The 2014 World Congress on
Advances in Civil, Environmental, and Materials Research (ACEM14)
Busan, Korea, August 24-28, 2014

Strength of Soft Clay Reinforced with Single and Group Bottom Ash Columns

*Muzamir Hasan[1)], Aminaton Marto[2)] and Masayuki Hyodo[3)]

[1)] Faculty of Civil Engineering and Earth Resources, Universiti Malaysia Pahang, Lebuhraya Tun Razak, 26300 Gambang, Kuantan Malaysia
[2)] Faculty of Civil Engineering, Universiti Teknologi Malaysia, Skudai, 81310 Johor, Malaysia
[3)] Department of Civil and Environmental Engineering, Yamaguchi University, Tokidawai 2-16-1, Ube, 755-8611, Japan

[1)] muzamir@ump.edu.my
[2)] aminaton@utm.my
[3)] hyodo@yamaguchi-u.ac.jp

ABSTRACT

Stone column could be implemented as a ground improvement technique where a portion of the soil is substituted with granular material such as sand or crushed rocks which is proven to improve bearing capacity and accelerate the dissipation of pore water pressure. This research was aimed to investigate the role of bottom ash columns in improving the shear strength soft reconstituted kaolin clay. This was done by determining the effect of area replacement ratio, height penetrating ratio and volume replacement ratio of a single bottom ash columns on the strength of kaolin clay reinforced with bottom ash column(s). The reinforced kaolin samples were tested using Unconfined Compression Test (UCT). Research variables include diameter and height of the bottom ash columns. Through the results of UCT, it can be concluded that the S_u generally increased with the increased in the height penetrating ratio but decreased after reaching an optimum improvement at 80 % of height penetrating ratio. The increment of S_u was also dependent on the area replacement ratio of bottom ash. However, excessive area replacement decreased the shear strength of the sample reinforced by group columns since the remaining width of the soil sample will be too thin to hold the columns. Generally it can be concluded that the shear strength of soft clay could be improved by the installation of bottom ash columns.

1. INTRODUCTION

The uncontrollable usage of non-renewable natural material and the production of

[1)] Senior Lecturer, Ph.D.
[2), 3)] Professor, Ph.D.

waste from human activities imposed negative impact to the earth. The main key to solve these problems is by utilization of waste in the construction industry. According to Muhardi et al. (2010), the power plant companies posed social and environmental problems to the society because of the magnification of disposal areas and the increased disposal expenses to accommodate the by-products that are transferred to end users. Because of that, the utilization of coal ash in construction industry is potentially the solution for the disposal problem of the material. Bottom ash formed the largest component of coal ash. Kumar and Stewart (2003) found out that the properties of bottom ash are almost similar to those of sand. By that, there is a good potential of utilizing this bottom ash as a substitute material to stone columns. By introducing Bottom Ash Column (BAC), the cost of a construction project will significantly reduce as well as the disposal area for bottom ash.

Construction on problematic soil such as soft clay requires ground improvement or modification to improve its mechanical properties. Stone columns could be used as a method where a portion of the soil is being replaced with granular material such as crushed rocks or sand. Stone columns are installed in soft cohesive soil because they can improve the bearing capacity, reduce settlement, and accelerate the dissipation of pore water pressure.

Previous research on soft clay improvement includes the use of single and group sand columns tested in laboratory or field. This includes installing single and group columns (up to four columns) in clays and loaded using either top plates of typical triaxial cells or model foundations. Earlier work done by Hughes and Withers (1974) proved to be very important in utilizing the small-scale laboratory tests to understand the behaviour of single stone column installed in soft clay. Sivakumar et al. (2004) and Ambily and Gandhi (2007) also had studied the behaviour of single columns installed in soft soil through small-scale laboratory tests. However, they used sand as the columns. Hence it is wise to refer to the stone and sand columns as granular columns.

Najjar et al. (2010) reported that most researchers conducted their tests in large one-dimensional loading chambers where there is no control of drainage in the specimens during the loading process. Usually the outputs from the tests are limited to improvements in the load carrying capacity of sand column or the clay–sand column hybrid system. Muir Wood et al. (2000), McKelvey et al. (2004), Black et al. (2006, 2007), Najjar et al.(2010), Sivakumar et al. (2004) and Ambily and Gandhi (2007) conducted tests on both single and group granular columns to investigate bearing capacity improvement of clay. Muir Wood et al. (2000) varied the diameter, length, and spacing of stone columns to study the transferred load to the surrounding clay. They used exhumation technique to study the deformed shapes of the columns. Sivakumar et al.(2000), Black et al. (2006), Juran and Riccobono (2001) conducted tests under full triaxial conditions where the loading rate and drainage were controlled during shear. McKelvey et al. (2002), McKelvey (2004) performed tests using transparent ‘clay-like’ material so that any deformation on the specimen and also the columns could be observed closely.

Hasan et al. (2011) initiated the study using bottom ash as a substitute material in stone columns in which series of Unconfined Compression Tests were conducted on kaolin clay reinforced with single and group bottom ash columns (BAC) to determine the short-term improvement of soft clay. The area replacement ratio (area of columns/area of clay specimen) used was 4 % (single column) and 16 % (group of 4 columns) while the height penetrating ratio (height of column/height of clay specimen) used was 0.6 and 1.0. They found that the undrained shear strength of the clay generally improved by the installation of BAC. However, for the clay installed with single column, the partially penetrating column gives higher improvement in the undrained shear strength than the fully penetrating column.

