Performance of Test Embankment with Geogrid and Prefabricated Vertical Drains

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ABSTRACT : The numerical modelling of an embankment, reinforced at its base with a geogrid and installed vertical drains at its foundation subsoil, has been carried out using the computer programme CRISP. The objective of this paper is to illustrate how simple concepts can be used with certain accuracy in numerical modelling of an embankment with geogrids and vertical drains. Geogrids are modelled without incorporating the stiffness component of bar element in the formation of global stiffness matrix. The actual stress-strain and direct shear behaviour of the geogrid is utilised externally, considering equilibrium and compatibility conditions. This procedure model the actual behaviour of the geogrid instead of the usage of approximated constitutive relations in FEM analysis. Two different models of vertical drains, based on consolidation theories, were studied in this analysis to incorporate 3-D axisymmetric behaviour of drains into 2-D behaviour in order to perform plane strain embankment analysis.

1 INTRODUCTION

The practice of inserting reinforcement at the base of embankments constructed on soft foundations has become increasingly common in recent years, since it is generally agreed that reinforcement can improve short term stability and reduce lateral movements (Rowe, 1984). In this paper, a simple geogrid model using strain compatibility and equilibrium conditions, is used to incorporate geogrid behaviour, to analyse reinforced embankments using the finite element method.

To perform finite element analysis with vertical drains, the model used by Kumamoto et al (1988) and Cheung et al (1991) is used in this analysis. This model provides a procedure to convert axisymmetric vertical drain behaviour to plane strain analysis by an equivalent vertical parallel drainage walls.

2 THE TEST EMBANKMENTS AT MUAR FLATS

A total of thirteen full scale field test embankments, nine 6m high and four 3m high, were authorized by the Malaysian Highway Authority (MHA) for construction at a section of the express highway which is located at Muar Flat in the valley of the Muar River by Malaysian Highway Authority (MHA).

Intensive subsurface exploration program was carried out to determine subsurface conditions. The soil testing program was carried out in three phases by the Malaysian Highway Authority (MHA) and the Asian Institute of Technology (AIT), Thailand. The soil profile at the location of trial is presented in Fig. 1. An embankment with vertical drains and geogrids, built in Muar Flats, Malaysia, is analysed in this study.

Depth (mRL)	Layer	Description	Dominant minerals determined by X-ray diffraction	Grain size (%) Clay	Silt	Sand	Gravel	Coefficient of horizontal permeability, k_h (m/sec)	Compression ratio $\frac{C_c}{1+e_o}$	Preconsolidation pressure, P_c (kPa)
+2.5 to −0.5	CRUST	Yellowish brown mortled red CLAY with roots, root holes and laterite concretions	—	62	35	3	0	—	0.3	110
−0.5 to −5.6	UPPER CLAY	Light greenish grey CLAY with a few shells, very thin discontinuous sand partings, occasional near vertical roots and some decaying organic matter (less than 2 %)	Koolinite Montmor Illite Quartz	45	52	3	0	4×10^{-9}	0.5	40
−5.6 to −15.3 / −15.9	LOWER CLAY PEAT	Grey CLAY with some snells, very thin discontinuous sand partings and some decaying organic matter (less than 2 %) Dark brown PEAT with no smell (carbon dated to 10,000 years BP)	Koolinite Montmor Illite Quartz	50	47	3	0	10^{-9}	0.3	50
−15.9 to −19.9	SANDY CLAY	Greyish brown sandy CLAY with a little decaying organic matter	—	20	36	44	0	2×10^{-7}	0.1	60
below −19.9	SAND	Dark grey very silty medium to coarse SAND (SPT greater than 20)	—	4	20	71	5	—	–	–

Fig. 1.- Typical Subsurface Profile at the Site

3 FEM ANALYSIS AND CRISP COMPUTER PROGRAM

CRISP (Gunn and Britto, 1987), a Critical State finite element programme developed at Cambridge University, Cambridge, England, has been used extensively to analyze geotechnical problems. The CRISP program allows undrained, drained or fully coupled (Biot) consolidation analysis. Different types of triangular and quadrilateral elements are available in CRISP (last modification in 1987).

3.1 Soil Parameters

Soil Parameters for the Malaysian Muar clay have been obtained from field and laboratory tests (MHA, 1988; Balasubramaniam, 1988 and Balachandran, 1990). The input soil parameters used to perform analysis using CRISP are shown in Table 1.

Table 1- Soil Parameters Used in CRISP Program.

(a) Embankment Foundation (Modified Cam-clay Model) - Undrained Analysis.

