	20	20	21	20	21	18	19	18	20	18	18	18	15	18	16	18
DMT layer II																
ϕ' (°)	x	x	x	x	32	35	35	35	–	–	–	–	32	x	x	x
M (MPa)	x	x	x	x	5	5	5	6	–	–	–	–	4	4	4	5
DMT layer III																
ϕ' (°)	33	34	34	34	33	34	34	35	–	–	–	–	33	45	34	35
M (MPa)	20	29	28	30	20	30	30	30	–	–	–	–	20	30	30	30

x parameters not possible to define with used procedure

The 3.8 m long column was constructed. The head of the column was 1.9–2.0 m in diameter and the maximum diameter of 2.8 m was recorded at a depth of 1.9 m (Figs. 2, 3).

CPTU and DMT were performed using a static probe Hyson 200 kN produced by Dutch company A.P. van den Berg Machinefabriek. The piezocones had base surface area equal to 10 cm², friction sleeve surface equal to 150 cm², apex angle 60° and a filter installed directly behind the cone tip (u_2). The soundings were made with a constant penetration velocity of 20 mm/s [15].

The following parameters were recorded continuously during the tests: cone resistance (q_c), sleeve friction (f_s), and excess pore water pressure (u_2). They were standardized and normalized [16–18] to the following values: corrected cone resistance q_t , friction ratio R_f , excess pore water pressure parameter B_q and normalized effective cone resistance Q_t . The soil type was determined in two stages. During the first stage, Harder-von Bloh procedure [19] was applied to divide soil into layers and localize them using the classification system proposed by the Department of Geotechnics at Poznan University of Life Sciences [20]. The second stage consisted in grouping the soil types by applying Hegazi–Mayne procedure [21]. In the second phase, the division was based on Hegazi–Mayne procedure [21] and the soil type was determined using Robertson's diagram [22]. This second phase was applied to confirm the

soil type indicated in the first stage. Deformation parameters, i.e., effective friction angle (ϕ'), effective cohesion intercept (c') were determined on the basis of Senneset and Janbu's procedure [23] whereas undrained shear strength—on the basis of Lunne et al. [16]. The latter, as well as Mayne [24, 25] served also to indicate deformation parameters, i.e., constrained modulus.

Dilatometric tests were performed with a flat plate dilatometer. Pressure values were recorded at 0.2 m intervals at increasing depth [15]. Based on these readings, the following parameters were indicated: non-dimensional mechanical properties (I_D), non-dimensional lateral stress indices (K_D) and dilatometric moduli (E_D). With these parameters it was possible to estimate the soil type and its mechanical parameters (ϕ' , M), applying the procedures prepared by Marchetti [26].

3 Results

The outcome of the tests could not be analyzed in the first superficial layer (layer I) due to the detrimental effect of heavy machinery and weather conditions. Layer IV consisted of sands where the compaction mechanism caused by the use of high energy impact was already recognized (e.g., [27]). Detailed parameters of layers II and III which have been subjected to extensive analysis are shown in Table 2.

Fig. 4 Results of CPTU tests (q_c) in different time intervals 2 and 3 m from the column center

During the column formation a significant (up to 50%) decrease of cone resistance q_c was noted in the closest vicinity of the column (i.e., up to 3 m from the column axis) in both layers (Fig. 4). The values of the cone resistance increased with time and exceeded the initial values in the weaker layer II by approximately 70–100% and by 30% in layer III (except 2 m from the column). Further from the column (at 4 m), the changes were only local, and 6 m from the center were not visible (Fig. 5).

After the column formation the highest q_c increases were measured in layer II in the distance of 4 m from the center (approximately 60%).

Soil friction angle and cohesion in the layer II at a distance of 2 and 3 m from the column increased after the column completion by 35 and 100%, respectively. At all the points in layer III parameters ϕ' and c' dropped by 10–50% during tamping and then increased after the column formation finally reached values that are similar to the initial values.

Similar changes were observed in the soil undrained shear strength (S_u). An increase of 90 and 48% was noted in the layer II, 3 and 6 m away from the column axis, respectively. Undrained shear strength in the layer III decreased in points located 3 and 4 m away from the

column axis (by maximum 20%) and increased in other points (by maximum 33%).

The constrained modulus in the layer II, 3 m from the column axis, increased by 90% whereas it did not change in other points. The value of the constrained modulus dropped in the layer III (10–15%) during the column formation process. After the construction, the increase was observed only in points located 6 m away from the column axis.

Pressure P_0 in layers II and III was increasing (up to 40%) already during the column formation (2, 3 m from the column) or after the formation process had been completed (6 m from the column)—Fig. 6. The maximum P_0 values were recorded in the layer II 1 day after construction (the values were up to 150% higher than the initial in the distance 6 m from the column center). The values gradually decreased over time reaching the initial values (layer II, 6 m from the column center) or exceeding these (30–120%) 30-day post-construction.

