STATIC AND DYNAMIC NON-LINEAR SOIL BEHAVIOUR BY THE BOUNDING SURFACE PLASTICITY MODEL FOR IRAQI SOIL

Dr. Omar Al-Farouk S. Al-Damluji
Assistant Professor
Department of Civil Engineering
University of Baghdad

Dr. Yousif J. Al-Shakarchi
Professor
Department of Civil Engineering University of Baghdad

Dr. Bushra S. Al-Busoda
Department of Civil Engineering
University of Baghdad

ABSTRACT

This paper studies the non-linear behaviour of a clayey Iraqi soil taken from Baghdad City, using the bounding surface plasticity model under static and dynamic loading. The bounding surface plasticity model needs a group of input parameters. Some of them are determined directly from triaxial and consolidation tests, while the others are evaluated from a parametric study using a computer program named EVAL.A series of advanced laboratory tests was performed after modifying and manufacturing some parts of the triaxial machine that are required for carrying out K_0-consolidated compression and extension tests. The soil test results are used in a computer program named EVAL to compute the soil properties and input parameters for the Iraqi clayey soil under consideration that are required in the bounding surface plasticity model. After the input parameters became available, one could use them in finite element computer programs for static and dynamic analyses. Two-finite element programs are chosen. The first program, named ACED used for solving coupled problems under static loading. While the second program, named DLEARN, is used for solving dynamic problem. The proposed abbreviation of Baghdad silty clay soil is (BBSC).. The program EVAL gives the input parameters for the Iraqi clayey soil that the bounding surface plasticity model needs.The values obtained of these parameters are as follows:

λ	κ	M_c	M_e/M_c	ν	R_c	R_e/R_c	A_c	A_e/A_c
0.064	0.017	1.20	0.676	0.4	2.70	0.85	0.05	0.80

الخلاصة

يقدم هذا البحث دراسة التصرف اللاخطي للتربة العراقية المأخوذة من مدينة بغداد عند تعرضها للأحمال الساكنة والحركية باستخدام نموذج السطح المحيط اللدن.يحتاج هذا النموذج الى عدد من المدخلات .بعض هذه المدخلات يحسب بطريقة مباشرة من فحص الأنضغاط وفحص الأستطالة الثلاثي المحاور.اما المدخلات الأخرى فيتم احتسابها بمساعدة برنامج حاسبة EVAL وبالمقارنة مع نتائج الفحوص المختبريةالى ان يتم الوصول الى اعلى درجة من التقارب بين نتائج البرنامج ونتائج الفحوص المختبرية. تم عمل دراسة لكل البحوث المنجزة التي تتطرق الى تصرف التربة اللاخطي باستخدام نموذج السطح المحيط اللدن وقد تبين من خلال ذلك انه ليس هنالك دراسة لتصرف التربة العراقية اللاخطي باستخدام النموذج المذكور وذلك لعدم

توفر المدخلات والخواص التي يحتاجها هذا النوذج ولعدم توفر الفحوص المختبرية المتكاملة العالية الدقة عن التربة العراقيةالتي يمكن بواسطتها الحصول على المدخلات التي يحتاجها نموذج السطح المحيط اللدن ، فقد أصبح الآن حاجة وضرورة لتوفير مثل هذه الفحوص واستخدام الخواص والمدخلات التي يحتاجها النموذج ثم الاستفادة منها في تحليل تصرف التربة اللاخطي تحت الأحمال الساكنة والحركية على تربة بغداد. أجريت مجموعة من الفحوص المختبرية المتقدمة على هذه التربة بعد إجراء بعض التحويرات على جهاز فحص الانضغاط الثلاثي المحاور وتصنيع عدد من الأجزاء اللازمة لإنجاز فحص الاستطالة وفحص الانضغاط على نماذج منضمة على خط K_0. وتم الاستفادة من نتائج الفحوص المختبرية في برنامج الحاسبة EVAL لاحتساب خواص التربة العراقية التي يحتاجها السطح المحيط اللدن ومقارنتها مع النتائج المختبرية وباستخدام أسلوب المحاولة والخطأ حتى الوصول إلى تطابق جيد بين نتائج البرنامج ونتائج الفحوص المختبرية. وبعد توفر الخواص والمدخلات الخاصة بالتربة العراقية التي يمكن أستخدامها لنموذج السطح المحيط اللدن، أصبح من الممكن استخدام هذه المدخلات في أي برنامج حاسبة يتضمن عمله نموذج السطح المحيط اللدن لحل كافة المسائل التي تصف تصرف التربة في حالة تعرضها للأحمال الساكنة أو الأحمال الحركية. وتم أختيار برنامج حاسبة يسمى ACED وهو برنامج حاسبة يعمل بطريقة العناصر المحددة لحل مسألة التربة تحت الأحمال الساكنة وقد تم أختيار مسألة الانضمام بأتجاه واحد . أما مسائل الاحمال الحركية فقد تم أختيار برنامج DLEARN وهو برنامج حاسبة يعمل بطريقة العناصر المحددة ويستعمل هذا البرنامج لحل مسائل الأحمال الحركية وقد تم أختيار مسألة باتجاه واحد عند تعرضها لاحمال الدالة الجيبية . وقد تم الحصول على مجموعة من الاستنتاجات من خلال إجراء هذا البحث أهمها أن تربة بغداد هي تربة طينية غرينية بنية اللون مفرطة الانضمام. كما تم الحصول ولأول مرة على المدخلات التي تصف التصرف اللاخطي لتربة بغداد باستخدام نموذج السطح المحيط اللدن والمبينة قيمها في الجدول الاتي:

