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Mohammed Y. Fattah,[1] *Kais T. Shlash,*[2] *and Maki J. Mohammed Al-Waily*[3]

Stress Concentration Ratio of Model Stone Columns in Soft Clays

ABSTRACT: In this work, laboratory experiments have been carried out to study the value of the stress concentration ratio, *n*, which is defined as the ratio of vertical stress acting on the stone column to that acting on the surrounding soil. A laboratory setup was manufactured in which two proving rings are used to measure the total load applied to the soil-stone column system and the individual load carried directly by the stone column. The foundation steel plates have 220 mm diameter and 5 mm thickness. These plates contain 1, 2, 3, and 4 holes. The spacing between all the holes equals twice the stone column diameter, *D*, center to center. The stone columns made of crushed stone were installed in very soft clays having undrained shear strength ranging between 6 and 12 kPa. Two length to diameter ratios *L*/*D* were tried, namely, *L*/*D*=6 and 8. The testing program consists of 30 tests on single, two, three, and four columns to study the stress concentration ratio and the bearing improvement ratio ($q_{treared}/q_{untreated}$) of stone columns. The experimental tests showed that the stone columns with *L*/*D*=8 provided a stress concentration ratio *n* of 1.4, 2.4, 2.7, and 3.1 for the soil having a shear strength cu=6 kPa, treated with single, two, three, and four columns, respectively. The values of *n* were decreased to 1.2, 2.2, 2.5, and 2.8 when the *L*/*D*=6. The values of *n* increase when the shear strength of the treated soil was increased to 9 and 12 kPa.

KEYWORDS: stone columns, group, stress concentration, laboratory model, soft clay

Introduction

Many methods for ground modification and improvement are available around the world now, including dewatering, compaction, preloading with and without vertical drains, grouting, deep mixing, deep densification, and soil reinforcement. Many of these techniques, such as dewatering, compaction, preloading, and grouting, have been used for many years. However, there have been rapid advances in the areas of deep densification (vibro-compaction, deep dynamic compaction, compaction piles, and explosive densification), jet and compaction grouting, deep mixing, and vibro-replacement and vibro-displacement in recent years. These methods have become practical and economical alternatives for many ground improvement applications (Raman 2005).

Of the many techniques of ground improvement, stone column has gained lots of popularity since it has been properly documented in the middle of the last century. As in most new ground improvement techniques that were developed in different countries, experience has preceded the development of theory and comprehensive guidelines. The stone column technique of ground treatment has proven successful in the following:

(1) Improving the slope stability of both embankments and natural slopes,
(2) Increasing the bearing capacity,
(3) Reducing the total and differential settlements,
(4) Reducing the liquefaction potential of sands, and
(5) Increasing the time rate of settlement.

Stone columns are used to support the structures overlying both the very soft to firm cohesive soils and also loose the silty sands having greater than about 15 % fines (Barksdale and Bachus 1983).

The high potential for the beneficial use of stone columns is mainly as a ground improvement technique to strengthen the weak and soft soil. This includes the area of highway, railway, and also airfield applications, which prompted a comprehensive investigation to determine how and why the system works so well and to develop appropriate design and construction guidelines. This has resulted in many empirical design concepts to be published for the purpose of designing the stone column.

The stone columns were first employed in Europe in the 1830s and have been used there extensively since the late 1950s. The practice was adopted in the United States since the early 1970s. This technique was proven, in general, to increase the bearing capacity within 150–300 % and reduces the settlements within 30–80 % (Priebe 1995).

The cement-soil mixed columns are also used for soft ground improvement. The soil ground treated by the deep cement mixing (DCM) in the field normally consists of cement-soil mixed columns and untreated soils. Although many attempts have been made, research on the consolidation behavior of the treated soil ground has been limited.

The experimental arrangement and the results of a small-scale physical composite foundation model test with instrumentation have been presented by Yin and Fang (2006). Based on the analysis, it was found that under the approximately rigid loading condition, the pressure carried by the DCM soil column and the untreated soft clay changed with time and external loading. For a specified loading stage, with the progressive increase of degree of consolidation of the untreated soft clay, the DCM soil column carried a progressively greater proportion of loading than the untreated soft clay. The failure of the composite foundation was caused mainly by the local failure of the DCM soil column.

An axisymmetric physical model test with full instrumentation was carried out by Fang and Yin (2007). The physical model ground consisted of a central cement-soil column and surrounding soft soil. The excess pore water pressures in the soil and the vertical

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[1]Ph.D., Dept. of Building and Construction Eng., Univ. of Technology, Baghdad 964, Iraq, e-mail: myf_1968@yahoo.com
[2]Ph.D., Professor, Dept. of Building and Construction Eng., Univ. of Technology, Baghdad 964, Iraq, e-mail: ktishlash1@yahoo.com
[3]Ph.D., Al-Musaib Technical Institute, Babylon, Iraq, e-mail: maki_jafar@yahoo.com

pressures carried by the DCM column and the untreated soil were recorded throughout the test. It is found that the improved ground consolidates faster than the pure soil ground. The major reason is considered to be that the DCM column reduces the vertical stress increment in the soil and results in a lower value of excess pore pressure. The decrease of excess pore water pressure in the middle of the soil seems to be controlled by the reduction of the total stress in the soil. Besides, a delayed pore water pressure increase was observed in the early period of the loading stage.

Stress Concentration

When the composite ground is loaded, studies have shown that the concentration of stress occurs within the stone column accompanied by the reduction in stress in the less stiff surrounding clayey soil (Bergado et al. 1996). This is due to the approximately same vertical settlement of the granular material and the surrounding soil.

Because of the higher stiffness, the stress concentrates on the column material and causes a difference in the vertical stress within the column and in the surrounding soil. Such a disparity or stress concentration is also evident from the results of the analysis. The stress distribution is generally defined in terms of a stress concentration ratio, n, as

$$n = \frac{\sigma_s}{\sigma_c} \tag{1}$$

where:

σ_s=stress in the column and

σ_c=stress in the surrounding soil (clay).

The area replacement ratio, a_r, representing the area of the clay foundation, A_c, replaced by a stone column, A_s, is given by Cheung (1998)

$$a_r = \frac{A_s}{A_s + A_c} \tag{2}$$

where:

A_s=cross sectional area of stone column and

A_c=area of clay per each sand column.

To give a realistic picture of the actual situation, the design of the stone column pattern needs to take into account the stress distribution between columns and soil.

The main objective of this work is to find an experimental base for the value of n by manufacturing a model of the single stone column with rigid instrumented loading plates such that the total load applied to the model footing and the load applied to the stone column can be measured separately.

Experimental Work

Soil Used

Soil samples were collected from a depth of 0.50 m from the ground surface of a site in the vicinity of Al-Musaib Technical Institute in Babylon, west of Iraq. The soil was subjected to routine laboratory tests to determine its properties. These tests include the following:

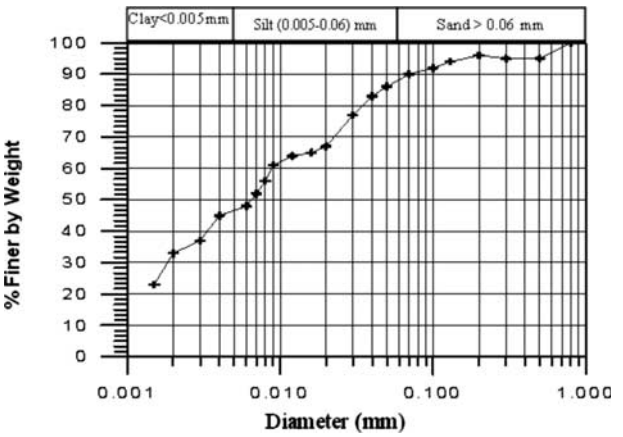


FIG. 1—*Grain size distribution of the soil used in preparing the model tests.*

(1) Grain size distribution (sieve analysis and hydrometer tests) according to ASTM D422 (2002) specifications and
(2) Atterberg limits (liquid and plastic limits) according to ASTM D4318 (2000) specifications.