Depth (m)	κ	λ	e_{cs}	M	ν'	K_w	γ_s
0.0 - 2.0	0.05	0.13	3.06	1.19	0.3	4.440E4	16.5
2.5 - 8.0	0.05	0.13	3.06	1.19	0.3	1.115E4	15.5
8.0 - 18.0	0.08	0.11	1.61	1.07	0.3	2.270E5	15.5

(b) Embankment Foundation (Modified Cam-clay Model) - Consolidation Analysis

Depth (m)	κ	λ	e_{cs}	M	ν'	K_w	K_h (m/s)	K_v (m/s)
0.0 - 2.0	0.05	0.13	3.06	1.19	0.3	16.5	1.5E-9	0.8E-9
2.5 - 8.0	0.05	0.13	3.06	1.19	0.3	15.5	1.5E-9	0.8E-9
8.0 - 18.0	0.08	0.11	1.61	1.07	0.3	15.5	1.1E-9	0.6E-9

4 MODELLING GEOGRID

Rowe (1983) presented a methodology to incorporate geogrid action in embankment analysis without introducing special interface element in finite element formulation. In this approach the soil and geogrid were examined separately and were related by conditions of compatibility and equilibrium. At the interface, for compatibilty condition the shear stresses are the same. When the shear stress reaches its limit value further stresses will be constant and may cause slippage.

The behaviour of a geotextile in direct shear apperatus can be closely modeled in the analysis of reinforced embankments. In this analysis an attempt was made to model the reinforcement action in the foundation of an embankment by using the stress-strain characteristics and the direct shear behaviour of the geotextile as constitutive relations of bar element and the interface element, respectively. Hence, the use of approximated constitutive relations to model bar elements and interface elements, in FEM analysis, is avoided. This model is graphically depicted as shown in Fig. 2.

Embankment loading is applied in several load increments. After the application of the first increment, the analysis was temporarily stopped and the displacement of corresponding finite element node which represents the geogrid are obtained as shown in Fig. 2. Then the strain developed inbetween two surface nodes, on application of the first load increment, is calculated as given in (1).

$$\varepsilon_{ij} = \frac{(X_i - X_j)}{L_{ij}} x100 \qquad (1)$$

where, L_{ij} is original length of the element side and ε_{ij} is the strain developed in the element side between nodes i and j.

Then by assuming compatibility conditions, the stresses developed in the geogrid was obtained separately from the stress - strain relationship and the direct shear behaviour of the geogrid. The tensile forces developed in the geogrid was calculated as given in (2).

$$T_{ij} = \sigma_{ij} x a_i \qquad (2)$$

where a_i is the cross sectional area of the geogrid. The interface shearing behavior was modelled using the shear behavior of the geogrid. The nodal boundary forces are then calculated by adding the tensile force and the shear force as given in (3).

$$F_{\xi} = \sigma_{ij}(x) x a_i + \tau_{ij}(x) x A_i \qquad (3)$$

where A_i is the surface area of geogrid in between two nodes per metre width.

The nodal forces calculated after the application of the first load increment is then applied as the initial boundary conditions in the analysis for second load increment. This procedure is repeated until the final load increment. The accuracy of this method of analysis will increase when the number of the load increment is more. The nonlinearity in

