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Prediction of geosynthetic reinforced wall performance using finite element analysis

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INTRODUCTION

The performance of reinforced soil walls during construction and external loading is the result of simultaneous responses of many localized interactions between the soil and structural materials that make up the wall. These interactions are generally very numerous and highly nonlinear, making overall wall performance complex and difficult to predict at both working stress and near-failure stress conditions. Two general approaches to wall performance prediction can be considered - relatively simple observationally based analysis methods, and relatively complex numerical methods like the finite element method (FEM).

The geosynthetic reinforced wall constructed for this symposium has several physical characteristics which cause it to fall at the outside limits of the range of actual structures from which the observationally based analysis methods have been derived. Some of these characteristics are: reinforcement spacing, 11 inches center-to-center (this is considerably smaller than used in previous models); reinforcement strength is as much as an order of magnitude weaker than is typical; considerable length of overlap at each reinforcement layer, 55% of the length of the reinforcement; location of the surcharge load; restraint provided at the bottom of the wall by the gravel base; and, restraint provided by the top of the wall by the loading frame. An additional limitation of the observationally based methods is that they generally provide little information on wall deformations. For this prediction exercise, the authors have decided to use observationally based methods only to estimate the surcharge level at wall failure.

The authors have selected FEM analysis to model the working stress performance of the test wall. Its primary advantage is the ability to model the multitude of simultaneous interactions that occur between wall components. The FEM code used in this prediction, SSCOMP (Seed and Duncan, 1984), has been used with some success by the authors and others to predict reinforced soil wall behavior (Collin, 1986; Adib, 1989 and Jaber, 1990). The code has not been previously used, however, to predict performance of structures with highly extensible reinforcement and continuous timber facings, which are characteristics of the symposium test wall. This prediction is therefore partially uncalibrated, as it represents a first attempt to apply the SSCOMP code to structures such as the test wall.

ANALYTICAL APPROACH

The analysis presented herein was performed using SSCOMP, a plane-strain finite element code for incremental modeling of soil placement and compaction. This program calculates stresses, strains and displacements in soil elements, and internal forces and displacements in structural elements by simulating the actual sequence of construction operations in a number of steps: The non-linear and stress dependent, stress-strain properties of the soil are approximated by varying the values of modulus and Poisson's ratio in accordance with calculated stresses in a double iterative process. For any increment of analysis, two iterations take place. The first time through, the soil modulus and Poisson's ration values are based on the stress conditions at the beginning of the increment. The second iteration uses adjusted soil properties based on average stresses during the increment.

Four types of elements are used in the program to model each component of the structure discretely, and to model soil structure interaction effects. They are:

* Soil Elements - which are four node, two dimensional isoparametric elements.

* Bar Elements - which are two node elements with axial stiffness only. These elements are used to model the reinforcements.

* Beam Elements - which are two node elements capable of exhibiting axial, bending, and shear stiffness. These elements are used to model the face of the wall.

* Interface Elements - these elements have zero thickness and are capable of modeling soil structure interface conditions through normal and shear springs.

FINITE ELEMENT MODEL

The test wall was modeled as a plane-strain, two dimensional problem for the finite element analysis. Figure 1 shows the nodal point mesh used for the analysis. The mesh consists of: 779 nodes; 338 soil elements; 201 bar elements; 23 beam elements, and 365 interface elements.

There are several areas of the mesh that required special attention. They are briefly identified and discussed in the following paragraphs.

* Geosynthetic-Soil Interface - The proper simulation of the geosynthetic-soil interface is critical to the validity of the F.E.M. analysis. In order to correctly model soil-reinforcement interaction, each layer of bar elements representing the geosynthetic was separated from the overlying and underlying soil by interface elements as illustrated in Figure 2. The use of interface elements allows the model to simulate relative movement between the reinforcement and soil and therefore the transfer of stress between these materials.

