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Finite element prediction of the performance of an instrumented geotextile-reinforced wall

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ABSTRACT: A geotextile-reinforced retaining wall 12.6 m high with a surcharge fill more than 5 m in height was constructed in Seattle, Washington. As the wall was higher than previously constructed structures of its type, it was extensively instrumented. Nonlinear finite element analyses were made to compare the numerically predicted wall response before and after surcharge placement with the measured behavior. Extra care was required in determining the appropriate mesh layout, material parameters, and loading sequence. In-situ geotextile moduli were back calculated by matching available instrumentation records with numerically obtained results. Confined geotextile stiffness was found to be two to four times greater than the value determined from unconfined tests. Predicted reinforcement force distributions, lateral wall displacements, and stress distribution in soil mass agreed well with instrumentation records. The numerically calculated soil stresses and reinforcement tensions provide insights into the mechanisms that dominated the wall behavior.

1 INTRODUCTION

There is now considerable experience in using approximate methods of analysis and design of reinforced soil walls. In conjunction with the normally adopted soil properties and safety factors, current design guidelines (e.g., Christopher et al., 1990) generally provide safe structures and acceptably small deformations under working stress conditions. These methods, however, were developed for walls of simple geometry. Consequently, the wide range of wall geometry, facings, backfill materials, and reinforcement characteristics now being used may result in a wall behavior different from that assumed in the design.

The construction and monitoring of a large number of full-scale field test walls could resolve many uncertainties, particularly when requirements dictate wall structures with nonstandard geometry or loadings that fall outside the range considered by the empirical design methods. However, instrumenting and adequately monitoring full-scale wall tests is costly.

Numerical simulations are an alternative for predicting the behavior of nonstandard projects, provided field test data are available for validation of parameters and procedures used in the analyses. A rational approach involves initial interpretation of instrumentation results, subsequent validation of the numerical model against the field data, and then numerical simulation of new design aspects. The purpose of this paper is to describe the validation of the finite element (FE) representation of a soil structure having two distinctive characteristics: extensible geotextile reinforcements, and a sloping backfill on top of the wall.

Although a number of successful FE analyses of metallic- and geogrid-reinforced soil retaining walls have been validated against field records, this is not the case for the more flexible geotextile-reinforced structures. A review by Yako and Christopher (1987) identified approximately 200 reinforced walls and slopes which had been constructed in North America using polymeric reinforcements. The number has certainly grown significantly since then. However, of the reviewed projects, only 13 had well-documented instrumentation. Of those, only five provided stress-strain information, and these were all geogrid-reinforced structures. Consequently, much of the field experience to date has provided only qualitative assessment of the design variables in geotextile-reinforced structures, and quantitative data is needed to substantiate design modifications. The FE analysis of the geotextile-reinforced wall presented in this study, referred to as the Rainier Ave. wall, adds quantitative information to the existing instrumentation records and provides a calibrated modeling procedure for future parametric studies.

Sloping backfills behind geosynthetically reinforced soil walls are common for projects with limited working area. The state-of-practice for design of soil walls reinforced with extensible inclusions has been to consider a Rankine failure surface as the locus of maximum tensile forces (Mitchell and Christopher, 1990). The same potential failure surface has been considered for the design of reinforced soil walls with both level and sloping backfills (Fig.1). The anchorage length for pullout resistance verification is taken as the reinforcement length behind this surface. Consequently, the correct identification of the potential failure surface has major implications on the verification of the wall internal stability. The location of the potential failure surface in the Rainier Ave. wall, before and after surcharge application, was investigated using the numerically obtained soil stress distribution and reinforcement forces.

After describing the wall and its instrumentation, the characteristics of the FE modeling (geometry definition, soil and reinforcement material parameters, incremental sequence of analysis) are presented. Finally, results from the analysis,

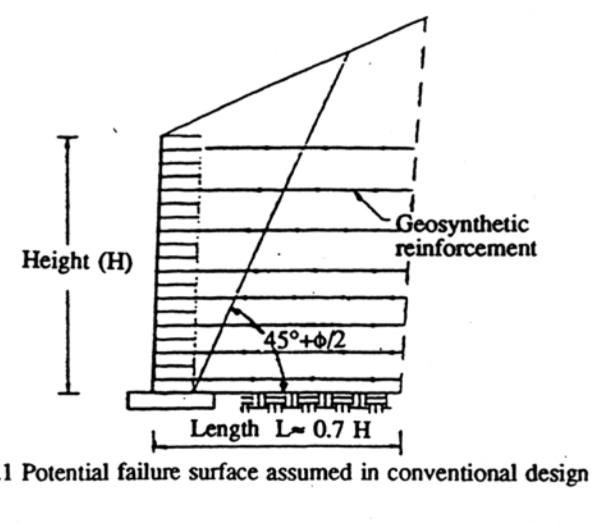

Fig.1 Potential failure surface assumed in conventional design

interpretation of mechanisms that dominated the wall behavior, and location of the potential failure surface are addressed.

