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PERFORMANCE OF A GEOTEXTILE-REINFORCED SLOPE USING DECOMPOSED GRANITE AS BACKFILL MATERIAL

GEOSSINTÉTICOS 95

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ABSTRACT

As part of a highway widening project, the Federal Highway Administration designed and supervised the construction of a permanent 1H:1V geotextile-reinforced slope 15.3 m high. Several characteristics were unique to the design: the structure was higher than usual geotextile-reinforced slopes, it involved the use of both a high modulus composite and a nonwoven geotextile, and it was constructed using indigenous soils (decomposed granite) as backfill material. The selected geotextiles were required to have not only adequate tensile strength but also appropriate in-plane drainage capacity to allow dissipation of pore water pressures that could be generated in the fill. An extensive program of instrumentation and construction monitoring was implemented to evaluate its performance. After presenting the characteristics of the project, this paper discusses the estimation and analysis of the strains in the geotextiles, monitored using mechanical extensometers attached to the reinforcements. Maximum strains are on the order of 0.2%, which are notably lower than the large geotextile strains at which the design strength would typically be developed.

RESUMO

A Administração Federal de Transportes dos Estados Unidos (FHWA) projetou e supervisou a construção de um aterro permanente de 15.3 m de altura (1H:1V), reforçado com geotêxteis, como parte de um projeto de enlargamento de autoestrada. Várias características foram singulares neste projeto: a estrutura é mais alta do que aterros convencionais reforçados com geotêxteis, envolve o uso de geocompostos de elevada resistência à tração assim como de geotêxteis não tecidos, e foi construida usando solos locais (granito decomposto) como material de aterro. Os geotêxteis foram selecionados não apenas com uma resistência adequada à tração, mas também com uma capacidade de drenagem apropriada para dissipar poro-pressões que venham a gerar-se no aterro. Um programa abrangente de instrumentação e de monitoramento foi implementado para avaliar o comportamento desta estrutura. Após descrever as características do projeto, apresentam-se neste artigo a estimativa e o análise das deformações nos geotêxteis, monitorados através de extensômetros mecânicos sujeitos aos reforços. As deformações máximas, em torno de 0.2%, são notoriamente menores do que as que correspondem à resistência usada no dimensionamento dos reforços.

1. INTRODUCTION

The project consists of a geotextile-reinforced slope designed as part of the widening of U.S. Highway 93 between Salmon, Idaho, and the Montana state line. The reinforced structure is a 1H:1V (45°) slope located in Idaho's Salmon National Forest along Highway 93. Esthetics was an important consideration in the selection of the retaining structures along scenic Highway 93, which has been recognized by a recent article in National Geographic (Parfit, 1992). The 172 m long and up to 15.3 m high geotextile-reinforced slope is vegetated, causing a minimum environmental impact to the Salmon National Forest. This structure, designed by the Western Federal Lands Highway Division, represents one of the highest geotextile-reinforced slopes in the U.S.

The slope was designed using geotextile reinforcements that were required to have not only adequate tensile strength but also appropriate in-plane drainage capacity to allow dissipation of pore water pressures that could be generated in the fill. In this way, an additional drainage systems was not necessary even though indigenous soils were used as backfill and groundwater seeping was expected from the excavation behind the fill. Due to the unique characteristics of this structure, the reinforced slope was considered experimental, and an extensive program of instrumentation and construction monitoring was implemented to evaluate its performance.

Figure 1 is a view of the completed geotextile-reinforced slope after the erosion control matting has been placed. The subgrade was completed in the 1993 summer season and the reinforced slope has performed as intended since then. A considerable amount of instrumentation data has been accumulated during the construction period, and post-construction performance is still being monitored at this writing.

