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FINITE ELEMENT STUDY OF GEOTEXTILE REINFORCED EMBANKMENTS

ETUDE PAR ELEMENTS FINIS DE MASSIFS RENFORCES PAR GEOTEXTILE

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SYNOPSIS: Results of a parametric study to investigate the applicability of the finite element method and to examine the effect of reinforcement modulus on the behavior of a reinforced embankment are presented. The various components of a reinforced soil system which include soil, interface and reinforcement are presented and modeled by the finite element method. The results show that both vertical and horizontal displacements in the embankment and foundation soil decrease with the increase in reinforcement modulus, and the overall stability of the embankment is improved. For the typical range of geotextile strength modulus of 818 to 3500 kN/m, the reduction of horizontal displacement is as high as 47 percent whereas the reduction of vertical displacement is only about 13 percent.

INTRODUCTION

A parametric study to investigate the applicability of the finite element method and to examine the effect of reinforcement modulus on the behavior of reinforced earth embankments is reported in this paper. The finite element analysis of reinforced earth embankments has been conducted previously by Rowe (1982), Schaefer and Duncan (1988) and others. The creep inclusive elasto-plastic constitutive material model for soil, and some of the elements used in this finite element study have not been applied previously to model the behavior of reinforced earth embankments.

A typical embankment reinforced with a single layer of geotextile and constructed over a very soft foundation soil is considered in this study. The entire analyses were performed using the nonlinear plane-strain finite element computer program TUSPIN (Tavassoli 1991). This program was developed at Tulane University by extensively modifying the computer program SPIN2-D (Borja 1984) to include new types of elements and material models for soil, reinforcement and interface. The Borja and Kavazanjian (1985) creep inclusive elasto-plastic soil model, a derivative of the Cam-Clay model capable of modeling soft clays under undrained, drained and consolidation is used to model both the embankment and the soft clay foundation soil. In the absence of creep, seven material parameters are required to define this constitutive model. These parameters are virgin compression index λ, recompression index k, slope M of the critical state line, reference void ratio e_a at unit preconsolidation pressure and hyperbolic stress-strain parameters a, b and R_f. The value of these parameters can be obtained from the results of isotropic triaxial tests. The interaction between soil and reinforcement is modeled by interface elements placed above and below the reinforcement. The parametric study was conducted by using the control parameters based on an undrained loading condition in the foundation soil and a drained loading condition in the embankment. In the parametric study reported in this paper, only the value of reinforcement modulus was changed while keeping the remaining parameters at their control values.

THE FINITE ELEMENT MODEL

The general layout of the typical embankment used in the parametric study is shown in Figure 1. The embankment under consideration is 3 meters high with symmetrical side slopes of 1 vertical on 2.1 horizontal and is reinforced with a single layer of geotextile placed on the original ground surface beneath the embankment. The embankment has a 6.1 meters wide crest and is constructed in five lifts of 0.6 meter high each. The foundation soil is assumed to be a thick deposit of soft clay with the water table located at the ground surface. Due to the symmetry of the embankment and the foundation soil, only one half of the problem is modeled in the finite element analyses.

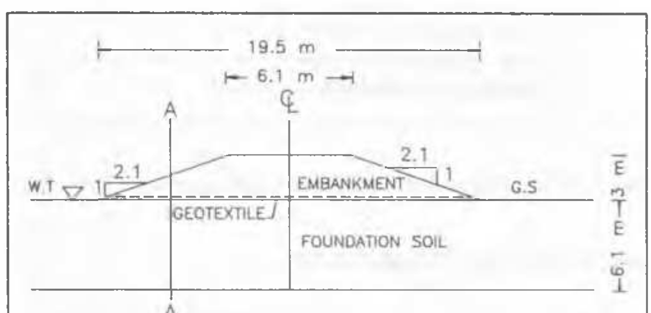

Fig. 1. Geometry of the control embankment.

