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Construction and Instrumentation of a Highway Slope Reinforced With High-Strength Geotextiles

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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 geotextiles were selected considering not only that adequate tensile strength should be provided but also that expected in-plane drainage capacity would be beneficial in dissipating pore water pressures that could be generated in the fill. An extensive program of instrumentation and construction monitoring was implemented to evaluate its performance. Preliminary monitoring results taken until eight weeks after completion of the fill indicate an excellent overall performance of the slope, that showed small deflections and reinforcement strains during the construction and early post-construction periods.

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 (Barrows and Lofgren, 1993). 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 not only were required to have adequate tensile strength but were also expected to provide 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.

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. The present project attempted to define the actual stress and strain distribution within the geotextile slope, both during and after construction, in order to evaluate the performance of the reinforced fill and the assumptions in its design. The results of the research may make it possible to determine if modifications to present design procedures are appropriate, especially for higher than conventional slopes.

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 9.5 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.

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. 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 in order to increase the external and compound stability. Methods of slope stability analysis, adapted to consider forces provided by the reinforcements, 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 1, the final design adopted two geosynthetically reinforced zones with a constant reinforcement spacing of 0.3 m (1 ft). At the highest cross-section of the structure, the reinforced slope has a total of 50 geotextile layers. A nonwoven geotextile (PP-20) was selected in the upper half of the slope, while a high strength composite geotextile (PPC-100) was used in the lower half. Both selected geosynthetic reinforcements were 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.

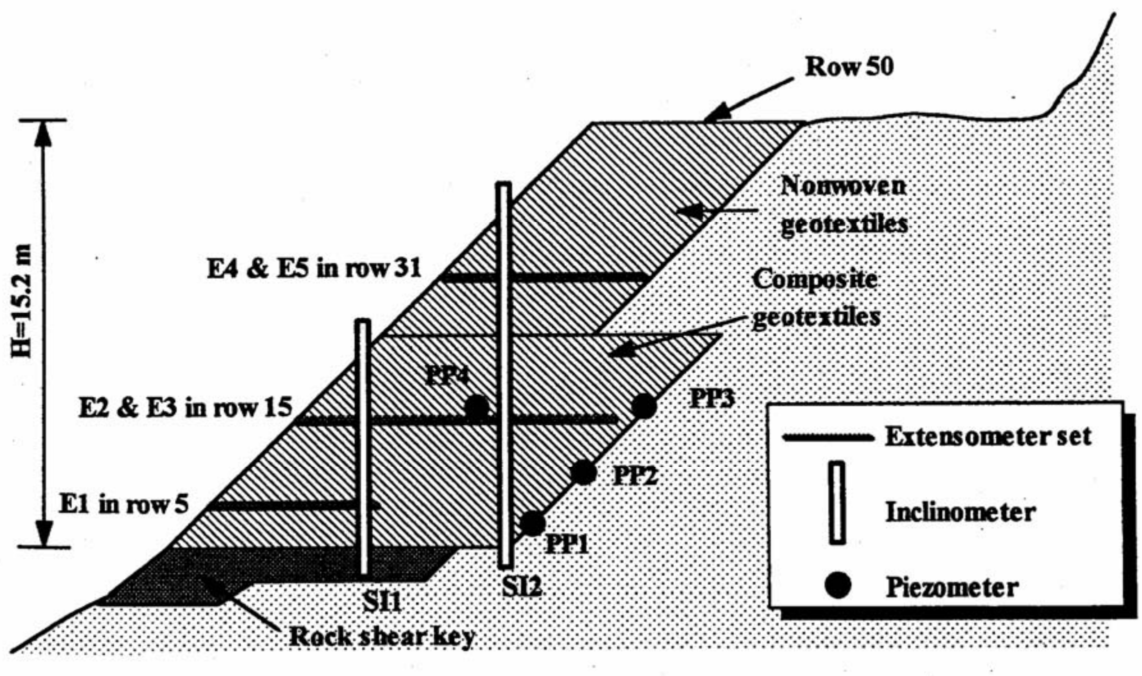

Figure 1. Cross section of the reinforced slope showing the instrumentation layout

