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A simple analytical method for calculation of bearing capacity of stone-column

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Abstract

The Stone-column is a useful method for increasing the bearing capacity and reducing settlement of foundation soil. The prediction of accurate ultimate bearing capacity of stone columns is very important in soil improvement techniques.Bulging failure mechanism usually controls the failure mechanism. In this paper, an imaginaryretaining wall is used such that it stretches vertically from the stone column edge. A simple analytical method is introduced for estimation of the ultimate bearing capacity of the stone column using Coulomb lateral earth pressure theory.Presentedmethodneeds conventional Mohr-coloumb shear strength parameters of the stone column material and the native soil for estimation the ultimate bearing capacity of stone column. The validity of the developed method has been verified using finite element method and test data. Parametric studies have been carried out and effects of contributing parameters such as stone column diameter, column spacing, and theinternal friction angle of the stone column material on the ultimate bearing capacity have been investigated.

Keywords: *Stone column, Bearing capacity, Soft soil, Bulging, Lateral earth pressure*

1. Introduction

The construction of structures such as a building, storage tanks, warehouse, earthenembankment, etc., on weak soils usually involves an excessive settlement or stability problems. To solve or reduce encountered problems, soil improvement may be considered. Various methods may be used for soil improvement. Three categories involving column type elements, soil replacement, and consolidationmay be considered [1].One effective method is stone-column referred to by other names such as granular column or granular pile.

Stone-column is useful for increasing the bearing capacity and reducing settlement of foundation soil. In addition, because of high permeability of stone column material,consolidation rate in soft clay increases. In stone-column construction, usually 15 to 35 percent of weak soil volume isreplaced with stonecolumn material. Design loads on stone-columns ordinarily vary between 200 to 500kN[1]. The confinement of stone-column is provided by the lateral stress due to the weak soil. The effectiveness of the load transmitted by stone-columns essentially depends on the lateral stress that exerts from the surrounding soft soil.

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Upon application of vertical stress at the ground surface, the stone column material and soil move downward together, resulting in stress concentration in the stone column because of higher stiffness of stonecolumn material relative to the native soil. Stone-columns are constructed usually in equilateral triangular pattern and in square pattern. The equilateral triangle pattern gives more dense packing of stone-columns in a given area.

Barksdale and Bachus[2]described three typesof failure, whichmay occur upon loading a stone column: bulging failure, shear failure, and punching failure. For bulging failure mechanism, Greenwood [3], Vesic [4], Hughes and Withers [5], Datye and Nagaraju [6], and Madhav et al [7] and for shear failure mechanism, Madhav and Vitkare [8], Wong [9], Barksdale and Bachus [2], and for punching failure mechanism, Aboshi et al[10]presented relationships for prediction of the ultimate bearing capacity of single stone-column.

The ultimate bearing capacity of stone columns originally depends on column geometry, stone column material properties, and properties of native soil. Normally the column length has a negligible effect on the long column ultimate bearing capacity. Since the applied load is transfered from the column into the surrounding native soil, a small portion of the load is transmitted to column the bottom. This has been found experimentally for long columns (Hughes and Withers [5];Pitt et al. [11]). In practice, stone-column diameter and length usually varies between 0.9-1.2 mand 4-10 m, respectively. For single isolated stone-column, with length to diameter ratio equal to or greater than 4 to 6 (long column), the most probable

failure mechanism is bulging failure.Various researchers have proposed the analysis of granular pile reinforced ground. Shahu et al. [12], [13] presented a simple theoretical approach to predict deformation behaviour of soft ground reinforced with uniform and non-uniform granular pile–mat system.Bouassida [14] , [15] presented a method for evaluation of the stone column bearing capacity by usingof limit analysis method. Lee and Pande[16]performed axi-symmetric finite elelement analysis to investigate load-settlelment characteristics of stonecolumns. They established equivalent material for in situ soil and stone column composit. In this research, they modified axi-symmetric condition for plane strain. Abdelkrim et al. [17] presented elastoplastic homogenization procedure for predicting the settlement of a foundation on a soil reinforced by stone columns. They used homogenizationstechnique and converted composite native soil and stone column to unit composit material. They also made some simplifications intheir calculation procedure. Physical model tests were also performed on stone columns (Wood et al., [18]; Ambily and Gandhi [19]).

In the present study, by using an imaginary retaining wall, a simple analytical method is developed for estimation of the bearing capacity of an isolated stone-column failed by bulging failure mechanism. Most of existing approaches for bulging mechanism need several mechanical parameters for prediction of ultimate bearing capacity. However, the new developed method, only needscohesion, internal friction angle, and density of the stone column material and native soil.

2. Bulging Failure Mechanism

In homogeneous soil reinforced by stone-columns, if the length to diameter of the column is equal to or greater than 4 to 6, the bulging failure occursat depth equal to 2 to 3 diameters of stone-columns (Fig. 1). However, there is numerical and experimental evidence indicating that even bulging can occur in shallower depth less than 2-3D(Pitt et al.[11]; Murugesan and Rajagopal [20]).Hughes et al. [21] observed the bulging failure by performing experiments.

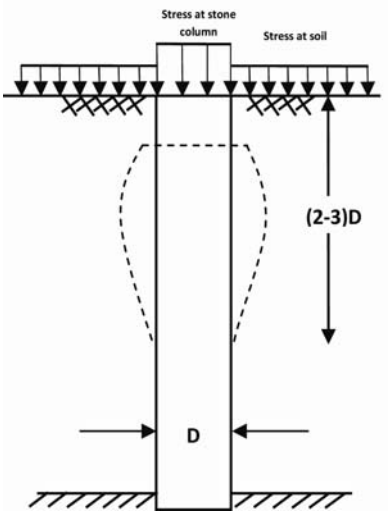

Fig. 1 Bulging failure mechanism

A limited number of theories havebeen presentedfor prediction of the ultimate capacity of a single stone-column supported by soft soilin form of $\sigma_1 = \sigma_3 K_{P_s}$. Here σ_1 is the vertical stress on stone clolumn, σ_3 is the lateral confining stress, and K_{P_s} is the passive lateral earth pressure coefficient offered by the stone column material (Greenwood [3]; Vesic [4]; Hughes and Withers [5]; Datye and Nagaraju [6]; Madhav et al. [7]). Most of early analytical solutions assume a triaxial state of stresses representing the stone-column and the surrounding soil.

The lateral confining stress that supports the stone-columns is usually taken as the ultimate passive resistance induced to the surrounding soil as the stone-column bulges outward against the soil. Since the column is assumed to be in a state of failure, the ultimate vertical stresstolerated by the stone-columnis equal to the coefficient of passive pressure, K_{P_S}, times the lateral confining stress. In other words:

$$\sigma_1 = \sigma_3 \tan^2(45 + \frac{\phi_s}{2}) = \sigma_3 \frac{1+\sin\phi_s}{1-\sin\phi_s} = \sigma_3 K_{P_s} \quad (1)$$

Where ϕ_s = internal friction angle of stone-column material.

