Reinforced Embankment over Very Soft Soil: Grassy Sound Highway Embankment

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ABSTRACT: A highway embankment approximately 2 km in length was constructed over very soft tidal marshes in the southern coastal region of New Jersey. A design utilizing high strength geotextiles in combination with vertical drains and staged-construction resulted in significant savings to the Contractor and the New Jersey Department of Transportation. The tensile strength of one of the geotextiles is among the highest ever to be used in the U.S.A. Strain gages were attached to the high strength geotextiles to monitor their performance. The highway embankment was instrumented with piezometers, settlement platforms, and slope inclinometers. The performance of the embankment in terms of both settlement and tensile strain developed in the geotextiles was in reasonably good agreement with the design predictions.

1 PROJECT DESCRIPTION

The original design of the new Route 147, near Wildwood, New Jersey, required wet excavation of very soft tidal marsh soils, sheet piling for retention, replacement with sand backfill and construction of the highway embankment in stages. The $62 million new section of highway in the southern coastal region of New Jersey is about 4 km long with two major bridges (see Figures 1 and 2). About half of the length of the new section of highway was to be built on very soft tidal marshes using staged construction, with embankment heights ranging from 2.8 to 4.7 m above existing grades.

A "Value-Engineering Study" performed for the General Contractor indicated that elimination of the costly wet excavation and sheet piling, and placement of the embankment fill directly on the marsh surface employing high-strength geotextiles would result in significant savings for the Contractor and the New Jersey Department of Transportation (NJDOT). (The concept of Value Engineering encourages the Contractor to find technically sound, but more cost-effective means of construction, with the savings being split between the Owner and the Contractor.)

The design changes involved placing two layers of high-strength geotextiles directly on the marsh soils to reinforce the embankment. The tensile strength of the

