Field Test of a Geotextile-Reinforced Levee

Reda M. Bakeer, Tarik A. Hadj-Hamou, Frank M. Duarte, and Gerard S. Satterlee

The field performance of a full-scale test section of a geotextile-reinforced embankment founded on a soil deposit of mostly soft clays is discussed. The test section was constructed in 1986 under the supervision of the U.S. Army Corps of Engineers as an enlargement prototype of an existing hurricane protection levee. The section was built to test the performance of the proposed design prior to commencing work on the entire 13-mi (20.8-km) span. The soil investigation conducted by the Corps included undisturbed and disturbed borings, soil sampling, and laboratory testing. The test section was fully instrumented with inclinometers, settlement plates, and piezometers. Displacement transducers were used to measure the deformations in the fabric. Field measurements consisted of horizontal movements, vertical settlements, pore pressures, and strains in the fabric. Measurements were taken during construction and continued for 2 years.

Soft and highly organic soils are common in Louisiana, where thousands of miles of embankments are built along highways or as levees for protection against river floods and hurricanes. Conventional construction of an embankment on soft soil may require using piles or replacing some of the soft material with sand or shells. In addition, excessive loss of marsh and dry land results from the use of very flat side slopes to assure the stability of an embankment. Recently, an alternative design was made possible by the introduction of geosynthetics. The inclusion of geogrids or geotextiles reinforces the embankment, reduces its size, controls its deformation, and increases its overall stability. The use of geosynthetics reduces the overall cost and destruction of marsh, as well as accelerating the construction process.

At present, the design of reinforced embankments is based on the classical concepts of earth pressure and slope stability, with minor modifications to account for the effect of the geosynthetics. The designer considers several potential failure wedges in the slope at the state of limit equilibrium and calculates their factor of safety. This approach is easy to apply, but it does not model the interaction of the fabric with the soil or the effect of the method of construction. It ignores the large deformations in the fabric required to develop a significant effect on the reinforced soil mass. Neither the stresses nor the strains at various parts of the embankment or the fabric could be determined by this approach. The finite element method is also used to model fabric-reinforced soils. Different models have been developed to account for nonlinearity, plastic failure, or creep, and various types of elements have been used to model the soil and fabric. However, since no reliable closed form solution is available, the results of a finite element analysis must be verified experimentally. The results of full-scale field models are the most reliable source for examining the performance of fabric-reinforced embankments, because scaling may affect small laboratory models. On the other hand, the number of field studies available in the literature is small due to the associated elaborate work, high cost, and long duration.

This paper presents the results of a field test on a full-scale fabric-reinforced embankment. The embankment under investigation is the Reach "A" test section of the hurricane levee at Tropical Bend in Louisiana. The levee was constructed and instrumented by the New Orleans District of the U.S. Army Corps of Engineers (COE) in association with Plaquemines Parish (county) of Louisiana in the period between October and December of 1986.

DESCRIPTION OF THE TEST SECTION

The levee under consideration is located near the Gulf of Mexico in Lower Plaquemines Parish in southeastern Louisiana. From a geological standpoint, the site is in the central Gulf coastal plain at the modern delta of the Mississippi River, where several other major deltas were also formed over the past 5,000 years (*1*). The main sediments of engineering interest in this area were deposited from the Pleistocene epoch to present time. These sediments are divided into three main categories: natural levee, point bar, and backswamp deposits. Slightly elevated ridges of natural levees extend along the banks of the Mississippi River. The constant migration of the river in the past resulted in the formation of point bar deposits, where the coarse materials are deposited downstream on the convex side of the river banks. On the other hand, backswamp deposits were formed by the deposition of fine sediments in the shallow ponded areas during bank overflows. These backswamp deposits contain thin laminated clays and silts of high organic nature.

The test section is a prototype for a proposed enlargement of the existing levee located between the Gulf of Mexico and a drainage canal, with its centerline 160 ft (48.8 m) off the centerline of the canal, as shown in Figure 1. The crown of the existing levee along this 13-mi (20.8-km) reach is approximately at elevation 7.5 ft (2.29 m) National Geodetic Vertical Datum (NGVD) and its centerline is 20 ft (6.1 m) off the centerline of the enlarged levee on the canal side. It is to be raised to elevation 14.5 ft NGVD (4.42 m) to protect the

R. M. Bakeer and T. A. Hadj-Hamou, Department of Civil Engineering, Tulane University, New Orleans, La. 70118. F.M. Duarte and G. S. Satterlee, U.S. Army Corps of Engineers, New Orleans District, New Orleans, La. 70160.

