Finite Element Analysis of Geocell Reinforced Embankment

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ABSTRACT: The use of geocell mattresses has proven to be an economic solution to the problem of constructing embankments over very soft ground. The combination of high tensile strength geogrid reinforcement erected into a 1.0 m deep cellular form and filled with granular material provides a stiff reinforced layer at the base of the embankment, permitting the maximum bearing capacity of the soft foundation soil to be mobilised. The paper describes the development of a Finite Element Model (FEM) for this type of construction and the results of a FEM analysis on a geocell reinforced embankment built over soft clay and peat at Dartford, England. Comparisons of data from field monitoring with that from the FEM analysis are also presented.

1 INTRODUCTION

A geocell mattress is an open cellular structure of vertical diaphragms of high strength geogrid reinforcement connected in a triangular pattern and filled with granular material, Fig. 1.

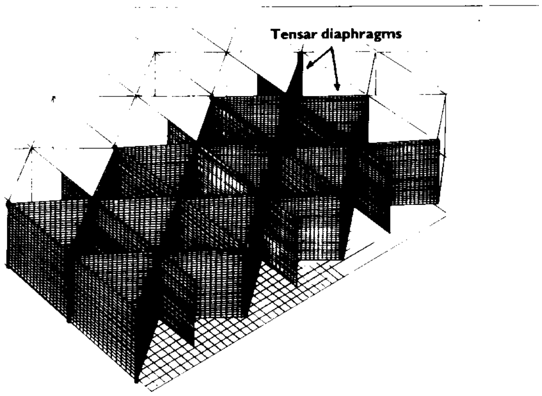

Fig. 1 Typical geocell mattress layout

The high strength mattress provides a stiff platform for embankment construction and minimises the effects of differential settlement of the underlying soil. The mattress intersects potential failure planes and its rigidity forces them to pass vertically through the mattress and deep into layers of the foundation soil.

A method to analyse geocell reinforced embankments in 2-D using the FEM was developed, using the combination of at least 2 elements to represent the mattress. One element representing the geogrid reinforcement is superimposed on a second element representing the soil material. The soil element which is weak in tension transfers tensile stresses exceeding its strength to the reinforcing element, which has high strength in tension, thus simulating the behaviour of soil confined in the cells of a geocell mattress.

A road embankment was built on very soft and highly compressible alluvial soils near the River Thames at Dartford, between July and December 1992. To ensure the stability of the embankment and avoid settlement problems a geocell mattress and vertical drains were installed under the embankment. Soil and monitoring data from this project were used in the model to compare predicted and actual performance of the structure.

2 THE SUPERIMPOSING ELEMENT

The geocell mattress was modelled by superimposing separate elements representing the soil and geogrid reinforcement. Common space and nodes are applied to each element, to achieve identical deformations on both elements. The soil element is modelled as a plane strain element, while the reinforcing element is modelled as a plane stress element corresponding to the physical shape of the geogrid, Fig. 2.

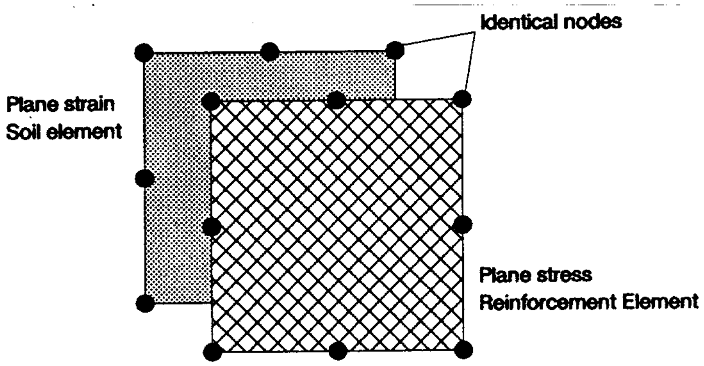

Fig. 2 The superimposing reinforcement element

The tensile strength of the reinforcing elements comes from the tensile strength of the vertical diaphragms of uniaxial geogrid. This leads to the use of orthotropic modelling of the reinforcing material (e.g. Valliappan, 1981)