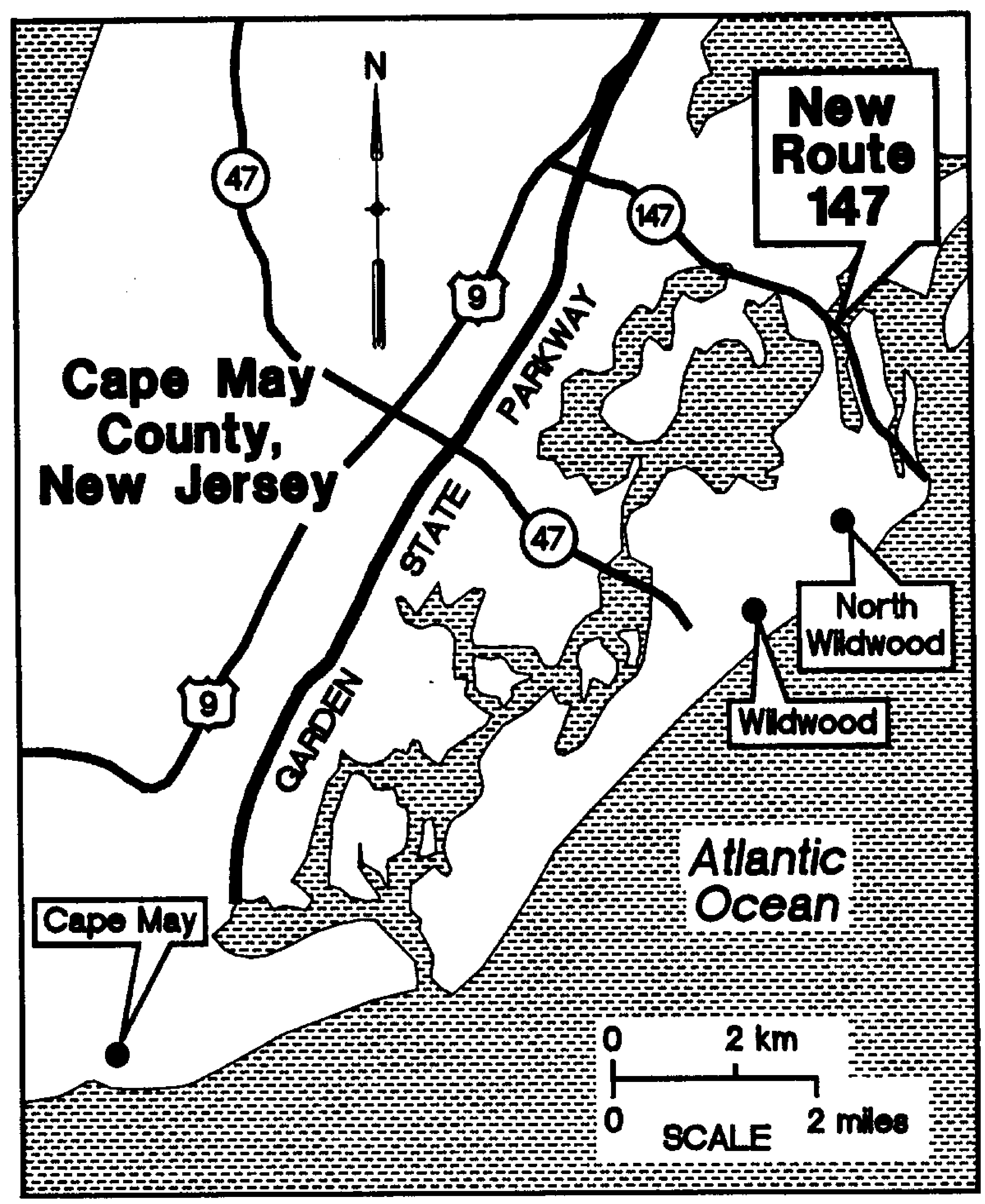

Figure 1 Wildwood, New Jersey

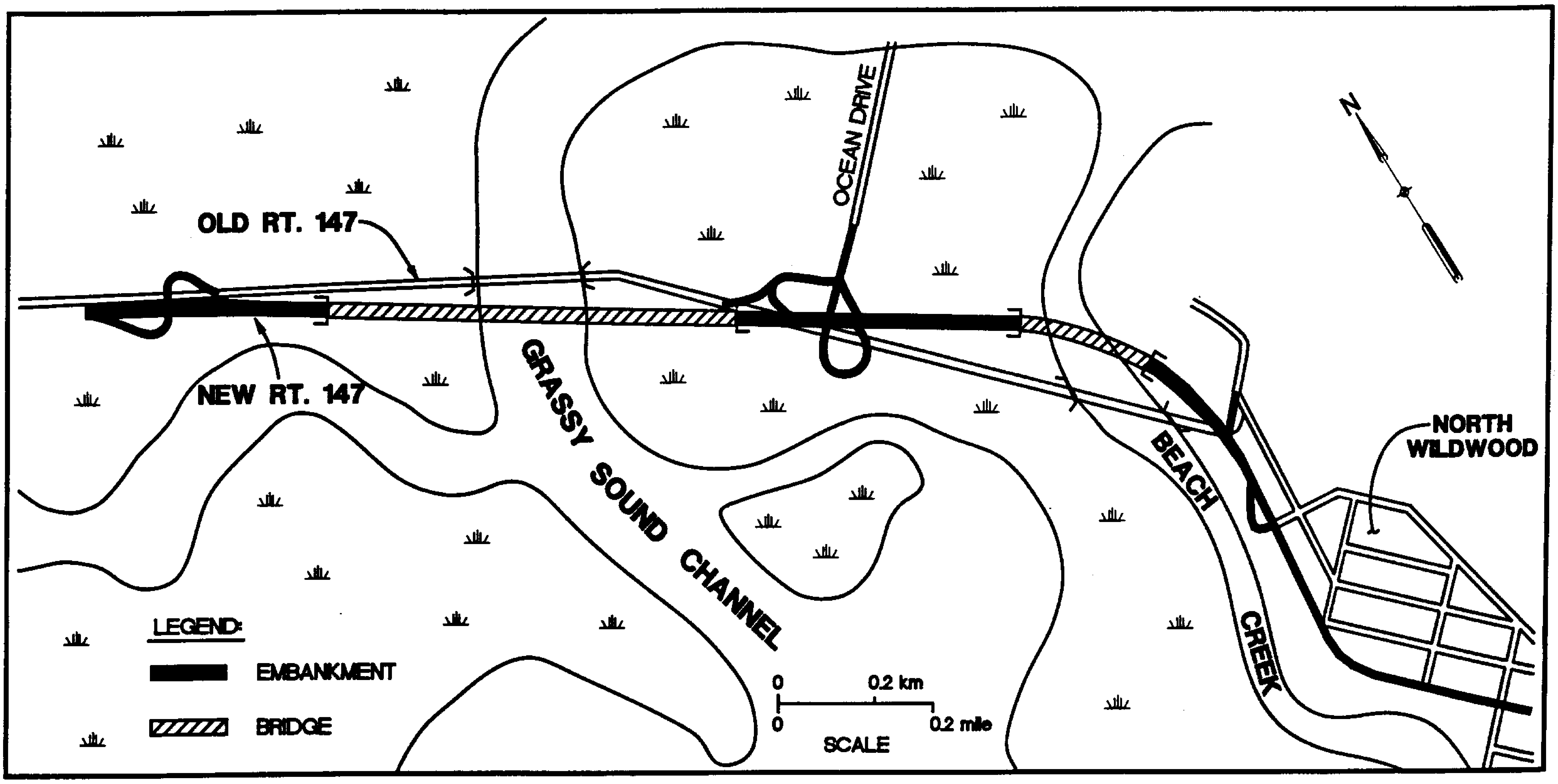

Figure 2 Route 147 Site Plan

first layer of geotextile, 730 kN/m (50,000 lb/ft), is among the highest ever used in the U.S.A. The NJDOT approved the "Value-Engineering Design" in Spring 1991, and during the summer fabric was placed and wick drains and monitoring instrumentation were installed. The embankment was constructed in 4 to 6 stages, varying from 1 to 1.8 m of fill per stage, to allow the 7-m-thick organic clay layer time to consolidate and gain strength. The waiting time between stages was on the order of 4 to 6 months. Total settlement for the embankment was on the order of 1.8 to 2.7 m, depending upon the height of embankment. The embankment was completed in Summer 1994.

