ANALYSIS OF EMBANKMENT ON GYPSEOUS SOILS

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الخلاصة:

تنتشر التربة الجبسية بشكل واسع في بلدنا وفي بلدان اخرى. لذا لا بد من دراسة تصرف هذه الترب لما يمكن ان تسببه من مشاكل للمنشأت المؤسسة على هذه الترب وخاصة عند تعرضها للماء الذي يسبب انهيار التربة نتيجة لغسل الاملاح منها. هذا البحث اهتم بدراسة تأثير عملية غسل التربة على استقرارية السدة الترابية المقامة على اساس يتكون من تربة جبسية. طريقة العناصر المحددة استخدمت في هذا البحث من خلال استخدام التحليل اللاخطي ذات الطابع التزايدي على الاجهاد في برنامج حاسوب. المتغيرات المستخدمة في هذا التحليل حسبت باستخدام معلومات جمعت لفحوصات مختبرية (فحص القص الثلاثي المحاور) لبعض الباحثين.

نتائج تحليل السدة الترابية المقامة على اساس يتكون من تربة جبسية تعرضت للغسل تبين بان هنالك زيادة ملحوظة في الازاحات والتشوهات للسدة واساسها المقامة عليه. في الختام يأكد هذا البحث على امكانية نجاح استخدام طريقة العناصر المحددة في تصميم وتحليل المنشأت المهمة المقامة على ترب جبسية تعرضت لتأثير عملية الغسل حيث يمكننا ذلك من امكانية التكهن بتصرف المنشأ وبصورة فعالة لوضع الحلول الملائمة لأي مشاكل قد تحدث نتيجة وجود الترب الجبسية.

ABSTRACT:

The gypseous soils are distributed in many regions in Iraq and other countries. Therefore, it is necessary to study the behavior of such soils due to the large damages that affects the structures founded and constructed in or on it.

This research is concerned with studying the effect of leaching soil process on the stability of an embankment erected on foundation gypseous soil. The finite element method is adopted in this research. The analyses carried out using a nonlinear, increment, and stress-dependent finite element computer program. The hyperbolic stress-strain parameters used in the finite element analyses are estimated by the data collected from triaxial compression tests of some researchers. The analysis of the embankment problem carried out, shows that the leaching process for foundation

gypseous soil increases the displacements and deformations of the embankment and its foundation.

Finally, this research necessitate the success using of the finite element method in design and analyses of the important structures and buildings erected on gypseous soils that may expose to the effect of leaching process. This means that there is possibility to predicate the behavior of structure by a powerful means to establish the suitable solutions for any problems that may be occurred as a result of the present gypseous soil.

NOTATIONS:

a	Hyperbolic constant for stress-strain relationship.
b	Hyperbolic constant for stress-strain relationship.
C	Cohesion.
CD	Consolidated drained (triaxial test).
CU	Consolidated undrained (triaxial test)
d	Parameter expressing rate of change of υ_i with strain.
D	Tangent compliance tensor.
[D]	Material flexibility matrix.
$e_{(ij)}$	Deviator of strain tensor $\underline{\varepsilon}$.
E_i	Initial tangent modulus.
E_t	Tangential modulus.
E_{ur}	unloading-reloading modulus value.
f	Value of tangent Poisso's ratio at zero strain $=\upsilon_i$.
F	Rate of change of υ_I and σ_3.
G	Value of at one υ_i atmospheric pressure.
K	Modulus number.
[K]	Global stiffness matrix.
K_{ur}	Unloading-reloading modulus number.
K_o	Lateral earth pressure coefficient at rest.
n	Exponent determining rate of variation of E_i with σ_3.
R_f	Failure ratio.
(r)	The global nodal displacement vector.
(R)	The global nodal force vector
$S_{(ij)}$	Deviator of stress tensor $\underline{\sigma}$.
P_a	atmospheric pressure.
ε	Strain.
γ	Unit weight.
σ_1, σ_3	Major and minor principal stresses
υ	Poisson's ratio
ϕ	Angle of shear resistance (internal friction
σ_1-σ_3	Deviator stress
$(\sigma_1-\sigma_3)_f$	Deviator stress at failure
$(\sigma_1-\sigma_3)_{ult}$	Asymptotic value of deviator stress

1. INTRODUCTION

The terms "gypsifereous soil" and "gypseous soil" are used to specify the soil that contains gypsum, Agronomists use the first, while civil engineers use the second.

There is no unique definition for gypseous soils used by civil engineers. It can be stated that a soil is a gypseous soil when it has gypsum content enough to change or to affect its engineering properties.

Gypseous soils are distributed in many regions in the world including Iraq. They cover about 20 % of the total area of Iraq [3].

Many problems which are related to construction on gypseous soils were observed. There are three main sources of such problems, first, the dissolution and transport of gypsum through soil, which causes a continuous loss of soil mass and increasing voids. A large reduction in shear strength and an increase in compressibility are the main results behind this phenomenon. The second is the variation of shear strength and compressibility characteristics of gypseous soils upon wetting and saturation. The third is the volume change accompanying the dehydration of gypsum or hydration of anhydrite.

The finite element technique has proved to be a powerful engineering analysis tool, and versatile numerical method of considerable potential for simulating a real problem in the field and the laboratory; because it intrinsically permits the realistic molding of more aspects of problems than do alternative techniques.

The finite element method has become widely accepted by the engineering profession as an extremely valuable method of analysis. Its application has enabled satisfactory solutions to be obtained for many problems which had been regarded as insoluble [14] and the amount of research effort currently being devoted to the finite element method ensures a rapidly widening field of application.