Pressure P_1 measured at 2 and 3 m from the column decreased during column formation in both layer II and III. The pressures subsequently increased during the last stage of the column formation or 1 day later (Fig. 7). This effect was more pronounced in layer II (up to 300%), while in

Fig. 5 Results of CPTU tests (q_c) in different time intervals 4 and 6 m from the column center

Fig. 6 Results of DMT tests (P_0 pressure) in different time intervals and distances from the column center

layer III the increase reached 30–50% (2, 3 m) and 100% (6 m) of the initial values.

Based on DMT testing an increase in the friction angle (up to 10%) and the constrained modulus (up to 50%) was

previously noted during the column formation (2 m from the column axis) or 1 day after construction (3, 6 m). Those values did not change afterwards.

Fig. 7 Results of DMT tests (P_1 pressure) in different time intervals and distances from the column center

Figures 2, 3, 4, 5 and 6 are presented for the purpose of individual analysis for the reader. Figure 2 shows q_c values, whereas Fig. 3 shows P_0 and P_1 readings before the column formation and 30 days later. These figures show what changes in the parameters are permanent. Additionally the parameter changes in different time intervals can be analyzed basing on Figs. 4 and 5 (for q_c), Fig. 6 (for P_0) and Fig. 7 (for P_1).

4 Discussion

The completed DMT and CPTU tests indicated that the surrounding soil softens during the dynamic replacement process. The range of the impacted zone may vary. The zone radius can be estimated as up to 2.5 times the diameter of the top of the column. The extent of the softening effect depends on soil condition. For cohesive soils, characterized by higher stiffness (layer III), the softening is considerably greater than for weaker soils (layer II). These findings are similar to those of Yee and Chua [5]. However, these authors have not determined the radius of the impacted zone.

The dynamic replacement construction is successful in improving the surrounding soil. Soil parameters after strengthening varies over time and are dependent on the initial soil condition as well as on the distance from the column. In this paper the authors examined cohesive silty soils, in which the increase of parameters were measured for 30 days. After that period, the values of mechanical parameters of layer II were higher (even by 100%) than before the ground improvement. However, soils in the layer III returned to the initial state.

The extent of strengthening zone was similar to the softening zone. Generally, the changes measured in q_c and in ϕ' , c' , S_u , M were similar to those described by Dumas et al. [10] and addresses in this paper. The dilatometer results are similar to those presented by Dumas et al. [10] and Bates and Merifield [3]. However, the cited authors did not examine the course of changes of the mentioned parameters within time, after the construction of the ground improvement. The parameters changes after 30-day post-construction can be explained by consolidation process implicated by high energy impacts. During the pounder drops in close vicinity of the column, soil is subjected to large deformations. That can lead to internal cracking in some regions. These regions can be identified with privilege filtrating paths which shorten the consolidation time.

5 Conclusion

This paper presents a summary of the results of unique (due to its wide character) field tests on DR column and surrounding soil. It has been shown that changes occurring in the soil surrounding a DR column are complex. They depend on the distance from the column, elapsed time, the type and the initial condition of the soil. During the strengthening process, the soil softens in close vicinity of the column, but then soil parameters increases over time. On the basis of the test results, it is possible to conclude that the less stiff the natural in situ soil is, the more significant the improvement becomes. The soil softening is less evident at greater distances from the column; however, the properties of surrounding soil are still improved with

time. These conclusions are true for the particular technique of DR column formation that is presented in this paper and the particular soil conditions investigated ($\sim 15\%$ sand, $\sim 79\%$ silt, $\sim 6\%$ clay).

Consideration of the pre-strengthening soil parameters in the design could underestimate soil-column interaction. This may happen even when in the design sophisticated constitutive model of soil was used [28]. The present increases in soil mechanical parameters have been identified for the ground surrounding a single DR column only and would be expected to be subjected to further increase if more columns were constructed. Nonetheless, if the acceptance tests are carried out too early then it might be a case that DR has not met its design criterion.

Acknowledgements This outcome has been achieved with the financial support of Polish National Research Center in Project No. 1989/B/T02/2011/40.

Open Access This article is distributed under the terms of the Creative Commons Attribution 4.0 International License (<http://creativecommons.org/licenses/by/4.0/>), which permits unrestricted use, distribution, and reproduction in any medium, provided you give appropriate credit to the original author(s) and the source, provide a link to the Creative Commons license, and indicate if changes were made.