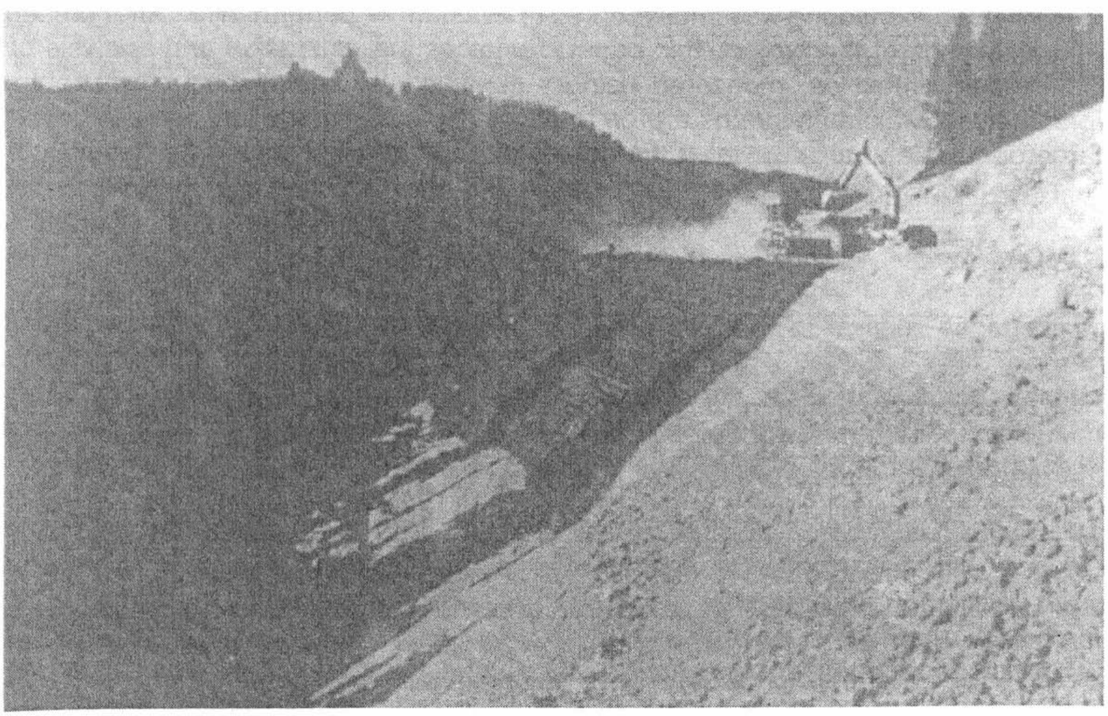

Figure 1. Finished reinforced slope with erosion matting in place.

As part of this instrumentation program, forty-five mechanical extensometers were placed on the geotextiles, two inclinometer tubes were installed to monitor horizontal movements within the reinforced zone, piezometers were installed to evaluate generation and dissipation of pore water pressures, and survey points were used to monitor face movements. After presenting the characteristics of the project and of the instrumentation program, this paper focuses on the interpretation and analysis of monitoring data obtained from extensometers during the construction period. They allowed interpretation of the geotextile strain distribution and evaluation of the location and magnitude of the maximum strains in the reinforcements.

2. DESIGN CONSIDERATIONS

Use of indigenous soils. On-site soil coming from excavation of the road alignment was to be used as backfill material. Subsurface drilling revealed that the majority of subsurface material on this project is granite bedrock that varies from hard, intact rock to highly decomposed, soil-like material. Preconstruction evaluation of the cutbank soil indicated a maximum density of 18 to 21 kN/m^3 and an optimum moisture content of 79.5 to 13.5%, as determined by Standard Proctor tests. Although the project specifications required the use of material with no more than 15% passing U. S. no. 200 sieve, internal drainage was a design concern. This was because of the potential seepage from the fractured rock mass into the reinforced fill, especially during spring thaw, coupled with the potential crushing of decomposed granite particles that may reduce the hydraulic conductivity of the fill. There is strong experimental evidence that permeable geosynthetics can more effectively reinforce indigenous soils (Zornberg and Mitchell, 1994; Mitchell and Zornberg, 1995). Consequently, instead of constructing a separate drainage system, the adopted design was to provide lateral drainage by using reinforcements with appropriate in-plane transmissivity.

Design methodology. Design of the geotextile-reinforced slope, done according to FHWA guidelines, included analysis of the external and internal stability (Christopher et al., 1990). The external stability was evaluated by analyzing the potential for sliding and for overall deep-seated slope failure. Limit equilibrium slope stability methods, adapted for analysis of reinforced slopes, were used to determine the required geotextile layer spacing and reinforcement tensile strength. The total reinforcement length that provides adequate pullout resistance was finally calculated. The selected geotextiles were evaluated by performing product specific creep tests and a construction damage assessment (Wayne and Barrows, 1994). The results were used to develop the partial factors of safety that estimate the geotextile allowable tensile strength.

