Evaluation the Efficiency of Installing Stone Columns in Ground Improvement of Al-Nassiriya City Soil

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Abstract

This paper investigates the performance of stone columns in soft soil of Al-Nassiriya city (one of Iraqi cities in the south). A geotechnical study of physical and mechanical properties of the soil has been carried out for a selected areas from Al-Nassiriya city. A finite difference Matlab program has been employed in simulation of installing stone columns in the aforementioned soil. Results of the study illustrate using stone columns reduces the settlement of the soil for different values of the modular ratio and replacement ratio, thus the Wmax maximum settlement ratio changes from 30% to 16% of the modular ratios (E_{c}/E_{s})=10 and 50 respectively. The effect of spacing and column width is taken into consideration.

All results are reasonable and harmonic with most last studies.

Keywords : Geotechnical ,stone columns , finite difference .

تقييم كفاءة استخدام الاعمدة الحجرية في تحسين تربة مدينة الناصرية

المستخلص

ان هذا البحث يدرس كفاءة استخدام طريقة تسليح تربة مدينة الناصرية الضعيفة بتقنية الركائز الحجرية. وقد تم عمل جميع فحوصات التربة المختبرية لنماذج منتقاة من احد المواقع في مدينة الناصرية وذلك لمعرفة الخواص الفيزيائية والميكانيكية للتربة .

و قدمت الباحثتان في هذا البحث محاكاة نموذج ميكانيكي لتربة مدينة الناصرية الضعيفة مسلحة بعمود حجري موضوع فوقها طبقة حبيبية باستخدام برنامج الماتلاب وحسب طريقة الفروق المحددة وبالاستفادة من نتائج الفحوصات المختبرية لتمثل بيانات ادخال المسألة.

لقد اثبتت النتائج فعالية تسليح تربة مدينة الناصرية بأعمدة حجرية ، اذ أعطى هذا النوع من التسليح تقليل من مقدار الهبوط الكلي وبالتالي زيادة في قيمة قابلية التحمل لتربة مدينة الناصرية .ولقد اعطت النتائج في هذه الدراسة توافقا مع دراسات سابقة كثيرة.

1. Introduction

Al-Nassiriyah city, the centre (capital) of Thi-Qar governorate is situated in the south of Iraq at about 225 miles (370 km) southeast of Baghdad. Recently many engineering projects have been performed as a result to rebuilding movement in Thi-Qar province. The basic construction problem is related to the foundation because the soil is soft and the underground water exists at shallow depth of the soil.

Based on our site observations, stone columns are adopted to increase load carrying capacity and reduce the total settlement In addition to the advantage of getting a drainage path for solving a problem of high underground water.

The main objectives for using stone column in soil reinforcement are, to increase the bearing capacity, reduce settlement, improve slope stability and the resistance to liquefaction (Greenwood ,1970; Baumann & Bauer,1974; Hughes, J. M. 0. & Withers ,1974; Priebe, 1976; Madhav & Vitkar,1978; Goughnour & Bayuk ,1979; Balaam & Booker ,1981; Schweiger & Pande ,1986; and Canetta, & Nova,1989). In 2010 Deb studied the advantage of providing a drainage path (as stone column behaves similar to sand column). Moreover soil reinforcement by stone column can be considered as a very economic solution for soil improvement (Al-Obaidy 2000).