Most of researchershave attempted to predict the value of surrounding confinement pressurein eq. (1). Vesic [4] introduced:

$$\sigma_3 = cF_c^{'} + qF_q^{'} \quad (2)$$

Wherec=cohesion, q= $(\sigma_1 + \sigma_2 + \sigma_3)/3$ =mean (isotropic) stress, at the equivalent failure depth, and $F_q^{'}$ and $F_c^{'}$ =cavity expansion factors. Vesic [4] presented a graph for calculation of expansion factors ($F_q^{'}$ and $F_c^{'}$) which are functions of the internal friction angle of the surrounding soil and the rigidity index, I_r. Vesic [4] expressed the rigidity index as:

$$I_r = \frac{E}{2(1+\nu)(c + q\tan\phi_c)} \quad (3)$$

Where E = modulus of elasticity of the surrounding soil, c = cohesion of surrounding soil, υ = Poisson's ratio of surrounding soil, and q is within the zone of failure.

Hughes and Withers [5] considered the bulging type failure in single stone-columns to be similar to the cavity expansion developed as in the case of a pressuremeter test. Therefore, eq. (4) may be used for computing σ_3 in frictionless soil as:

$$\sigma_3 = \sigma_{r_c} + c[1 + Ln\frac{E_c}{2c(1+\nu)}] \quad (4)$$

Where σ_{r_0} = total in-situ lateral stress (initial) and E_c = elastic modulus of the soil.

Eq. (4) gives the ultimate lateral stress if c, φ, E_c, and ν are known. In the present research, assuming the bulging failure mechanism, only the first two are required to determine the bearing capacity of an isolated stone-column reinforced soil.

In realstone columns are using in-group, but in the literature there is no comprehensive appraoch for estimation of the ultimate bearing capacity of group of stone columns. Barksdale and Bachus [2] presented a simple approach by approximating the failure surface with two straigth rupture lines. The develpoted approach did not consider the possibility of a local bulging failure of the individual column. Hence, the approach is only applicable for firm and stronger cohesive soils having an undrained shear strength greater than 30-40 KPa. However, it is useful for approximately determining the relative effects on ultimate bearing capacity design variables such as column diameter, spacing. However, in soft and very soft cohesive soils, the ultimate bearing capacity of stone column is predicted using the capacity of a single, isolated column located in-group and to be multiplied by the number of columns[2].

3. Developed Analytical Method

Fig. 2 illustrates a shallow foundation constructed on stone column reinforced surrounding native soil. Stone columns are usually used in rows and groups with square or triangular configurations to support raft foundations or embankments. An isolated stone column acts in an axisymmetrical ring, which is surrounded by native soil in a ring shape. The thickness of these rings may be determined such that the area replacement ratio in the model is kept constant similar to real situation in the field. In addition, the center-to-center distance between rings is kept equal to the spacing between columns in the field (Elshazly,[22][23]). When stone columns are used in groups, they may be idealized in plane strain condition. Since stone columns are constructed at center to center, S, for analysis in the plane strain condition, stone-columns are converted into equivalent and continuous strips having a width of W (Fig. 2).

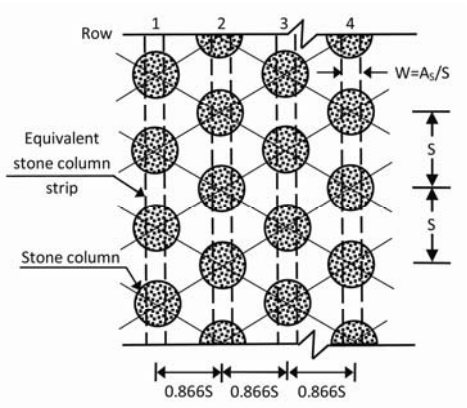

Fig. 2 Stone-column strip idealization

Others have used the conversion of three-dimensional objects to equivalent and continuous stone-column strips. For example, Barksdale et al. [2], Christoulas [24], Han et al. [25], and Abusharar et al. [26] used this idealization for analysis of slope stability reinforced with stone columns. Zahmatkesh et al. [27] used this technique for evaluating the settlement of soft clay reinforced with stone columns. Deb [28], [29] considered plane strain condition for group of stone columns for predicting the behavior of granular bed-stone column-reinforced soft ground. The above hypothesis was also used for soil-nailed walls in excavations. In this application, for numerical modeling of soil-nailed walls, nails which are actually discrete elements are replaced with ‘‘equivalent element as plate or cable’’. Therefore, discrete nails are replaced with continous element extended to one unit width ([30] and[31]).

Based on the above hypothesis, for stone column analysis, W, the width of continuous strips for each row of stone columns is determined using (Fig. 2):

$$W = \frac{A_S}{S} \tag{5}$$

Where A_S =the horizontal cross sectional area of the stone-column, S=center to center distance between two subsequent stone columns.

The failure mechanism is assumed as shown in Fig.3.

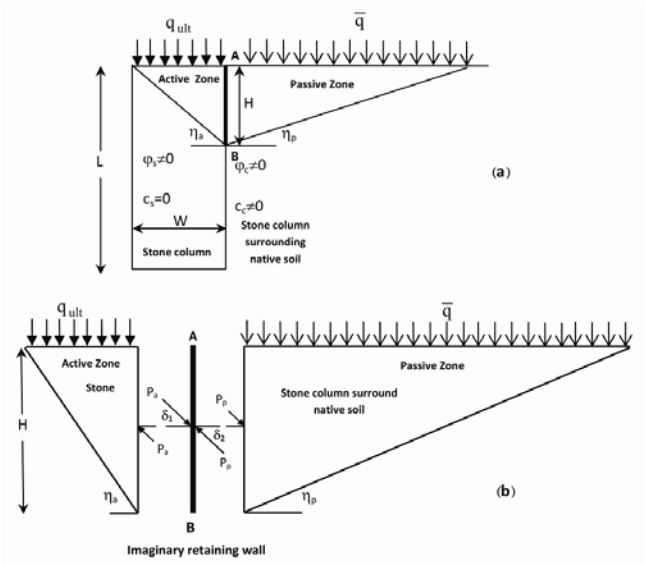

Fig. 3 Imaginary retaining wall conception

A vertical imaginary retaining wall is assumed to pass the foundation edge, as considered originally by Richard et al. [32] for determination of the seismic bearing capacity of strip foundation on homogeneous granular soil. The stone column is subjected to vertical pressure originated from the foundation. As a result, at the failure stage, the stone column material exerts an active thrust on imaginary wall AB (Fig.3). This force is determined using the Coulomb lateral earth pressure theory and properties of the

stone column material. The active wedge makes angle η_a with the horizontal direction. The imaginary wall is assumed to push the soil on the right had side. As a result, the native soil reacts by its passive pressure state. The passive wedge makes angle η_P with the horizontal direction.

