Hyperbolic stress-strain parameters for non-linear Finite Element Analyses of stone column constructed in soft soil

Dr.Shlash K. T. Prof. University of Technology
Al-Hity A. A. Researcher University of Technology
Al-Ani Taher M.A. Researcher Anbar University.

المقدمة:

يَعتمدُ سلوكُ إجهادِ -انفعالَ أيّ نوع من التُرَبِ على عدد مِنْ العواملِ المختلفةِ وتتضمن ذلك الكثافةِ، محتوى الماءِ، التركيب البلوري، حالات التصريفِ، حالات الانفعالِ (انفعال مستوي ، triaxial)، مــدّة التحميــل، تأريخ إلا جهاد، الضغط المحصور، و إجهاد القصّ. في العديد مِنْ الحالاتِ يجب بأخذ بنظر الاعتبار حــساب هذه العواملِ المهمة وذلك بانتقاء نماذجِ معينة للتربةِ وفحصها تحت الشروطِ الــتي تُطــابق حالــة النمــوذج حقليا".وهذا من الصعب حتى ولو عمل النموذج بدقة وعلى أية حال فان النموذج عموما" يكون معــرض الى تشكيلة واسعة من الاجهادات الغير خطية الغير مرنة وتعتمد على مقدار ضغط الانحــسار (Confining Pressure) المسلط على النموذج أثناء الفحص. ولكي يؤدي تحليل هذه الاجهادات نحتاج إلى تقنيات لكي تفسر لنا هذه السمات المهمة لسلوك التربة.قام العالمان (Duncan)و(Chang) بتطوير الإجراءات العملية المبسطة لتمثيل سلوك اجهادات التربة المعتمد على الإجهاد اللاخطي بشكل أو حالة التي سهلت بشكل جيــد استعمالها في تحليل الاجهادات المتزايدة بطريقة العناصر المحددة.

التقنيات المستخدمة لتمثيل سلوك التربة اللاخطي على اعتبار أن التربة في الحالة الغير مرنة باستعمال العلاقــة واحدة للتحميل الأساسي (Primary Loading) والأخرى للتحميل (Loading) أو لإفراغ التحميل (Unloading).

Abstracte:

The stress-strain behavior of any type of soil depends on a number of different factors including density, water content, structure, drainage conditions, strain conditions (i.e., plane strain, triaxial), duration of loading, stress history, confining pressure, and shear stress. In many cases it may be possible to take account of these factors by selecting soil specimens and testing conditions which simulate the corresponding field condition. Even when this can be done accurately, however, it is commonly found that the soil behavior over a wide

range of stresses is nonlinear, in elastic, and dependent upon the magnitude of the confining pressure employed in the tests. In order to perform stress analysis of soils, it is desirable to employ techniques, which account for these important aspects of soil behavior.

Introduction:

Duncan and **Chang** [2] have developed a simplified, practical procedure for representing nonlinear, stress-dependent soil stress-strain behavior in a form which is very convenient for use in incremental finite element stress analysis.

Techniques for Representing Nonlinear soil behavior:

The procedure accounts for inelastic soil behavior by utilizing one relationship for primary loading and other for unloading or loading.

Primary Loading: using the hyperbolic stress-strain relationship proposed by Kondner [5]. It was shown that the tangent modulus for primary loading **(Et)** could be related to the principal stresses (**σ_1** and **σ_3**) by

$$Et = \left[1 - \frac{Rf(1-\sin\phi)(\sigma_1 - \sigma_3)}{2C\cos\phi + 2\sigma_3 \sin\phi} \right]^2 Ei \qquad \ldots(1)$$

In which **C** and **ϕ** are the Mohr-Coulomb shear strength parameters, **Ei** is the initial tangent modulus value, and **Rf** is the failure ratio or ratio between the compressive strength **$[(\sigma_1 - \sigma_3)_f]$** and the asymptotic stress difference for the hyperbolic stress-strain curve **$[(\sigma_1 - \sigma_3)_{ult}]$**. The variation of the initial tangent modulus value with confining pressure was represented by an empirical equation suggested by **Janbu** [6].