Finite Element Mesh

The finite element mesh used to model the typical embankment is shown in Figure 2. The geometry of the mesh is selected to assure the accurate modeling of the embankment, to minimize the effect of the boundaries on the behavior of the embankment and to minimize the computational cost. The bottom boundary of the mesh is located at a depth of 6.1 meters below the ground surface of the foundation soil. This depth was selected from the results of a parametric study (Tavassoli 1991) which consisted of varying the foundation depth until little or no displacement in the vicinity of the bottom boundary was observed. The left side boundary of the mesh is extended to approximately 16.5 meters beyond the toe of the embankment and the right side boundary of the mesh is aligned along the centerline of the embankment to reflect the effect of symmetry.

The finite element mesh consists of 803 nodes of which 196 are pore pressure nodes to model the undrained loading condition in the foundation soil at the end of the embankment construction. The mesh consists of 268 elements and is divided into 9 element groups. These element groups represent foundation soil, lower interface, geotextile reinforcement, upper interface and the embankment. The element group representing the foundation soil consists of 162 eight-noded quadrilateral isoparametric elements each with four pore pressure nodes at the four corners. Two other element groups contain the upper and lower interface elements. Each of these interface element groups consist of 16 six-noded isoparametric interface elements with zero thickness and is placed between soil elements and reinforcement elements. Sixteen three-noded isoparametric bar elements are included in an element group to model the behavior of the geotextile reinforcement. Each of these bar elements is attached to an upper and a lower interface element. The remaining five element groups which consist of 58 eight-noded isoparametric elements with no pore pressure capability model the five lifts used in the construction of the embankment.

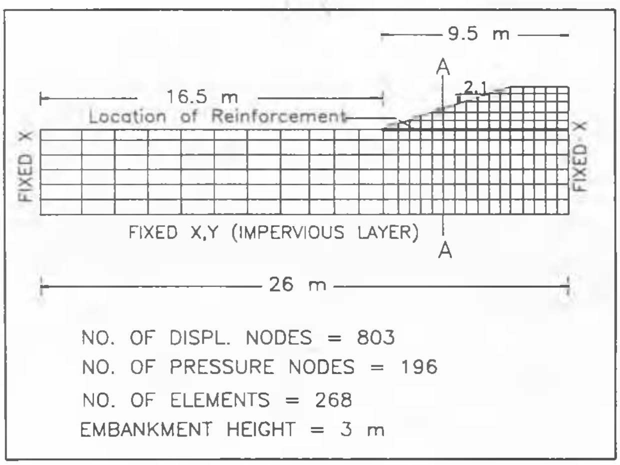

Fig. 2. Finite element mesh of the control embankment.

Material Model For Foundation Soil

The foundation soil is assumed to be a thick deposit of soft clay with unit weight of 17.3 kN/m^3 and an overconsolidation ratio of 1.5. It is modeled in the analyses with an elasto-plastic Cam-Clay material model using control properties established from the published literature for the Boston Blue Clay (Borja 1984). The control material parameters for the foundation soil are virgin compression index $\lambda = 0.15$, recompression index $\kappa = 0.06$, slope of critical state line $M = 1.05$, reference void ratio $e_a = 1.74$ at a preconsolidation of 1 kPa and hyperbolic stress-strain parameters $a = 0.0062$, $b = 2.73$ and $R_f = 0.9$. The undrained shear strength of the foundation soil is assumed to be 6.7 kPa at the ground surface, and to increase linearly with depth. The horizontal permeability ($K_x = 0.002$ m/day) is assumed to be four times higher than the vertical permeability ($K_y = 0.0005$ m/day) which is a typical condition for such a thick deposit.

Material Model For Earth Embankment

The earth embankment material is a silty sand with a unit weight of 17.3 kN/m^3. The embankment is modeled as a drained elasto-plastic Cam-Clay material using the control properties reported by Borja (1984). The control parameters for the embankment material are $\lambda = 0.025$, $\kappa = 0.005$, $M = 1.5$, $e_a = 2.5$ at preconsolidation pressure of 1 kPa, $a = 0.0012$, $b = 0.77$ and $R_f = 1.0$. The embankment is constructed in five lifts using forty load increments for each lift to ensure convergence. The embankment is assumed to have a drained loading condition since no significant excess pore pressure develops in the embankment during the construction due to its relatively high permeability.