For active wedge (Fig. 3), the value of η_a is computed using, [33]:

$$\eta_a = \varphi_s + \tan^{-1}\left(\frac{-\tan\varphi_s + C_1}{C_2}\right)$$
$$C_1 = \sqrt{\tan\varphi_s\left(\tan\varphi_s + \cot\varphi_s\right)\left(1+\tan\delta_1\cot\varphi_s\right)} \quad (6)$$
$$C_2 = 1 + \left(\tan\delta_1\left[\tan\varphi_s + \cot\varphi_s\right]\right)$$

Where, φ_s is the internal friction angle of the stone column granular material.

Similarly, the value of η_P for native soil with internal friction angle φ_c and cohesion c_c, can be calculated by,[33]:

$$\eta_P = -\varphi_c + \tan^{-1}\left(\frac{\tan\varphi_c + C_3}{C_4}\right)$$
$$C_3 = \sqrt{\tan\varphi_c\left(\tan\varphi_c + \cot\varphi_c\right)\left(1+\tan\delta_2\cot\varphi_c\right)} \quad (7)$$
$$\mathrm{C_4} = 1 + \left(\tan\delta_2\left[\tan\varphi_c + \cot\varphi_c\right]\right)$$

The active force, Pa, is computed using:

$$P_a = \frac{1}{2}K_{as}\gamma_s H^2 + q_{ult}K_{as}H \quad (8)$$

Where k_{as}=lateral active earth pressure coefficient, γ_s =unit weight of stone material, and H=the virtual wall height.

The passive force is determined from:

$$P_p = \frac{1}{2}K_{pc}\gamma_c H^2 + \bar{q}\ HK_{pc} + 2c_c\sqrt{K_{pc_c}}H \quad (9)$$

Wherek_{pc}=lateral passive earth pressure coefficient, γ_c =unit weight of stone material, and $\bar{q}$ =surcharge pressure on passive region surface.

The values of K_{as} and K_{p_c} are expressed, respectively, as,[34]:

$$K_{as} = \frac{\cos^2(\varphi_s)}{\cos(\delta_1)\left[1+\sqrt{\frac{\sin(\varphi_s+\delta_1)\sin(\varphi_s)}{\cos(\delta_1)}}\right]^2} \quad (10)$$

$$K_{pc} = \frac{\cos^2(\varphi_c)}{\cos(-\delta_2)\left[1-\sqrt{\frac{\sin(\varphi_c+\delta_2)\sin(\varphi_c)}{\cos(-\delta_2)}}\right]^2} \quad (11a)$$

$$K_{pc_c} = K_{pc}\left(1+\frac{c_w}{c_c}\right) \quad (11b)$$

Where c_w is the wall-soil interface cohesion and varies between $0.3c_c$ for stiff soilto c_c for soft soil. In the absence of experimental data, $c_w = 0.45c_c$ may be used [35]. CP2Code[36], limits a maximum value of 50 kPa for c_w. Table 1 shows c_w values for active and passive conditions.

Table 1 Value of c_w in terms of value of c_c based on CP2 code [36]

Value of c_c	Active state	Passive state
$c_c < 50\ \ kPa$	$c_w = c_c$	$c_w = 0.5c_c$
$c_c \geq 50\ \ kPa$	$c_w = 50\ \ kPa$	$c_w = 25\ \ kPa$

In eqs.(6), (7), (10), and (11), characters δ_1 and δ_2 represent the friction angle of stone-column material or native soil with imaginary rigid retaining wall, respectively. In this research, $\delta_1 = \varphi_s/2$ and $\delta_2 = \varphi_c/2$ are assumed as suggested by Richard et al. [32].

The height of the imaginary wall is given by:

$$H = W\tan\eta_a = \frac{A_s}{S}\tan\eta_a \quad (12)$$

The equilibrium euation for the forces in the horizontal direction on the face of the imaginary rigid retaining wall gives:

$$\mathrm{P_a}\cos\delta_1 = \mathrm{P_p}\cos\delta_2 \quad (13)$$

Substituting eqs. (8) and (9) into eq. (13) gives:

$$q_{ult} = \frac{\cos\delta_2}{\cos\delta_1}\frac{\left(\frac{1}{2}K_{pc}\gamma_c H + \bar{q}\ K_{p_c} + 2c_c\sqrt{K_{pc_c}}\right)}{K_{as}} - \frac{1}{2}\gamma_s H \quad (14)$$

Simplifying eq. (14) and substituting for $H = W\tan\eta_a$ leads to:

$$q_{ult} = c_c\left(2\frac{\cos\frac{\varphi_c}{2}}{\cos\frac{\varphi_s}{2}}\frac{\sqrt{K_{pc_c}}}{K_{as}}\right) + \bar{q}\left(\frac{K_{p_c}}{K_{as}}\frac{\cos\frac{\varphi_c}{2}}{\cos\frac{\varphi_s}{2}}\right) + \frac{1}{2}W\gamma_c\left(\frac{\cos\frac{\varphi_c}{2}}{\cos\frac{\varphi_s}{2}}\frac{K_{pc}}{K_{as}} - \frac{\gamma_s}{\gamma_c}\right)\tan\eta_a \quad (15)$$

Eq. (15) is similar to the conventional bearing capacity relationship for shallow foundations, given by:

$$\mathrm{q_{ult}} = \mathrm{c_c N_c} + \bar{\mathrm{q}}\ \mathrm{N_q} + \frac{1}{2}\mathrm{W}\gamma_{\mathrm{c}}\mathrm{N}_{\gamma} \quad (16)$$

Where $N_c = 2\frac{\cos\frac{\varphi_c}{2}}{\cos\frac{\varphi_s}{2}}\frac{\sqrt{K_{pc_c}}}{K_{as}}$ $N_q = \frac{K_{p_c}}{K_{as}}\frac{\cos\frac{\varphi_c}{2}}{\cos\frac{\varphi_s}{2}}$

$$N_\gamma = \tan\eta_a\left(\frac{\cos\frac{\varphi_c}{2}}{\cos\frac{\varphi_s}{2}}\frac{K_{pc}}{K_{as}} - \frac{\gamma_s}{\gamma_c}\right)$$

Fig. 4 presents variation of N_c coefficient versus ϕ_s for various friction angles of native soilhaving a cohesion of 50 kPa or less. Fornative soil having a cohesion greater than 50 kPa, N_c coefficient must be calculated using eq. (16). Figs. 5 and 6show the variation of N_q and N_γ coefficients versus the friction angleof the stone column material, respectively.