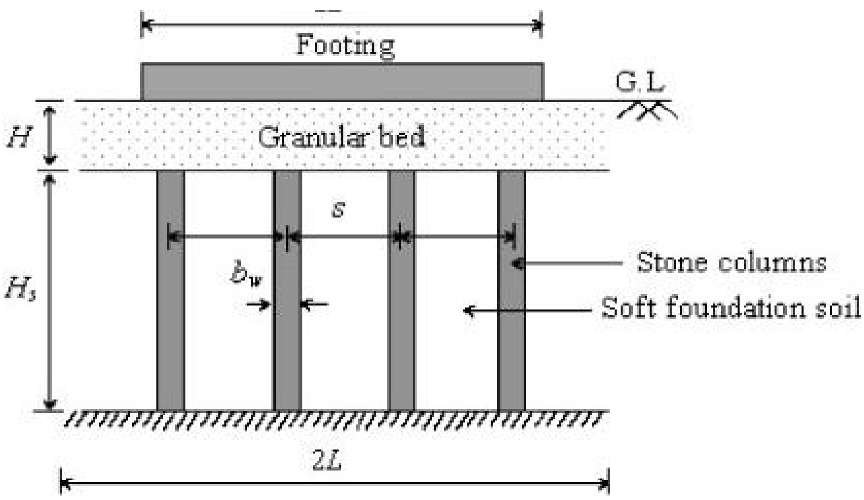
A number of publications have been written on the development of theoretical solutions for estimating settlement and bearing capacity of soft soils reinforced by stone columns. Many researchers simulated their studies of stone columns by finite element method, see : Acharya, et al, 2005; Al-Obaidy, 2005; Elshazly et al 2007& 2008; Frikha et al 2008; Sadek and Shahrour 2008; and Hassen et al, 2010. Whereas others introduced their numerical model using finite difference method such as: Buggy et al,1994; Taberlet et al, 2006; Deb et al, 2007; El Shamy, 2007; Han et al, 2007; Murali, et al 2007; and Deb, 2010.

A granular layer of sand or gravel is usually placed over top of the stone columns (Mitchell, 1981) as shown in Figure (1). This granular layer or sand bed acts as drainage layer and also distributes the stresses coming from the embankments.. The embankment has been modeled by Pasternak shear layer with variable thickness as proposed by Sharma in 1989. The granular layer and the soft soil have been idealized by the Pasternak shear layer and spring-dashpot system, respectively as described by Shukla and Chandra , 1994 :[see Figure (2)].

In the present study, the mechanical model shown in Figure (2) have been adopted and a numerical study is conducted to investigate the efficiency of installing stone columns in Al-

Nasiriyah city soil in increasing load carrying capacity and reducing the amount of settlement of the soil.

In the present paper, there are two steps, The first one is performing laboratory tests to get the input data which represent the case of Al-Nasiriyah city soil and then using these data in the second step. The second one is programming the case study using Matlab program to simulate the case of Al-Nasiriyah soil improvement by reinforcing it by stone columns have different modular ratio and replacement area.



Figure(1). A granular fill-stone column-reinforced soft soil system.

