ANALYSIS OF STONE COLUMN IN SOFT SOIL BY FINITE ELEMENTS METHODS

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الخلاصة:

تتضمن هذه الدراسة إجراء تحليل لتصرف الأعمدة الركامية (Stone Column) باستخدام طريقة العناصر المحددة (Finite Element Method) في محاولة لتوفير بعض القواعد والمعلومات والمعادلات التي تساعد مهندس الجيوتكنيك في تصميم أسس تستند إلى طبقات طينية ضعيفة سلحت بالأعمدة الركامية. تم اعتماد العنصر الرباعي متماثل المحور(Axisymmetric quadrilateral element) لتمثيل التربة والأعمدة الركامية بينما استعمل عنصر التداخل الأحادي البعد (One-dimensional interface element) لتمثيل الاحتكاك بين الأعمدة الركامية والتربة.استعمل الموديل ا للاخطي (nonlinear inelastic stress-dependent) لتمثيل التربة والاحتكاك في منطقة التداخل خلال مراحل التحميل المختلفة (incremental solution) من خلال معلومات تم الحصول عليها من فحص الضغط الثلاثي المحاور.أجريت التحليلات في البداية على مسالة نموذجية(Basic problem) لتوضيح تصرف العمود الركامي حيث تم اختيار أبعاد المسالة المتاخمة(Boundary condition) وتم اختيار متغيرات خواص التربة ومنطقة التداخل. بعد ذلك تم العمل باتجاه دراسة تأثير بعض المتغيرات الخاصة بأبعاد العمود الركامي وخواص مادته وعلاقة ذلك بخواص التربة الطينية المحيطة به.

لوحظ من خلال النتائج أن زيادة طول العمود الركامي وزيادة نسبة جساءة مادته إلى جساءة التربة المحيطة تلعب دورا" مهما"في تقليل الهبوط وفي زيادة التحمل الأقصى للعمود الركامي.

Abstract

This paper includes an analysis to asses the behavior of stone columns using the finite element method and to provide bases and information helping geotechnical engineers to design foundations resting on weak soils reinforced with stone column.

The axisymmetric quadrilateral element is adopted in the finite element program to simulate the soft soil and the stone column while the one-dimensional element is used to simulate the soft soil and the stone column-soil interface. The nonlinear inelastic stress-dependent model is used to simulate the behavior of the soil and the interface throughout the incremental loading stages adopting nonlinear parameters obtained from triaxial and direct shear stress.

The analysis is carried first on a selected basic problem, to clarify the nonlinear of the column, in which a selected geometry, boundary condition, and material properties for both soil and interface as chosen.

The rest of the analysis is grouped into the effect of some of the parameters concerning the geometry of the stone column and the material of column and adjacent soil are investigated.

It was found that the increase in stone column length and in relative stiffness of stone column material to soil play an important role in increasing ultimate capacity of the stone column and in reducing settlements.

Introduction

There are several methods to improve the geotechnical characteristics of soft soils like, sand drains, dewatering, sand pile and stone columns. The widely used technique during the last three decades is the last one which proved to be the best technique.

As a soil improvement method, the stone column technique has three purposes:

i. To reduce the settlement of highly compressible soil such as soft clay.
ii. To accelerate the stage of primary consolidation
iii. To enhance the bearing capacity.

This research is concerned with the last of these.

1- The Purpose of the Present Study

The theoretical study developed to supplement existing knowledge concerning the design of a stone column throughout the study. The emphasis is placed on the practical aspects of stone column design, contraction factor and parametric study.

The mesh model was selected to carry out a parametric study to assess the influence of some significant factor on the general behavior of stone, a detailed discussion of the contraction, utilization and limitations of stone column will be given in this research.

2- Analysis of the Basic Problem

2-1 Introduction

To carry out an analysis of the soil-structure stone column problem under various types of loading and boundary conditions, a finite element computer program required to simulate the construction of the real problem as close as possible.

The parametric study will provide guidelines for designers to understand the behavior of soil-stone columns system under different conditions, and it will be of most important value if the **standard** or **basic** problem is chosen in such a way that it represents typical field conditions and dimensions.

2-2 The Basic Problem

To carry out the mentioned parametric study, the basic problem must be introduced after taking some important facts into consideration. It must simulate the real problem as close as possible, must be simple, and the analysis of which needs the smallest possible time and effort. In simulating the desired real problem, problem geometry and material characteristics must be considered in the basic problem simulation.

2.2.1 Problem Geometry

The basic axisymmetric model chosen for the intended study shown in Fig. (1) Involves the cylindrical unit cell of soil and stone column. The soil was assumed to be homogeneous, soft clay underlain by a firmer stratum. Single stone column with a diameter equal to one meter and a length equal to 14 meter was embedded in the 24-m thick soil. Pressure was applied uniformly on the stone column. The thickness of the soil left below the tip of the stone column is taken to be 10 m. Because, it is known that the stress distribution appears at distance equal to 6D (where D is the diameter of the stone column) (1), and for more safety it was taken to be 10 m. According to the (2.1) stress distribution method, the stresses reaching the lateral distance from the center of the stone column equal to (D+Z)/2, thus a depth (Z) equals to 10 m and D equals to 1 m. The lateral distance is taken to be 9 m, for more safety. The boundary site conditions of the unit cell domain are shear free with no radial movements at the lateral sites and fixed with no vertical and horizontal movements at the bottom. The final geometry and the mesh of the basic problem are shown in Fig. (2), which consists of 266 nodal points and 216 two-dimensional quadrilateral elements.