Reinforcement layout. Widening of the original road was achieved by turning the existing 2H:1V nonreinforced slope into a 1H:1V reinforced slope. The specified geotextile strength was varied with the height of the slope to more closely match theoretical design strength requirements. As shown in Figure 2, the final design adopted two geosynthetically reinforced zones with a constant reinforcement spacing of 0.3 m (1 ft). A high strength composite geotextile was selected in the lower half of the slope, while a nonwoven geotextile was used in the upper half. At the highest cross-section of the structure, the reinforced slope has a total of 50 geotextile layers. Since a detailed subsurface investigation revealed low-strength decomposed granite zones, a reinforced rock shear key was built at the base of the reinforced slope to increase deep-seated global stability.

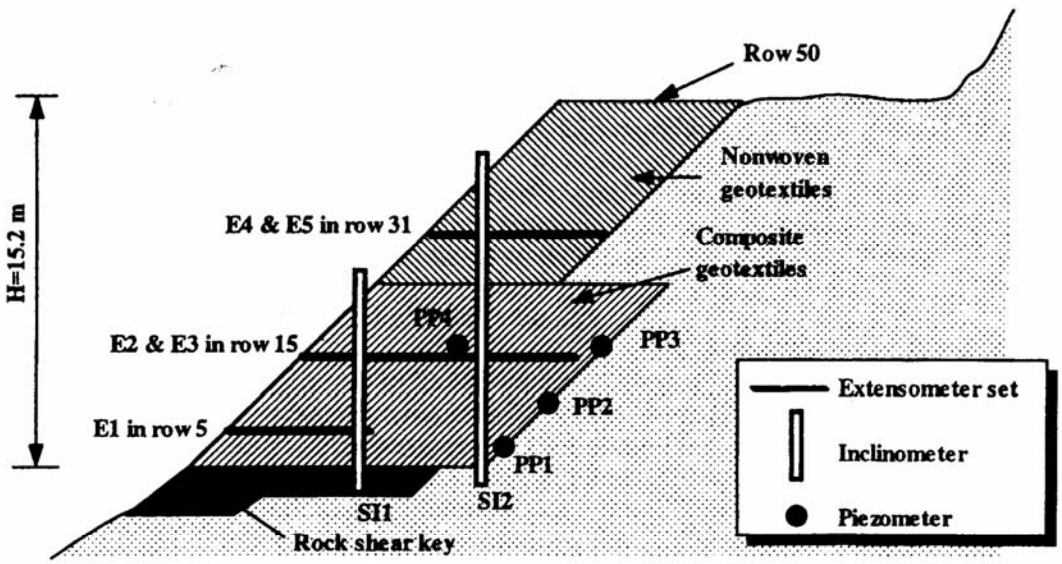

Figure 2. Cross section of the reinforced slope showing the instrumentation layout

Geotextile selection. The decision to use a reinforced soil slope was based on the ease of construction, the anticipated lower cost as compared to more conventional structures, and the reduced environmental impact of this solution. The selected geosynthetic reinforcements were the nonwoven geotextile PP-20 and the composite PPC-100, both manufactured by Polyfelt. The PP-20 material, with an ultimate tensile strength of over 20 kN/m, is a polypropylene continuous filament needle punched nonwoven. The PPC-100, with an ultimate tensile strength over 100 kN/m, is a polypropylene continuous filament nonwoven geotextile reinforced by a biaxial network of high-modulus yarns. Both materials exhibit a typical in-plane hydraulic transmissivity of 0.006 l/s/m under 200 kPa of normal stress. The composite geotextile was chosen for the lower half of the slope given the design need of combining the reinforcing benefits of high-modulus geosynthetics and the hydraulic advantages of nonwovens.

3. INSTRUMENTATION PROGRAM

A comprehensive monitoring program was designed to evaluate the performance of the reinforced soil slope during and after construction. Figure 2 shows the location of the instrumentation used in the monitoring program of the geotextile-reinforced slope. Since most instruments measure conditions at only one point, a large number of measurement points was required to evaluate parameters of interest over the entire section of the structure. Instrument readings were taken during construction of the reinforced slope and continued until approximately eight weeks after the completion of the fill. Observations restarted after the spring thaw to evaluate the long-term performance of the structure.