Black et al. (2006) stated that clay samples reinforced with an isolated, fully penetrating column showed strength increases of 33 % compared with sample without column. Maakaroun et al. (2009) found that for fully penetrating columns, the increase in undrained shear strength ranged from 13 to 19.5 % and from 67.5 to 75 % for area replacement ratios of 7.9 and 17.8 %, respectively.

In a number of past studies, sand columns of different lengths were used to investigate the effect of the column penetration on the improvement of load-carrying capacity of the specimens. Many of the researchers have come up with the 'critical column length' idea where the column beyond this length will not improve the capacity of the clay. The value for 'critical column length' as proposed by Hughes and Withers (1974), Muir Wood et al. (2000), McKelvey et al. (2002), McKelvey (2004), Narasimha Rao et al. (1992) occurred between 4 and 8 times the diameter of the column.

The study by Black et al. (2006) concluded that for fully penetrating sand columns, the insertion of sand columns increases the load carrying capacity of the soft clays, reduces settlement, and decreases the generation of excess pore-water pressure during undrained loading. The results also showed that the improvement was dependent on the undrained shear strength of the clay, angle of internal friction of the column material, and the geometric characteristics (diameter and spacing) of the sand columns. Generally it has been shown that the relative increase in strength due to the presence of sand columns is independent of the column configuration (no effect by group column) and is only dependent on area replacement ratio of the column.

The aim of the current research is to determine the suitability of bottom ash as material in granular column for improving the strength of soft clay. This paper discusses the results of the improvement in shear strength to the soft clay reinforced with single and group BAC.

2. 2 MATERIAL CHARACTERISTICS AND PREPARATION METHODS

2.1 Material Characteristics

Bottom ash used for this study was collected from Tanjung Bin Power Plant in Johor, Malaysia. For the clay, commercially available kaolin of Grade S300 was used to produce repeatable homogenous soft clay samples. This kaolin that possesses platy structure is hydrophilic and ready to be mixed with water to form slurry for producing homogeneous soft clay. Table1shows the basic and mechanical properties of bottom ash and kaolin used in this study.

Table 1 Basic and mechanical properties of bottom ash and kaolin

Material	Test	Parameter	Value
Bottom Ash	Soil Classification	AASHTO	A-1-a (0)
	Standard 'Light' Compaction	Maximum Dry Density Optimum Moisture Content	1.34 Mg/m^3 23.5 %
	Relative Density	Maximum Index Density Minimum Index Density Relative Density	0.98 Mg/m^3 0.78 Mg/m^3 13.31%
	Small Pycnometer	Specific Gravity	2.35
	Constant Head Permeability	Coefficient of Permeability	1.59 x 10^{-3} m/s
	Direct Shear	Cohesion Angle of Shear Resistance	6.6 kPa 38 °
Kaolin	Soil Classification	AASHTO USCS (Plasticity Chart)	A-6 (4) ML
	Atterberg Limit	Liquid Limit Plastic Limit Plasticity Index Shrinkage Limit	37 % 26 % 11 % 27 %
	Standard 'Light' Compaction	Maximum Dry Density Optimum Moisture Content	1.60 Mg/m^3 20 %
	Specific Gravity	Specific Gravity	2.65
	Falling Head Permeability	Coefficient of Permeability	3.4 x 10^{-10} m/s
	Vane Shear	Undrained Shear Strength	7 kPa

2.2 Sample Preparation

The soft clay was prepared using customised compaction method and the BAC had been installed in the soft clay using the replacement method.

2.2.1 Soft Clay Preparation

The kaolin was air dried and then mixed with 20 % of water which is the optimum moisture content of the kaolin obtained from standard compaction test. After uniform mixing of the soil, it was poured into the customized steel mould and compacted in three layers. Each layer had been compacted with five free fall blows of a 3.1 kg customized steel hammer. The customized mould was designed so that the amount of clay using inside it will be compressed into a 50 mm diameter and 100 mm high specimen. By using this procedure, the uniformity of each specimen could be maintained since the mass of the soil and the volume of the mould are almost the same although it is believed that there could be some minor losses in the mass during the process. The specimens were then extruded out from the mould and stored in a special case and left for at least 24 hours to stabilize the pore pressure inside the specimen before initiating the Unconfined Compression Test.

2.2.2 Installation of Bottom Ash Columns

In preparing for the installation of BAC for the reinforced specimens, the holes for the installation of BAC were drilled using drill bit of respective diameter with the specimens still inside the mould to prevent it from expanding (as shown in Fig.6). The bottom ash was installed in the holes to achieve relative density of 13.31 %.

Since the specimen is soft and sensitive, the process of installation and densification of the bottom ash was very challenging. Through the results of several pilot tests, it was decided that the raining method was the best way to create homogeneous BAC in the clay specimens. The bottom ash was densified by pouring it into the pre-drilled hole by free fall from a predetermined height. The falling height in the raining method was fixed at about 10 mm from the tube to the surface of the clay specimens.

In order to maintain a uniform density in each BAC, the mass of bottom ash used to fill the pre-drilled hole had been based on the volume of pre-drilled hole (as shown in Table 2). By referring to this method, the same density of 8.15×10^{-4} g/mm^3 had been produced for every specimen in this study.