The finite element method has developed simultaneously with the increasing use of large, high-speed computers, which have made these methods efficient and economical, and with the growing emphasis on numerical methods for engineering analyses. Now, however, due to availability of high-speed computers and powerful numerical analytical techniques such as the finite element method, it is possible to approximate nonlinear inelastic soil behavior in stress analyses. In order to perform nonlinear stress analyses of soils, however, it is necessary to be able to describe the stress-strain behavior of the soil in quantitative terms, and to develop techniques for incorporating this behavior in the analyses.

In this study, the finite element method used to analysis an embankment constructed on foundation gypseous soil.

2. CONSTITUTIVE MODELING

A constitutive model or law represents a mathematical model that describes the behavior of a material. In other words, a constitutive model simulates physical behavior that has been perceived mentally [7].

Constitutive models or laws of engineering materials play a significant role in providing reliable results from any solution procedure. Their importance has been enhanced significantly with the great increase in development and application of many modern computers based techniques such as the finite element, finite difference and boundary integral equation methods.

The simplest constitutive laws used in engineering are linear such as the Hooke's law. These laws are valid only for a very limited class of materials because most engineering systems are nonlinear and complex. The influence of the nonlinear response becomes more prominent in the case of materials that are influenced by factors such as state of stress, residual or initial stresses, volume change under shear stress history or stress paths, inherent and induced anisotropy, change in the physical state, and fluid in the pores. Different constitutive laws based on different concepts have been proposed. Each model can be valid within its own local realm, and that no valid universal constitutive model has yet been developed for all material under all conditions. In this study, the hyperbolic stress–strain relationships are used to describe the behavior of natural and leached gypseous soils.

3. HYPERBOLIC STRESS-STRAIN MODEL

It is the most widely used for soil behavior representation. The model was proposed by **Kondner (1963)[11]**, and developed by **Duncan and Chang (1970)[9]**, in an attempt to provide a simple framework encompassing the most important characteristics of soil stress-strain behavior, using the data available from conventional laboratory tests such as UU-triaxial compression test or CU-triaxial compression test. The hyperbolic stress-strain relationships were developed for use in incremental finite element analyses. In each increment of such analyses, the stress-strain behavior of the soil is treated as being linear, and the relationship between stress and strain is assumed to be governed by the generalized Hooke's law of elastic deformations, which may be expressed as follows for conditions of plane strain [15]:

$$\begin{Bmatrix} \Delta\sigma_x \\ \Delta\sigma_y \\ \Delta\tau_{xy} \end{Bmatrix} = \frac{E_t}{(1+\nu_t)(1-\nu_t)} \begin{bmatrix} (1-\nu_t) & \nu_t & 0 \\ \nu_t & (1-\nu_t) & 0 \\ 0 & 0 & (1-2\nu_t)/2 \end{bmatrix} \begin{Bmatrix} \Delta\varepsilon_x \\ \Delta\varepsilon_y \\ \Delta\gamma_{xy} \end{Bmatrix} \quad \text{....(1)}$$

where:

$\Delta\sigma_x$, $\Delta\sigma_y$ and $\Delta\tau_{xy}$ = are the increments of stress during a step of analysis.

$\Delta\varepsilon_x$, $\Delta\varepsilon_y$ and $\Delta\gamma_{xy}$ = are the corresponding increments of strain.

E_t = is the tangent value of Young's modulus.

υ_t = is the tangent value of Poisson's ratio.

The value of both E_t and υ_t in each element change during each increment of loading in accordance with the calculated stresses in that element, in order to account for three important characteristics of the stress-strain behavior of soil, namely non linearity, stress-dependency, and inelasticity.

Kondner (1963)[11,] has shown that the stress-strain curves for a number of soils, both clay and sand, could be approximated reasonably accurate by hyperboles like the one shown in figure (1). This hyperbola can be represented by an equation of the form:

$$(\sigma_1 - \sigma_3) = \frac{\varepsilon}{a + b\varepsilon} \quad \text{.............(2)}$$

where: σ_1 and σ_3 : are the major and minor principal stresses.

ε : is the axial strain.

a and b : are constant related to initial tangent modulus, E_i, and the asymptotic stress, $(\sigma_1 - \sigma_3)_{ult}$, respectively.

$$a = \frac{1}{E_i} \quad \text{.............(3)}$$

$$b = \frac{1}{(\sigma_1 - \sigma_3)_{ult}} \quad \text{.............(4)}$$

Kondner (1963)[11], has shown that the value of the coefficients a and b may be determined most readily if the stress-strain data plotted on transform axes, as shown in figure (2) when equation (2) is rewritten in the following form:

$$\frac{\varepsilon}{(\sigma_1 - \sigma_3)} = a + b\varepsilon \qquad \text{.............(5)}$$

It may be noted that a and b are respectively, the intercept and the slope of the best-fit resulting straight line. The asymptotic stress value, $(\sigma_1 - \sigma_3)_f$, by means of a factor R_f as follows:

$$(\sigma_1 - \sigma_3)_f = R_f (\sigma_1 - \sigma_3)_{ult} \qquad \text{.............(6)}$$

where:

R_f : is the failure ratio, which always has a value less than unity. For a number of different soils, the value of R_f has been found to be between 0.5 and 0.9 [15].

By expressing the parameters a and b in terms of the initial tangent modulus value and the compressive strength, equation (2) may be rewritten as:

$$(\sigma_1 - \sigma_3) = \frac{\varepsilon}{\left[\dfrac{1}{E_i} + \dfrac{\varepsilon R_f}{(\sigma_1 - \sigma_3)_f}\right]} \qquad \text{.............(7)}$$

On the other hand initial tangent modulus E_i is related to the confining stress as follows:

$$E_i = K P_a \left(\frac{\sigma_3}{P_a}\right)^n \qquad \text{.............(8)}$$

where:

E_i = initial tangent modulus.