Forty-five single-point mechanical extensometers were placed on the geotextiles to measure local displacement of the geotextile and to evaluate the strain distribution as well as the location and magnitude of maximum tensile strains. The extensometers consisted of metal rods attached to the geotextile at increasing lengths from the slope, and extended to the front face in a stiff PVC casing to protect them from soil overburden. Figure 3 shows the end bearing plate of a mechanical extensometer already attached to the nonwoven geotextile. The end bearing plates were placed at increasing distances from the wrapped around face (Figure 4), at nominal intervals of 610 mm between them. Relative displacements between the extensometer anchor plate and the slope face were measured to the nearest 0.025 mm (0.001 inch). The extensometers were concentrated in the area of the predicted potential failure surface, as defined by the limit equilibrium analysis used in the structure design. As indicated in Figure 2, the extensometers were mounted on the composite geotextile layers 5 and 15, and on the nonwoven geotextile layer 31, located at elevations 1.22 m, 4.27 m, and 9.14 m. Extensometer sets were installed at two parallel cross-sections of the reinforced structure in order to provide sufficient redundancy to explain possible anomalous data as well as to account for possible damages of some instruments during construction. Extensometer sets E1 (with five single-point extensometers), and sets E2 and E4 (ten extensometer each), were installed in one of the instrumented cross-sections. Extensometer sets E3 and E5, analogous to E2 and E4, were additionally installed in a parallel cross-section. The provision of considering redundant instruments proved to be crucial to the success of the instrumentation program, as several single-point extensometers in set E5 were lost during compaction operations.

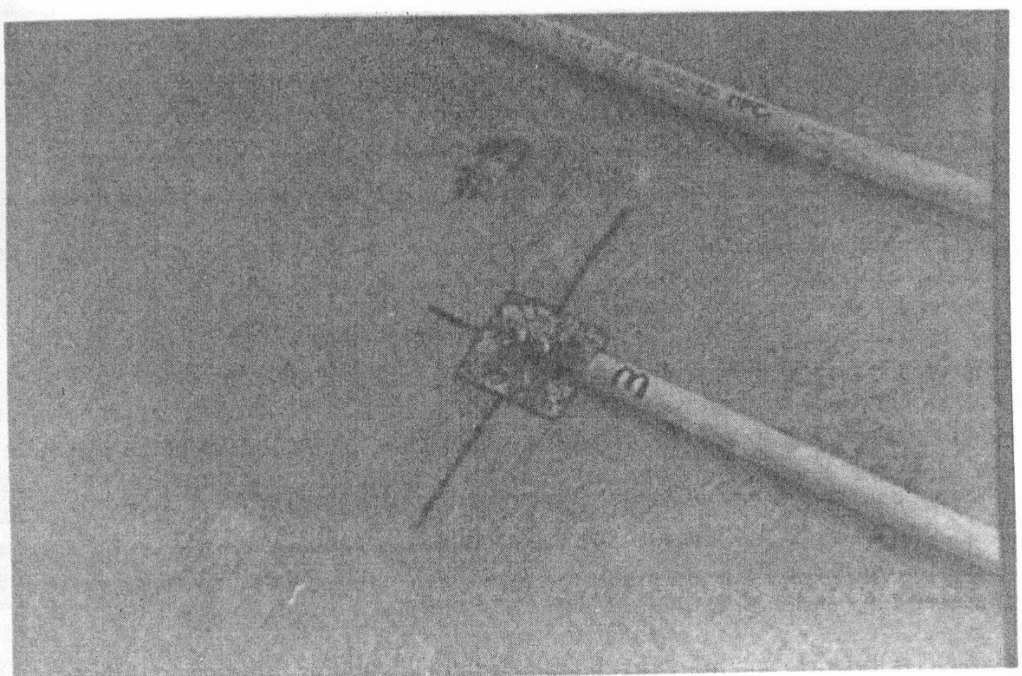

Figure 3. End bearing plate of a mechanical extensometer attached to a geotextile.

Also as part of the instrumentation program, two inclinometer tubes were installed to monitor horizontal movements within the reinforced zone both during and after construction. These inclinometers were installed at 7.3 m and 11.9 m from the toe of the reinforced slope, and daylighted on top of geotextile rows 24 and 39 respectively. Movements of the slope face were monitored by survey points located in four vertical rows in the vicinity of the instruments. Each survey point consisted of a short piece of rebar embedded between two reinforcing layers. Additionally, four electronic piezometers were installed to evaluate generation and dissipation of pore water pressures that could develop either during construction or after rainfall events. Groundwater seepage is expected from the excavation behind the fill, mainly during the spring runoff.