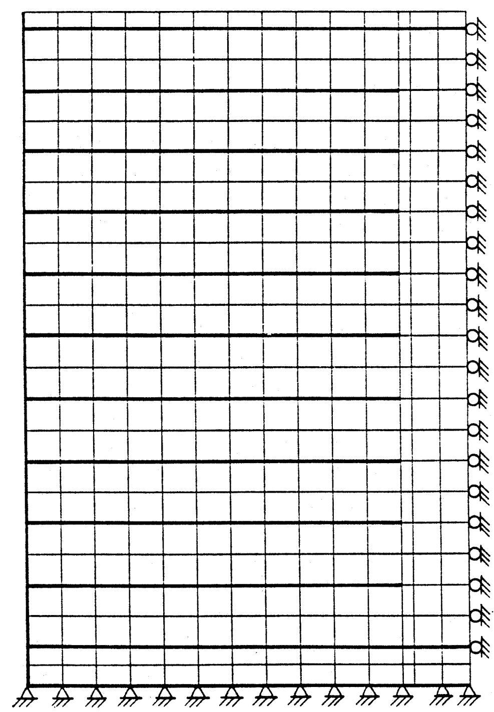

Figure 1. Finite Element Mesh

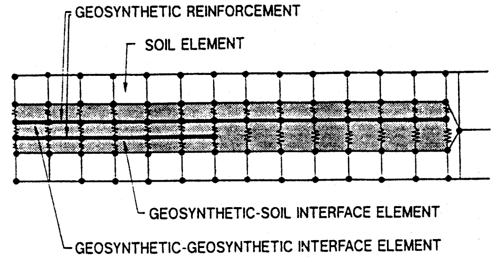

Figure 2. Geosynthetic-Soil Interface Model

* Geosynthetic-Geosynthetic Interface - the length of the overlap between geosynthetics used in the construction of the test walls (55% of the reinforcement length) is believed to have a significant effect on the wall performance and was, therefore, modeled as two bar elements, one directly above the other, connected by interface elements to allow movement (slippage) between geosynthetics. The bar elements have zero thickness and therefore occupy the same location in space. Figure 2 shows the mesh configuration used at each reinforcement layer where an overlap was present. The top and bottom layers of reinforcement (which contained no overlap) were modeled as single layers of bar elements.

* Timber Wall Face - Geosynthetic-Soil Interface - Figure 3 illustrates, schematically, the modeling of conditions at the juncture between the wall face and the geosynthetic. Each normal spring shown actually represents both a normal and shear spring. Nodes 82, 83, 98, 105 and 120 all occupy the same initial location. This is also true of nodes 84, 99, 106 and 121. Interface element I-30 and I-65 permit modeling of shear movement at the soil-wall face interface; interface elements I-31 and I-51 model pullout shear movement between the soil and geosynthetic, and interface element I-37 allows movement between reinforcements. Nodes 82, 98 and 105 are forced to have the same horizontal displacements at all times in the analysis.

* Bottom of Test Box - Gravel-Geosynthetic Interface - In the test chamber, the bottom of the box was prepared by gluing a fine gravel to the steel base and spreading a thin layer of gravel on top of this base until a smooth surface was achieved. The geosynthetic was then placed on top of this surface. In the finite element model, this bottom interface

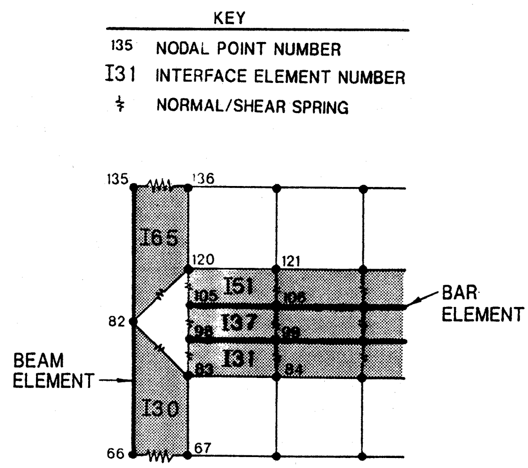

Figure 3. Wall Face-Geosynthetic-Soil Interface Model

was modeled as a rigid foundation with interface elements to allow movement between the base and the geosynthetic.