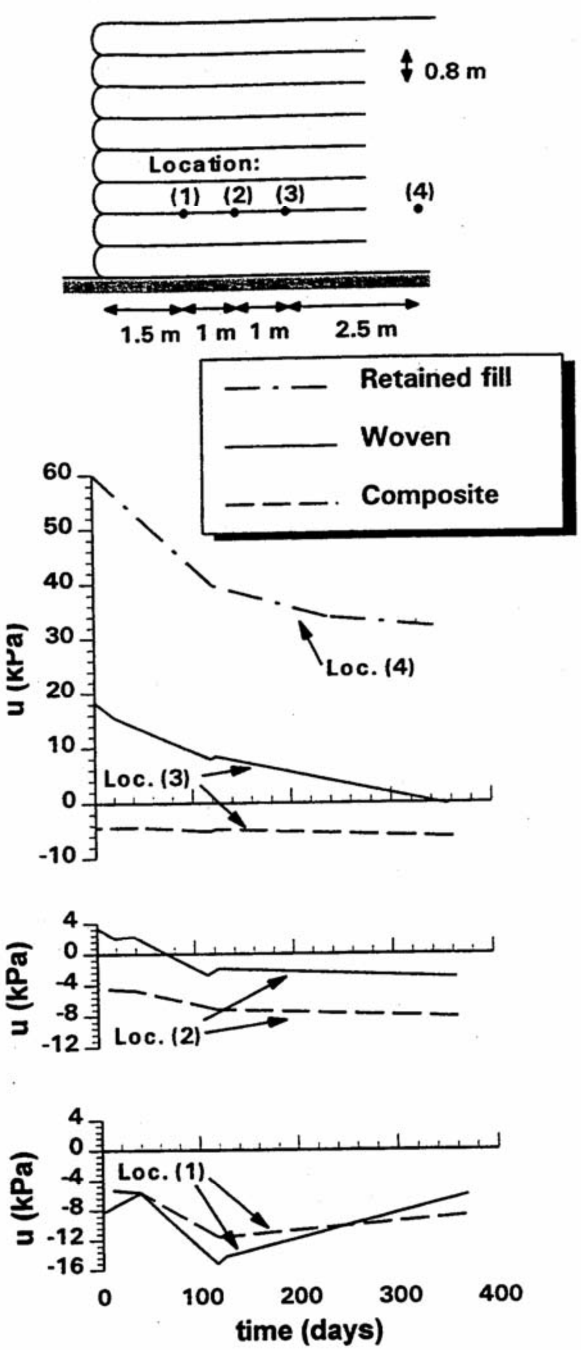

Figure 2. Pore water pressures (u) in the Rouen reinforced wall, along a woven and a nonwoven/geogrid composite, within a silty backfill (redrawn after Perrier et al., 1986)

Basis for geosynthetic 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. On the other hand, the use of reinforcements with appropriate in-plane transmissivity was specified in order to deal with potential seepage from the fractured rock mass. The lateral drainage provided by the reinforcements would avoid the need of a separate drainage system. There is no general design methodology for reinforced soil structures built with poorly draining soils. Nevertheless, since a number of this type of reinforced structures has already been constructed, many lessons can be learned from past experience. Permeable geosynthetics were specified for this FHWA project based on the experimental evidence that these reinforcements can more effectively reinforce poorly draining soils. A good example that supports this decision was provided by a 5.6 m high experimental structure built in Rouen, France, in which pore water pressures were monitored within the silt backfill (Perrier et al., 1986). The structure consisted of sections reinforced with woven geotextiles and a section reinforced with a composite nonwoven bonded to a polyester geogrid. Figure 2 shows positive and negative pore water pressures as a function of time recorded at different locations within the fill. The pressure sensor inside the embankment and beyond the reinforcement region recorded placement excess pore water pressures of as much as 60 kPa at the end of construction. Along the woven geotextile, 3.5 m from the wall face, positive pore water pressures on the order of 20 kPa were registered at the end of construction and dissipated in 350 days. Along the composite geotextile, on the other hand, negative pore water pressures were registered over the whole length of the reinforcement even at the end of construction. As indicated in the figure, pore water pressures along the composite geotextile were systematically lower than those recorded along the non-draining woven textile. Additional evidences that good structure performance is dependent on maintaining a low water content in poorly draining backfills was provided by Tatsuoka et al. (1990), Burwash and Frost (1991), and Huang (1992), among others. However, practice has led theory in the use of poorly draining backfill for reinforced soil structures, and a number of research needs should still be addressed in order to formulate a consistent design methodology (Zornberg and Mitchell, 1994; Mitchell and Zornberg, 1994).

