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Performance of Improved Ground by Deep Mixing Technique: State of the Art Review

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***Abstract*—Nowadays, construction on weak soils is challenging due to urbanization of the world. To improve the poor engineering properties of such soils, the deep soil mixing technique has been widely employed as a soil stabilizing technique in many countries due to the applicability for use in many types of soils, speedy execution and resulted high ground strength. In this technique, the soil is mixed mechanically with a stabilizing agent commonly, cement in a powder or slurry form by a special machine equipped with mixing blades forming a hard-treated soil column. Generally, the stabilized soil which is produced, has a higher strength, lower compressibility and lower permeability than the native soil before stabilization which leads to improvement of bearing capacity and reduction of settlement. In this paper, several previous works by numerous researchers related to the ultimate bearing capacity of deep mixing ground are reviewed including experimental works and analytical methods.**

***Keywords*— Soil mixing; Soil-cement column; Bearing capacity; Settlement**

I. INTRODUCTION

The deep mixing method refers to an in-situ soil modification technique that uses a stabilizing agent to improve bearing capacity and reduce settlement [1]. It has been widely applied in Japan, UK, Poland, Malaysia, USA and other countries serving a wide extent of applications such as road and rail embankment stabilization projects [2]; encapsulation to treat the contaminated soil [3]; developing soft areas such as the Carriageway, Trasa Zielona in Poland [4]; etc.

Fig. 1, shows the sequence of column installation in five steps [5]. Firstly, positioning the mixing shaft over the desired location. Secondly, the mixing shaft penetrates the soil and continue mixing until the desired treatment depth. Afterwards, cementitious agent is injected through auger or along the shaft, which is rotating in the horizontal plane and mixes soil and binder constantly. The next step is a withdrawal phase, when injection and rotation are continued till ground surface or designed depth. Finally, the treated column is complete and reactions taking place between soil and the stabilizing agent increasing the strength of the ground.

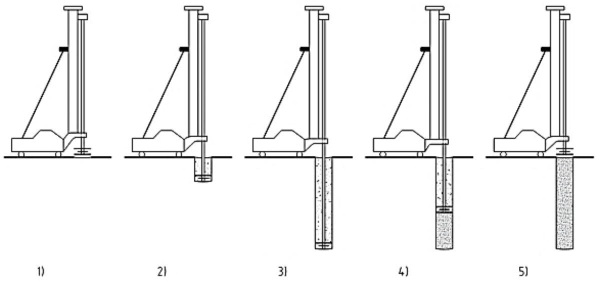


Fig. 1. Sequence of DM Installation [5]

In common practice, soil columns have diameters ranges from 0.5-1.75 m and spacing from 1-1.75 m and lengths varies from 10 to 30m. The improvement of the soil mainly depends on increasing the stiffness of the native soil as a result of physical load transfer between the columns and surrounding untreated soil. The replacement area ratio (α) is the ratio between the total sectional area of the columns to the area of the ground occupied by the columns as expressed in equation (1) and shown in Fig. 2 [6]. In common treatment applications in the United States and the Scandinavia, (α) varies between 10 to 30% [7].

$$\alpha = \sum \frac{A_p}{d_1 \times d_2} \qquad (1)$$

Where: A_p is the sectional area of a soil-cement column and d_1 and d_2 are the plan dimensions of the improvement.

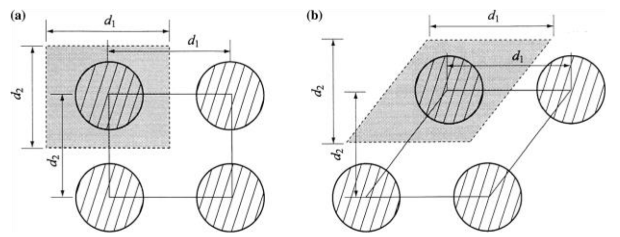


Fig. 2. Evaluation of replacement area ratio (α) in (a) Rectangular pattern, (b) Staggered pattern [6]

Chemical reactions occur between the stabilizing agent and the soil, producing cemented composite material that increases the strength of the soil, providing bearing resistance and settlement performance. Broms and Boman [8], suggested that the stabilized soil behaves as a composite foundation. A composite foundation is generally referring to a foundation formed of two different materials having different stiffnesses. The average strength of the untreated soil and cement column is regarded as the dominant factor in design for a composite foundation [4, 6, 9]. However, Terashi and Tanaka proposed a non-composite method in which the cement column acts as piles [10]. To understand the method well especially with regard to the prediction of bearing capacity and modes of failure, a review of previous works by numerous researchers including experimental work and analytical methods are discussed in the following section.