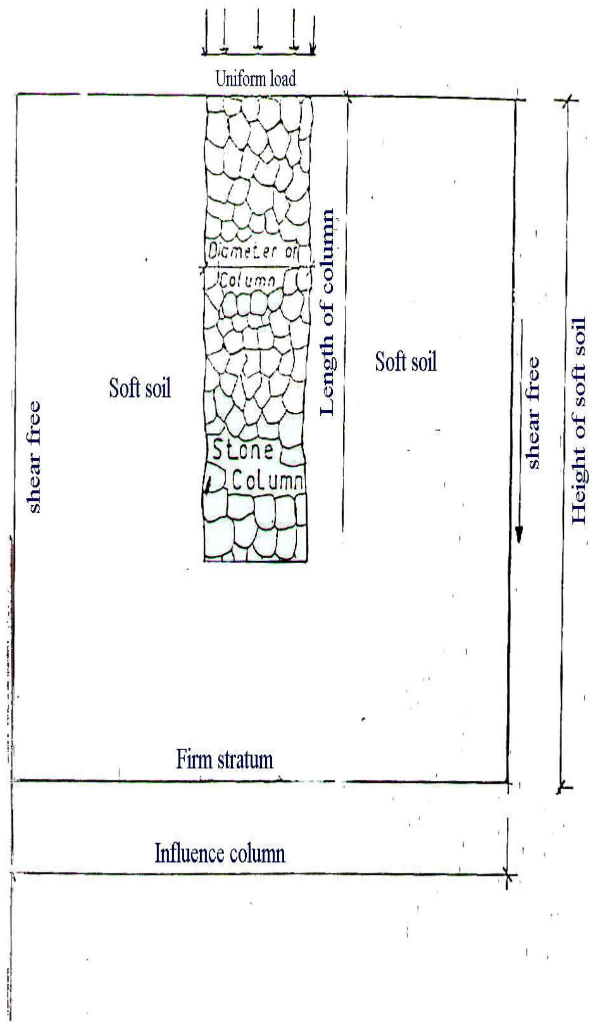


Fig. (1) The basic axisymmetric model chosen for the intended study

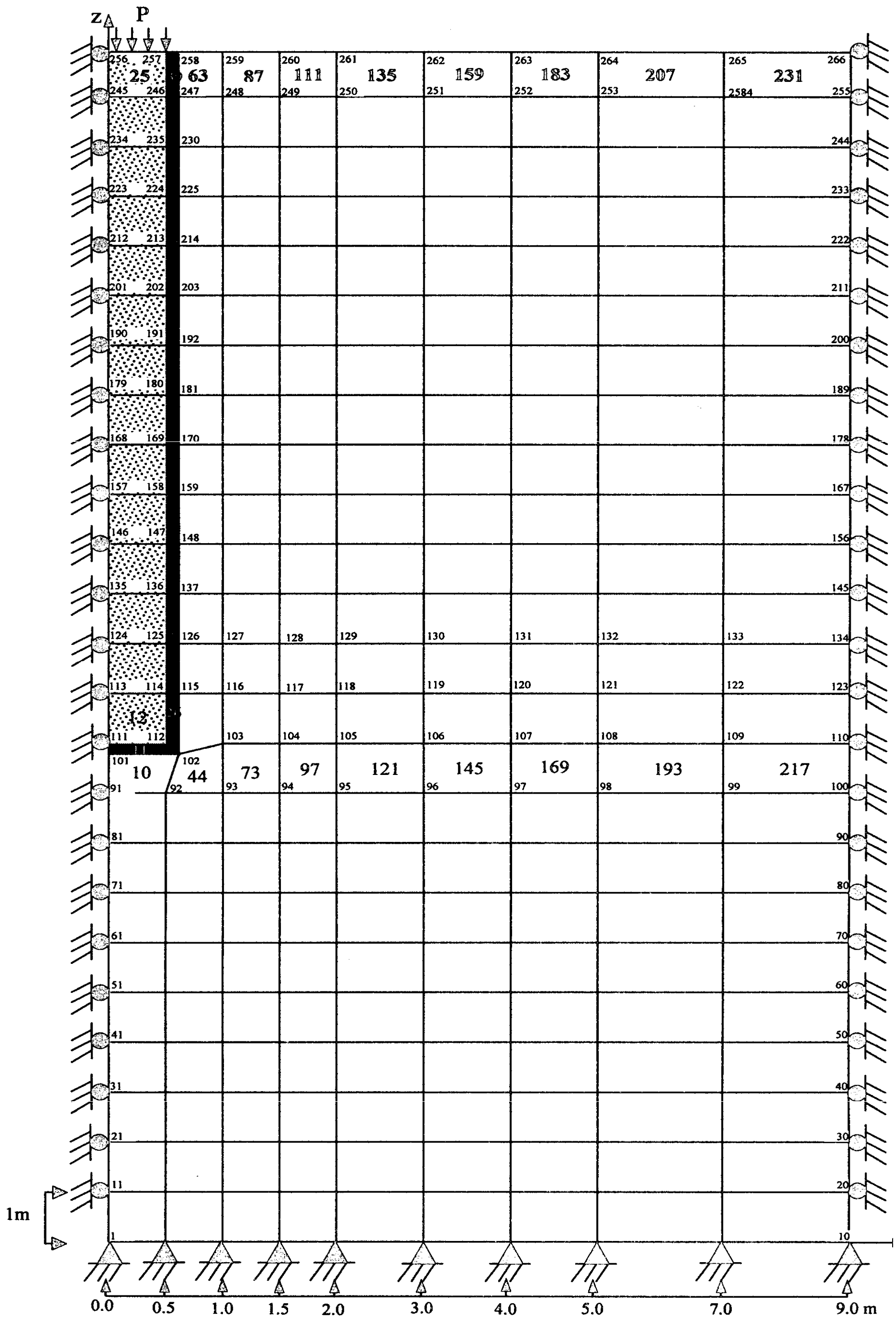


Fig. (2) The final geometry and the mesh of the basic problem

2.2.2 Material Characteristics

The surrounding soil is a c-Φ soil with a coefficient of at rest condition, K_o, of 0.8 which has a unit weight of 17 kN/m^3. Stone columns with a coefficient of at rest condition of 0.36, which has a unit weight of 20 kN/m^3. Both stone column and soil material are also adopted in this study and are approximated by the incremental procedure for calculating the stress-dependent tangent modulus After Shlash (1979)(2) (section 3.). The parameters used in the analysis of the basic problem are shown in Table (1).

Table (1). Nonlinear material characteristics used inthe analysis of the basic problem.

Parameters	Soil (soft clay)	Material of column (stone)
Unit weight kN/m^3	17	20
At rest pressure coefficient, k_o:	0.8	0.36
Cohesion intercept kN/m^3	25	0.0
Angle of internal friction Φ:	0	49
Poisson's ratio, ν :	0.45	0.30
Nonlinear modulus of elasticity		
k	150	2200
k_{ur}	450	2640
n	0.65	0.2
Rf	0.9	0.85

2.2.3 Loading

The stone column is loaded under equal increments of 100 kN/m^2. The loading is continued until the stone column failure occurs. The failure is defined to occur when a maximum settlement of 10% of the stone column diameter is reached (3).

2.2.4 Parametric Study

In view of the lesson provided by the literature review which presents the known previous work in the realm of stone columns, taking into consideration the suitability to finite element analyses, the following items were decided to be a parametric study set for the present work.

3.1 Results of the Basic Problem Analysis

The distribution of the stresses generated in the interface elements surrounding the pile is shown in Fig. (3) From which is possible to notice the peak values of the shear stresses occurring at the bulging failure at 4 m below top of the stone column due to the relative motion between the column and the in-situ soil. The indicated shear stress at the column-soil interface was computed by considering the variation in vertical stress increase in the stone from one increment to the next multiplied by the column cross sectional area divided by the outside surface area of the columns between the centers of these two columns. This magnitude of shear stress is considered to apply at mid distance between centers of the two elements under consideration. The largest of theses resulting shear stresses for all calculations was less than 13 kN/m^2. Regarding the shear stress distribution, there appears to be in good agreement with shear stress values obtained by (4) .