* Wall Face - the timber wall face was modeled using continuous beam elements. This is believed to be an accurate representation of the actual face which consists of 4" x 5 1/2" timber beams connected to each other by 1/2" thick plywood and 3" long deck screws.

* Loading Increments - the construction of the wall was modeled by 23 construction increments. The surcharge loading was applied in increments of 0.3 psi up to a surcharge of 3 psi and increments of .5 psi up to a total surcharge of 15 psi. It was necessary to apply the surcharge load in these relatively small increments to restrict premature failure of soil elements near the top of the wall that initially have small confining stresses relative to surcharge load increments.

* Loading Frame - Figure 4 shows the loading frame used for the test walls. This frame has a rigid plate, located

directly behind the timber wall face, that slides vertically with the top surface of the wall. The function of this plate is to add confinement to the sand above the top of the wall face. This added confinement changes the performance of the wall by restricting lateral movement of the soil. Stress is rapidly transferred to the top layer of geosynthetic. This soil structure interaction effectively restrains the top of the wall from large lateral deformations. Figure 5 shows schematically the model used to simulate the above described condition at the top of the wall. The restraint provided by the steel plate is modeled by restricting the movement of nodal points 749 and 765 in the horizontal direction.

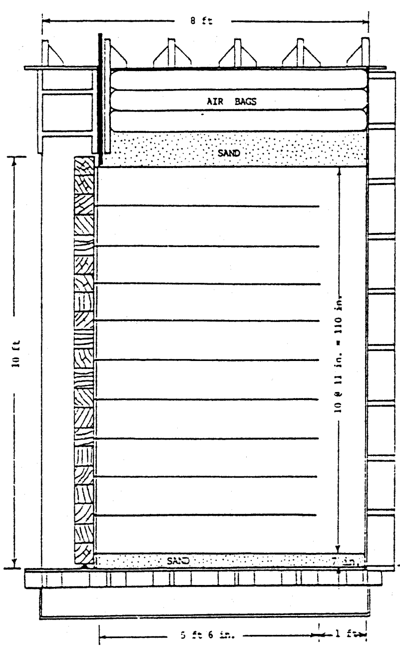

Figure 4. Configuration of the Test Wall and the Loading Facility

MATERIAL PROPERTIES
Soil

The hyperbolic model (Duncan and Chang, 1970) was used to describe the non-linear, behavior of the soil used to construct the test

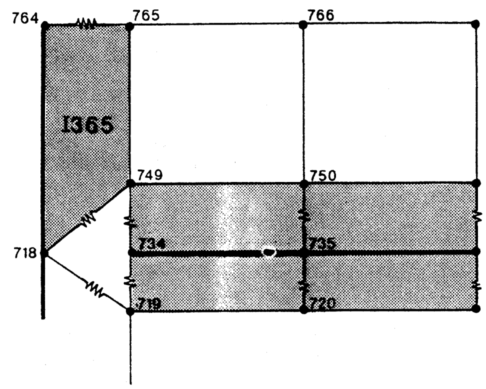

Figure 5. Top of Wall - Schematic Model

Table 1. Summary of Soil Hyperbolic Parameters

PROPERTY	
Unit Weight (PSF)	107
Modulus No., K	1000
Unload Mod. No., K_{ur}	1500
Modulus Exponent, N	.67
Failure Ratio R_f	.85
Bulk Mod. No., K_b	700
Bulk Mod. Exponent, M	.40
Cohesion, C(psf)	0
Friction Angle	38
Friction Angle per log cycle change in $sigma_3$	2
Coeff. of Earth Pressure at rest	.38

Table 2. Summary of Interface Properties

PROPERTY	1	2	3
Interface Adhesion (psf)	0	0	0
Interface Friction	27	24	25
Change Friction Angle per log cycle normal stress increase	1	1	0
Normal Spring Coeff.	$1x10^8$	$1x10^8$	$1x10^8$
Shear Spring Coeff. (10^3)	17.5	17.5	5.0
Unload Shear Spring (10^3)	26.3	26.3	7.5
Shear Exponent, N	0.8	0.8	0.5
Failure Ratio R_f	0.9	0.9	0.7

1 - Geosynthetic-Soil Interface
2 - Geosynthetic-Geosynthetic Interface
3 - Timber Face-Soil Interface

wall. The hyperbolic parameter values used in the analysis (Table 1) were obtained by graphically fitting them to the triaxial test results for the soil used to construct the wall.

