



# Effect of Stress Concentration Ratio on Shear Strength of Soft Soils Improved with Stone Columns

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## Abstract

Numerous studies have been performed on the behavior of stone columns subjected to vertical loads. However, little research has been focused on determining the shear strength and equivalent shear strength parameters of cohesive soils reinforced with stone columns. The performed research in this area mostly includes numerical analyses which do not consider the effect of stress concentration ratio, and the value of stress concentration ratio in such analyses is assumed to be 1. In this study, an experimental investigation has been carried out on stone columns subjected to vertical and shear loads, with the purpose of determining the actual value of stress concentration ratio. The experiments were performed in a large direct shear device. In this study, single, square and triangular arrangements of stone columns with three different area replacement ratios under three vertical loads were investigated. Soft clay was used as bed material, and stone column material was either crushed stone or fine-grained sand. Results showed that stress concentration ratio decreased with the increase in stone column diameter and vertical stress and the value of stress concentration ratio is highest for square arrangement and lowest for single stone columns. The equivalent shear parameters obtained from experiments and those predicted by analytical relationships were compared at stress concentration value of 1 and stress concentration values obtained from the experiments. Results also showed that the shear strength value and the equivalent shear strength parameters measured from experiments were higher than those predicted by analytical relationships.

**Keywords** Stone column · Stress concentration ratio · Equivalent shear strength · Stone column arrangement · Direct shear device

## 1 Introduction

Stone column installation is one of the most popular methods of soil improvement and improving subsurface soil condition. Among the benefits of using stone columns are increased slope stability, increased bearing capacity and shear strength, and reduced settlement and consolidation time. Stone column-reinforced ground performs as a composite system with higher strength and stiffness compared to the initial soil bed (Alamgir et al. 1996; Murugesan and

Rajagopal 2010). Stone columns are generally designed to carry vertical loads applied by the overlying structures. However, in cohesive ground or loose non-cohesive ground, lateral flow (Barksdale and Bachus 1983) might occur in soil and result in lateral thrust in stone columns. Lateral flow causes stone column to bear shear stresses. Several studies including analytical and numerical (Hosseinpour et al. 2014; Nazari and Ghazavi 2014; Zhang et al. 2013; Chen et al. 2015; Castro 2017) and experimental (Deb et al. 2011; Vekli et al. 2012; Ghazavi and Nazariafshar 2013; Nazariafshar et al. 2017, 2019; Mehrannia et al. 2018) investigations have been carried out to evaluate the vertical bearing capacity of stone columns. The obtained results from these studies show that the most important factors influencing the bearing capacity of stone columns are area replacement ratio, type of the soil surrounding stone column, friction angle of stone material and stone column length. However, few studies have been conducted to investigate the shear strength of grounds improved with stone columns. Gniel and

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Bouazza (2008) used triaxial tests to model reinforced stone columns and evaluate the effect of key parameters such as reinforcing length and reinforcing materials in increasing shear strength. Results showed an increase in specimen stiffness and strength and reduced lateral expansion. Najjar et al. (2010) used triaxial device to experimentally investigate the increase in bearing capacity of soft clay due to the installation of sand column in the middle of the specimen with different diameters and heights. Results showed that with the increase in diameter and length of stone columns, the strength of specimen increased. Although stress concentration ratio is an important parameter in calculations, it is not possible to be calculated in triaxial tests.

Limited direct research has been performed to evaluate the shear strength of grounds reinforced with stone columns. Murugesan and Rajagopal (2008) performed a series of plane strain experimental tests to evaluate the behavior of non-reinforced and reinforced stone columns under shear loading and reported that shear strength value increased due to the presence of non-reinforced and reinforced stone columns. Mohapatra et al. (2016) performed several direct shear tests which are performed on granular columns with and without encasement in a shear box. Tests are conducted at different normal pressures. Two different diameters of columns, three types of encasements and three different plan configurations are studied in this research work. Increase in shear strength was observed with an increase in area replacement ratio.

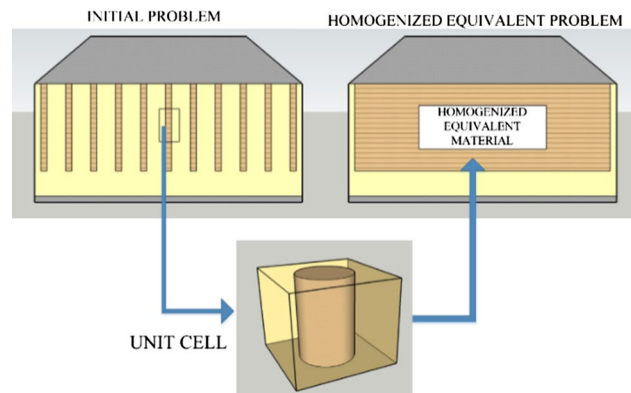
Calculating stress concentration ratio is very effective in predicting settlement parameters and analyzing stability in grounds improved with stone columns. However, there is not any exact solution available that can provide an appropriate estimation of stress concentration ratio. The performed experimental studies on stone columns have focused on determining stress concentration ratio under vertical loads (Barksdale and Bachus 1983; Goughnour and Bayuk 1979; Ambily and Gandhi 2006; Han and Ye 2001; Ng 2014); and not any of the experimental research determined the value of stress concentration ratio in stone columns subjected to both vertical and shear loads.

Various methods have been proposed to simplify the geometry of stone column problems and to avoid complicated calculations. One of these methods is the equivalent area method. In this approach, stone column and its surrounding soil are replaced with a homogenous soil with equivalent parameters. Equivalent parameters are obtained by calculating the weighted average of soil and stone column material parameters. One of the influencing parameters in calculating equivalent parameters by analytical relationships is stress concentration ratio. The value of stress concentration ratio has not been calculated in laboratory studies under shear loads, and it was considered to be the conservative value of 1. This study was the first study to record the real

variation of stress concentration ratio in stone columns during the experiment under vertical and shear loads, using a special instrument installed on top of the specimen. In this study, the shear strength of soft clay reinforced with stone column was experimentally investigated in direct shear device. The effects of different parameters including stone columns arrangement, area replacement ratio, stone columns material, and vertical stress were investigated. Moreover, stress concentration ratio was evaluated for different arrangements of stone columns, and the results were used to evaluate and compare the difference between the equivalent shear strength and shear strength parameters obtained from experiments and those predicted by analytical relationships. This study was the first study to compare the strength parameters obtained from experiments and those predicted by analytical relationships. Results showed that using analytical relationships is conservative, and it is required that the accurate value of stress concentration ratio be calculated and used in the relationships.

## 2 Equivalent Shear Strength Concept

In evaluating the bearing capacity, settlement and stability of grounds improved with stone columns, the stone column and the surrounding soil are considered as a unit cell. Unit cell model is widely used in numerical analyses. The unit cell in stone column problems is heterogeneous due to the presence of stone column and the surrounding soil. One of the simplifying methods is to convert the heterogeneous unit cell to a homogenous one. In homogenization method, the stone column and the surrounding soil are replaced with an equivalent homogenous soil with improved properties (Fig. 1). Using this method simplifies the geometry of the problem. The simplest approach of calculating the parameters of equivalent homogenous soil is to calculate the weighted average



**Fig. 1** Principle of the homogenization method applied to column-reinforced soils

of soil and column parameters at the corresponding area replacement ratio.

The equivalent shear strength in unit cell can be calculated by Eq. (1):

$$\tau_{ave} = A_r \cdot \tau_c + (1 - A_r) \cdot \tau_s \tag{1}$$

where  $\tau_{ave}$  is equivalent shear strength,  $A_r$  is area replacement ratio,  $\tau_c$  is shear stress of stone column material, and  $\tau_s$  is shear stress of bed material.

Area replacement ratio ( $A_r$ ) which is defined as the ratio of stone column cross section area to the area of the entire improved zone is introduced by Eq. (2):

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

where  $A_c$  is the total cross section area of stone columns, and  $A_s$  is the cross section area of the soil surrounding stone columns.

Regardless of the effect of stress concentration ratio, shear strength parameters in equivalent shear strength method can be calculated using Eq. (3):

$$[\tan \varphi]_{ave} = \frac{\gamma_c \cdot A_r \cdot \tan \varphi_c + \gamma_s \cdot (1 - A_r) \cdot \tan \varphi_s}{\gamma_{ave}} \tag{3}$$

In Eq. (3)  $\gamma_{ave} = A_r \gamma_c + (1 - A_r) \gamma_s$ , and  $\gamma_{ave}$  is equivalent unit weight,  $\gamma_c$  is unit weight of stone column material,  $\gamma_s$  is unit weight of bed material,  $\varphi_{ave}$  is equivalent friction angle,  $\varphi_c$  is friction angle of stone column material, and  $\varphi_s$  is friction angle of bed material.

Cooper and Rose (1999) and Abusharar and Han (2011) proposed Eq. (4) for equivalent friction angle without considering the effect of stress concentration ratio:

$$\varphi_{ave} = \tan^{-1} (A_r \cdot \tan \varphi_c + (1 - A_r) \cdot \tan \varphi_s) \tag{4}$$

Cooper and Rose (1999), Christoulas et al. (1997), and Mestar and Riou (2004) proposed Eq. (5) for calculating equivalent friction angle, and Eq. (6) for calculating equivalent cohesion, without considering the effect of stress concentration ratio:

$$\varphi_{ave} = A_r \varphi_c + (1 - A_r) \varphi_s \tag{5}$$

$$C_{ave} = A_r C_c + (1 - A_r) C_s \tag{6}$$

where  $C_{ave}$  is equivalent cohesion,  $C_c$  is cohesion of stone column material, and  $C_s$  is cohesion of bed material.

As presented in Eqs. (3)–(5), the effect of stress concentration ratio is not considered in calculating shear strength parameters, and all shear strength parameters of equivalent soil are calculated assuming that the stress concentration ratio is equal to 1. Priebe (1978) proposed Eq. (7) to include

the effect of stress concentration ratio in calculating equivalent internal friction angle:

$$\varphi_{ave} = \tan^{-1} (\omega \cdot \tan \varphi_c + (1 - \omega) \cdot \tan \varphi_s) \tag{7}$$

In Eq. (7)  $\omega = \frac{A_r n}{1 + A_r(n-1)}$ ,  $\varphi_{ave}$  is equivalent friction angle,  $\varphi_c$  is friction angle of stone column material,  $\varphi_s$  is friction angle of bed material,  $A_r$  is area replacement ratio, and  $n$  is stress concentration ratio.

Stress concentration ratio can be presented by a dimensionless factor ( $n$ ) given by Eq. (8):

$$n = \frac{\sigma_c}{\sigma_s} \tag{8}$$

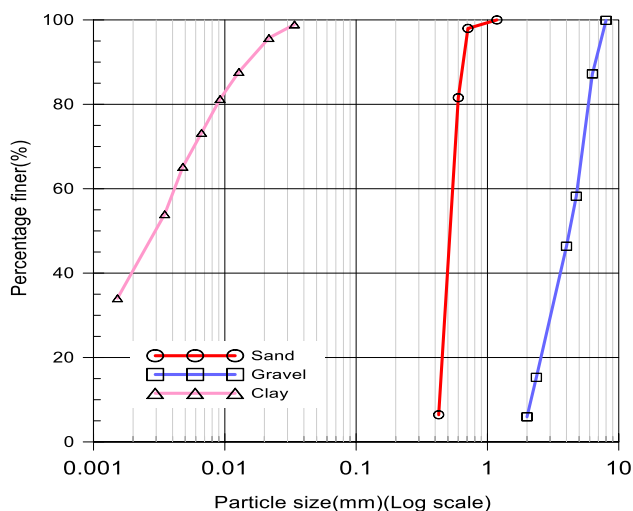
where  $n$  is stress concentration ratio,  $\sigma_c$  is the stress acting on stone column, and  $\sigma_s$  is the stress acting on soil mass.