The test results show that the soil consists of 10 % sand, 42 % silt, and 48 % clay as shown in Fig. 1. According to the unified soil classification system, the soil is inorganic sandy silty clay designated as CL. Table 1 shows the physical properties of the soil.

The natural calcium carbonate, $CaCo_3$ (limestone), crushed stone was used as a backfill material. The size of the crushed stone was chosen in accordance with the guidelines suggested by Nayak (1983), where the particle size is about 1/6 to 1/7 of the diameter of stone columns. The minimum particle size is 4 mm and the maximum particle size is 10 mm. Figure 2 illustrates the grain size distribution of the crushed stone used in the stone columns.

The Test Setup

Steel Container—The model tests were carried out in a test tank manufactured of steel with dimensions of $1100 \times 1000 \times 800$ mm^3, made of steel plates 6 mm in thickness as shown in Fig. 3. The container is sufficiently rigid and exhibited no lateral deformation during the preparation of the bed of soil and during the tests.

The Loading Frame—Figure 4 shows the details of the complete setup, which consists mainly of steel container, loading frame, dial gauges, and accessories.

TABLE 1—*Physical properties of the treated soil.*

Property	Value
Liquid limit (LL)	44 %
Plastic limit (PL)	22 %
Plasticity index (PI)	22 %
Specific gravity (G_s)	2.72
Percent passing sieve no. 200	90 %
Sand content	10 %
Silt content	42 %
Clay content <0.005 mm	48 %
Maximum dry unit weight, kN/m^3	17.8
Symbol according to Unified Soil Classification System	CL

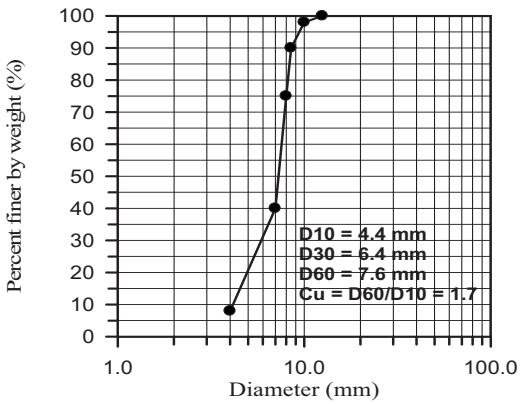
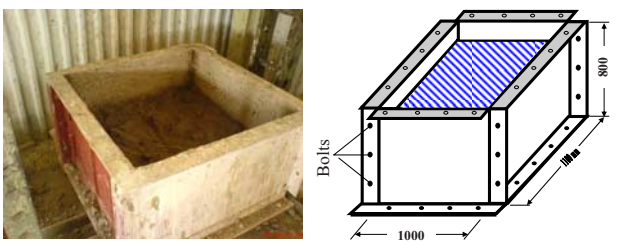


FIG. 2—*Grain size distribution of the crushed stone used in the model stone columns.*



(a) Photograph **(b)** Isometric view

FIG. 3—*Steel container used in the tests. (a) Photograph. (b) Isometric view.*

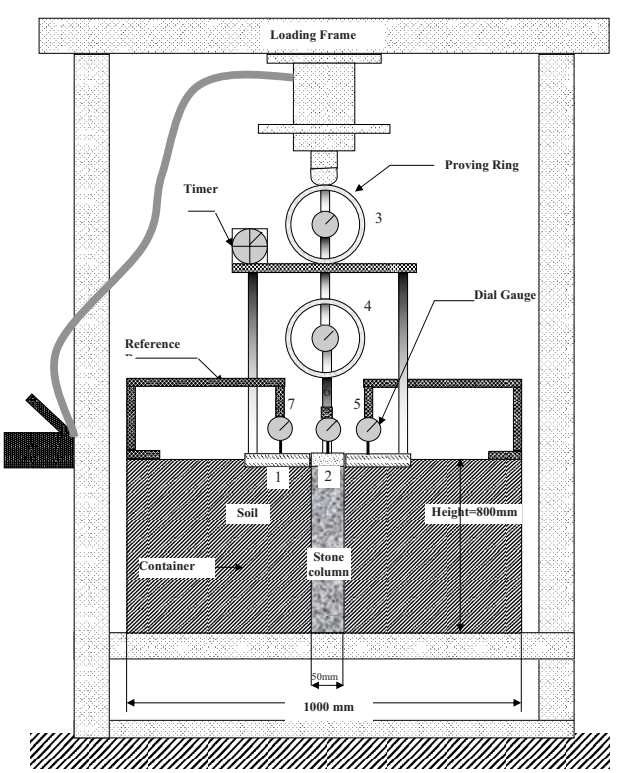


FIG. 4—*Schematic diagram of the experimental setup.*

FIG. 5—*Foundation plates and accessories.*

The Foundation Plates and Accessories—Figures 5 and 6 show the details of the foundation plates and accessories used for carrying out the loading tests. The foundation consists of a plate with a diameter of 220 mm and a thickness of 5 mm and having a hole in the middle or more than a hole (2, 3, and 4) holes distributed as shown in the figures (spacing between all columns=2*D*, center

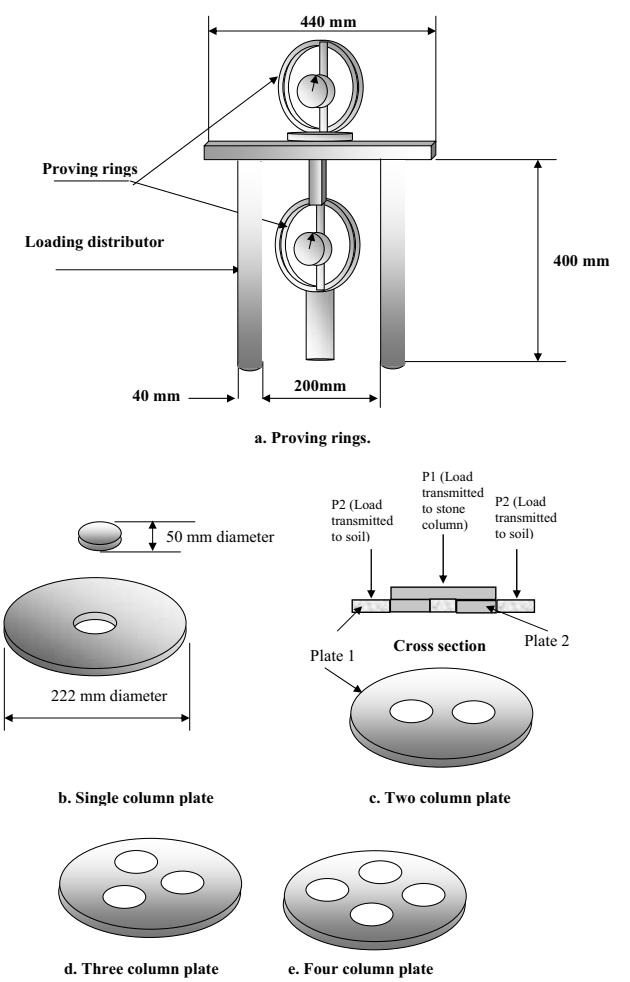
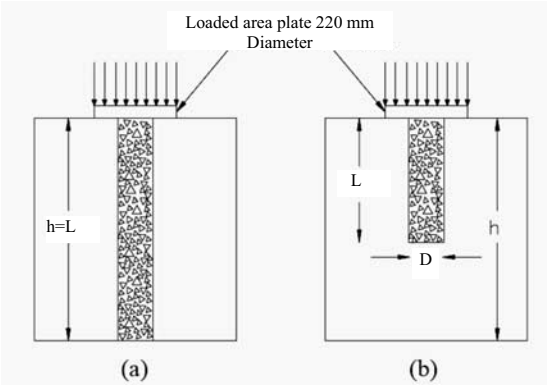