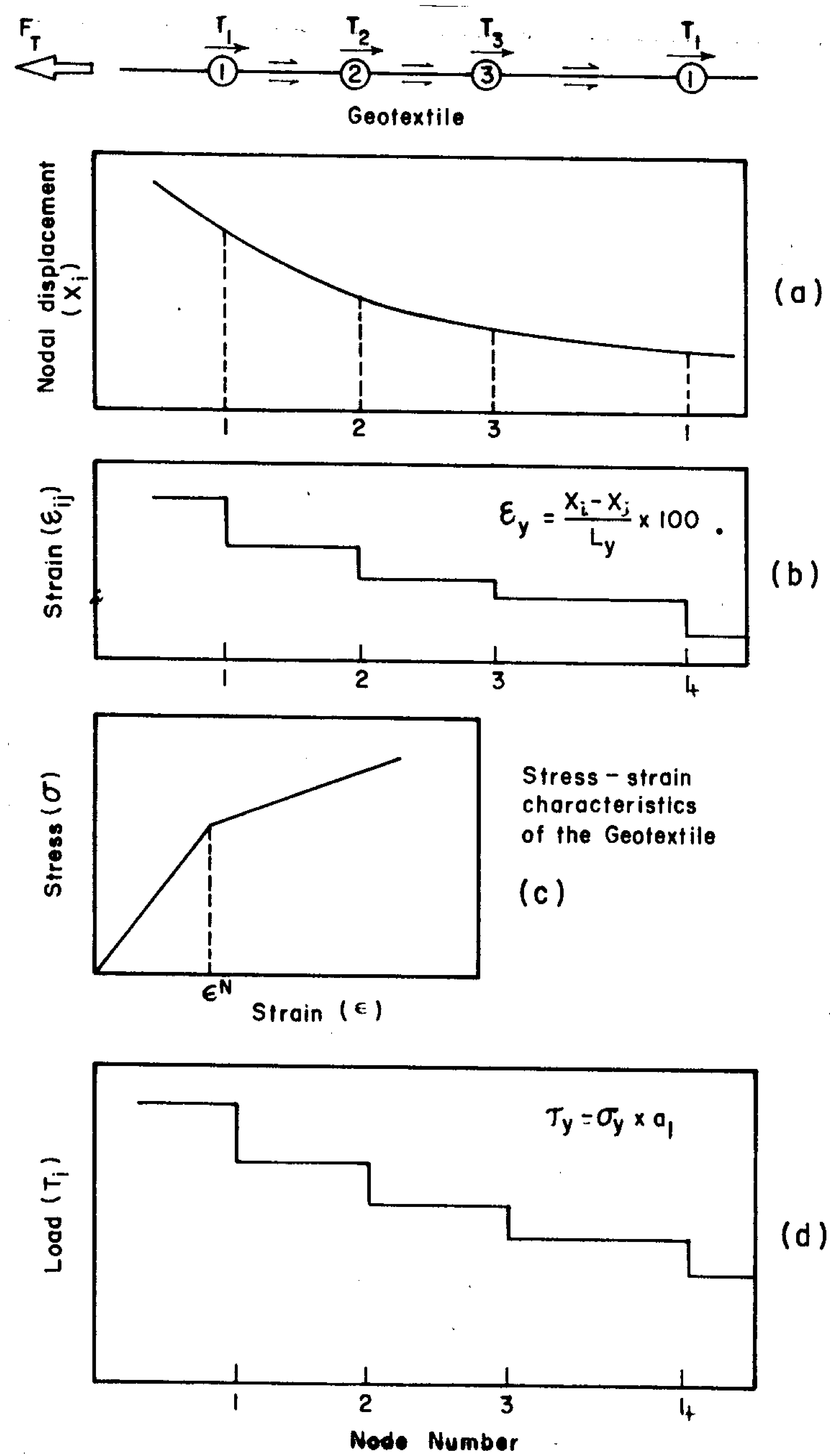

Fig. 2.- Geogrid Modelling Procedure Used in the Analysis

constitutive relations can be accurately modelled. Slippage that occur in higher loads inbetween geogrid and the interface is not considered in the analysis.

5 MODELLING OF VERTICAL DRAINS

In practice, embankment analysis is performed assuming plane strain conditions. When band drains are used to accelerate consolidation behavior, the actual three dimensional axisymmetric behavior of band drains should be transformed to an equivalent two dimensional condition to perform plane strain analysis.

A vertical drain system can be approximately converted into a parallel drainage wall system (Fig. 3) based on proper analytical consolidation theories by adjusting the space of drain walls and the horizontal coefficient of permeability of soil within the improved zone. A method proposed by Cheung et al (1982) was used in this study. There were two cases tried in this analysis as proposed by Cheung et al., (1991).