Table 2 Detail on Densification Process for Installing Bottom Ash Columns in Kaolin Specimens

Column Diameter (mm)	**Column Length (mm)**	**Volume (mm^3)**	**Mass of Bottom Ash (g)**	**Density (g/mm^3)**
10	60	4712.39	3.84	8.15×10^{-4}
	80	6283.19	5.12	
	100	7853.98	6.50	
16	60	10602.88	9.83	
	80	16084.95	13.11	
	100	17671.46	16.38	

2.2.3 Pattern and Size

For single columns, the columns were installed at the centre of the specimens; while for group of columns, the triangular pattern was chosen as it was much easier to maintain the location of the columns to be installed especially in terms of spacing in between the columns. The spacing in between the columns was chosen by evaluating the area ratio and also the ratio of the column area to the overall clay area. This is by locating the columns to be in the middle between the geometric centers of the kaolin specimens to its boundary in order to transfer load evenly to each column.

The diameter of the column (D) and the particle size of granular material (d) play an important role in choosing the appropriate size of the column to be used in model tests. According to Muir Wood et al. [9], it is desirable to have a ratio of D/d in model tests to be similar to that found in the prototype structures being modelled. They used D/d between 52 to 83. In practice, stone columns are formed at typical diameters, D = 0.6 to 1 m of crushed rock or gravel with typical particle size, d = 25 to 50 mm, so that it makes the ratio D/d to be in between 12 and 40. The column diameters used in this study were 10 and 16 mm, while the particle sizes of bottom ash were between 0.6 to 2.36 mm. The ratio D/d in the model tests therefore had the values between 4 to 17. Although the values for lower range of model tests were slightly smaller than those typical in practice, it is unavoidable as there is a limitation on the diameter of the column to be used to avoid boundary effects. Fig. 1 shows the specimens already installed with single and group bottom ash columns, respectively.

(a) Single column (b) Group columns

Fig. 1 Clay specimen reinforced with (a) single and (b) group bottom ash columns

The diameter of bottom ash columns used in this study was varied from 10 to 16 mm. As a result, the area ratio, defined as the area of the column to the area of the specimen (A_c/A_s) was 4 and 10.24 % respectively. As for group column, A_c/A_s was 12 and 30.72 %, for the 10 mm and 16 mm columns, respectively. The height penetration ratio, defined as the ratio of the height of the column to the height of the specimen (H_c/H_s), was varied from 0.6, 0.8 and 1.0. Fig. 2 illustrates the arrangement of single and group columns of 10 and 16 mm diameters, respectively in the soil specimens.

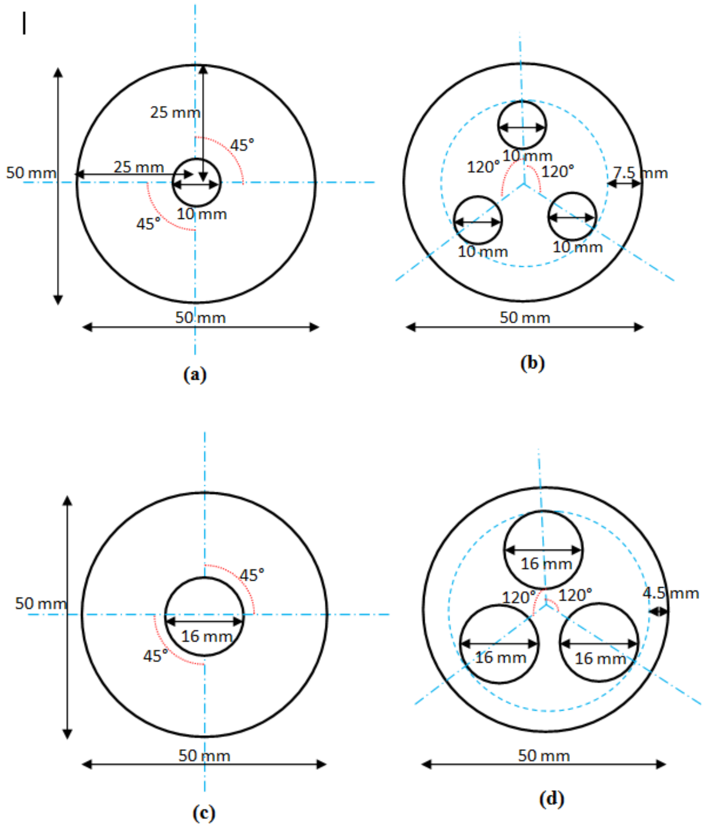

Fig. 2 Detail column arrangement for single and group bottom ash columns installed in clay specimens

3. SHEAR STRENGTH ANALYSIS

To determine the shear strength for soft clay samples reinforced with bottom ash columns, the Unconfined Compression tests were conducted. The summary of the results from the tests are shown in Table 3. Four sets of test were done on every penetration ratio to obtain the average values of the shear strength. Besides, sample without any reinforcement of bottom ash columns was used as controlled sample. Thus, a total of fifty-two (52) samples were prepared and tested.