$$Ei = KPa\left(\frac{\sigma_3}{Pa}\right)^n \qquad \ldots(2)$$

In which the modulus number **K** and the exponent are both pure numbers and **Pa** is the value of atmospheric pressure expressed in appropriate unit. The values of the five parameters **C**, **ϕ**, **Rf**, **K** and **n** may be determined conveniently from the results of a series of triaxial or plane strain compression tests. The drainage conditions employed in the compression tests are chosen to correspond to the condition to be analyzed.

Unloading-Reloading: for unloading and reloading, many soils are nearly linear and elastic and their behavior may thus be accurately represented by a single modulus which is independent of the percentage of strength mobilized. The value of this unloading-reloading modulus, **Eur**, has, however, been found to berated to the value of confining pressure in the same manner as shown by Eq. (2). For the initial tangent modulus, the value of the exponent, **n**, in this relationship has been found to have essentially the same value for unloading and reloading as for primary loading. The value of the modulus number for unloading-reloading, **Kur**, may be determined readily from the results of these involving one or cycles of unloading and is always somewhat larger that the modulus number for primary loading.

Stress-Strain parameters for crushed stone-sand soils:

The crushed stone soils used in this study were well graded and density and the drain triaxial tests were performed on a number of these undistributed specimens at effective confining pressures of 2, 4, and 6 kg/cm^2 to determine stress-strain parameters for primary loading. The variations of stress difference with axial strain in these tests are shown in Figure (1).