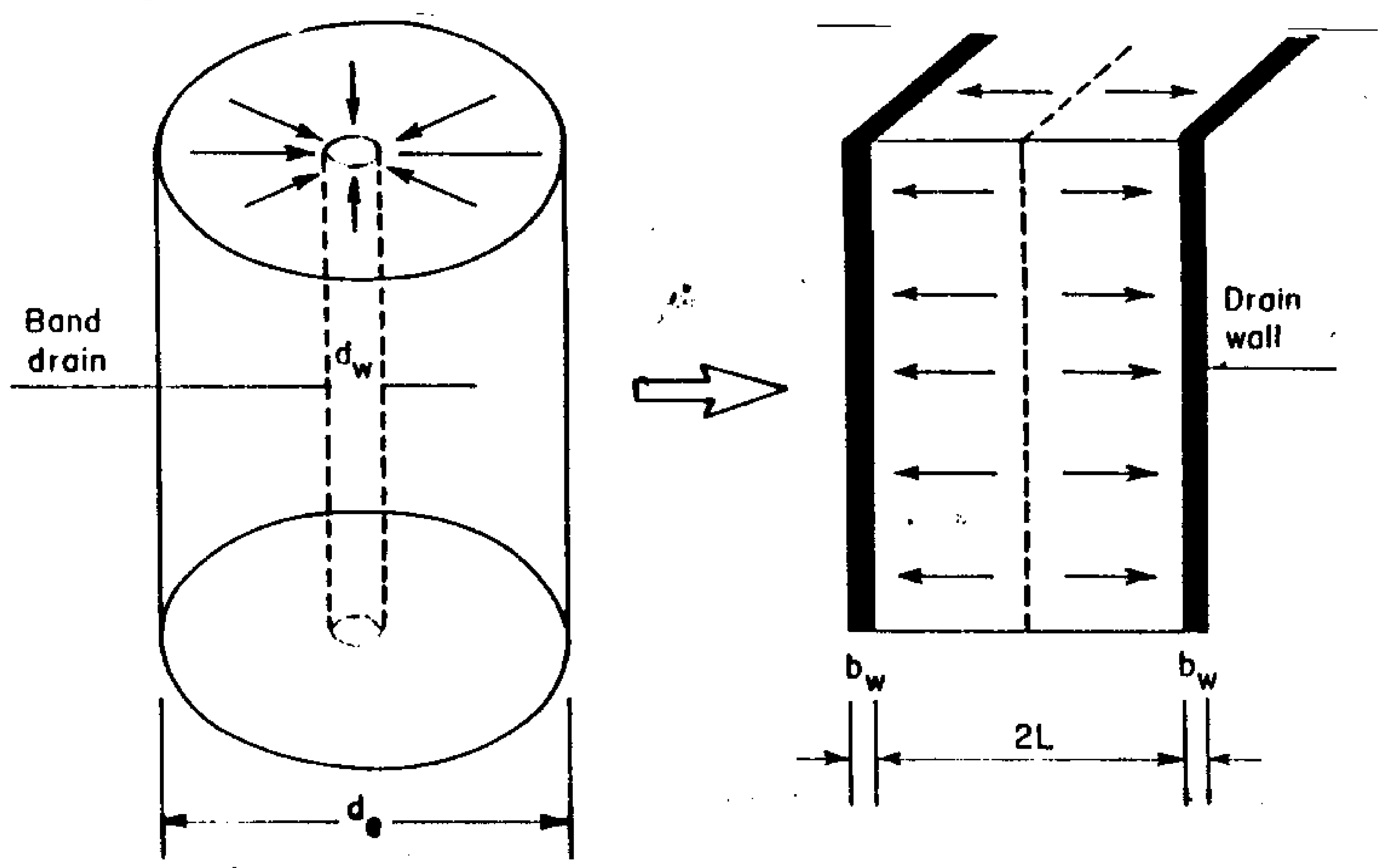

Fig. 3.- Vertical Drains Conversion Model

6 METHOD OF ANALYSIS

The analysis of the embankment treated with vertical drains and geogrid was performed in three stages, namely: (1) without any improvements, (2) with vertical drains only, and (3) with vertical drains and geogrids.

The embankment construction was simulated as vertical loads on the surface. The embankment loading pattern, observed in the field, was idealized as shown in Figs. 4 & 5 to simulate for the FEM analysis. The loading has been done effectivelv in two stages.