2.1 BEHAVIOUR UNDER VERTICAL LOADING

An example of the behaviour of the combined element is carried out by testing one combined element of the size 1 m by 1 m, loaded vertically. The soil parameters used in the test are $\nu=0.3$, $E=10^5$ kN/m^2, $c=0$ kPa, $\phi=30°$, and dilation angle $\psi=0°$. The parameters for the reinforcing material were estimated from the properties of Tensar SR80 geogrids used for the vertical diaphragm of which the short term, quality control, tensile strength is 80 kN/m and the long term tensile strength is 32.5 kN/m.

The result of the test in Fig. 3 shows increase of the soil capacity to resist vertical loading relative to the capacity of the same soil without any reinforcement.

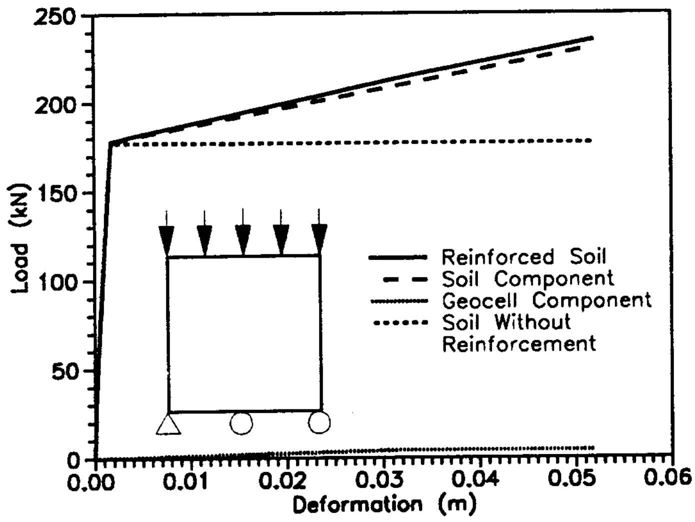

Fig. 3 Load-displacement responses of reinforced soil element and unreinforced element

The curve in Fig. 3 also indicates that the total capacity of the reinforced soil is greater than the sum of the individual response of the components, i.e. the soil and the reinforcing sheet. The vertical load is mainly carried by the soil component of the reinforced soil element, due to the increase of the confining pressure given by the reinforcing element to the soil element.

3 THE DARTFORD ROAD EMBANKMENT

The geology of the area where the Dartford road embankment was built is described as Alluvium overlying Flood Plain Gravels and Upper Chalk. A surface layer of Made Ground was also expected as a result of industrial development in the area. The Alluvium was described as blue grey marsh clay interstratified with beds of amorphous and fibrous peats and underlain by sands and gravels. The soft alluvial deposits, typically 6 to 9 metres thick, show longitudinal, transverse and depth variations in lithology and thickness (Payne, 1993). Profiles of the moisture content and the unit weight of the soils at Sta. 1550 are shown in Fig. 4.