λ	κ	M_c	M_e/M_c	ν	R_c	R_e/R_c	A_c	A_e/A_c
0.064	0.017	1.20	0.676	0.4	2.70	0.85	0.05	0.80

كذلك فقد تم وصف تصرف التربة تحت أحمال حركية وساكنة في تطبيقات مختلفة.وتبين من خلال تلك التطبيقات كفاءة نموذج السطح المحيط اللدن في وصف تصرف التربة عند تعرضها الى أحمال ساكنة وأخرى حركية كذلك اتضح من خلال التطبيقات دقة المدخلات التي تم الحصول عليها وهذا يثبت دقة النتائج المختبرية.

KEY WORDS
Bounding surface, plasticity model, static, dynamic, soil behaviour,non-linear

EXPERIMENTAL WORK

A series of classification, engineering and ultrasonic tests are performed in this study as shown in the testing program of **Fig. (1).**

The specific gravity tests are performed according to BS 1377:1975, test No.6.B.The liquid limit tests are carried out according to BS 1377:1975, test No.2.A using the cone penetrometer method, while the plastic limit is determined according to BS 1377:1975, test No.3. **Table (1)** shows the results of the physical tests.

Table (1) Results of Physical Tests

Depth (m)	Specific Gravity	Liquid limit %	Plastic limit %	Plasticity Index %	Wet Unit Weight Kn/m^3	Natural Water Content %
3-3.5	2.73	54	24	30	20.0	23
5-5.5	2.75	58	27	31	20.3	25
7-7.5	2.73	51	26	25	20.1	24

Standard consolidation tests are carried out according to ASTM 2435-70. These tests are conducted to study the behaviour of the soil during loading and determining its stress history. The results of these tests are shown in **Table (2).**

Table (2) Results of Standard Consolidation Tests

Depth (m)	C_c	C_s	P_c(kPa)	P_o(kPa)	OCR
3-3.5	0.162	0.001	250	47.5	5.26
5-5.5	0.156	0.037	115	68.7	1.67
7-7.5	0.300	0.048	155	145.5	1.065