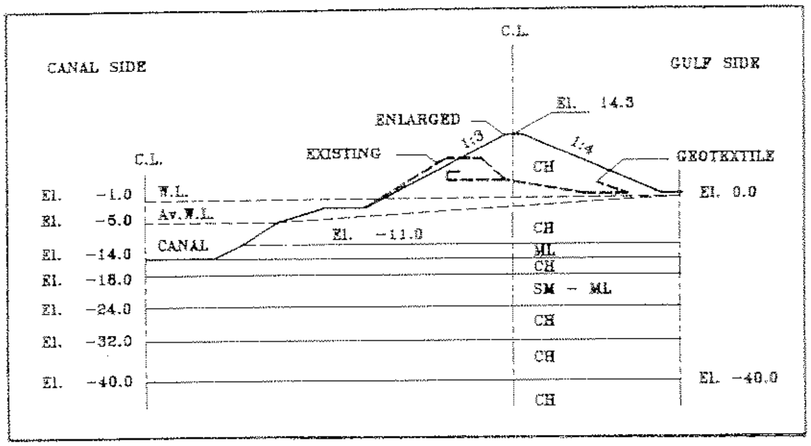

FIGURE 1 Cross section of test levee.

canal side of the levee against a 100-year storm after compensating for consolidation settlement. The 500 ft (152.4 m) long test section is reinforced with high-strength polyester geotextile. The water table is at elevation −1.0 ft (−0.305 m), NGVD, whereas the average water level in the canal is at elevation −5.0 ft (−1.52 m). The crown of the levee is 8 ft (2.13 m) wide and its base is 122 ft (37.19 m) wide. The levee has side slopes of 1V (vertical) on 4H (horizontal) on the Gulf side and 1V on 3H on the canal side.

Soil samples, laboratory tests, and a thorough soil investigation were conducted by the COE New Orleans District. The simplified soil profile in Figure 2 shows that the top layer between the ground surface [elevation 0.0 and elevation −11.0 ft NGVD (−3.35 m)] is mostly highly organic clay (CH). Two thin layers of silt of low plasticity (ML) extend between elevations −11.0 and −14.0 ft (3.35 and 4.27 m) and elevations −21.0 and −24.0 ft NGVD (6.4 and 7.32 m). A thin layer of silty-sand 3 ft (0.91 m) thick exists between the bottom ML layer and a second 4 ft thick (1.22 m) CH layer. The soil deposit below elevation −24.0 ft (7.3 m) consists of layers of highly organic clay (CH). The borrow materials used in the levee construction consist of poorly graded river sand for the core and silty clay for the impermeable cover.

Soil samples were collected from two soil borings on the Gulf side at 4.2 and 5.5 ft (1.28 and 1.68 m) on the centerline of the new levee and one soil boring 69.0 ft (21 m) off its centerline on the canal side. Properties of the CH clay layers were obtained from the results of consolidated undrained (R) and unconsolidated undrained (Q) triaxial tests at different confining pressures. Settlement calculations were based on the results of consolidation tests performed on clay samples (CH) from different depths. A summary of the average soil properties accumulated from the different tests is shown in Table 1. Examination of the consolidation curves, water content, and Atterberg limits indicated that the CH layers were all normally consolidated except for the top layer, which was slightly overconsolidated under the weight of the existing levee. Variation of the shear strength and wet density of the subsoils with depth are also plotted in Figure 2.

Analysis of the existing levee was performed by COE using conventional slope stability analysis (2–4). Their study indicated that the existing levee has a safety factor of 1.1. Raising the crown to the new elevation with the same side slopes would induce failure in the levee, because the factor of safety would decline to 0.8 for a slide toward the canal and 0.85 for a slide toward the Gulf. Stabilizing the levee with symmetrical berms was not feasible because of its closeness to the canal. Enlargement of the existing levee using conventional methods implied that the centerline of the levee had to be relocated about 120 ft (36.6 m) toward the Gulf into the marsh, in order to accommodate the required flat slopes and berms. The top 10 ft (3.05 m) of highly compressible soil would have to be excavated and replaced with sand using hydraulic dredging. Clay would have to be hauled to form the levee cover and bring the section to its design grade.