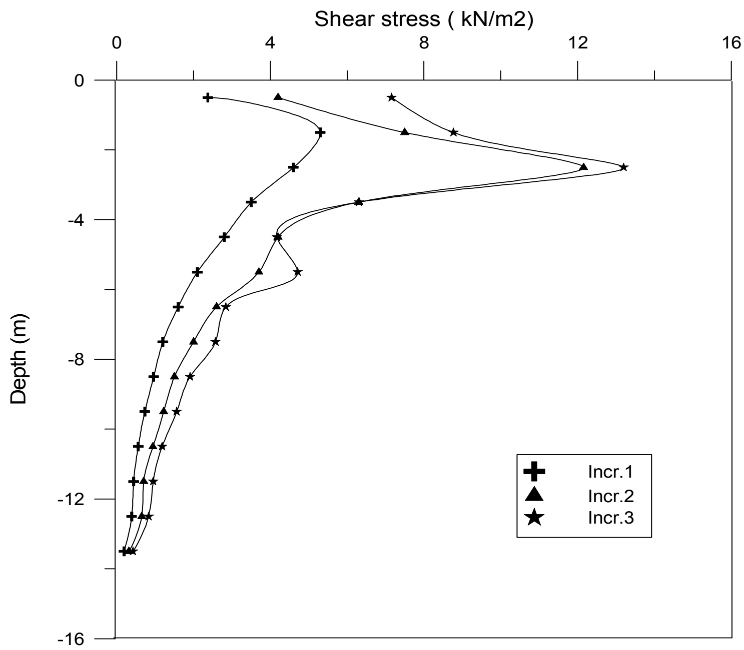


Fig.(3) Shear stress in interface element along the plane of stone col.

Figure (4) shows the lateral stresses in the interface element with depth. It is noticed that the lateral stresses increases with depth due to the column dilation, which applies lateral stresses to the surrounding soil that is resisted by passive pressure. Thus, there is a triaxial stress system with column conventional theory passive pressure implies an increase of pressure with depth.

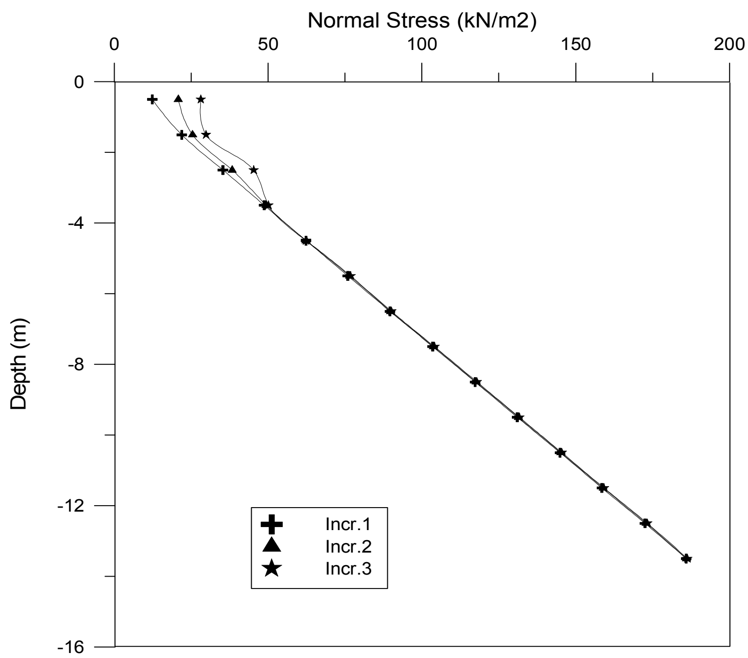


Fig.(4) Normal stress in interface element along the plane of stone col.

Figure (5) shows the shear stresses in elements along the stone column. The peak value of the shear stress occurs at the bulging failure and it is developed due to the relative motion between the soil and the stone column.

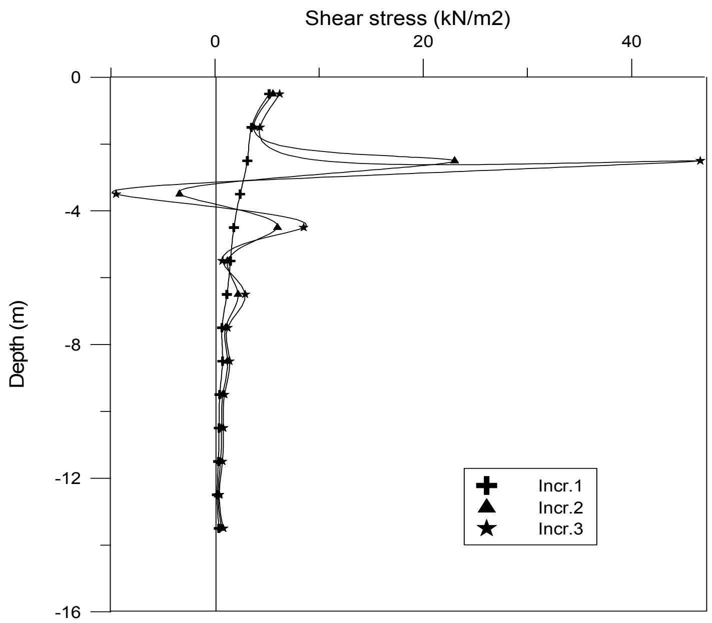


Fig.(5)Shear stress in element along the plane of the stone col.

Figure (6) shows the shear stresses in elements along the stone column. Lateral stresses are found to be increasing linearly with depth due to the fact that when the load is applied from the structure's footing, it tends to concentrate on the column as the stronger element of the composite foundation soil.

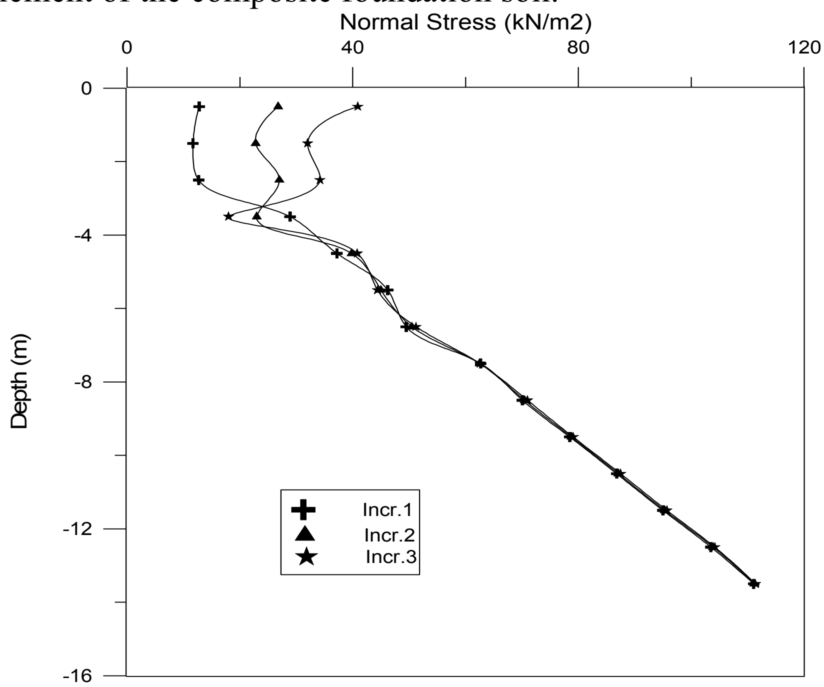


Fig.(6): Normal stress in element along the plane of stone col.