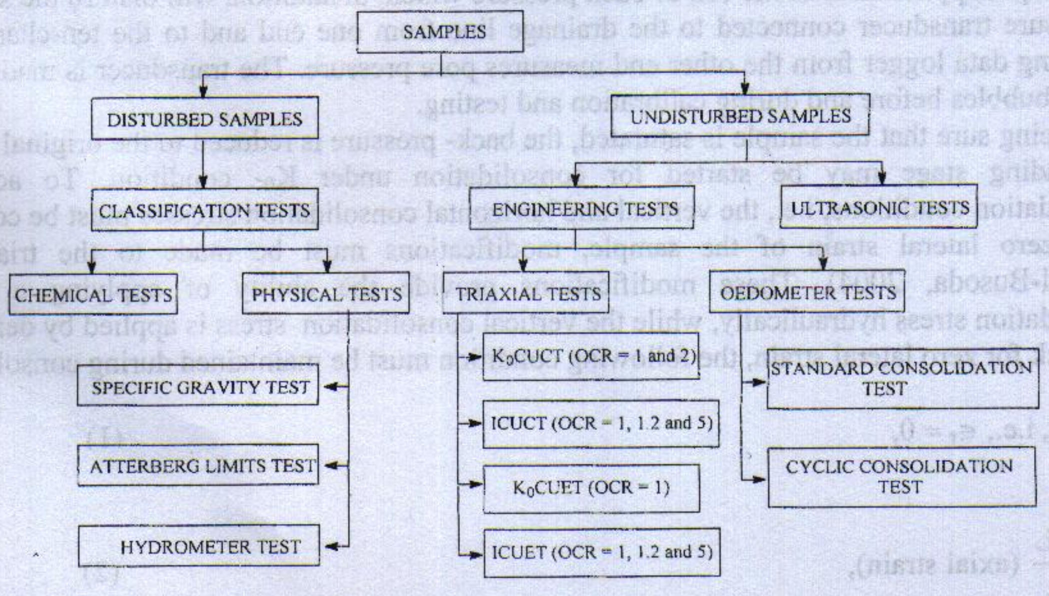

Fig. (1) Testing Program

In series C, D, E and F, various triaxial tests are performed under different conditions as shown in **Table (3).** These tests have few similar steps in common while most are different. In the similar steps, the system is flushed first and filled with distilled-deaired water. After that, a cylindrical triaxial sample of 38mm in diameter and 76 mm in height is placed with its filter paper (as side drains), if any, and membrane. The use of side drains is to accelerate the equalization of excess pore water pressure. After setting the sample, water may be allowed to run from the source up between the membrane and the sample to facilitate the removal of air before the loading cap is seated, as recommended by Bishop and Henkel (1962).

Table (3) Engineering Tests

Series mark	Test description
A	Standard consolidation tests LIR = 1,. LID = 1.
B	Cyclic consolidation tests (at least two cycles of unloading-reloading LIR = 1, LID = 1.
C	K_0 consolidated undrained triaxial compression tests (K_0CUCT → OCR=1 and 2)
D	K_0 consolidated undrained triaxial extension tests (K_0CUET → OCR=1).
E	Isotropically consolidated undrained triaxial compression tests (ICUCT → OCR = 1, 1.2 and 5).
F	Isotropically consolidated undrained triaxial extension tests (ICUET → OCR = 1, 1.2 and 5).

After the apparatus is assembled, a back pressure is used for saturation. The back pressure is always a little less than the confining pressure to ensure that the effective pressure remains positive. The back pressure is applied by means of the self-compensating mercury pressure system. The application of cell and back pressures should be in small increments to achieve an equalization and uniformity of stresses throughout the sample and to avoid swelling and compression of the sample during rapid applications of the cell or back pressure which, in addition, will disturb the sample.

A pressure transducer connected to the drainage line from one end and to the ten-channel digital indicating data logger from the other end measures pore pressure. The transducer is made free from any air bubbles before and during calibration and testing.

After being sure that the sample is saturated, the back- pressure is reduced to the original values and the loading stage may be started for consolidation under K_0- condition. To achieve K_0-consolidation conditions, i.e., the vertical and horizontal consolidation stresses must be controlled to attain zero lateral strain of the sample, modifications must be made to the triaxial setup (see Al-Busoda, 2004). These modifications provide the ability of applying a horizontal consolidation stress hydraulically, while the vertical consolidation stress is applied by dead loads.

To check for zero lateral strain, the following condition must be maintained during consolidation:

$$\epsilon_a = \epsilon_v, \text{ i.e., } \epsilon_r = 0, \tag{1}$$

where

$$\epsilon_a = \frac{\Delta L}{L} \text{ (axial strain)}, \tag{2}$$

$$\epsilon_v = \frac{\Delta V}{V} \text{ (volumetric strain), and} \tag{3}$$

ϵ_r = lateral strain.

Also, vertical and horizontal consolidation stresses must be increased gradually maintaining K_0-conditions.

It is important to note that the value of K_0 is computed using some empirical equations and considering the average value see **Table (4).**

After the completion of the consolidation stage, the shearing stage is started at a convenient rate of strain depending on the conditions of shearing.

Table (4) Empirical Equations for Evaluating K_0-Values

Empirical equation	K_0 value	K_0 (Average)	Reference
$K_0 = 0.4 + 0.007$ (PI), $0 \le PI \le 40$	0.61	0.61	Brooker and Ireland (1965)
$K_0 = 0.19 + 0.233\log(PI)$	0.534		Alphan (1967)
$K_0 = \lambda + \alpha(OCR - 1)$ At OCR = 1 $\lambda = 0.54 + 0.0044(w_L - 20)$, $\alpha = 0.09 + 0.001(w_L + 20)$	0.69		Sherif and Ishibashi (1985)

As for the differences in the steps of testing, they are mentioned henceafter. If the sample is sheared in an undrained compression state, then the back pressure valve is closed and the sample is sheared with a rate of strain of 0.12 mm/min with pore water pressure measurements. Furthermore, if the sample is sheared in an undrained extension state, then there some modifications must be made on the triaxial setup so that it becomes possible to conduct the test. The modifications include manufacturing some connective parts and a few others arrangements on the triaxial cell, similar to those done by AL-Kaisi (1978).

The procedure followed for the extension loading test is according to that followed by Bishop and Henkel (1962) and Al- Kaisi (1978).