Table 3 Results from Unconfined Compression test

Sample	Number of Columns	Column Diameter (mm)	Area Ratio, A_c/A_s (%)	Column Height (mm)	Column Height Penetrating Ratio H_c/H_s	Average Shear Strength (kPa)	Improvement of Shear Strength (%)
Controlled Sample							
C	0	0	0	0	0	6.52	-
Singular Columns							
S1060	1	10	4.00	60	0.6	9.11	39.72
S1080	1	10	4.00	80	0.8	17.39	166.72
S10100	1	10	4.00	100	1.0	10.70	64.11
S1660	1	16	10.24	60	0.6	8.11	24.39
S1680	1	16	10.24	80	0.8	16.30	150.00
S16100	1	16	10.24	100	1.0	10.51	61.20
Group Columns							
G1060	3	10	12.00	60	0.6	9.51	45.86
G1080	3	10	12.00	80	0.8	15.26	134.05
G10100	3	10	12.00	100	1.0	10.56	61.96
G1660	3	16	30.72	60	0.6	7.91	21.32
G1680	3	16	30.72	80	0.8	10.31	58.13
G16100	3	16	30.72	100	1.0	7.46	14.42

For the controlled sample which is the sample without columns, the average shear strength is 6.52 kPa. Meanwhile, for clay samples reinforced with single 10 mm diameter bottom ash column with 60 % height penetrating column, the average shear strength is 9.11 kPa, which indicates an increase of 39.72 %. As for height penetrating ratio of 80 %, for single 10 mm diameter column the average shear strength is 17.39 kPa which brings the improvement to 166.72 %. For fully penetrating column, the average shear strength is 10.70 kPa which makes the improvement to 64.11 %.

Meanwhile, for single 16 mm diameter columns with height penetrating ratio of 60 %, the average shear strength is 8.11 kPa which brings the strength improvement of 24.39 % compared to that of the controlled sample. As for the column with height penetrating ratio of 80 %, the average shear strength indicates the value of 16.3 kPa as it improves 150 %. For fully penetrating column, the average shear strength is 10.51 kPa which makes the improvement of shear strength to be 61.20 %.

In both cases for single column, the trends for different height penetrating ratio for both different column diameters are almost similar whereas the differences are only minor between the two. The average shear strengths increased until it reached the height penetrating ratio of 80 % and then it decreased when the height penetrating ratio is 100 %.

For clay samples reinforced with three bottom ash columns in group, the columns were arranged in triangular pattern. For 10 mm diameter bottom ash column with 60 % height penetrating column, the average shear strength is 9.51 kPa, which indicates an increase of 45.86 %. As for the height penetrating ratio of 80 %, for group 10 mm diameter column, the average shear strength is 15.26 kPa which brings the improvement to 134.05 %. For fully penetrating column, the average shear strength is 10.56 which makes the improvement to 61.96 %.

For group of three 16 mm diameter columns with the height penetrating ratio of 60 %, the average shear strength is 7.91 kPa which brings the strength improvement of 21.32 % compared to the controlled sample. As for the column with height penetrating ratio of 80 %, the average shear strength value is 10.31 kPa as it improves 58.13 %. For fully penetrating column, the average shear strength is 7.46 kPa which makes the improvement of shear strength to be 14.42 %.

Table 4 shows the values of the maximum deviator stress, qu and axial strain for the controlled sample and specimens reinforced with singular and group of three bottom ash columns that had been tested under unconfined compression test. For both single and group columns, it is found out that the undrained shear strength increased after the specimens had been reinforced by a group of three bottom ash columns. The result also indicates that for both single and group of bottom ash columns, the reinforcement increases the strength of the specimens. The results implicate that the axial stiffness of the reinforced specimens had insignificant increments.

Table 4 Peak deviator stress and axial strain from UCT

Area Replacement Ratio, A_c/A_s	**Height Penetrating Ratio, H_c/H_s (%)**	**Unconfined Compressive Stress, q_u (kPa)**	**Average Axial Strain (%)**
0	0	13.04	1.35
4.00	60	18.23	1.84
	80	34.78	1.83
	100	21.40	1.86
10.24	60	16.23	1.79
	80	32.60	1.86
	100	21.03	1.86

3.1The Effect of Area Replacement Ratio

The graph of shear strength versus area replacement ratio, A_c/A_s is shown in Figure 3. Results indicated that the shear strength decreases as the diameter of the bottom ash column increases. For the singular bottom ash column, the shear strength for area replacement ratio is 4 % greater compared to 10.24 % area replacement value. This result contradicted the results done by Najjar et al. (2010) and Black et al. (2007). The effects of area replacement ratio on undrained shear strength from previous and current study are shown Table 5.