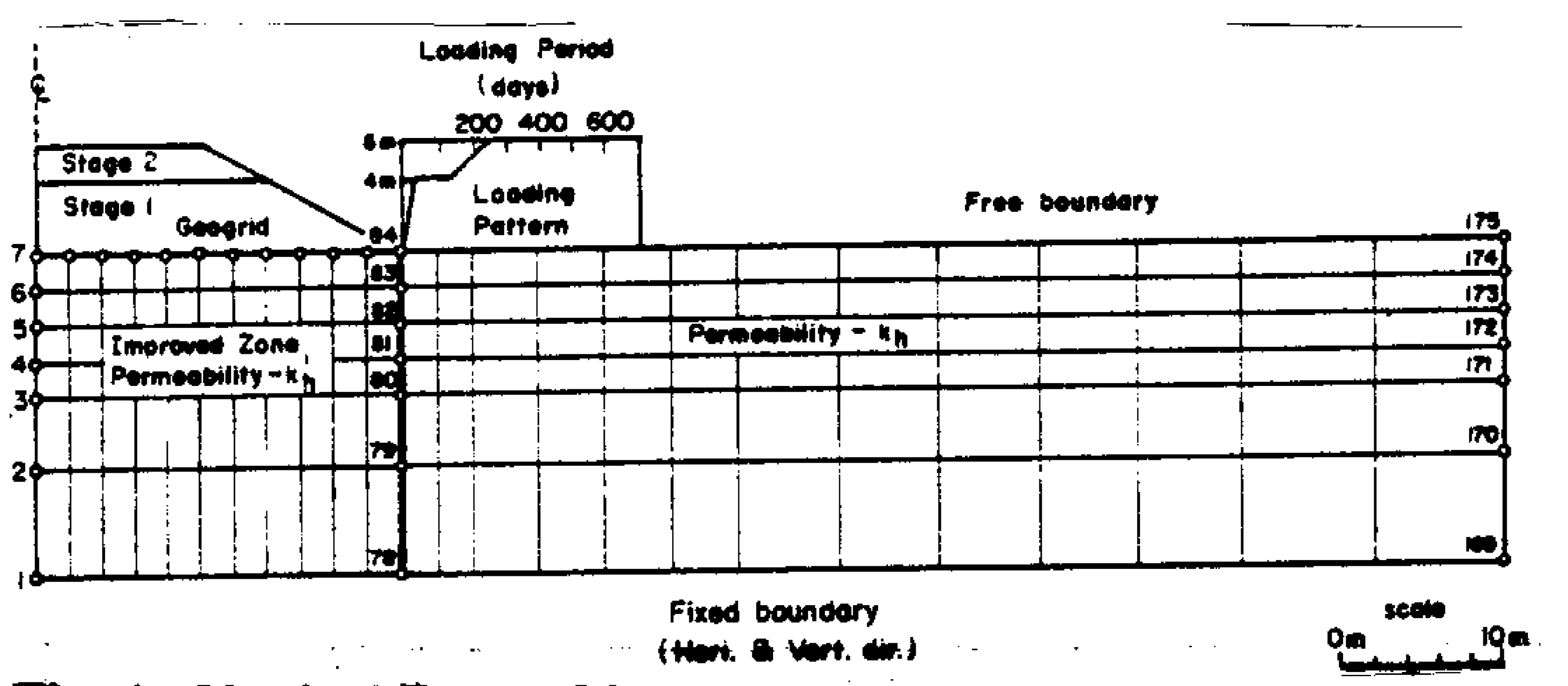

Fig. 4.- Vertical Drains Model, CASE A: FE Modelling of the Embankment

The settlement and lateral deformation behaviour of the embankment at the end of each loading and consolidation stage were computed seperately in order to establish deformation profile with stage loading and to compare the field measurements at each step. Then the analysis was repeated with vertical drains.