References

- Han J (1998) Ground modification by a combination of dynamic compaction, consolidation, and replacement. In: Proceedings of the 4th international conference on case histories in geotechnical engineering, St. Louis, pp 341–346
- Kwiecień S, Sękowski J (2012) Kolumny kamienne formowane w technologii wymiany dynamicznej. Wydawnictwo Politechniki Śląskiej, Gliwice
- Bates L, Merifield R (2010) Evaluation of the CPT for assessing ground improvement by dynamic replacement. In: Proceedings of the 2nd international symposium on cone penetration testing, Huntington Beach, pp 2–52
- Gunaratne M, Mullins G, Stinnette P, Thilakasiri S (1997) Stabilization of florida organic material by dynamic replacement. Report No. 665
- Yee Y, Chua C (2009) Ground improvement techniques for east coast expressway phase 2, Malaysia. In: Ground improvement technologies and case histories, pp 705–712
- Lo K, Ooi P, Lee S-L (1990) Unified approach to ground improvement by heavy tamping. *J Geotech Eng* 116(3):514–527. doi:10.1061/(ASCE)0733-9410(1990)116:3(514)
- Hamidi B, Varaksin S, Nikraz H (2010) Dynamic replacement for constructing embankments and walls on soft soil. In: Proceedings of the 3rd international conference on problematic soils, Adelaide, pp 105–112
- Godlewski T, Saloni J (2006) Wzmocnienie podłoża gruntowego kolumnami na przykładzie odcinka Trasy Siekierskiej w Warszawie. *Zeszyty Naukowe Politechniki Białostockiej—Budownictwo* 28(2):337–348
- Stinnette P, Gunaratne M, Mullins G, Thilakasiri S (1997) A quality control programme for performance evaluation of dynamic replacement of organic soil deposits. *Geotech Geol Eng* 15:283–302. doi:10.1023/A:1018467810200
- Dumas J, Morel J, Beaton N (1993) Dynamic compaction using select fill displacement methods. In: Proceedings of the 3rd international conference on case histories in geotechnical engineering, St. Louis, pp 1067–1077
- Sołowski W, Sloan S, Kanty P, Kwiecień S (2013) Numerical simulation of a small scale dynamic replacement stone column creation experiment. In: III International conference particles, pp 522–533
- Danilewicz A, Sikora Z (2015) Numerical simulation of crater creating process in dynamic replacement method by smooth particle hydrodynamics. *Studia Geotechnica et Mechanica* 36(3):3–8. doi:10.2478/sgem-2014-0022
- Koohsari H, Alielahi H, Najafi A, Adampira M (2016) Evaluation of factors affecting the estimated improvement depth of dynamic compaction using fuzzy method and PSO. *Soils Found.* doi:10.1016/j.sandf.2016.08.012
- Krzemiński M, Vincent P, Racinais J, Mitchell D (2017) Proceedings of the 19th international conference on soil mechanics and geotechnical engineering, Seoul
- Tschuschke W, Kroll M (2011) Analiza teoretyczna i doświadczalna wpływu wymiany dynamicznej na otoczenie gruntowe. Report a grant no. 1989/B/T02/2011/40, Poznań
- Lunne T, Robertson PK, Powell JJM (1997) Cone penetration testing in geotechnical practice. Powell, Blackie Academic and Professional, London
- Jamiolkowski M, Ghionna VN, Lancellotta R, Pasqualini E (1998) New correlations of penetration tests for design practice. In: Proceedings of 1st international symposium on penetration testing, pp 263–296
- Worth CP (1998) Penetration testing—a more rigorous approach for interpretation. In: Proceedings of the ISOPT-1, pp 303–311
- Harder H, Von Bloh G (1998) Determination of representative CPT-parameters. In: Proceedings of the conference the penetration testing in U.K., Birmingham, pp 237–240
- Młynarek Z, Tschuschke W, Wierzbicki J (1997) Klasyfikacja gruntów podłoża budowlanego metodą statycznego sondowania. Materiały konferencyjne XI Krajowej Konferencji Mechaniki Gruntów i Fundamentowania 2, Gdańsk, pp 119–126
- Hegazy YA, Mayne PW (2002) Objective site characterization using clustering of piezocone data. *J Geotech Geoenviron Eng* 128(12):986–996
- Robertson PK (1990) Soil classification using the cone penetration test. *Can Geotech J* 27:151–158. doi:10.1139/g90-014
- Senneset K, Janbu N (1985) Shear strength parameters obtained from static cone penetration test. ASTM Special Technical Publication No. 883, San Diego
- Mayne PW (2001) Stress-strain-strength-flow parameters from enhanced in situ tests. In: Proceedings of the international conference on in-situ measurements of soil properties and case histories, Bali, pp 27–48
- Mayne PW (2007) A synthesis of highway practice. Transportation Research Board of the National Academies No. 386, Washington
- Marchetti S (1997) The flat dilatometer: design applications. In: Proceedings of the 3rd international geotechnical engineering conference, Keynote lecture, Cairo University, pp 421–448
- Berry A, Visser A, Rust E (2000) State of the art review of the prediction of ground improvement using impact compaction equipment. In: Conference paper of South Africa transport conference, 17–20 July, South Africa
- Sternik K (2017) Elasto-plastic constitutive model for over consolidated clays. *Int J Civil Eng.* doi:10.1007/s40999-017-0193-8