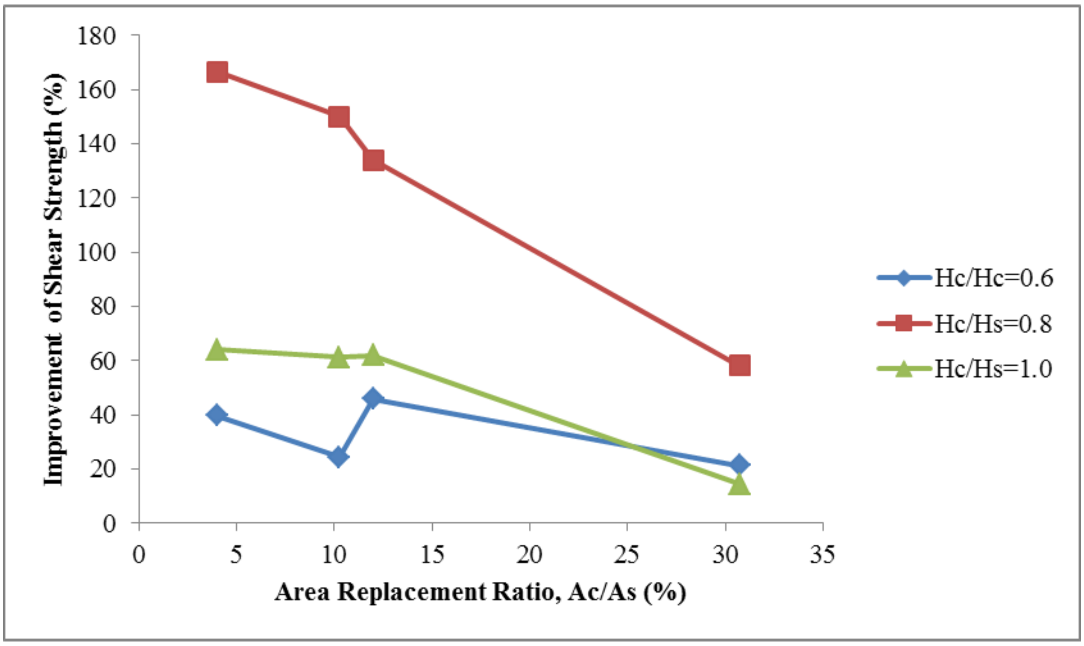

Fig. 3 Shear strength versus area replacement ratio

Table 5 Effect of area replacement ratio for fully penetrating columns on undrained shear strength

Researcher	Area Replacement Ratio, A_c/A_s (%)	Undrained Shear Strength Improvement (%)
Najjar *et al*. (2010)	7.90 17.80	19.50 75.00
Black *et al*. (2007)	10.00 12.00	33.00 55.00
Current study (2013)	4.00 10.27	64.11 61.20

	12.00	61.96
	30.72	14.42

However, the result is in line with the previous study done by Tandel et al. (2012) and Murugesan et al. (2010) where the reason behind the situation is that the decreased performance is due to the mobilization of higher confining stresses in smaller bottom ash column. The higher confining stresses in the column are, the higher the stiffness of smaller diameter.

Besides, based on the results, it shows that the group columns with 12 % and 30.72 % of area replacement ratios have smaller improvement of shear strength compared to the single columns with the area replacement ratios of 4 % and 10.24 %. This is due to the fact that the area replacement ratio of column is too big. When the vertical load is distributed into the columns, the columns became bulged as the remaining width of the soil sample was too thin to hold the columns. In addition, Kempfert and Gebreselassie (2007) stated that the decreased performance in shear strength is due to the columns in a group having an axis symmetrical influence zone (cylindrical in shape). In other words, the decreased performance in shear strength is due to the overlapping of influence zone.

The result also suggests that by drilling out large portion of soil from the samples before the bottom ash columns were installed could affect the natural state of the soil while decreasing the initial shear strength of the soil itself. Since there are constraints due to the size of the samples which had been scaled down, this suggests that it played an important part in the shear strength of the soils.

3.2Effect of Column Penetration Ratio

To investigate the possible influence of height over diameter of column ratio to undrained shear strength, the improvement of undrained shear strength was plotted versus the height over diameter of column ratio in Fig. 4. For comparison, data by Maakaroun et al. (2010) was plotted on the same figure. As proposed by past researchers, “the critical column length” which is between 4 to 8 times the diameters of the column (D_c) was marked as the grey area in the figure.

The results are in accordance with the hypothesis of a critical column length beyond which the increase in undrained shear strength becomes relatively negligible. The highest increase in each different area ratio was within the grey area. Generally, there were higher increases in strength that were spotted for single columns compared to group columns. For area ratios of 4 % and 12 %, the highest improvements of undrained shear strength were when it reached 8Dc. As for the area ratios of 10.24 % and 30.76 %, the maximum improvement was achieved at $5D_c$.

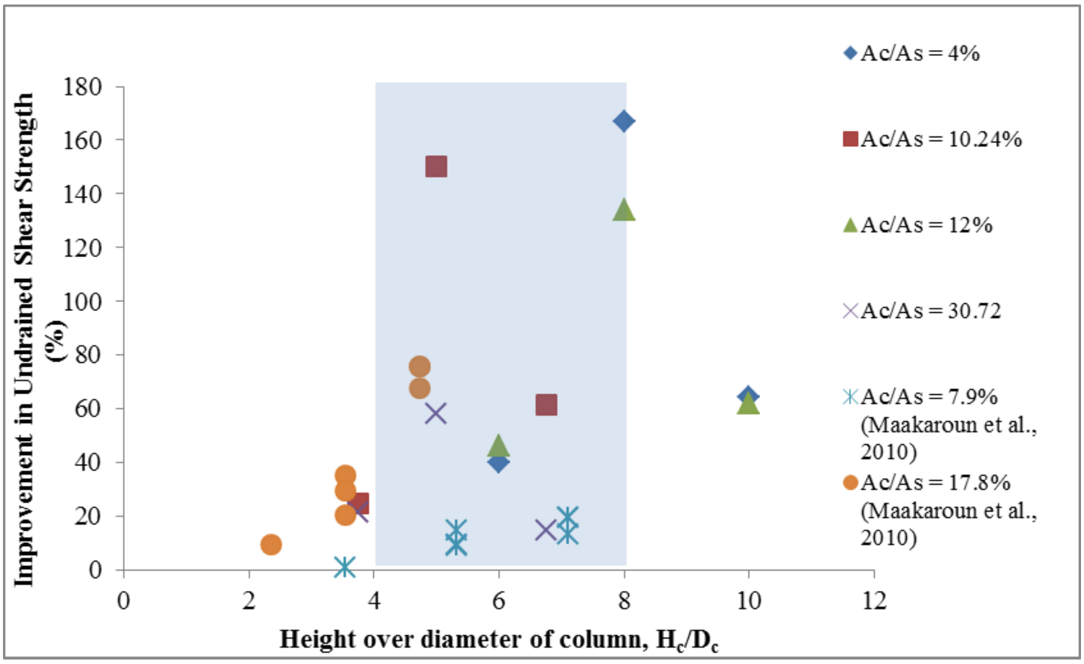

Fig. 4 Effect of ratio of column height to diameter on undrained shear strength

The improvement of shear strength by height penetrating ratio for single and group bottom ash columns by unconfined compression test is shown in Fig. 5. There was consistency detected in height penetrating ratio (H_c/H_s) as the highest improvement was shown when H_c/H_s = 0.8 for all the samples. The results may suggest that the height penetrating ratio might play more significant role in improving the undrained shear strength of the clay soil compared to the height over diameter of the column ratio.