σ_3 = minor principal stress.

P_a = atmospheric pressure, having same units as σ_3.

K = modulus number.

n = exponent determining rate of variation of (E_i) and (σ_3).

(K) and (n) are to be determined experimentally [9] by plotting E_i - logσ_3 curve for several triaxial test, figure (3).

If it is assumed that failure will occur with σ_3 constant and considering Mohr-Coulomb failure criterion, then:

$$(\sigma_1 - \sigma_3)_f = \frac{2c\cos\phi + 2\sigma_3 \sin\phi}{1-\sin\phi} \qquad \text{.............(9)}$$

Where C and ϕ are Mohr-Coulomb strength parameters. Combining equations (8) and (9) with equation (7), provides a mean of relating stress to strain and confining pressure by means of five parameters K, n, c, ϕ, R_f and as follows:

$$(\sigma_1 - \sigma_3) = \frac{\varepsilon}{\dfrac{1}{KP_a\left(\dfrac{\sigma_3}{P_a}\right)^n} + \dfrac{\varepsilon R_f(1-\sin\phi)}{2c\cos\phi + 2\sigma_3\sin\phi}} \qquad \text{.....(10)}$$

To afford for incremental stress analysis, tangential modulus (E_t) is needed and can be expressed as:

$$E_t = \frac{d(\sigma_1 - \sigma_3)}{d\varepsilon} \qquad \text{.............(11)}$$

and if considering equation (7), then

$$E_t = \frac{\dfrac{1}{E_i}}{\left[\dfrac{1}{E_i} + \dfrac{R_f\varepsilon}{(\sigma_1-\sigma_3)_f}\right]^2} \qquad \text{..............(12)}$$

It is useful to eliminate (ε) so as to be able to express a non (0, 0) stress-strain initial condition, and through suitable substitutions where:

$$\varepsilon = \frac{(\sigma_1 - \sigma_3)}{E_i\left[1 - \dfrac{R_f(\sigma_1-\sigma_3)}{(\sigma_1-\sigma_3)_f}\right]} \qquad \text{.............(13)}$$

is obtained from rewriting equation (7) then substituting equation (13) in equation (12) yields:

$$E_t = \left[1 - \frac{R_f(\sigma_1-\sigma_3)}{(\sigma_1-\sigma_3)_f}\right]^2 E_i \qquad \text{.............(14)}$$

Substituting equations (8) and (9) in equation (14), then E_i can be expressed as follows:

$$E_t = \left[1 - \frac{R_f(1-\sin\phi)(\sigma_1 - \sigma)}{2c\cos\phi + 2\sigma_3\sin\phi}\right] K P_a \left(\frac{\sigma_3}{P_a}\right)^n \qquad \text{.............(15)}$$

This expression may be employed very conveniently for any arbitrary initial stress conditions, since it is related directly to any stress level. Equation (15) can be employed in total or effective stress analyses depending on parameters obtained from laboratory tests.

In the case where three-dimensional stresses and strains are involved, it may be desirable to include the effect of intermediate principal stress (σ_2) in the failure criterion or in the stress-strain relationship of the soil [9]. In the above mentioned equations the assumption that $\sigma_2 = \sigma_3$ is considered which is simulating triaxial test conditions.

Stress-strain behavior of soil on unloading-reloading can be approximated with a high degree of accuracy as being linear and elastic [10]. This linear behavior is suggested to be independent of the value of the deviator stress. The unloading–reloading modulus value was found to depend only on the confining pressure (σ_3) as shown in figure (3).

$$E_{ur} = K_{ur} P_a \left(\frac{\sigma_3}{P_a}\right)^n \qquad \text{.............(16)}$$

where:

E_{ur} : unloading-reloading modulus value.

K_{ur} : the corresponding modulus number.

n : exponential determining rate of variation of (E_{ur}) with (σ_3) and can be taken as the same value as for primary loading [10].

4. THE FINITE ELEMENT COMPUTER PROGRAM USED

A computer program formulated by [2] in FORTRAN language was used in the finite element analysis carried out during this research. The program allows for four different types of elements to be used in the finite element mesh in solving soil, structure, or soil-structure interaction problems under plane or axisymmetrical conditions. The type of element considered in this research was the two dimensional quadrilateral element. The behavior of the soil and the interface can be approximated by several models [8]. The model which is considered in this work, is the hyperbolic, incremental, stress-dependent nonlinear technique [9]. The modulus is stress dependent and considered loading path whether loading or unloading.

Simulation of construction sequence could be achieved using incremental solution technique [8]. The nonlinear analysis technique was based on the mixed procedure in the evaluation of stresses and strain, where several iterations could be performed for any increment of loading.

The sign convention for stresses, numbering of element nodes and stress-strain relationship are shown in figure (4).

The program presents the results of analysis as displacements of nodal points and the value and direction of stresses developed at the centroid of each element at the end of each solution increment. Auxiliary programs to draw the finite element mesh of the problem before and after load application are also provided.

5. Estimation of Hyperbolic Stress-Strain Parameters

To estimate hyperbolic stress-strain parameters required for nonlinear finite element analysis, the data collected is grouped into triaxial compression tests of foundation natural and leached gypseous soils carried out by [1].

From this data, the parameters (C, ϕ, K, n, R_f, K_{ur}), which are required by Duncan-Chang model, 1970 can be obtained to analyze the behavior of the selected embankment by finite element method.