The instrumentation included settlement platforms, piezometers, and inclinometers in the embankment; and strain gages installed on the high-strength geotextile to measure fabric strains directly and to provide insight on the reinforcement behavior.

2 SITE AND SUBSURFACE CONDITIONS

The project site is a flat tidal marsh with ground surface elevations typically ranging from +0.7 to +1.0 m (mean sea level), traversed by tidal creeks up to 2 m in depth. Mean tides in the area vary from a high of +1.2 m to a low of -0.6 m. In most areas, the marsh surface is a fibrous rootmat capable of supporting foot traffic.

2.1 Very Soft Organic Clay

Underlying the rootmat is a very soft organic clay stratum generally 6 to 7 m in depth, the upper portion (top 1 m) of which is fibrous with moisture content as high as 400 percent. Becoming less fibrous with depth and containing occasional shells, the material was classified as CH with some OH. The moisture content ranged from 90 to 240 percent, liquid limit from 90 to 200 percent, plastic limit from 30 to 80 percent, and plasticity index from 50 to 130 percent. Unconsolidated-undrained (UU) triaxial shear tests yielded strength values ranging from 4.8 kPa (100 psf) for the fibrous portion, to 1.4 kPa (30 psf) at the top of the soft clay, to 9.6 kPa (200 psf) at the bottom, indicative of a very soft consistency. These undrained shear strengths correlate well with values expected for a normally consolidated clay using normalized strength parameters (Ladd, 1991). Total unit weights of the organic clays ranged from 11 to 14 kN/m^3 (70 to 90 pcf).

2.2 Underlying Soils

Beneath the very soft organic clay stratum is a fine sandy silty clay/clayey silt ranging in thickness from 0.8 to 1.5 m, generally stiff in consistency as indicated by pocket penetrometer readings and UU tests. Average values of moisture content, liquid limit, and plastic limit were 27, 35, and 18 percent, respectively.

Silty medium to fine sand with varying amounts of fine gravel underlies the stiff clay, and is in a dense to very dense condition as indicated by Standard Penetration Test (SPT) values. The stratum varies in thickness and is interbedded with layers of stiff to very stiff clay to a depth of about 30 m, where a very dense sand and gravel layer occurs (SPT values greater than 100 blows per 0.3 m).

3 DESIGN

3.1 Value Engineering Design vs. Original Design

The original design required 1.8 m of wet-excavation of the tidal marsh with temporary sheeting for soil retention; replacement with 1.8 m of clean sand; placement of a 1 m-high working pad; installation of wick drains and instrumentation; and then construction of the embankment with geogrid reinforcement. The primary motivation for the Value Engineering (VE) design was to eliminate the costly wet-excavation of the tidal marsh and temporary sheeting prior to constructing the embankment. The contractor could take advantage of the existing rootmat by placing the geotextile on the tidal marsh and then constructing the embankment. Also, the VE design reduced the staged construction of the embankment by one stage (i.e., six to five) in most areas of the site. Inclinometers were added to the original instrumentation program which consisted only of piezometers, observation wells and settlement platforms.

3.2 Methodology

The principles and procedures discussed in FHWA (1989), Bonaparte and Christopher (1987), and Koerner (1990), were used in the VE design of the reinforced embankment over soft ground. Potential modes of failure that were evaluated included bearing capacity, slope stability (failure through reinforcement), lateral sliding (on geotextile) and pullout of the geotextile. In addition, vertical settlement of the embankment was estimated. The geotextile was specified with partial factors of safety for creep, installation damage, durability, and to account for holes punched by the installation of vertical drains. The computer programs "STBREF" (Humphrey and Holtz, 1986, and Reyna et al., 1992) and "STABGM" (Duncan, Low and Schaefer, 1985) were used for slope stability analyses with tensile reinforcement.

3.3 Typical Section

The width of the reinforced embankment was typically 30 to 36 m at the base and 15 to 18 m at the top. Side slopes typically varied from 2H:1V to 3H:1V. Embankment heights ranged generally from 2.8 to 3.7 m, with a maximum height of 4.7 m (5.8 m including surcharge) at one bridge abutment.

3.4 Bearing Capacity

The very soft organic clay foundation required that the embankment be constructed in stages to preclude a bearing capacity failure (i.e., bearing capacity governed how much fill can be placed for each stage). Based on the width of the embankment and depth of the soft clay stratum, and using the procedure in Bonaparte and Christopher (1987), a bearing capacity factor (N_C) of 6 was chosen for design. Shear strength increase in the very soft clay layer was estimated using the results of consolidated-undrained (CU) triaxial tests. A CU cohesion value of 1.9 kPa (40 psf) and friction angle of 13 degrees was used in the design. Due to the large anticipated settlements of the embankment, the bearing capacity calculations were performed in conjunction with the settlement analysis. The bearing capacity analyses indicated that the embankment fill should be placed in not more than 1.4 m-high stages, with the first stage being 1 m in thickness.