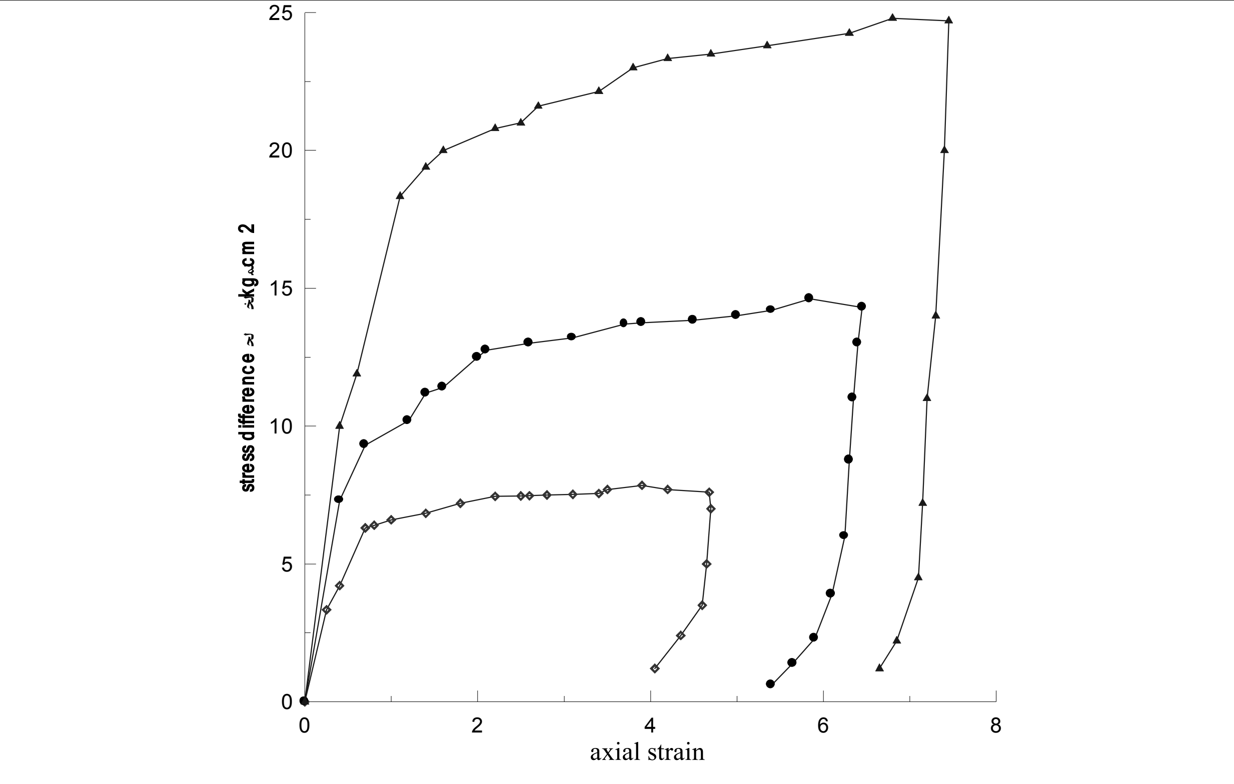

Fig.(1) Stress-Strain Curve from comparison triaxial tests on crashed stone (drained)

It may be noted that each specimen was unloaded after the peak strength was reached so that values of unloading modulus could be calculated. The strength parameters determined from these tests were C = 0.016 kg/cm^2 $\cong$ 0, and $\phi = 42^o$.

The stress-strain data determined in these tests have been on transformed axes in Figure (2) for the purpose of determining the values of initial tangent modulus, **Ei**, and asymptote value of stress difference **[(σ_1 - σ_3)$_{ult}$]**.