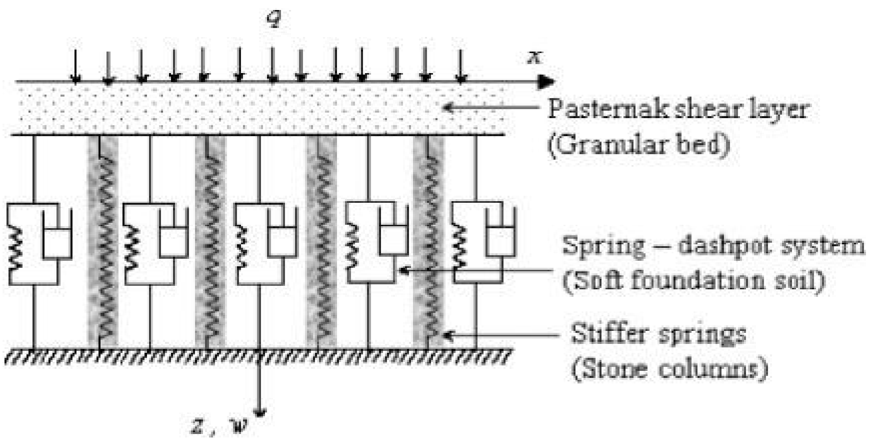


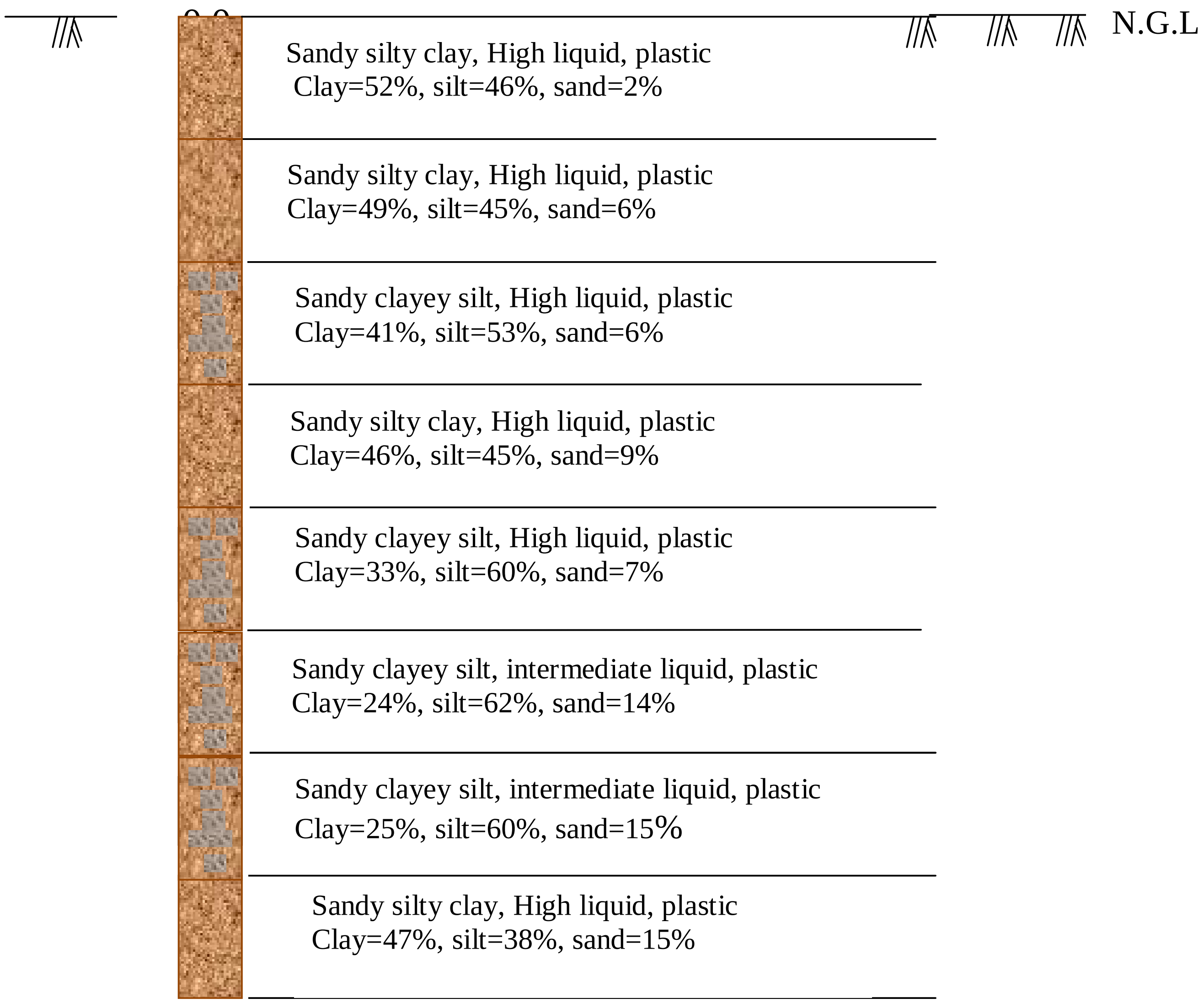
Figure (2). The mechanical Model (Pasternak shear layer-stiffer spring-spring & dashpot).