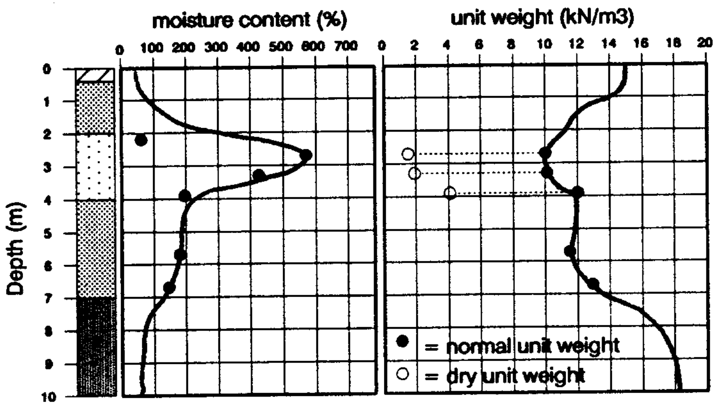

Fig. 4 Soil profiles at Sta. 1550

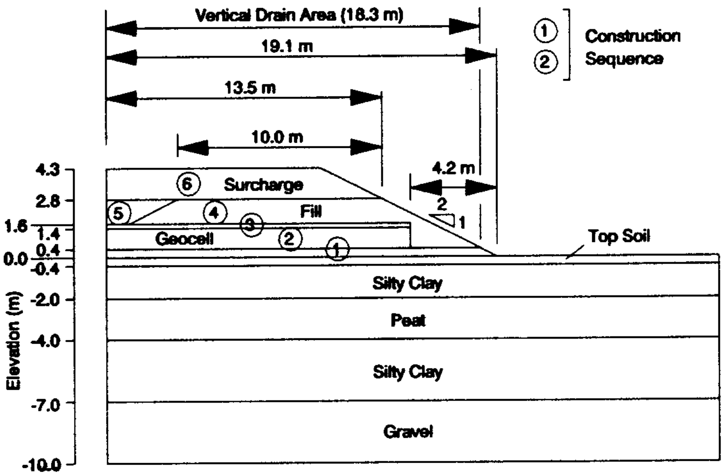

Fig. 5 The structure of Dartford road embankment and its construction sequence at Sta. 1550

The structure and dimensions of the embankment at Sta. 1550 together with the notation of the construction sequence is given in Fig. 5, in which the vertical drain area is the region where the vertical band drains were installed. The drains were typically about 9 metres long, installed in a diamond pattern with 1.5 metres spacing.

3.1 MODELLING THE SOIL

Modelling the soil in undrained or partially drained conditions requires a rather complex constitutive model to be used, as the simple elastic-perfectly plastic Mohr-Coulomb constitutive model would generally underestimate the generation of excess pore pressures in the soil (Hicks and Wong, 1988). In the present analysis, the double hardening constitutive model Monot (Molenkamp, 1981), was used. However, due to data limitations, not all of the soil types in the analysis were modelled with Monot. Only the upper silty clay, the peat, and the lower silty clay layers were modelled with Monot. The rest of the soils which data were not known were modelled with the Mohr-Coulomb constitutive model using estimated parameter values.

Modelling soil with Monot constitutive model, however, requires the model to be calibrated with a number of laboratory soil tests, sufficient to bring out the essential features of the soil behaviour. The more complete the data available, the better the model could be 'fine-tuned'. Example of the results of the calibration on the peat material, based on triaxial undrained tests, with isotropic consolidations of 30 and 60 kPa, are shown in Figs. 6-7, in which t and γ are the shear stress and the strain invariant, respectively, and defined as follows:

$$t = \left[\frac{1}{3}\left\{(\sigma_x-\sigma_y)^2+(\sigma_y-\sigma_z)^2+(\sigma_z-\sigma_x)^2\right\}+2\{\tau_{xy}^2+\tau_{yz}^2+\tau_{zx}^2\}\right]^{\frac{1}{2}} \qquad (1)$$

$$\gamma = \left[\frac{1}{3}\{(\varepsilon_x-\varepsilon_y)^2+(\varepsilon_y-\varepsilon_z)^2+(\varepsilon_z-\varepsilon)^2\}+\frac{1}{2}\{\gamma_{xy}^2+\gamma_{yz}^2+\gamma_{zx}^2)\}\right]^{\frac{1}{2}} \qquad (2)$$