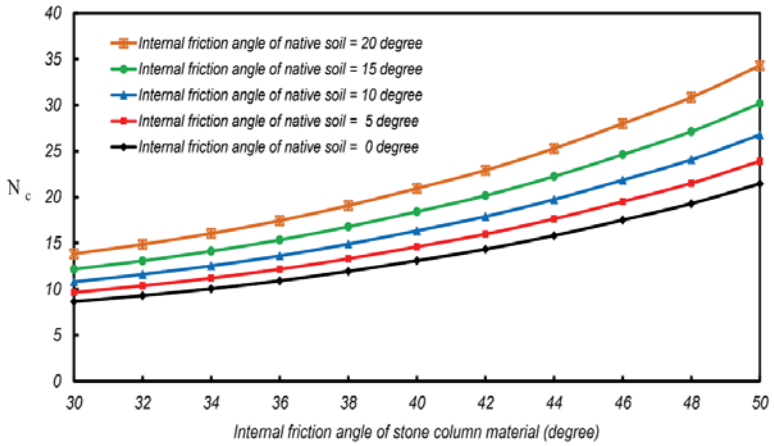

Fig. 4 Variation of N_c versus stone column material friction angle for various native soil friction angles (Cohesion of native is less than 50 kPa**)**

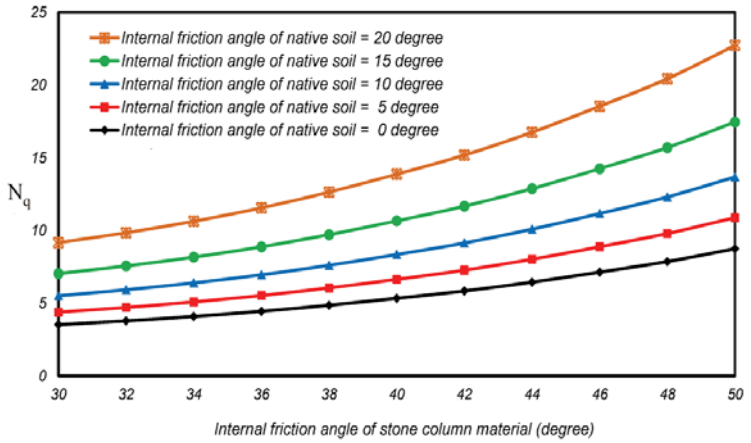

Fig. 5 Variation of N_q versus stone column material friction angle for various native soil friction angles

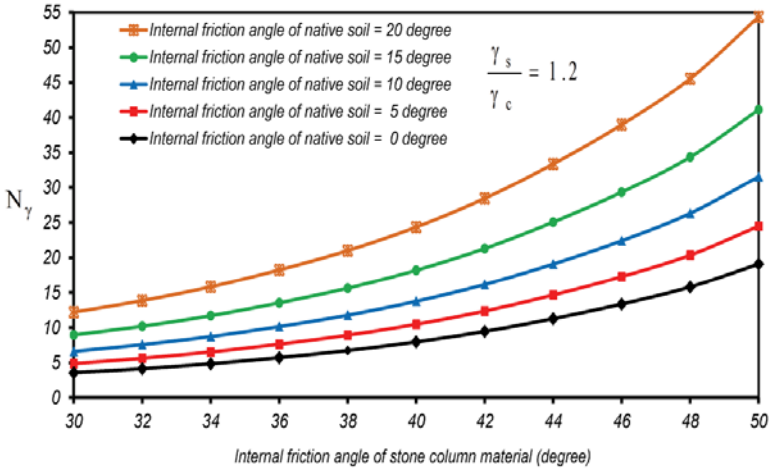

Fig. 6 Variation of N_γ versus stone column material friction angle for various native soil friction angle ($\frac{\gamma_s}{\gamma_c}=1.2$)

4. Evaluation of New Simple Method

4.1. Comparison with vesic analytical method

An example considered for comparisonthe results of new simple method with those of Vesic method. The unit weight of the native soil is $\gamma_c = 17KN/m^3$ and that of the stone-columnis $\gamma_s = 19KN/m^3$. The stonecolumn diameter is $D = 1$ m, and center-to-center distance for stonecolumns is $S = 3$ m. For analysis, six types for native soil are assumed. Soil 1 has $c_c = 30\ kPa$ and $\varphi_c = 0^\circ$. Soil 2 has $c_c = 40\ kPa$ and $\varphi_c = 0^\circ$, soil 3 has $c_c = 50\ kPa$ and $\varphi_c = 0^\circ$, soil 4 has $c_c = 70\ kPa$ and $\varphi_c = 0^\circ$, soil 5 has $c_c = 30\ kPa$ and $\varphi_c = 5^\circ$, and soil 6 has $c_c = 50\ kPa$ and $\varphi_c = 5^\circ$. In Vesic's method, for all six types of soils, the Poisson ratio and young modulus are assumed to be 0.35 and 11c, respectively. In Vesic method, for calculating the ultimate bearing capacity for stone-columns, $F_q^{'}$ and $F_c^{'}$ are assumed as below: For soil 1,2,3, and 4, $F_c^{'} = 2.4$ and $F_q^{'} = 1$. Soil 5 and 6 have $F_c^{'} = 2.4$ and $F_q^{'} = 1.25$ .Fig. 7 shows results of analysisfor six types of native soils and for different internal friction angles for stone-column materials. As seen, the new simple method gives relatively similar data to those of Vesic method, especially for cohesive soils. The minimum and maximum ratios of new method data to those of Vesic method varies 90% to 109%, as seen in Fig.7. In the developed method, for prediction of the stone column bearing capacity, only shear strength parameter of stone column and native soil materials are required, whereas in the Vesic's method, in addition to these, the Poisson ratio and young modulus of the soil are also required.This may be considered the superiority of the new method to that presented by Vesic.

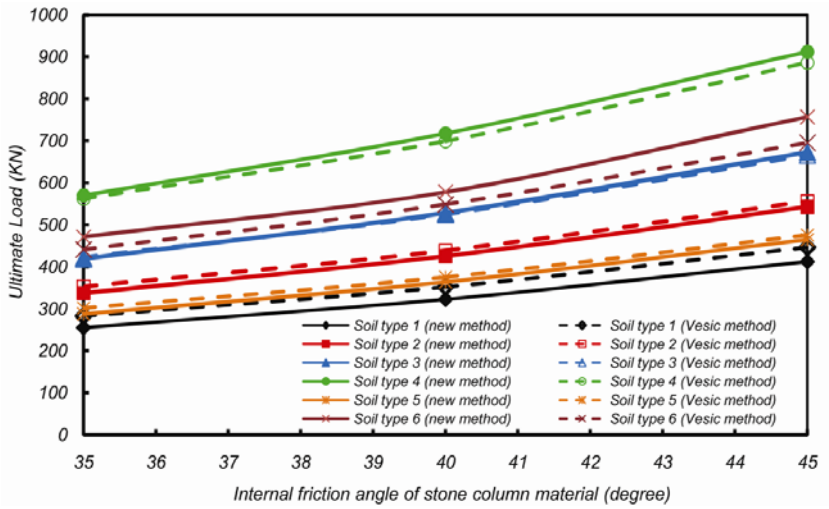

Fig. 7 Comparison between bearing capacity value determined from new method and Vesic method

4.2. Comparison with numerical results

Some analyses were carried out using finite element method based on PLAXISto compute the ultimate bearing capacity of stone columns. The constructed numerical soil-column system behavior was validated using experimental data on a real single stone column performed by Narasimha Rao et al. [37].They used a test tank with 650 mm diameter. The clay thickness was 350 mm. A stone column having a diameter of 25 mm and a length of 225 mm was constructed at the center of the clay bed. The column was loaded with a plate with diameter equal to twice the diameter of the stone column. Properties of clay and stone are shown in Table 2.