Geotextile Reinforcement

The single layer of geotextile reinforcement is modeled as a linear elastic materiel with failure assumed to occur at five percent strain level. This strain level is commonly used by the U.S. Army Corps of Engineers (USACE 1989) in designs as the failure criterion for geotextile. The reinforcement is assumed to have a thickness of 0.3 cm and a strength modulus that varies from 0 to 3500 kN/m. The control parameter for the geotextile modulus is set at 1600 kN/m. The reinforcement modulus is equivalent to Young's modulus of the geotextile reinforcement multiplied by its thickness which is used in the computation of the tensile force in the reinforcement in terms of force per unit width. A reinforcement modulus of zero represents the case of a conventional unreinforced embankment.

Interface Between Soil And Geotextile

The material model for the interface between the geotextile and soil is assumed to be governed by a hyperbolic model (Clough and Duncan 1971). The bond strength of the interface is governed by the Mohr-Coulomb slip criterion, and once the bond strength is exceeded, the shear stiffness of the interface is automatically reduced to a value close to zero to model a slip condition. The bond strength between the foundation soil and the reinforcement is governed by the undrained shear strength at the foundation soil surface while the bond strength between the reinforcement and embankment is governed by friction angle of the embankment material (Rowe 1984). The lower interface is assumed to have a $\phi = 0^o$, adhesion $C = 6.7$ kPa and a failure ratio $R_f = 0.9$ while the control parameters for upper interface are $\phi = 37^o$, $C = 0$ kPa and $R_f = 0.9$. The control values of the shear stiffness and normal stiffness of the interface are 9420 kN/m^3 and 1.5×10^6 MN/m^3, respectively. A high normal stiffness is used to prevent any significant vertical deformation in the interface. The control value of shear stiffness of the interface resulted in no significant relative movement in the interface.

Boundary And Loading Condition

The bottom boundary of the finite element mesh is located far enough from the surface of the foundation soil to be affected by the construction of the embankment, as discussed previously. Accordingly, it is restrained in the horizontal and vertical directions throughout the analyses. The left and right side boundaries of the mesh are fixed in the horizontal direction only. These boundary conditions model the infinite extension of the foundation soil on the left hand side of the embankment, and the condition of symmetry along the centerline of the embankment. In addition, no drainage is allowed at the bottom boundary of the foundation soil (impervious). However, drainage is allowed at the surface of the foundation soil. The embankment construction is simulated by placing five lifts in five construction days and by applying 40 load increments for each lift. Although drainage is allowed at the ground surface boundary, the low permeability of the foundation soil and the rapid construction period of five days model the undrained loading condition that exists during and at the end of the embankment construction.

EFFECT OF REINFORCEMENT MODULUS

Four different analyses are performed on the typical embankment using reinforcement strength moduli ranging from 0 to 3500 kN/m. A conventional embankment with the control parameters and no reinforcement (strength modulus of 0) was modeled in case 1. Analyses of cases 2 through 4 represent embankments reinforced with geotextile moduli of 818, 1600 and 3500 kN/m, respectively. A reinforcement modulus of 818 kN/m is within the range of strength modulus for Advance Type I geotextile (613-818 kN/m), reinforcement modulus of 1600 kN/m is close to value of strength modulus of Stabilenka 200 (1900 kN/m), and a reinforcement modulus of 3500 kN/m is slightly higher than the strength modulus of high tenacity woven polyester geotextile (3360 kN/m). In each of the analyses, horizontal displacement along profile A-A located at distances of 6.7 meters away from the centerline of the embankment and vertical displacement at the embankment centerline at the ground surface are compared (see Fig. 1). In addition, tensile force and tensile strain in the reinforcement are compared.