3.5 Settlement

Based on consolidation tests, it was decided that the 7-m-thick organic clay stratum would be analyzed for vertical settlement using a modified compression index (compression ratio) of 0.42 for the top 2.4 m of the organic clay stratum and a value of 0.33 for the lower 4.6 m. The very soft clay was found to be normally consolidated. A staged-construction settlement analysis was performed using the fill thickness for each stage determined from the bearing capacity analysis. Due to large estimated settlements, the vertical consolidation from each previous stage was taken into account by decreasing the organic clay stratum thickness. Total vertical settlement was estimated to be on the order of 2.1 to 2.7 m, depending on the height of embankment.

3.6 Stability

Slope stability analyses of the staged construction were performed with the most critical situation being at the end-of-construction of the last stage. The Simplified Bishop Method of Slices was used. The reinforcement force gained from the geotextile was applied halfway between horizontal and tangential to the slip circle at the base of the slice where the circle intersected the geotextile (FHWA, 1989).

The shear strength of the organic clay for the initial analyses and area outside the embankment was assumed to be 2.9 kPa (60 psf) for the top 0.6 m to 1.4 kPa (30 psf) at a depth of 0.9 m and increasing to a value of 7.2 kPa (150 psf) at the bottom of the stratum as shown on Figure 3. The CU cohesion and friction angles described in Section 3.4 were used to estimate the shear strength increase due to consolidation of each stage of the embankment. A friction angle of 32 degrees was estimated for the sandy embankment fill. For the typical embankment section with 2H:1V slopes and height of 3.7 m, it was found that an allowable geotextile tensile strength of 335 kN/m (23,000 lb/ft) was required for a factor of safety of 1.3 at the end-of-construction of the final stage. Lateral stability of the reinforced embankment was found to have satisfactory factors of safety.

3.7 High-Strength Geotextile

Partial factors of safety selected for the high-strength polyester geotextile were FS_{CREEP} of 2.25, $FS_{INSTALLATION\ DAMAGE}$ of 1.1, and $FS_{DURABILITY}$ of 1.0. In addition, the allowable strength of any geotextile punctured by the vertical drains was reduced by approximately 30 percent to account for the reduction in area. Also, the high-strength geotextiles were specified such that strains would not exceed 5 percent for the design section.

In the typical section, a high-strength polyester geotextile with an ultimate tensile strength of 730 kN/m (50,000 lb/ft) was designed to be placed on the existing ground. The first fill stage, approximately 1 m in thickness, would be placed and the vertical drains then installed, puncturing the lower geotextile. Then a second high-strength polyester geotextile with an ultimate tensile strength of 438 kN/m (30,000 lb/ft) would be placed before placing the second fill stage.

The high-strength geotextile was to be placed in continuous panels with the warp direction perpendicular to the centerline of the highway embankment. The seam strengths were specified to be 53 kN/m (3600 lb/ft).