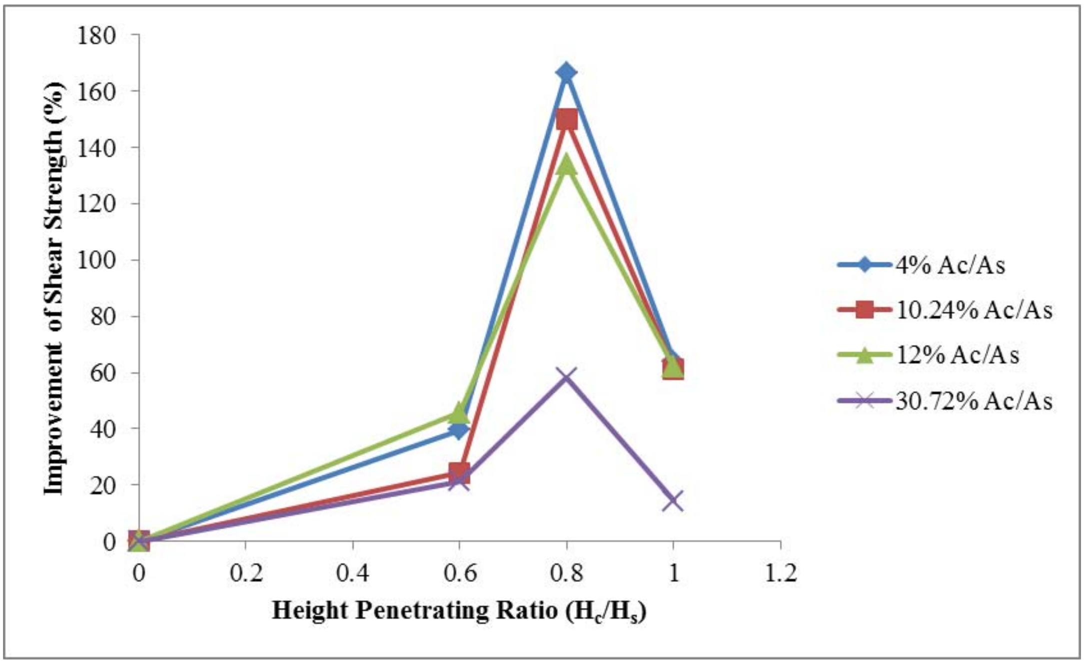

Fig. 5 The improvement of undrained shear strength by height penetrating ratio for single and group bottom ash columns

According to Muir Wood et al. (2000), the interactions between the individual stone columns, the loaded area and the surrounding soil can be understood as the behaviour of `piles' with non-linear, sand-like axial stiffness properties. If a column is sufficiently short for significant load to be transmitted to the base of the column then it will penetrate the underlying clay. As the column length increases, the penetration reduces because fewer loads pass to the base of the columns. This mode can be directly compared with the behaviour of the sand column as a pile.

Axially, like compressible piles (Fleming et al., 1985), beyond a certain length no further load can reach the base of the pile or stone column because it has all been shed through shaft friction: this has been observed in the physical models. The difference from a simple pile analogy is that the radial restraint from the surrounding soil is important in its influence on the mobilization of shear strength within the pile or column. The condition was illustrated in Fig. 6.