FIG. 6—*Schematic diagram of the foundation plates: (a) Proving rings; (b) single column plate; (c) two column plate; (d) three column plate; and (e) four column plate.*



Fully penetrated stone column, L/D = 8 Partially penetrated stone column, L/D = 6

FIG. 7—*Loading of the composite bed. Fully penetrated stone column, $L/D = 8$. Partially penetrated stone column, $L/D = 6$.*

to center of columns).

Figure 6(*c*) illustrates the method by which the loading distributor fit the two column plates to keep the loading center on the same line of footing center. The same thing was done for the three column plate and the four column plate as shown in Fig. 6(*d*) and 6(*e*), respectively.

Model Preparation and Testing

Preparation of the Bed of Soil—Prior to the preparation of the bed of soil, a relationship was obtained between the water content and the undrained shear strength of the soil. This relation will help maintain the required shear strength for each model. The shear strength was measured by using the Swedish fall cone pentrometer.

Following this stage, the bed of the soil was prepared as follows.

(1) The natural soil was first crushed with a hammer to small sizes and then left for 24 h for air-drying; further crushing was carried out by using a crushing machine.
(2) The air-dried soil was divided into 10 kg groups.
(3) Each group was mixed gradually and thoroughly with sufficient amount of water corresponding approximately to the water content range of 24–35 %.
(4) After mixing with water, the soil was placed in layers inside the steel container and each layer was tamped with a special tamping hammer of 50×50 mm^2 in size. The final thickness of each layer was about 50 mm. The procedure was continued till the final thickness of the bed of soil.
(5) After the completion of the preparation of the bed of soil, it was covered tightly with nylon sheets and left for four days as curing period.

Construction of Stone Columns—At the end of the curing period, the following steps were used in construction of the stone columns.

(1) The top of the soil bed was leveled.
(2) The position of the stone column(s) to be placed was properly marked with respect to the loading frame. A hollow PVC tube, with external diameter of 52 mm and 2 mm in thickness, coated with petroleum jelly was inserted vertically to the required depths (40 mm in fully penetrated stone column or $L/D = 8$ and 30 mm in partially penetrated stone column or $L/D = 6$), [the critical length is usually about four times the column diameter (Greenwood and Kirsch 1983)]. More details about both cases (fully and partially penetrated stone columns) are shown in Fig. 7. The tube was then slowly withdrawn and twisted during the lifting process.
(3) The soil was removed from the tube and samples of the soil at different depths were taken for water content measurement.
(4) The crushed stone was poured into the hole in layers and each layer was compacted gently by using a 30 mm diameter tamping rod. The unit weight of the compacted crushed stone was measured to be 16.3 kN/m^3.

Model Testing Procedure

The model tests were carried out according to the testing program as follows.

(1) Calibrating the proving rings used in the testing program by applying various known static loads on it and recorded the readings of dial gauges. This procedure was repeated for many times to get more accurate readings.
(2) The footing assembly 220 mm diameter consists of two plates (1 and 2 in Fig. 4), one of them on the stone column(s) and the other on the surrounding soil. These plates were placed in position so that the center of the footing coincides with the center of the hydraulic jack.
(3) Two proving rings (3 and 4 in Fig. 4) with an accuracy of 0.01 mm/division were set such that the total load applied to the model footing, and the load applied to the stone column can be measured alone.
(4) Three dial gauges (5, 6, and 7 in Fig. 4) with an accuracy of 0.01 mm/division were fixed in position to measure the settlements of both plates.
(5) Loads were then applied through a loading disk in the form of load increments.
(6) During each load increment, the readings of the two dial gauges corresponding to two proving rings (3 and 4 in Fig. 4) were recorded.
(7) The dial gauge readings (5, 6, and 7 in Fig. 4) were recorded at the end of the period of each load increment.
(8) Each load increment was left for 2.5 min.
(9) The load increments were continued until the total settlement reached 50 mm (100 % of the stone column diameter).
(10) For comparison purposes, the loading tests were performed in the container on the untreated soil only.

Figure 8 presents a stone column model after completion of the tests.

Presentation of Results and Discussion

In this paper, 30 model tests are conducted according to the testing program to examine the behavior of each case. The cases include single stone column and groups of columns consisting of two, three, and four columns. All figures are presented to account for the

FIG. 8—*Two columns, $L/D=8$, after the test.*

stress concentration ratio n and the "bearing improvement ratio" ($q_{\text{treated}}/q_{\text{untreated}}$).

In the stage of bearing analysis, the obtained bearing capacity ratio (q/cu) representing the obtained bearing capacity to the undrained shear strength of the natural soil is plotted versus the settlement ratio (S/B), the settlement of the footing to its diameter.

The bearing improvement ratio ($q_{\text{treatd}}/q_{\text{untrated}}$) attained by the stone columns is calculated at the same value of settlement under applied stress for both models, untreated soil and soil treated with stone columns.

Definition of Failure

Most researchers consider the stone column to behave as a pile. Therefore, the criteria proposed for defining the failure load of the pile can be adopted for stone columns. There are many approaches used to define the ultimate bearing capacity and failure of stone column. The most important five of them are as follows.

(1) De Beer (1967) method (as reported by WinterKorn and Fany (1975)). The bearing capacity is taken at the break point of two interesting straight lines of different slopes after plotting the load-settlement relationship in log-log plot. This break point represents failure. This method was adopted by Al-Qyssi (2001).
(2) Terzaghi method, where failure was defined as the load corresponding to 10 % of the model footing width (or pile diameter). This method was adopted by Zakariya (2001).
(3) Tangent method, in which definition of failure is based on the intersection of the two tangents of load-settlement curve. The first tangent corresponds to the initial part of the curve while the second is tangent to the lower flatter portion of the curve.
(4) Hughes and Withers (1974) method. The ultimate load carrying capacity (true failure, equals to 26 times the undrained cohesion of the clay) was reached at a vertical displacement of 58 % of the stone column diameter. Al-Mosawe et al. (1985) found that the ultimate load carrying capacity was reached at a vertical displacement of 60 % of the diameter of the stone column.
(5) Rao et al. (1997) method. The capacity is taken as the load corresponding to a settlement equal to 0.1 times the diameter of the stone column.