Table 2 Material properties used in Plaxis program for validation

Parameter	Clay	Stone column
Shear strength ,c_u (KPa)	20	0
Internal friction angle	0	38
Modulus of elasticity (KPa)	2000	40000
Poisson ratio	0.45	0.30

In the present paper, an axisymmetric finite element analysis was carried out using Mohr-Coulomb failure criterion for clay and stone materials. In the finite-element discretization, 15-noded triangular elements with boundary conditions as introduced in test were used.In all numerical analysis no interfaceelements were used Atthe interface

between the stone column and soft clay, because thedeformation ofthe stone columns is mainlyby radial bulging and no significant shear is possible.In addition,theinterface between a stone column and clay is a mixed zonewhere the shear strength properties can vary depending on themethod of installation.As this is not precisely known, an interfaceelement is not used[19].A similar finite-element analysis without an interface element carried out by Mitchell and Huber[38],Saha et al.[39], and Murugesan and Rajagopal[20].

Fig.8 compares the results obtained from the laboratory model test reported by Narasimha Rao et al.[37] and the finite element analysis carried out by the authors. As seen, the load-settlement variation obtained from the finite element analysis is in good agreement with those obtained from tests.

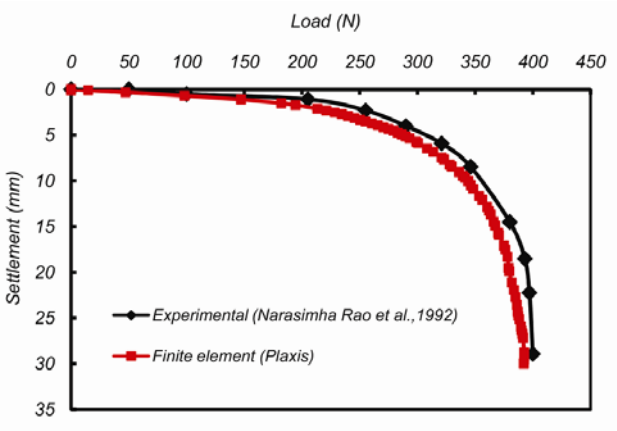

Fig. 8 Comparison between FE analysis and tests

An example is considered for comparison the results of the new simple method with those of finite element method. The unit weight of the native soil is $\gamma_c = 17\ kN/m^3$ and that of the stone-column is $\gamma_s = 19\ kN/m^3$. The stone column diameter is $D = 1$ m, and center-to-center distance for stone columns is $S = 3$ m. For analysis, three types for native soil are assumed. Soil 1 has $c_c = 30\ kPa$ and $\varphi_c = 0^\circ$. Soil 2 has $c_c = 40\ kPa$ and $\varphi_c = 0^\circ$, soil 3 has $c_c = 50\ kPa$ and $\varphi_c = 0^\circ$.Fig.9 compares the results obtained from analytical and numerical methods. As seen, the new simple method gives relatively similar data to those of the FE method.

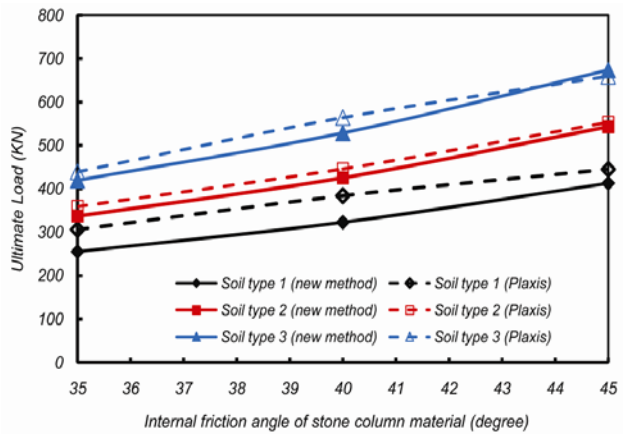

Fig. 9 Comparison between bearing capacity values determined from new analytical method and FE method

4.3. Comparison with experimental results

Cases 1-5

For cases 1-5, an investigation on the behavior of granular piles with different densities and properties of gravel and sand on soft Bangkok clay was carried out byBergado and Lam [40]. Table3 shows that for the same granular materials, the bearing capacity increases with increasing the number of blows per layer, resulting in an increase in densities and friction angles. The average deformed shaped of the granular piles is typically bulging type and all of granular piers have an initial pile diameter of 30 cm. Soft Bangkok clay had an undrained cohesion of $c_u = 15\ kPa$ and internal friction angle of $\varphi_c = 26^\circ$ ([41],[42]).

Table 3 Properties of granular piles

Test No:	Case 1	Case 2	Case 3	Case 4	Case 5
Proportion of sand in volume	1	1	1	0.3	0
Proportion of gravel in volume	0	0	0	1	1
SPT	20	15	10	15	15
In-situ average density (kN/m^3)	17	16.1	15	19.4	17.4
Friction angle (deg)	38.2	36.9	35.6	37.7	43.3
Measured ultimate load (kN)	33.3	30.8	21.7	31.3	36.3
Predicted ultimate load using new method (kN)	28.8	27.5	25.8	28.5	38.1
Deviation between predicted and measured load	-14%	-11%	19%	-9%	5%

Table 3 shows the results for the ultimate bearing capacity of granular piles, calculated by new simple method and reported from experimental load test. The deviations between the data are also shownwith respect to the measured data in Table 3 where the positive and negative sings represent over and under estimations, respectively, with respect to the measured data. As seen, there is a good agreement between predicted and measured data.