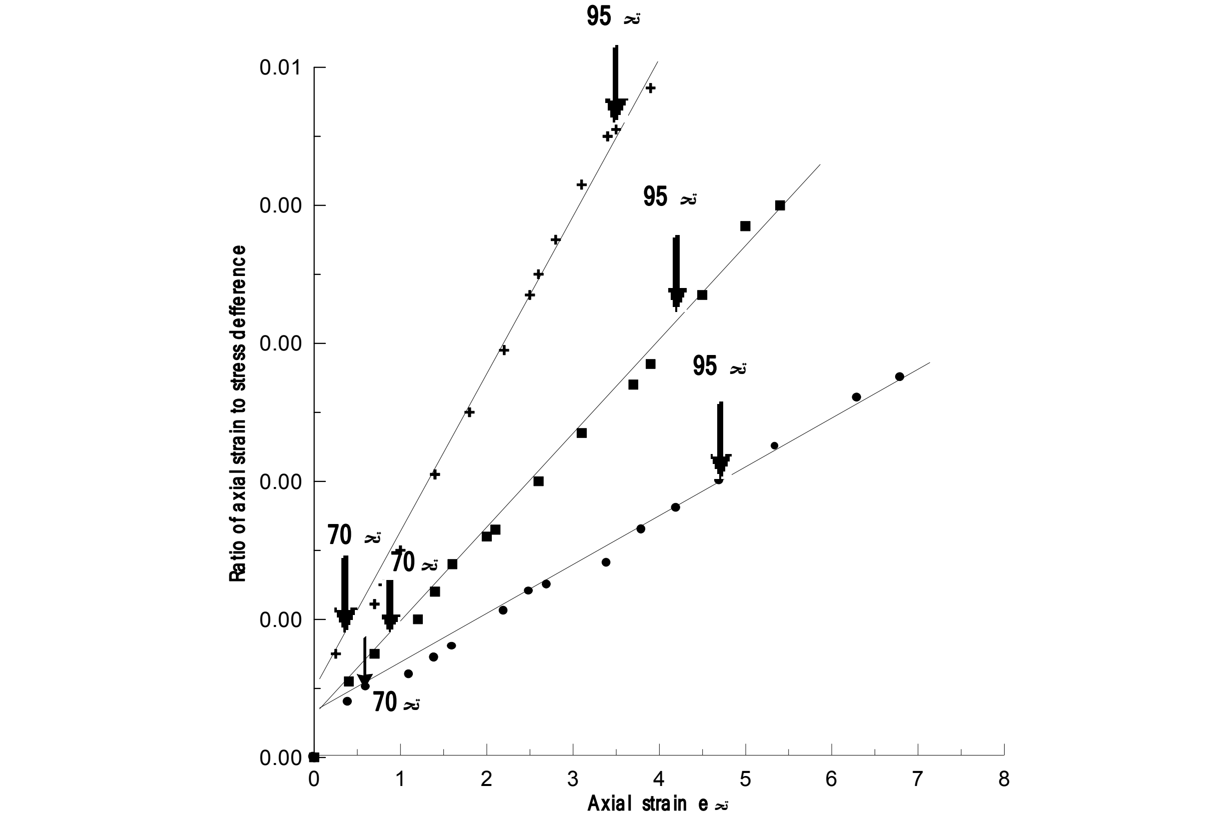

Fig. (2) Transformed hyperbolic Stress-strain relationship from triaxial tests on crashed stone (drained).

It may be noted that data diverge somewhat from a linear relationship at both low and high

values of strain indicating that the stress-strain curves for these tests are not precisely hyperbolic is shape. In accordance with the finding of pervious studies (**Duncan and Chang**)[2], the hyperbola were chosen so that they intersected the stress-strain curves at the origin **[(σ_1 - σ_3) = 0, ε = 0]** and at the points corresponding to 70% and 95% of the strength mobilized. The value of **Rf**, which are a measure of the difference between the values of **$(\sigma_1 - \sigma_3)_{ult}$** and the values of stress difference at failure, **$(\sigma_1 - \sigma_3)_f$**, were found to be 0.86. The values of **Ei** have been plotted against the corresponding values of **σ_3** in Figure (3) for the purpose of determining appropriate values of the parameters **K** and **n**. Because the samples tests were not perfectly homogenous, encompassing a wide range of grain size and gradation, the experimental data are scattered an appreciable degree, as indicated by the line of three point shown in Figure (3). The corresponding value of **K** and **n** were determined to be 2200 and 0.2 respectively, as indicated in Figure (3).