II. Experimental Studies

An early study of the behavior of the improved ground by a group of treated-soil columns was discussed based on the laboratory model test data and field measurements by Terashi and Tanaka [10]. Ten physical modelling tests were studied with a range of replacement area ratio (a_s) ranging from 13 to 32% and a strength ratio (q_{ut}/q_{uu}) ranging from 11 to 173 (q_{ut} and q_{uu} are unconfined compressive strength of treated and untreated soil respectively). The size of the tank used for the tests was 3.5 m (width) x 9 m (length) x 4 m (depth) for L-series as shown in Fig. 3 and 0.5 m (width) x 1.5 m (length) x 1m (depth) for S-series. Columns were installed in two conditions, fully and partially penetrating. In the preparation of the model ground, casing pipe was inserted to the prescribed positions of pre-consolidated model ground and soil inside the casing was removed using an auger. Then, the casing was filled with soil-cement mixture and casing pipe was pulled up. Rigid plate was used to load the model ground while rate of displacement was controlled. Although, the installation method did not represent in situ deep mixing installation, it is practical and workable for a laboratory small-scale setup as it provides a uniform column strength. The reactions of the soil-column (P_t) and the surrounding soil (P_u) were measured independently via load cells. It was noted that the stress–strain curve has a clear peak for strong column (L-2) having (q_{ut} = 1040 kPa) and a strain-softening behavior where the residual failure decreases significantly which refers to brittle failure as shown in Fig. 4. They deduced that the brittle behavior was certainly governed by the deep mixing columns as it is known that unimproved clay possesses a ductile characteristic. Also, displacements of individual column at the overall peak load was not concurring as progressive failures in the columns occurred. They found that all the fully penetrated columns were broken during the failure of the model ground. They found that the reactions of the soil-column (Pt) was 55–80% of (q_{ut}) regardless the magnitude of (a_s), while for the surrounding soil (P_u) was approximately equal to the bearing capacity of a shallow foundation on clay soil.

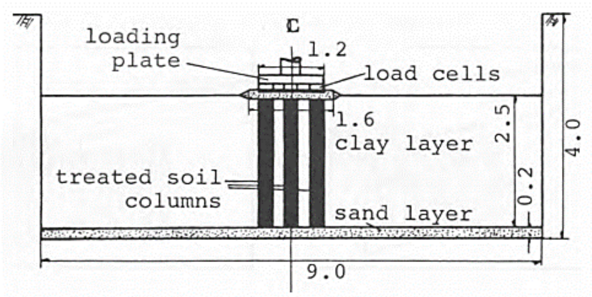


Fig. 3. Model ground for L-series [10]

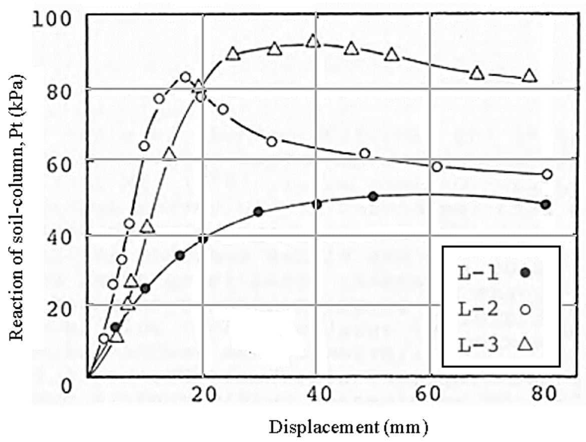


Fig. 4. Relationship between reaction of soil-column and displacement for L-series (modified after [10])

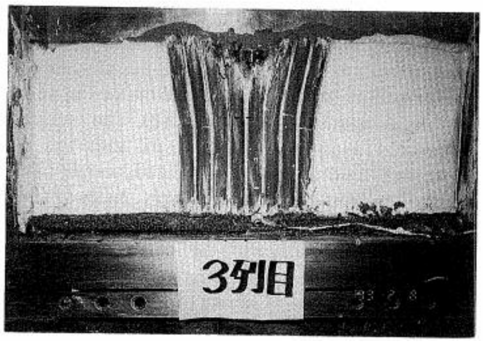
Finally, Terashi and Tanaka [10] concluded that the soil columns in the improved ground acted as foundation piles of low strength material having a bearing capacity expressed as per equation (2):

$$P_{ult} = a_s\,P_t + (1 - a_s)\,P_u = a_s\,P_t + (1 - a_s)\,N_c\,c_{uu} \qquad (2)$$

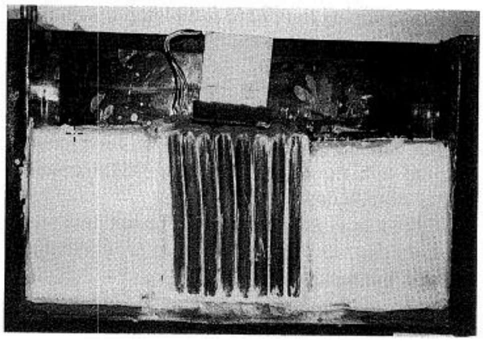
Where: the reaction of the column, (P_t) is decided by the smaller value between the bearing capacity and the compressive strength of the column, (N_c) is bearing capacity factor for shallow foundation on clay, (c_{uu}) is the cohesion of untreated soil.

Kitazume et al. [11] performed a series of centrifuge model (30g) tests to investigate the effects of various combinations of vertical and horizontal loads and different column strengths on the failure behavior of a prototype of a composite breakwater on fully penetrated columns with high improvement area ratio (79%). The column installation mechanism differs from the method implemented by Terashi and Tanaka [10], as they prepared the columns outside the ground model inside acrylic pipes of 20 mm inner diameter and 250 mm length. The columns mix was made from a mixture of Kawasaki clay and normal Portland cement, followed by pouring inside pipes and vibrating on a vibrating table for a few minutes. The columns were extracted after one week curing by an electric motor jack, then left for further weeks of curing again at room temperature in a wet condition. They reported that, a brittle behavior was obtained for vertical load tests matching the findings of Terashi and Tanaka [10]. They also concluded that, different failure modes were observed depending not only on the external load conditions but also on the location of each column. Fig. 5(a) shows combination of rupture breaking and several shear failures under columns just beneath the caisson due to vertical loading. Fig. 5(b), 5(c) and 5(d) show the failure patterns observed in the inclined loading tests which depends on column strength and vertical load component. Fig. 5(b) shows the rupturing failure mode of a low strength column (885 kPa) and relatively large normalized vertical load value where each segment of the failed columns remained straight. Contrary, when a column of high strength and a small applied vertical load, the entire column mass inclined towards the loading direction with some local failure as shown in Fig. 5(c). For extremely high strength column, the ground exhibits a collapse failure in which the entire column group inclined towards the loading direction without any rupture failure as shown in Fig. 5(d).