2 FINITE ELEMENT ANALYSIS OF REINFORCED SOIL STRUCTURES

The FE analyses in this study were done using the code SSCOMP developed originally by Seed and Duncan (1984), and subsequently modified by Collin (1986) and Jaber (1989) for analysis of reinforced soil structures. Additional modifications were implemented for the purposes of this study. SSCOMP is a general, plane strain, soil-structure interaction program for static analyses of geotechnical structures including consideration of compaction-induced stresses and deformations.

Nonlinear stress-strain and volumetric strain behavior of soil is modeled in SSCOMP using the hyperbolic formulation proposed by Duncan et al. (1980). The program allows the modeling of compaction induced stresses using a hysteretic model for stresses resulting from cyclic loading under condition of no lateral deformations. Reinforcements are modeled using elastic bar elements. Soil-structure interaction is modeled using interface elements capable of transferring shear stresses between the soil and the reinforcement. The revised code used in this study is a UNIX version that handles problems with a larger number of degrees of freedom than previous versions, generates output files for postprocessing of results, and deals slightly differently with soil elements in tension. SSCOMP has been used successfully by previous investigators to predict the behavior of large model walls in centrifuge tests and full-scale instrumented reinforced soil walls. A review of the reinforced soil structures already modeled using SSCOMP and validated against instrumentation records is presented by Zornberg and Mitchell (1993).

A number of FE analyses of soil structures reinforced with geosynthetics (mainly with geogrids) have been reported in the literature. Some of these studies investigate the performance of hypothetical reinforced soil structures (e.g., Chalaturnyk et al., 1990; Ho and Rowe, 1993), while others validate the FE results for geogrid-reinforced structures against field records (e.g., Adib, 1988; Bathurst et al., 1992). For the more flexible geotextile-reinforced structures, only a limited number of FE studies were identified in the technical literature (e.g., Adib, 1988). Considering this limited modeling experience, extra care was required for the determination of appropriate mesh layout, material parameters, and analysis sequence for the geotextile-reinforced structure under study.

The FE modeling of soil structures reinforced with extensible reinforcements differs from the modeling of structures with inextensible reinforcements. Some modeling aspects are actually simpler in the first case:

- Geosynthetic reinforcements are generally continuous sheet layers and, consequently, they can be naturally assumed as plane strain structural elements. This is generally not the case for inextensible reinforcements, such as steel strips or bar mats, which have a three-dimensional layout.
- While rigid facing structures can play an important role in the overall wall behavior, in the case of more flexible structures the wall response is dominated by soil and reinforcement characteristics. The selection of facing parameters in these analyses is consequently less important.

On the other hand, several modeling aspects are more complex in the FE analysis of geosynthetically reinforced soil structures:

- The effect of confinement and time on the tensile strength and the stiffness of geosynthetics is not fully understood. The available information generally consists of results from wide width tensile tests. These data, obtained by testing unconfined geosynthetic samples under high strain rates, are not representative of in-service conditions.
- Since geosynthetically reinforced soil structures generally have smaller reinforcement vertical spacing than structures reinforced with inextensible inclusions, a higher mesh discretization is required. Moreover, a fine mesh should be used to represent each compacted soil layer due to the high interface friction between soil and geosynthetics.
- FE results of geotextile walls are more sensitive to the stress-strain-strength behavior of the backfill soil than in the case of structures reinforced with inextensible reinforcements. The stiffer reinforcements and facing structures used in those structures dominate the wall response behavior.
- Analytical procedures were developed to estimate compaction-induced earth pressures for situations in which geostatic soil stresses represent conditions of no lateral deformation (Seed and Duncan, 1984). However, the ability of this model to simulate compaction-induced pressures on soil structures with extensible inclusions and flexible facing requires additional investigation.
- The backfill material undergoes larger lateral deformations in geosynthetically reinforced soil walls than in structures reinforced with inextensible elements. Consequently, zones of highly mobilized shear stresses and, possibly, of failed elements may develop. Although the accuracy of the soil behavior modeling may be compromised, the presence of highly mobilized shear stresses is not so adverse as in nonreinforced earth structures. This is because, zones of highly mobilized shear stresses do not imply a failure mechanism if ultimate tensile strength has not been reached also in the reinforcements.