An alternative design using geotextiles and showing significant advantages over the conventional solution was proposed and analyzed by COE (3). Stability analysis was performed using the wedge method of analysis (2) to determine the tensile strength of the fabric required to provide a safety factor of 1.3. A woven polyester fabric with a tensile strength of 1,700 lb/in (297.7 kN/m) at 5 percent strain was selected for the final design shown in Figure 1. Laboratory tests were performed at Drexel University on samples of different geotextile fabrics to determine which would meet the design requirements. A summary of the properties of the fabric is given in Table 2. Based on the available information, the most critical circular failure was found to be toward the canal and

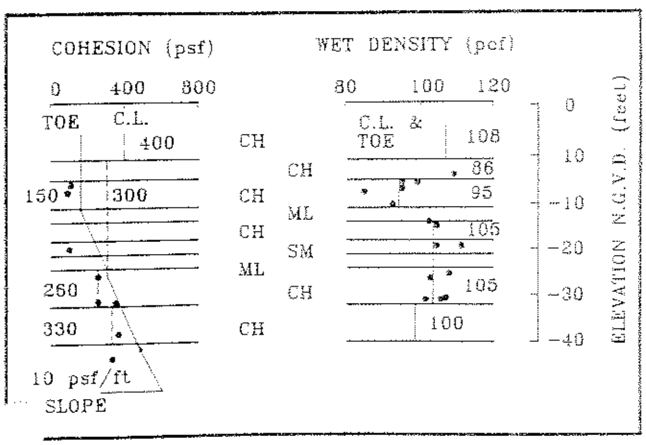

FIGURE 2 Shear strength and densities in subsoils.

TABLE 1 AVERAGE PROPERTIES OF THE SUBSOILS

SOIL LAYER[1]	1	2	3	4	5	6
THICKNESS (ft)	11	3	4	3	3	>16
TYPE USCS[2]	CH	ML	CH	SM	ML	CH
LL	110	NA[3]	61	NA	NA	71
PL	24	NA	22	NA	NA	22
PI	86	NA	39	NA	NA	49
ø (degrees)	22.0	25.0	22.0	30.0	25.0	0.0
c (psf)	290	200	200	0	200	300
γ_d (pcf)	51.9	NA	69.6	NA	NA	64.4
γ (pcf)	95.0	117.0	106.0	122.0	117.0	102.4
e_o	2.44	NA	1.45	NA	NA	1.65
W_c (%)	89.4	NA	52.6	NA	NA	60.1
S_r (%)	99.1	NA	98.7	NA	NA	97.7

[1] Refer to Figure 1

[2] Unified Soils Classification System

[3] Not Available

TABLE 2 PROPERTIES OF THE GEOTEXTILE FABRIC

Property	Value
Tensile warp @ 5 percent strain	1,700 lb/in.
Tensile warp @ ultimate	3,793 lb/in.
Tensile fill @ ultimate	1,188 lb/in.
Seam strength @ ultimate	486 lb/in.
Polyester thread	6 stitches/in.
Creep elongation @ 500 hours	1.22 percent
Friction angle	
Levee to levee	30 degrees
Silty-sand to marsh (organic clay)	14 degrees

passing through the bottom CH layer at elevation −40 ft (12 m).

It was decided to test a full-scale section before commencing the actual construction, in order to obtain more information on field performance and to provide guidelines for current and future designs. The test section was constructed by Plaquemines Parish under COE supervision according to the schedule given in Table 3 and Figure 3. The existing levee was enlarged by maintaining its landward toe on the canal side and backfilling into the marsh toward the Gulf (see Figures 1 and 3). The top 3 ft (0.9 m) of the existing levee (Region 2 in Figure 3), were degraded to elevation +5.0 ft NGVD (1.52 m) to provide a wide working platform and additional anchorage for the fabric.

Four rolls of fabric, each weighing 1.5 tons (1,360 kg) were used to cover Region 1 in Figure 3. The geotextile had an unseamed length of approximately 70 ft (21.34 m) in the warp direction. The fabric was rolled in place by a crawler-mounted hoe and stretched by hand over the degraded levee surface, its Gulf side slope, and the marsh grass on the Gulf side, and submerged in the ponded areas. Sand was placed over the fabric in Region 3 to a maximum height of 4 ft (1.22 m) to form the core of the new levee. Both ends of the fabric were folded back over the sand to increase the anchorage and pull-out resistance. The clay cover was placed over the sand to the specified design grade in Regions 2 and 4-8 according to the sequence shown in Figure 3 and Table 3. Borrow material was hauled to the site by dump trucks and spread using bulldozer and a crawler-mounted hoe. All soils were placed in their natural state, and compaction was provided by the repeated motion of the construction equipment.