### 3 Material Properties

In this study, clay material was used as bed material and 2 different materials including crushed gravel and fine-grained sand were used as stone column material. Clay material properties are illustrated in Table 1. To determine the water content corresponding to the desired cohesion, a series of small direct shear tests were performed on specimens with different water contents. Accordingly, at water content value of 25.4%, the obtained shear strength parameters of clayey soil were  $c = 21$  kPa and  $\varphi = 6$ . Large direct shear tests were carried out to determine the shear strength parameters of stone column materials. Properties of sand and crushed gravel are presented in Table 1. The gradation curve of stone column materials and clay bed material is illustrated in Fig. 2. Given that the diameters of model stone columns were smaller than those in the field, it was required that the particle dimensions be reduced by an appropriate scale to simulate the real behavior of stone columns. Nayak (1983)

**Table 1** Properties of materials

Parameters	Clay	Sand	Gravel
Specific gravity	2.7	2.66	2.7
Liquid limit (%)	44	–	–
Plastic limit (%)	20	–	–
Maximum dry unit weight (kN/m <sup>3</sup> )	17	17.5	17
Minimum dry unit weight (kN/m <sup>3</sup> )	15.1	15.4	14.7
Bulk unit weight (kN/m <sup>3</sup> )	18	16.5	16.5
Water content (%)	29.2	–	–
Relative density (%)	–	56	80
Cohesion	21	3	11
Friction angle	6°	31°	35°
Unified system classification (USCS)	CL	SP	GP



**Fig. 2** Particle size distribution for stone column and clay materials

and Fattah et al. (2010) proposed that the ratio of column diameter to stone particle diameter should be between 1/6 and 1/7. However, studies by Fox (2011), Stoeber (2012) and Mohapatra et al. (2016) showed that a ratio of 1/10 is suitable. In addition, according to ASTM-D-4767, for triaxial test specimens, the ratio between the largest stone particle diameter and sample diameter should be 1/6. In this study, the ratio of particle diameter to stone column diameter was chosen as 1/6, and based on the stone column diameters presented in Table 3, stone particles with diameters ranging from 2 to 8 mm were selected.

## 4 Experimental Testing

### 4.1 Testing Device

A large direct shear device with in-plane dimensions of 305\*305 mm and height of 152.4 mm was used for physical modeling and doing the experiments (Fig. 3). All specimens were sheared under a constant horizontal displacement rate of 1 mm/min. One of the objectives of this study was to determine the stress concentration ratio of stone columns with different arrangements (Table 2). For this purpose, the large direct shear device was modified and two miniature load cells with capacity of 5 kN were used to record the forces imposed on stone column and the surrounding soil. The miniature load cells were mounted on the rigid loading plate with dimensions of 305\*305 mm<sup>2</sup> and thickness of 30 mm. Schematic image of installation miniature cells is illustrated in Fig. 4. Four different steel loading plates were provided for the stone column arrangements tested in this study. Two miniature load cells were placed on each loading plate, one for measuring the stress acting on the stone



**Fig. 3** Direct shear device and installed equipment

**Table 2** Summary of experimental tests

Test no.	Stone column arrangement	Area replacement ratio Ar (%)	Test name
1	CL	0	CL
2	Center—diameter 124.5 mm	13.3	CL-G-C-13.3%
3	Center—diameter 145 mm	17.7	CL-G-C-17.7%
4	Center—diameter 169 mm	24	CL-G-C-24%
5	Square—diameter 62.5 mm	13.3	CL-G-SQ-13.3%
6	Square—diameter 72.5 mm	17.7	CL-G-SQ-17.7%
7	Triangle—diameter 72.5 mm	13.3	CL-G-TR-13.3%
8	Triangle—diameter 83.5 mm	17.7	CL-G-TR-17.7%
9	Center—diameter 124.5 mm	13.3	CL-S-C-13.3%
10	Center—diameter 145 mm	17.7	CL-S-C-17.7%
11	Center—diameter 169 mm	24	CL-S-C-24%
12	Square—diameter 62.5 mm	13.3	CL-S-SQ-13.3%
13	Square—diameter 72.5 mm	17.7	CL-S-SQ-17.7%
14	Triangle—diameter 72.5 mm	13.3	CL-S-TR-13.3%
15	Triangle—diameter 83.5 mm	17.7	CL-S-TR-17.7%
16	Gravel	100	G
17	Sand	100	S

SQ square, TR triangle, C center, CL clay, G gravel, S sand

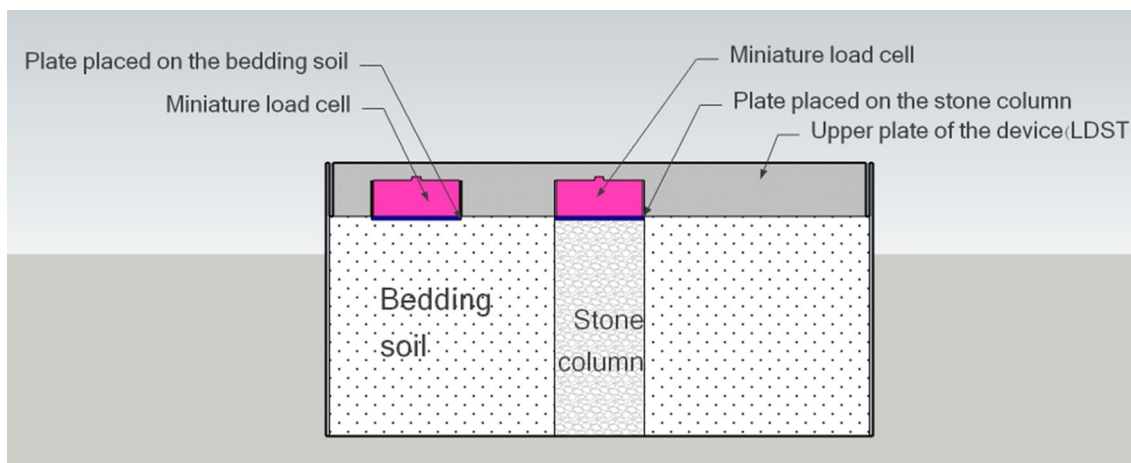


Fig. 4 Schematic image of installation of miniature cells

column, and the other for measuring the stress acting on the surrounding soil. For each arrangement pattern, one of the load cells was placed on the stone column and the other one was placed on the soil, and the stress concentration ratio was calculated. To accurately record the applied loads, a plate with 2 mm thickness was placed on stone column and the surrounding soil at load cell location after constructing the specimen.

### 4.2 Testing Program

The main aim of this research was to study the variation of stress concentration ratio and its relationship with shear strength parameters of stone column-reinforced cohesive soil subjected to vertical and shear loading. Table 2 summarizes the details of the experiments performed in this study.

Experiments were performed at three different area replacement ratios (13.3, 17.7 and 24%), and three different arrangements of stone columns (single, square and triangular). Experiments were carried out under normal stresses of 35, 55 and 75 kPa to obtain Mohr–Coulomb failure envelope. For example, installation pattern and location of stone columns with area replacement ratio of 17.7% are illustrated in Fig. 5. In Table 2, abbreviation letters are used to name the tests. The first letter of the names represents soil bed material, the second letter represents column material, the third letter indicates installation pattern, and the last number refers to replacement ratio. In this study, 4 series of experiments were carried out to ensure the repeatability of the achieved results, and excellent consistency was observed between the results obtained from different experiments.

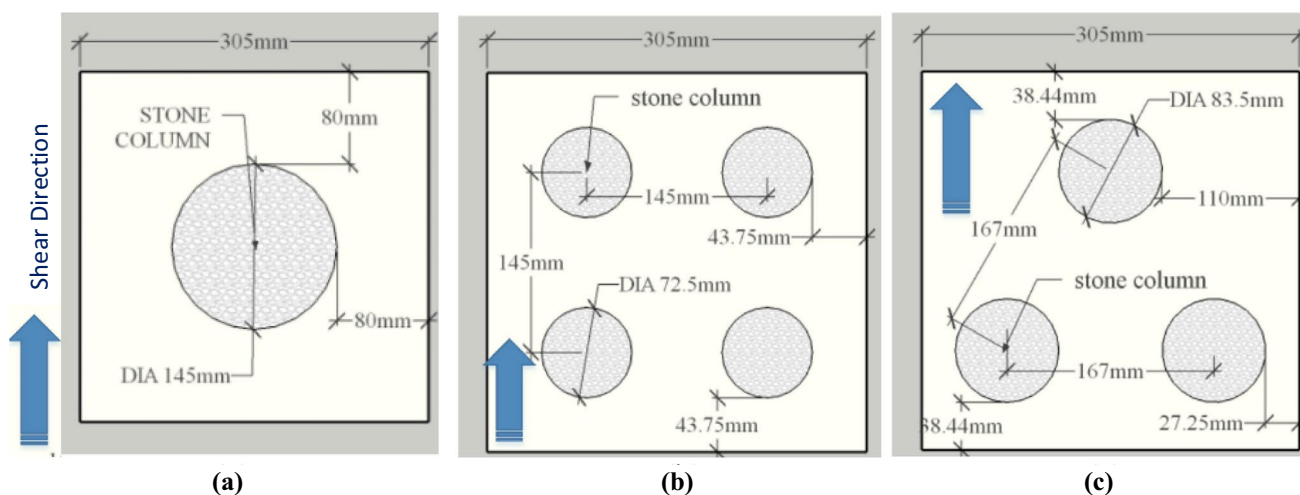


Fig. 5 Layout of a single, b square, c triangular arrangement pattern stone columns

All tests were continued until the lower part of direct shear device reached a horizontal displacement of 30.5 mm, which was equal to 10% of sample length (ASTM D-3080 standard). To avoid boundary effects, stone columns were installed at a distance of at least 38.5 mm from the inner walls of the box in shear direction (Fig. 6). Considering this limitation, the maximum distance between stone columns was selected as 2D (D was equal to stone column diameter).

### 4.3 Clay Bed

For preparation of clay bed, first the amount of water required for the moisture content of 25.4% was added to the soil and the bag was sealed. The soil specimens were then kept for at least 1 week, and in the meantime, the specimen was kneaded several times to ensure the uniform distribution of moisture throughout the soil. Before preparing the clay bed, the inner walls of the direct shear device were covered with a thin layer of grease to minimize friction between clay and the walls. As the experiments were carried out in undrained condition, a thin layer of plastic was placed at the bottom and side walls of the device to avoid moisture content loss during the experiments. The soil was compacted in the direct shear box in five 3-cm-thick layers. For this purpose, the inner walls of the box were divided into five 3-cm-thick segments, and an amount of soil proportional to a 3-cm clay layer with dry unit weight of  $18 \text{ kN/m}^3$  was weighed, placed in the box and compacted using a special hammer. A special hammer with mass of 2 kg and contact area of  $15 \times 15 \text{ cm}^2$  was used for soil compaction. A series of steel bars with diameter of 10 mm and length of 10 mm were installed under the hammer to knead each clay layer properly during compaction, to eliminate trapped air voids and to improve the compaction quality and bond of clay layers. After preparing the clay bed, the surface of clay layer was

leveled. In all experiments, samples were taken from clayey soil before, during and after the experiment to control water content and to ensure that the desired value was achieved.

### 4.4 Installation of Stone Column

In this study, all stone columns were installed by replacement method. Open-ended steel pipes with height of 200 mm and diameters equal to stone columns diameters were used to construct stone columns. The wall thickness of steel pipes was selected as 2 mm to reduce disturbance during pipe installation. The inner and outer surfaces of the pipes were coated by a thin layer of oil to reduce friction, to facilitate the movement of pipes in the soil layer and to avoid soil disturbance. The pipes were then pushed into the soil layer until reaching the bottom of the shear box and the soil inside the pipes were removed using a spiral steel auger and then the pipes were pulled out slowly from the ground. Stone column material was weighed based on the material unit weight ( $16.5 \text{ kN/m}^3$ ) and poured into the holes and compacted in five 3 cm-thick layers with the same compaction energy of  $67 \text{ kJ/m}^3$  in all tests. The observations during compaction showed that this energy level provided the desired unit weight without causing lateral expansion of the stone columns.