3 THE RAINIER AVENUE WALL

To provide for a preload fill in an area of limited right-of-way, the Washington State Department of Transportation designed and supervised the construction of a 12.6 m high geotextile-reinforced retaining wall. As the wall was higher than any geotextile-reinforced wall built previously and supported a 5.3 m high surcharge fill, an extensive instrumentation program was developed to evaluate its performance (Christopher et al., 1990; Allen et al., 1991). The objectives of the monitoring program were to observe the stress and strain distributions within the reinforced soil wall, and to evaluate the wall response due to the inclined surcharge fill.

Reinforcement requirements for the Rainier Ave. wall were determined based on a conventional tieback wedge analysis (Mitchell and Christopher, 1990), and a reinforcement spacing of 0.38 m was adopted. The specified geotextile strength was varied with the height of the wall to more closely match theoretical design strength requirements. Accordingly, four different polypropylene slit film woven and polyester multifilament woven geotextiles were selected as reinforcements.

Bonded resistance strain gauges and mechanical extensometers were installed in the geotextiles to evaluate the strain distribution as well as the location and magnitude of maximum tensile stress. Differences were observed between the strain gauge and the extensometer records. Maximum geotextile strains obtained from strain gauge measurements were approximately 0.5%, while maximum strains measured by the extensometers were on the order of 0.7 to 1.0%. The extensometers incorporate strain occurring in the geotextile macrostructure, including local effects such as creases and folds. Additionally, since the extensometers were not rigidly fixed to the fabric, but were only wired to the geotextile, it was possible for the extensometer to move relative to the geotextile (Christopher et al., 1990; Allen et al., 1991). Based on these considerations, only strain gauge records were considered in this study. Nevertheless, since the glue used to fix the gauges is often stiff relative to the geotextile, measured strains are expected to be lower than the actual field strains.

Inclinometer tubes were installed at the face of the wall, within the reinforced soil section, and behind the reinforced section to monitor the horizontal movement of the wall. Optical and photogrammetric surveys of the wall face were also made during and after construction. Finally, vertical stresses beneath the wall were measured using Glötzl stress cells.

4 MODELING CHARACTERISTICS

The FE mesh for the analysis of the Rainier Ave. wall was established based on a sensitivity study of the mesh discretization, on the need for retrieving numerical results at the location of instrumentation devices, and on practical limitations of program capacity and run times. The FE mesh selected for the final analysis consisted of 1698 nodes, 1661 plane strain elements for soil representation, and 561 bar elements for simulation of the reinforcements.

Mesh discretization between reinforcement layers was found essential for the proper representation of the soil layer behavior. As contact efficiency between geotextiles and soil is high, displacement compatibility between soil and reinforcement elements is a reasonable assumption. Even during pullout, failure would probably occur within the soil mass and not along the soil/reinforcement interface. Consequently, high discretization of the backfill soil between two reinforcement layers was used in the analysis.

One of the most important parameters to be selected in a FE analysis of a reinforced soil wall is the in-situ tensile stiffness of the geotextiles. However, methods commonly used for determination of geosynthetics' mechanical properties do not replicate the operational conditions of the geotextile in the field. Results from unconfined wide width strength tests, available for the geotextiles used in the structure under study, may grossly underestimate the in-situ tensile stiffness. A significant increase in stiffness and strength develops in geotextiles when they are confined by the soil. Unfortunately, it is essentially impossible to determine the in-situ stiffness directly from the instrumentation data. However, numerical back calculation offers an alternative for this determination, and the in-situ geotextile stiffness was estimated in this way for this study. The unconfined modulus values at 5% strain obtained from wide width strength tests are summarized in Table 1. These moduli are the lower bound for the in-situ stiffness values of the geotextiles used in the wall under study. Additionally, geotextile stiffness values were estimated based on the results of pullout tests (Zornberg and Mitchell, 1993). Stiffness values from pullout testing provided upper reference values for geotextile moduli under working stress conditions.