FIELD INSTRUMENTATION

The instruments were installed at similar locations along two stations (659+00 and 660 + 00) 100 ft (30.5 m) apart. The instruments were installed prior to the placement of the sand layer. Construction was halted for a week to install the instruments and for another 5 days after the levee reached elevation

TABLE 3 CONSTRUCTION SCHEDULE OF THE TEST SECTION

DATE[1]	ACTIVITY	REGION
10-17	Started degrading levee to El.[2] +5.0	2
10-20	Finished degrading levee to El. +5.0	2
10-21	Started installing fabric	
10-22	Finished installing fabric	
10-22	Replace clay over fabric on levee	2
10-22	Hauled sand cover on fabric (1908 yd^3)	3
10-23	Hauled sand cover on fabric (840 yd^3)	3
10-24 TO 11-02	Shutdown for soil borings and instrumentation	
11-03	Hauled clay cover (1368 yd^3)	4
11-04	Hauled clay cover (1908 yd^3)	4
11-07	Hauled clay cover (696 yd^3)	5
11-10	Hauled clay cover (1416 yd^3) El. +10.0	5
11-11 TO 11-16	Shutdown for instruments readings & evaluation	
11-17	Hauled clay cover (1536 yd^3)	6
11-18	Hauled clay cover (2208 yd^3)	7
11-19	Hauled clay cover (1602 yd^3)	8
11-20	Hauled clay cover (816 yd^3)	8
11-21	Dressed out levee	
11-24	Dressed out levee	
12-08	Fertilized levee	
12-09	Seeded levee	

1 Calendar year 1986

2 Elevation in feet N.G.V.D.

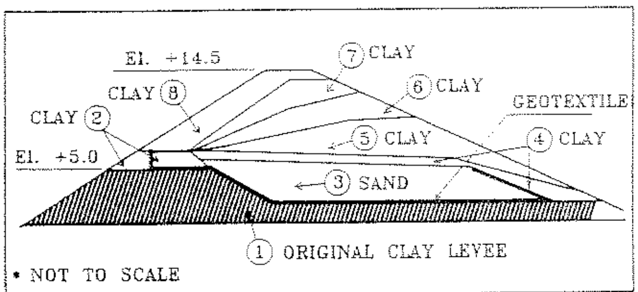

FIGURE 3 Construction procedure of test section.

+10 ft NGVD (3.05 m) to check the instruments and evaluate the performance of the section. Readings of the instruments were recorded using an electronic recording device and backed up by hand-written forms during construction and for the next two years after completion.

Six inclinometers were installed by COE at the two stations. At each station, one inclinometer was installed near the canal (11 or 12), at the crown of the levee on the Gulf side (13 or 14), and on the side slope of the levee on the Gulf side (15 or 16). At station 660+00, shown in Figure 4, inclinometers 11, 13, and 15 were located at distances of 83.0, 5.5, and 30.0 ft (25.30, 1.68, and 9.14 m) off the centerline of the new levee, respectively. The inclinometer's tips above the ground surface were at elevations 3.8, 19.8, and 13.6 ft NGVD (1.16, 6.04, and 4.15 m), whereas their bottom tips were at elevations −101.2, −95.5, and −96.8 ft (−30.85, −29.11, and −29.50 m), respectively.

Four settlement plates, 4 ft by 4 ft (1.22 m) each, were installed by Plaquemines Parish on the surface of the fabric below the sand core at the same two stations. At station 660+00, shown in Figure 5, settlement plates S-1 and S-3 were located 5 and 25 ft (1.52 and 7.62 m) off the centerline of the new levee. The original elevations of the settlement plates after installation were 3.53 and 1.82 ft (1.08 and 0.56 m), respectively.