The pattern of vertical displacement along the ground surface is similar in all cases with a small heave occurring immediately beyond the toe of the embankment and a maximum vertical displacement occurring beneath the centerline of the embankment at the ground surface. A maximum vertical displacement of 13.5 cm occurs at the centerline of the unreinforced embankment. The maximum vertical displacement decreases to 11.7 cm at the centerline of the embankment as the reinforcement modulus increases from 0 to 3500 kN/m resulting in a 13 percent reduction.

The pattern of horizontal displacement along profile A-A is similar in all cases with the maximum horizontal displacement occurring near the ground surface and the minimum displacement occurring close to the base of the mesh. A maximum horizontal displacement of 18.1 cm occurs along profile A-A at a depth of 0.9 meter below the ground surface of the conventional embankment. The maximum horizontal displacement decreases to a minimum of 9.5 cm at the same location as the reinforcement modulus increases to 3500 kN/m for a 47 percent reduction.

Development of the tensile force is similar in all cases, namely the tensile force in the reinforcement increases with distance from the embankment toe toward its centerline. A maximum tensile force of 20 kN/m occurs in the reinforcement at the embankment centerline in case 2 where a reinforcement modulus of 818 kN/m is used. As the reinforcement modulus increases from 818 to 3500 kN/m in case 4, the tensile force in the reinforcement at the centerline of the embankment increases to 47 kN/m. The tensile strain in the geotextile also increases gradually from zero at the embankment toe to a maximum value at the centerline of the embankment. However, as the reinforcement modulus increases, tensile strain decreases in the reinforcement. Case 2 with a reinforcement modulus of 818 kN/m results in a maximum strain of 2.4 percent in the reinforcement at the centerline of the embankment. The strain decreases gradually to a minimum of 1.25 percent at the centerline of the embankment in case 4 where a reinforcement modulus of 3500 kN/m is used. Accordingly, a reduction of 48 percent in the maximum tensile strain is achieved by about a four fold increase in the geotextile modulus. In addition, the strain computed in all cases are well below the five percent strain typically used in designs as the failure criterion for geotextile reinforcement. The maximum horizontal displacement, maximum vertical displacement, tensile force and tensile strain versus reinforcement modulus for analyses of cases 1 through 4 are plotted in Figures 3 through 6, respectively.

Maximum horizontal displacement along profile A-A near the toe is normalized by the width of the embankment. In addition, maximum vertical displacement at the centerline of the embankment is normalized with respect to height of the embankment. The normalized ratios give the percent of displacements that have taken place with respect to dimensions of the embankment. Table 1 lists the maximum horizontal displacement, vertical displacement and the resulting normalized ratios. In this table L is the maximum horizontal displacement at profile A-A, D is the maximum vertical displacement at centerline, W is the width of the embankment (19.5 m), H is the height of the embankment (3 m) and F is the maximum tensile force at the centerline of the embankment.

Table 1. Calculated response for various reinforcement modulus.

Modulus (kN/m)	L (cm)	D (cm)	L/W (%)	D/H (%)	F (kN/m)
0	18.1	13.5	0.94	4.4	0
818	15.2	13.2	0.8	4.3	20
1600	12	12.7	0.63	4.2	32
3500	9.5	11.7	0.49	3.8	46.7

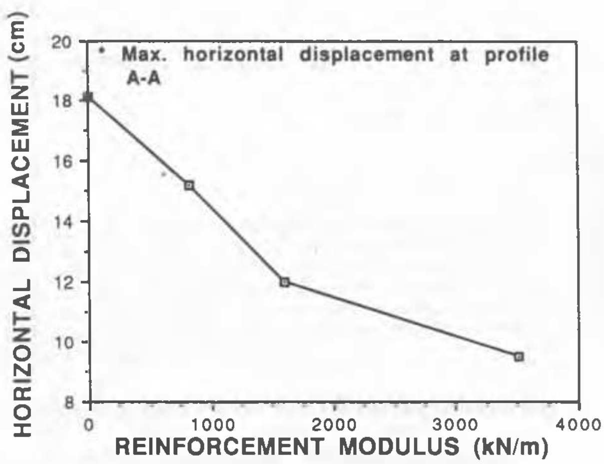

Fig. 3. Effect of reinforcement modulus on horizontal displacement.