Table 1. Geotextile parameters

Distance from top of wall	Geotextile type	*a)* Wide width strength	*b)* Strain at peak tension	*c)* Modulus at 5%strain	*d)* Back calculated Stiffness	Ratio *(d)/(c)*
0-3	Polypropylene (PP) slit film woven	31	21	198	438	2.21
3-6	PP stitch-bonded (2 layers) woven	62	16	453	1237	2.73
6-9	PP stitch-bonded (3 layers) woven	92	17	662	2767	4.18
9-12	Polyester multi-filament woven	186	18	1068	3791	3.55

The stress-strain-strength characterization ot the backfill soil plays a more relevant role in geosynthetically reinforced walls than in stiffer metallic-reinforced structures. Therefore, special care was taken in estimating model parameters for the backfill material. The soil constitutive relationship used for the analysis is a modified version of the hyperbolic model proposed by Duncan, et al. (1980). Soil material properties at any increment during the FE analysis are based on the current stress state and the previous stress history of each element. The nonlinear, stress-dependent model assumes that stress-strain curves for soils can be approximated by hyperbolas as shown in Fig.2. Calibration of the backfill soil was performed using data from triaxial tests. Obtained parameters are presented in Table 2. The results of two triaxial test series were available, and comparable values were obtained using each series separately, showing that the soil samples were representative. Model predictions using parameters obtained from the calibration were able to capture both the pre-failure stress strain behavior and the failure stress level in each test (Fig.2). The good representation of the deviatoric stress-strain behavior by the hyperbolic model is evident.

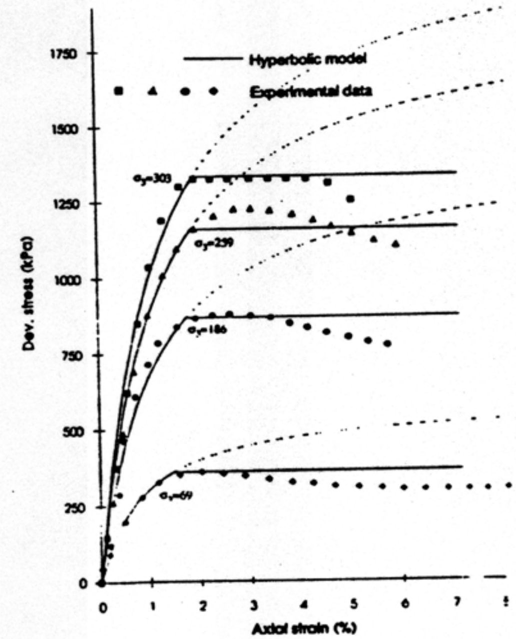

Fig.2 Hyperbolic representation of backfill soil

Table 2. Hyperbolic soil parameters

Parameter	Parameter definition	Backfill	Foundation
K	Young's modulus coeff.	913	1900
n	Young's modulus exponent	0.6	0.25
R_f	Failure ratio	0.64	0.7
c (kPa)	Cohesion	0.0	48.9
ϕ_0 (°)	Friction angle at 1 atm.	46.1	40.0
$\Delta\phi$ (°)	Friction angle reduction param.	5.3	0.0
K_B	Bulk modulus number	250	450
m	Bulk modulus exponent	0.8	0.0
K_{ur}	Unload-reload modulus coeff.	1485	2850
γ (kN/m^3)	Unit weight	21.1	20.5
K_0	At-rest lateral earth press. coeff.	0.35	0.41

Volume change data are required for determination of bulk modulus. However, since the compacted soils were tested in unsaturated conditions, volume change information was not available. Thus, bulk modulus parameters determined by Boscardin et al. (1990) for a similar compacted granular material were used. Hyperbolic parameters for the foundation soil, also listed in Table 2, were estimated from a review prepared by Duncan et al. (1980). The sensitivity of the FE results to the selected foundation properties showed that they only have a minor effect on the results.

SSCOMP incorporates zero thickness interface elements capable of representing soil/structure interface conditions by modeling the relative movement between soil and structure. This element is made up of a normal spring and a nonlinear stress dependent shear spring. However, the use of these elements incorporates additional degrees of freedom to the analysis. Since a parametric study showed that the use of interface elements had only minor influence on the results, the final study was done without interface elements. The assumption of displacement compatibility between soil and reinforcements is justified by the high interface shear strength of geotextiles and by the fact that geotextiles can tolerate the same large strains as soils prior to failure.

Beam elements have been used previously to simulate the facing of reinforced soil structures. Only minor influence was observed in the results if beam elements are used to represent the flexible face. Based on this parametric study, no beam elements were considered in the final analysis of the Rainier Ave. wall.