## 5 Results

### 5.1 The Effect of Using Stone Columns on Shear Strength

Figures 7, 8, 9, 10, 11, 12 and 13 illustrate variations of shear stress versus horizontal displacement for stone columns with different materials (sand and gravel), different

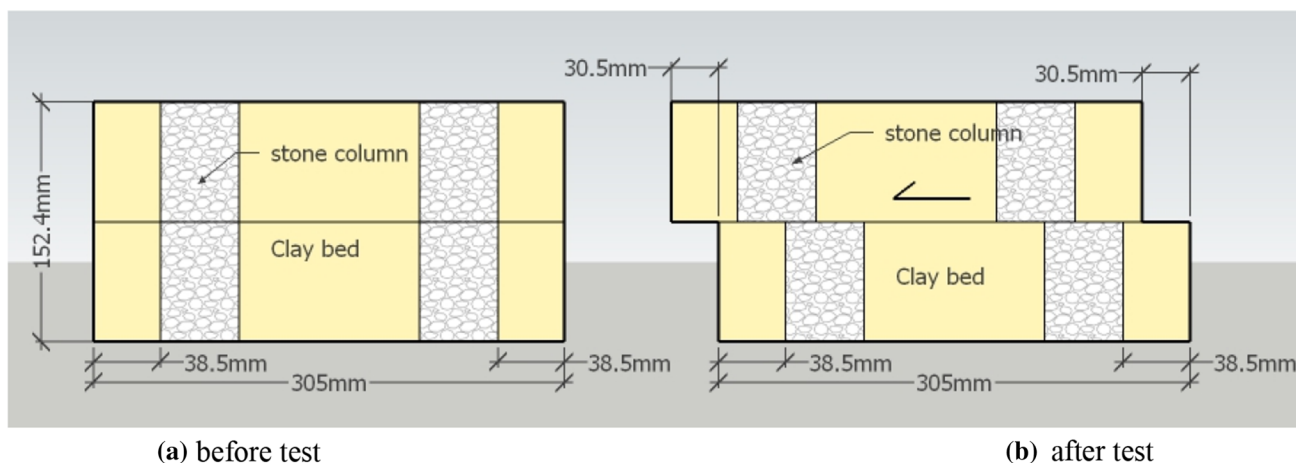


Fig. 6 Minimum distance between stone column and inner wall of shear box in square and triangular arrangement patterns

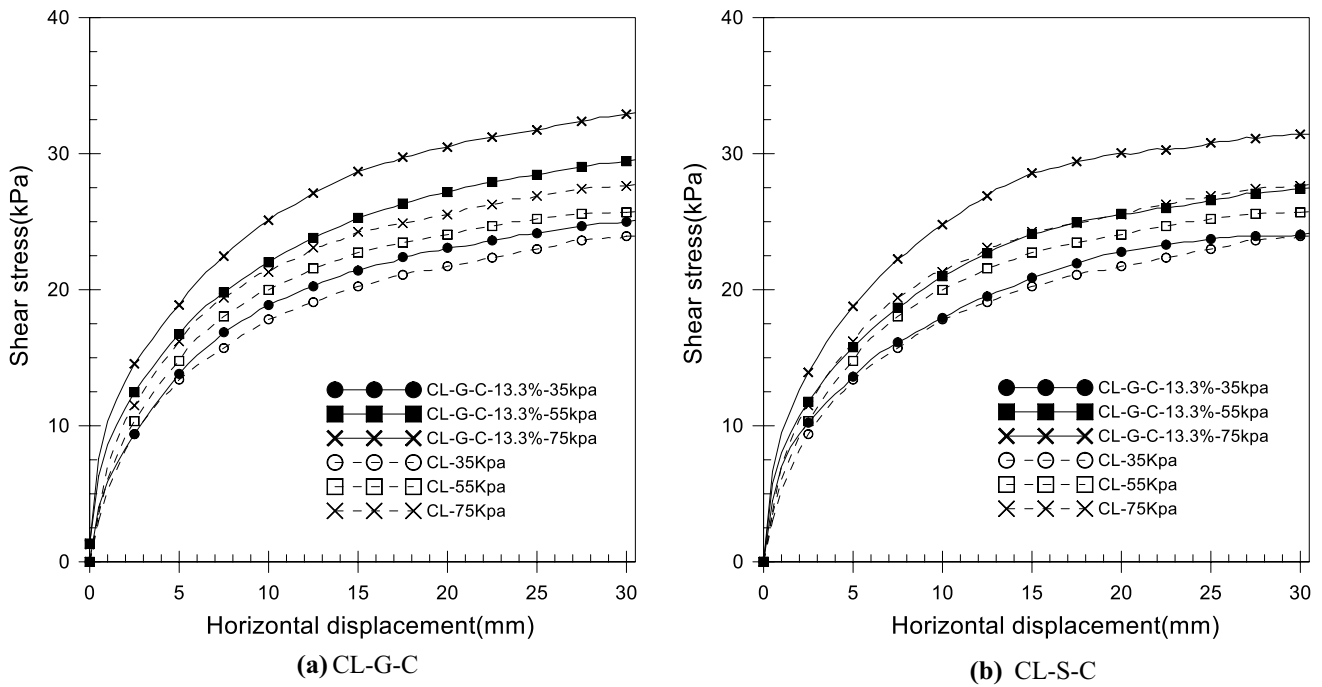


Fig. 7 Shear stress versus horizontal displacement  $Ar = 13.3\%$

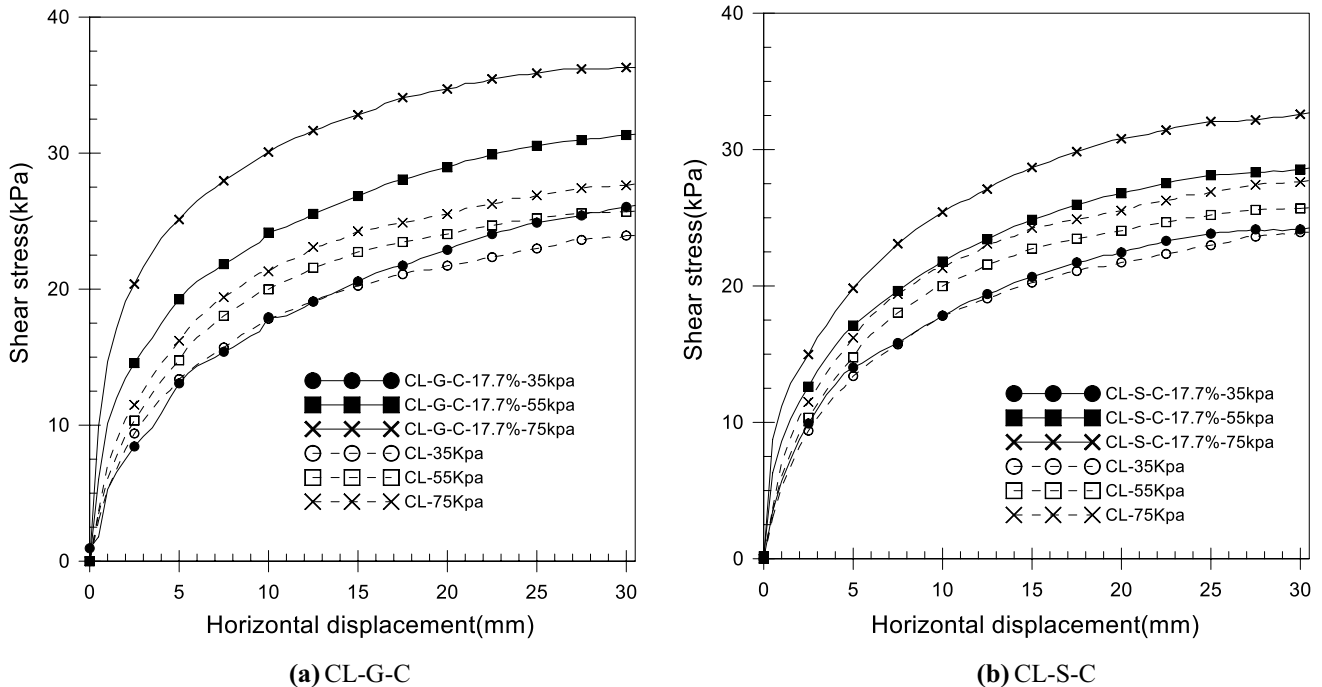


Fig. 8 Shear stress versus horizontal displacement  $Ar = 17.7\%$

installation patterns, and different area ratios, under normal stress values of 35, 55 and 75 kPa. Results from Figs. 7, 8, 9, 10, 11, 12 and 13 show that stone column installation leads to an increase in the shear strength value of soft clay

bed and the slope of shear stress-horizontal displacement curves increased, showing that the overall stiffness improved due to stone column installation. Stone column and soft clay bed perform as a compound body and mobilize higher shear