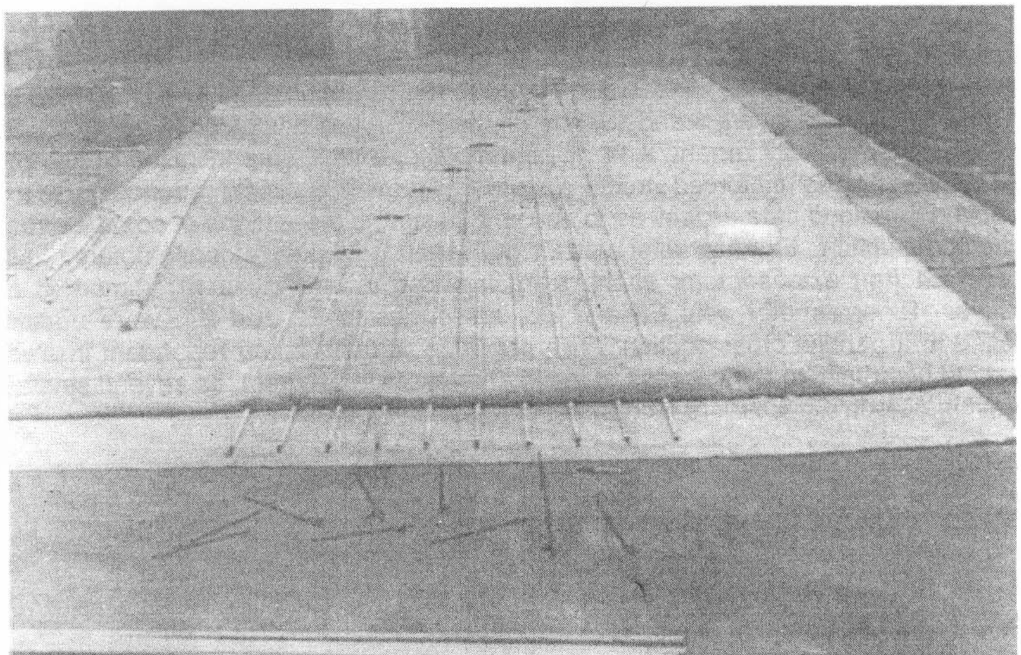

Figure 4. Geotextile layer instrumented with extensometers ready for placement in the field.

4. MONITORED STRAINS IN THE GEOTEXTILES

Important results have already been collected from the different instruments installed in the reinforced slope. However, because of space limitations, only the monitoring results obtained from the mechanical extensometers will be covered in this paper. For a complete analysis including global structure deformations from inclinometers, face movements from survey measurements, as well as the pore water pressures monitored from using piezometers the reader is referred to Zornberg (1994).

Mechanical extensometers measure the relative displacements between the slope face and the extensometer plate anchored to the geotextile within the fill. Since the instruments in each extensometer set are installed at increasing lengths from the face, displacements between extensometer plates and, consequently, geotextile strains can be determined. Consistent with results obtained from inclinometer data, no post-construction movements were noticeable from the extensometer measurements taken after completion of the fill construction.

Since no damage during construction operations was experienced in any of the extensometers from sets E1, E2 and E4, all installed at the same cross-section, calculations to define geotextile displacements and strains from extensometer measurements were based mainly on the results from these three instrument sets. Information from extensometer sets E3 and E5 was used to verify the correctness of individual measurements.

Figure 5 shows horizontal geotextile displacements obtained from the extensometer set E2, located 4.27 m above the slope base in row 15, along with superimposed smooth curves defined by fitting of the raw data as explained later in this section. The figure shows displacement distributions at different fill elevations, indicated by number of rows placed during construction. Post-construction measurements obtained during eight weeks following the end of construction are not indicated in the figure as there was essentially no time-dependent movements. Extensometer displacement distributions similar to the one shown for extensometer set E2 were obtained for the other extensometer sets.