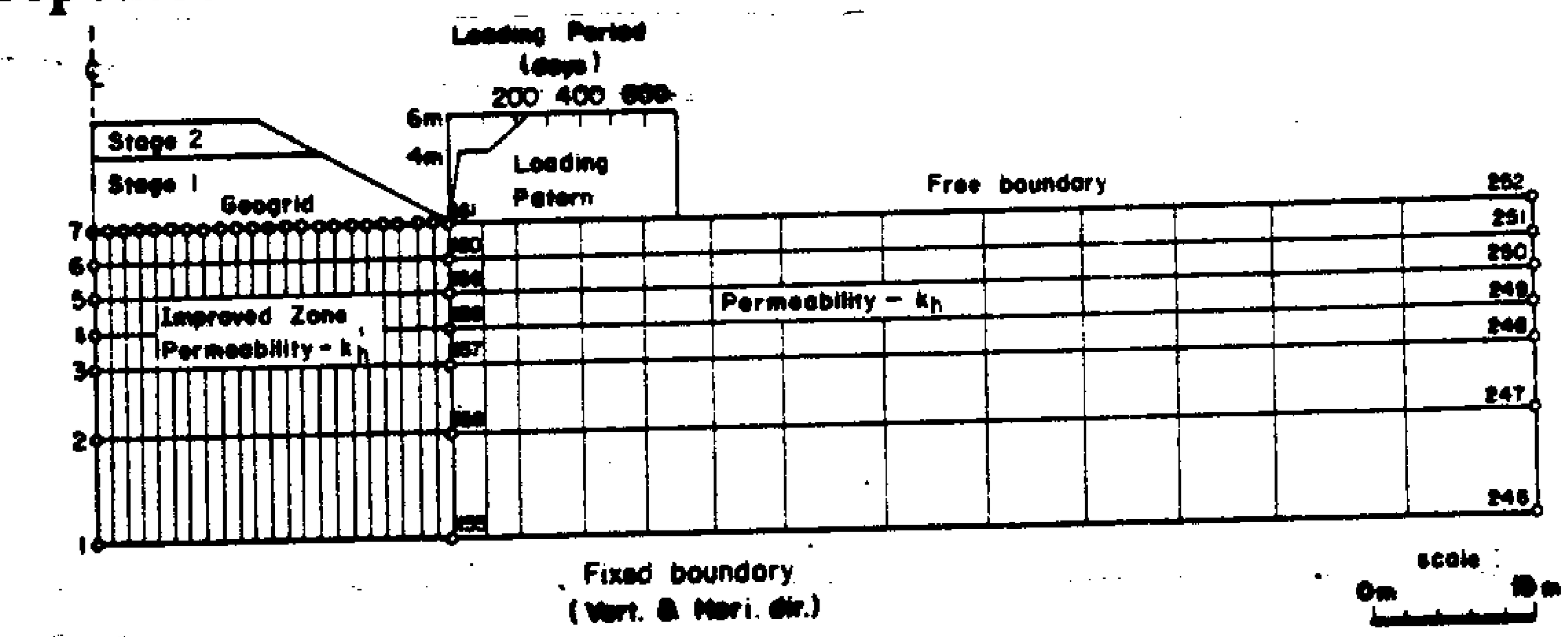

Fig. 5.- Vertical Drains Model, CASE B: FE Modelling of the Embankment

Finally, the geogrid, at the base of the embankment, was introduced with vertical drains. The geogrid model used in this analysis was decribed before.

7 RESULTS AND DISCUSSION

Initially, the analysis was performed with the absence of vertical drains and geogrids. Undrained and coupled consolidation analysis were performed. In the field, the embankment load was applied in two stages. About 0.21 m immediate and 0.29 m consolidation settlements were obtained at the end of stage 1. Similarly, 0.48 m immediate and 0.61 m consolidation settlements were obtained at the end of stage 2. Lateral deformations below toe of the embankment was also calculated for both stages of loading. During stage 1 loading, the maximum lateral deformation observed was 0.21 m and for stege 2, 0.39 m was observed.

Similar analysis was repeated with band drains in the embankment foundation. Two models (case A and Case B) were used to transform axisymetric drain behaviour to plane strain conditions. It was found that the predicted settlement in case A is about 10% less than that of case B at the end of stage 1. Similarly, about 8% difference in settlement observed at the end of stage 2. In both cases, heave was not observed at the toe of the embankment.

Finally, the analysis was extended to incorporate geogrid also. Geogrid was modelled as disscused before. Stress developed in the interface of the geogrid was calculated externally from the displacement and thus the strain developed at the embankment foundation. Both models of vertical drains, case A and case B, were analysed with geogrid seperately.