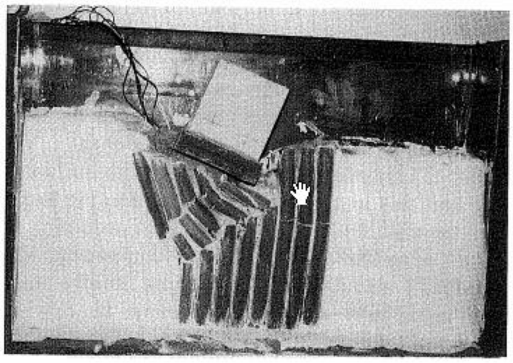
Bouassida and Porbaha [12-13] performed a (1g) modelling to analyze the ultimate bearing capacity of ground improved by fully penetrated columns using kinematic approach of the yield design theory. Only single improvement area ratio of (18.8%) was adopted with different column strengths. Kaolin clay was used as soft ground and a lower drainage layer of Toyoura sand. The installation technique of soil cement columns followed same procedures that carried out by Kitazume et al. [11] as columns were constructed and cured outside the soil. A schematic diagram of a typical scaled model of the improved ground is shown in Fig. 6.



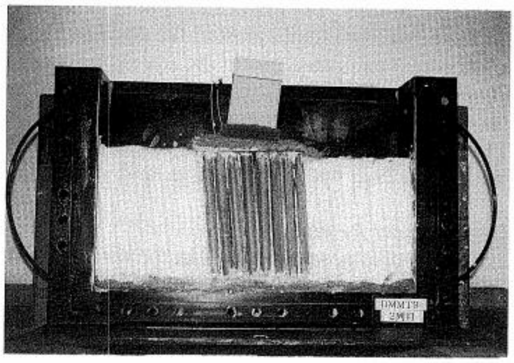
(a) Vertical loading case (quc = 758 kPa)



(b) Inclined loading on low strength columns case (quc = 885 kPa)

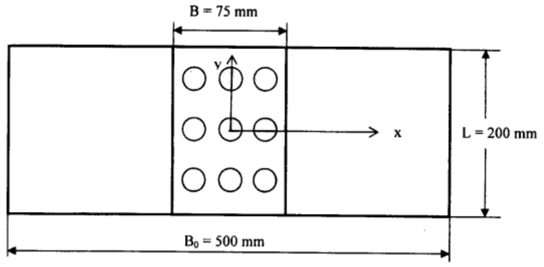


(c) Inclined loading on high strength columns case (quc = 2050 kPa)

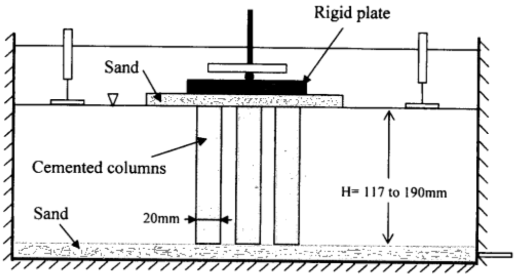


(d) Inclined loading on extremely high strength columns case (quc = 27200 kPa)

Fig. 5. Failure modes of deep mixing columns under vertical and horizontal loading [11]



(a) Plan



(b) Elevation

Fig. 6. A schematic diagram of a typical scaled model of the improved ground [12-13]

A kinematic approach based on yield design theory was used to validate the experimental results and comparing it with the one proposed by Broms method and others analyzed by the static approach of yield design theory as Bouassida et al. [14]. As per the results obtained by Bouassida et al. [14] the untreated soil and cement columns are assumed as purely cohesive materials having equal unit weight, then the bearing capacity factor bounded as:

$$\frac{Q^*}{c_{us}\,S} \geq 4 + 2\eta(K_c - 1) \qquad (3)$$

Where: (Q*) denotes the theoretical extreme vertical load at failure of the reinforced ground; (cus) is the cohesion of soft clay; (S) is the area of the footing; (η) is the improvement ratio area; (K_c) is the cohesion ratio of the column to the soil (c_{uc}/c_{us}).

In order to derive the bearing capacity factor upper bound, a failure mechanism of five blocks was considered, then the bearing capacity factor was bounded as:

$$\frac{Q^*}{c_{us}\,S} \leq 2\sqrt{2} + \sqrt{[1+\eta(K_c - 1)][2 + \eta(K_c - 1)]} \qquad (4)$$

Fig. 7 shows the relationship between the bearing capacity Factor (BCF) and the cohesion ratio (K_c), and it is obvious that when (K_c) increases, the theoretical upper bound values are clearly a little bit more than those recorded from experiments. Both equations (3 & 4) show better prediction for bearing capacity from laboratory results compared to Broms [15]. The relative difference between upper bound solutions and Broms approach was found to range from 10 to 14%.