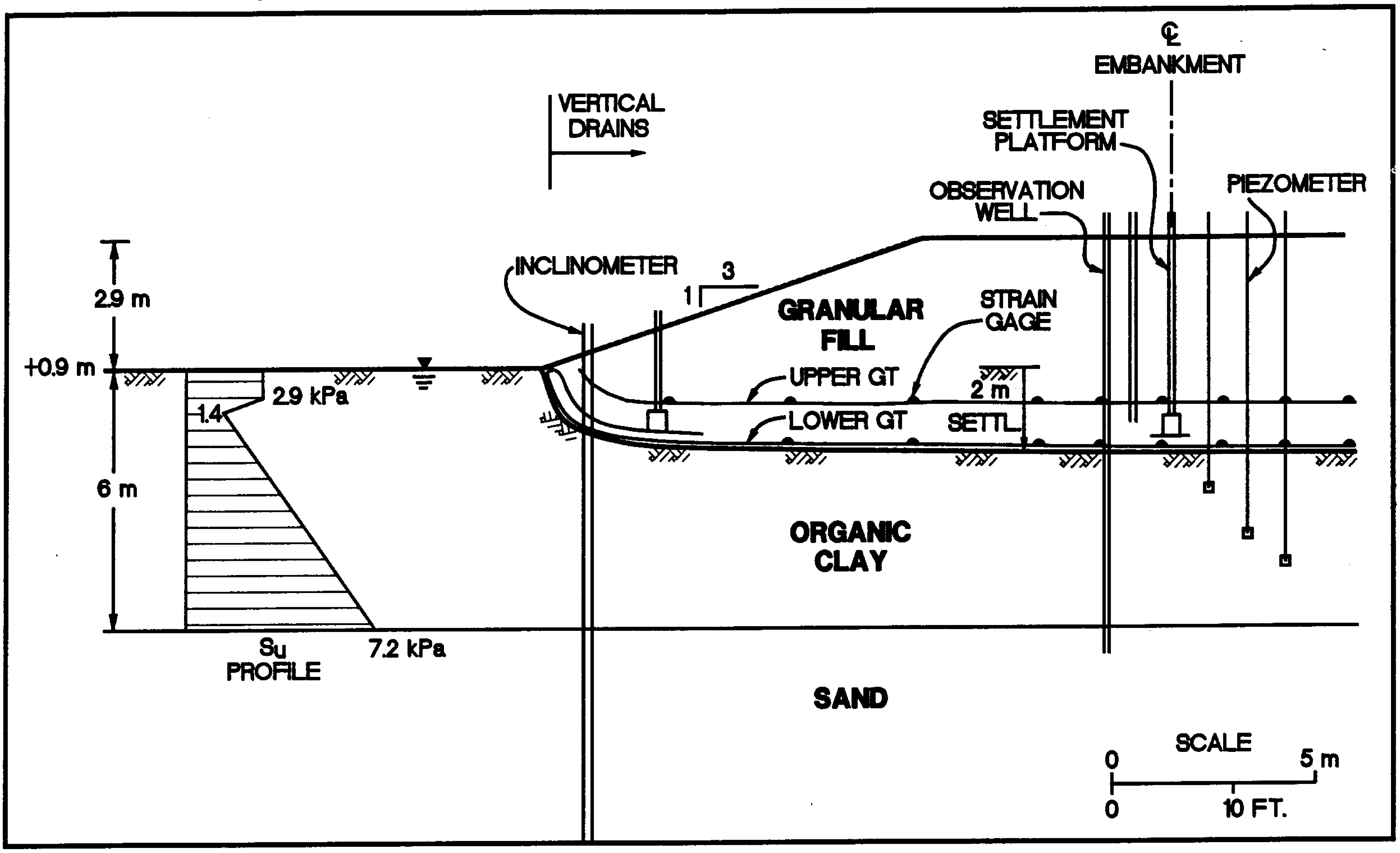

Figure 3 Strain-Gaged Section of Embankment

Tensile strengths that were required to be reached at or before 5 percent strain were: lower geotextile - 292 kN/m (20,000 lb/ft), upper geotextile - 175 kN/m (12,000 lb/ft), and in the fill direction for both - 53 kN/m (3,600 lb/ft).

Both geotextiles were required to have a sand to geotextile interface friction angle of 24 degrees. Pullout resistance was checked and although factors of safety were sufficient, the NJDOT, in their review, recommended that the bottom geotextile be wrapped over the first 0.3 m of fill and folded back a distance of 4.6 m.

3.8 Vertical Drains

Vertical drains were required at 0.76 m triangular spacing as part of the original design. Although it was believed that such close spacing might cause excessive soil disturbance and remolding, it was decided not to increase the spacing. Consolidation tests indicated that the vertical coefficient of consolidation was on the order of 0.0005 to 0.0018 m^2/day (0.005 to 0.019 ft^2/day). Horizontal consolidation tests indicated that the horizontal or radial coefficient of consolidation was on the order of 1.5 to 2 times the vertical coefficient of consolidation. Based on these results, it was believed that the original design estimate of six months for 90 percent consolidation was reasonable, though possibly slightly conservative.

4 CONSTRUCTION

4.1 High-Strength Geotextile Properties

The two high-strength geotextiles used on this project, Comtrac 750.285 and 450.140, consisted of high tenacity polyester yarns and were manufactured by Huesker Inc. The geotextiles weighed 1.35 and 1.15 kg/m^2 (40 and 34 oz/yd^2), respectively. The two geotextiles met the tensile requirements described in Section 3.7.

The seams were dual 401 chain stitches sewn with a Union Special 80200Z sewing machine using a Tex 450 polyester thread. A "prayer" style seam was installed having an ultimate strength of 53 kN/m (3,600 lb/ft).

4.2 Strain Gage Installation

Gages type EP-08-40CBY-120, manufactured by Micro-Measurements Division, were used. Their nominal length is 10.2 cm (4 in.). The procedure to attach the gages on similar polyester geotextiles is described in detail in Fowler and Leshchinsky (1990) and Schimelfenyg, Fowler and Leshchinsky (1990). Though sixteen gages were attached to the lower geotextile, only eleven survived the field installation.

After the vertical drains were installed, the full length of cable for each gage was placed in a trench at the top of the first fill stage and then attached to multiple dial switches in a remote readout unit located 2 m beyond the toe of slope. Approximately 60 percent more than the required length of cable was snaked in the trench in a "zig-zag" to allow for settlement of the embankment. By the end of the project (three years after installation), three of the remaining eleven gages on the bottom layer were inoperable. This is believed to be due to damage from driving the vertical drains near the gages or from a leak in the silicone waterproofing of the strain gages.

On the upper geotextile layer, all sixteen gages were operable throughout the duration of the project, since no vertical drains were installed and because experience had been gained from the lower layer installation.

4.3 Geotextile Installation

Unique geotextile deployment methods were developed by Atlantic Construction Fabrics, Inc. for construction over the extremely soft soil, tidal creeks and uneven ground. Sewing machines were mounted on specially constructed swamp buggies to negotiate and sew on the adverse terrain. Series of blocks and tackle combined with heavy equipment were used to move sewn panels. Panels as large as 45 m by 91 m weighing 5900 kg were deployed in one move.

4.4 Embankment Construction

Stage one fill, consisting of clean sand, was placed in approximately two 0.46 m lifts and compacted only with a low-ground-pressure bulldozer (approximately 295 g/cm^2 (4.2 psi)). The fill was placed along the toe of slopes and then filled in. Virtually no mud waves were observed during fill placement, with the exception of one area of the site where there was very little rootmat.