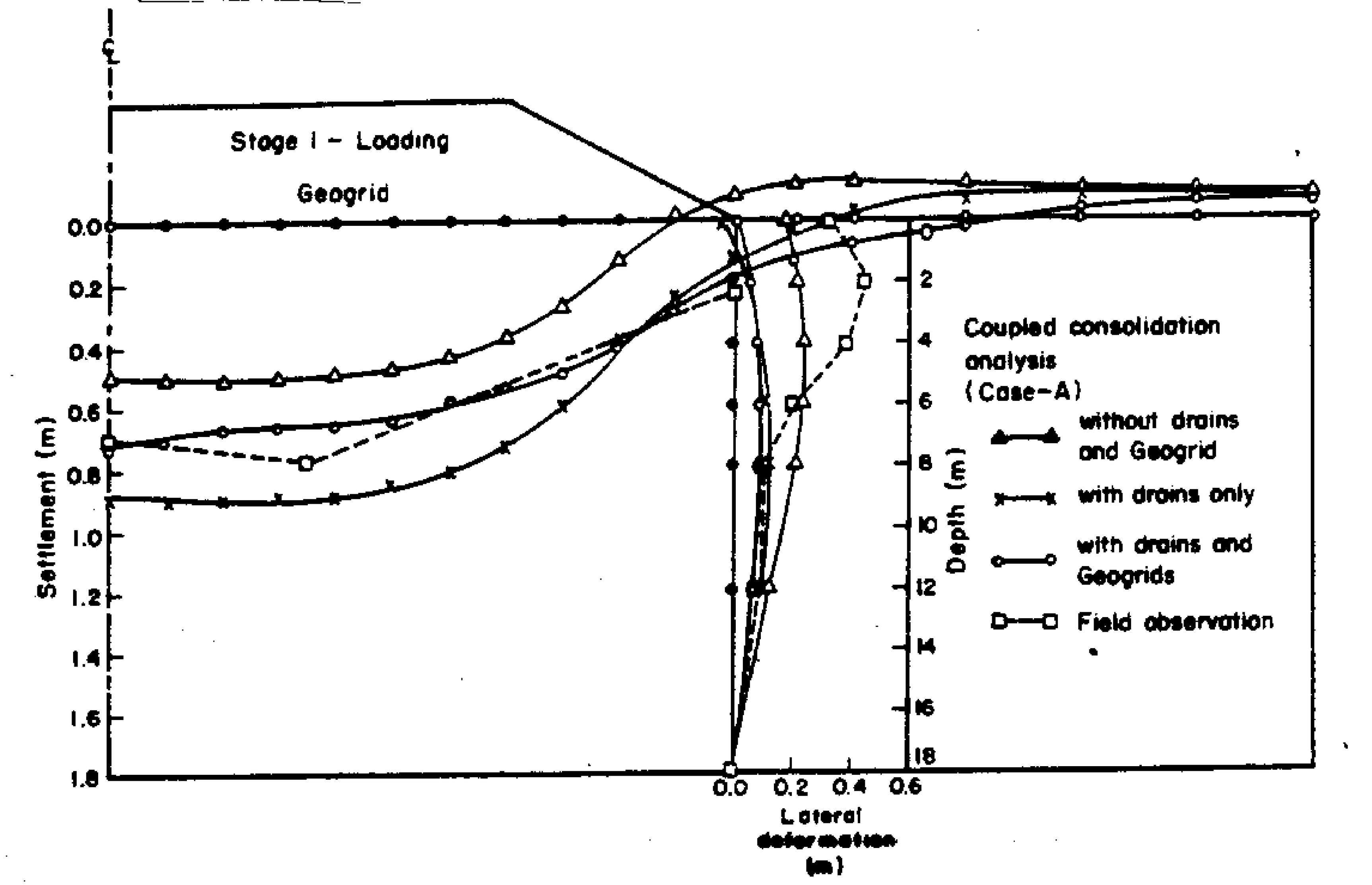

Fig. 6.- Vertical Drains Model, CASE A: Deformation Pattern (Stage 1)