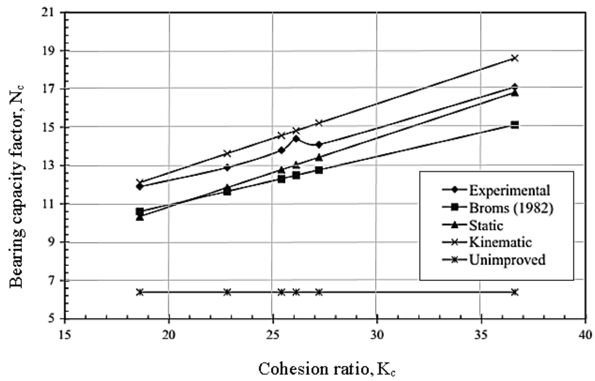
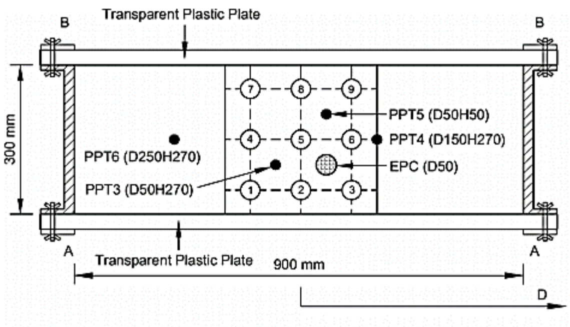
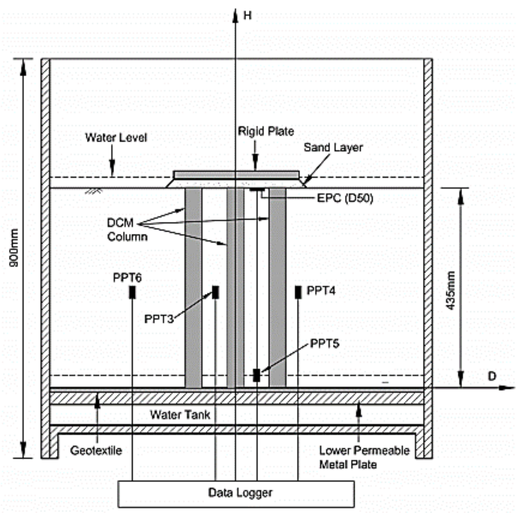


Fig. 7. BCF variation vs Kc ratio (modified after [12-13])

Yin and Fang [16] conducted a plane-strain physical modelling to investigate the bearing capacity and failure patterns of improved soil by a fully penetrated column group with relatively low replacement area ratio of (12.6%). Pore pressure transducers and earth pressure cells were utilized in order to record responses of pore water pressure and earth pressures in the soil at several locations as shown in Fig. 8. Hong Kong marine clay (HKMC) was used for preparing the soft soil ground and same procedures similar to those of Terashi and Tanaka [10] were adopted for column installation.



(a) Plan



(b) Cross section

Fig. 8. Model arrangement and instrumentation [16]

They reported that progressive softening behavior was observed for the model ground since the vertical pressure decrease gradually after the peak. Also, at the initial period of loading (up to approximately 1 min), larger pore water pressure in the soil are recorded due to the fact that the saturated soil shared more vertical foundation stress. While as time increases, more foundation pressure is shared by columns leading to a decrease in pore water pressure in the soil.

Yin and Fang [16], reported a simple “weighted method” and Broms method [15] which are suggested to calculate the bearing capacity of a rigid footing on the composite ground. The weighted method is expressed in the form of equation (5):

$$q_u = c_{uc}\alpha + c_{us}(1-\alpha) \tag{5}$$

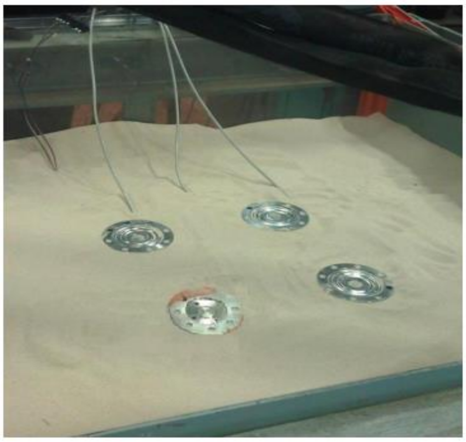
Where: (c_{uc}) and (c_{us}) denote the undrained shear strengths of the column and clay respectively and (α) is area replacement ratio. Meanwhile, Broms method is expressed as [12, 13, 15] in the form of equation (6):

$$q_u = 0.7 q_{uc}\alpha + \lambda\, c_{us}(1-\alpha) \tag{6}$$

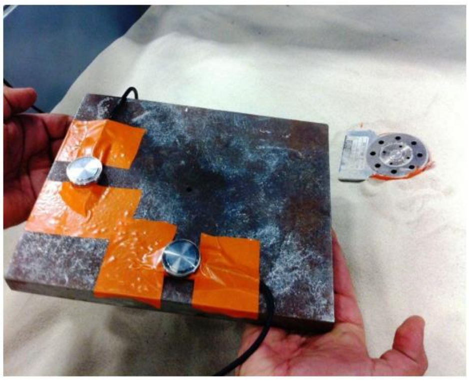
Where: (q_{uc}) is the unconfined compressive strength ($q_{uc}=2c_{uc}$), λ is a non-dimensional coefficient of a value of 5.5 as suggested by Bergado et al. [17]. Finally, Yin and Fang [16] concluded from comparing their results with the obtained values from these two equations and they found that their results match well with Broms method (equation 6) with a maximum relative difference of around 15%.