2. Soil properties from experiments

Soil specimen has been taken from bore holes in Al -Shumukh area in Al-Nasiriyah city and number of soil samples have been tested. The laboratory tests are carried out in the National Centre For Construction Laboratories (NCCL) at Thi-Qar governorate & the soil mechanics laboratory at Civil Engineering Department, College of Engineering, University of

Baghdad. The geotechnical properties for soil are determined according to the American society for testing and material (1984) and standard procedures adopted by the British standard (BS1377; 1975).

Figure (3) shows the soil profile for site study, while Table(1) represents a complete summary of all laboratory test results which belong to Al- Shumukh area soil which has been simulated to be reinforced with stone column.



Figure(3). Al -Shumukh area soil profile.

Table (1). The physical and mechanical properties of the soil of Al -Shumukh site in Al-Nasiriyah city.

Depth of sample (m)	Index tests			Mc	Ic	Specific gravity	Total density g/cm^3	Swelling potential	Mechanical Properties	
	LL	**PL**	**PI**						**C** kg/cm^2	ϕ^0
0.0 - 0.5	62	32	30	36	0.866	2.7		9.618		
0.5 - 1.0	57	28	29	32	0.862	2.7	1.82	8.485	0.5	11
1.0 - 1.5	55	25	30	37	0.6	2.7		8.14		
1.5 - 2.0	57	29	28	28	1.035	2.68		7.423		
2.0 - 2.5	58	30	28	30	1	2.7	1.79	6.025	0.3	10
2.5 - 3.0	43	26	17	27	0.941	2.68	1.9	1.532		
3.0 - 3.5	42	25	17	30	0.705	2.7	1.85	1.559		
3.5 – 4.0	50	31	19	24	1.368	2.7	1.9	2.935		

3.Theoretical analysis

The uniform load of intensity (q) which is applied on the foundation of 2B width at time (t>0)is expressed by Deb et al in 2007 for the same mechanical visco - elastic model shown in Figure (2) as follow:

$$q = q_s - GH\frac{\partial^2 w}{\partial x^2} \qquad (1)$$

Where:

q_s: vertical stress acting on the saturated soft foundation soil

H: thickness of the granular layer placed over the soft soil

$w(x,t)$: vertical displacement

G: shear modulus of the granular layer which can be expressed based on hyperbolic shear stress- shear strain response proposed by Gosh and Madhav (1994) as:

$$G = \frac{G_0}{\left[1 + \frac{G_0 \left/ \partial w \middle/ \partial x\right.}{\tau_u}\right]^2}$$

Where G_0 and τ_u : initial modulus of the shear layer and ultimate shear resistance of the granular layer respectively.

The vertical stress q_s at time (t>0) considering the hyperboilic non-linear stress-displacement relationship and the consolidation effect of the soft soil can be expressed as Deb et al in 2007 as the following equation:

$$q_s = \frac{k_{s0} w}{U\left[1 + k_{s0}\left(w/q_u\right)\right]} \qquad (2)$$

Where:

k_{s0}: initial modulus of a subgrade reaction of the saturated soft soil

q_u: ultimate bearing capacity of the saturated soft soil

U: average degree of consolidation at any time (t)

By substitution eq. 2 in eq.1 that yields:

$$q = \frac{k_{s0} w}{U\left[1 + k_{s0}\left(w/q_u\right)\right]} - GH\frac{\partial^2 w}{\partial x^2} \qquad (3)$$

Since the vertical stress acting on the stone column is $q_c = k_{c0} w$

(where: k_{c0}: modulus of the subgrade reaction of the stone column)

Eq.3 can be written in the following form to govern the differential equation within the stone column region:

$$q = k_{c0} w - GH\frac{\partial^2 w}{\partial x^2} \qquad (4)$$

Using the non-dimensional parameters as:

$X = x/B$, $W = w/B$, $G^* = GH/k_{s0}B^2$, $G_0^* = G_0H/k_{s0}B^2$, $q^* = q/k_{s0}B$, $q_u^* = q_u/k_{s0}B$, $\tau_u^* = \tau_u H/k_{s0}B^2$, $\alpha = k_{c0}/k_{s0}$, $q_s^* = q/k_{s0}B$, $q_c^* = q_s/k_{s0}B$, $q_c^* = \alpha w$

The governing differential equation at time t>0 can be expressed in a non-dimensional form as:

$$q^* = CW - G^*\frac{\partial^2 w}{\partial x^2} \qquad (5)$$

Where:

$$C = \frac{1}{U\left[1 + \left(W/q_u^*\right)\right]} \quad \text{within the soft foundation}$$

$$= \alpha \quad \text{within the stone column region}$$

The modulus of the subgrade reaction of the soft soil (spring stiffness) can be expressed in terms of the modulus of elasticity and Poisson ratio ,see Bowels in 1988:

$$k_{s0} = \frac{E_s}{H_s\left(1 + \mu_s\right)\left(1 - 2\mu_s\right)} \qquad (6)$$

Where :

H_s: thickness of the soft soil

E_s: modulus of elasticity

μ_s: Poisson's ratio

The modulus of the subgrade reaction of the stone column using stone column length equal to the length of soft soil according to the above equation will be:

$$k_{c0} = \frac{E_c}{H_c(1+\mu_c)(1-2\mu_c)} \tag{7}$$

Where :

H_c: length of stone pile

E_c: modulus of elasticity of the stone column

μ_c: Poisson's ratio of the stone pile

Thus the subgrade modulus or spring constant ratio (α) can be calculated as:

$$\alpha = \frac{(1+\mu_s)(1-2\mu_s)}{(1+\mu_c)(1-2\mu_c)}\frac{E_c}{E_s} \tag{8}$$

The degree of consolidation of the stone column-reinforced soft soil at any time has been calculated as mentioned by Deb et al in 2007 , in which the width of the plain strain unit cell is taken to be equal to the diameter of the unit cell in the axi-symmetric condition , they introduce An approximate solution if U is greater than 30% to be expressed as:.

$$U = 1 - \frac{8}{\pi^2}\exp^{-[8/F(N_{PL})]T'_r \; -[\pi^2/4]T'_v} \tag{9}$$

Where

$T'_r = c'_r\, t/4B_e^2$: a modified time factor in the radial flow, B_e is half plain-strain unit cell width

$T'_v = c'_v\, t/H_s^2$: a modified time factor in the vertical flow

$F(N_{PL}) = 2/3$, Npl is width ratio (=2Be/b_w), b_w is width of the stone and Cv are modified coefficient of consolidation in radial and vertical directions respectively

$$c_r = c_r\left(1+\beta\frac{1}{N_{pl}^2-1}\right) and \quad c_v = c_v\left(1+\beta\frac{1}{N_{pl}^2-1}\right)$$

c'_r and c'_v are modified coefficient of consolidation in radial and vertical directions

β: is the stress concentration ratio as the consolidation complete

$= \xi\frac{E_c}{E_s}$ where ξ is the Poisson ratio factor which equal to: $\frac{(1+\mu_s)(1-2\mu_s)(1-\mu_c)}{(1+\mu_c)(1-2\mu_c)(1-\mu_s)}$

4. Presentation of the implemented program

The application of the theory developed above is obtained with the help of the MATLAB Virgin R2008b , its software offering various functions and facilities to compile and to visualize complex mathematical operations. The implemented program is subdivided into different logic subprograms accomplishing specific tasks from data input to output visualisation .