Figure 6 shows the results of laboratory triaxial test results for the sand used in the test wall and the computed results from the hyperbolic model parameters used for the analysis.

Interface Properties

A hyperbolic representation similar to that used for the soil was used for the interface elements. The hyperbolic parameters of the geosynthetic-soil interface were developed from the direct shear test results. The pullout box test results were not used in the formulation of the interface properties, as it was felt that the size of the box was too small to provide meaningful results. Table 2 lists the interface properties used in the F.E.M. model. Figure 7 shows the results of laboratory direct shear test (geotextile to soil) and the computed results from the hyperbolic model.

The properties for the geotextile-geotextile interface and the gravel-geotextile interface were estimated as there was no test information available. The properties used for the timber face-soil interface were based on previous work (Collin, 1986) and are also listed in Table 2.

Structural Elements

The wall face, a combination of 4" x 5 1/2" timber beams and a 1/2" sheet of plywood, was modeled as linear-elastic beam elements with a flexural stiffness (EI) of 25,000 pounds inches squared per inch. This value was obtained from face beam load tests provided for the symposium.

The geosynthetic reinforcement was modeled using linear-elastic bar elements which had axial (tensile) but no flexural stiffness. An axial stiffness (EA) of 400 pounds per inch was used. This value is appropriate for modeling the load-deformation behavior of the geosynthetic reinforcement used in the test walls up to a strain of approximately 5%.

PREDICTION AT END OF CONSTRUCTION

The finite element analysis at the end of construction indicates that the stress and strains within the reinforced soil mass are within the working stress limits for the materials used to construct the wall. Figure 8 shows the predicted strain in the geotextile vs. distance from the face of the wall. The predicted strains are on the order of 1 percent, as would be expected for a polymer reinforced soil wall. It is interesting to note that the strains in the 3 foot

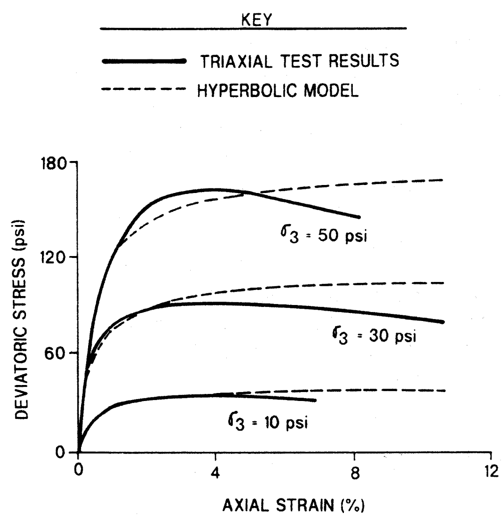

Figure 6. Triaxial Test Results - Actual vs. FEM Model

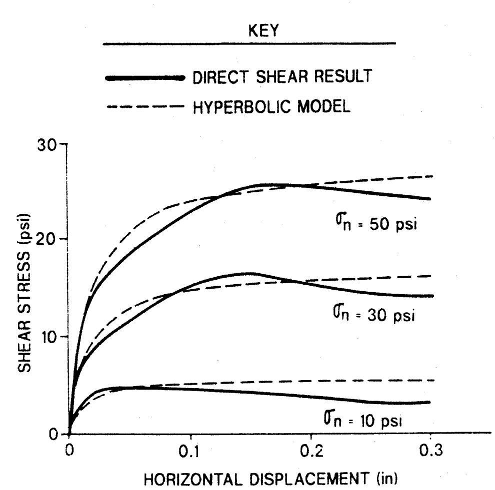

Figure 7. Direct Shear Test Geotextile-Soil - Actual vs. FEM Model

long overlap section of geotextile mirror the strains in the same portion of the 5 foot, 6 inch long reinforcement.