Dhaybi and Pellet [18] developed a reduced scale model in order to analyze the effect of soil mixing (SM) reinforcement on shallow foundations. The soil studied was Hostun dry sand in loose and dense state and the column installation is done by sinking a steel tube in soil and remove the sand inside the tube using vacuum. Then the soil-cement mixture is poured into the tube and cured for 7 and 14 days. Fig. 9 shows the test tank and its instrumentation which was accomplished by two stress sensors installed between the plate and the soil to measure the soil pressure, a small force sensor placed between the head of the column and the steel plate to measure the applied pressure on the column and three displacement sensors installed on the plate to check its horizontal.

(a) Test tank



(b) Force sensors



(c) Stress sensors

(d) Displacement sensors

Fig. 9. Test tank and instrumentation [18]

Fig. 10 shows results of loading tests carried on dense sand. Tests (1, 2 and 5) were repeated to verify the good reproducibility of the tests. It is noted that soil columns increased the bearing capacity of foundations and greatly reduces the extent of settlements. For loose soil, a sharp increase in vertical stress takes place at the beginning for a small amount of vertical displacement as shown in Fig. 11. This is due to the presence of a stiff element under the foundation which takes the most part of the applied load then the curve slopes become lower. Fig. 12 shows the load-settlement average results of all tests. They also reported that the trend of unreinforced case in dense sand is similar to the reinforced foundation in loose sand loaded at day 7 which shows the importance of the reinforcement since we could have the same behavior of a dense soil with a stiffer behavior at the beginning of vertical displacement. Difference between bearing capacity at 7 and 14 days in both cases (loose and dense sand) is around 23 %. They Attributed this difference to cement hydration which raises the column stiffness and ability to take higher load.

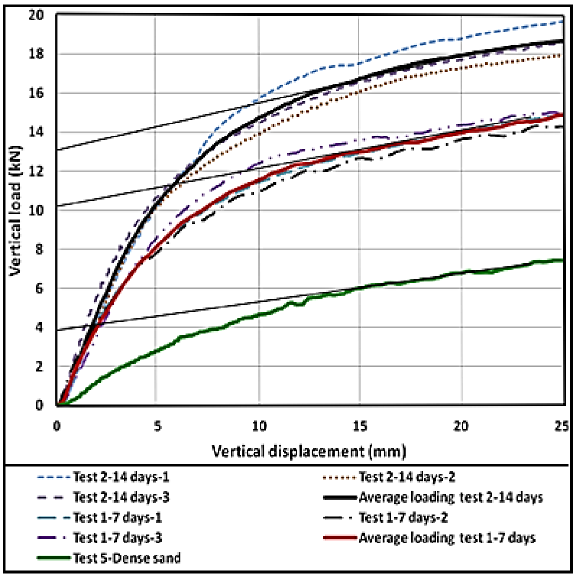


Fig. 10. Loading test results of dense sand-reinforced and unreinforced cases (modified after [18])

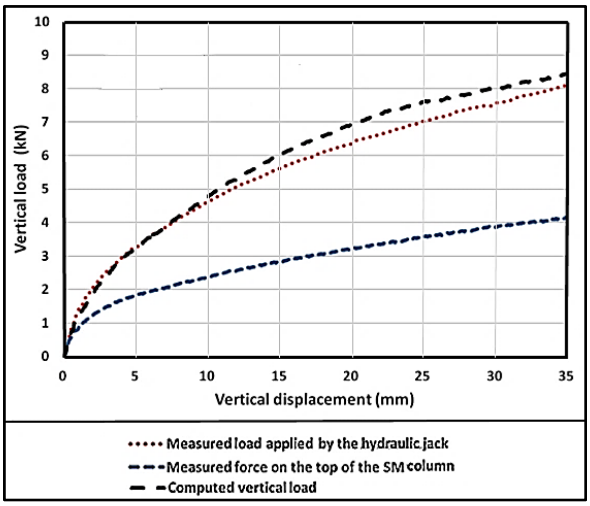


Fig. 11. Loading test result 3 - loose soil mass (modified after [18])

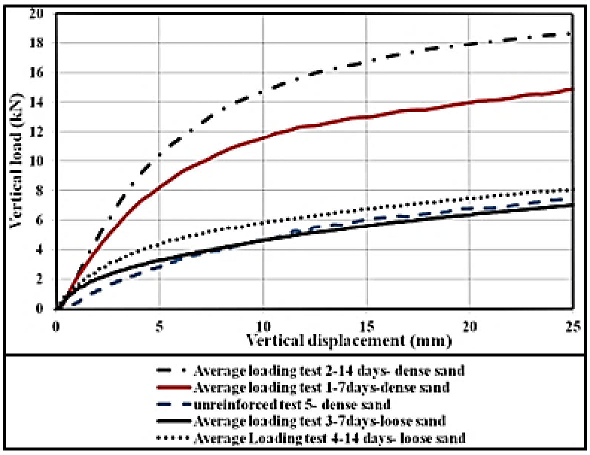


Fig. 12. Loading test results carried on dense sand, loose sand and unreinforced dense sand (modified after [18])

Banadaki et al. [19] conducted a total of 13 physical model tests to determine the ultimate vertical bearing capacity of peat soil improved by end bearing and floating cement deep mixing columns. Three area improvement ratios of 13.1, 19.6, and 26.2 % were considered and four length/depth ratios of 0.25, 0.5, 0.75 and 1.0. The geometry of the test setup and the configuration of the DM columns are shown in Fig. 13. The continuous replacement method was considered in preparing soil columns in which columns were installed and cured inside the soil unlikely the method conducted by Yin and Fang [16] where columns were constructed and cured out of the soil model.