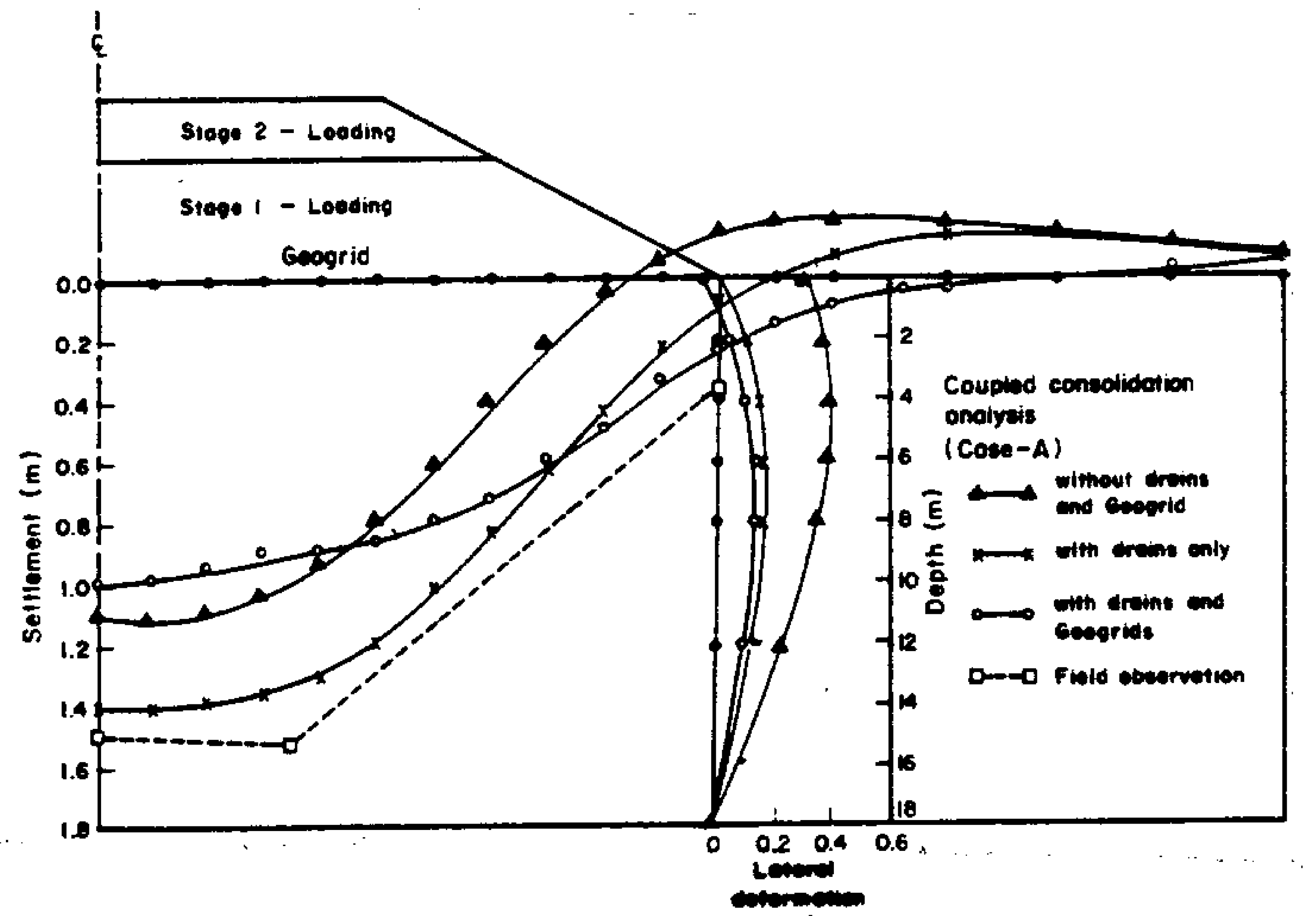

Fig. 7.- Vertical Drains Model, CASE A: Deformation Pattern (Stage 2)

Similar predictions made for case B as for case A. In this case, FEM results under predicted the settlement for both stage 1 and stage 2. Predicted settlement was 14% less than that of field settlement, at the end of stage 1, below the center of the embankment. For stage 2, about 30% of under prediction obtained when compared to field observations as in the case A. Similar predictions were made for lateral deformations. It can be observed that the lateral deformations for case B is marginally lower than that of case A. As for the case A, no heave observed at the vicinity of the embankment.

In both cases, predictions from FEM analysis are matching with the field observations at the end of stage 1. But about 30% of under predictions were observed in stage 2. This identical diference in both cases may be due to the slipping of geogrid in higher loads, temperature effect and the creep of the geogrid material.

In case A, the introduction of vertical drains icreased the consolidation settlement by 0.4 m and the introduction of geogrids reduced the consolidation settlement by 0.2m at the end of stage 1. At the end of stage 2, vertical drains increased the consolidation settlement by 0.3 m while the geogrid reduced the consolidation settlement by 0.4 m. Similar observations for case B is also shown in Table 5. This shows that, for high embankments, if slippage is not allowed, geogrids are more effectively control the post construction settlements.

8 CONCLUSIONS

The finite element analysis coupled with Critical State Soil Mechanics theory was performed to predict the settlement and lateral deformation behaviours of an embankment, treated with vertical drains and geogrid, using CRISP computer program. Two different vertical drain models, case A and case B, were adopted in FEM analysis to transfer 3-D axisymmetric into 2-D plan strain condition. Settlements obtained in case A (i.e., considering a vertical drainage line beneath the toe of the embankment and assigning an equivalent value of permeability to the zone actually comprising these drains) was less than the settlements obtained from case B (i.e., considering vertical drainage lines beneath the embankment foundation arranged in equivalent spacing and assigning a different equivalent value of permeability to the zone comprising these drains). At the end of stage 1, settlements obtained in case A is 15% less than that obtained in case B, and this difference is about 10% at the end of construction. On comparison of final results (analysis incorporating geotextiles) with field measurements, the model used in case A is an appropriate one to convert 3-D axisymetric drain behaviour into 2-D plane strain conditions for analytical purposes. Accurate predictions were obtained upon usage of this model

A geogrid model, which couple the geogrid and soil-geogrid interface properties externally through eqilibrium and strain compatibility conditions, was adopted in FEM analysis. The predictions were perfect at the end of stage 1. Therefore, it may be concluded that, for low embankments (< 3 m), geogrids can be successfully modelled using strain compatibility and equilibrium conditions as an alternative to the use of interface and bar elements in FEM analysis. For higher embankments (> 3 m), slipage and creep behaviours of geogrid need to be considered in order to model the geogrids. Settlements obtained without incorporating slipage and geogrid creep in the analysis lead to an under-estimation of about 30%.

The geogrid at the base of an embankment, besides reducing the lateral spread at the toe, reduces the heave in its vicinity. Further, it enhances the stability of the embankment on step loading and ensure a positive control of the differential settlement of the embankment.

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