CONSTRUCTION

Slope construction, performed using conventional construction equipment, took place during the summer of 1993. The original slope was excavated back to a 1H:1V side slope, and the base for the embankment was graded to a smooth condition. The rock shear key was constructed by depositing, spreading, and then compacting the rock material with a vibratory roller (Figure 3). The rock shear key was reinforced with welded wire mesh having a vertical spacing of 0.45 m. The selection of a welded wire mesh reinforcement was based on the large openings required to accommodate the size of the rock material in the shear key (up to 380 mm). Although construction took place during the dry summer season, seepage appearing as weeps at the base of the cut slope emerged from the fractured rock mass.

No special expertise was required for slope construction, and a crew of five members without previous experience in reinforced soil construction placed an average of three layers per day along the instrumented, 172 m long slope. In each lift, backfill material was spread with a medium sized bulldozer and oversized rocks (greater than 100 mm) were then removed. Each layer had to be compacted to 95% of maximum density, as determined by Standard Proctor tests, and the water content of the backfill was specified to be within 3% of the optimum. These compaction requirements were easily achieved by the contractor using static compaction methods: a grid roller was used for compaction of most of the fill, and a small walk-behind compactor was used close to the facing. Special care was required when working around the inclinometer tubes during slope construction. The geotextile at each lift level was placed with the longitudinal direction perpendicular to the slope, overlapping adjacent rolls a minimum of 0.60 m. Although initial design did not consider wrapping of the geotextiles at the slope face, the geotextiles were eventually wrapped in order to satisfy National Forest Service requirements. A single layer forming system inclined 45° was used, and holes were made through the geotextile on the face to permit vegetation.

Placement of the top layer (layer 50), was finished approximately one month after placement of the initial layer. An erosion control matting was subsequently placed on the slope and anchored to protect the face until vegetation is well established. Figure 4 is a view of the completed geotextile-reinforced slope after the erosion control matting has been placed. The

Figure 3. Rock shear key under construction

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.

INSTRUMENTATION PROGRAM

A comprehensive monitoring program was designed to evaluate the performance of the reinforced soil slope during and after construction. The instruments were placed at the highest cross-section of the geotextile-reinforced structure with the objectives of:

- evaluating the stress and strain distribution within the reinforced soil slope;
- assessing the effect of lateral drainage provided by geosynthetics with adequate in-plane transmissivity;
- evaluating the deformation response of the structure;
- investigating the long-term performance of the geotextile-reinforced structure; and
- providing a reference base for future designs with the possibility of improving design procedures and/or reducing costs.