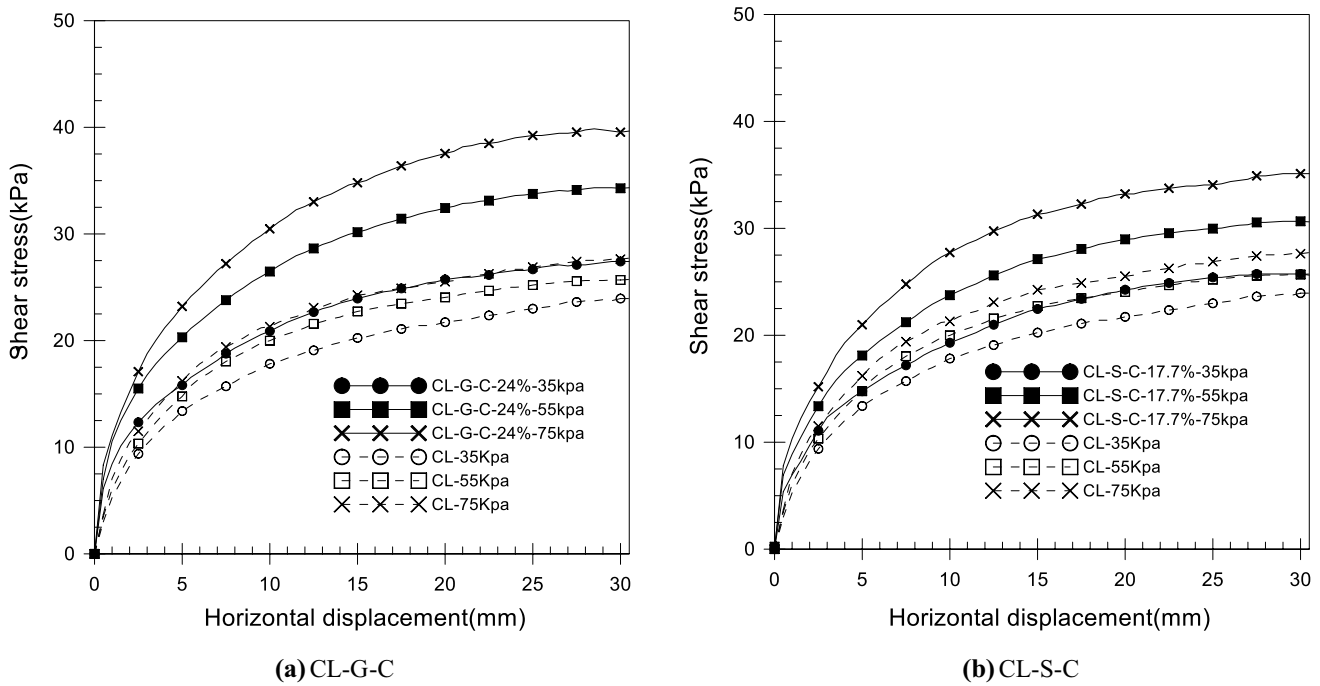


Fig. 9 Shear stress versus horizontal displacement  $Ar = 24\%$

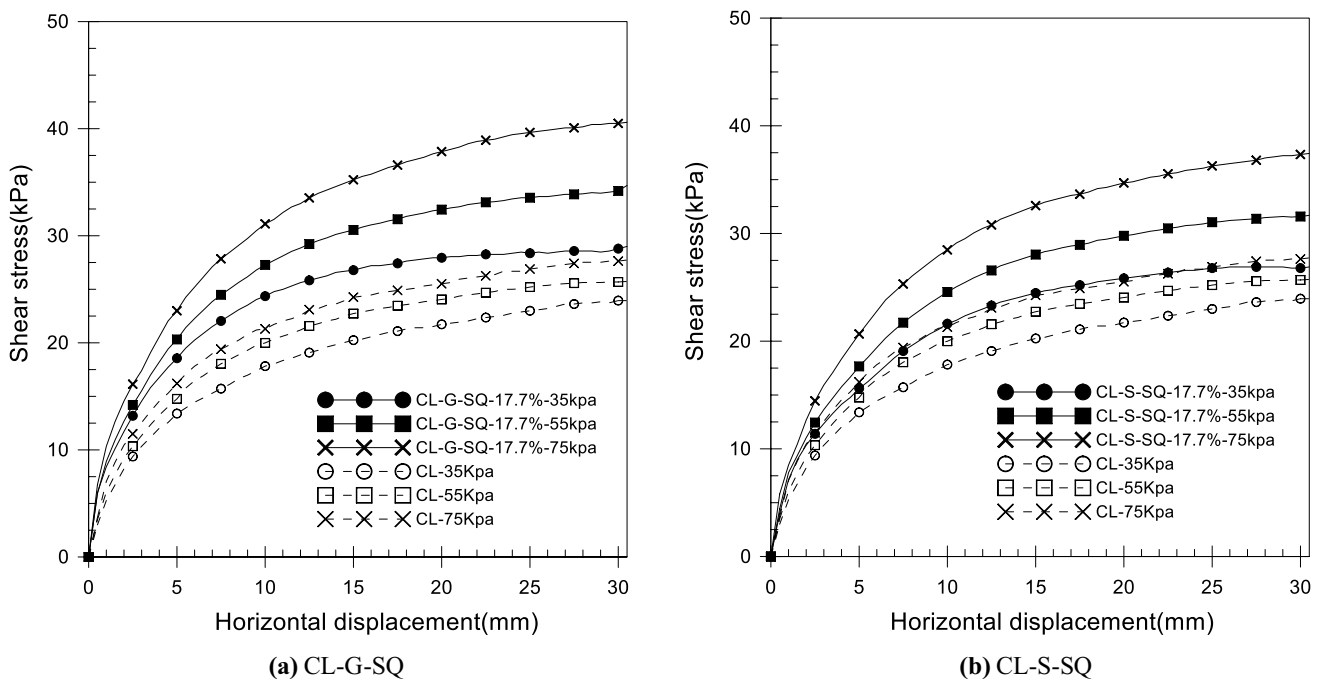


Fig. 10 Shear stress versus horizontal displacement  $Ar = 17.7\%$

strength compared to soft clay bed itself. With the increase in area replacement ratio, shear strength increases due to the increase in stone column area effective in shear surface. Results show that in tests with the same area replacement ratio, the amount of shear strength increase is higher for

stone columns installed in square and triangular patterns in comparison to single stone columns, and the highest value is for square pattern. One of the reasons of shear strength increase in square and triangular patterns is the increased confining pressure on the soil between stone columns. This



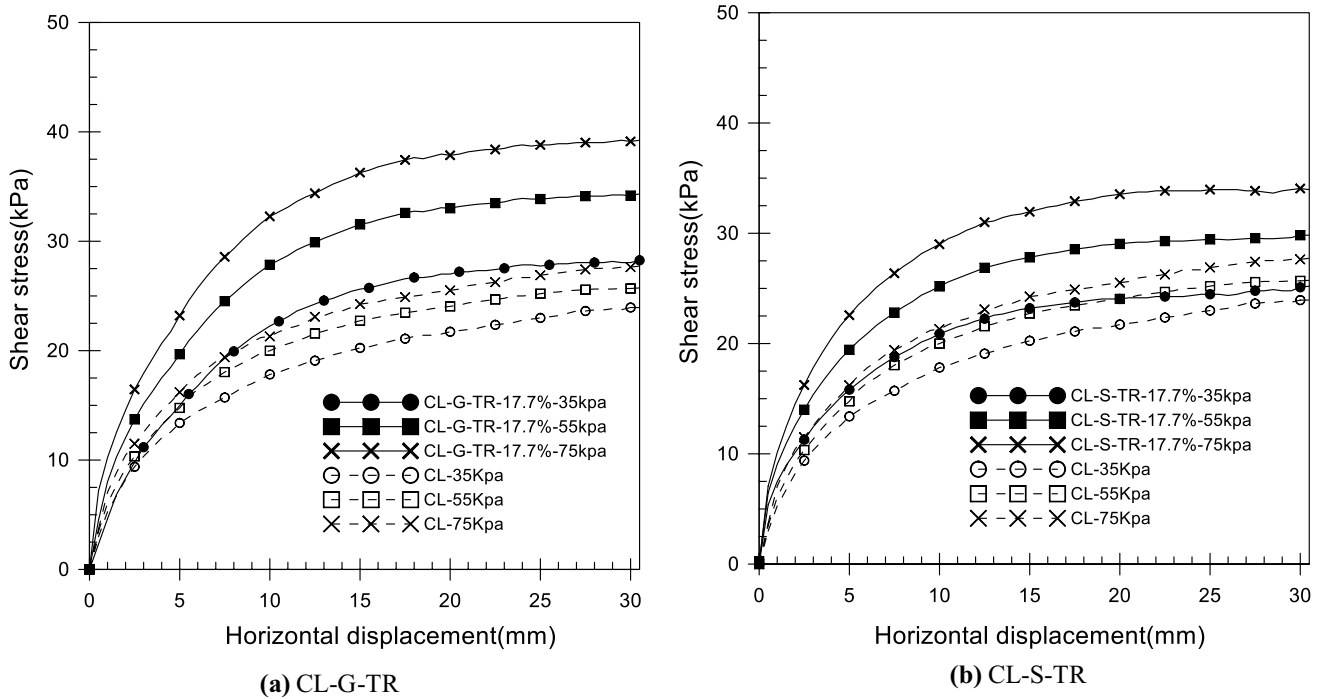


Fig. 11 Shear stress versus horizontal displacement  $Ar = 17.7\%$

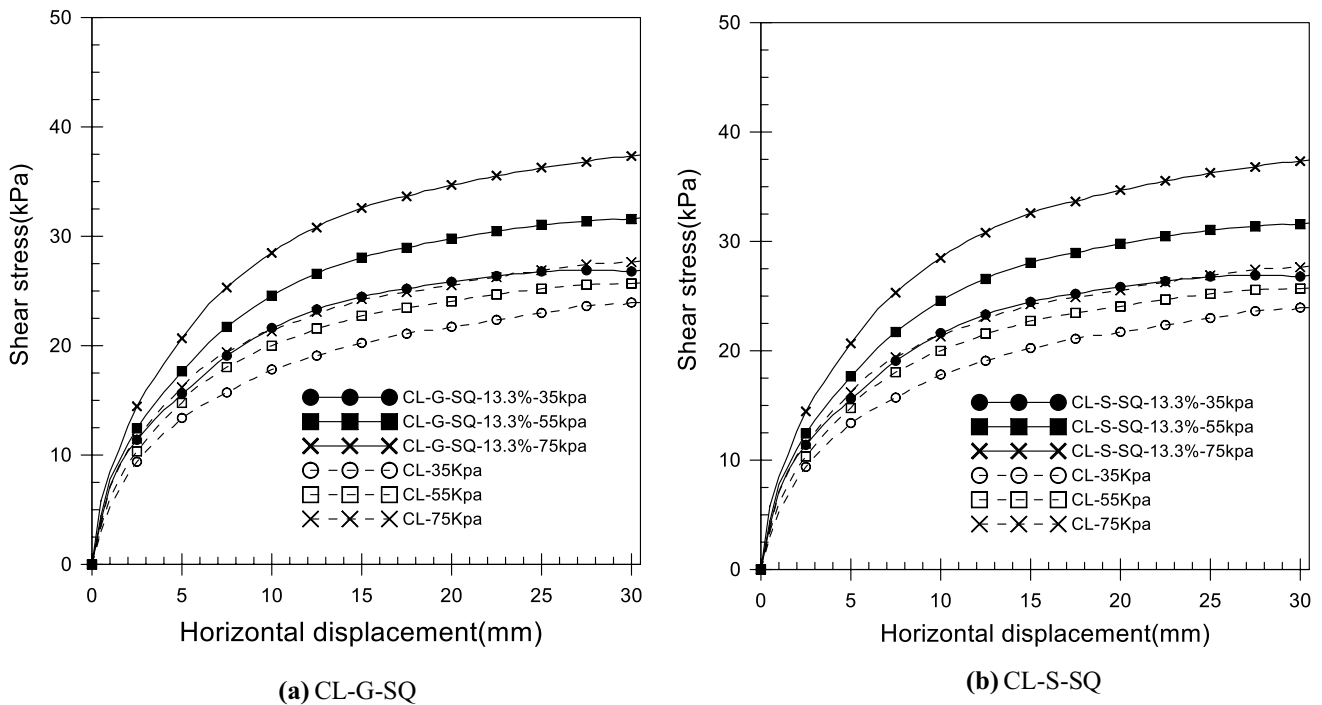


Fig. 12 Shear stress versus horizontal displacement  $Ar = 13.3\%$

makes the soil between stone columns mobilize higher shear strength compared to the soil surrounding single stone column. Another reason is the increase in lateral surface area

of stone columns resulted from changing the arrangement from single column to square and triangular patterns. The increased lateral surface area improves the lateral force on

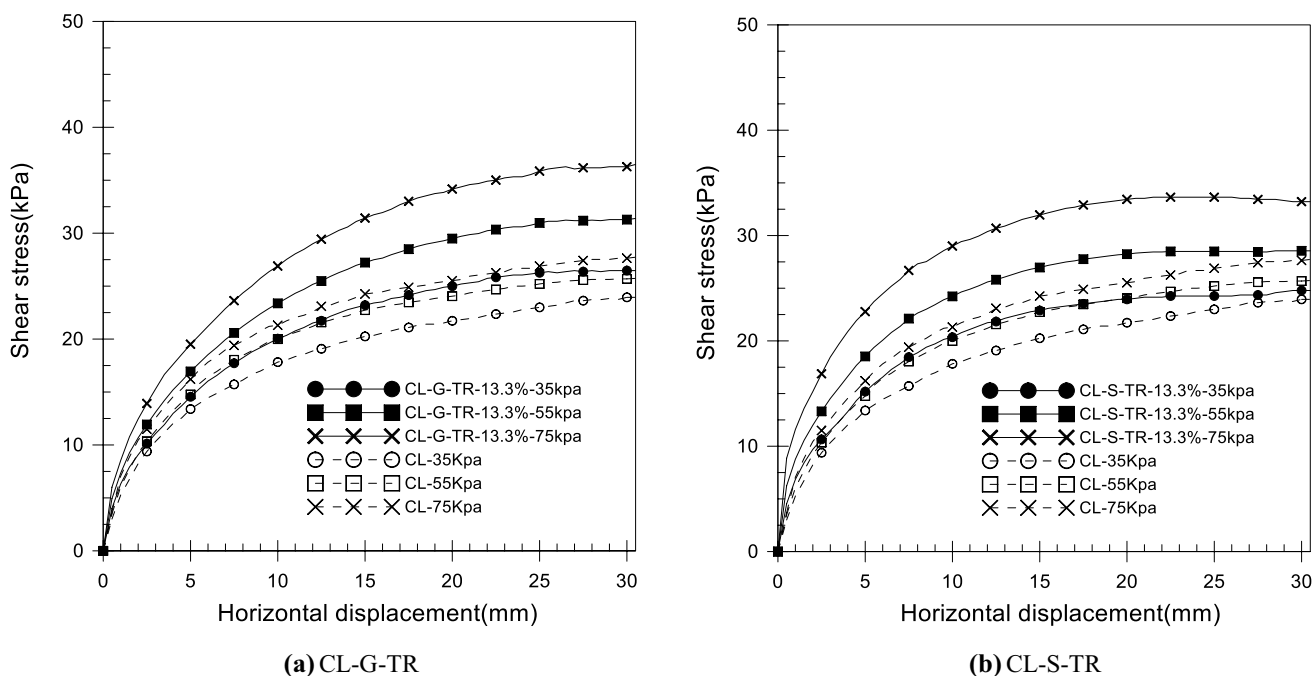


Fig. 13 Shear stress versus horizontal displacement  $A_r = 13.3\%$

stone column and results in a higher shear strength mobilization of stone material. Results from Figs. 7, 8, 9, 10, 11, 12 and 13 show that the amount of increase in shear strength was not significant for area replacement ratios lower than 15%. However, for area replacement ratios higher than 15%, the amount of increase in shear strength was noticeable.