Non-linear and stress dependent material properties are modeled in SSCOMP by using an incremental analysis procedure that follows the actual construction sequence of the earth structure. Selection of the number of analysis increments is a trade off between improved representation of non-linear stress-dependent modulus values and increased computer time. After a study of sensitivity, the placement of soil layers 0.38 m thick was adopted. Sloping backfill surcharge loads were modeled by applying equivalent distributed loads on the top of the wall.

5 RESULTS FROM THE ANALYSIS

As indicated, selected modeling parameters and procedures for the wall under study were evaluated by studies of sensitivity of the results. The final step in the calibration process consisted on selecting appropriate values of in-situ stiffness for the different geotextiles used in the wall. The in-situ moduli were selected based on the agreement observed between the numerical results and the instrumentation records of the Rainier Ave. wall. Back calculated confined geotextile stiffness values obtained after the calibration process are shown in Table 1.

Ratios between the numerically back calculated and the experimental unconfined stiffness values are also shown in Table 1. For the reasons noted earlier, the back calculated values are higher than the values obtained from wide width testing. The increase is expected to be dependent on both the material type of the woven geotextiles and on the in-situ confining pressures. For polypropylene materials, confined stiffness increases from roughly twice the unconfined value in the upper reinforced zone (0 to 3 m from the top of the wall) to roughly four times the unconfined value in the third zone (6 to 9 m). The increase in the only polyester material used as reinforcement (in the zone from 9 to 12 m from top of the wall) is less than four times the unconfined modulus. Due to differences between field and laboratory conditions (field construction damage, geotextile degradation, representativity of laboratory tests) back calculated in-situ moduli are expected to be lower than the values from laboratory pullout tests. Back calculated moduli are approximately 65% the value obtained from pullout testing (Zornberg and Mitchell, 1993).

Reinforcement tensions predicted from the FE analysis are shown in Fig.3, where they are compared with tension distributions obtained from strain gauge measurements. Since these field measurements represent a lower bound of the actual strains, in-situ reinforcement stiffness was selected so that predicted values represent an upper envelope (and not an average) of field strain records. The matching is good, and both numerical and field results reflect a similar tension increase after surcharge placement.

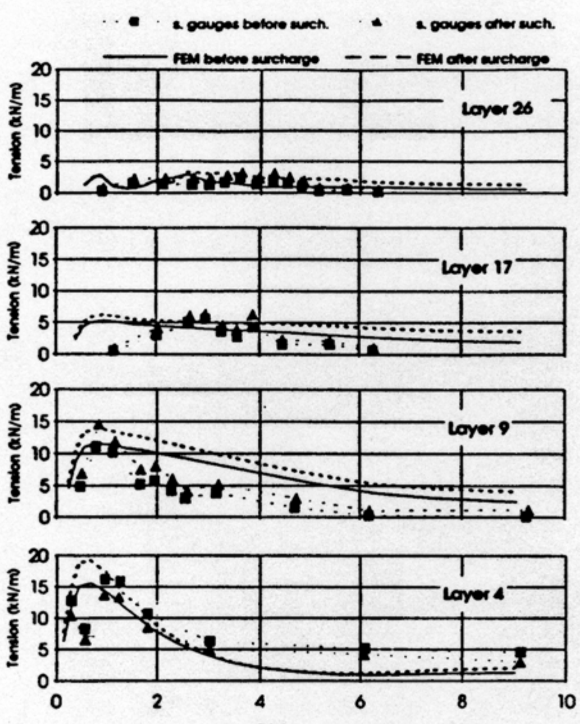

Fig. 3 Tension distribution along geotextile reinforcements

An inclinometer tube was installed within the reinforced section 2.7 m behind the wall face. Measurements from this inclinometer are considered the most reliable records available for lateral displacements. Lateral deformations at the location of the inclinometer within the reinforced section are presented in Fig.4. Displacements at each nodal point represent total displacements relative to the initial position of the node. The agreement between numerically obtained displacements and inclinometer measurements is very good. Both numerical results and field values show a lateral displacement increase of roughly 2.5 cm caused by the surcharge. The good agreement between numerical results and the different instrumented responses of the wall supports the selection of parameters and procedures made for the analysis.