The hyperbolic model parameters for natural and leached gypseous soils which used in this study is shown in table (1)

6. Finite Element Analysis of Embankment

6.1 Problem geometry

The construction of an embankment (8m) height and (1:1.5) slope, to be made into a stratum, 16m thick, of natural and leached gypseous soils used in this study.

The finite element mesh used in the analysis is shown in figure (5) consists of 138 nodes and 116 two-dimensional quadrilateral elements. The mesh extended horizontally away from the toe of the embankment to twice the width of the embankment. The mesh also extends downwards to twice the embankment height. This is to assure coverage of all the foundation zones that are appreciably affected by the application of the embankment weight during and after construction [5,8].

The nodal points along the bottom boundary of the mesh are assumed to be fixed both horizontally and vertically. The nodes on the right and left ends of the mesh are fixed in the horizontal direction while they are free to move in the vertical direction. All interior nodes are free to move horizontally and vertically.

The discretization selected allowed processing the foundation to be of up to six layered strata of different properties.

6.2 Material characterization

In the cases analyzed, the profile consisted of two main zones of different material:

a. The embankment zone.
b. The foundation zone.

The behavior of soil material is considered to be nonlinear stress dependent. Stress-strain relationship for the tangent modulus is in accordance with Duncan-Chang model. The Mohr-Coulomb failure criterion is used as the indicator for element failure.

The coefficient of lateral strain, Poisson's ratio, is also considered to be nonlinear stress-dependent for the evaluation of the tangent Poisson's ratio of the soil [12].

The embankment mass is composed of gypseous soil compacted at optimum moisture content. The parameters representing the nonlinear behavior of this soil shown in table (2) for data obtained from triaxial test carried out by ***(Al-Kaisi, 1997)***[2]. The foundation material for natural and leached gypseous soils in table (2) also shows the nonlinear soil parameters for the foundation and simulated according to the data obtained from triaxial tests carried out by ***(Al-Busoda,1999)***[1].

6.3 Embankment simulation

The embankment construction in the analysis is simulated by four lifts. The initial stresses within the soil media are calculated on the basis of ($\sigma_v = \gamma h$) and ($\sigma_h = k_o \gamma h$).

The coefficient of lateral earth pressure at rest, k_o is initially evaluated using the equation:

$$k_o = \frac{\nu}{1-\nu} \qquad \text{..............(17)}$$

6.4 Results of analysis

Two conditions are analyzed during this research, the first condition concerned with an embankment resting on natural gypseous soil and the second condition concerned with an embankment resting on leached gypseous soil.

The values of the model parameters tabulated in table (2) were adopted in these conditions.

Figure (6) shows the shear stress contours at the end of embankment construction. It can be seen that the magnitude of maximum shear stress increase but its location does not change when the embankment is resting on leached gypseous soil.

The displacement vectors are markedly increase after leaching process compared to natural gypseous soil as in figure (7). The maximum displacements concentrate in the centerline under the effect of embankment load. Also, it can be noticed that the displacements decrease gradually when getting a way from the toe of embankment.

From figure (8) which represents the deformation of embankment and foundation soil at the end construction for natural and leached gypseous soil, it can be observed that the deformation increase after leaching process but no clear heave was observed in the ground surface in front of the embankment toe.

The vertical displacement at node number 134 of figure (5) of embankment variation with the construction stages for two conditions is shown in figure (9). From this figure, it can be seen that vertical displacements largely increase after leaching process. This behavior may be attributed to removal of the cementing material from the natural soil leading to a large decrease in the shear strength causing high settlement with embankment construction stages.

A summery, the shear stresses, vertical and horizontal displacements largely increase while the factor of safety reduces when the embankment constructed on foundation consist of leached gypseous soil.

7. CONCLUSIONS

The finite element analyses of the embankment problem selected under foundation of natural and leached gypseous soils lead to the following conclusions:

1. The leaching process of the gypseous soil foundation resulted in a radical drop in stability of this embankment which leading to complete failure.
2. The leaching process for gypseous soil foundation largely increases the displacements and deformations of the embankment and its foundation.
3. Finally, this research necessitate the success using of the finite element method in design and analyses of the important structures and buildings erected on gypseous soils expose to the effect of leaching soil process. This means that there is possibility to predicate the behavior of structure by a powerful means to establish the suitable solutions for any problems that may be occurred as a result of the present gypseous soil.

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Table (1): Hyperbolic model parameters for various gypseous soils[13].

Site Location	Reference	Leaching Condition	Test Type	Hyperbolic model parameters				
				C , kPa	ϕ °	K	n	R_f
Tikrit	Al-Busoda, 1999	Natural	CD	46	34.4	120	0.7	0.76
=		Leached	CD	1.28	34.5	60	1.2	0.72

Table (2): Material characteristics used in the finite element analysis.

Parameters	Foundation Soil		Embankment Material
	Natural Soil	Leached Soil	
Unite weight, γ, kN/m^3	15.3	16.2	20.4
Cohesion, c, kN/m^2	46	1.28	81
Angle of internal friction, ϕ, degrees	34.4	34.5	34
Nonlinear modulus			
K	120	60	717
n	0.7	1.2	0.28
R_f	0.76	0.72	0.95
K_{ur}	-	-	1812
d(ε_1 in %)	0.05	0.018	0.04
F	0.05	0.23	0.37
G	0.091	0.43	0.45

Table (3): Effect of leaching on embankment and its foundation.