## 5.2 Improvement of Shear Strength Parameters

Shear strength parameters obtained from tests with different stone column installation patterns and different area replacement ratios are presented in Table 3. Results showed that the shear strength parameters of soil bed were improved by stone column installation. According to Table 3, for stone columns with the same replacement ratio, the maximum increase in friction angle refers to stone columns with square patterns and the minimum value refers to single stone columns. Friction angle had an increasing trend with increase in area replacement ratio, particularly at area ratios higher than 15%. However, the increased value was slight at area replacement ratios lower than 15%. Also, using gravel columns resulted in higher shear strength parameters compared to using sand columns.

Shear strength obtained from experiments and average shear stress obtained from Eq. (1) for different stone column materials, installation patterns, and area ratios are presented in Figs. 14, 15 and 16. Results show that shear stress increases with increase in normal stress and replacement ratio. Also, the amount of increase in shear strength is

maximum for square pattern and minimum for single stone columns. Figures 14, 15 and 16 illustrate the comparison between the equivalent shear strength values obtained from experiments and those obtained from analytical relationships (Eq. 1). Results show that in all installation patterns, shear strength values obtained from experiments are higher than those predicted by analytical relationships.

The presented results show the difference between shear strength and friction angle values predicted by analytical relationships and those obtained from experiments. The difference between friction angle values predicted by Eqs. (3)–(5) and those obtained from experiments is illustrated in Table 4. Results show that for single stone columns with replacement ratios lower than 15%, shear strength parameters obtained from experiments are in good agreement with those predicted by Eqs. (3)–(5). However, the difference between these results increases with the increase in replacement ratio. For square and triangular patterns, shear parameters obtained from experiments are higher than those predicted by Eqs. (3)–(5). In other words, the analytical relationships underestimate the value of shear strength in square and triangular patterns at all area replacement ratios. This difference can be explained by the following reasons: 1. Although shear strength increases by stone column installation, the soft soil bed surrounding stone column cannot provide enough lateral pressure to mobilize shear strength of stone column material. However, analytical relationships assume that the shear strength of stone column material is fully developed.

**Table 3** Shear strength parameters obtained from experiments

Test no.	Test name	Area replacement ratio Ar (%)	Cohe-sion (kPa)	Internal friction angle (degree)
1	CL	0	21	6
2	CL-G-C-13.3%	13.3	18	11
3	CL-G-C-17.7%	17.7	17	14
4	CL-G-C-24%	24	18	17
5	CL-G-SQ-13.3%	13.3	19	15
6	CL-G-SQ-17.7%	17.7	19	17
7	CL-G-TR-13.3%	13.3	18	14
8	CL-G-TR-17.7%	17.7	19	16
9	CL-S-C-13.3%	13.3	18	10
10	CL-S-C-17.7%	17.7	18	12
11	CL-S-C-24%	24	18	14
12	CL-S-SQ-13.3%	13.3	18	14
13	CL-S-SQ-17.7%	17.7	18	15
14	CL-S-TR-13.3%	13.3	18	12
15	CL-S-TR-17.7%	17.7	18	13
16	G	100	12	35
17	S	100	5	31

2. Analytical relationships do not consider stone column properties and the interaction between stone column and the surrounding soil. 3. The analytical relationships used for predicting equivalent shear strength parameters are based on one-dimensional equal strain assumption, and their validity for 3D calculations should be investigated. 4. Analytical relationships do not consider stress concentration ratio ( $n$ ). Equations (3)–(5) are proposed based on the assumption that stress concentration ratio is equal to 1. However, in reality, the value of stress concentration ratio is higher than 1.

Table 5 presents the difference between cohesion values predicted by analytical relationships (Eq. 6) and those obtained from experiments with different stone column installation patterns. Results show that, for all stone column installation patterns, the cohesion values predicted by analytical relationships are almost equal to those obtained from analytical relationships. It is worth mentioning that in calculating cohesion values from Eq. (6), the effect of cohesion of stone material has been included.

### 5.3 The Effect of Stress Concentration Ratio on Equivalent Shear Strength

The values of stress concentration ratio corresponding to different stone column installation patterns, replacement ratios and normal stresses are illustrated in Figs. 17, 18, 19, 20, 21, 22 and 23. Results show that the value of stress concentration ratio at first increases when the load is applied, then reduced during the experiment, and finally reaches the value of 1 at the end of the experiments. Stress concentration ratio values obtained from experiments shows that the stress concentration ratio decreases with increase in normal stress. The decrease in value of stress concentration ratio is due to the lack of adequate lateral confining pressure from the surrounding soft clay layer, and by increasing vertical stress, bulging occurs in stone column. Also, the value of stress concentration ratio in square pattern is higher than that in triangular pattern. Stress concentration ratio in gravel columns is higher than that in sand columns. One of the reasons that shear strength increases with changing installation pattern from single column to square and triangular pattern is that in square and triangular patterns the lateral surface area of stone columns increases. The increased lateral surface area improves the lateral force on stone columns and results in a higher shear strength mobilization of stone material. Stress concentration ratio decreases with increase in stone column diameter.

The effect of stress concentration ratio is not considered in Eqs. (3)–(5), and equivalent shear strength parameters are calculated assuming that the value of stress concentration ratio is equal to 1. Priebe (1978) proposed Eq. (7) to include the effect of stress concentration ratio in  $\varphi_{ave}$  calculations.

Comparison between shear strength parameters obtained from experiments and those predicted by Eqs. (3)–(5) and Eq. (7) is presented in Table 6. Results show that for single stone columns, the measured shear strength values are equal to those predicted by Eq. (7) considering stress concentration ratios of 1.2, 1.5 and 1.6 for gravel, and 1.1, 1.3, and 1.5 for sand, at replacement ratios of 13, 17.7 and 24%, respectively. For stone columns installed in square pattern, the measured shear strength values are equal to those predicted by Eq. (7) considering stress concentration ratios of 2.5 and 2.4 for gravel, and 2.5 and 2.3 for sand, at replacement ratios of 13.3 and 17.7%, respectively. For stone columns installed in triangular pattern, the measured shear strength values are equal to those predicted by Eq. (7) considering stress concentration ratios of 2.12 and 2.05 for gravel, and 1.8 and 1.6 for sand, at replacement ratios of 13.3 and 17.7%, respectively. Results show that for single stone columns stress concentration ratio is equal to 1 at replacement ratios lower than 15%, and higher than 1 at replacement ratios higher than 15%, whether gravel or sand is used as column material. In group stone columns with square and triangular arrangements,

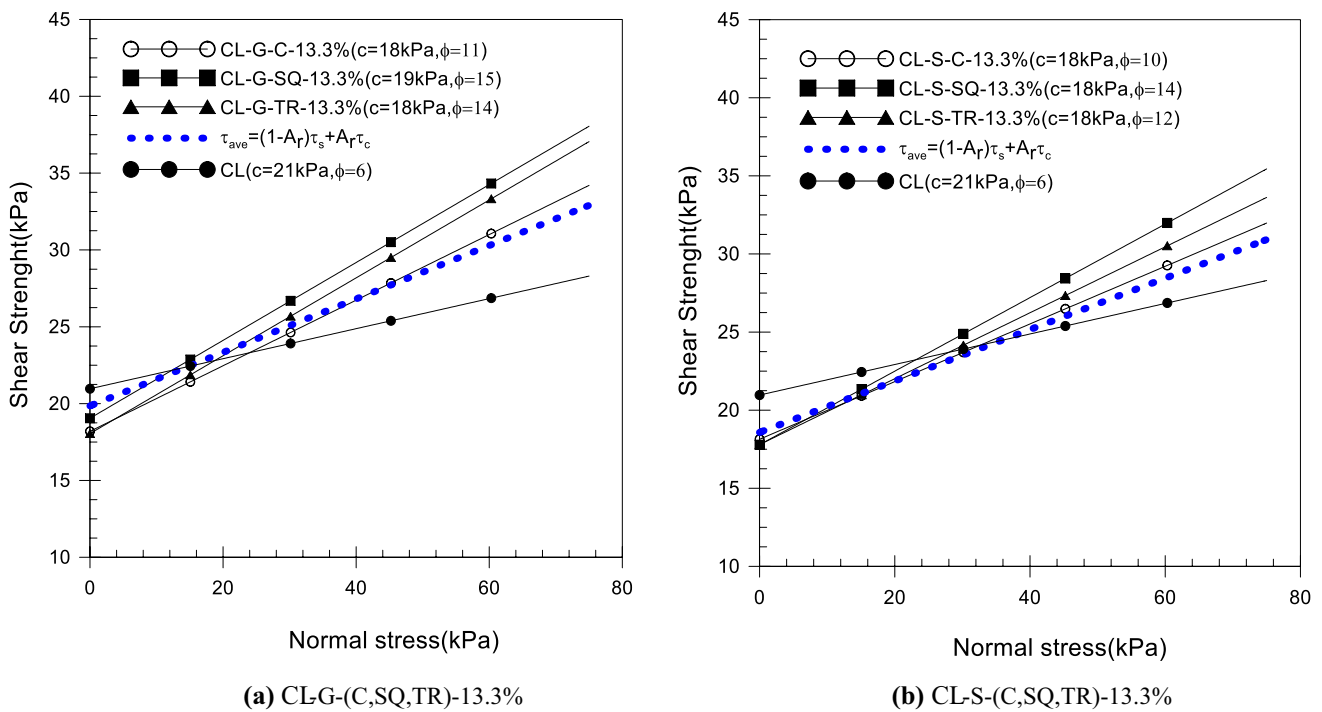


Fig. 14 Mohr–Coulomb failure envelopes

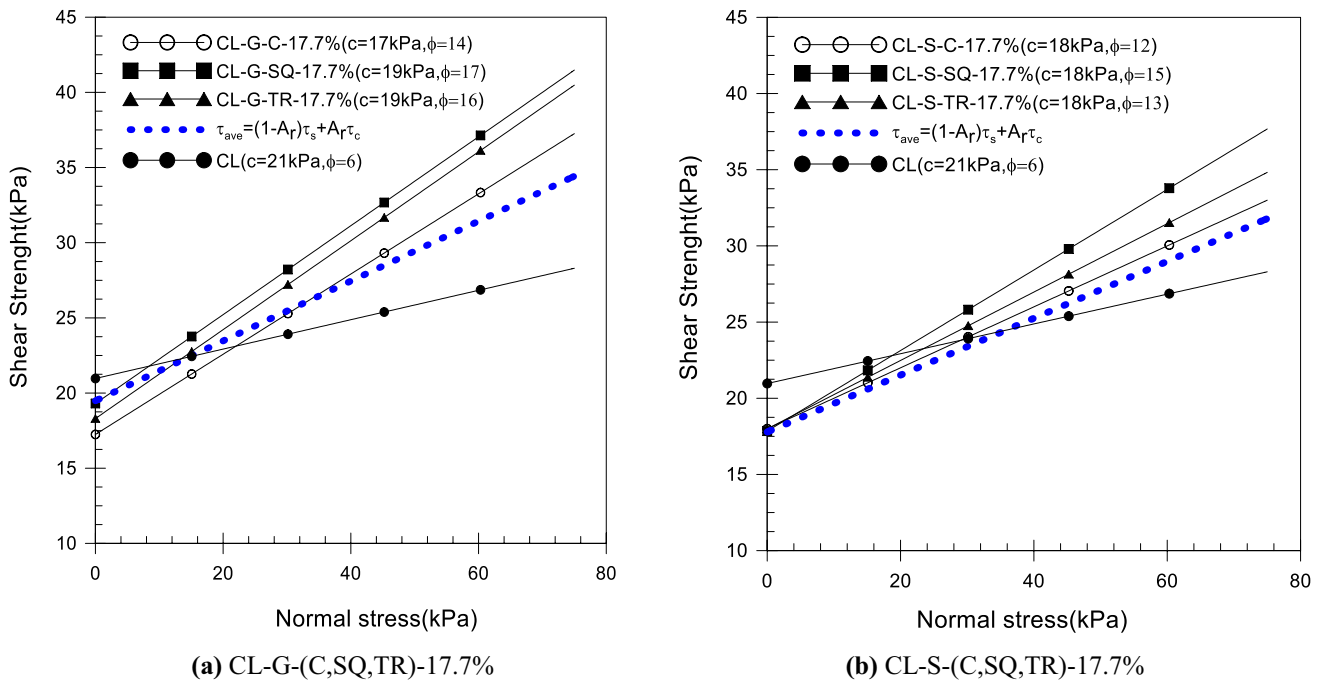


Fig. 15 Mohr–Coulomb failure envelopes

stress concentration ratio is higher than 1 whether gravel or sand is used as column material.