Figure 9 presents the bearing ratio—settlement relationships for the case of single stone with $L/D=8$ constructed in a very soft clay with shear strength, cu=12 kPa. In these figures, the pervious criteria for the definition of failure were used. In Fig. 10, a new criterion was proposed in which the bearing ratio (failure) is defined when the settlement reaches 50 % of the diameter of the stone column or 11 % of the diameter of the model footing (the least value of the two) (Al-Waily 2007). This definition is compatible with Hughes and Withers (1974) and Al-Mosawe et al. (1985).

A comparison between Figs. 9 and 10 with the shape of load-settlement for the case of local shear failure reveals that the treated and untreated clays showed a local shear failure in spite of the improvement by the stone columns.

After examining the previous methods and by inspection of the behavior of the stress-settlement relation for the untreated and treated soil in the present work, it was found that this behavior indicates that the settlement increases in a low rate with the increase of stress until the settlement equals approximately half the diameter of the stone column diameter. After that, the increase in settlement was in a steeper rate.

Improvement of Stress Concentration Ratio

The stress concentration ratio is calculated during the incremental loading applied up to a final value. The variation of stress concentration ratio n versus the stress increments is determined. The stress concentration ratio n is plotted versus the bearing ratio in soil treated with single, two, three, and four columns for three soil conditions (cu=6, 9, and 12 kPa) and for stone column with stone only, $L/D=8$, and stone column with stone only, $L/D=6$.

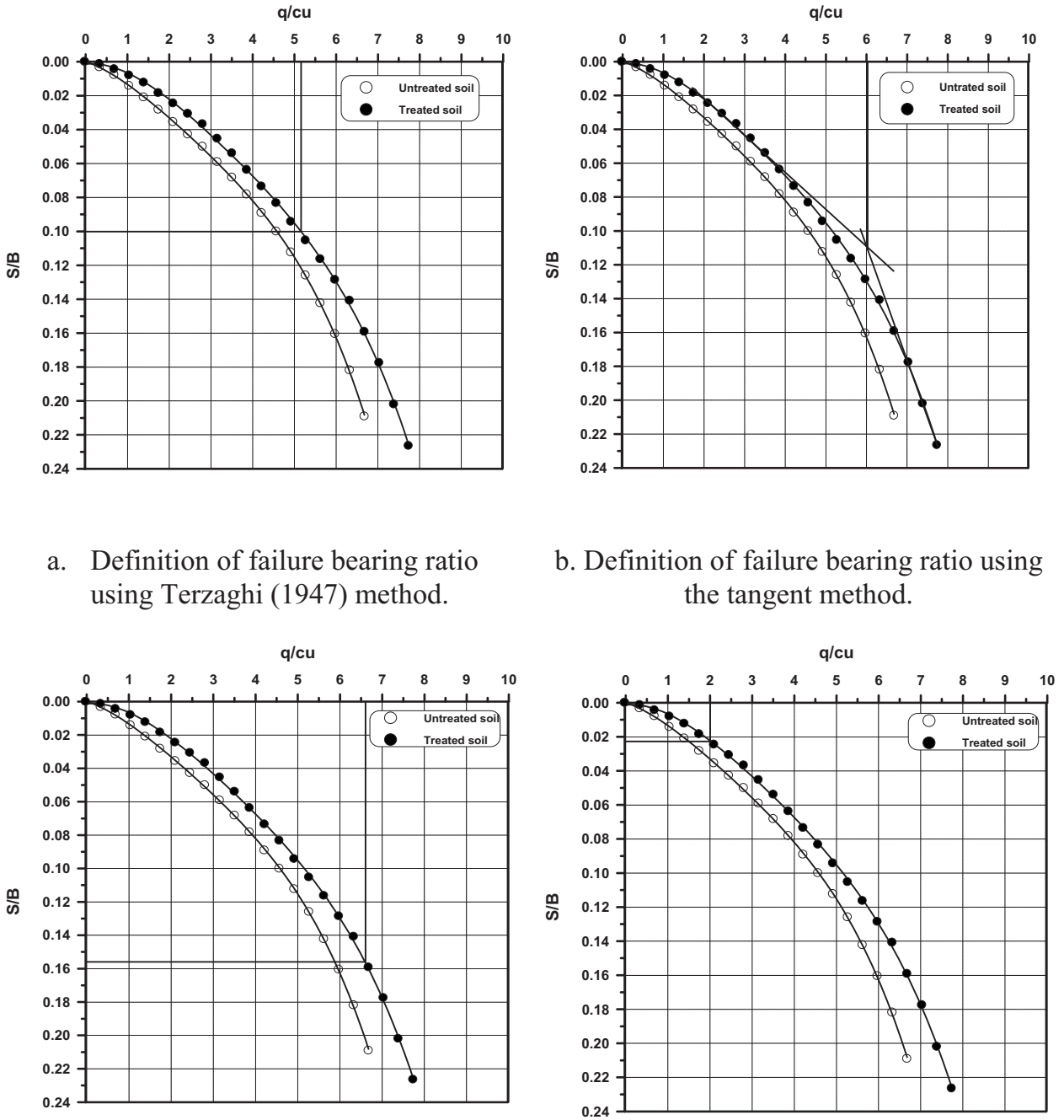
Stress Concentration Ratio n at Various Shear Strengths and Various (L/D) Ratios

Figures 11–16 show the relationship between the stress concentration ratio n and the bearing ratio (q/cu) for twenty four model tests of soil treated with single stone column and groups of 2, 3, and 4 stone columns. In these figures, the stone columns are constructed in very soft clays having three shear strengths (cu=6, 9, and 12 kPa). Two values of the length to diameter ratio are maintained $L/D=6$ and $L/D=8$. These figures showed the same general tendency that the stress concentration ratio n reached a peak value at a point located approximately at $q/\text{cu}=2$ then the behavior becomes in two ways as follows.

(1) The value of n was reduced gradually with increasing the bearing ratio (q/cu) in treated soil with 6 kPa shear strength.
(2) The value of n was reduced suddenly with increasing the bearing ratio (q/cu) in treated soil with 9 or 12 kPa shear strength.

After that, the n reached a plateau at the end of the test in both conditions. It can be noticed that the value of n was increased when the number of stone columns was increased. The n values are 1.2, 2.2, 2.5, and 2.8 in soil having a shear strength of 6 kPa, treated with single, two, three, and four stone columns at $L/D=6$, respectively (Fig. 11).

Also, it can be noticed that the values of n increase with increasing the L/D ratio. The n values at failure point are 1.4, 2.4, 2.7, and 3.1 in soil having a shear strength of 6 kPa, treated with single, two, three, and four stone columns at $L/D=8$ (Fig. 14). The stress concentration ratio values are 2.5, 2.6, and 3.4 at three shear strength



a. Definition of failure bearing ratio using Terzaghi (1947) method.

b. Definition of failure bearing ratio using the tangent method.

c. Definition of failure bearing ratio using Hughes and Withers (1974) method.

d. Definition of failure bearing ratio using Rao et al. (1997) method.

FIG. 9—*q/cu versus S/B for the soil treated with single stone column with cu=12 kPa, L/D=8. (a) Definition of failure bearing ratio using the Terzaghi (1947) method. (b) Definition of failure bearing ratio using the tangent method. (c) Definition of failure bearing ratio using Hughes and Withers (1974) method. (d) Definition of failure bearing ratio using Rao et al. (1997) method.*

values of 6, 9, and 12 kPa, respectively, for the soil treated with three stone columns with $L/D=6$ (Figs. 11–13) and the n values are increased to 2.7, 2.8, and 3.2 for the soil treated with three stone columns at $L/D=8$ (Figs. 14–16).