The governing differential equation (eq.5 in the previous section)has been programmed using finite difference method by dividing the length L/B into n number of elements with (n+1) number of node points (i =1,2,3,4 to n);thus, the mesh size (ΔX) can be expressed as, ΔX . the governing differential equation in a finite difference form, for an interior node (i, j) (where i and j are the indices for space and time, respectively) as expressed below:

$$q_i^* = C_{i,j}W_{i,j} - G_t\left[\frac{W_{i-1,j} - 2W_{i,j} + W_{i+1,j}}{(\Delta X)^2}\right] \tag{10}$$

where

$$C_{i,j} = \frac{1}{U_j\left[1+\left(W_{i,j}/q_u^*\right)\right]} \quad \textit{within the soft foundation soil}$$

$$= \alpha \quad \textit{within the stone column region}$$

One half of the system is depended as the problem is symmetric,. Thus, at the centre of the loaded region X =0 or x=0, due to symmetry, the slope ∂W/∂X will be zero. The width of the granular fill considered in the analysis is sufficient enough so that at the edge [at X =L/B (or x =L)] the slope ∂W/∂X of the settlement-distance profile will also be zero.

The continuity at the edge of the stone columns is automatically satisfied. The loading conditions considered are given as:

$$q_i^*(X) = q* \qquad for|X| \leq 1.0$$

$$= 0 \qquad for \; |X| \rangle \; 1.0 \tag{11}$$

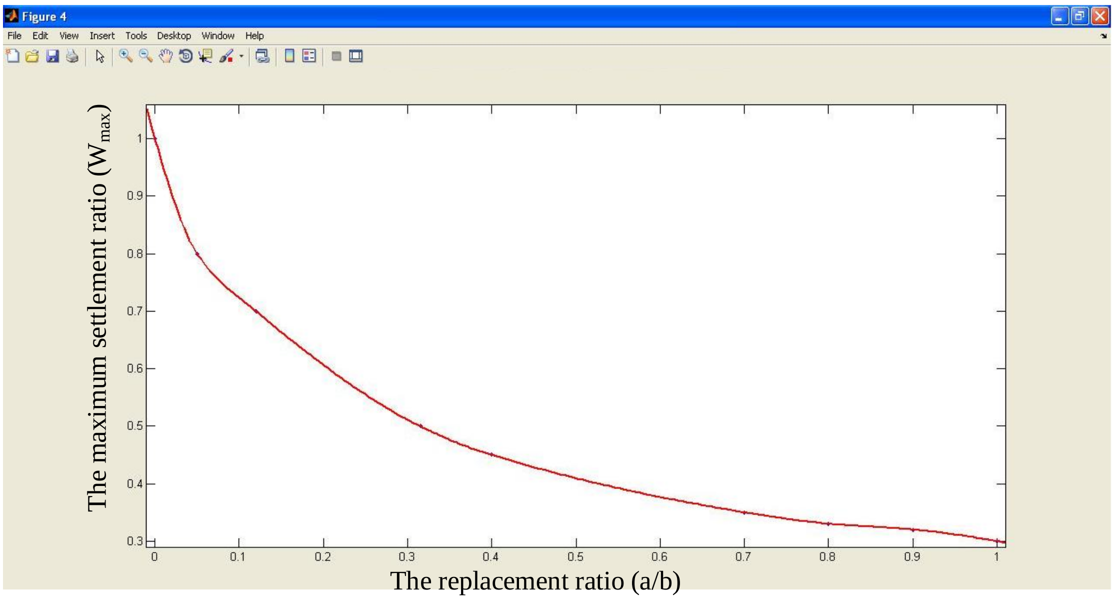
5. Results and discussion

Figure (4) illustrates the variations of maximum settlement ratio of the improved ground to that of an unimproved ground with replacement ratio a/b (where a=bw/2 and b=s/2, where s is the center to center distance between the stone columns) .It can be observed from Figure (4) that, when a/b is zero (no stone column) no reduction in the maximum settlement has been observed, but as the value of a/b increases settlement decreases and when a/b is 1, in that case, the soft foundation soil is completely replaced by stone columns, and reduction of the settlement is maximum at that point. As the stiffness of the stone columns is increased compared with the soft foundation soil, settlement is reduced further. This is because of the fact that as stiffness of the stone columns is increased more stress is transferred onto the stone column from the soft soil due to their stiffness difference which causes more settlement reduction. However, after a/b=0.5 , no significant change in the amount of the maximum settlement ratio.