It can be deduced from the achieved results that using maximum stress concentration ratio in calculations leads

to unrealistic results, and using a stress concentration equal to 1 is conservative. Thus, the ratio used in analytical

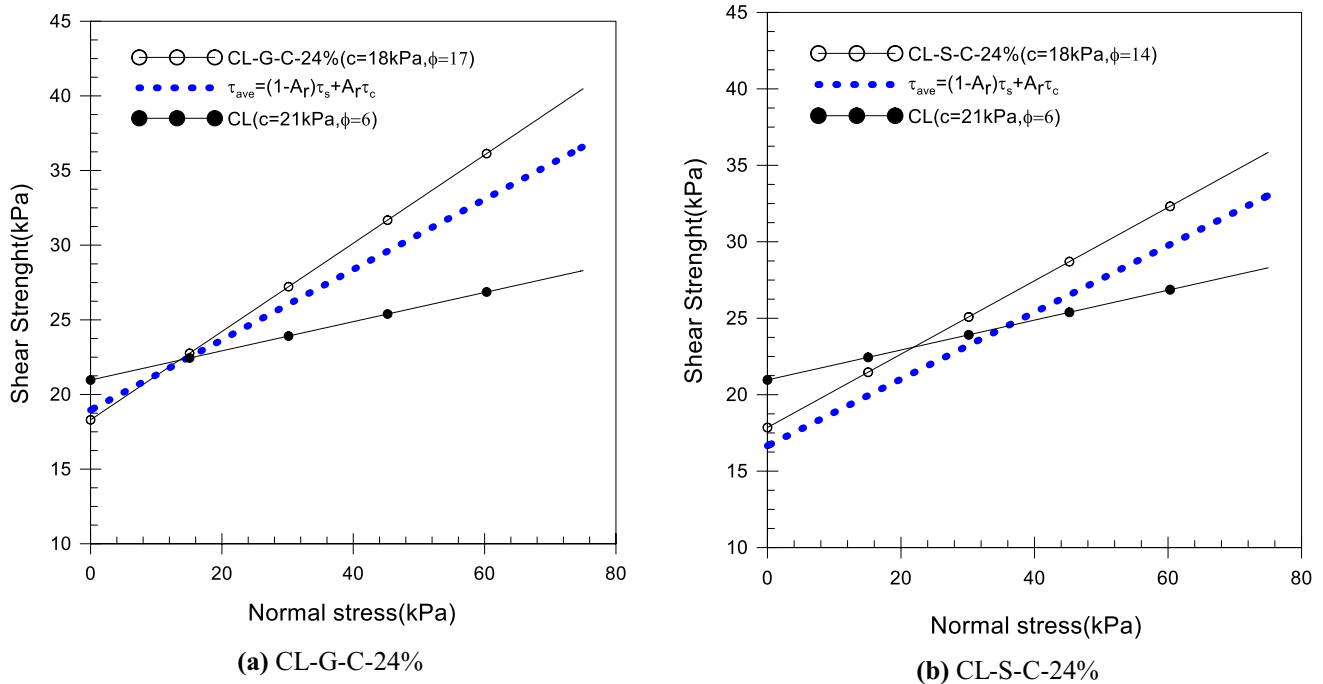


Fig. 16 Mohr–Coulomb failure envelopes

**Table 4** Difference between friction angle values obtained from experiments and analytical relations

Test no.	Test name	Internal friction angle (degree)			
		Test result	Equation (3)	Equation (4)	Equation (5)
1	CL	6	–	–	–
2	CL-G-C-13.3%	11	10	10	10
3	CL-G-C-17.7%	14	12	12	11
4	CL-G-C-24%	17	13	14	13
5	CL-G-SQ-13.3%	15	10	10	10
6	CL-G-SQ-17.7%	17	12	12	11
7	CL-G-TR-13.3%	14	10	10	10
8	CL-G-TR-17.7%	16	12	12	11
9	CL-S-C-13.3%	10	10	10	9
10	CL-S-C-17.7%	12	11	11	10
11	CL-S-C-24%	14	12	12.6	12
12	CL-S-SQ-13.3%	14	10	10	9
13	CL-S-SQ-17.7%	15	11	11	10
14	CL-S-TR-13.3%	12	10	10	9
15	CL-S-TR-17.7%	13	11	11	10
16	G	35	–	–	–
17	S	31	–	–	–

relationships should have a value between the maximum stress concentration ratio and the stress concentration ratio corresponding to 10% horizontal displacement.

## 6 Conclusions

The present work pertains to the development of understanding the behavior of stone columns subjected to lateral (shear) loading. The behavior of stone columns was

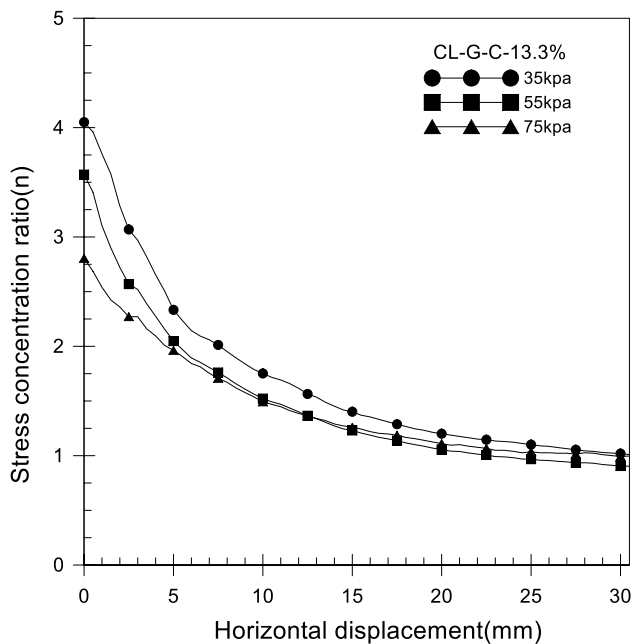
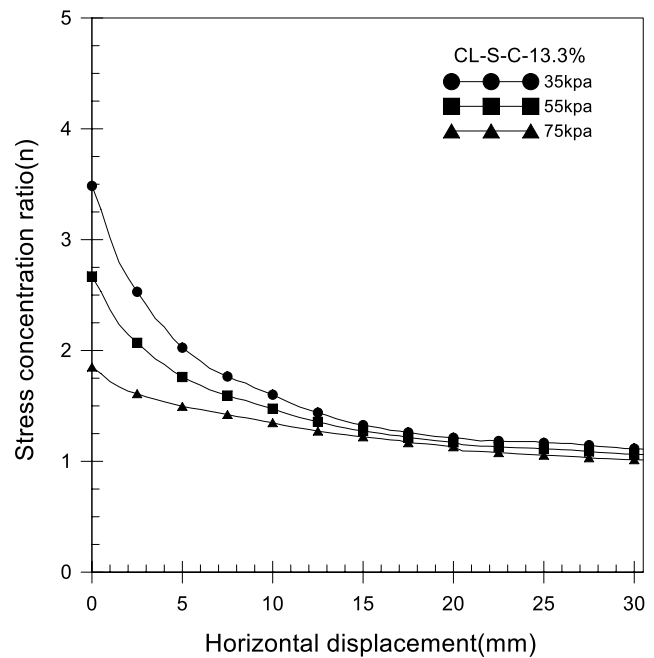
**Table 5** Difference between cohesion values obtained from experiments and analytical relations

Test no.	Test name	Cohesion (kPa)	
		Test result	Equation (7)
1	CL	21	–
2	CL-G-C-13.3%	18	19.7
3	CL-G-C-17.7%	17	19.2
4	CL-G-C-24%	18	18.6
5	CL-G-SQ-13.3%	19	19.7
6	CL-G-SQ-17.7%	19	19.2
7	CL-G-TR-13.3%	18	19.7
8	CL-G-TR-17.7%	18	19.2
9	CL-S-C-13.3%	18	18.6
10	CL-S-C-17.7%	18	17.8
11	CL-S-C-24%	18	17
12	CL-S-SQ-13.3%	18	18.6
13	CL-S-SQ-17.7%	18	17.8
14	CL-S-TR-13.3%	18	18.6
15	CL-S-TR-17.7%	18	17.8
16	G	11	–
17	S	3	–

studied using laboratory large direct shear by performing tests with different area replacement ratios, different stone column installation patterns, and different normal stresses, and their results are compared to show the improvement of

lateral load carrying capacity of stone columns. Based on the observed results, the following conclusions are made:

1. For stone columns with the same modified area ratio, the installation pattern plays a slightly important role in increasing the shear strength. Group stone columns mobilized higher shear strength compared to single stone columns. Stone columns with square arrangement mobilized the highest increase in shear strength value while single stone columns experienced the lowest. The reason of this behavior is that the lateral surface of stone columns increases when the installation pattern is changed from single column to square pattern. As a result, lateral confining pressure on the soil between stone columns increases.
2. Shear strength increases with the increase in modified area ratio in all stone column installation patterns. In gravel stone columns, shear strength increase was in the range of 18 to 25% under normal stress of 35 kPa, 11 to 30% under normal stress of 55 kPa and 21 to 46% under normal stress of 75 kPa, for different stone column installation patterns. In sand columns, shear strength increase was in the range of 4 to 13% under normal stress of 35 kPa, 8 to 25% under normal stress of 55 kPa and 14 to 36% under normal stress of 75 kPa, for different stone column installation patterns. The rate of shear strength increase was noticeable for area replacement