Case 6

A large-scale test was conducted by Maurya et al. [43] on a stone column in India.The stonecolumns were installed in a triangular pattern with $S = 4$ m, $D = 0.9$ m, and length of $L = 6.6$ m. For stone column material, the density was $\gamma_s = 22$ kN/m^3 and the friction angle was $\varphi_s = 46^\circ$ .Laboratory tests on soil samples collected from marine clay strata indicated that the cohesion valuesvaried 5 to 12 kPa, liquid limit ranged 69% to 84%, the plastic limit was 25% to 32%, and the in natural moisture contents varied 40% to 68%. The ultimate bearing capacity of native soil was 34 kPa. Field load tests were carried out on stone columns using real footings.The loaded area was larger than the cross-sectional area of the stone column. This is because applying the load over an area greater than the stone column increases the vertical and lateral stresses in the surrounding soft soil. As a result, it reflects the insitu condition under raft foundation or embankment.A reinforced concrete footing (RC) was constructed on the sand blanket.The diameter of the RC footing in case of single column wasequal to the spacing of stone columns, i.e. 4m, with center ofthe footing coinciding with the center of the column.The ultimate load was about 800kN for the single column test at a corresponding settlement of about 23mm.If the average cohesion of the soft soil is assumed8.5 kPa, the developed simple method givesthe stone column ultimate bearing capacity of $q_{ult} = 414$ kPa. If this value is multiplied by the cross sectional area of the stone column and added to the net area of the RC footingmultiplied by 34 kPa (the ultimate bearing capacity of native soil), the ultimate load becomes about 670kN. This differs only -16% from the measured capacity.

Case 7

Narasimha et al. [37] carried out a small-scale physical model test on a single stone column. The test tank used in their experiment had 650 mm diameter. The clay thickness was 350 mm. A stone column having a diameter of 25 mm and a length of 225 mm wasconstructed at the center of the clay bed. The column was loaded with a plate of diameter equal to twice the diameter of the stone column. The undrained shear strength ofthe clay was 20 kPa and the internal friction angle of the stone column material was 38°.

The experimental results showed that the ultimate bearing load carried out by the single stone column was 350 N. The bearing support offered by the clay soil in contact with the loading plate is obtained $q_{ult} = cN_c = 5.7 \times 20$ $kPa = 114$ kPa, using Terzaghi method. The developed simple method gives $q_{ult} = 241$ kPa. If this value is multiplied by the cross sectional area of the stone column and added to the net area of the loading plate multiplied by 114 kPa, the ultimate load becomes about 286 kN. This differs only -18% from the measured ultimate load.

CaseS 8 To 10

Murugesan and Rajagopal[44] carried out a large-scale physical model test on a single stone column.The test tank used in their experiment was cubic and dimensions of $1.2 \times 1.2 \times 0.8$ m. For stone column material, the density was $\gamma_s = 16$ kN/m^3 and the friction angle was $\varphi_s = 41.5^\circ$ .The undrained shear strength of clay was 2.5 kPa determined from in situ vane shear strength In the laboratory, the strength and the plasticity index of the clay were measured 2.22 kPa and 32, respectively. The clay saturated density was $\gamma_c = 16.88$ kN/m^3 .

Murugesan and Rajagopal [44] tested three single stone-columns having diameters of 5, 7.5, and 10 cm. The length of all three stone columns was 60 cm. The load was applied on a plate having a diameter equal to twice the column diameter. The experimental results show that the ultimate load tolerated by single stone columns and native soil are 110 N, 320 N, and 620 Nfor stone-columns having diameters 5, 7.5, and 10 cm, respectively. The bearing support offered by clay in contact with the loading plate was $q_{ult} = cN_c = 5.7 \times 2.22$ $kPa = 12.65$ kPa , using Terzaghi method. The developed simple method gives $q_{ult} = 31$ kPa for stone-columns with different diameter. If this value is multiplied by the cross sectional area of the stone column and added to the net area of the loaded plate multiplied by 12.65 kPa, the ultimate load becomes about 135 N, 304 N and 541 N for stone-columns with diameters of 5, 7.5, and 10 cm, respectively.These differs only 23%, -5% and-13% from the measured forces for 3 columns, respectively.As shown for above tencases, the new method over-estimates the ultimate load for four cases and under-estimates for six cases (Table 4). Therefore, obviously the developed simple method has capabilities to determine the ultimate load carried by a stone column and thus is used subsequently to perform further analyses on stone columns.

Table 4 Difference between measured and predicted values for ultimate loads carried by stone columns

CASE No:	Case 1	Case 2	Case 3	Case 4	Case 5	Case 6	Case 7	Case 8	Case 9	Case 10
Measured ultimate load (N)	33300	30800	21700	31300	36300	800	350	110	320	620
Predicted ultimate load using new method (N)	28800	27500	25800	28500	38100	670	286	135	304	541
Deviation between predicted and measured load	-14%	-11%	19%	-9%	5%	-16%	-18%	23%	-5%	-13%

5. Parametric Study

A series of parametric studies has been carried out using the developed method. The unit weight of the native soil and stone-column were taken $\gamma_c = 15\ kN/m^3$ and $\gamma_s = 18\ kN/m^3$, respectively. Also, diameter and center-to-center distance of stone column were taken , 0.8 m, and 2 m, respectively.Fig. 10 shows variation of stone column ultimate load versus native soil cohesion for various stone material friction angles. Results shows, the stone-column axial bearing capacity increaseswith increasing the internal friction angle of stone-column material.

Fig. 11 shows the variation of the ultimate axial load versus the stone column diameter for various stone material friction angles. In Fig.11, the native soil shear strength was assumed to be $c_c = 50\ kPa$. As observed, the internal friction angle of the stone material is more effective on the ultimate load for stone columns with greater diameter.There is also a negligible effect on the ultimate load for stone columns with diameters less than 0.6 m.

Fig. 12 shows the effect of space between stone columns with diameters of 0.6 m, 0.8 m, 1 m and 1.2 m for native soil with undrained shear strength of $c_c = 40\ kPa$ and stone column material with internal friction angle of $\varphi_s = 42^\circ$. As seen, the ultimate load carried by the stone column decreases by increasing stone column center-to-center distance of columns, especially by increasing the diameter of stone columns. In addition, the limiting ultimate load of the column decreases up to S/D=2-3.For S/D greater than 2-3, the reduction in the ultimate load is negligible. These findings are wellin accordance with experimental resultsreported by Ambily et al.[19]. They observed that as column spacing increases, the axial capacity of the column decreases and the settlement increases up to s /d=3. Beyond this, the change is negligible.