These figures also demonstrate that the n was increased generally with increasing the shear strength of the treated soil; this behavior is clear when the shear strength of 6 kPa was increased to 12 kPa. The n values are 2.3, 2.4, and 2.6 for the soil treated with two stone columns ($L/D=8$) at the three different shear strengths (Figs. 14–16). The results obtained from these figures are accessible in Table 2.

The results obtained in Figs. 11–16 are in agreement with Aboshi et al. (1979), Greenwood and Kirsch (1983), Mitchell and Huber (1985), Barksdale (1987), Juran and Guermazi (1988), Ber-

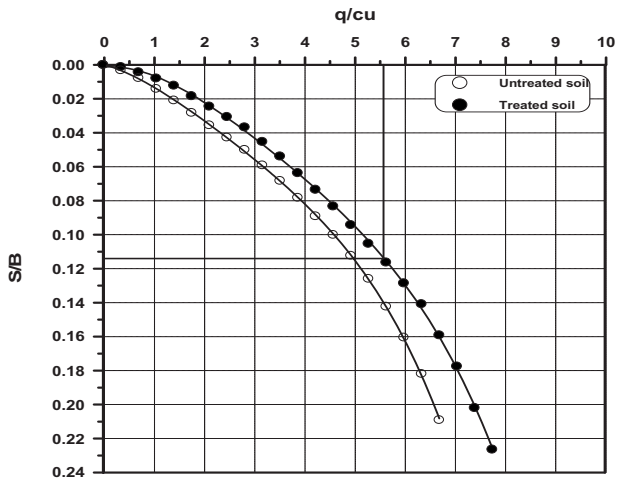


FIG. 10—*q/cu versus S/B for the soil treated with single stone column with cu=12 kPa, L/D=8, definition of failure bearing ratio is based on the present method.*

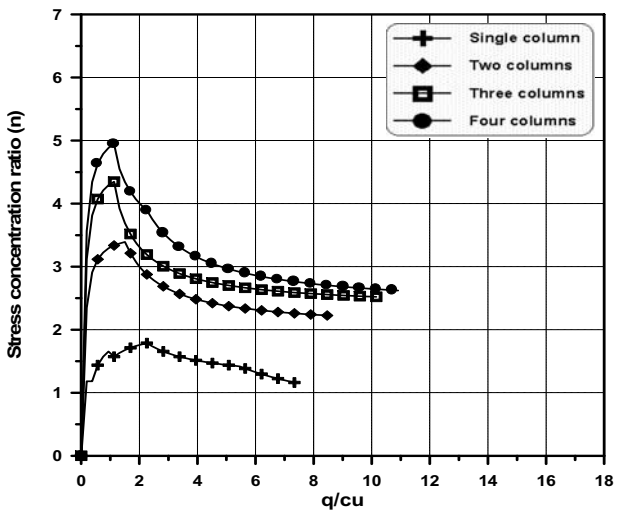


FIG. 12—*Stress concentration ratio (n) versus q/cu for the soil treated with stone columns, cu=9 kPa, L/D=6.*

gado et al. (1996), and Kirsch and Sondermann (2002). However, Stewart and Fahey (1994) from the laboratory study reported contradicting relationship.

Bearing Improvement Ratio

Figures 17, 19, 21, 23, 25, and 27 relate the bearing ratio (q/cu) with the deformation ratio S/B for untreated soil and soil treated with single, two, three, and four stone columns having L/D ratio of 6 and 8, respectively. The surrounding soil was prepared at undrained shear strength of cu=6, 9, and 12 kPa, respectively. These models were tested 24 h after the preparation. The figures demonstrate that the stone column in all bearing ratios shows significant difference in the behavior corresponding to S/B ratio.

The figures also indicate that when the shear strength of the soil decreases, the effect of the stone column becomes more visible and a clear increase in q/cu ratio is noticed. This behavior is attributed to the truth that the calculation of stresses is dependent on the stress applied to the soil that was replaced from the zone of stone column only, disregarding the stress applied to the soil surrounding the column. Thus, the effect of the improvement seemed clearly in the treated soil of low shear strength.

The bearing improvement ratio achieved by the stone columns is presented by the relationship between the ratio ($q_{\text{treated}}/q_{\text{untreated}}$) and the S/B ratio. It can be noticed from ($q_{\text{treated}}/q_{\text{untreated}}$) in Figs. 18, 20, 22, 24, 26, and 28 that the bearing improvement ratio ($q_{\text{treated}}/q_{\text{untreated}}$) ranges from 1.20 to 2.18 for the soil having cu =6 kPa treated with single stone column with L/D=6 and with four stone columns of L/D=6, respectively at S/B=11 % (Fig. 18). The ratio ($q_{\text{treated}}/q_{\text{untreated}}$) ranges from 1.18 to 1.88 for the soil having cu=9 kPa treated with single stone column of L/D=6 and with four stone columns of L/D=6, respectively (Fig. 20).

The ratio $q_{\text{treated}}/q_{\text{untreated}}$ ranges from 1.19 to 1.62 for the soil having cu=12 kPa treated with single stone column with L/D=6 and with four stone columns L/D=6, respectively (Fig. 22).

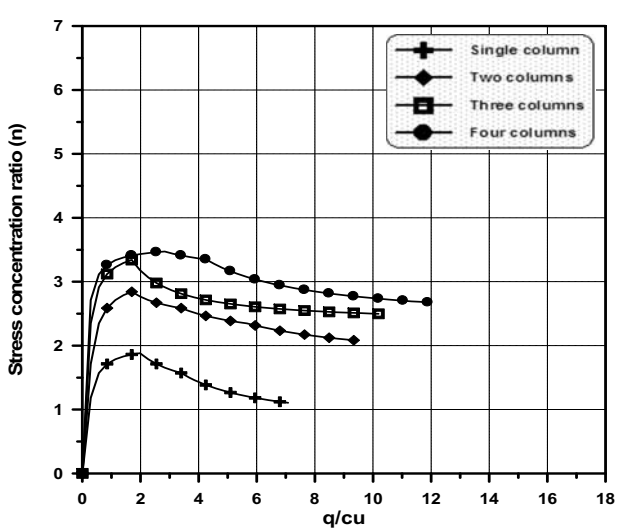


FIG. 11—*Stress concentration ratio (n) versus q/cu for the soil treated with stone columns, cu=6 kPa, L/D=6.*

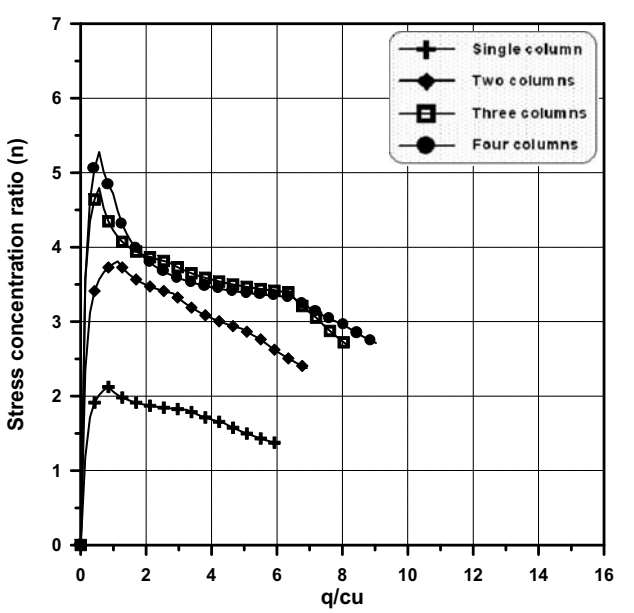


FIG. 13—*Stress concentration ratio (n) versus q/cu for the soil treated with stone columns, cu=12 kPa, L/D=6.*