Case	Min. F. O. S.	Max. Vertical Settlement, (cm)	Max. Horizontal Displacement, (mm)	Max. shear stress, kPa
Natural Soil (CASE-I)	2.0	48	161	38
Leached Soil (CASE-II)	1.37	82.7	599	40.2

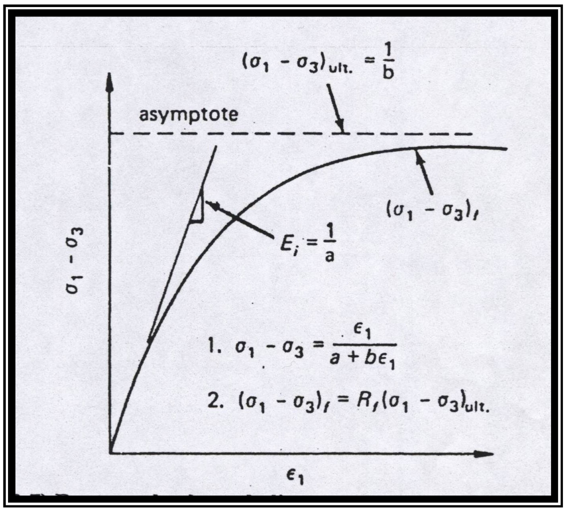


Figure (1): Hyperbolic stress-strain Curve[11].

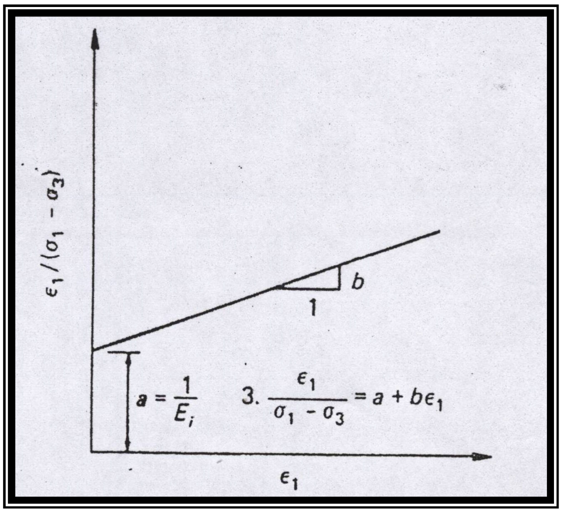


Figure (2): Transformed hyperbolic stress-strain curve [11].

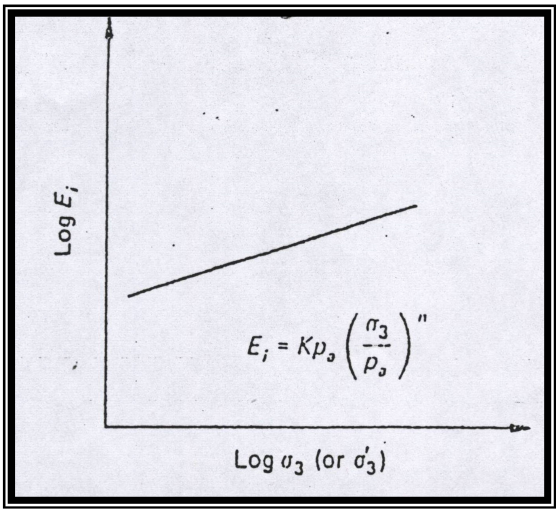


Figure (3): Relation between initial modulus and confining stress for computation of tangent modulus [6].