Figure (7) shows the vertical stresses with depth in elements along the stone column, from it we can notice that the first curve taken as arching v.s due to a smaller loading level (125 kN/m^2) applied on the stone column surface that means the bulging does not justify and the transfer stress does not occur. There appears to be in good agreement with results obtained by Mitchell, and Huber (1985)(1).

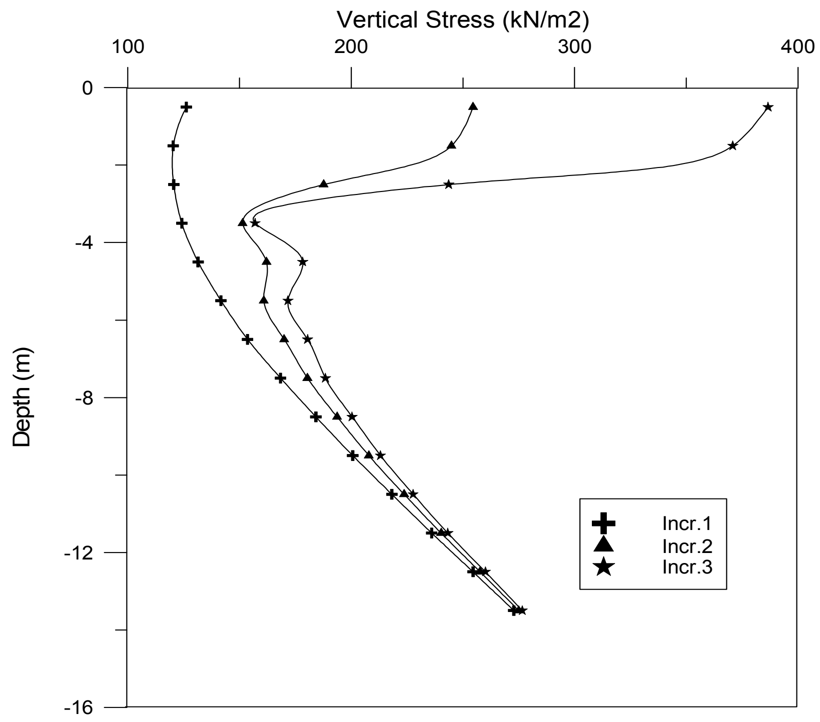


Fig.(7): Vertical stress in element along the plane of stone col.

At higher loading level (400 kN/m^2), the peak values of the vertical stresses occur at the upper 2 m the stone column and the curve of vs. squeezing which means that the vertical stress occurs at low values due to the transfer stress between the stone column and the native soil. After this the vertical stress increases with depth due to the increase of the geostatic stress in the element with depth.

From both lateral and vertical stresses developed in the soil around the stone column as shown in Figs. (5), (6), and (7) one can notice the difference of the stress values as compared with the lateral and the vertical stresses developed in stone bulging. This may by attribute to the high soil flexibility compared with the stone column flexibility, which causes stresses to be transferred to the more rigid structure element (stone column element).

Figure (8) shows the load-settlement curve of the stone column from which it can be noticed that the first portion of the curve approximately is linear and then it becomes curved then it drops after approaching the ultimate capacity (Q_u = 310 kN/m^2). The curve's drops means that the bulging failure has occurred. After that the curve tends to be approximately in a linear region with increasing value of settlement under increasing load.

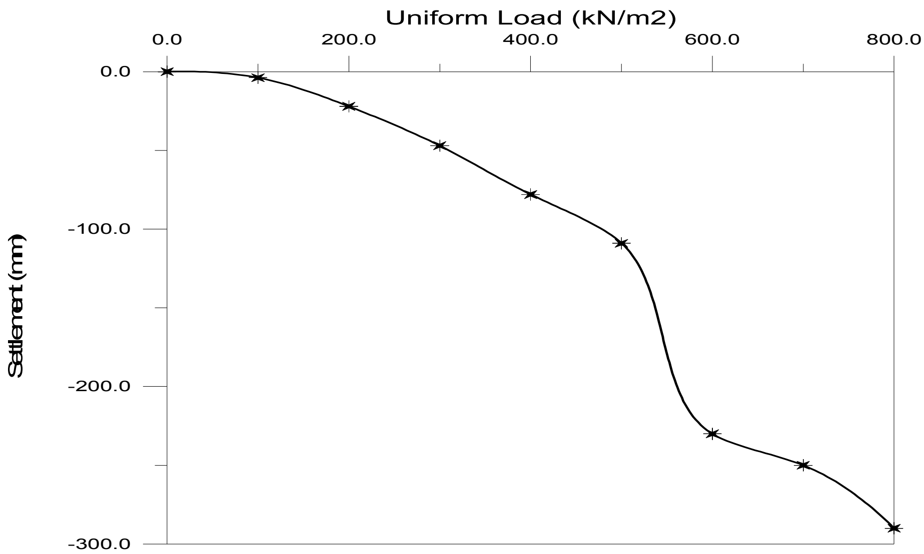


Fig.(8) Uniform load-settlement curve

The vertical, shear and lateral stress contour lines of the basic problem for the stone column and soil around it, we can notice bulging of the column clearly and the shear stress in the element of the column clearly as shown in Fig. (9) Below.

Figure (10) shows the vertical, shear and lateral stress contour lines for the soil around the stone column only.