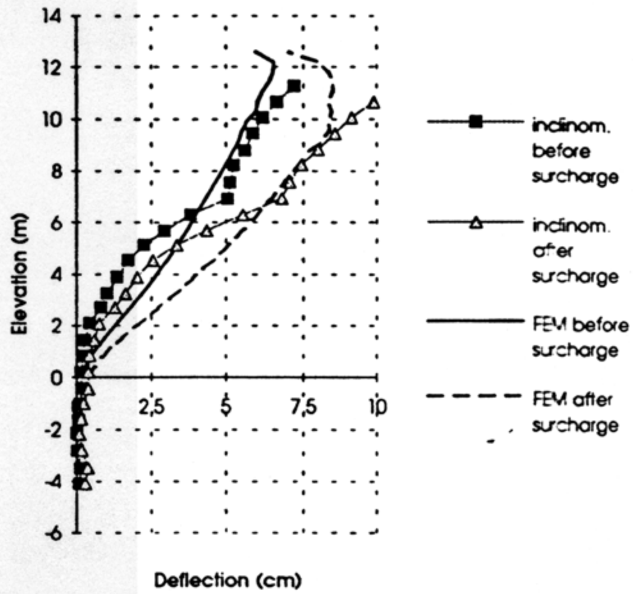

Fig.4 Lateral displacements at the location of the inclinometer

Vertical stresses beneath the wall measured using Glötzl stress cells showed that stresses at the toe were approximately 20% higher than in the middle and back of the wall. However, these measured vertical stresses were inconsistently lower than the average overburden pressure, both before and after placement of the sloping backfill surcharge (Allen et al., 1991). Measured vertical stresses are shown in Fig.5. Vertical stresses predicted by the FE analysis and, as a reference, theoretical average overburden pressures before and after the surcharge are also indicated. The numerical results show a pattern very similar to the distribution obtained from cell pressures. Moreover, predicted values are consistent with the average overburden pressure at both construction stages. The vertical pressures measured by the stress cells were shifted until the average measured vertical stress matched the average overburden pressure. The agreement between these corrected field measurements and the numerical prediction is very good. This suggests that the FE solution correctly represents vertical soil stress distributions, even for walls higher than conventional.

Predicted vertical stresses in the reinforced soil mass were lower than average overburden stresses both near the structure facing and at the beginning of the retained soil. On the other hand, predicted vertical stresses in the reinforced zone away from the facing were higher than overburden pressures, suggesting that frictional resistance along reinforcement anchorage length is higher than that calculated assuming overburden pressures. The numerical results also showed that just behind the reinforced zone predicted horizontal stresses

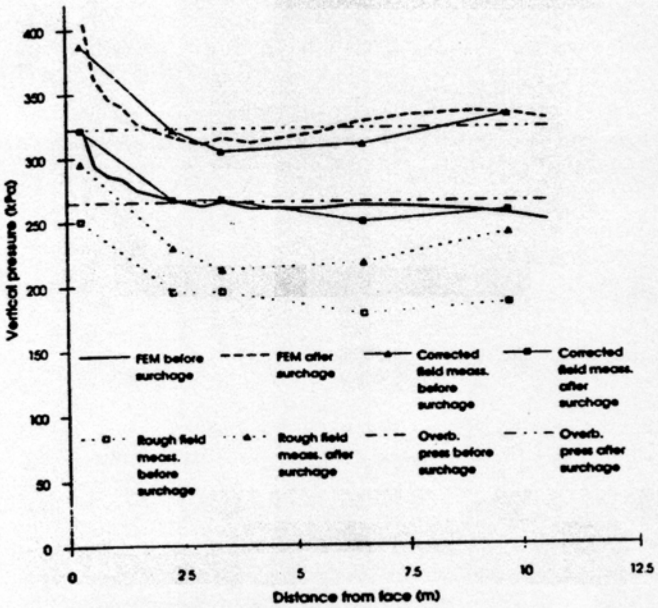

Fig.5 Predicted and measured vertical stresses

were in agreement with pressures estimated by the active Rankine coefficient. Mobilized shear stresses near the facing were high due to the large lateral displacements undergone by the reinforced soil mass. However, the presence of zones with highly mobilized shear stresses did not cause an imminent state of failure. The complete distribution of vertical stresses, lateral earth pressure coefficients, and mobilized shear stresses is presented by Zornberg and Mitchell (1993).