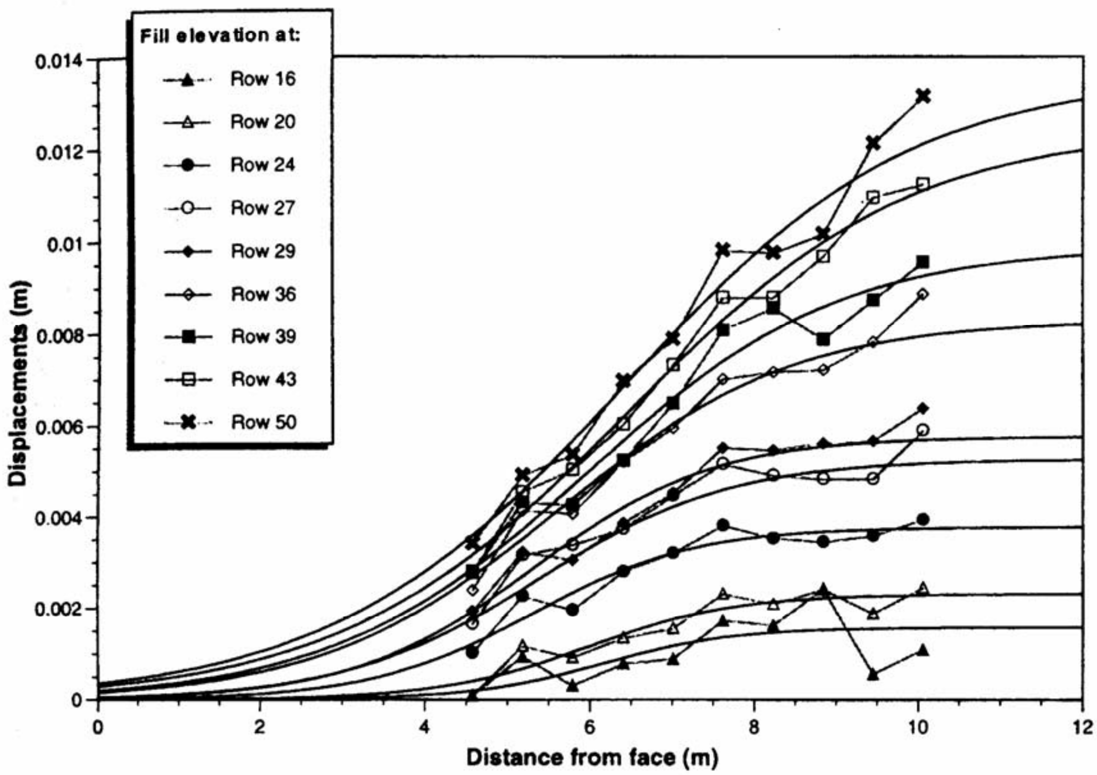

Figure 5. Lateral displacements measured by extensometers attached to geotextile layer 15 (Extensometer set E2)

The pair of inclinometers installed within the reinforced zone allowed for determination of differential soil movement between them. This was particularly useful to cross-check inclinometer displacements with the displacements obtained from extensometers mounted on the reinforcements. Considering the location of extensometers and inclinometers (Figure

2) this cross-check is particularly useful at the level of extensometer set E2 (4.27 m high). Figure 6 shows the relative horizontal displacements between inclinometers SI1 and SI2, at the level of extensometer set E2. The progress in relative displacements with increasing fill elevation obtained from extensometer readings agrees very well with the displacement progress obtained from the inclinometer monitoring results. This validation supports the accuracy of the displacements interpreted from both inclinometer and extensometer measurements.

The superimposed smooth curves in Figure 5 were used to evaluate the geotextile strain distribution. Geotextile strain values can be obtained by calculating relative movements between extensometers and dividing them by the distance between measuring points. However, the use of raw extensometer displacement data to perform these calculations render unclear reinforcement strain distributions since minor scatter in the displacement trend results in major oscillations in the calculated strain distribution. Consequently, the raw extensometer displacement information was initially smoothed by fitting the data to a monotonically increasing curve in order to better define the strain distribution. The expression used to fit the extensometer displacements is a sigmoid curve defined by:

$$d = \frac{1}{a + b\, e^{-cx}} \tag{1}$$

where d is the extensometer displacement, x is the distance from the structure face to the extensometer anchoring plate, and a, b, and c are parameters to be defined by fitting the curve to the raw data using the minimum squares technique.

The geotextile strain distribution can be obtained analytically from the derivative of the displacement function, and is indicated in Figure 6 for the case of extensometer set E2. The figure shows the strain distribution at different construction stages during the construction period. Post-construction strain distribution obtained from readings taken eight weeks after construction of the fill essentially superimpose the distribution indicated for row 50.