The results of triaxial tests are shown in **Table (5).**

Table (5) Values of Angle of Internal Friction ϕ'.

Depth (m)	Isotropically consolidated specimens		K_0-consolidated	
	Compression CIUCT	Extension CIUET	Compression CK_0UCT	Extension CK_0UET
3-3.5	29°	27°	28°	26°
5-5.5	29°	29°	28°	28°
7-7.5	32°	28°	31°	27°
ϕ'_{av}	30°	28°	29°	27°

It is important to note that the value of the angle of internal friction for specimens that are consolidated isotropically seem to be larger than those under K_0-consolidation. Also, the value of ϕ for extension tests are lower than those obtained from compression tests. Similar results were found by Al-Kaisi (1978), Bjerrum (1972) and D'Appolonia et al. (1975).

One of the most useful coefficients in soil engineering practice is Skempton's pore pressure parameter at failure (A_f), since it is used to predict the induced pore pressures if the change in total stresses is known. The values of A_f obtained from triaxial tests vary from 0.5 to 1.0 for compression, which is within the range of normally consolidated clays as cited by Skempton (1954). While for extension, they vary from 0 to −0.2 which lies in the range of overconsolidated clays. The reason behind such behaviour for extension tests may be attributed to the procedure followed in performing the tests. When unloading during the test, the soil conditions change from normally consolidated initially to overconsolidated at the final step.

RESULTS OF EXPERIMENTAL WORK

The grain size distribution curve and the Casagrande chart show that the soil is a fine grained deposit. It is an inorganic silty clay of high plasticity (CH).The results of standard consolidation tests show that the soil has a compression index ranging from 0.156 to 0.30 which is a narrow one. These results are similar to those of Chicago silty clay and somewhat near the compression index of Bostton blue clay as presented by Holtz and Kovacs (1981). The values of the recompression index are considered to be within the reasonable range, whereas the values of C_s lie outside the range of 0.005 to 0.05 and should be considered questionable.The results of the triaxial compression tests (isotropically and K_0-consolidated undrained triaxial compression tests) show that the soil has an average value of angle of internal friction equal to 30° and 29° respectively.

The results of triaxial extension tests (isotropically and K_o- undrained triaxial extension tests) show that the soil has an angle of internal friction equal to 28° and 27° .It is important to note that the value of the angle of internal friction for specimens that are consolidated isotropically seem to be larger than those under K_0-consolidation. Also, the value of ϕ for extension tests are lower than those obtained from compression tests. Similar results were found by Al-Kaisi (1978), Bjerrum (1972) and D'Appolonia et al. (1975)..

Finally, the results of triaxial compression tests given herein agree with those of Al-Saadi (1989) performed on Al-Zaafarania soil while the results of triaxial extension tests are similar in behaviour to those of Al-Kaisi (1978).Also, the values of pore water pressures decrease in triaxial extension tests due to the decrease of stresses through the test.

Table (6) represents the results of ultrasonic tests. The shear moduli obtained are within the range for silty clay mentioned by Bowles (1996). Therefore, this confirms the reality of the results obtained from ultrasonic tests.

Table (6) Results of Ultrasonic Tests

Depth (m)	V_p (m/sec)	V_s (m/sec)	E (Kp_a)	G (kP_a)	υ	K (kP_a)
3-3.5	412	134	1.01×10^5	35104	0.44	2.85×10^5
5-5.5	1238	516	1.4×10^6	511212	0.40	2.26×10^6
7-7.5	966	998	1.28×10^6	483608	0.32	1.17×10^6

CONSTITUTIVE RELATIONS

A significant development in constitutive modeling was achieved when Dafalias applied the bounding surface concept to soils (Dafalias and Herrmann, 1982; Dafalias, 1986),. Instead of a classical yield surface, a bounding surface is defined so that the actual stress is mapped to the imaginary stress on the bounding surface. The distance between the real and imaginary stresses is used to specify the plastic modulus of the actual stress states in terms of bounding plastic modulus. The bounding surface theory allows a smooth transition of stress in an elastic state, within and on the bounding surface. The model is very relevant for simulating the behaviour of overconsolidated cohesive soils. Therefore, the bounding surface plasticity model is chosen for analysis in the present research. The bounding surface plasticity model needs a total of 15 or 17 parameters. These are M_c, M_e, κ, λ and v the traditional material parameters within the critical state soil mechanics context. The κ and λ can be determined by consolidation and rebound of oedometer specimens. v can be obtained from triaxial or ultrasonic tests. M_c and M_e can be reached at from the friction angle at failure in compression and extension triaxial tests. The remaining are the new material parameters. The R_c, R_e, A_c, A_e and T determine the shape of the bounding surface in compression and extension, the first two for ellipse 1, the second two for the hyperbola and T for ellipse 2.The R parameter is a material property different for each soil and will be found by a parametric study .The value T=-0.1 was chosen rather arbitrary and based on the fact that different values of T had little effect on the response, because very quickly the image stress moves from ellipse 2 to the hyperbola, Dafalias and Herrmann(1982). The five parameters, C, S, h_c, h_e and h_o are related to the response for overconsolidated states. The first determines the projection center I_c. The second determine the size of the elastic nucleus, a value of S =1 can be used for soils. This value causes the elastic nucleus to shrink to a point (the projection center) and thereby permits inelastic behaviour to occur at any point within or on the bounding surface. h_c and h_e are the values of the shape hardening factor h in compression and extension respectively. h_o is typically set equal to the average of h_c and h_e. Also, m can be taken equal to 0.02 ,Dafalias and Herrmann (1982).