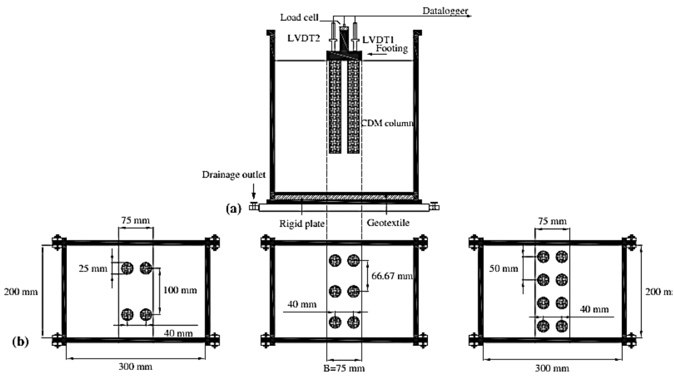


Fig. 13. A schematic diagram of test model, (a) Test setup (b) Column configuration [19]

They defined the ultimate bearing capacity of the reinforced soil by a dimensionless ratio of bearing capacity factor (BCF) expressed as:

$$BCF=\frac{q_u}{c_{us}} \quad (7)$$

Where: (q_u) is the ultimate bearing capacity of stabilized peat and (c_{us}) is the undrained shear strength of soil.

For the end-bearing columns, the ultimate bearing capacity increased up to 200, 229 and 240 % using area improvement ratios of 13.1, 19.6, and 26.2 %

respectively compared to unimproved peat. While for floating columns, it had an average increase of 60% approximately.

Fig. 14 compares the bearing capacity factor (BCF) obtained from experimental results with different analytical methods in case of end bearing columns. It was clearly noted that with the increase of the area improvement ratio, the (BCF) of stabilized soil increases which was attributed to higher stiffness of soil using end-bearing columns. For floating columns, the results of the UBC were compatible with Broms method showing insignificant relative difference between them as shown in Fig. 15.

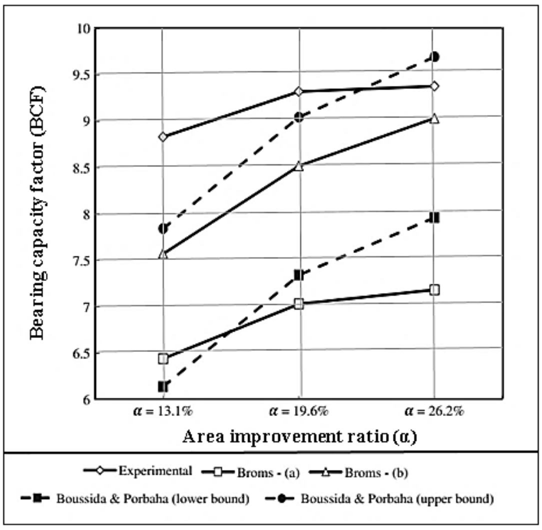


Fig. 14. BCF of peat improved by end bearing columns soil (modified after [19])

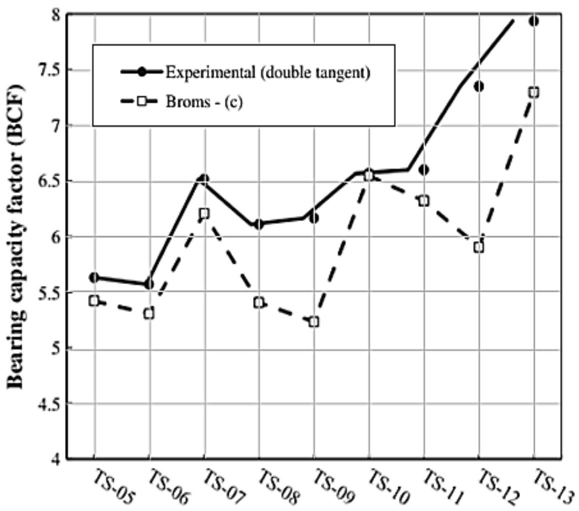
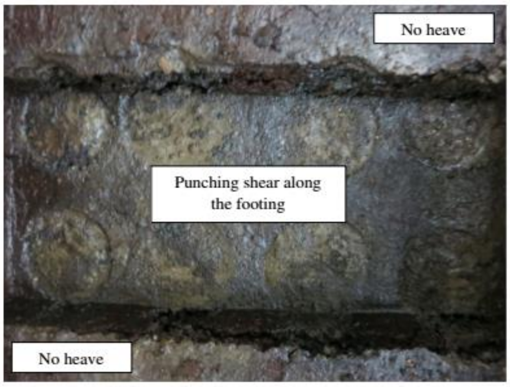


Fig. 15. BCF of peat improved by floating columns soil (modified after [19])

They observed a punching shear in the case of floating columns, while in the case of end-bearing columns, progressive cracks and small heave were observed around the footing as shown in Fig. 16.