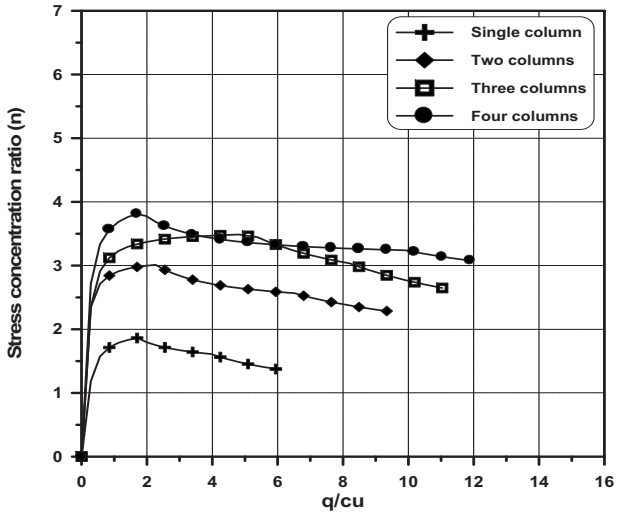


FIG. 14—*Stress concentration ratio (n) versus q/cu for the soil treated with stone columns, cu=6 kPa, L/D=8.*

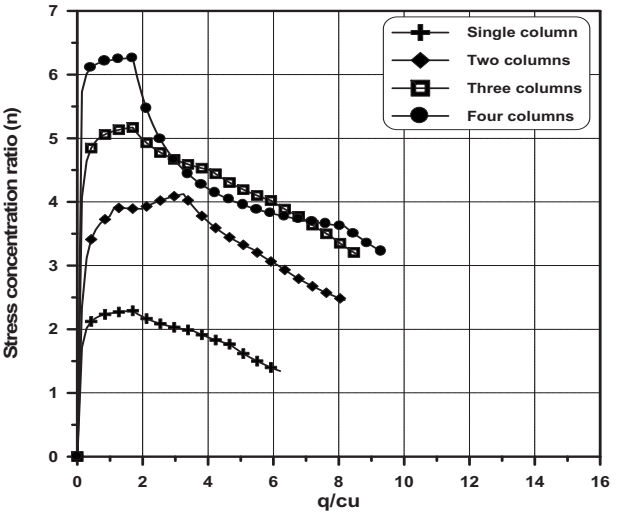


FIG. 16—*Stress concentration ratio (n) versus q/cu for the soil treated with stone columns, cu=12 kPa, L/D=8.*

It can be concluded from the previous values that the bearing improvement ratio is increased with increasing the number of stone columns by a percentage ranging between 20 % and 100 %. The results obtained from Figs. 18, 20, 22, 24, 26, and 28 are presented briefly in Table 3.

Conclusions and Recommendations

The following points are drawn from the tests.

(1) The value of stress concentration ratio *n* increases with increasing the shear strength of the treated soil.
(2) The crushed stone columns with $L/D=8$ provided a stress concentration ratio *n* of 1.4, 2.4, 2.7, and 3.1 for the soil having a shear strength, cu=6 kPa, treated with single, two, three, and four columns, respectively. The values of *n*

TABLE 2—*The stress concentration ratio (n) values for the soil treated with stone columns.*

Shear	Single Column	Two Columns	Three Columns	Four Columns
Stone column $L/D=6$				
cu=6 kPa	1.2	2.2	2.5	2.7
cu=9 kPa	1.5	2.3	2.6	2.7
cu=12 kPa	1.6	2.8	3.4	3
Stone column $L/D=8$				
cu=6 kPa	1.4	2.4	2.9	3.2
cu=9 kPa	1.5	2.6	3.1	3.6
cu=12 kPa	1.8	3.1	3.8	3.6

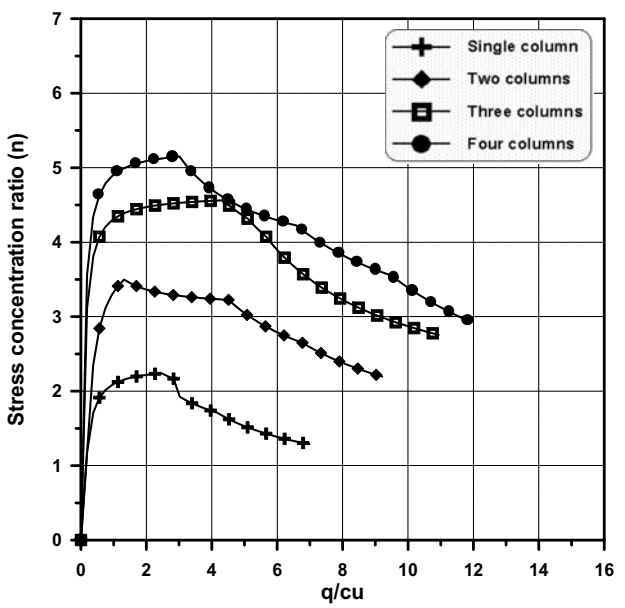


FIG. 15—*Stress concentration ratio (n) versus q/cu for the soiltreated with stone columns, cu=9 kPa, L/D=8.*

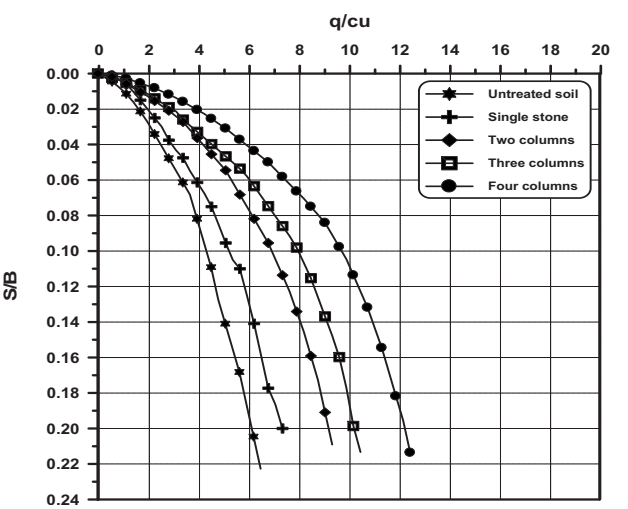


FIG. 17—*q/cu versus S/B for the soil treated with stone column, cu=6 kPa, L/D=6.*