In geotechnical engineering libraries, there are complete records of laboratory soil test results for many soils (Boston blue clay, Chicago clay, etc.). Therefore, there is a high capability of using these results for simulating and modeling their behaviour. Also, there is a capability of getting appropriate input parameters for high quality soil models. Thus, a precise description and good ability of analysis could be obtained. This positive point could be used in design and analysis in practice.

For cohesive soils, program EVAL has been written for obtaining the input parameters that are required by the bounding surface plasticity model , Dafalias and Herrmann (1986). EVAL can be used for predicting the behaviour of homogeneous soil samples subjected to arbitrary stress and strain histories for either drained or undrained conditions. EVAL performs, essentially, single-element incremental-iterative finite element analyses of soils under homogeneous states of stress and strain. The soil test results are used in computer program EVAL to compute the soil properties and input parameters for the Iraqi clayey soil under consideration that are required in the bounding surface plasticity model. A trail and error procedure is used until reaching an acceptable convergence with experimental results. The program EVAL gives the input parameters for the Iraqi clayey soil that the bounding surface plasticity model needs.The values obtained of these parameters are shown in **Table (7).**

Table (7) Complete Set of Input Parameters of Bounding Surface Plasticity Model for Baghdad Brown Silty Clay (BBSC)

λ	0.064	R_e/R_c	0.85
κ	0.017	A_e/A_c	0.80
M_c	1.2	C	0.0
M_e/M_c	0.676	S_p	1.0
v	0.4	m	0.02
R_c	2.7	h_e/h_c	0.5
A_c	0.05	h_c	2.0
T	-0.1	h_o	1.5

FINITE ELEMENT APPLICATION

It is useful to make use of the parameters given in **Table (7)** in solving practical engineering problems and studying the behaviour of the Iraqi clayey soil under static and dynamic loading.

Static Problem

To analyze a static problem, the finite element computer program ACED used by Al-Damluji (1981) and developed and verified by Al-Ebady (2001) is chosen. The ACED program is used to solve one -dimensional consolidation problems of soils using the bounding surface plasticity model.

Dynamic Problems

To study the behaviour of the Iraqi clayey soil under dynamic loading, a finite element program is chosen. This program is named DLEARN, Hughes (1987). This program was developed and verified by Al-Tae'e(2001). The developed program (DLEARN) is used here for solving dynamic problems under sinusoidal

Applications and Discussions

The geotechnical engineering problems that are selected to be applied in a static state are one-dimensional consolidation. While, in the dynamic state, the behaviour of a one-dimensional column of soil under sinusoidal loading is studied.

Applications on Static Problems

A one-dimensional plane strain consolidation problem is solved in this section. The problem is similar to that solved by Desai and Siriwardane (1984). Isoparametric eight-nodded elements have been used instead of triangular elements that were used by Desai and Siriwardane.
The same problem was solved by Al-Ebady (2001), but it was not solved an Iraqi soil and the internal input parameters were assumed and collected from different references.
The finite element mesh is shown in **Fig. (2).** The width of loading B is assumed to be equal to 0.333 m. An external surface load (p_0 = 50 kPa) is applied at the top surface of the model. The input parameters used in this problem are shown **in Table (7)** as found in the previous section.
Fig. (3) shows the variation of surface settlement with time. It is clear that the settlement is little at early times and increases logarithmically until it reaches the final settlement after "1631" days. Moreover, the distribution of pore water pressure jumped over the value of the applied stress at the moment of application of load. This may be attributed to the Mandel Cryer effect (Scott, 1978). Then it decreases with time due to the dissipation of water out of the soil.