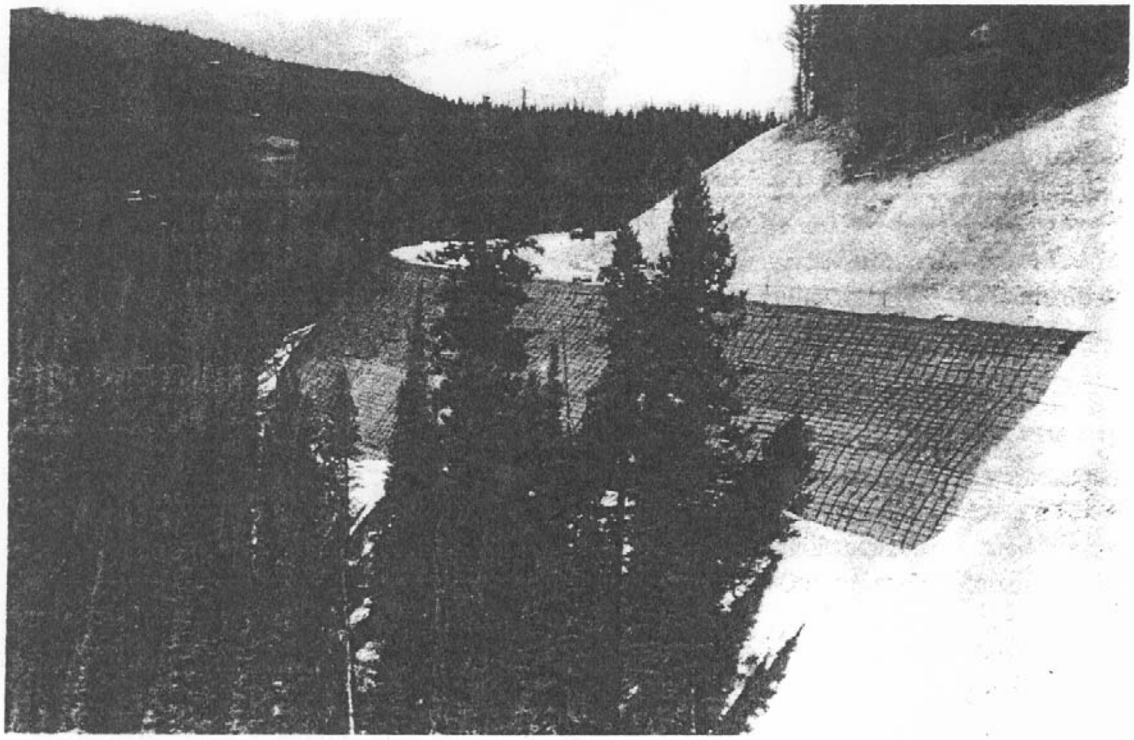

Figure 4. Finished reinforced slope with erosion matting in place

To achieve these goals, the parameters selected for monitoring were the global strain distribution in the geotextiles, with special attention to the magnitude and location of maximum strains, horizontal movements within the reinforced soil mass, movements in the slope face, and pore water pressures within the fill.

Figure 1 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.

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. The inclinometer tubes grouted into the drill casings in the rock shear key for anchorage. During construction of the geotextile-reinforced slope, inclinometer casings were added in 1.52 m sections and backfill was hand-placed and compacted around the inclinometer casings.

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. Slope movements in response to increase in overburden were monitored through changes in offset distances, to the nearest 3 mm (0.01 ft), from control points at the toe of the fill to survey points on the structure face.

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. 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 1, 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.

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. Three piezometers were installed at the back of reinforcement layers 2, 7, and 15 at elevations 0.3 m, 1.8 m, and 4,27 m respectively. A fourth piezometer was installed within the fill between geotextile rows 15 and 16 (4.4 m high) to test the ability of permeable geotextiles in controlling the pore pressure generated within the fill (Figure 1). Pore water pressure was measured at each sensor every 30 minutes, and the maximum, minimum, and average values were stored daily using a multi-channel data logger.

PRELIMINARY INSTRUMENTATION RESULTS

Although the geotextile-reinforced slope is still being monitored, some significant performance behavior has already been recorded. Considering that information recorded through the spring runoff is not available at this writing, and because of space limitations, only preliminary results of the global deformations of the slope and of the geotextile strain distribution will be presented here. For a complete analysis and interpretation of the presented instrumentation records the reader is referred to Zornberg (1994). Although the main focus of these results is on the slope performance during construction, post-construction behavior obtained until two months after the end of construction will also be discussed.

Global structure deformations. Global deformations of the geotextile-reinforced slope were determined from the two inclinometers installed within the reinforced zone and from the survey points located on the structure face. The inclinometers measured horizontal deflections within the reinforced fill to the nearest 0.025 mm (0.001 in), providing a precise evaluation of displacements caused by increasing overburden and of possible post-construction movements. While an approximately linear increase in displacement readings was obtained during the construction period, the progress of lateral displacements until eight weeks following the end of construction clearly showed no increasing displacement tendency, which would be indicative of time-dependent movement. Similar trends were observed from the displacement readings obtained during the construction and post-construction periods from the two inclinometer readings.

Lateral deflections obtained from inclinometers SI1 and SI2 at the end of the construction period are indicated in Figure 5. Inclinometers measure the total horizontal movement relative to the bottom of the inclinometer casing, which is a fixed reference. The displacement profile obtained for inclinometer SI1 shows a relatively uniform rotation of approximately 0.2 degrees, with a maximum horizontal displacement at the end of construction of less than 25 mm (1 inch). This ultimate horizontal displacement is very small, representing a horizontal movement on the order of 0.16% of the height of the geotextile-reinforced slope. A slight kink can be observed in the lateral displacement profile at an approximate height of 2.7 m.

For inclinometer SI2, the maximum lateral displacement obtained at the end of construction is approximately 19 mm (3/4 inch), and was located at the top of the inclinometer. This deflection corresponds to a rotation of less than 0.1 degrees, and represents a movement on the order of 0.12% of the height of the structure. The lateral displacement profile shows a prominence at an approximate height of 4 m, probably caused by localized constraints in the inclinometer tube originated by oversized aggregates or local overcompaction. However, this feature is localized and appears not to affect the general displacement trend. A kink can also be observed in the inclinometer SI2 profile at an approximate height of 6.7 m.