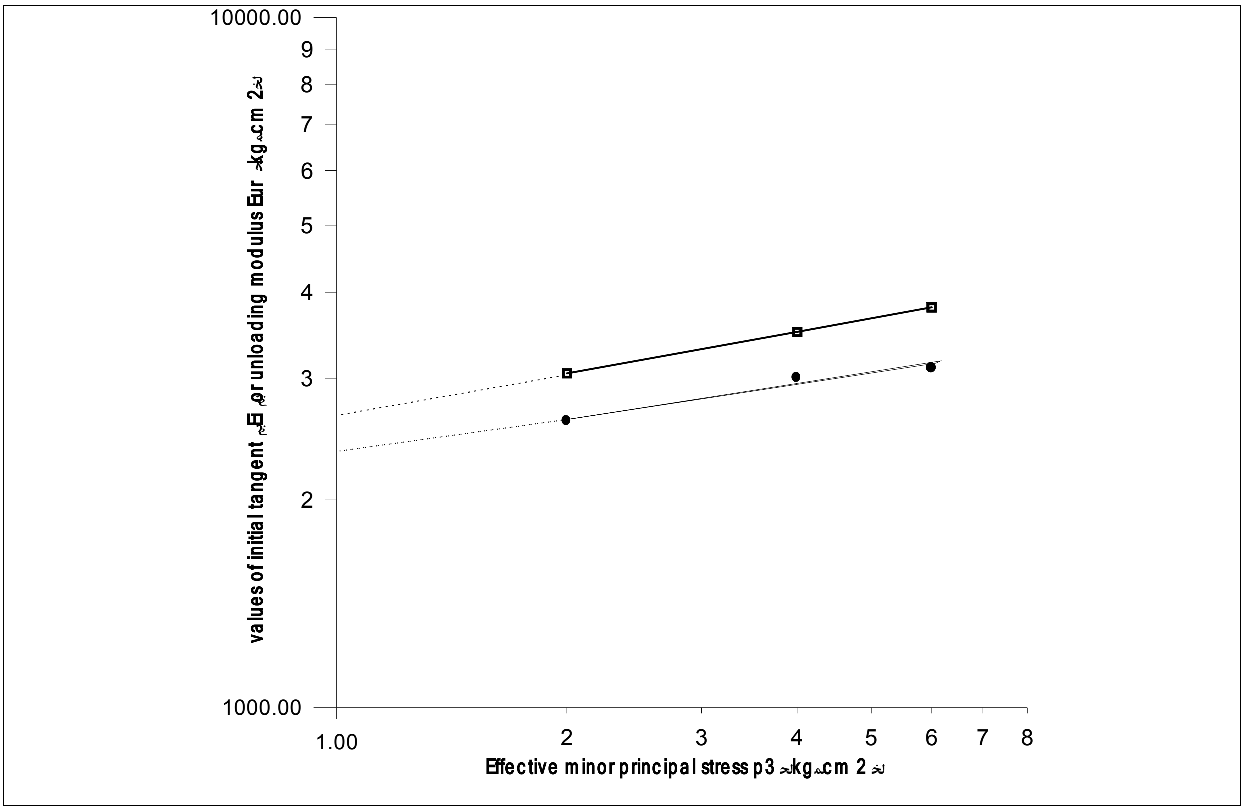

Fig. (3) Variation of values of initial tangent modules and unloading modules with effective minor principle stress for crashed stone (drained).

In order to study the unloading behavior of crushed stone soils, the values of unloading modulus determined from other tests were plotted against the corresponding values of minor principal stress as shown in Figure (1), an the average of range of values was selected for use in the analysis. The corresponding values of **Kur** was found to be 2650 and **n** = 0.2.

Stress-Strain parameters for clay under undrained condition:

Previous studies of initial tangent modulus values for clay under undrained test condition calculated by **Ladd**(7,8), have shown that the modulus values may be related to consolidation pressure by

$$Ei = KPa\left(\frac{\sigma_{3C}}{Pa}\right)^{n} \qquad \ldots(3)$$

In which $\boldsymbol{\sigma_{3C}}$ is the minor principal stress during consolidation and the value of the exponent, **n** is usually found to be close to unity. Unfortunately, not enough undisturbed specimens were available to determine the values of both **K** and **n** in this equation for site clays, and it was necessary to assume that the value of **n** was unity to compute corresponding values of **K** from the results of the tests conducted.

The stress-strain curve for unconsolidated-undrained triaxial compression tests on the undistributed specimens of clay is shown in Figure (4). Although, the compressive strength of the specimens is 14 kg per sq. cm.