The location of the maximum geotextile tension defines the beginning of reinforcement anchorage length needed to satisfy pullout requirements. The locus of the maximum reinforcement tension, obtained numerically at each geotextile layer, is shown in Fig.6. Results obtained before and after placement of the sloping backfill surcharge are indicated in the figure. The Rankine line, which is the conventionally assumed locus for walls with geosynthetic reinforcements, is also indicated. Two observations can be made based on the observed distribution: a) although there is some scattering near the top of the wall, no tension is observed after the surcharge is placed, and b) the Rankine line is a conservative approximation for the actual position of the maximum reinforcement tensions; i.e., the actual maximum tension locus is inside the plane defined by an angle of $45°+\phi/2$ from the horizontal.

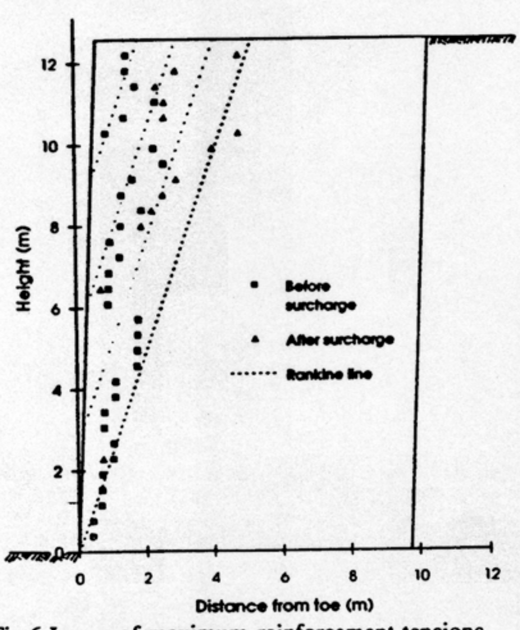

Fig.6 Locus of maximum reinforcement tensions.

Reinforced soil structures with sloping backfill surcharges are generally designed assuming the same design failure surface, independently of the presence of a slope surcharge behind the wall. As shown in Fig.6, numerical results obtained for the Rainier Ave. wall verify that actual failure surface is inside the Rankine plane, both before and after surcharge placement. This locus does not show, however, a smooth configuration. An interpretation of the mechanism that may have originated the observed pattern is suggested with dotted lines in Fig.6. Since the Rainier Ave. wall was designed with four vertical zones of different reinforcement strength, each zone can be interpreted as a composite material placed on a stiffer base. Different potential failure surfaces appear to develop at the interface between the zones. This pattern is favorable to pullout safety, resulting in a steep composite maximum reinforcement tension line. Walls with several vertical reinforced zones may then be especially advantageous for projects in which the design is conditioned by pullout requirements.

The magnitude of maximum reinforcement tension with depth is indicated in Fig.7. The straight lines in the figure represent reinforcement tensions estimated using the active Rankine coefficient, the active coefficient calculated using Coulomb approximation for infinite slope, and the at-rest lateral earth pressure coefficient. The predicted maximum tensions for the horizontal backfill (no surcharge) case match reasonably well the Rankine line, while tension values for the sloping surcharge case are conservatively estimated by the active coefficient for infinite slope. The sudden changes observed in the maximum tensions correspond to the boundaries between different reinforced zones.

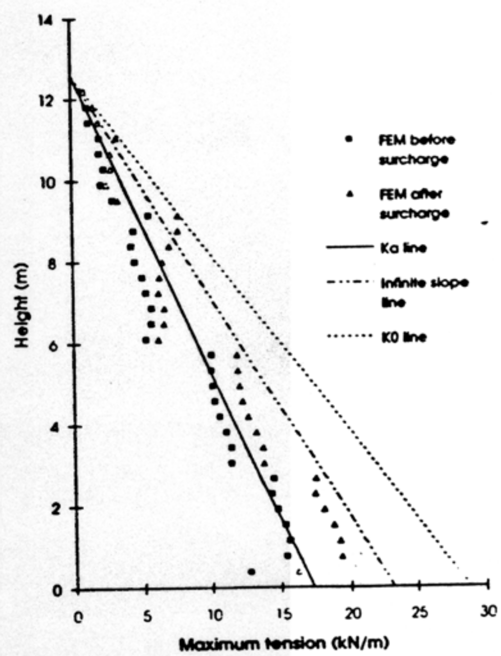

Fig.7 Maximum reinforcement tension versus depth

6 LOCATION OF THE CRITICAL PLANAR SURFACE

The most critical potential slip surface in a reinforced soil wall is assumed to coincide with the locus of maximum tension forces in the reinforcements. This locus has been assumed to be linear in structures with extensible reinforcements. A parametric study on the location of the potential failure surface would also be simplified if the failure surface is assumed to be planar. However, such a planar failure surface is difficult to define from the data observed in Fig.6. A systematic methodology is then used to determine this critical plane.