**(a)** CL-G-C**(b)** CL-S-C**Fig. 17** Stress concentration ratio for Ar = 13.3%

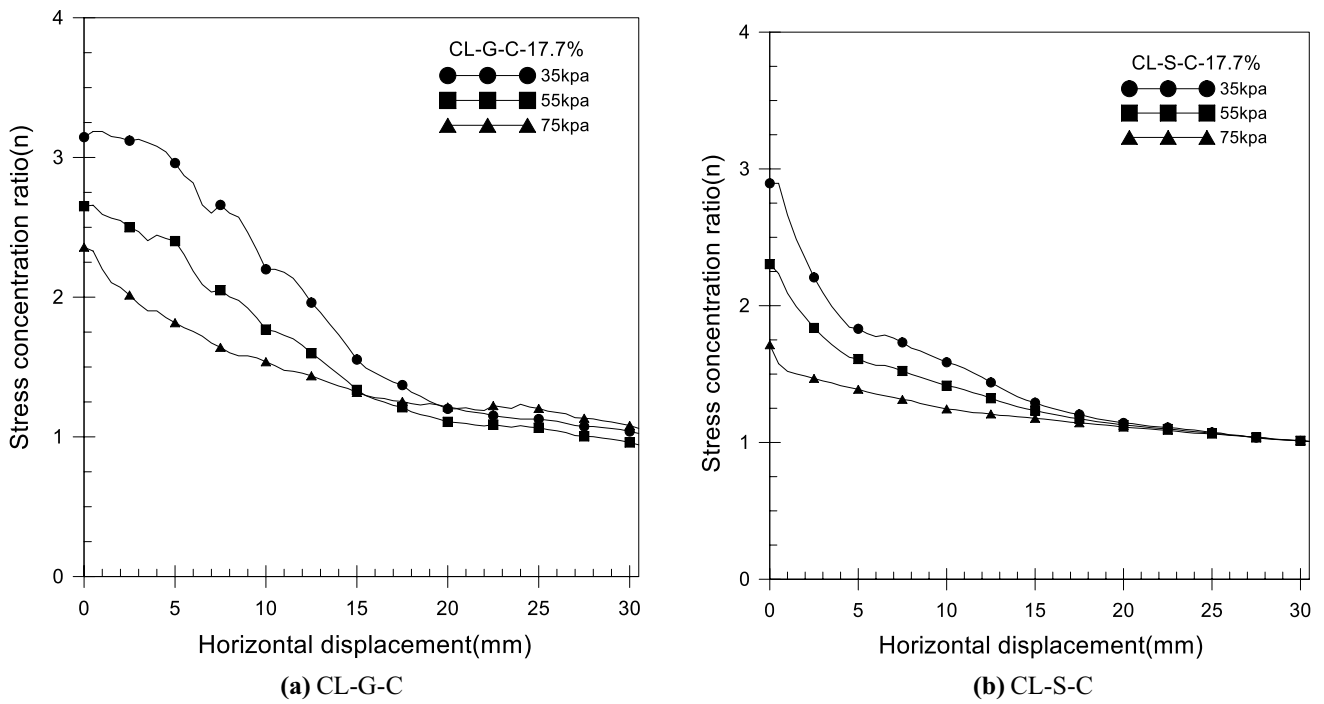


Fig. 18 Stress concentration ratio for Ar = 17.7%

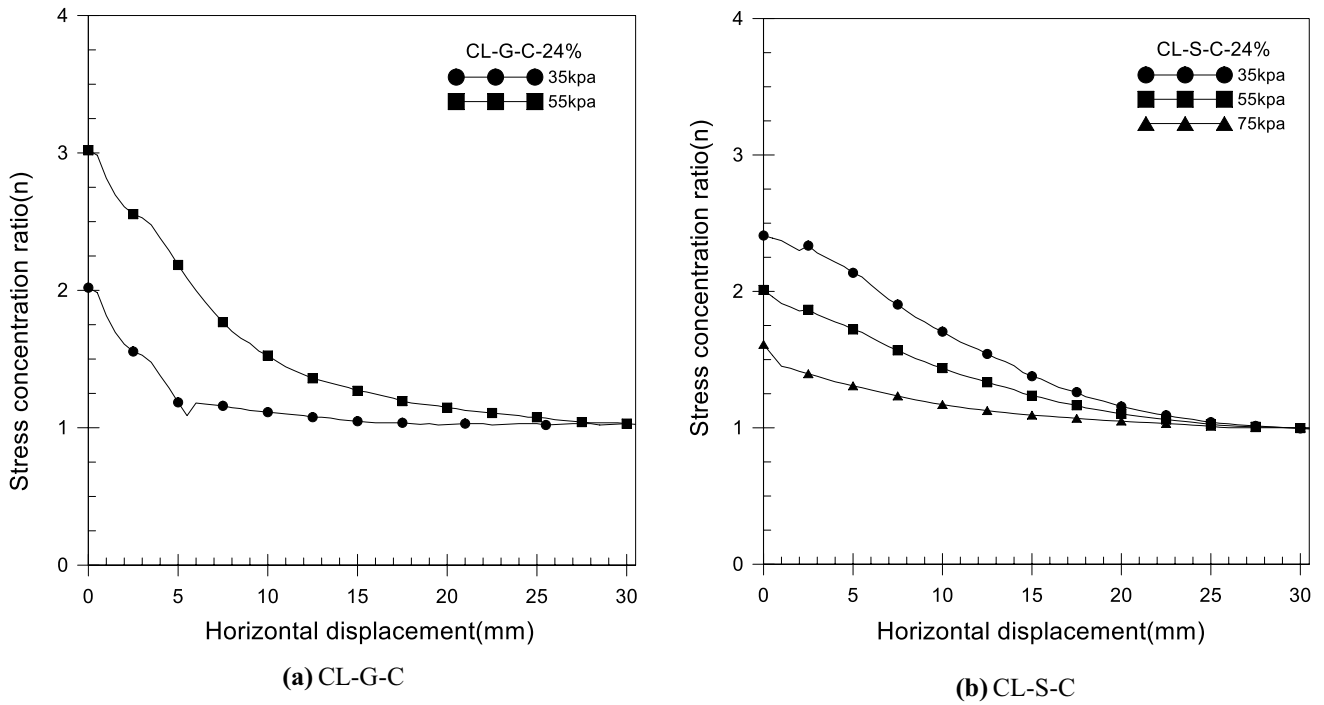


Fig. 19 Stress concentration ratio for Ar = 24%

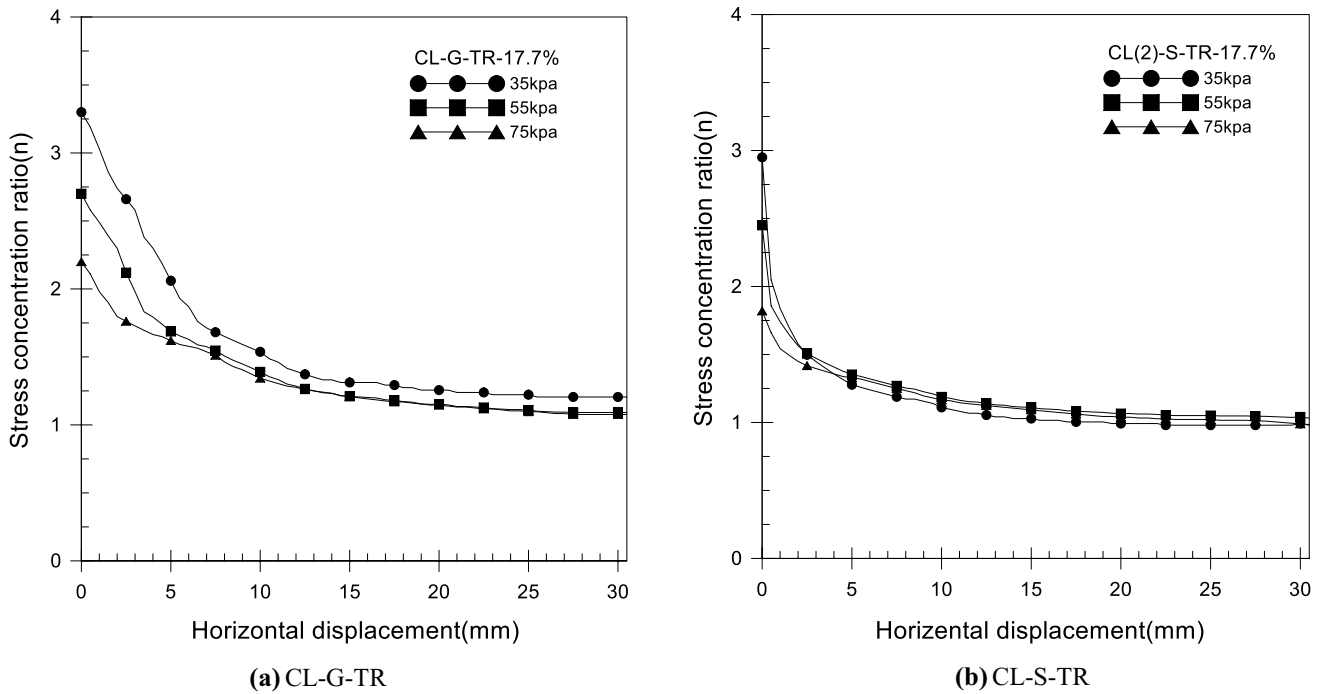


Fig. 20 Stress concentration ratio for Ar = 17.7%

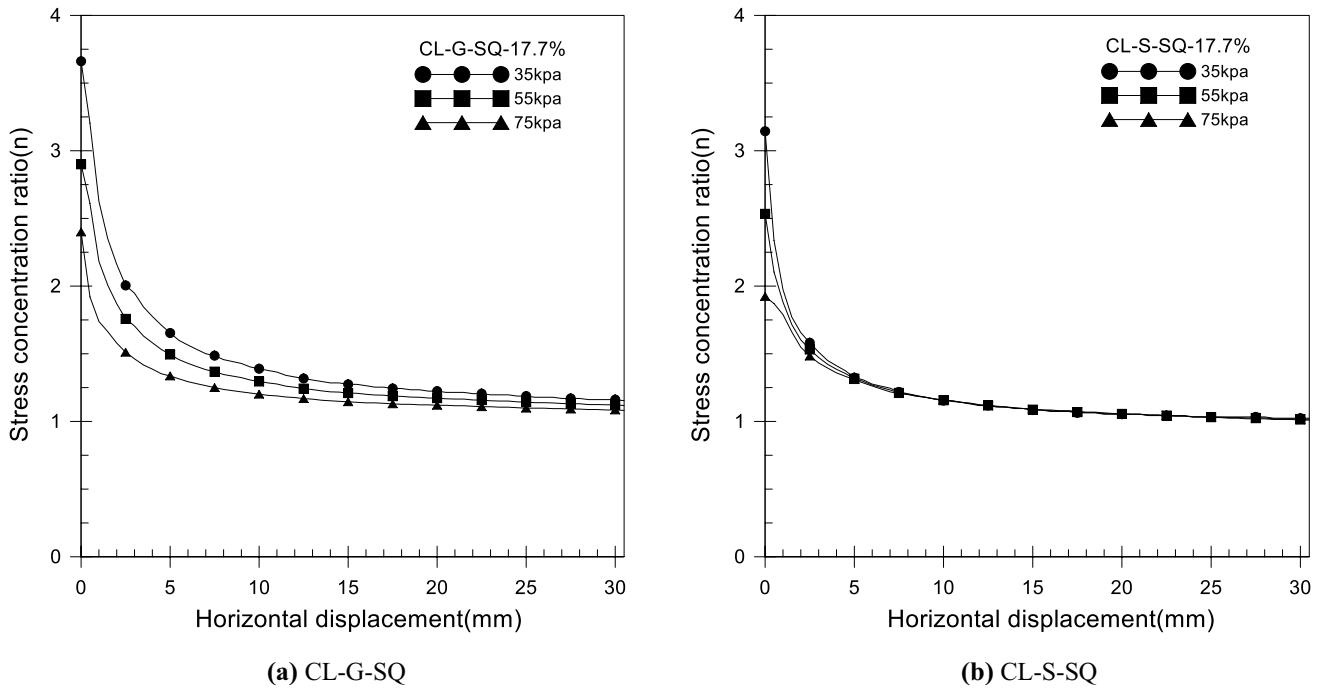


Fig. 21 Stress concentration ratio for Ar = 17.7%

ratios greater than 15%, and slight for area replacement ratios below 15%.