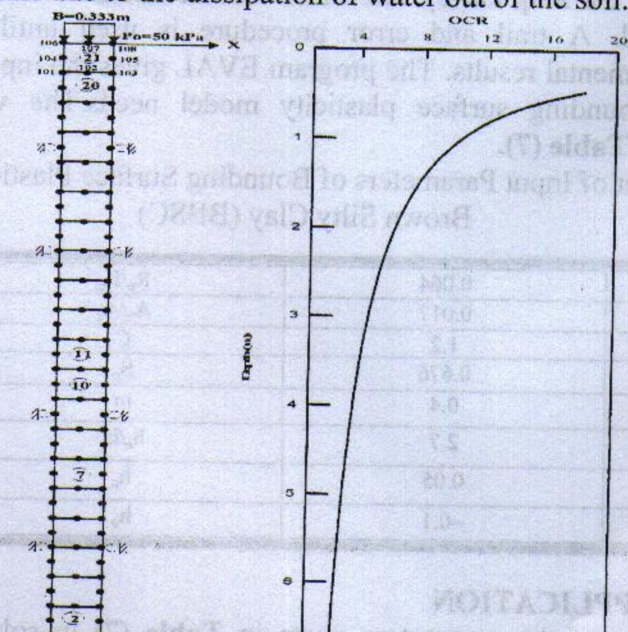

Fig. (2) Finite Element Mesh for a One-Dimensional Consolidation Problem.

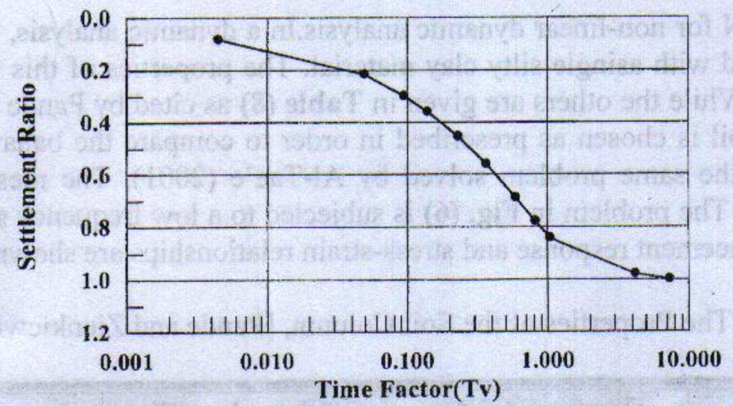

Fig. (3) Variation of Surface Settlement Ratio (Settlement at any Time/Final Settlement) with Time.

Furthermore, the distribution of pore water pressure with depth is shown in **Fig. (4)** at different times. Uniform isochrones are shown. Also, the pore water pressure is zero at the top surface and at a depth of 7 m due to the two-way drainage boundary condition and has a maximum value at the mid depth of the soil column.

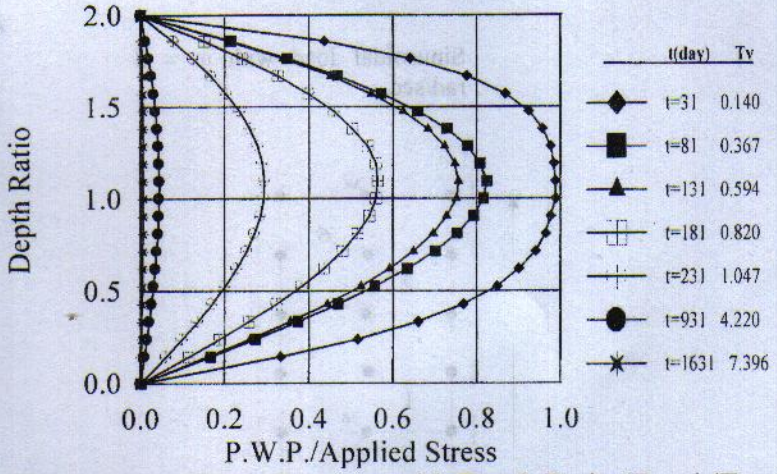

Fig. (4) Variation of Excess Pore Water Pressure with Depth Ratio (Depth/Total Depth) at Different Times.

Figure (5) shows the variation of pore water pressure with time at element (7).

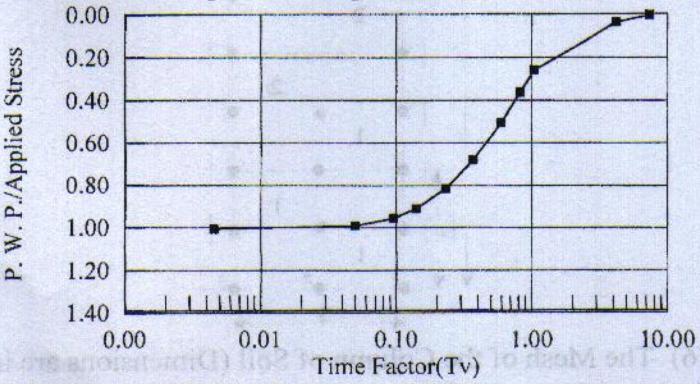

Fig. (5) Variation of Excess Pore Water Pressure with Time at a Typical Element (Element 7).