The location of potential failure surfaces in nonreinforced soil structures has already been investigated using FE analyses (e.g., Duncan, 1992). In these studies, the numerically predicted shear stresses along a trial surface are compared to the ultimate shear stresses available along that surface. This approach can be extended to reinforced soil structures to investigate planar trial failure surfaces at a range of angles (β) from the horizontal.

The Reinforcement Tension Summation (RTS) is determined by adding the tensions of each reinforcement layer along each trial surface. The value of the Factor of Safety along each trial surface can also be determined using numerically obtained soil stresses and reinforcement tensions. The surface with the maximum RTS is the critical planar surface since, considering simplifying assumptions, it can be formally demonstrated that the plane with a minimum Factor of Safety corresponds to the surface with a maximum RTS (Zornberg and Mitchell, 1993).

The critical surfaces, before and after surcharge placement, obtained after the search process are shown in Fig.8. RTS values at each trial plane are also indicated in the figure. The results verify that the anchorage length for pullout safety can be conservatively estimated, both before and after surcharge, using a potential failure surface defined by the theoretical Rankine line.

Using calibrated input parameters and modeling procedures obtained from the analysis of the Rainier Ave. wall, a parametric study was done to investigate the effect of sloping backfill surcharges on the performance of geosynthetically reinforced soil walls. Results from this parametric study will be described in a subsequent paper. A Factor of Safety on the order of 3 was calculated for the Rainier Ave. wall along the critical planar surface (Zornberg and Mitchell, 1993). Such a high factor shows that current design procedures for geotextile-reinforced structures are conservative.

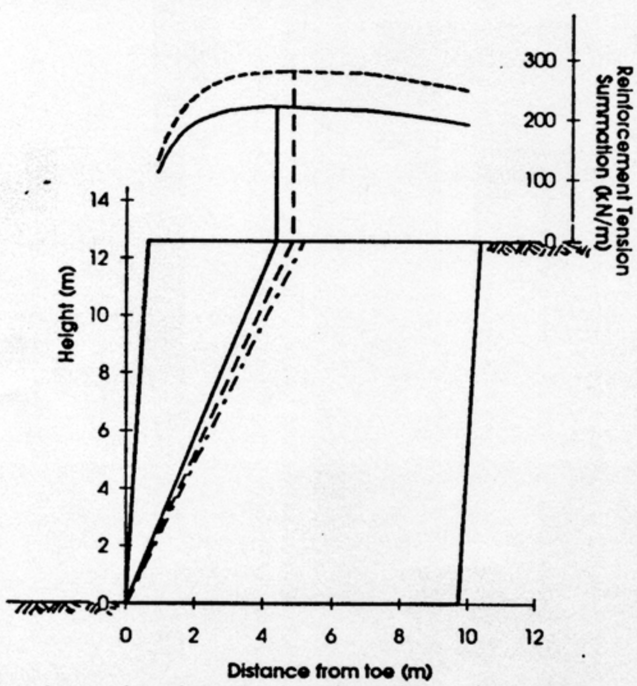

Fig.8 Location of the planar surface with maximum RTS

7 CONCLUSIONS

The FE analysis of a well instrumented geotextile-reinforced soil wall with a sloping backfill surcharge was performed in this study. The following lessons can be learned:

- The numerical results are in agreement with the different instrumented responses of the wall (geotextile tension distribution, lateral displacements, vertical stresses).
- In-situ geotextile stiffness, back calculated by matching available instrumentation records with FE results, was found to vary from twice to four times the values determined from unconfined wide width tensile tests.
- Numerical results showed that maximum reinforcement forces can be appropriately estimated using the Rankine active coefficient for the horizontal backfill case. The use of the Coulomb active coefficient for infinite backfill slope predicts conservatively the maximum reinforcement forces after surcharge placement.
- The locus of the maximum reinforcement tensions suggests the development of multiple potential failure surfaces when the wall is designed with zones of different reinforcement strengths.
- The locus of the maximum reinforcement tensions for the Rainier Ave. wall, both before and after surcharge placement, is inside the conventionally assumed Rankine line.
- A high factor of safety, calculated along the critical planar surface using reinforcement tensions and soil stresses from the FE analysis, reflects conservatism in the wall design.

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