3. Shear strength values predicted by analytical relationships are lower than those obtained from experiments.

It is conservative to use analytical relationships to calculate shear strength parameters. It should be noted that these relationships consider that stress concentration ratio is equal to 1. Results have shown that the exact



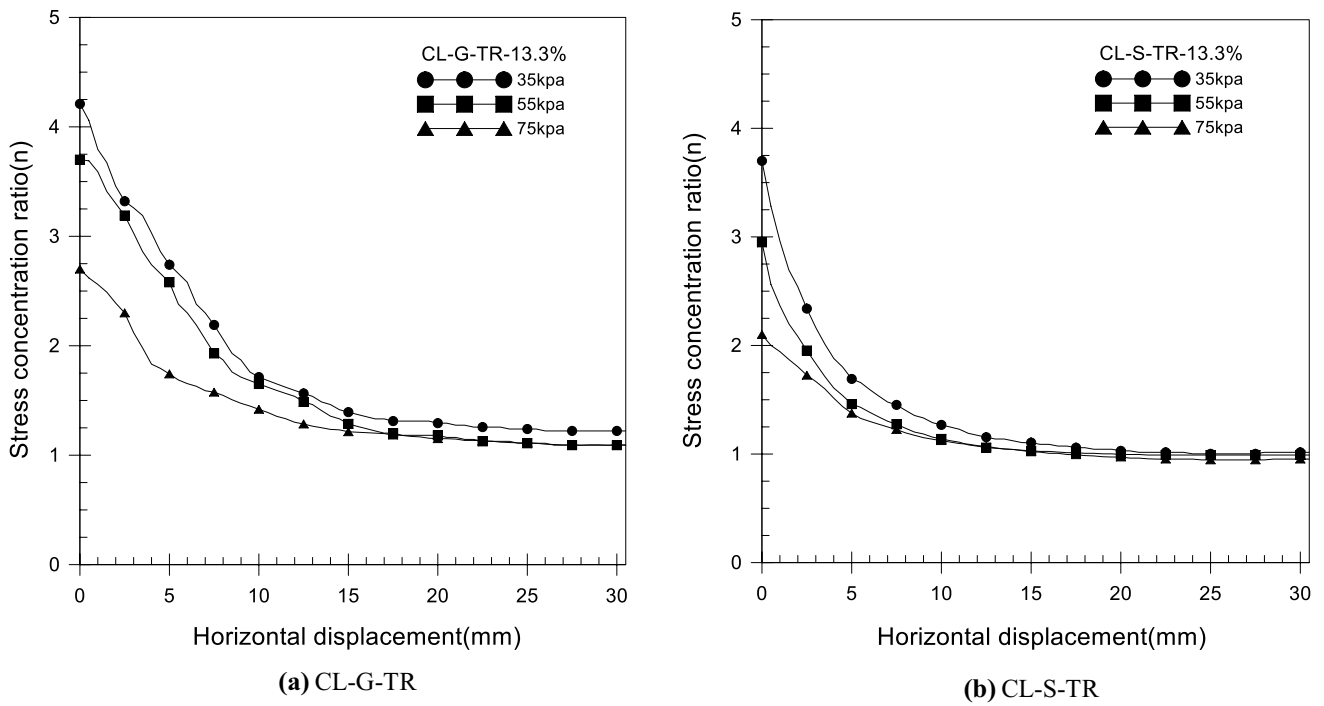


Fig. 22 Stress concentration ratio for Ar = 13.3%

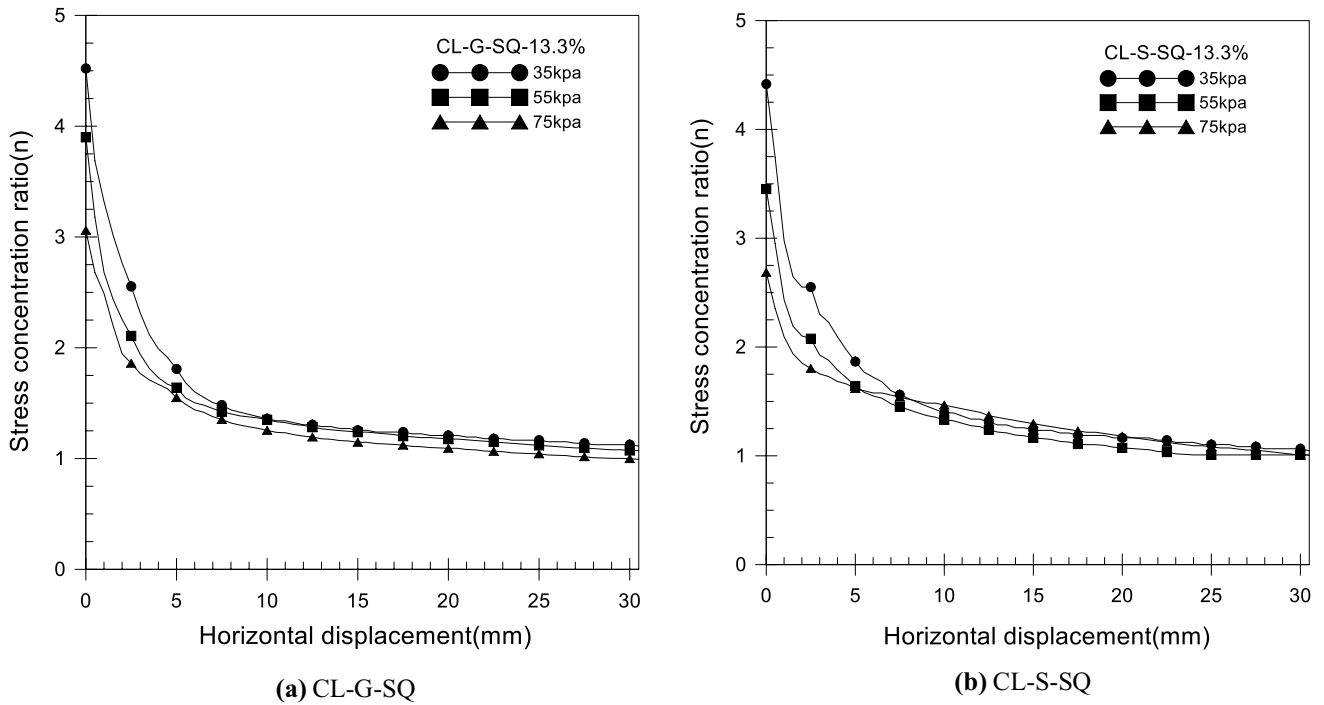


Fig. 23 Stress concentration ratio for Ar = 13.3%

**Table 6** Comparing the results of the experiments with the analytical relations

Vertical stress (kPa)	Arrangement pattern	Area replacement ratio Ar (%)	Internal friction	Stress concentration ratio ( <i>n</i> )	The friction angle resulting from analytic relations with consideration (SCR) (Eq. 7) (degree)			SCR ( <i>n</i> ) value required in relation 7	The friction angle resulting from analytic relations without consideration (SCR) (degree)						
					Max	Min	<i>n</i> =1		<i>n</i> =2	<i>n</i> =3	Equation (3)	Equation (4)	Equation (5)		
35	CL-G-C	24	17	3	1	17	1.6	13.9	18.5	21.5	1.6	12.9	13.9	12.9	
55				2	1.1			11.9	15.9	18.7		1.5	12	11.9	11.1
75				–	–			10.4	13.7	16.3		1.2	10.5	10.4	9.8
35	CL-G-C	17.7	14	3.1	1.02	14	2.4	11.9	15.9	18.7	2.4	12	11.9	11.1	
55				2.7	1			10.4	13.7	16.3		1.2	10.5	10.4	9.8
75				2.4	1.1			11.9	15.9	18.7		2.4	12	11.9	11.1
35	CL-G-C	13.3	11	4	1.1	11	3.9	10.4	13.7	16.3	3.9	10.5	10.4	9.8	
55				3.6	1			11.9	15.9	18.7		2.4	12	11.9	11.1
75				2.8	1			10.4	13.7	16.3		2.5	10.5	10.4	9.8
35	CL-G-SQ	17.7	17	3.7	1.2	17	3	10.4	13.7	16.3	3	10.5	10.4	9.8	
55				3	1.1			11.9	15.9	18.7		2.05	12	11.9	11.1
75				2.4	1.1			10.4	13.7	16.3		2.1	10.5	10.4	9.8
35	CL-G-SQ	13.3	15	4.5	1.1	15	3	11.9	15.9	18.7	3	12	11.9	11.1	
55				3.9	1			10.4	13.7	16.3		2.1	10.5	10.4	9.8
75				3	1			11.9	15.9	18.7		2.05	12	11.9	11.1
35	CL-G-TR	17.7	16	3.3	1.2	16	2.7	10.4	13.7	16.3	2.7	10.5	10.4	9.8	
55				2.7	1			11.9	15.9	18.7		2.1	10.5	10.4	9.8
75				2.2	1			10.4	13.7	16.3		2.1	10.5	10.4	9.8
35	CL-G-TR	13.3	14	4.2	1.2	14	3.7	10.4	13.7	16.3	3.7	10.5	10.4	9.8	
55				3.7	1			11.9	15.9	18.7		2.1	10.5	10.4	9.8
75				2.7	1			10.4	13.7	16.3		2.1	10.5	10.4	9.8
35	CL-S-C	24	15	2.4	1	15	2.7	12.6	16.5	19.1	2.7	11.7	12.6	12	
55				1.9	1			10.9	14.3	16.7		1.3	11	12.9	10.4
75				1.6	1			10.9	14.3	16.7		1.3	11	12.9	10.4
35	CL-S-C	17.7	12	2.9	1	12	1.9	10.9	14.3	16.7	1.9	11	12.9	10.4	
55				1.9	1			9.7	12.5	14.6		1.1	9.8	9.7	9.3
75				1.6	1			9.7	12.5	14.6		1.1	9.8	9.7	9.3
35	CL-S-C	13.3	10	3.5	1.1	10	2.7	9.7	12.5	14.6	2.7	9.8	9.7	9.3	
55				2.7	1			10.9	14.3	16.7		2.3	11	12.9	10.4
75				1.9	1			10.9	14.3	16.7		2.3	11	12.9	10.4
35	CL-S-SQ	17.7	15	3.1	1	15	2.5	10.9	14.3	16.7	2.5	11	12.9	10.4	
55				2.5	1			10.9	14.3	16.7		2.3	11	12.9	10.4
75				2	1			10.9	14.3	16.7		2.3	11	12.9	10.4

Table 6 (continued)

Vertical stress (kPa)	Arrangement pattern	Area replacement ratio Ar (%)	Stress concentration ratio (n)	Internal friction	The friction angle resulting from analytic relations with consideration (SCR) (Eq. 7) (degree)			The friction angle resulting from analytic relations without consideration (SCR) (degree)					
					Max	Min	n=1	n=2	n=3	Equation (3)	Equation (4)	Equation (5)	
35	CL-S-SQ	13.3	4.4	1	13.42	9.7	12.5	14.6	2.5	9.8	9.7	9.3	
55			3.5	1									
75			2.7	1									
35	CL-S-TR	17.7	3	1	13	10.9	14.3	16.7	1.6	11	12.9	10.4	
55			2.5	1									
75			1.8	1									
35	CL-S-TR	13.3	3.7	1	12	9.7	12.5	14.6	1.8	9.8	9.7	9.3	
55			3	1									
75			2.1	1									

- value of stress concentration ratio must be calculated and applied in the relationships.
- The comparison between shear parameters obtained from experiments and those predicted by analytical relationships show that in stone columns arrangements with area replacement ratio lower than 15%, the stress concentration ratio is equal to 1, and in stone column arrangements with area replacement ratios higher 15%, the stress concentration ratio varies between 1.5 and 1.6 for gravel columns and between 1.3 and 1.5 for sand columns. The value of stress concentration ratio is higher than 1 in stone columns with square and triangular arrangements. In stone columns with square arrangement, the stress concentration ratio varies between 2.4 and 2.5 for gravel columns and 2.3 and 2.5 for sand columns, and in triangular pattern it varies between 2.05 and 2.12 for gravel columns and between 1.6 and 1.8 for sand columns.
  - The slope increase in shear strength-horizontal displacement curves shows that the soil-stone column system has higher stiffness than soft clay bed, and this stiffness varies with modified area ratio and installation pattern. The stiffness values in square arrangement are greater than individual arrangement.

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