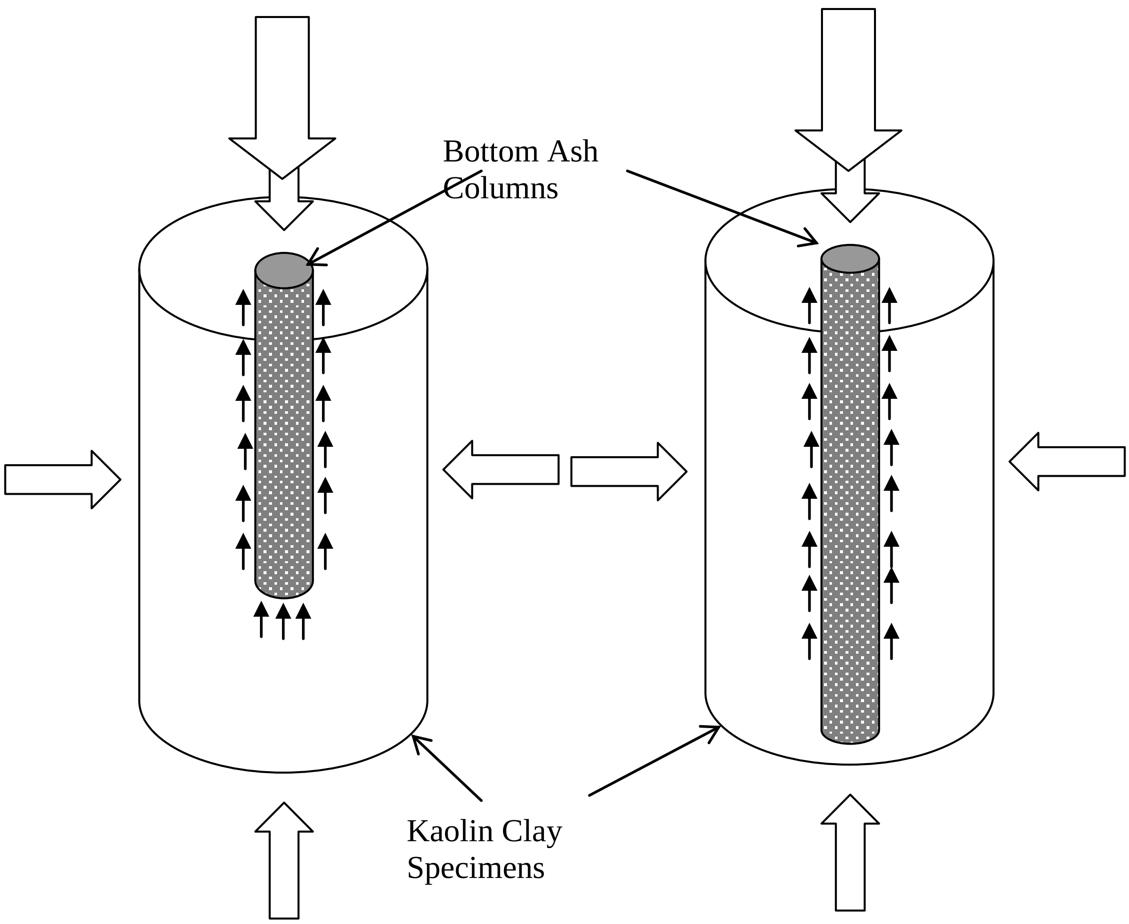

Fig. 6 Schematic diagrams of soft kaolin clay reinforced with partially and fully penetrating granular columns, confined by surrounding soil pressure

3.3Effects of Volume Replacement Ratio

Fig. 7 shows the improvement of undrained shear strength with volume replacement ratio (V_c/V_s). From the figure, the installation of bottom ash columns improved the undrained shear strength of kaolin samples. However, the improvement is not in line with the increase of volume replacement ratio. After V_c/V_s exceeds 10 %.

Since a large portion of soils were drilled and taken out from the samples, it affects the natural state of the soil. This resulted in the shear strength of the samples to decrease. Because the samples during the test were not imposed with any confining pressure, the tendency for the soil to collapse is much higher.

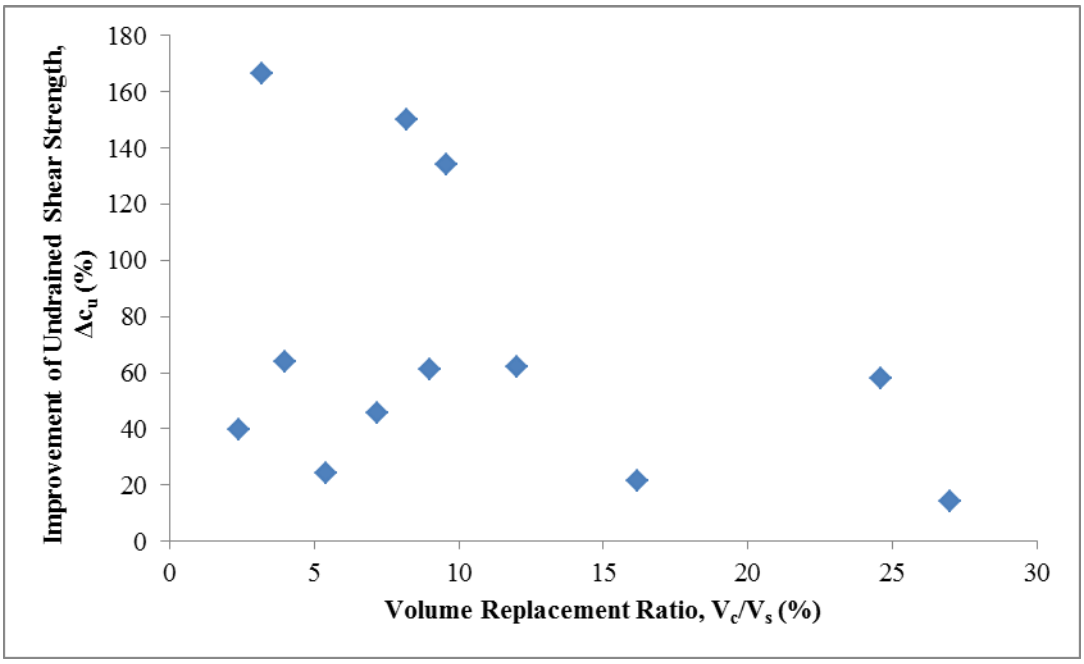

Fig. 7 Improvement of undrained shear strength with volume replacement ratio

4. CONCLUSIONS

Conceptually, the bottom ash column is proposed for the improvement of shear strength and compressibility of soft soil for the construction purposes based on the potential of bottom ash column to act as a combination of pile and vertical drain, besides being stronger materials to replace soft or weak soils. The granular bottom ash column acts principally as a reinforcement material while the void in between the bottom ash particles could accelerate the dissipation of pore water pressure. However, the primary selection criteria for bottom ash are supposed to be its strength while its drainage capability is secondary. The strength and stiffness of bottom ash was exploited to improve shear strength by replacing portion of soft soil within the samples. The result shows significant improvement in shear strength and compressibility of soft kaolin clay samples reinforced with bottom ash columns, compared to the unreinforced or ‘controlled’ sample. When bottom ash, which is a stiffer material than soft soil, was inserted as vertical column, it is clear that the strength behaviour of the soft soil was greatly modified.