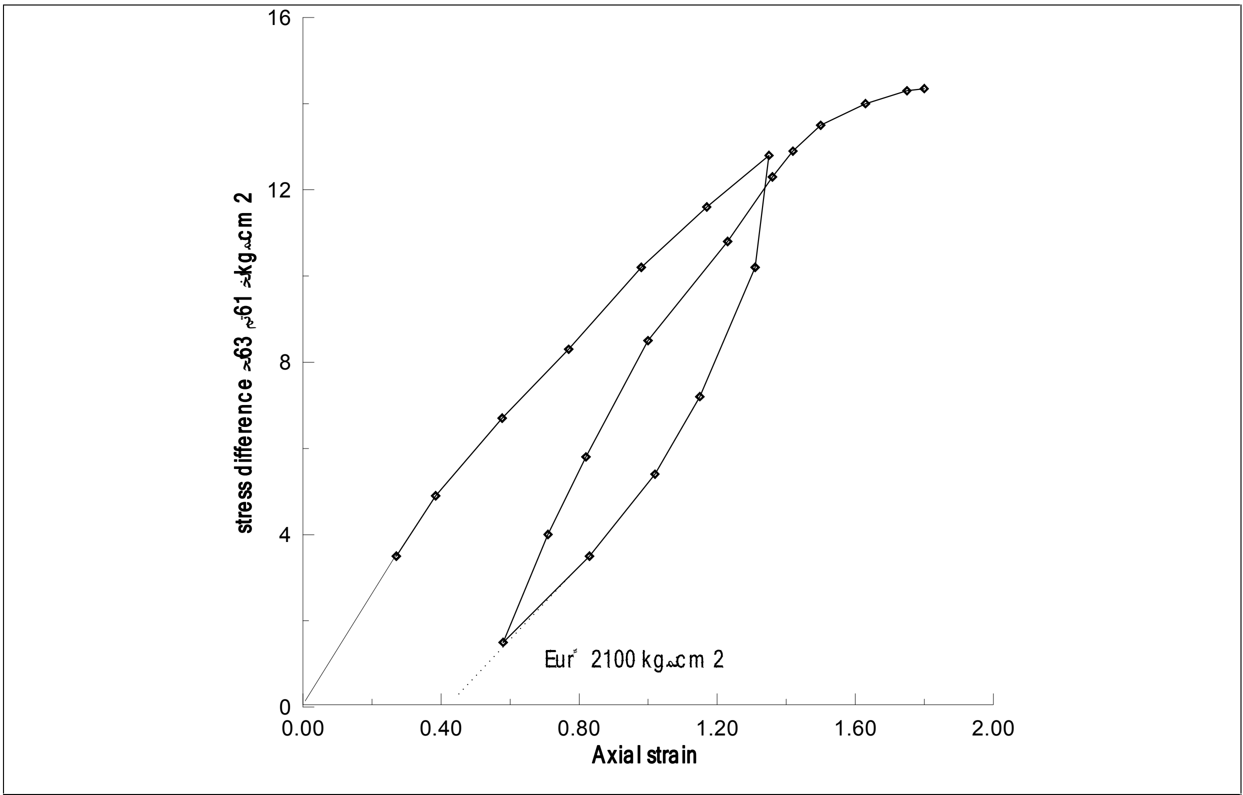

Fig. (4) uncosoildation-undrained test on clay

To relate the average values of initial tangent modulus, **Ei**, and unloading-reloading modulus **Eur**, to consolidation pressure, it was necessary to estimate the value of $\boldsymbol{\sigma_{3C}}$, in the ground at the locations from which these specimens were obtained. The over-consolidation ratio (which is obtained by dividing the maximum past effective pressure **Pc**, to which the clay has been stressed by the consolidation effective pressure, **Pc`**, at which the determination of **A** is carried out) at this depth was found to be about 1.5, and the value of coefficient of earth pressure at rest, $\mathbf{K_o}$, was estimated to be 0.8 using the relationship between over-consolidation ratio and $\mathbf{K_o}$ determined by (**Brooker and Ireland**) [1]. Thus, because the effective overburden pressure at sample depth was 12 kg/cm^2, it was estimated that the value of $\boldsymbol{\sigma_{3C}}$ is 9.6 kg/cm^2. Using the values of **Eur** determined from stress-strain curves in Figure (4), and values of **Ei** determined from Figure (5), and assuming that the value of the exponent n, was equal to unity. The value of **K** is found to be 150, and the value of **Kur** is found to be 220.