(a) End bearing columns



(b) Floating columns

Fig. 16. Failure of peat improved by mixing columns [19]

Ahsan et al. [20] carried out a small-scale test to study the effect of cement column in improving soft soil. Natural soil clay sample was collected from KUET in order to form the clay bed. They used a hand rotating mixing device to install a 6" cement column as shown in Fig. 17. The test model was tested in “Universal Testing Machine” to record the load-settlement curve as shown in Fig. 18. The results indicate that the bearing capacity of soft soil is increased significantly by using a single cement column. It was observed that the load intensity value of the cement column is 822.28 kPa with a relatively very low cement content (7%) of volume of soil in cement column, whereas in case of without cement column it is only 172.38 kPa as shown in Fig. 19.

Fig. 17. Mixing device for installation of cement column [20]

Fig. 18. Universal testing machine [20]

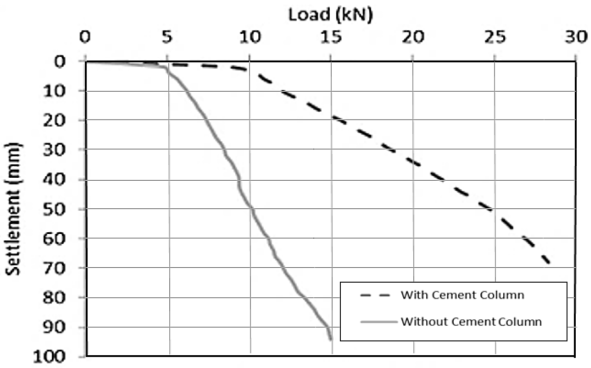
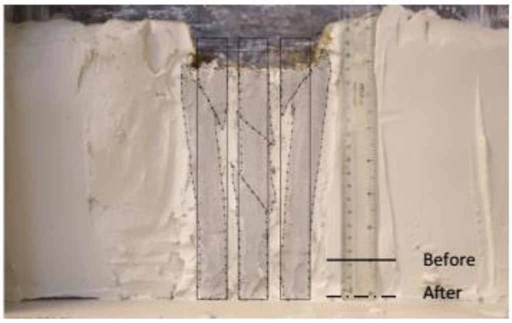


Fig. 19. Load settlement curve (modified after [20])

Rashid et al. [21] carried out a series of (1g) physical model tests to investigate the failure behavior of improved ground by fully and partially penetrating soil columns. Two types of loading were applied, rigid and flexible footings. A single improvement area ratio was used of a value of (26%). Speswhite kaolin clay was adopted for preparing the clay soil model, and the same procedures carried out by Kitazume et al. [11] for the installation of columns inside the soil model was followed. Particle image velocimetry (PIV) and close-range photogrammetry were used to measure the soil displacements and monitor the mechanism pattern under the failure load. In order to validate the bearing capacity results, a simulation of the model test using Limit-State: GEO software was used.

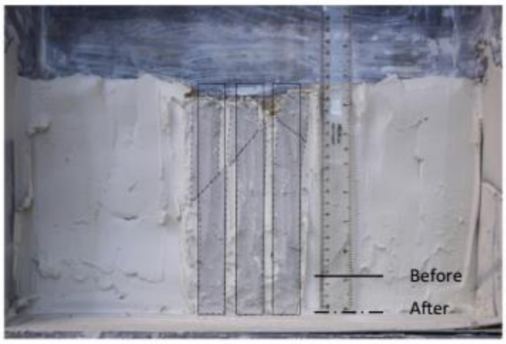
It was clearly noted that the unimproved soil ground either with a rigid or a flexible footing possessed a ductile behavior. While for the full penetration case, the improved ground showed a brittle behavior which was attributed to the differences in strength between the soil and the column and the end-bearing effect. For the partial penetrating soil columns, they observed that the composite ground possesses a ductile behavior, which could be attributed that the group of columns was bearing on soft ground.

A significant improvement of bearing capacity was noticed in the full and partial penetration cases reaching 168% and 80% respectively, compared to the bearing capacity in the untreated ground. In the full penetration case, a combination of shear and bending failure were observed while a bending failure only was observed in the partial penetration case as shown in Fig. 20.

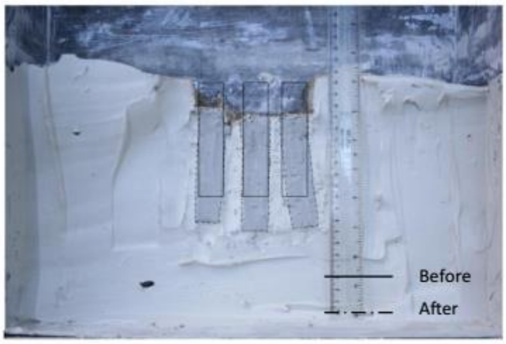


(a) Rigid footing with fully penetrating columns

(b) Rigid footing with partially penetrating columns



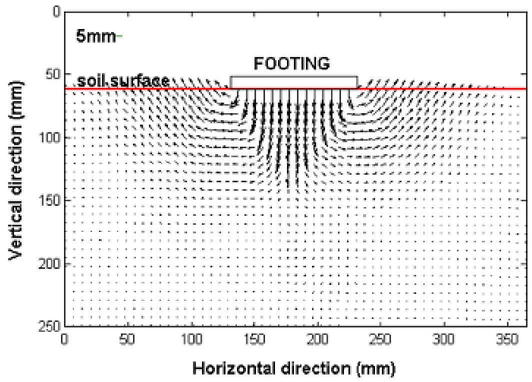
(c) Flexible footing with fully penetrating columns