The evaluation of face movements at various stages during fill placement was made by measuring offset distances from survey points to control points located at the toe of the structure. Face movements at the end of construction, obtained from four cross-sections, are indicated in Figure 6. It should be noted that, while inclinometers measure total deflections relative to a fixed reference, survey readings are incremental, in that the measured deflections are relative to the

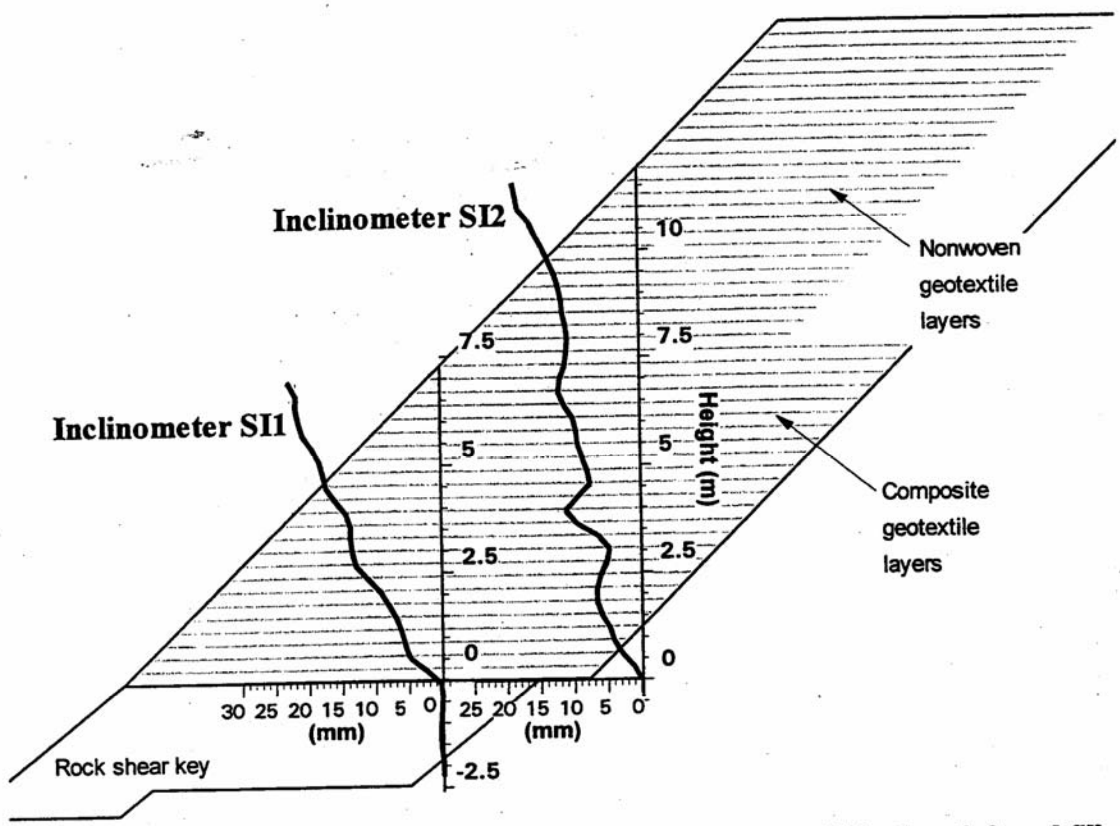

Figure 5. Horizontal deflections obtained from inclinometers within the reinforced fill

initial readings taken for each specific survey point. This is the basic reason for the difference in shape between the inclinometer and survey displacement profiles. Moreover, survey measurements evaluate the total face movements, that include not only a horizontal displacement component, but incorporates also a vertical component. Inclinometer measurements are significantly more accurate than survey data, measured to the nearest 3 mm (0.01 ft). Nevertheless, survey measurements show very small total face movements, which is consistent with results obtained from the more accurate inclinometer and extensometer data.

Geotextile strain distribution. 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 estimated.