The movement of the wall face has been plotted two ways; first to show the cumulative displacement of the face at the end of construction; and secondly to show the actual displacement of each timber member after its placement. Once again, as might be expected under working stress conditions, the maximum predicted deflections are approximately 1 percent of the height of the wall or approximately 1 inch.

The predicted earth pressures with respect to depth below the top of the wall are also shown on Figure 8. The dashed line represents the computed lateral earth pressure based on an active condition. The predicted earth pressure is approximately 2/3 of the active earth pressure. This phenomenon has been observed in previous work (Collin, 1986) for reinforced soil walls, reinforced with relatively flexible tensile members, and may be due to an apparent decrease in vertical soil pressure at the face of the wall because of friction between the soil and face.

PREDICTION AT FAILURE

Prior to predicting the surcharge load required to cause failure of the reinforced soil wall, failure must be defined. Is failure the total collapse of the wall, the rupture of a reinforcement, a quantified amount of wall displacement, or is some other characteristic behavior more appropriate? The authors have chosen, based on past experience from centrifuge model test (Mitchell et al., 1988), to define failure as the appearance of a well defined failure surface within the reinforced soil mass. Such a definition of failure is particularly applicable to walls reinforced with highly deformable reinforcements such as geotextiles or geogrids.

The finite element code used for this analysis is not a good tool for predicting the failure mechanism of a reinforced soil wall or the development of a failure surface because of the inherent limitations of the soil model and its applicability for "small deformations" only. However, it may give some indication of the surcharge load at which failure occurs by exhibiting a large number of failed soil elements.

The predicted failure mechanism of the test wall is based on observations made on centrifuge model tests loaded to failure by Mitchell et al. (1988). Due to the rigidity of the face and the deformability of the reinforcements, the failure mode is expected to be a rotation of the face around the bottom of the wall and the appearance of a well-defined failure surface well before collapse of the wall. The predicted failure surface (Figure 9) is expected to be inclined at an angle equal to, or slightly larger than, Rankine's active angle for the backfill soil, namely 45 + $\phi/2$ (approximately 65 degrees).

The failure surcharge load is estimated using a limit equilibrium method based on one developed by Jaber and Mitchell (1991). This method is based on moment equilibrium at incipient failure, assuming that the wall has moved enough to develop active earth pressure conditions for the full height of the wall, and that significant redistribution of stress between reinforcements occurs shortly before failure resulting in almost equal tensile contribution from the reinforcements at all levels of the wall, as observed in centrifuge tests. The moment equilibrium calculation to estimate the failure surcharge load is described below:

Active Moment: $M_A = K_a \gamma \frac{h^2}{2} \frac{h}{3} + K_a q_f h \frac{h}{2}$

where: K_a = active earth pressure coefficient
= $\tan^2 (45 - \phi/2) = 0.238$
γ = unit weight of soil = 107 pcf
h = height of wall = 120 in.
q_f = surcharge load at failure

Resisting Moment: $M_R = N R_T \frac{h}{2}$

where: N = Number of Reinforcements
= 21 (including both overlaps)
R_T = Tensile resistance of reinforcements at "failure"
h = height of wall

At "failure", $M_A = M_R$ yields: $q_f = \frac{NR_T}{K_a h} - \frac{\gamma h}{3}$

The failure surcharge load, q_f, depends on R_T, the tensile resistance of the reinforcements at failure. For R_T = 20 to 25 pounds per inch (i.e. the geotextile tensile resistance at 10 to 15 percent strain), q_f = 13 to 16 pounds per square inch. This failure load is confirmed in the finite element analysis by an abundance of failed soil elements within this load range.

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