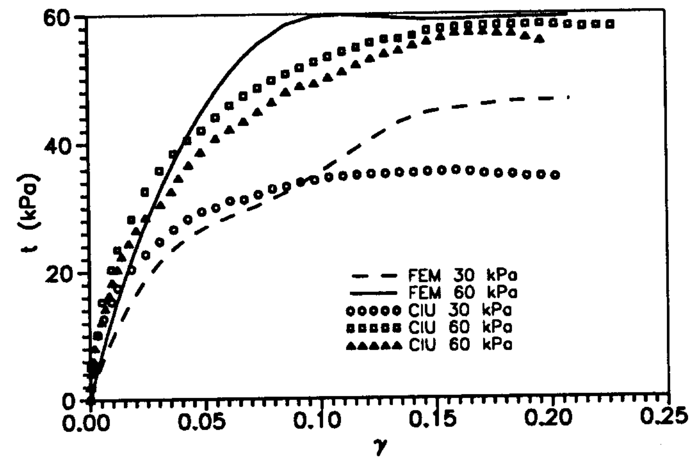

Fig. 6 Calibration of the peat material on t-γ curve

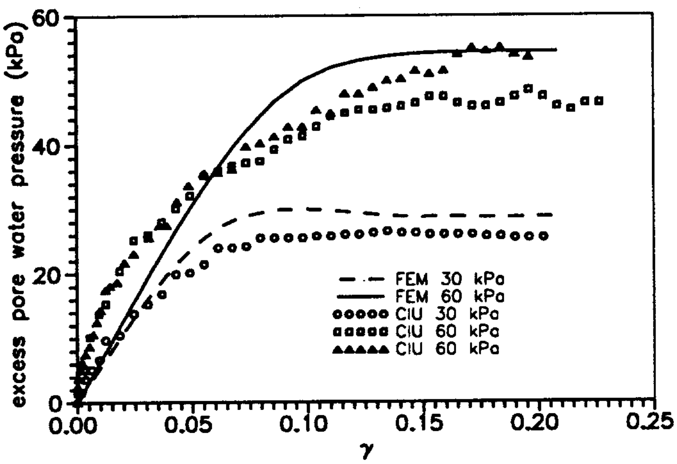

Fig. 7 Calibration of peat material on excess pore water pressure - γ curve

3.2 THE FINITE ELEMENT ANALYSIS

Analysis of the Dartford road embankment was carried out by simulating the construction sequence of the embankment, by adding new layers of elements as the construction progressed. The sequence and rate of the construction was approximated by the curve shown in Fig. 8, which is a series of construction activities and rest periods between constructions.

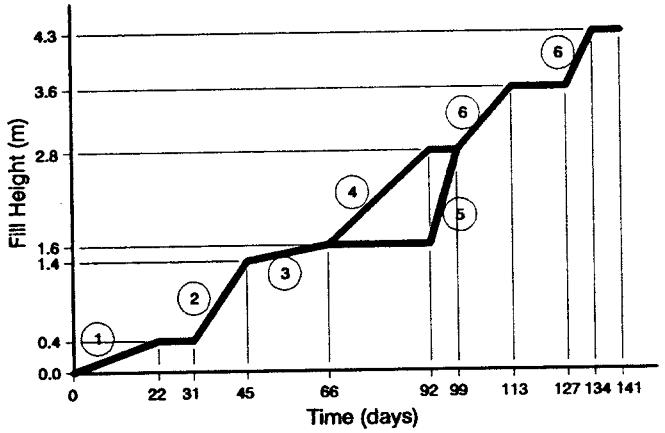

Fig. 8 The approximation of the embankment construction rate at Sta. 1550

In the analysis, no special treatment was given to the area where the vertical drain were installed. Permeabilities of the soil layers were estimated to yield similar pattern of response as the generated pore pressure recorded on site.

The recorded and calculated excess pore water pressures (epwp) during the construction of the embankment is shown in Fig. 9., and the finite element mesh and the calculated epwp contour at the end of construction at day 141, is shown in Fig. 10.

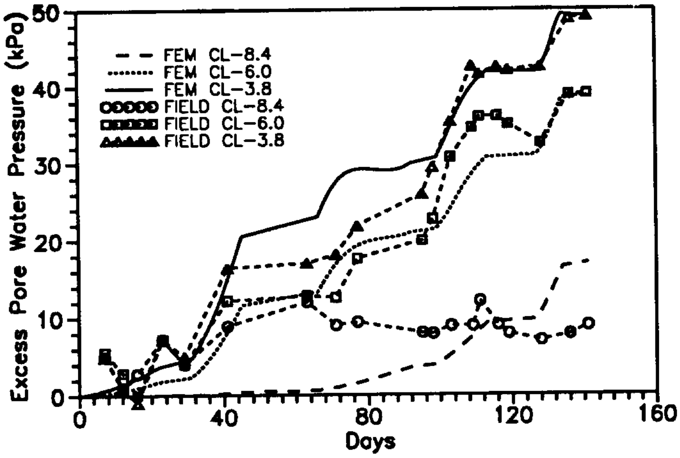

Fig. 9 The response of recorded and calculated epwp.