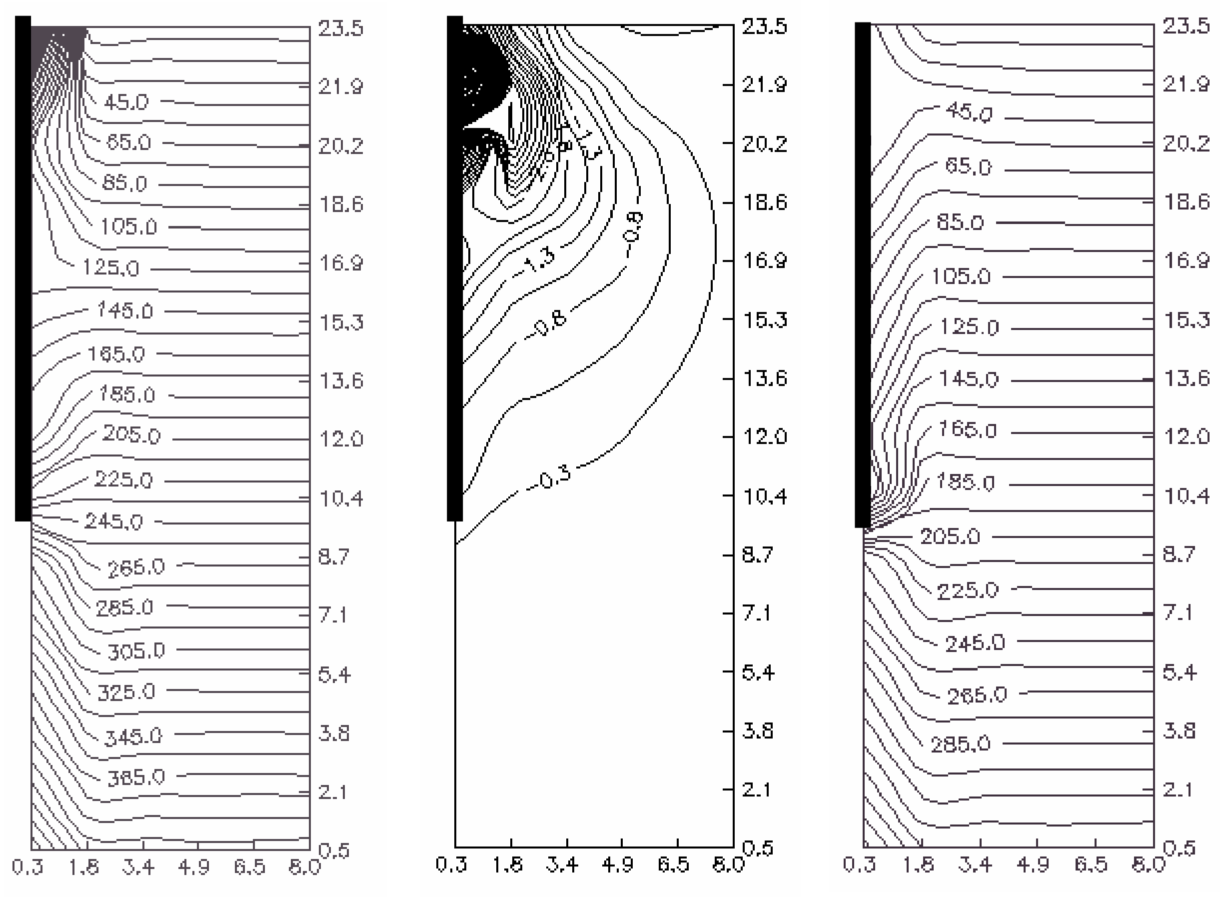


Fig. (9-a) Vertical Stresses **Fig. (9-b) Shear Stresses** **Fig. (9-c) Normal Stresses**

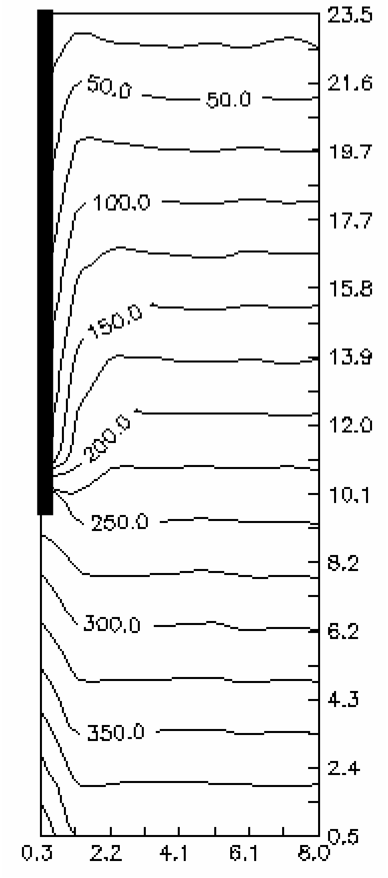
Fig.(9) vertical, shear and lateral stress contour lines for the soil around the stone column

Figure (9 a) shows the stresses to be transferred to the more rigid structure (stone column).

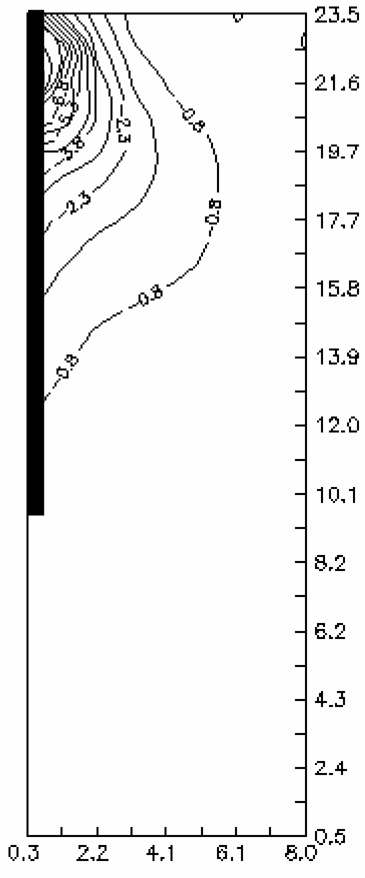
Figure (9 b) shows the contour lines of shear stresses from which it can be seen that the lines concentrate near the stone column surface and the slip surface occurs between the stone column and soil (clay). The maximum shear stresses are noticed in the bulging column failure.

Figure (10) shows the vertical, shear and lateral stress contour lines for the stone column at altitude load, it can also be seen that the adjoining soil elements move downward with the elements of the stone column. The elements of soil under the tip of stone column move downward under the stone column.

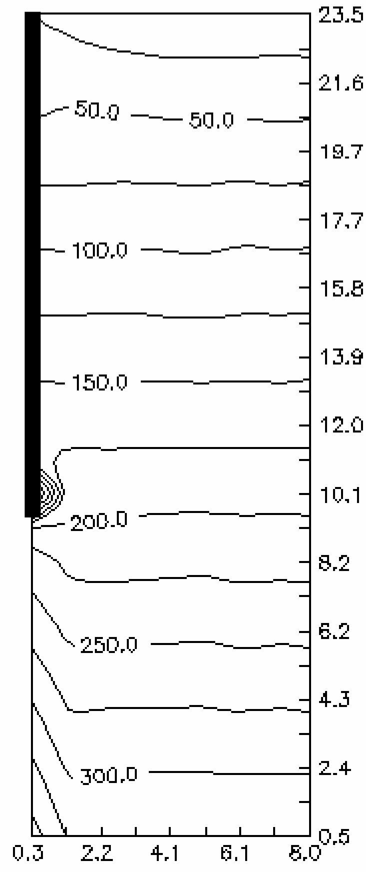
The displacements in stone column in upward are greater than downward due to bulging failure. It can be noticed that the displacement is stone column is greater than the displacements in soil elements. This difference indicates that a slip exits between stone column and the surrounding soil.