A total of eight piezometers were installed by COE at stations 659+00 and 660+00. The piezometers at station 660+00

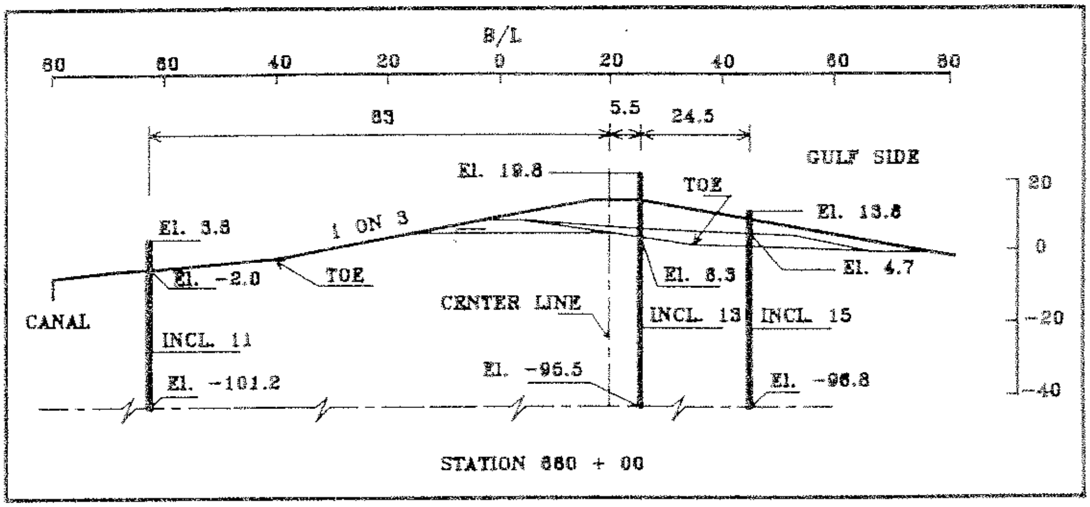

FIGURE 4 Layout of inclinometers.

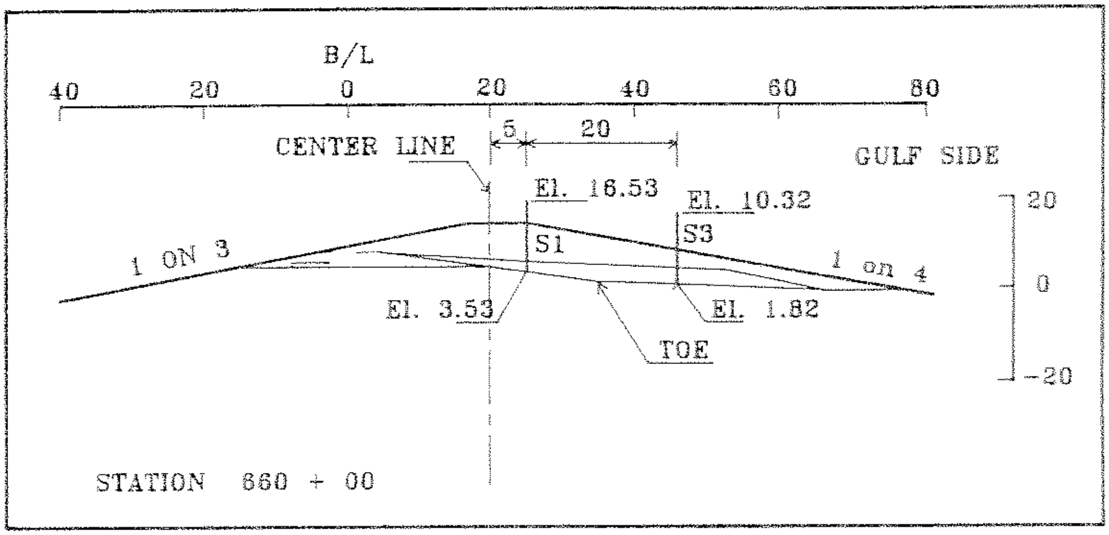

FIGURE 5 Layout of settlement plates.

were placed along an axis 5.5 ft (1.68 m) off the centerline of the new levee on the Gulf side, as shown in Figure 6. The tips of piezometers 1A, 1B, 1C, and 1D were at elevations −4.8, −10.8, −20.8, and −30.3 ft NGVD (−7.02, −3.29, −6.34, and 9.24 m), whereas their riser pipe elevations were 17.61, 17.34, 17.45, and 17.36 ft NGVD (5.37, 5.29, 5.32, and 5.29 m), respectively. The tips of the A piezometers were at the top clay layer (CH), whereas the tips of the B piezometers were at the bottom of the second CH layer, just above the silt layer (ML). The tips of the C piezometers were in the center of the silty sand layer (SM) between a CH layer and the second ML layer. The tips of the D piezometers were 6 ft (1.83 m) deep into the bottom CH layer located below the second ML layers.