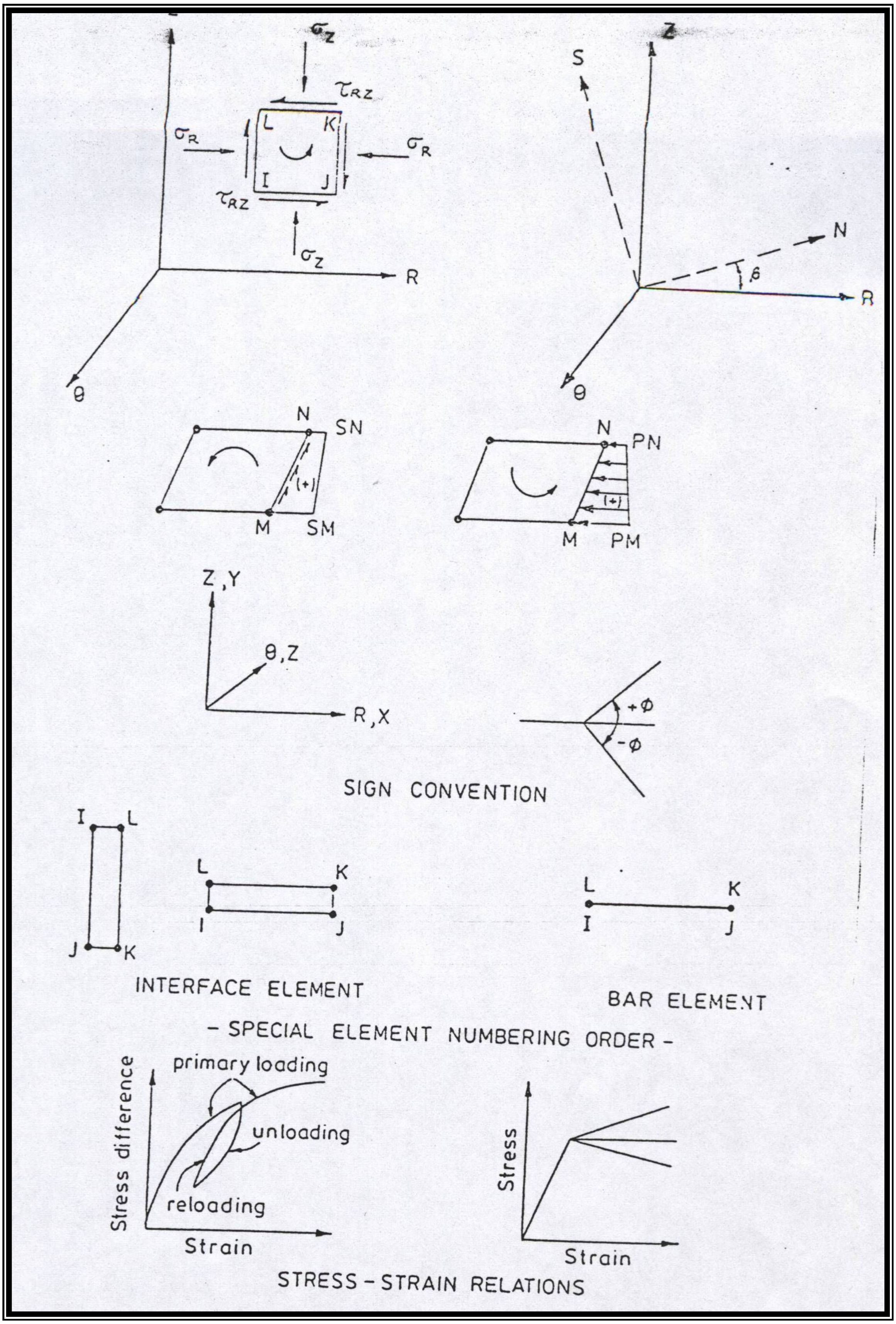


Figure (4): Sign convention, element numbering, and stress-strain relationship adopted in the finite element program [4].

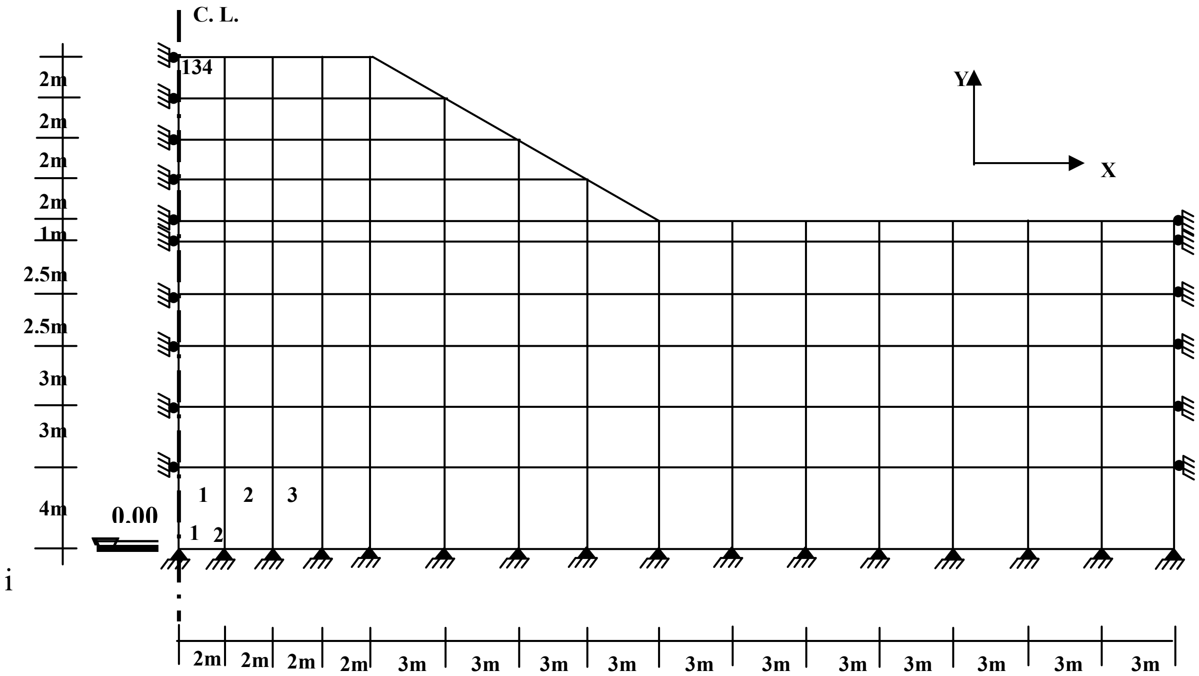
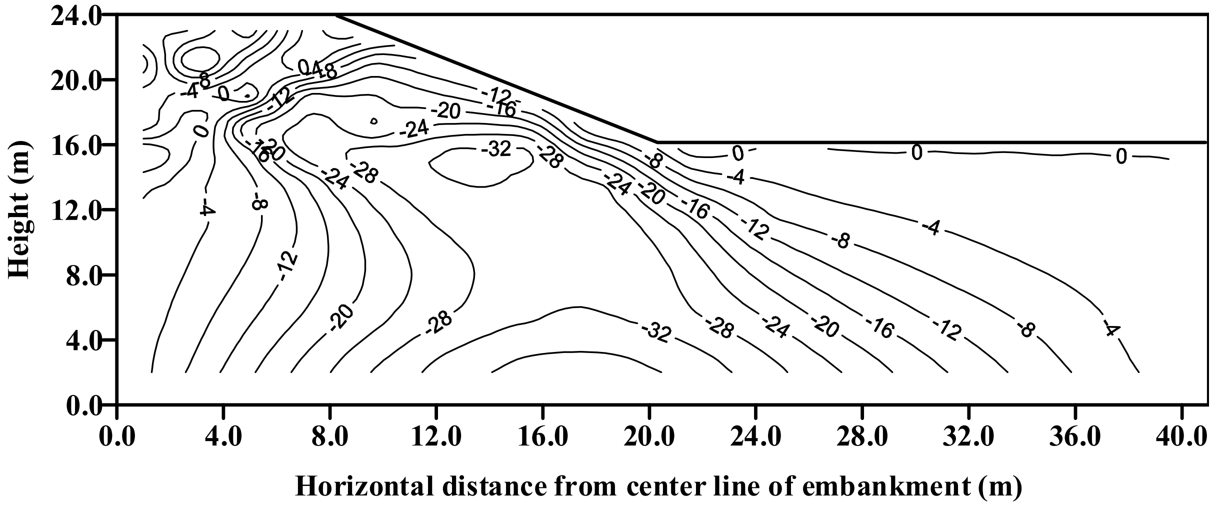
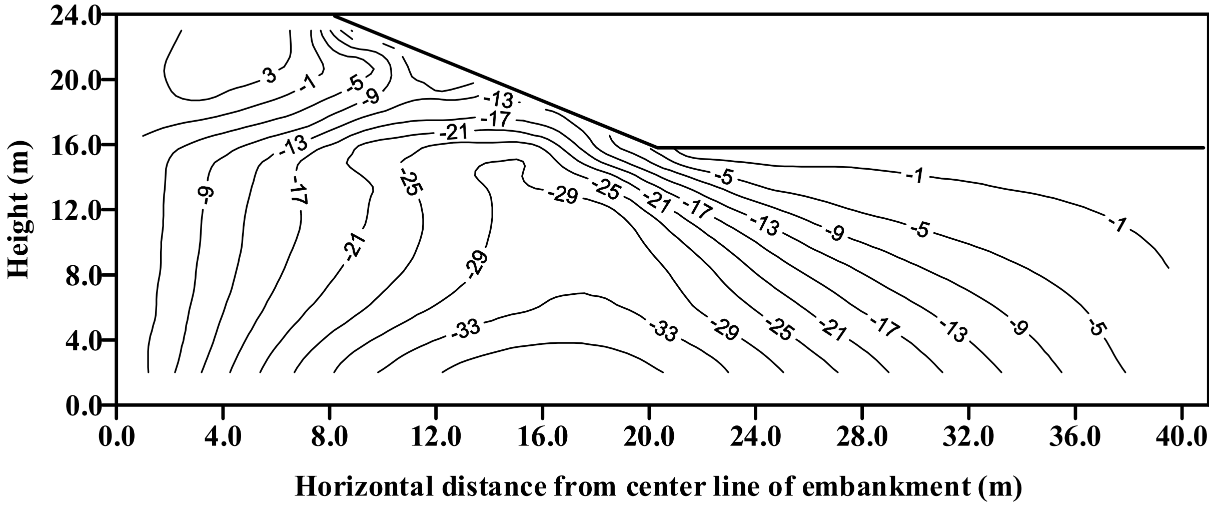