It can be concluded that through the unconfined compression test a conclusion can be made such that the height penetrating ratios have significant effect to the undrained shear strength of reconstituted soft clay reinforced with bottom ash column(s). The strength generally increases with the increase in the height penetrating ratio but decreases after reaching an optimum improvement at 80 % of height penetrating ratio. The conclusion is that if a column is sufficiently short for significant load to be transmitted to the base of the column then it will penetrate the underlying clay. As the column length increases, the penetration reduces because fewer loads pass to the base of the columns. Plus, beyond a certain length no further load can reach the base of the column because it might have all been shed through the column’s shaft friction. The increment of undrained shear strength is also dependent on the area replacement ratio of bottom ash. Without confining pressure, the higher the ratio, the higher the strength occurred. However, excessive area replacement decreased the shear strength of the sample reinforced by group columns. This is because the remaining width of the soil sample is too thin to hold the columns. Hence, when the vertical load is distributed to the columns, they just collapsed due to insufficient lateral pressure to support.

This research will probably give another alternative for a new type of soil improvement method especially in the construction over soft clay area. The method proposed in this research can be economically scaled down towards cost reduction, in foundation and embankment construction, particularly when utilizing by-product materials such as bottom ash. However, full-scale field tests are required to fully understand the nature of improvement occurred.

ACKNOWLEDGEMENT

The authors would like to acknowledge the Universiti Malaysia Pahang (UMP) for financing this research through the Research University Grant Scheme, Project Number RDU 140373. The cooperation given by all parties involved in this research is greatly acknowledged.

REFERENCES

Ambily, A. P.; Gandhi, S. R. (2007), “Behavior of stone columns based on experimental and FEM analysis,” Geotechnical and Geoenvironmental Engineering, **133**(4), 405–415.

Black, J.; Sivakumar, V; Madhav, M. R.; McCabe, B. (2006), “An improved experimental setup to study the performance of granular columns,” Geotechnical Testing Journal, **293**, 193–199.

Black, J.; Sivakumar, V.; McKinley, J. D. (2007), “Performance of clay samples reinforced with vertical granular columns,” Canadian Geotechnical Journal, **44**, 89-95.

Fleming, W. G. K, Weltman, A. J., Randolph, M. F. and Elson, W. K. (1985), "Piling Engineering," England: Surrey University Press.

Hasan, M; Marto, A.; Hyodo, M.; Makhtar, A. M. (2011), "The strength of soft clay reinforced with singular and group bottom ash columns," Electronic Journal of Geotechnical Engineering, **16** (M), 1215-1227.

Hughes, J. M. O.; Withers, N. J. (1974), "Reinforcing of soft cohesive soils with stone columns," Ground Engineering, **7** (3), 42–49.

Juran, I.; Riccobono, O. (2001), "Reinforcing soft soils with artificially cemented compacted-sand columns," Geotechnical Engineering, **117** (7).

Kempfert, H. G. and Gebreselassie, B. (2007), "Excavations and Foundations in Soft Soils," Springer.

Kumar, S; Stewart, J. (2003), "Evaluation of Illinois pulverized coal combustion dry bottom ash for use in geotechnical engineering applications," Journal of Energy Engineering, **129** (2), 42–55.

Maakaroun, T; Najjar, S.S.; Sadek, S. (2009), "Effect of sand columns on the load response of soft clays." Proc. International Foundation Congress and Equipment Expo, 217-224.

McKelvey, D. (2002), "The performance of vibro stone column reinforced foundations in deep soft ground," Ph.D. Thesis, Queen's University Belfast, U.K.

McKelvey, D.; Sivakumar, V.: Bell, A.; Graham, J. (2004), "Modelling vibrated stone columns in soft clay," Geotechnical Engineering, **157**, 137-149.

Muhardi; Marto, A; Kassim, K. A.; Makhtar, A. M.; Lee, F. W.; Yap, S.L. (2010), "Engineering characteristics of Tanjung Bin coal ash," Electronic Journal of Geotechnical Engineering. **15** (K), 1117-1129.

Muir Wood, D; Hu, W; Nash, F. T. (2000), "Group effects in stone column foundation: model tests," Geotechnique, **50** (6), 689–698.

Murugesan, S., and Rajagopal, K. (2010), "Studies on the behavior of single and group of geosynthetic encased stone columns," Journal of Geotechnical and Geoenvironmental Engineering, **136**, 129 – 139.

Najjar, S. S.; Sadek, S.; Makaroun, T. (2010), "Effect of sand columns on the undrained load response of soft clays", Geotechnical and Geoenvironmental Engineering, **136** (9), 1263–1277.

Narasimha Rao, S.: Prasad, Y. V. S. N.; Hanumanta Rao, V. (1992), "Use of stone columns in soft marine clays," Proc. 45th Canadian Geotechnical Conference, Toronto: Canadian Geotechnical Society.

Sivakumar, V.; McKelvey, D; Graham, J; Hughus, D. (2004), "Triaxial tests on model sand columns in clay," Canadian Geotechnical Journal, **41**, 299–312.

Tandel, Y. K., Solanki, C. H., and Desai, A. K. (2012), "Numerical modelling of encapsulated stone column-reinforced ground," International Journal of Civil, Structural, Environmental and Infrastructure Engineering (IJCSEIERD), **2** (1), 82 – 96.