After the 0.9 to 1 m of fill for the first stage was placed, settlement platforms, piezometers, observation wells and inclinometers were installed. Then vertical drains were installed at 0.76 m triangular spacing extending approximately 0.6 m into the alluvial sand layer. Installation, with a vibrating mandrel mounted on a specially fabricated low-ground-pressure rig, proceeded without difficulty through the high-strength geotextile. The instrumentation layout is shown on Figure 3.

5 MONITORING

5.1 Vertical and Horizontal Displacement

Settlement platforms were surveyed regularly to measure vertical settlement. Figure 4 shows the actual vertical settlement versus time for the strain-gaged location of the embankment. It should be noted that the strain-gaged location of the embankment, 2.9 m in height, was constructed in four stages with vertical settlements on the order of 2 m. In general, total vertical settlement ranged from 1.8 to 2.7 m, depending upon the height of the embankment.

The maximum horizontal displacement measured from the inclinometers, located at the toe of slope, ranged from 0.4 to 0.5 m at elevation -1.7 m. A maximum horizontal displacement versus time plot for the strain-gaged location of the embankment is shown on Figure 4.

5.2 Organic Clay Shear Strength

Triaxial shear strength (UU) testing was performed at the end of each stage in various locations to verify that the organic clay stratum had gained strength and was sufficiently strong for the next fill stage. UU test results indicated that the organic clay had gained more strength than was anticipated using the CU test results. This resulted in larger fill stages (up to 1.8 m in thickness) in one location during the latter stages of construction.

5.3 Strain in High-Strength Geotextiles

Monitoring of the strain gages was performed using a 4-1/2 digit ohm-meter. This technique, suggested and validated by Leshchinsky and Fowler (1990), is reliable and easy to use in the field. For the strain gages used, Leshchinsky and Fowler (1990) showed that strain elongation in percent is equal to 0.4 times the change in gage resistance in ohms relative to an initial value measured immediately after installation of the geotextile and before any fill placement. Consequently, monitoring the change in gage resistance in the field enabled the determination of the corresponding strain. Tensile force at a particular strain can be determined from the tensile force versus elongation curves of the geotextiles.