Figure (5): The finite element mesh used in the analysis.

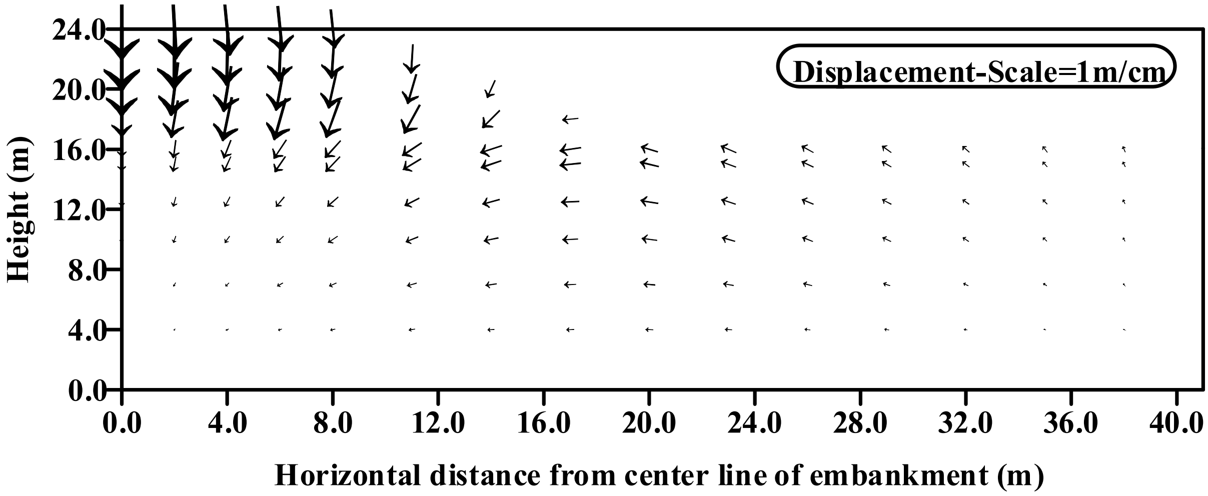


A- Natural Soil (CASE-I)