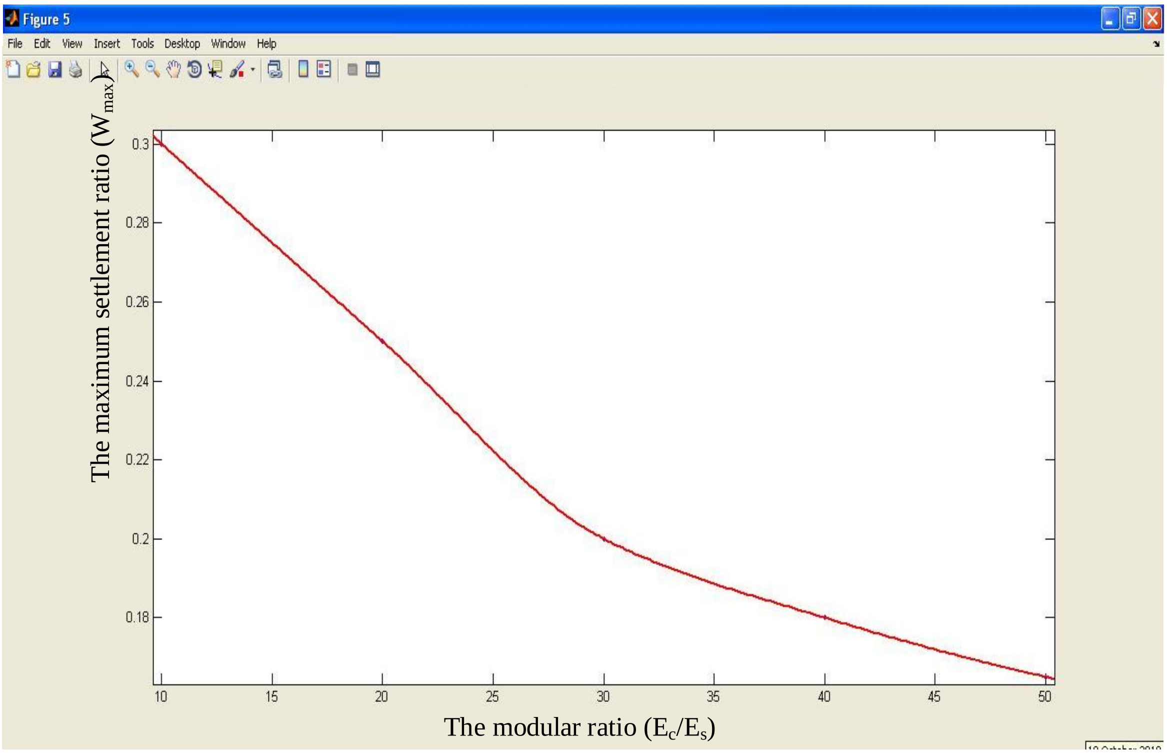
Figure (5) shows the effect of the modular ratio Ec/E_s on the amount of the maximum settlement. Obviously, it can be seen that as the modular ratio Ec/E_s increases as the maximum settlement decreases, thus the value of the maximum settlement ratio ranges between 0.3 and 0.15 for modular ratios Ec/E_s from 10 to 50. That may be explaining how the stiffer stone column carries more load which subjected on the composed cell of stone column and its surrounding area.

The relationship between maximum settlement and the ratio of spacing and the width of the stone column s/b_w for different values of the modular ratio is shown in Figure (6). There is a remarkable change in the value of the maximum settlement with respect to the ratio of s/b_w.