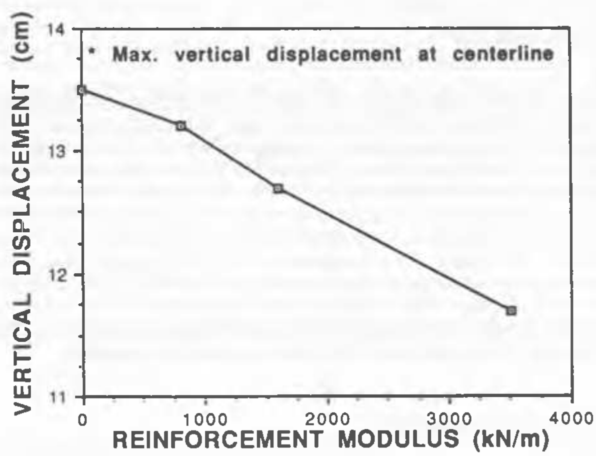

Fig. 4. Effect of reinforcement modulus on vertical displacement.

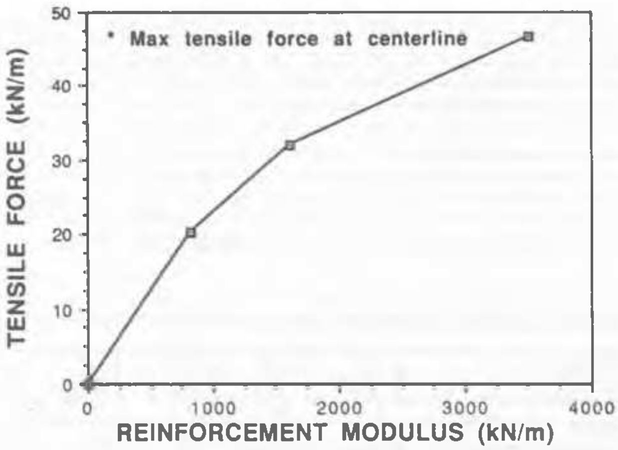

Fig. 5. Effect of reinforcement modulus on tensile force.

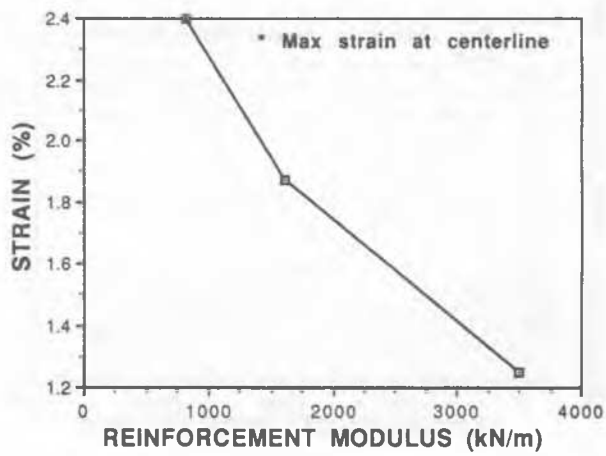

Fig. 6. Effect of reinforcement modulus on tensile strain.

CONCLUSIONS

Parametric study reported in this paper shows that for an undrained loading condition in the foundation soil, horizontal displacement decreases as the reinforcement modulus increases. For the typical range of geotextile modulus of 818 to 3500 kN/m, the reduction of maximum horizontal displacement is about 47 percent. However, for very high reinforcement modulus, the effectiveness of reinforcement in increasing the stability of the embankment might diminish if slip in the interface takes place. The magnitude of vertical displacement also decreases as reinforcement modulus increases. However, the reduction of vertical displacement is only about 13 percent for the range of geotextile modulus discussed above. In addition, tensile force in the reinforcement increases as the reinforcement modulus increases.

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