Consistent with results obtained from inclinometer data, no post-construction movements were noticeable from the extensometer measurements taken after completion of the fill construction over the eight week monitoring period after construction. Since no damage during construction operations was experienced in any of the extensometers from sets E1, E2 and E4,

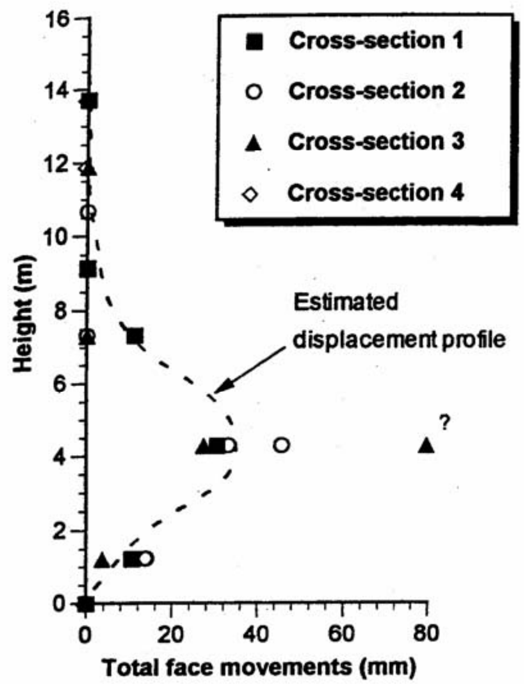

Figure 6. Total face movements at the end of construction as determined from survey measurements

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.

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 1) this cross-check is particularly useful at the level of extensometer set E2 (4.27 m high). The progress in relative displacements with increasing fill elevation obtained from extensometer readings agree 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.

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 displacement function, in order to better define the strain distribution (Zornberg, 1994). The geotextile strain distribution can be obtained analytically from the derivative of the displacement function. Although adding significant fluctuations to the strain distribution, calculations done using the raw relative movements between extensometer plates, provided a trend similar to the one obtained by smoothing the raw data.

Average strain values between the inclinometer locations, estimated using the extensometer readings, were compared to the average strain values calculated using inclinometer readings. A very good agreement was found between the results obtained from these two different instrumentation sources, providing confidence on the estimated strain distributions.

Figure 7 shows the strain distribution at the end of construction obtained using readings from the extensometers in the different instrumented reinforcement layers. The calculated 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.