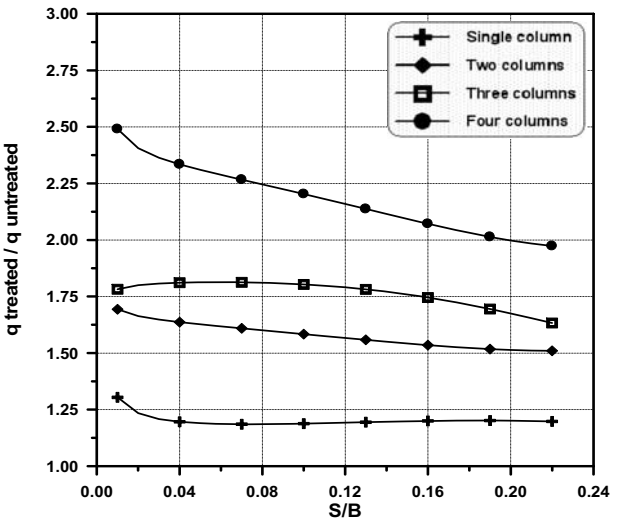


FIG. 18—*Bearing improvement ratio versus S/B for the soil treated with stone columns, cu=6 kPa, L/D=6.*

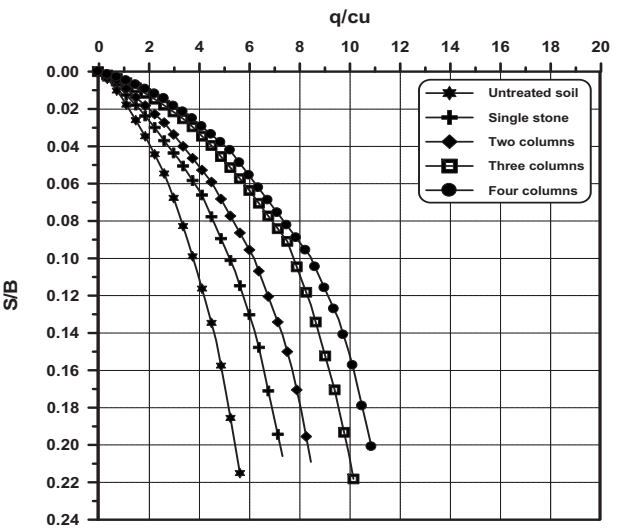


FIG. 19—*q/cu versus S/B for the soil treated with stone column, cu=9 kPa, L/D=6.*

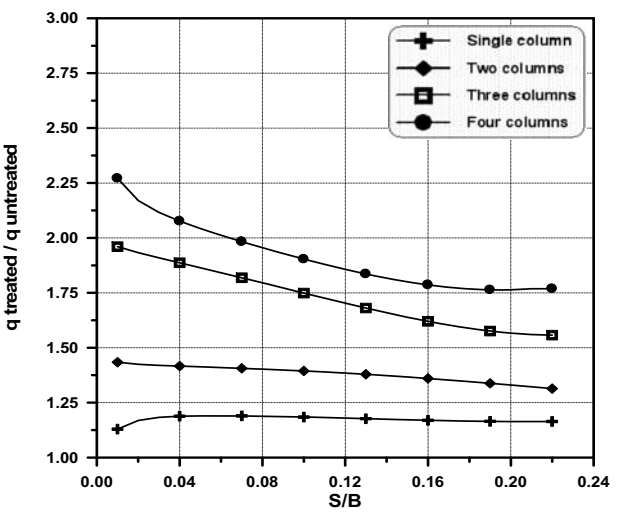


FIG. 20—*Bearing improvement ratio versus S/B for the soil treated with stone columns, cu=9 kPa, L/D=6.*

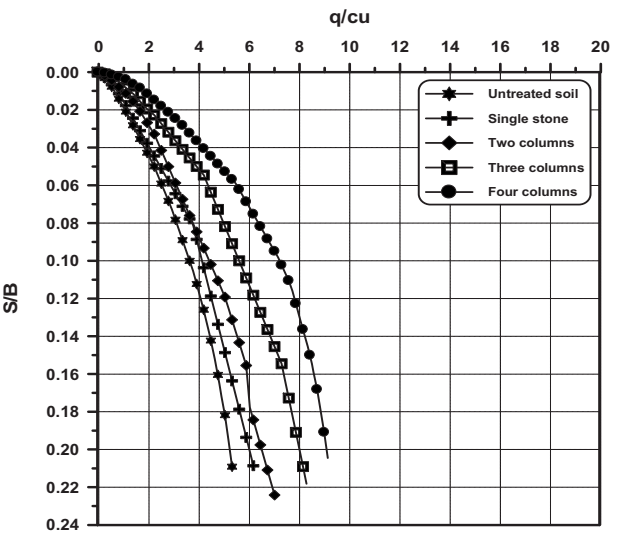


FIG. 21—*q/cu versus S/B for the soil treated with stone column, cu=12 kPa, L/D=6.*

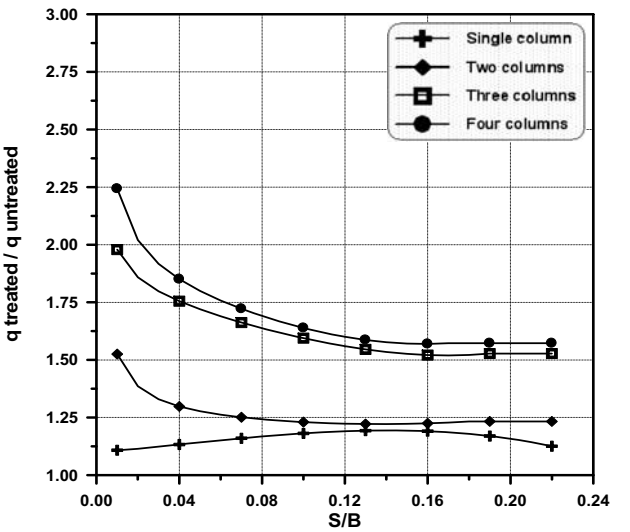


FIG. 22—*Bearing improvement ratio versus S/B for the soil treated with stone columns, cu=12 kPa, L/D=6.*

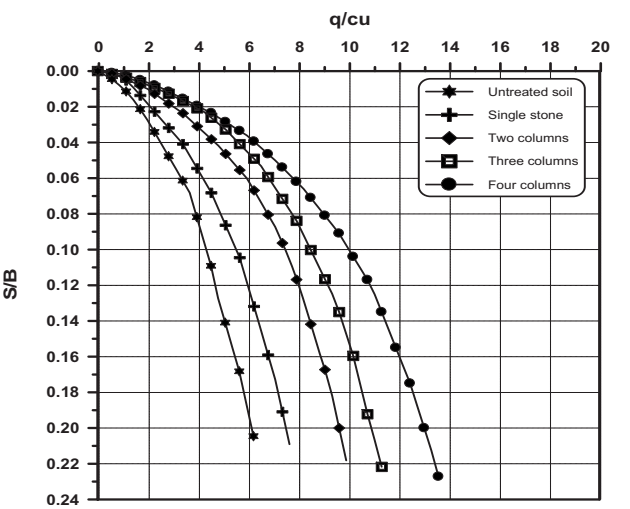


FIG. 23—*q/cu versus S/B for the soil treated with stone column, cu=6 kPa, L/D=8.*