The maximum strain ε_{max} at the different construction stages and its location x_{max} from the slope face can be determined analytically using the parameters a, b, and c that define the best-fitting curve for the extensometer data. The expressions are:

$$\epsilon_{max} = \frac{c}{4\,a} \tag{2}$$

$$x_{max} = -\ln\left(\frac{a}{b}\right) \cdot \left(\frac{1}{c}\right) \tag{3}$$

Figure 7 shows the strain distribution at the end of construction obtained using readings from the extensometers in the different instrumented reinforcement layers. The maximum strains at the end of construction are 0.12 % for layer 5 (1.22 m high), 0.20 % for layer 15 (4.27 m high), and 0.16 % for layer 31 (9.14 m high). The strain levels in all the instrumented geotextile layers are very low and, in all cases, there was almost no change in the geotextile strain distribution during the eight weeks following the end of construction.

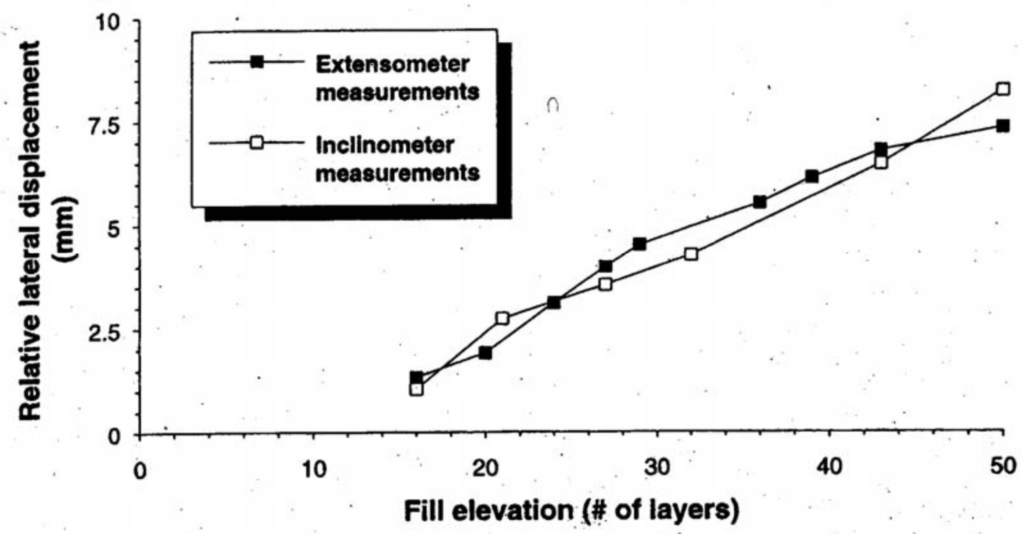

Figure 6. Development of relative displacements with increase in fill height (measured by inclinometers and extensometers).