Fig(10-a)
Vertical stress (kN/m^2)



Fig(10-b)
Shear stress (kN/m^2)



(Fig(10-c)
Lateral stress (kN/m^2)

Fig. (10) Vertical, shear and lateral stress contour lines for the stone column at altitude load

3.2 Results of Analysis

3.2.1 Effect of the Depth Factor (λ)

To study the effect depth of the stone column, the problem was solved for five different lengths of the stone column, λ = 0, 0.3, 0.6, 0.9 and 1.0, (where λ is defined as the ratio of the length of stone column to the height clay layer).

The ultimate bearing capacity of stone column increases with increasing depth factor (λ).The ultimate bearing load recorded in case is λ = 0.9, and 1.0 **(12% &25%)** above the case of depth factor of λ = 0.6 (the basic problem) and forλ = 0.3 about **25%**less than result of Basic Problem. The capacity slightly increases with the increase of the depth factor because of the change in mechanism of load transfer. Figure (11), (12), and (13) show the uniform load-settlement curves.

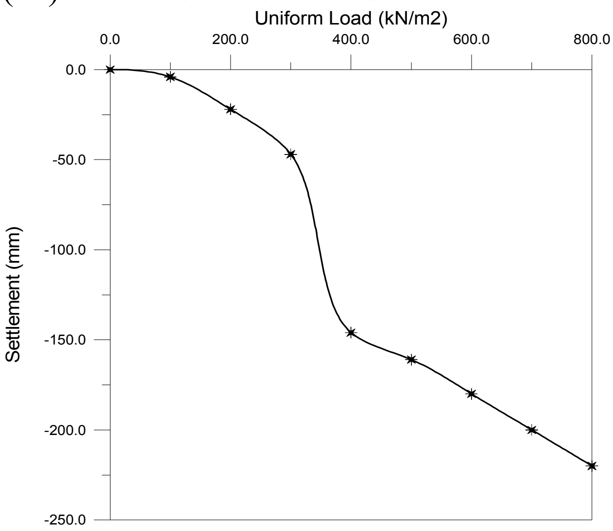


Fig.(11) Uniform load-settlement curve forλ = 0.3

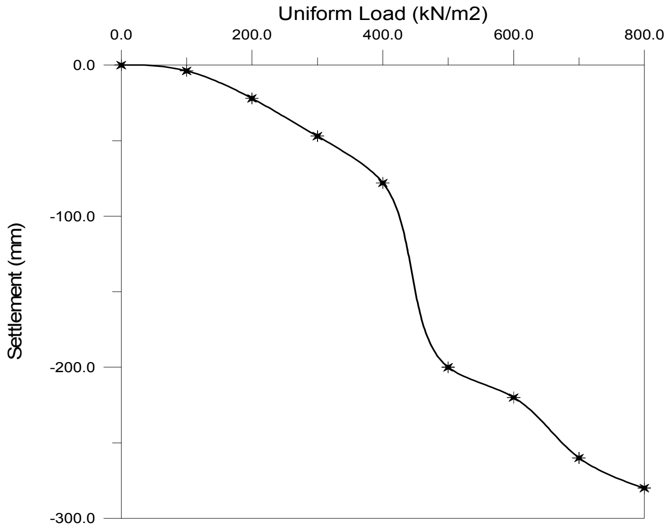


Fig.(12) Uniform load-settlement curve forλ = 0.9

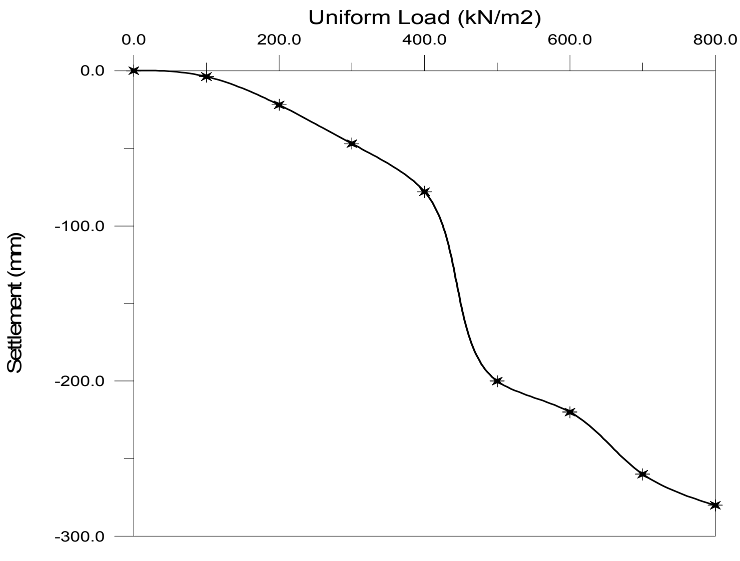


Fig.(13) Uniform load-settlement curve forλ =1

3.2.2 Effect of Diameter of the Stone Column

Four different diameters of the stone column were studied, namely, D =0.6, 08, 1.0 and 1.2. The figure shows that the settlement decreases as the diameter increases. Figure (14), (15), (16), (17) and (18) show the uniform load-settlement curves for the diameters mentioned above.

It is observed that the bearing capacity is less than that of the basic problem (D=1.0) by about **21%** for the case of D = 0.6 and **3.5%** for the case of D = 0.8 and more than the basic problem (D =1.0) by about **13.5 %** for the case of D = 1.2.This indicates that the effective diameter ranges from 0.8 to 1.1 m agreeing well with the prediction of most investigators as Terashi, Khazume, and Okada, (1991)(5).