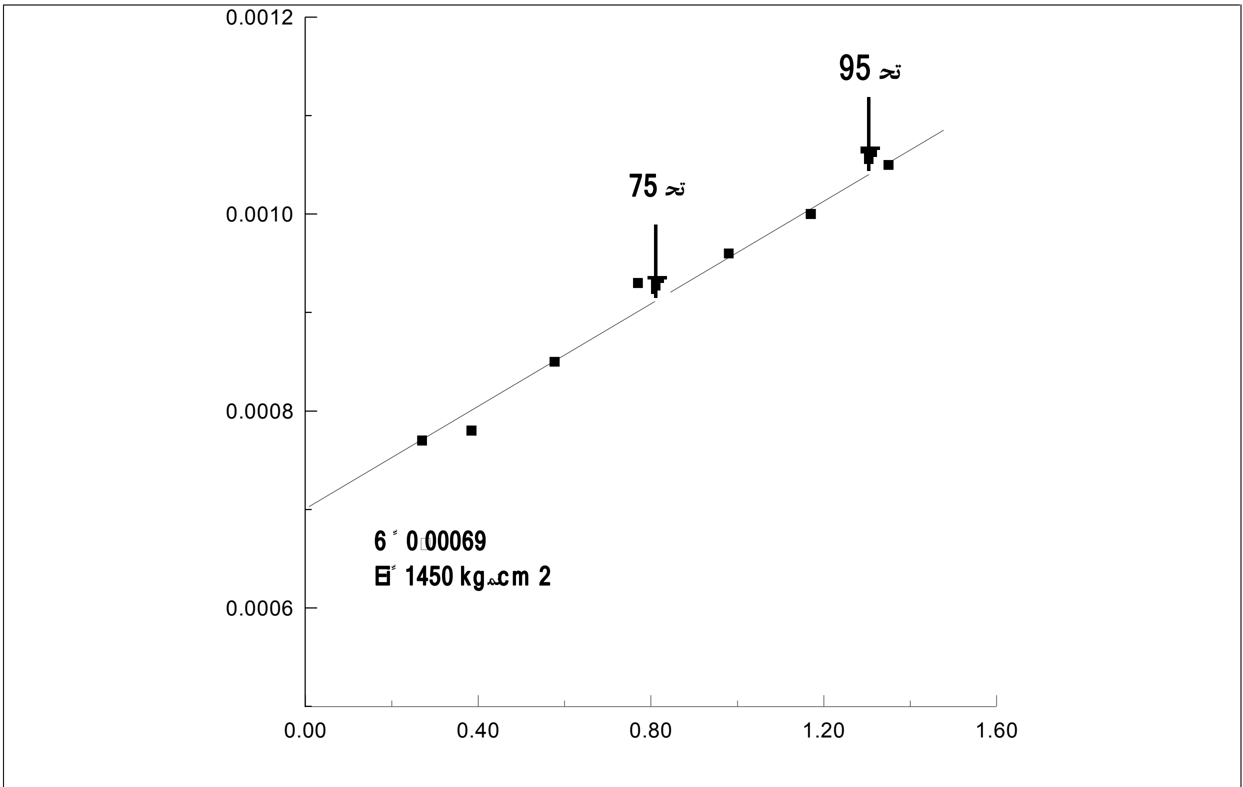

Fig. (5) Transformed hyperbolic stress-strain relationship for UU-tests on clay.

Because the clays were virtually saturated in-situ and therefore nearly in compressible under undrained conditions it was assumed for purposes of analysis that the value of Poisson's ratio these clays under undrained loading condition was 0.4.

Results of Analysis:

The nonlinear behavior of the in-situ soil and stone column crashed stone was approximated well by hyperbolic stress-strain and volume change parameters determined using the methods of Duncan and Chang.

Assuming of the properties and deformation parameter for each of the two-soil type is presented in Table (1). The soil parameters listed in this table were obtained by averaging the results of several triaxial compression tests for each soil. **Duncan and Kulhawy** (3), **Duncan and Wong** (4), **Mitchell and Huber** (6) , **Ladd and Richard** (8) in agreement with parameters for semi-similar soils publish the soil parameters.

Table (1) Summary of soil properties and deformation parameters

Soil type	Test condition	C (kn/m^2)	ϕ (degree)	Rf	K	Kur	n	ν
Clay	Undrained	25	4*	0.93	150	220	1*	0.49
Crushed stone	Drained	0	42**	0.86	2200	2450	0.2	0.3

* Taken (0.65)
** Taken 49

Comparison with theoretical results:

Mitchell and Huber (6) obtained experimental results and use it in their theoretical analyses of a stone column of 12.5m length, and 1m diameter. There results shown in Fig. (6).

The author also used an experimental data that he had get from agriculture college of AL- Anbar university, that proposed to be built in 1999, the experimental results taken to analyze a stone column with 14 m length and 1 m diameter as shown in Fig.(6). The results obtained by the author were agree with theoretical results obtained by **Mitchell and Huber** [(6)] .