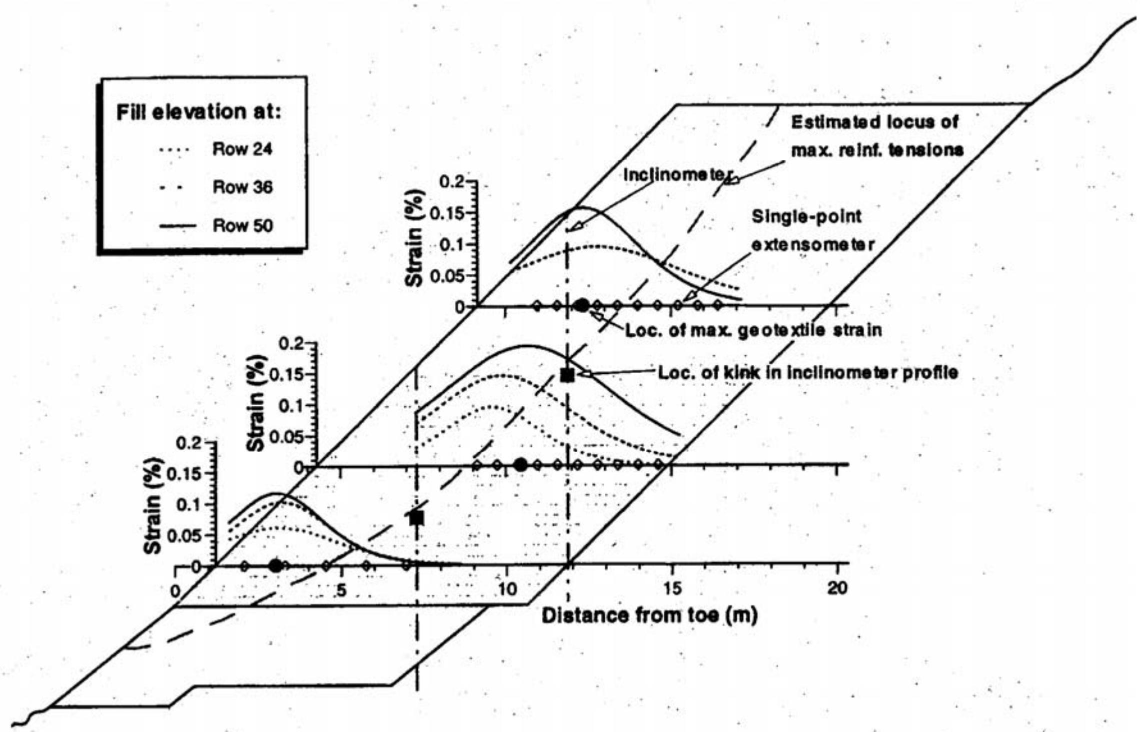

Figure 7. Distribution of strains during construction in each instrumented geotextile layer.

5. IMPLICATIONS ON THE DESIGN

Limit equilibrium methods have been conventionally used in the analysis of reinforced soil slopes to determine the required geotextile layer spacing and reinforcement tensile strength. These methods are techniques for conventional slope stability analysis, adapted to take into account the stabilizing moment created by the reinforcements. Figure 8 indicates the location of maximum geotextile strains at the end of construction in each of the instrumented layers (solid circles). One of the possible loci of maximum reinforcement tensions that can be inferred from these features of the instrumentation results is indicated in the figure. Although the strain levels are too low to expect a well defined line of maximum tension in the reinforcements, the indicated locus agrees with the critical surface defined by conventional limit equilibrium analysis. The field instrumentation results appear to be consistent with the use of limit equilibrium methods as a design basis for geotextile-reinforced soil slopes.

The maximum geotextile strains observed during construction and up to eight weeks following the completion of slope construction are on the order of 0.2%. These are significantly low strain levels, mainly if we consider that extensometers report global strains, comparable with the soil strains obtained from inclinometer readings. Global strains are higher than the local strains that may actually occur in the geotextile layers because extensometer readings incorporate the effect of geotextile macrostructure, and local effects such as geotextile creases and folds.

These strain levels are notably lower than the relatively large geotextile strains at which the design strength would typically be developed. The small maximum strains obtained from monitoring records of this and other geosynthetically reinforced slopes indicate that current design factors of safety are extremely conservative. The geotextile strain levels are much lower than those assumed in current design to define the required tensile strength of the reinforcements. In order to further evaluate the performance of the structure under working stress conditions, deformation analyses should be pursued and the stress-strain relations of geotextiles at low strain levels, mainly under confined condition, should be investigated.

The time-dependent properties of the reinforcement will be examined in more detail at the completion of the monitoring program. However, as shown by all extensometer and inclinometer data monitored up to eight weeks after construction, the geotextile has performed without any time-dependent degradation, and no creep movements were detected.

6. CONCLUSIONS

The instrumentation program detailed in this paper has evaluated the development of geotextile strains of a 1H:1V slope 15.3 m high, constructed using decomposed granite as backfill material. High modulus composites and nonwoven geotextiles were selected as reinforcements since they have the adequate in-plane drainage capacity to allow dissipation of pore water pressures that could be generated in the fill. To evaluate the structure performance, an extensive monitoring program was implemented that included the installation of inclinometers, mechanical extensometers, piezometers and survey points.

Results from the instrumentation program indicate an excellent performance of the slope, with small global deflections and low geotextile strain levels (on the order of 0.2%). A procedure for representation of extensometer measurements was developed, which assisted in the interpretation of the geotextile strain distributions. Cross-check of extensometer and inclinometer measurements showed very good agreement, providing confidence on the monitoring results. No time dependent movements were observed during continuous monitoring for eight weeks after completion of construction. The locus of maximum reinforcement tensions estimated from field instrumentation results is consistent with the location defined using design methods for internal stability analysis of reinforced slopes based on limit equilibrium. The low strain levels observed in the experimental structure under study suggest that further cost reductions could be achieved by reducing factors of safety.

ACKNOWLEDGEMENTS

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GEOSSINTÉTICOS 95

2º SIMPÓSIO BRASILEIRO SOBRE APLICAÇÕES DE GEOSSINTÉTICOS

São Paulo, junho de 1995

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