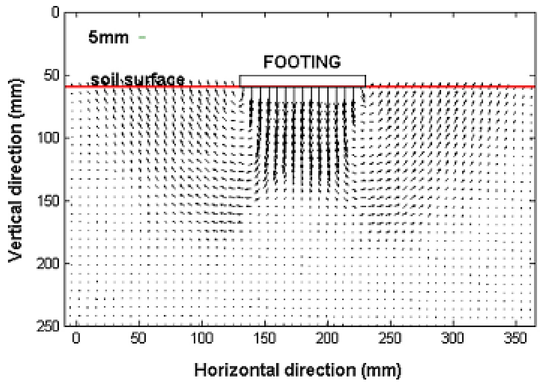
(d) Flexible footing with partially penetrating columns

Fig. 20. Failure patterns of fully and partially penetrating columns under vertical loading [21]

From the PIV analysis, it was observed that the failure wedge of the footing for the untreated and fully penetrating case extended to approximately 100% of the footing width and 150% for the partially penetrated columns as shown in Fig. 21. Also, considerable differences between the failure patterns produced by Limit-State: GEO compared to the PIV analysis due to the chosen material model, the type of loading required to develop the failure mechanism, and the 3D column movement away from the viewing window.



(a)Rigid footing with fully penetrating columns



(b) Rigid footing with partially penetrating columns

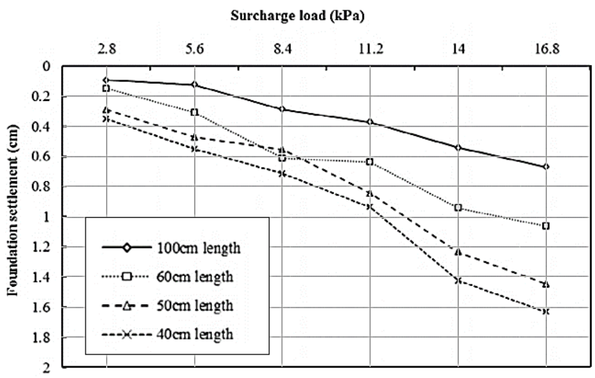
Fig. 21. Ground deformation produced by PIV analysis [21]

Yao et al. [22] conducted a physical model test to investigate the foundation settlement of soft soil (silt) improved by deep mixed columns. The studied parameters were column length (40, 50, 60 and 100 cm) and area replacement ratio (2.3, 5.3 and 9.3%). The cement mixture was put into the PVC mold layer by layer then the mold was lifted step by step with the filling of cement mixture as shown in Fig. 22. They reported that the foundation settlement decreased as longer column was used for all area replacement ratio as shown in Fig. 23(a). Also, the foundation settlement was significantly enhanced as the area replacement ratio increased from 2.3% 9.3% with same column length and surcharge load as shown in Fig. 23(b).

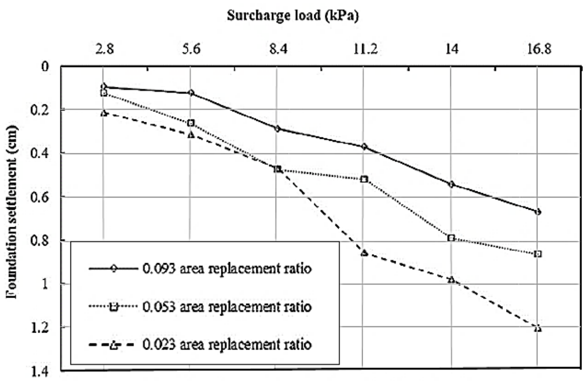
(a) Columns installation

(b) Loading procedures

Fig. 22. Experimental testing procedures [22]



(a) Area replacement ratio = 9.3%



(b) Column length = 100 cm

Fig. 23. Foundation settlement-load curve (modified after [22])

III. Conclusion

This paper sheds light on the studies done by numerous researchers on using the deep soil mixing technique which can be used as a successful method of weak soil improvement. From these studies, it is obvious that the deep mixing method provides a better alternative technique to the traditional methods of weak soil improvement in terms of applicability for use in different types of soils and the resulted high ground strength. The ultimate bearing capacity of the improved ground has been investigated in several experimental studies based on two analytical analysis including Broms and weighted method which mainly depend on the strength performance of the mixing columns. Most researchers have studied the ultimate bearing capacity of fully penetrating deep mixing columns. While a few other researchers have studied the ultimate bearing capacity of the deep mixing columns under the partially penetrating condition. Regarding the failure mode of the improved ground with deep soil mixing columns, it was reported by many investigators that it depends on the column strength and the vertical applied load. In this respect, a good attempt was done by (Rashid et al., 2015) using (PIV) and close-range photogrammetry despite of the differences compared with the results obtained by GEO software. So, it may be recommended to use a 3D numerical modelling or a transparent soil physical model for better observing and understanding the mode of failure in the case of using deep soil mixing columns technique as a method of weak soil improvement.

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