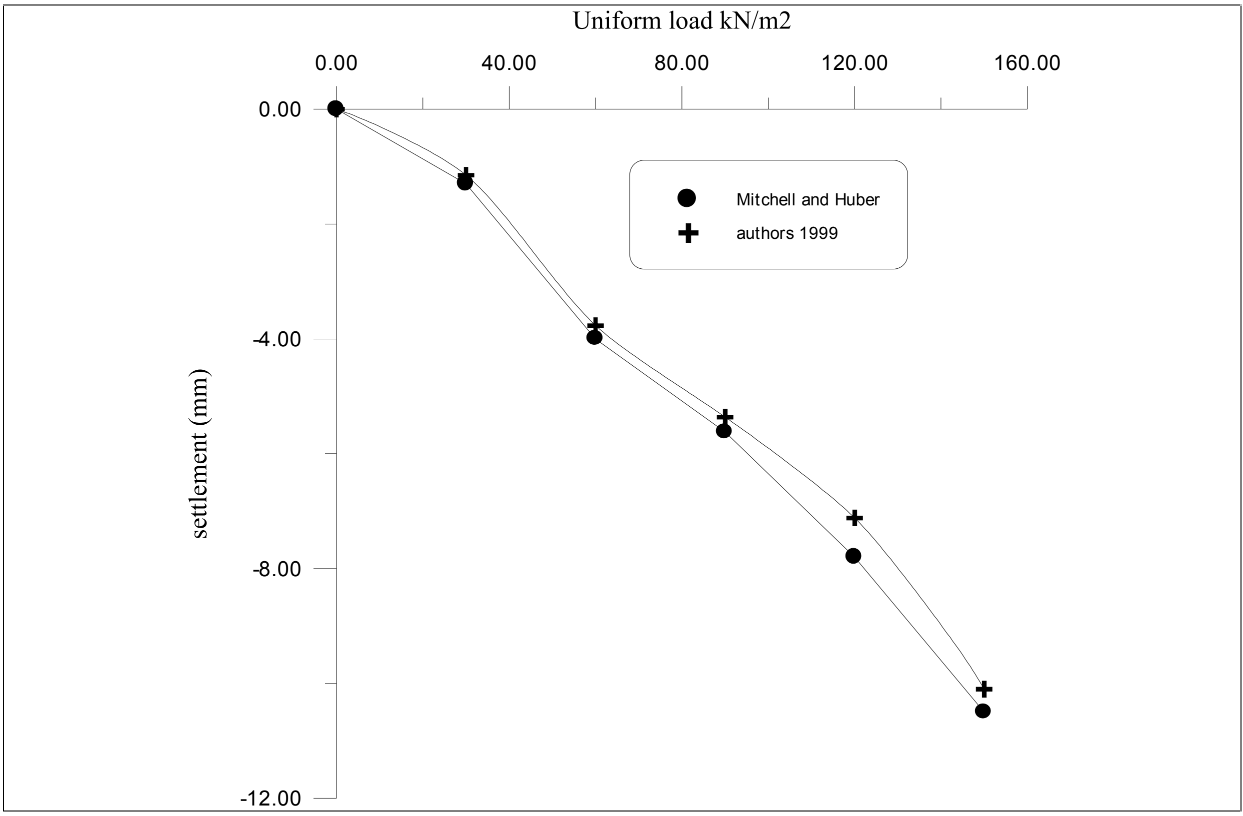

Fig. (6) Comparison Curve between author and Mitchell and Huber 1985.

Conclusion:

The analyses of stone column in soft soil show that the finite element method can be very useful for analysis of complex problems involving stresses and displacement in soil masses. These analyses are based on simplified, practical nonlinear stress-strain relationship for soils using parameters whose values may be determined from the results of standard laboratory tests on undistributed samples.

The results of this analysis correspond very closely with observed behavior of the stone column constructed within soil, with regard to both the magnitude of soil displacement and the development of regions of building failure.

Acknowledgment

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