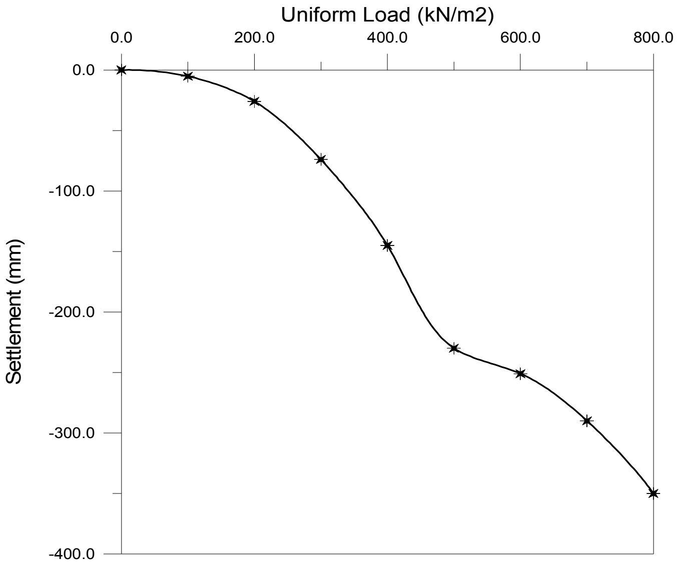


Fig.(14) Uniform load-settlement curve for stone column for basic problem Dim.=0.6

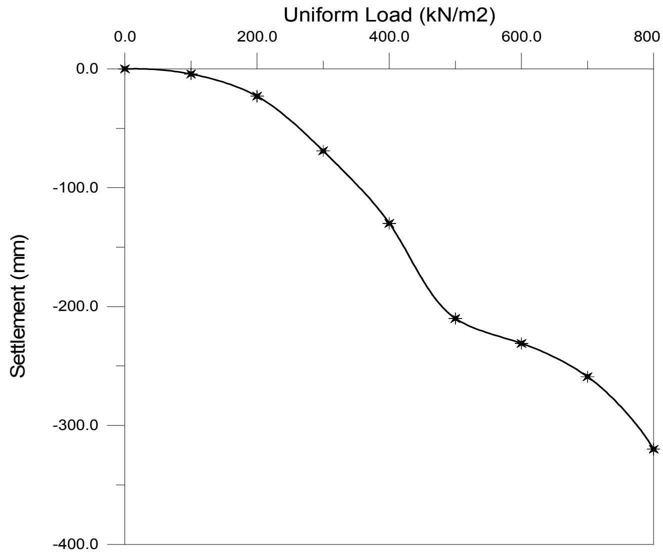


Fig.(15) Uniform load-settlement curve for stone column for basic problem Dim.=0.8

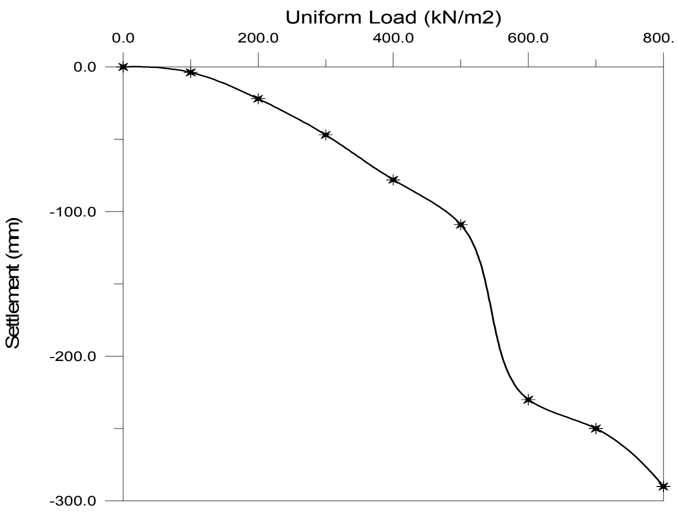


Fig.(16) Uniform load-settlement curve basic problei Dim. of stone col. =1.0 (m).

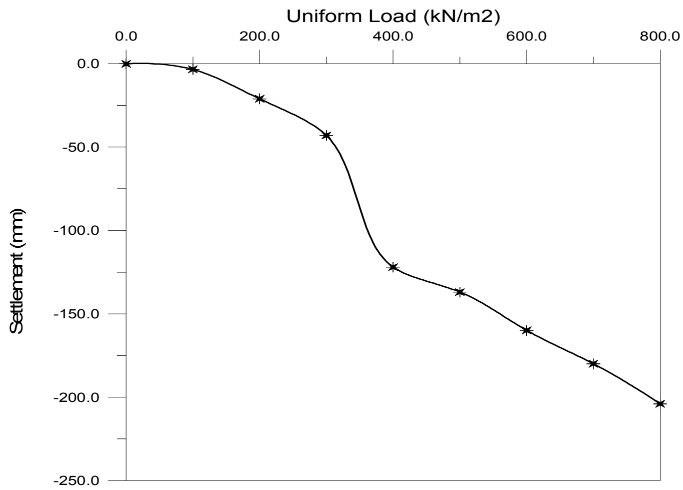


Fig.(17) Uniform load-settlement curve for stone column for basic problem Dim.=1.2

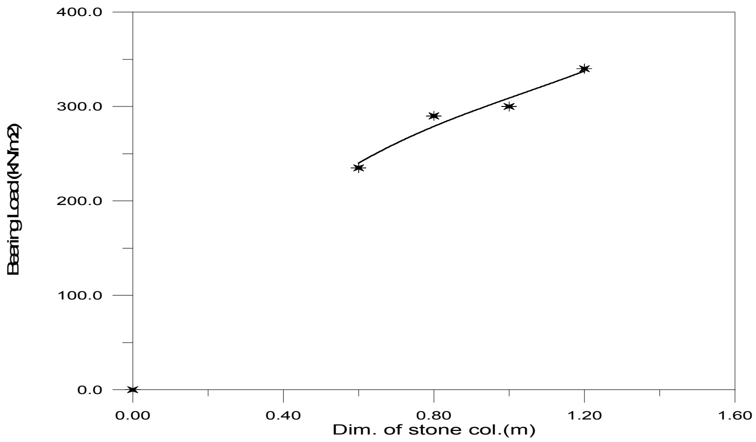


Fig.(18)Bearing load - Dim. of stone col. curve for basic problem.

3.2.3 Effect of Angle Friction of the Stone Column

Figure (19) depicts the relationship between the angle of friction of the stone column and the settlement at 400 kN/m^2 uniform load. The figure shows clearly that the settlement decreases as the angle of friction increases. The figure illustrates that for the same angle of friction, the effect of Poisson's ratio of the soil is insignificant. There are agreements with results obtained by Duncan and Chang, (1976)(6).