Two types of strain gauges were installed on the fabric during construction along three profiles. Two of these profiles, at stations 659+00 and 660+00, contained mechanical displacement transducers manufactured and installed by U.S. Army Waterways Experiment Station (WES). The third profile was equipped with foil gauges (linear voltage displacement transducers or LVDTs). The mechanical strain gauges were attached to plates previously installed on the fabric at WES, whereas the foil gauges were glued to the fabric at the factory. A sketch of the layout of the foil gauges used in this study is shown in Figure 7. The foil gauges were placed along three rows spaced 4 ft (1.22 m) apart with each row containing five gauges spaced equally at 8 ft (2.44 m). The center row consisted of the CT gauges (CT-1 through CT-5). The left row contained LT gauges, whereas the right row included RT gauges. These gauges were used to measure the lateral strains in the fabric, perpendicular to the test section. A fourth group of longitudinal foil gauges (CL-1 through CL-5) was installed adjacent to the center row of lateral foil gauges to measure the longitudinal strains along the test section, as shown in Figure 7. All foil gauges were calibrated after installation and normalized to an initial reading of 0.9 percent strain. This

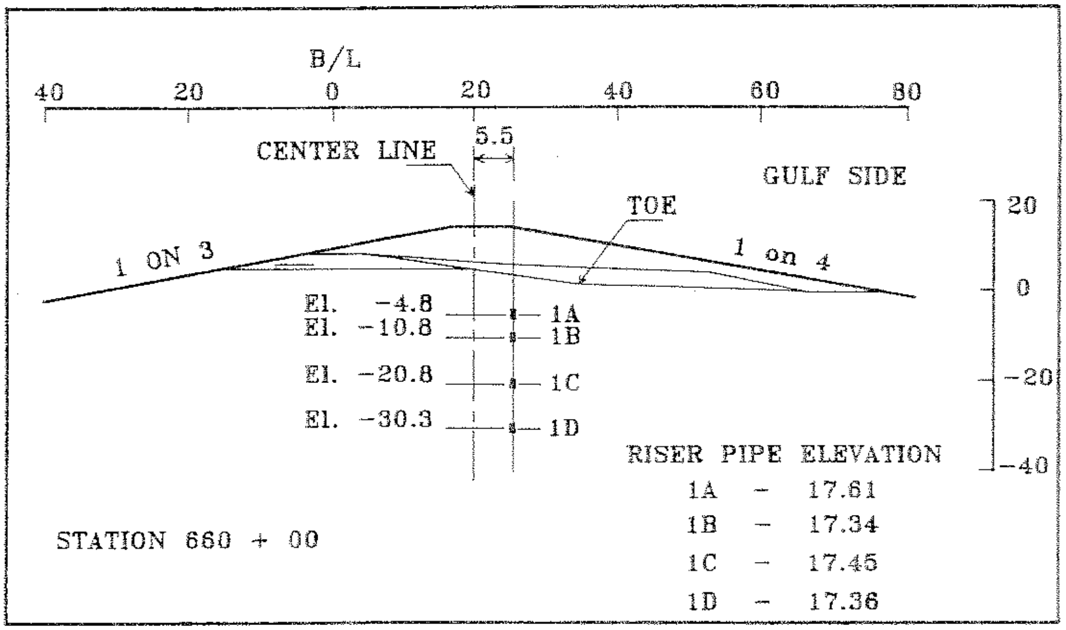

FIGURE 6 Layout of piezometers.