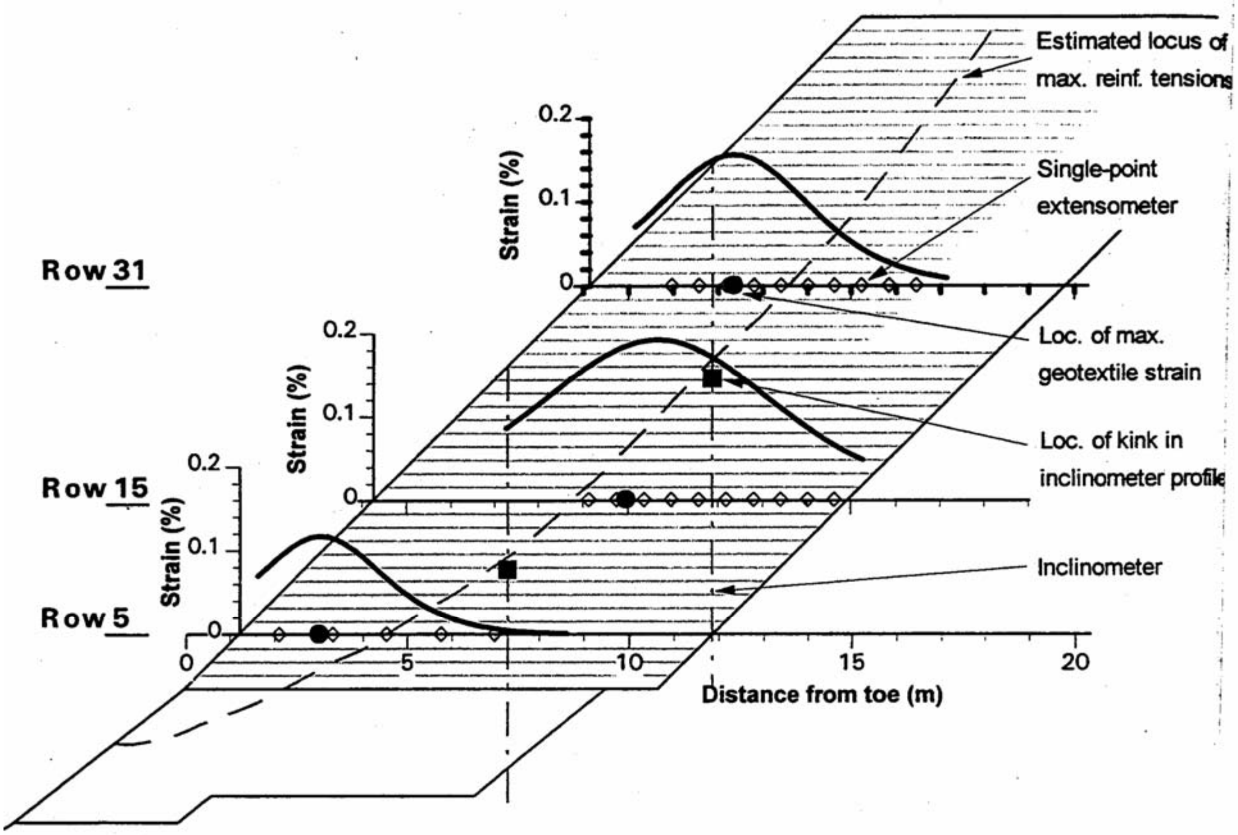

Figure 7. Distribution of strains at the end of construction calculated from extensometer readings

IMPLICATIONS ON THE DESIGN

Suitability of current design approach. Limit equilibrium methods have been conventionally used in the analysis of reinforced soil slopes to determine the required geotextile layer spacing length 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. For internal stability verification purposes, the location of the potential failure surface in reinforced soil structures has been generally identified as the locus of the maximum stresses (or strains) in the reinforcements. Figure 7 indicates one of the possible loci of maximum reinforcement tensions that can be inferred from the instrumentation results. The indicated locus agrees with the critical surface defined by conventional limit equilibrium analysis. The location of the previously mentioned kinks in inclinometer profiles SI1 and SI2, also align well with this estimated surface, providing additional field evidence of its possible location. These results suggest that limit equilibrium methods are an appropriate design basis for the analysis of internal stability of reinforced soil slopes. The location of potential failure surfaces not crossing the instrumented reinforcements cannot be evaluated from the collected data. Consequently, compound or external failures cannot be inferred from the information provided by the inclinometer and extensometer readings. However, external stability considerations during design stages of the structure led to the adoption of a rock shear key at the base of the reinforced slope.

<u>Geotextile strain levels</u>. 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. However, the low strain levels obtained from field monitoring are consistent with previous experiences of well instrumented geotextile-reinforced structures. A good example is the case of a geotextile-reinforced wall 12.6 m high with a surcharge fill more than 5 m in height (Christopher et al., 1990). Although this temporary structure was designed with low factors of safety and it was a vertically faced wall, the maximum global strains obtained from extensometer readings were less than 1%. The strain levels in the structure under study are expected to be comparatively lower since, both structures having a comparable height, the Idaho slope is a permanent structure designed with higher factors of safety and was constructed with a 1H:1V slope face. The small maximum strains obtained from the monitoring results of this structure indicate that representative reinforcement parameters should be obtained at strain levels lower than those assumed in current design and that further investigation of the stress-strain relations of geotextiles, mainly under confined condition, should be pursued.

<u>Global structure deformation and post-construction performance</u>. As monitored by the two inclinometers installed within the fill and confirmed by survey measurements, very small lateral deflections (less than 25 mm) occurred in the reinforced fill. These small horizontal displacements were validated by cross-checking the inclinometer data displacements obtained from extensometers mounted on the reinforcements. 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.

CONCLUSIONS

The instrumentation program detailed in this paper has evaluated the performance 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 considering their potential in-plane drainage capacity to allow dissipation of pore water pressures that could be generated in the fill. To evaluate the performance of the structure, an extensive monitoring program was implemented that included the installation of inclinometers, mechanical extensometers, piezometers and survey points.

Time required for the construction of the instrumented embankment was substantially less than other retaining structures of comparable size constructed on the same project. Preliminary results from the instrumentation program reveal an excellent performance of the slope, that showed small deflections and reinforcement strains, as well as negligible post-construction movements. However, results from slope displacement and geotextile strain monitoring have not yet covered the most critical period that corresponds to the spring thaw. The locus of maximum reinforcement tensions estimated from field instrumentation results tend to agree with the location of potential failure surfaces defined by limit equilibrium methods used for internal stability analysis of reinforced slopes. Long-term performance is still being monitored, and further interpretation of the instrumentation records will provide additional evaluation of the design assumptions, assessment of the ability of permeable geosynthetics to reinforce indigenous soils, and insight into the mechanisms that dominate the behavior of geotextile-reinforced slopes.

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