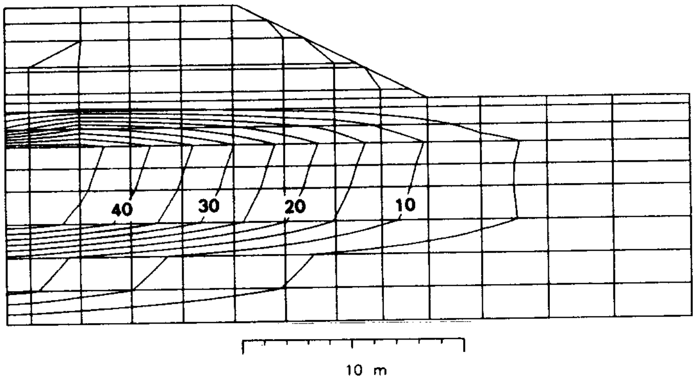

Fig. 10 Contour of epwp in kPa at day 141

Although the epwp can be approximated reasonably well, the calculated vertical and horizontal deformations, however, show considerable difference with the recorded values on site. Figs. 11-12 show the comparison between the calculated and the recorded values of these deformations.

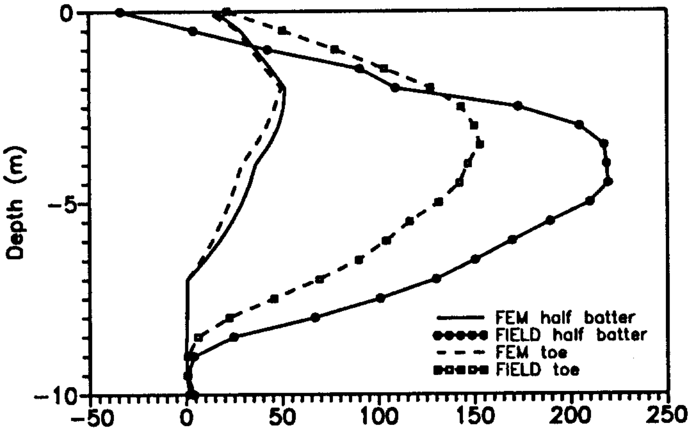

Fig. 11 Calculated and recorded values of horizontal deformations under the points at half batter and the toe of the slope

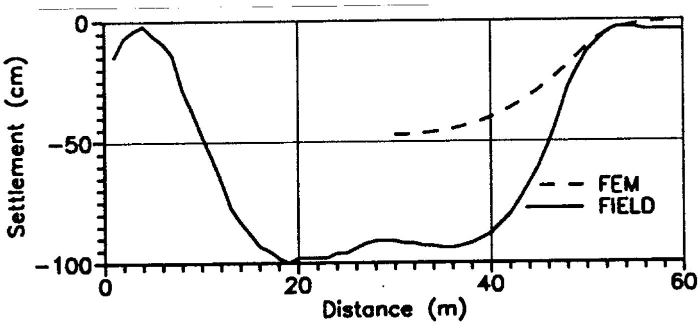

Fig. 12 Vertical deformations under the embankment

4 CONCLUSIONS

The superimposing element technique, gives the desired response of increasing the soil strength by the increase of confining pressure. Further research is still needed to observe the possibility of using the 2-D idealisation in the design of embankment with geocells.

The FEM analysis of the Dartford road embankment suffers due to the limitation of the soils data available. It is difficult to single out the cause of the discrepancies between the recorded and the calculated behaviour of the embankment because the number of unknown parameters which have to be assumed, is quite significant. However, the analysis shows the potential of using the FEM to observe many aspect of the behaviour of embankments being constructed at any stage of the construction activities.

5 ACKNOWLEDGEMENT

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