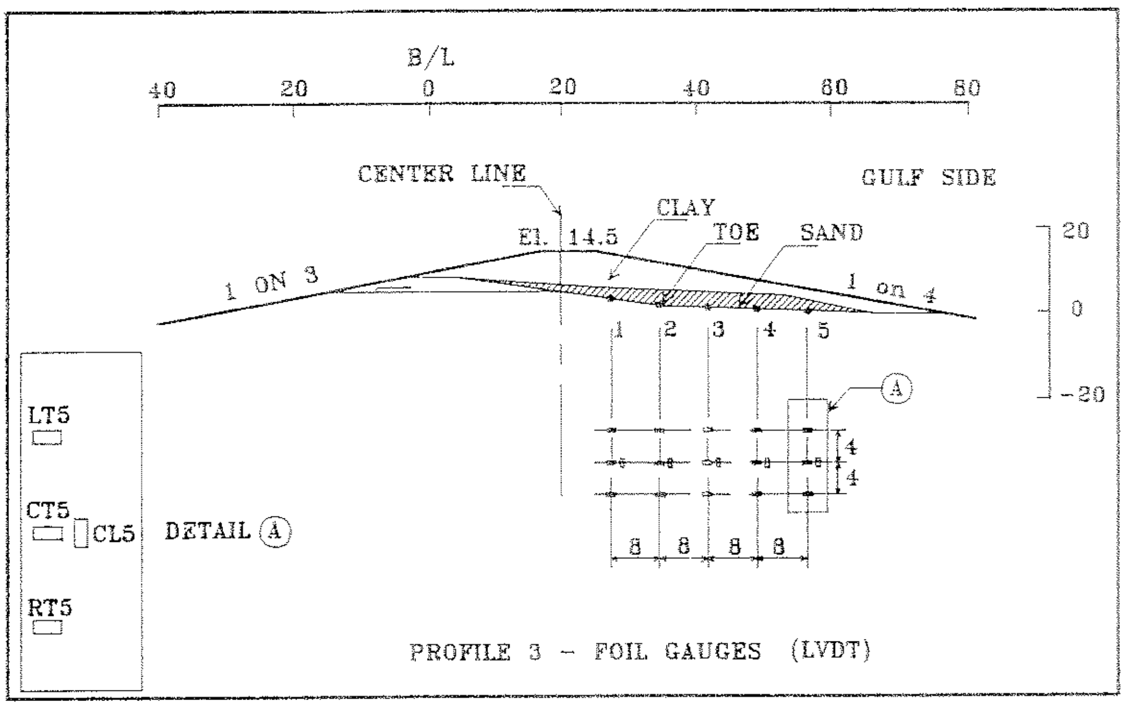

FIGURE 7 Layout of foil gauges.

offset value is approximately equal to that obtained in the laboratory tests conducted at Drexel University (5) on the fabric used in this study.

RESULTS OF THE FIELD TEST

The data obtained from the displacement transducers (WES gauges) were inconclusive: some of them jammed and the readings of others were inconsistent. The readings from the foil gauges were reasonably coherent but, unfortunately, no foil gauges were placed on the canal side or along the folds. The foil gauges measured a noticeable increase in strain with the progress of construction. The strains continued to increase slowly with time after the completion of construction for up to 400 days. However, no measurable increase was recorded by most of the gauges after 300 days. Table 4 shows a summary

TABLE 4 NET STRAINS IN THE FABRIC AFTER 300 DAYS

FOIL GAUGE GROUP	STRAIN READINGS OF FOIL GAUGES (%) CT	LT	RT	MAX DIFF	AVERAGE	CL
1	1.40	0.80	1.20	0.60	1.133	0.40
2	1.60	0.80	1.00	0.80	1.133	1.10
3	1.50	1.30	1.40	0.20	1.400	0.50
4	1.00	1.60	0.30	1.30	0.967	1.40
5	2.30	2.00	2.60	0.60	2.300	1.90
AVERAGE						1.06

of the strains in the fabric measured by the foil gauges after 300 days.

Typical lateral strains measured in the fabric by the foil gauges during the first 300 days are shown on Figure 8 for Group 5 (CT-5, LT-5, and RT-5). The readings of other groups of gauges followed similar patterns. The lowest strain of 0.8 percent was measured by gauges LT-1 and LT-2, and the maximum lateral strain was 2.6 percent at gauge RT-5. On average, a maximum strain of 2.3 percent was measured by Group 5 of lateral gauges, located near the fold on the Gulf side, and a minimum strain of 0.97 percent at Group 4. A gradual decrease in strain was observed toward the slope on the canal side, except for an inconsistent reading of Group 4. The 300 days readings of RT-4 and CT-4 were small (0.3 and 1.0 percent) in comparison with the reading of 1.6 percent at LT-4 (maximum difference of 1.5 percent) and the average readings of Groups 3 and 5 of 1.4 and 2.3 percent, respectively. In addition, lower strains were observed at Group 2 of gauges, located at the toe of the existing embankment (bend in the fabric), than in Groups 1 and 3, located before and after the toe.