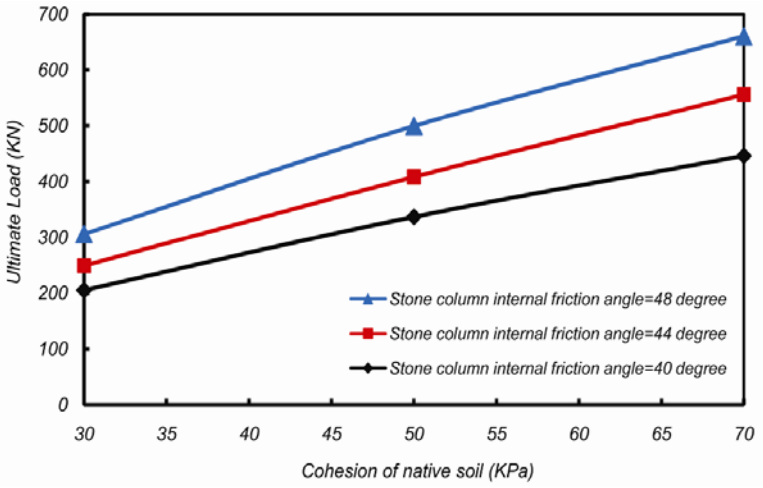

Fig. 10 Variation of stone column ultimate load versus native soil cohesion for various stone material friction angles

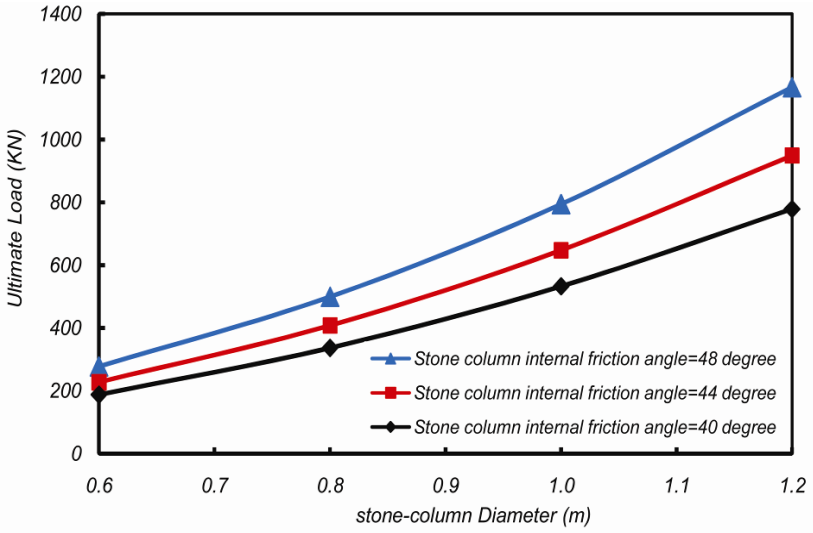

Fig. 11 Variation of stone column ultimate load versus stone column diameter for various stone material friction angles

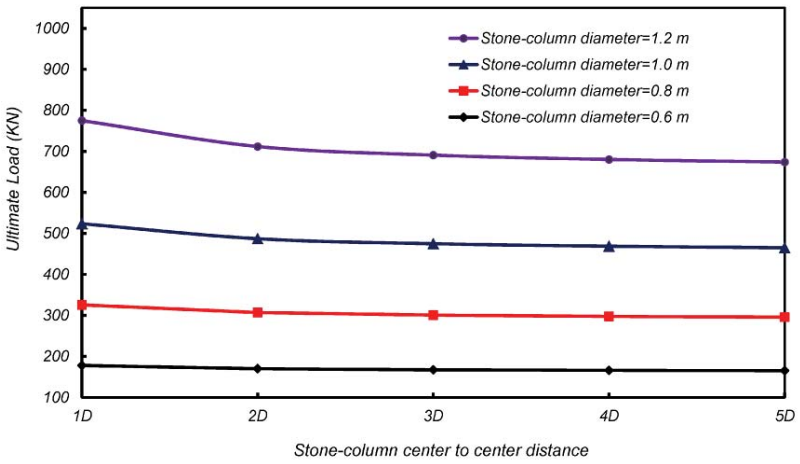

Fig. 12 Variation of stone column ultimate load versus stone column spacing for various stone material diameters

6. Conclusions

A simple method has been presented for determination of the ultimate load carried by stone columns. In the presented method three-dimentional problem of stone columns is converted in two-dimentional problem by using of traditional equivalent stone column strip .The method is based on the lateral earth pressure theorem and requires conventional Mohr-coloumb shear strength parameters of the stone column material and the native soil to be reinforced.The method also requires geometry parameters including diameter and spacing of the stone columns.The method predictions were verified using finite element numerical method and test data reported from available tests carried out by other researchers and showed reasonable agreement.

Parametric studies were carried out to determine the role of influencing parameters. The following concluding remarks may be extracted from the developed method:

1- The stone columnbearing capacity increases with increasing the friction angle of the stone material and the stone column diameter.

2- The stone column capacity decreases by increasing the stone columncenter to center distance to S/D=3 and beyond this value, the decrease of the stone capacity is negligible.

3- The use of stone columns is more efficient in softer cohesive soils.

The developed method is very simple, efficient and is very useful for estimation of the stone column ultimate bearing capacity. Although the predictions made by the developed simple solution are satisfactory, more laboratory and field tests and sophisticated numerical analyses are required to quantify the predictions of the developed solution.

References

[1] Kempfert H.G, Gebreselassie B. Excavations and Foundations in Soft Soils, Springer, 2006, pp. 461-521.

[2] Barksdale R.D, Bachus R.C. Design and construction of stone column, Report No. FHWA/RD-83/026, National Technical Information Service, Springfield, Virginia, 1983.

[3] Greenwood D.A. Mechanical improvement of soils below ground surface, Proceedings Of Ground Improvement Conference, Institute of Civil Engineering, 1970, pp. 9-29.

[4] Vesic A.S. Expansion of cavities in infinite soil mass, Journal of Soil Mechanics and Foundation Engineering Division, ASCE, 1972, No. SM3, Vol. 98, pp. 265-290.

[5] Huges J.M.O, Withers N.J. Reinforcing of soft cohesive soils with stone columns, Ground Engineering, 1974, No. 3, Vol. 7, pp. 42-49.

[6] Datye K.R, Nagaraju S.S. Installation and testing of rammed stone columns, Proceedings of IGS specialty session, 5th Asian Regional Conference on Soil Mechanic and Foundation Engineering, Bangalor, India, 1975, pp. 101-104.

[7] Madhav M.R, Iyengar N.G.R, Vitkar R.P, Nandia A. Increased bearing capacity and reduced settlements due to inclusions in soil, Proceedings of International Conference on Soil Reinforcement, Reinforced and other Techniques, 1979, pp. 239-333.

[8] Madhav M.R, Vitkar R.P. Strip footing on weak clay stabilized with a granular trench or pile, Canadian Geotechnical Journal, 1978, No.4, Vol. 15, pp. 605–609.

[9] Wong H.Y. Vibroflotation – Its effect on weak cohesive soils, Civil Engineering (London), 1975, Vol. 82, pp. 44-76.

[10] Aboshi H, Ichimoto E, Harada K, Emoki M. The composer- A method to improve the characteristics of soft clays by inclusion of large diameter sand columns, Proceedings of International Conference on Soil Reinforcement, E.N.P.C, 1, Paris, 1979, pp. 211–216.