Applications on Dynamic Problems

The selected dynamic loads In this sub-section, the behaviour of Iraqi clayey soil under dynamic are sinusoidal load for the one-dimensional problem. The application is carried out using the computer program DLEARN for non-linear dynamic analysis.In a dynamic analysis, a non-homogeneous soil column is assumed with asingle silty clay material. The properties of this material are those of the Iraqi clayey soil. While the others are given in **Table (8)** as cited by Pande and Zienkicwicz (1982). This column of soil is chosen as prescribed in order to compare the bahaviour of the Iraqi clayey soil with that of the same problem solved by Al-Tae'e (2001). The mesh of the soil column is shown in **Fig. (6).** The problem in **Fig. (6)** is subjected to a low frequency sinusoidal load at the top surface. The displacement response and stress-strain relationships are shown in **Figs. (7) and (8).**

Table (8) The Properties of the Soil Column, [Pande and Zienkicwicz, 1982].

Material zone		Elastic modulus (MN/m^2)	Poisson's ratio v	Unit weight (t/m^3)
1	Alluvium	200	0.4	2.09
2	Hydraulic fill sand	90	0.41	2.02
3	Clay core	927	0.4	2.01

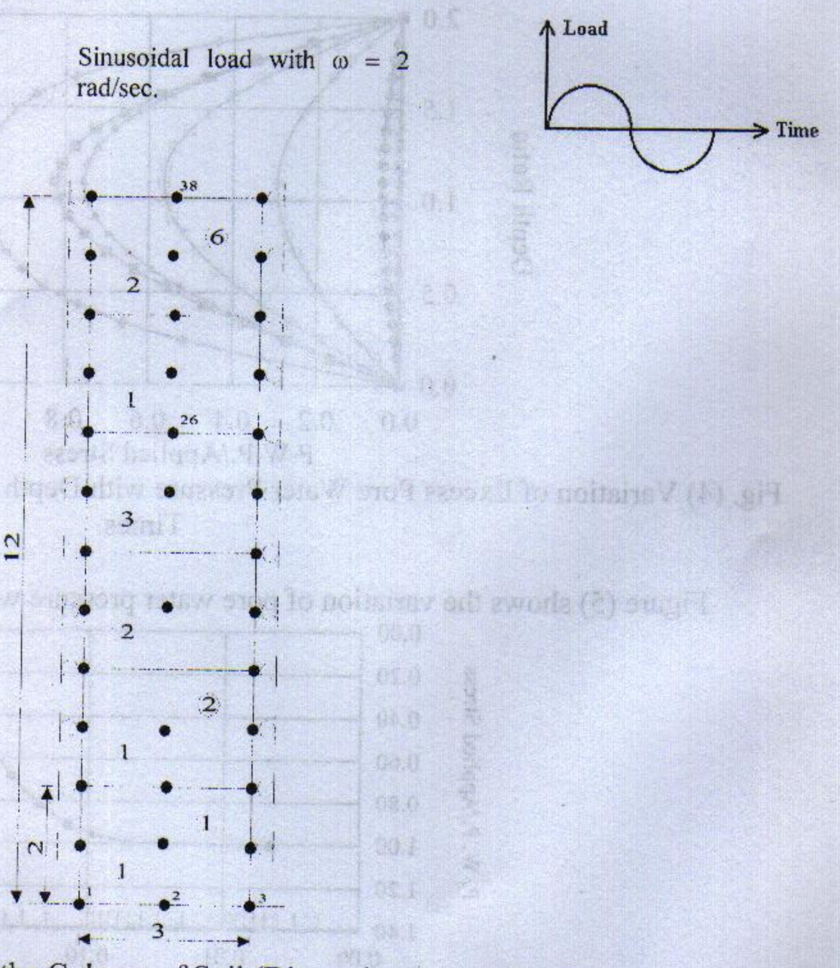

Fig. (6) The Mesh of the Column of Soil (Dimensions are in Meters).