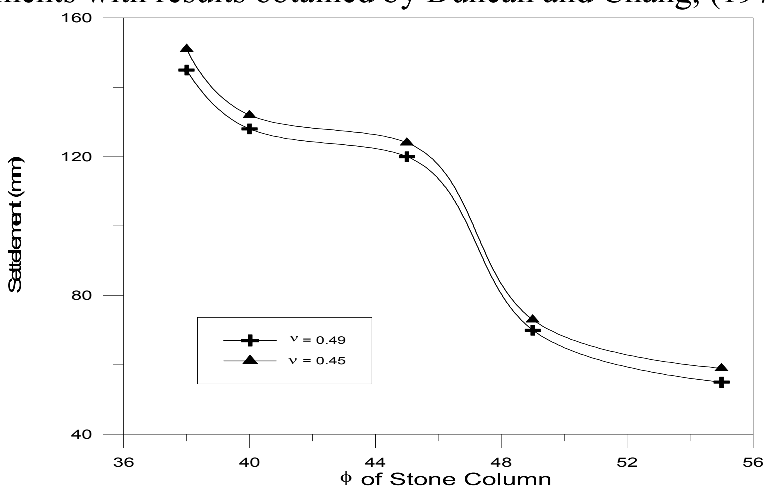


Fig. (19) Relationship between angle frection of stone col. with setlment

3.2.4 Effect of Poisson's Ratio

Figure (20) shows a plot of settlement versus Poisson's ratio of the stone column for two Poisson's ratio of the soil. The figure shows that the settlement decreases as Poisson's ratio of the column increases. The effect of Poisson's ratio of the soil is again, small.

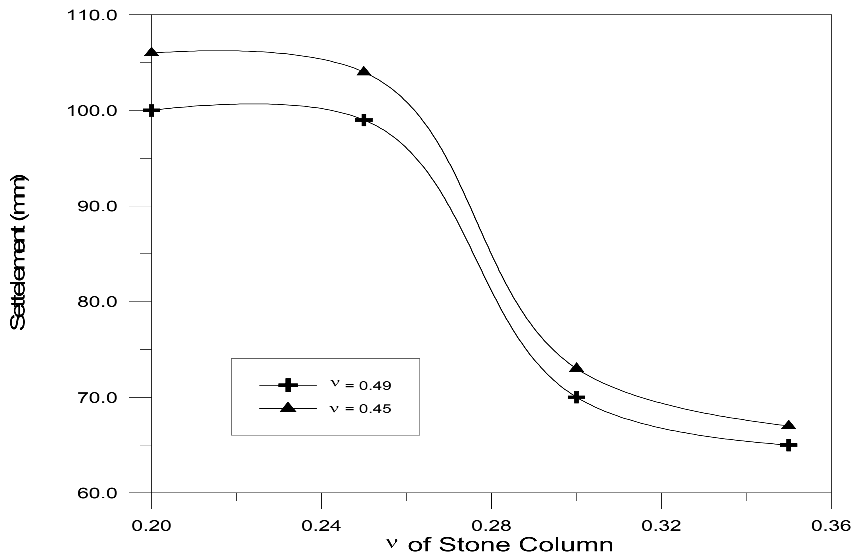


Fig. (20) Relationship between the possion ratio of stone col. with settelment

3.2.5Effect of Stiffnesses

3.2.5.1 Effect of Stiffness on Ultimate Bearing Capacity

Figure (21) shows the uniform load-settlement curve for the basic problem with different three column stiffnesses. For K =1800, the bearing capacity was **10%** less than the basic problem, K = 2200. For K = 2500, the bearing capacity was 30% above that of the basic problem. Hence, as the stiffness of the column increases, the ultimate bearing capacity increases.

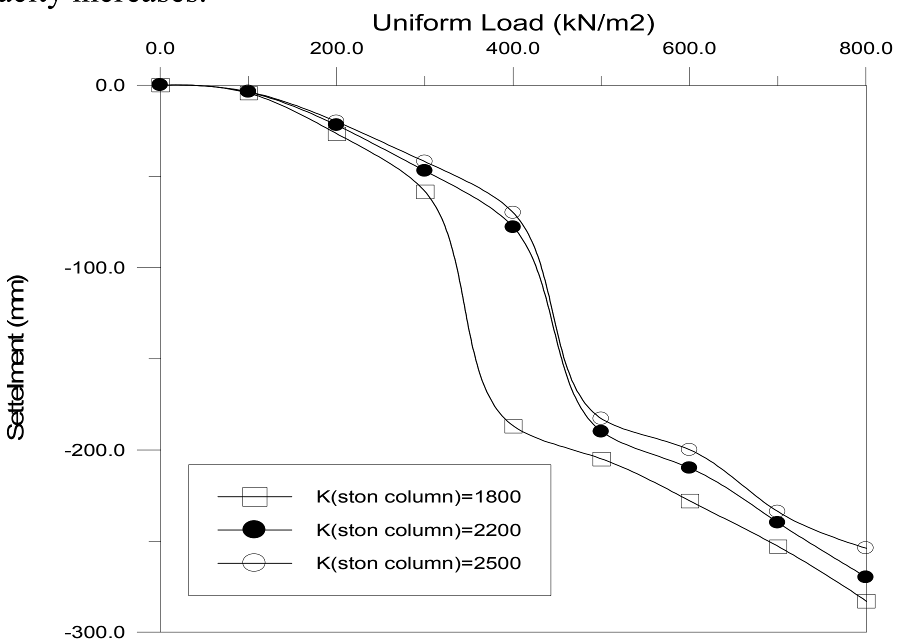


Fig.(21) Uniform load-settlement curve of the basic problem for soil at K=150, C=25 and ν =0.49.

4. Conclusion

The general behavior of soil- structure interaction problem dose not depends on a single parameter. The relative importance of a particular parameter depends on the effect of that particular parameter on the behavior of the soil-structure system relative to the variation of other parameters, from this idea ,one can see that a particular parameter may have a significant effect in certain condition but has not so in other conditions.

From the results of the basic problem and parametric study, the following conclusions can be drawn:

1- The ultimate bearing capacity of a stone column increases with increases depth factor (λ). The effect of depth factor is most significant than the effect of diameter of stone column. Also, it can be pointed out that the increase in length or diameter is usually not economical competitive with conventional deep foundations. Furthermore construction of very deep stone column is considered by many investigator pose series construction problems including stabilization of the hole and insuring that uncontaminated stone gets to the bottom and properly defined. The ultimate bearing capacity for basic problem λ=0.6, D_f =1.0 recorded 300 kN/m^2 .the ultimate bearing capacity for λ=0.3 recorded (25%) less than basic problem while for λ =0.9 and 1.0 the ultimate bearing capacity is (12%) and (25%) greater than the basic problem respectively. Furthermore the ultimate bearing capacity for D_f =0.6 and 0.8 is (21%) and (3.5%) less than basic problem respectively while D_f =1.2 the ultimate bearing capacity is (13.5%) further than basic problem.

2- The stiffness of stone column material is major factor in reducing the settlement of the treated soil and any increase in stiffness lead to increase the ultimate bearing capacity, as the increase in stiffness of soil leads to decrease the vertical and lateral displacement.

3- The influence of stone column material Poisson's and angle of internal friction ratio become more effective on the magnitude of settlement than Poisson's ratio of the soil ,furthermore the settlement decreases as the stone column Poisson's ratio increases.

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