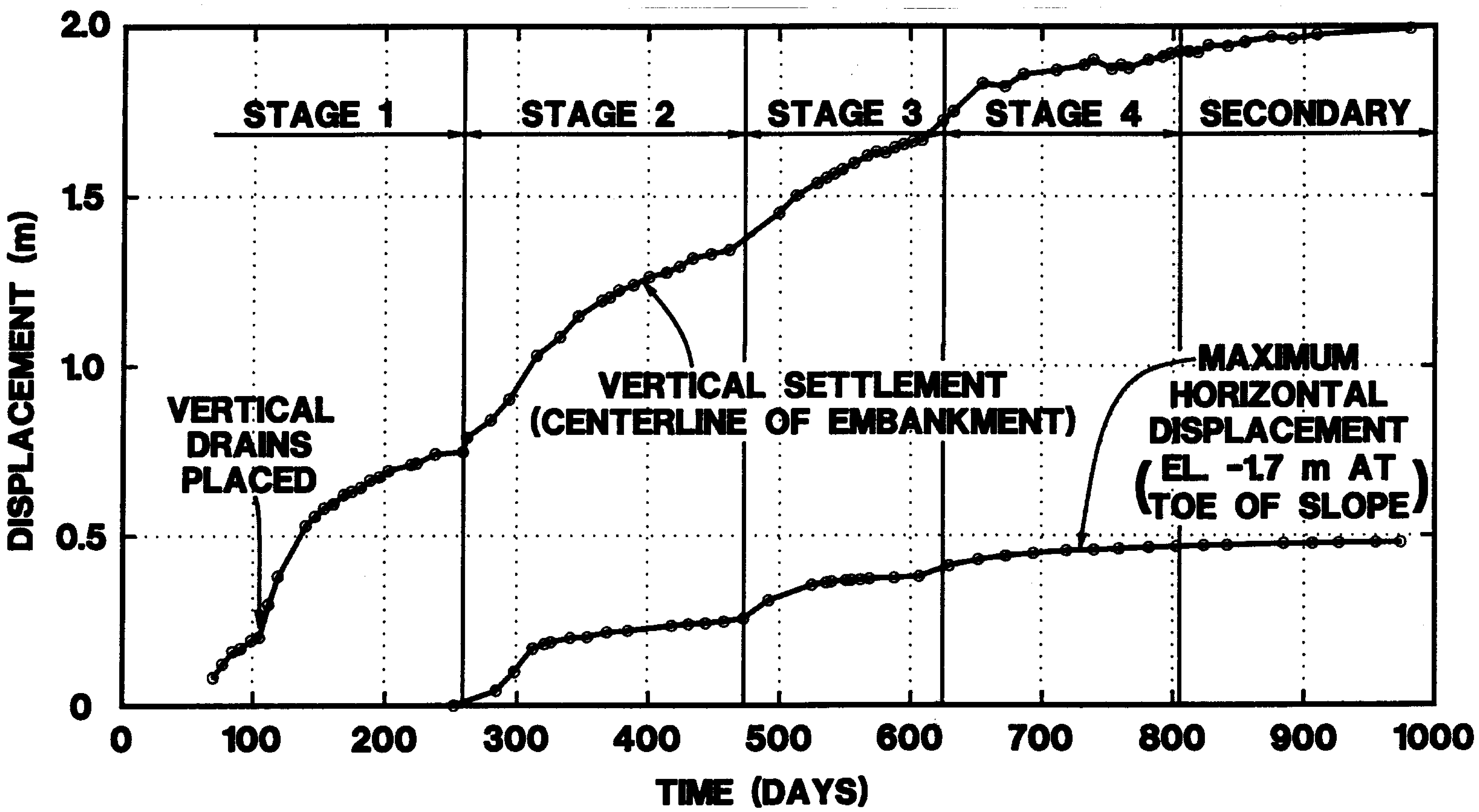

Figure 4 Displacement vs. time in strain-gaged location of embankment

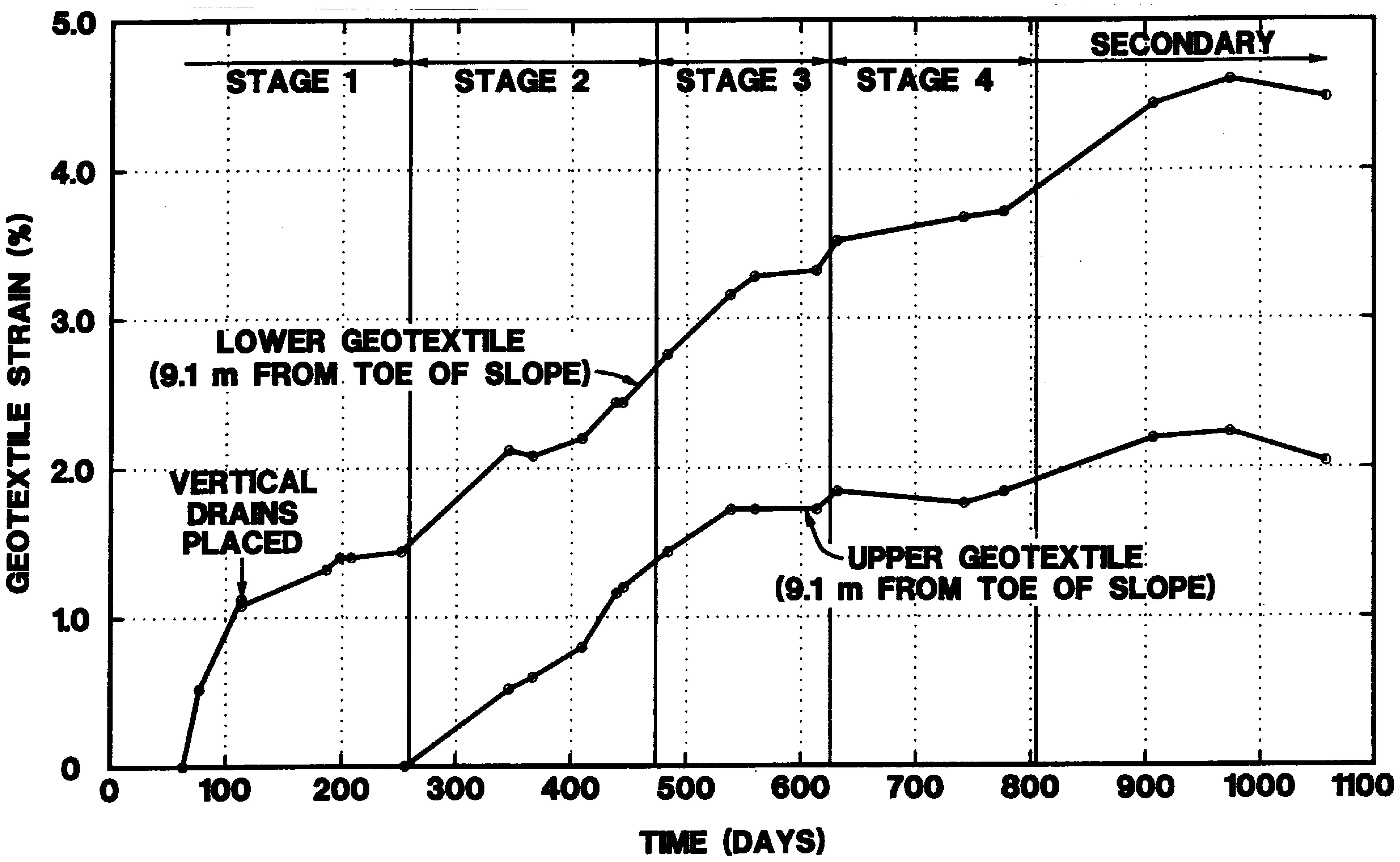

Figure 5 Geotextile strain vs. time

Six months after the end-of-primary consolidation of the fourth and final stage, the lower geotextile reached a maximum strain of approximately 4.6 percent and the upper geotextile reached a maximum strain of 2.2 percent. Plots of strain versus time, and strain versus distance from toe of slope, are shown on Figures 5 and 6, respectively.

6 LESSONS LEARNED

An embankment was designed and constructed over very soft ground utilizing staged construction and high-strength geotextiles, and monitored with extensive instrumentation. The following lessons were learned from this project.