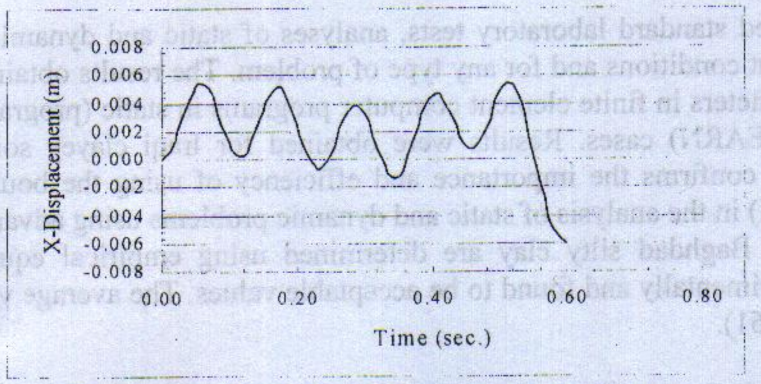

Fig. (7) x-Displacement Response at Node (26).

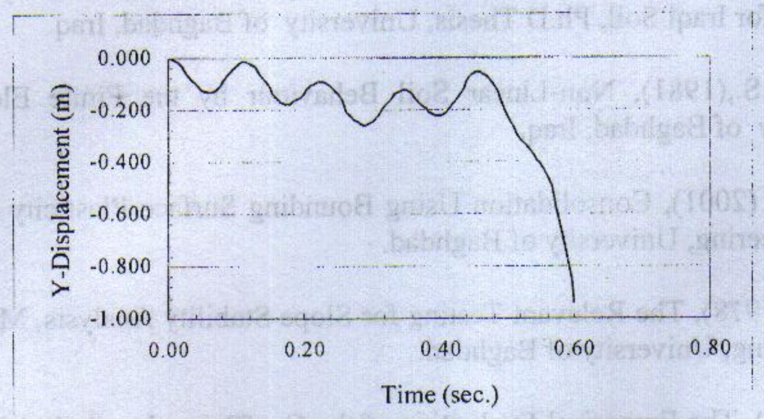

Fig. (8) y-Displacement Response at Node (26).

Evaluation of the Obtained Results

The results obtained in this work for static problems are similar in behaviour to those found by Al-Ebady (2001) whereas the results obtained for dynamic problems are similar in behaviour to those obtained by Al-Tae'e (2001). These results confirm the importance and the efficiency of using the bounding surface plasticity model (BSPM) in the analysis of static and dynamic problems using high quality laboratory tests. On the other hand, the results obtained here are for Iraqi clayey soils which are not available in literature before. This represents an addition of unavailable information in references.

CONCLUSIONS

Through the results of this work, several conclusions can be drawn:

1- The Baghdad soil is brown silty clay. The proposed abbreviation of it is (BBSC). It is of high plasticity (CH soil) and highly overconsolidated near the top surface.

2- From the ultrasonic test, results for the Iraqi clayey soil were obtained such as compression wave velocity (872m/sec), shear wave velocity (549m/sec), bulk modulus (1.24×10^6 kPa), shear modulus (343308 kPa), Young's modulus (9.27×10^5kPa) and Poisson's ratio (0.39) .

3. Using the laboratory test results, high quality input parameters for the bounding surface plasticity model were obtained using the computer program (EVAL). The values obtained of these parameters are as follows:

λ	κ	M_c	M_e/M_c	ν	R_c	R_e/R_c	A_c	A_e/A_c
0.064	0.017	1.20	0.676	0.4	2.70	0.85	0.05	0.80

4- From advanced standard laboratory tests, analyses of static and dynamic problems are allowed under different conditions and for any type of problem. The results obtained in (4) could be used as input parameters in finite element computer programs in static (program ACED) and dynamic (program DLEARN) cases. Results were obtained for Iraqi clayey soil from both programs. These results confirms the importance and efficiency of using the bounding surface plasticity model (BSPM) in the analysis of static and dynamic problems using advanced laboratory tests.

5- K_o values of Baghdad silty clay are determined using empirical equations .The values are checked experimentally and found to be acceptable values. The average value of K_o found in this research is (0.61).

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SYMBOLES

A_f	Skempton's pore pressure parameter at failure
ASTM	American Society of Testing Materials
BBC	Boston Blue Clay
C_c	Compression index
C_s	Swelling index.
E	Modulus of elasticity.
G	Shear modulus
K	Bulk modulus.
K_0	Coefficient of lateral earth pressure at rest
LID	Load Increment Duration
LIR	Load Increment Ratio
M	Slop of the critical state line in p-q space
OCR	Over Consolidation Ratio
P_0	Overburden pressure.

P_c	Preconsolidation pressure
ε_a	Axial strain.
ε_v	Volumetric strain.
ε_r	Lateral strain.
ϕ'_{TC}	Angle of internal friction from triaxial compression test.
ϕ'_{TE}	Angle of internal friction from triaxial extension test.
κ	Slop of swelling of the e-lnρ plot.
λ	Slope of consolidation line in the e-lnρ plot.
ν	Poisson's ratio.
ν_c	Compression wave velocity.
ν_s	Shear wave velocity.