[11] Pitt John M, White David J, et al. Highway application for rammed aggregate piles in Iowa soils, Iowa Department of Transportation, Final Repot, 2003.

[12] Shahu J.T, Madhav M.R, Hayashi S. Analysis of soft ground-granularpile-granular mat system, Computers and Geotechnics, 2000, Vol. 27, pp. 45-62.

[13] Shahu J.T. Non-uniform granular pile–mat foundations: analysis and model tests, Geotechnical and Geological Engineering, 2006, Vol. 24, pp. 1065–1087.
[14] Bouassida M, De Buhan P, Dormieux L. Bearing capacity of a foundation resting on a soil reinforced by a group of columns, Ghotechnique, 1995, No. 1, Vol. 45, pp. 25-34.
[15] Bouassida M, Jellali B, Porbaha A. Limit analysis of rigid foundations on floating columns, International Journal of Geomechanics, ASCE, 2009, No. 3, Vol. 9, pp. 89-101.
[16] Lee S, Pande G.N. Analysis of stone column reinforced foundations, International Journal for Numerical and Analytical Methods in Geomechanics, 1998, Vol. 22, pp. 1001-1020.
[17] Abdelkrim M, De Buhan P. An elastoplastic homogenization procedure for predicting the settlement of a foundation on a soil reinforced by columns, European Journal of Mechanics A/Solids, 2007, Vol. 26, pp. 736–757.
[18] Wood D.M, Hu W, Nash D.F.T. Group effect in stone column foundations: model test, Ghotechnique, 2000, No. 6, Vol. 50, pp. 689-698.
[19] Ambily A.P, Gandhi Shailesh R. Behavior of stone columns based on experimental and FEM analysis, Journal of Geotechnical and Geoenvironmental Engineering, ASCE, 2007, No. 4, Vol. 133, pp. 405-415.
[20] Murugesan S, Rajagopal K. Geosynthetic-encased stone columns: Numerical evaluation, Geotextiles and Geomembranes, 2006, Vol. 24, pp. 349–358.
[21] Hughes J.M.O, Withers N.J, Greenwood D.A. A field trial of reinforcing effect of stone column in soil, Proceedings of Ground Treatment by Deep Compaction, Institution of Civil Engineers, London, 1976, pp. 32-44.
[22] Elshazly H, Elkasabgy M, Elleboudy A. Effect of inter-column spacing on soil stresses due to vibro-installed stone columns: interesting findings, Geotechnical and Geological Engineering, 2008, Vol. 26, pp. 225–236.
[23] Elshazly H, Hafez D.H, Mossaad M.E. Reliability of conventional settlement evaluation for circular foundations on stone columns, Geotechnical and Geological Engineering, 2008, Vol. 26, pp. 323–334.
[24] Christoulas ST, Giannaros CH, Tsiambaos G. Stabilization of embankment foundations by using stone columns, Geotechnical and Geological Engineering, 1997, Vol. 15, pp. 247–258.
[25] Han J, Oztoprak S, Parsons R.L, Huang J. Numerical analysis of foundation columns to support widening of embankments, Computers and Geotechnics, 2007, Vol. 34, pp. 435–448.
[26] Abusharar S.W, Han J. Two-dimensional deep-seated slope stability analysis of embankments over stone column-improved soft clay, Engineering Geology, 2011, Nos. 1-4, Vol. 120, pp. 103-110.
[27] Zahmatkesh A, Choobbasti A. Settlement evaluation of soft clay reinforced with stone columns using the equivalent secant modulus, Arab Journal of Geoscience, 2010, No. 1, Vol. 5, pp. 103-109.
[28] Deb K. Modeling of granular bed-stone column-improved soft soil, International Journal for Numerical and Analytical Methods in Geomechanics, 2008, Vol. 32, pp. 1267–1288.
[29] Deb K, Chandra S, Basudhar P.K. Response of multilayer geosynthetic-reinforced bed resting on soft soil with stone columns, Computers and Geotechnics, 2008, Vol. 35, pp. 323–330.
[30] Fan Chia.Ch, Luo J.H. Numerical study on the optimum layout of soil–nailed slopes, Computers and Geotechnics, 2008, Vol. 35, pp. 585–599.
[31] Kvasnicka P, Matesic L. Analysis of an example a nailed wall in soft clayey soil, Geotechnical Hazards, Balkema, Rotterdam, 1998, pp. 547-552.
[32] Richards Jr.R, Elms D.G, Budhu M. Seismic bearing capacity and settlements of foundations, Journal of Geotechnical Engineering, ASCE, 1993, No. 4, Vol. 119, pp. 662-674.
[33] Richards R, Elms D.G. Seismic behavior of tied back walls, Report 87-8, Department of Civil Engineering, University of Canterbury, Christchurch, N.Z, 1987.
[34] Tschebotarioff G.P. Soil mechanics, foundations and earth structures, McGraw-Hill, NewYork, N.Y, 1951.
[35] Whitlow R. Basic soil mechanics, 2nd Edition, Longman Scientific &Technical, UK, 1990.
[36] Civil engineering code of practice, Earth retaining structures, The Institution of Structural Engineers, London, 1951, No. 2 (CP2).
[37] Narasimha R.S, Madhiyan M, Prasad Y.V.S.N. Influence of bearing area on the behavior of stone columns, Proceedings of Indian Geotechnic Conference, Calcutta, India, 1992, pp. 235-237.
[38] Mitchell J.K, Huber T.R. Performance of a stone column foundation, Journal of Geotechnical Engineering, 1985, No. 2, Vol. 111, pp. 205–223.
[39] Saha S, Santhanu S, Roy A. Analysis of stone column in soft ground, Proceeding, Indian Geotechnical Conference, Bombay, India, 2000, pp. 297–300.
[40] Bergrado D.T, Lam F.L. Full scale load test of granular piles with different densities and different proportions of gravel and sand in the soft bangkok clay, Journal of Soils and Foundations, 1987, No. 1, Vol. 27, pp. 86-93.
[41] Budhu M. Soil mechanic and foundation, John wiley & sons, INC, 2000.
[42] Balasubramian A.S, Chaudhry A.R. Deformation and strength characteristics of soft bangkokclay, Journal of Geotechnical Engineering Division, ASCE, 1978, No. 9, Vol. 104 , pp. 1153-1167.
[43] Maurya R.R, Sharma B.V.R, Naresh D.N. Footing load tests on single and group of stone columns, 16th International Conference on Soil Mechanics and Geotechnical Engineering, Osaka, Japan, 2005, pp. 1385-1388.
[44] Murugesan S, Rajagopal K. Studies on the behavior of single and group of geosynthetic encased stone columns, Journal of Geotechnical and Geoenvironmental Engineering, ASCE, 2010, No. 1, Vol. 136, pp. 129-139.