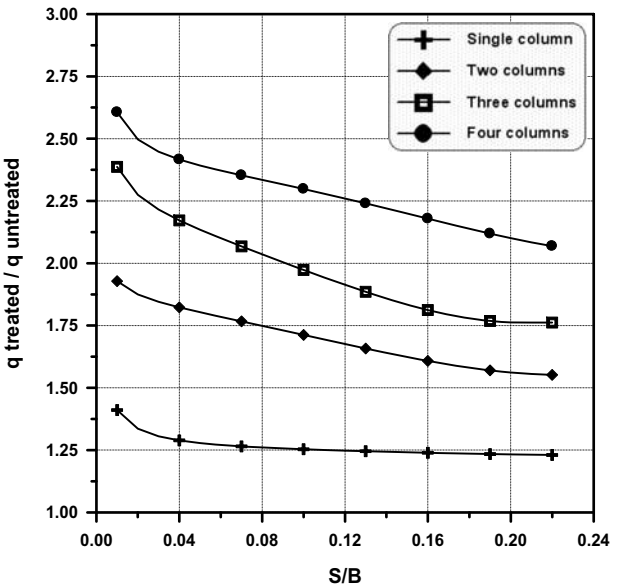


FIG. 24—*Bearing improvement ratio versus S/B for the soil treated with stone columns, cu=6 kPa, L/D=8.*

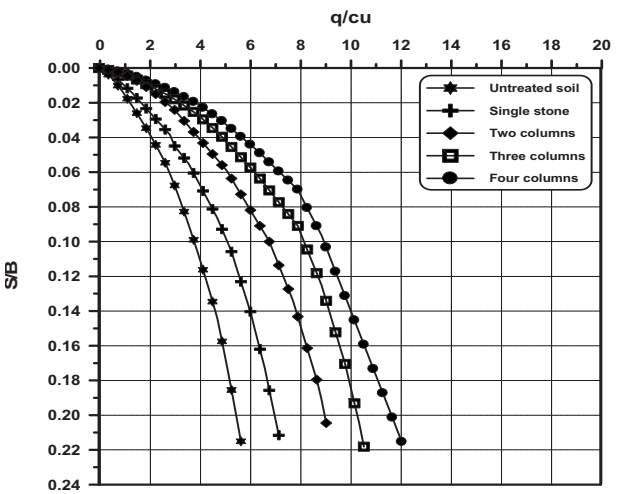


FIG. 25—*q/cu versus S/B for the soil treated with stone column, cu=9 kPa, L/D=8.*

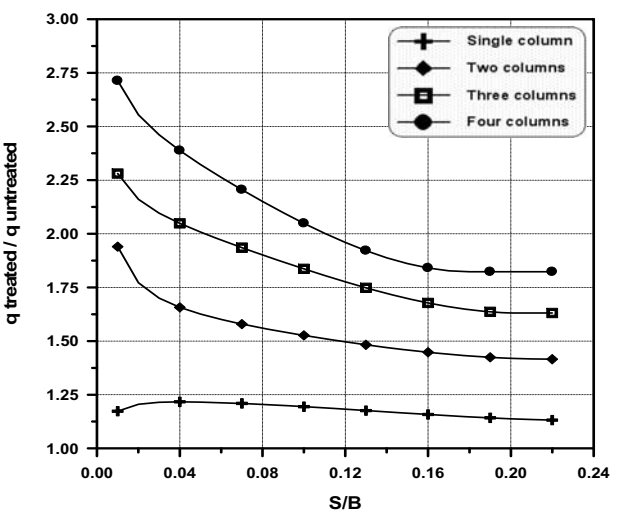


FIG. 26—*Bearing improvement ratio versus S/B for the soil treated with stone columns, cu=9 kPa, L/D=8.*

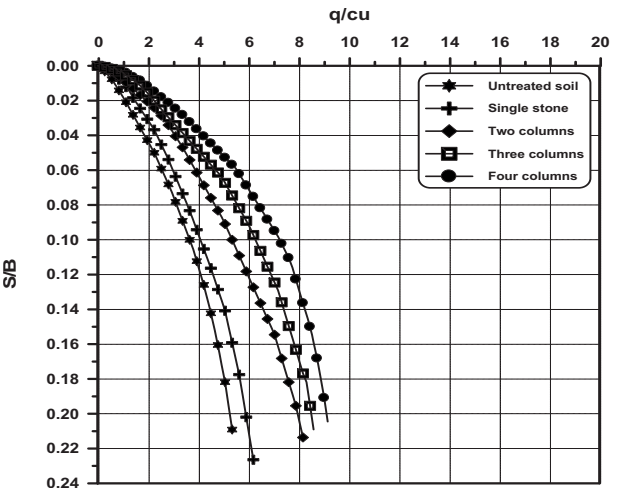


FIG. 27—*q/cu versus S/B for the soil treated with stone column, cu=12 kPa, L/D=8.*

were decreased to 1.2, 2.2, 2.5, and 2.8 when $L/D=6$.

(3) The value of the bearing improvement ratio decreases with increasing the shear strength of the treated soil.

(4) The stone columns with $L/D=8$ provided an increase in the bearing improvement ratio ($q_{\text{treated}}/q_{\text{untreated}}$) of 1.25, 1.7, 1.94, and 2.28 for the soil of shear strength cu=6 kPa treated with single, two, three, and four columns, respectively. The values of ($q_{\text{treated}}/q_{\text{untreated}}$) are decreased to 1.20, 1.58, 1.80, and 2.18 when $L/D=6$.

The following recommendations can be given for the cases at which stone columns in groups are suggested for the soil improvement.

(1) The values of stress concentration ratio n vary from 1.2 to 3.8 for stone columns having L/D ratio of 6 and 8 and constructed in very soft clays with shear strength of 6–12 kPa. The stone columns of $L/D=6$ are recommended for higher values of stress concentration ratio.

(2) The fully penetrated stone columns with $L/D=8$ provide higher bearing improvement ratio than partially penetrated

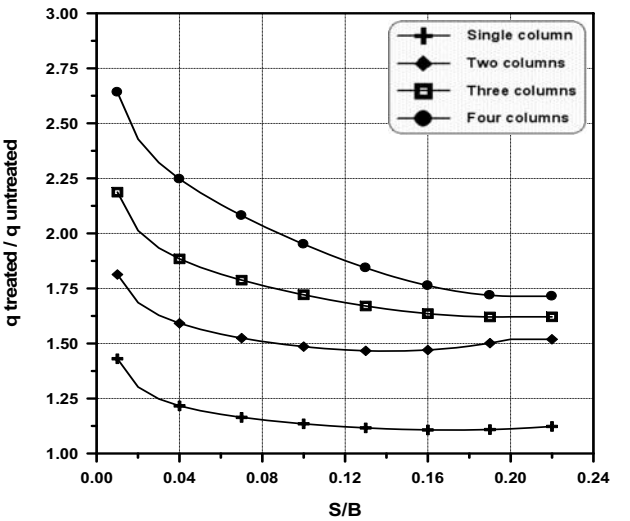


FIG. 28—*Bearing improvement ratio versus S/B for the soil treated with stone columns, cu=12 kPa, L/D=8.*

TABLE 3—*Bearing improvement ratio for the soil treated with stone columns.*

Shear	Single Column	Two Columns	Three Columns	Four Columns
(Stone) column $L/D=6$				
cu=6 kPa	1.20	1.58	1.80	2.18
cu=9 kPa	1.18	1.39	1.73	1.88
cu=12 kPa	1.19	1.23	1.57	1.62
(Stone) column $L/D=8$				
cu=6 kPa	1.25	1.70	1.94	2.28
cu=9 kPa	1.19	1.51	1.80	2.00
cu=12 kPa	1.13	1.48	1.70	1.80

stone columns for the case of single or two, three, and four column group by about 4–7 %. For economic design, partially penetrated stone columns with $L/D=6$ are recommended.

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