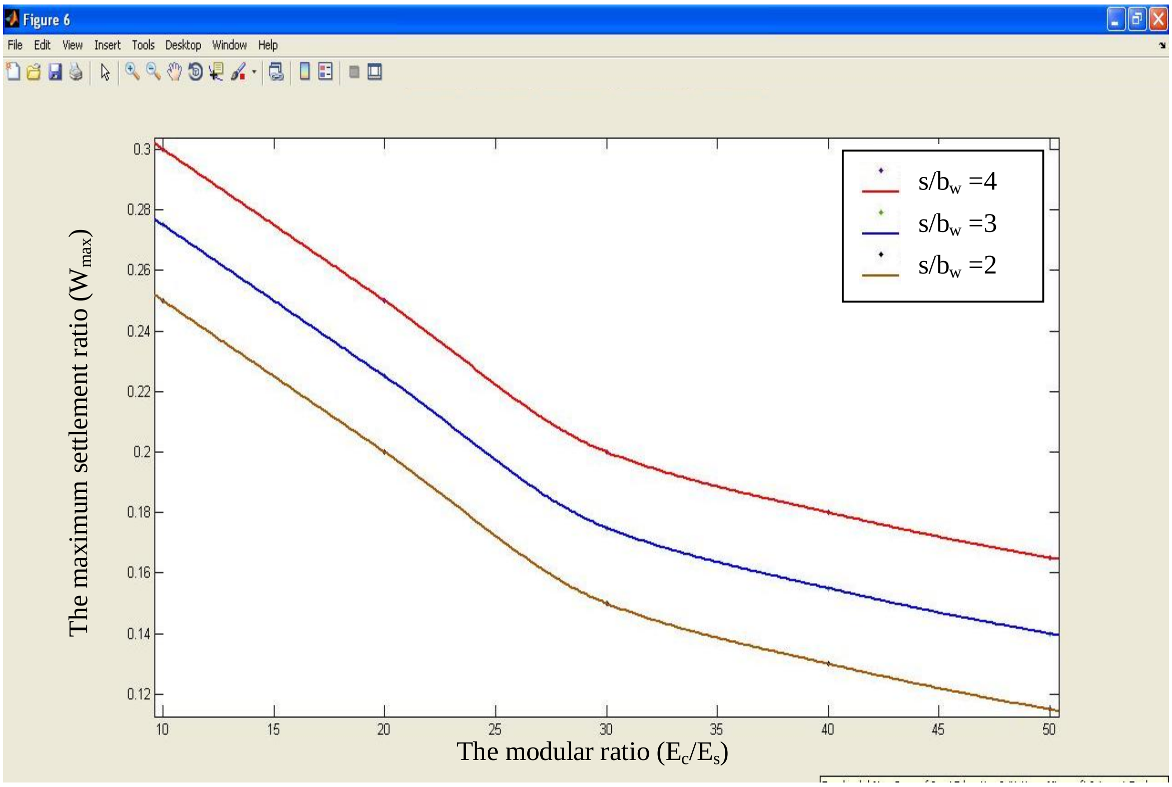
The results of this study are harmonic with last the studies results such as Balaam and booker 1981; Alamgir, et al 1996 and Al-Obaidy, 2005.



Figure(4). The maximum settlement ratio versus the replacement ratio.



Figure(5). The maximum settlement ratio versus the modular ratio (E_c/E_s).



Figure(6). The maximum settlement ratio versus the modular ratio for different values of s/b_w .

6. Conclusion

From the above discussions it can be said that the installation of stone column in Al-Nasiriyah soil is quite useful to reduce the amount of settlement and subsequently increasing the amount of the bearing capacity , thus the maximum settlement ratio changes from 30% to 16% of the modular ratios (E_c/E_s) =10 and 50 respectively. In addition there is an advantage of getting a drainage path for the stone column acts as sand drain for solving a problem of high underground water in Al-Nasiriyah soil. Moreover the material of stone column is very cheap if compare with others material ,so it can be considered an economic solution for ground improvement.

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