1. The performance of the embankment to date, despite very large vertical settlements has been satisfactory. No mud-waving beyond the toe of slopes or cracking of the embankment has been observed.
2. The actual vertical settlements were slightly less, yet very close to the predicted settlements.
3. The actual time for primary consolidation of the construction stages, estimated using procedures from Asaoka (1978), was reasonably close to the predicted time given the scatter of the laboratory consolidation tests.
4. The actual undrained shear strengths of the very soft organic clays determined from UU tests at the end of each stage was slightly higher than predicted using CU test results.

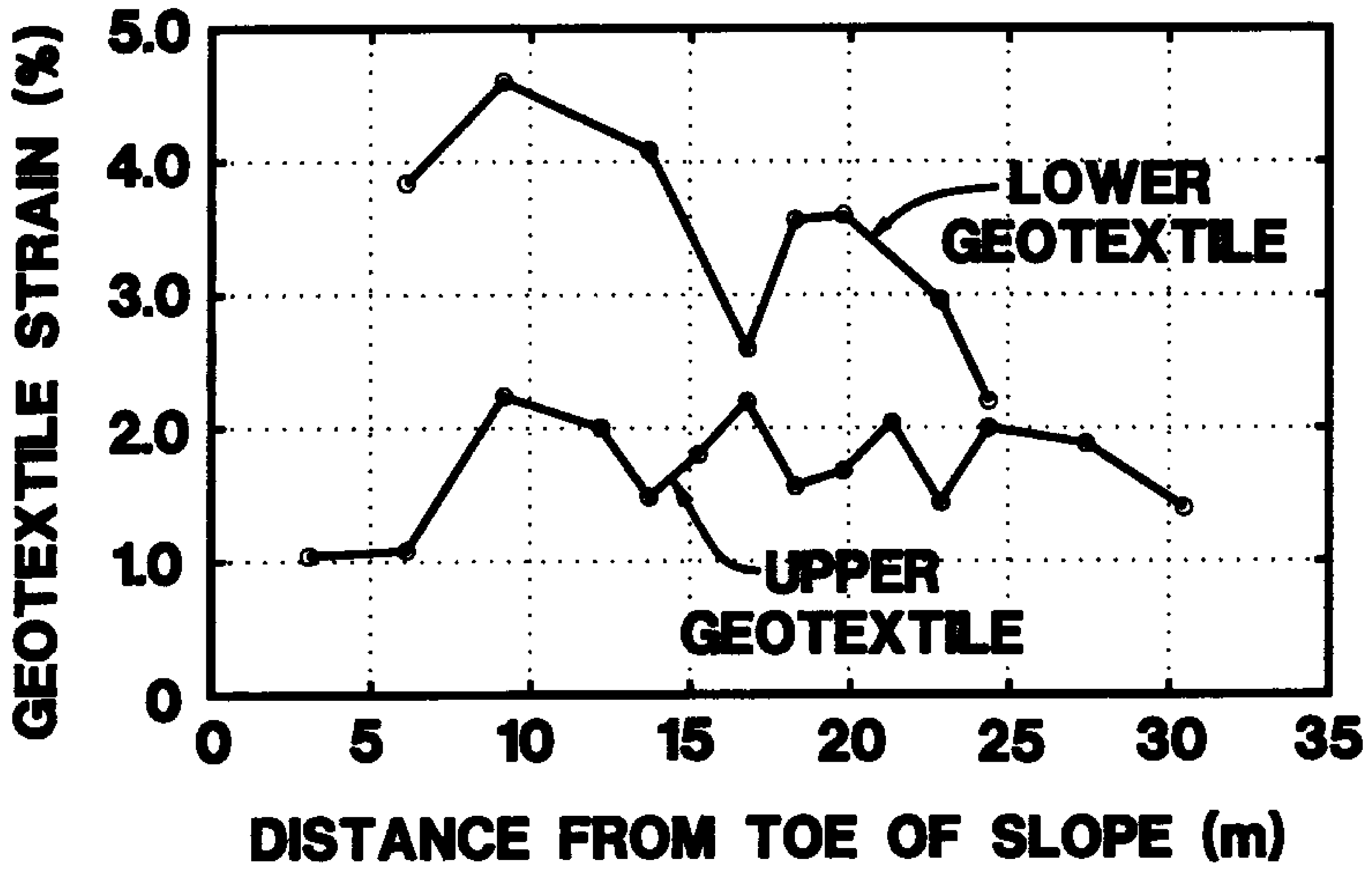

Figure 6 Geotextile strain vs. distance from toe of slope (six months after end-of-primary consolidation of fourth and final stage)

5. The actual strain developed in the high-strength geotextiles was slightly less than predicted.

6. In general, the performance of the embankment in terms of magnitude and time rate of settlement, shear strength of the soft foundation soils and tensile strain developed in the geotextiles was in reasonably good agreement with the design predictions.

7. The utilization of strain-gages to monitor strains in geosynthetics should be considered in certain critical applications such as staged construction, in the same way that piezometers, settlement platforms, inclinometers, and shear testing are used to monitor the performance of the embankment and foundation soils.

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