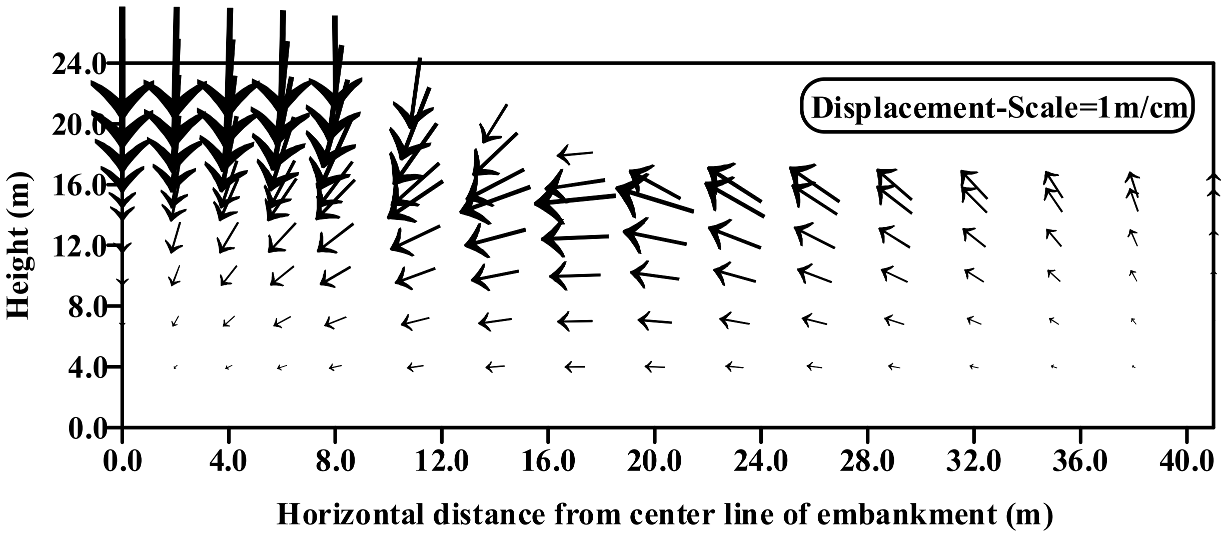


B- Leached Soil (CASE-II)

Figure (6): Contour of shear stresses at the end of embankment construction.



A- Natural Soil (CASE-I)



B- Leached Soil (CASE-II)

Figure (7): Displacement vectors at the end of embankment construction.

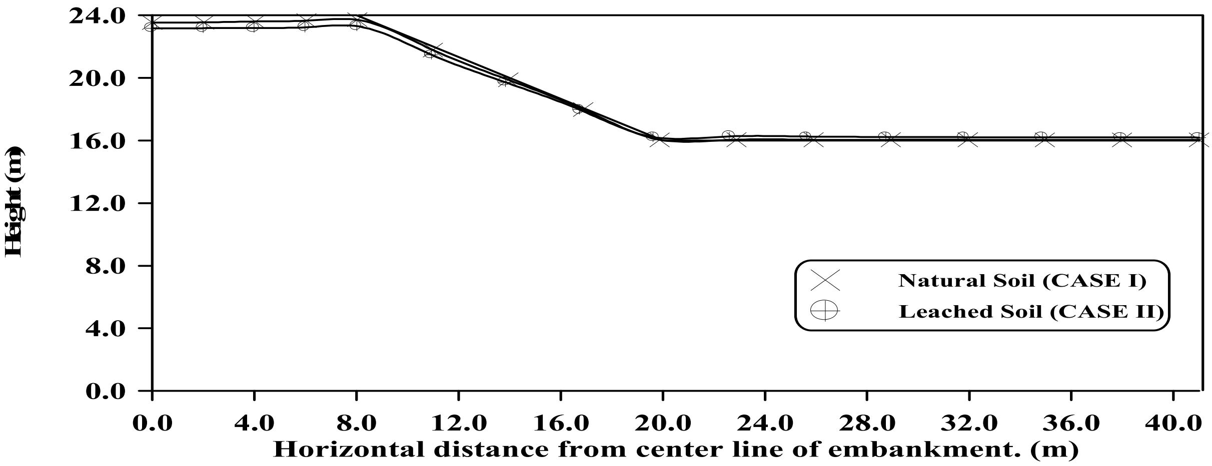


Figure (8): Deformation of embankment and foundation soil at the end of construction.

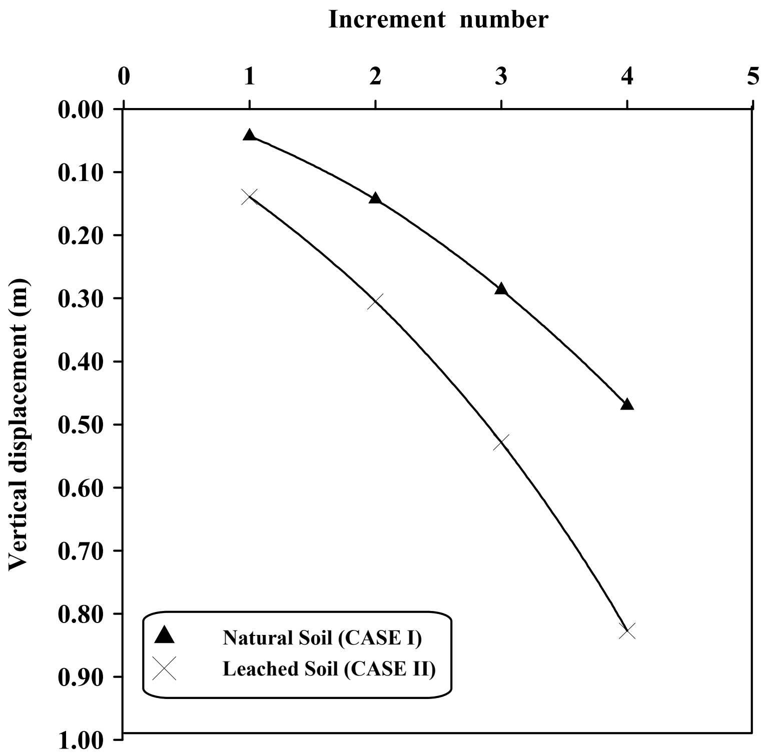


Figure (9): Vertical displacement at node number 134 of fig. (5) of embankment variation with increment number.