The longitudinal strains in the geotextile did not follow a particular pattern and ranged from a minimum of 0.4 percent at CL-1 to a maximum of 1.9 percent at CL-5, as shown in Table 4. The variation in the readings could be attributed to the different amount of fabric stretching during construction or to the different soil conditions along the 500 ft (152.5 m) span. An average of 1.06 percent strain was observed along the longitudinal axis of the levee, which represents 46 percent of the maximum lateral strain measured by Group 5 and 94 percent of the minimum lateral strain measured by Group 1.

No appreciable movement was detected by inclinometers 11 and 12, installed near the canal. Consistent patterns of progressive lateral deformation toward the Gulf were observed

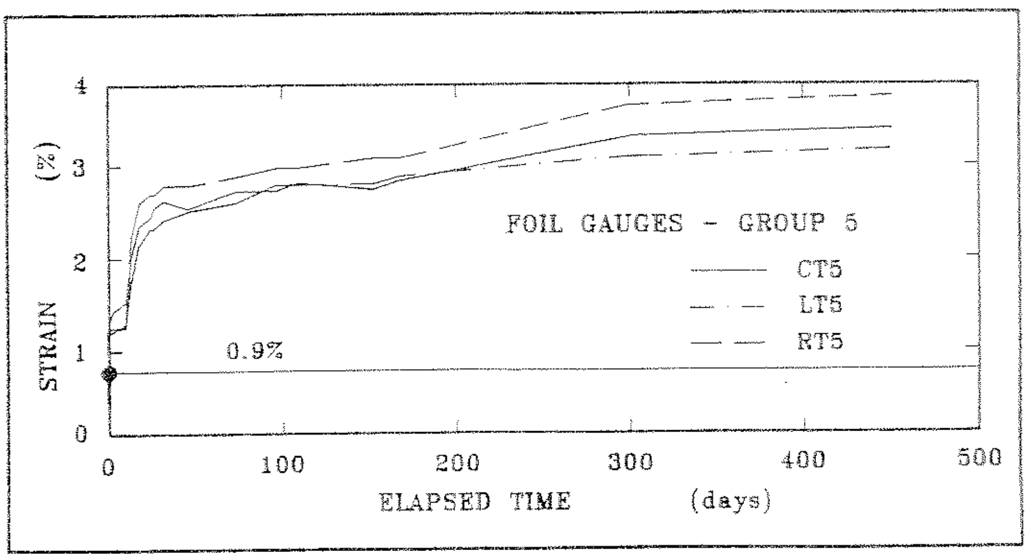

FIGURE 8 Strains in the fabric.

in all inclinometers with time, as shown in Figures 9 and 10. The horizontal movement measured by the inclinometers after 400 days is also shown in Table 5 at four selected elevations. Inclinometers 13 and 14, near the center of the embankment, experienced a horizontal movement toward the Gulf of 13 and 8.5 in. (330 and 216 mm), respectively. The average movement of 10.75 in. (273 mm) detected by these inclinometers is equivalent to 12.8 percent of the 7 ft (2.13 m) wide crown. The horizontal movement at the surface of the slope on the Gulf side was measured by inclinometers 15 and 16 as 14.3 and 10.5 in. (363 and 267 mm) toward the Gulf, respectively, for an average value of 12.4 in. (315 mm). This average reading represents 1.35 percent of the levee width of 76.4 ft (23.3 m) at that elevation (8.9 ft, 2.72 m). The horizontal movement at the fabric level was also toward the Gulf and ranged from 6.8 in. (173 mm) at inclinometer 14 to 14.4 in. (366 mm) at inclinometer 15. The average horizontal movement of 10.6 in. (0.27 m) corresponds to a 1.26 percent lateral strain in the 70 ft (24.4 m) long fabric. This value is consistent with the average reading of all foil gauges of 1.4 percent.

A plane of relatively large horizontal movement and potential failure surface was detected along the interface between the top CH clay layer and the first ML silt layer at approximately elevation −11 ft (−3.4 m). No appreciable horizontal movement, more than 0.5 in. (12 mm), was measured in the subsoils below elevation −30 ft (−9.1 m) by inclinometers 13 and 14, or below elevation −60 ft (−18.2 m) by inclinometers 15 and 16. All inclinometers showed movements in the subsoils toward the Gulf except for inclinometer 14, which

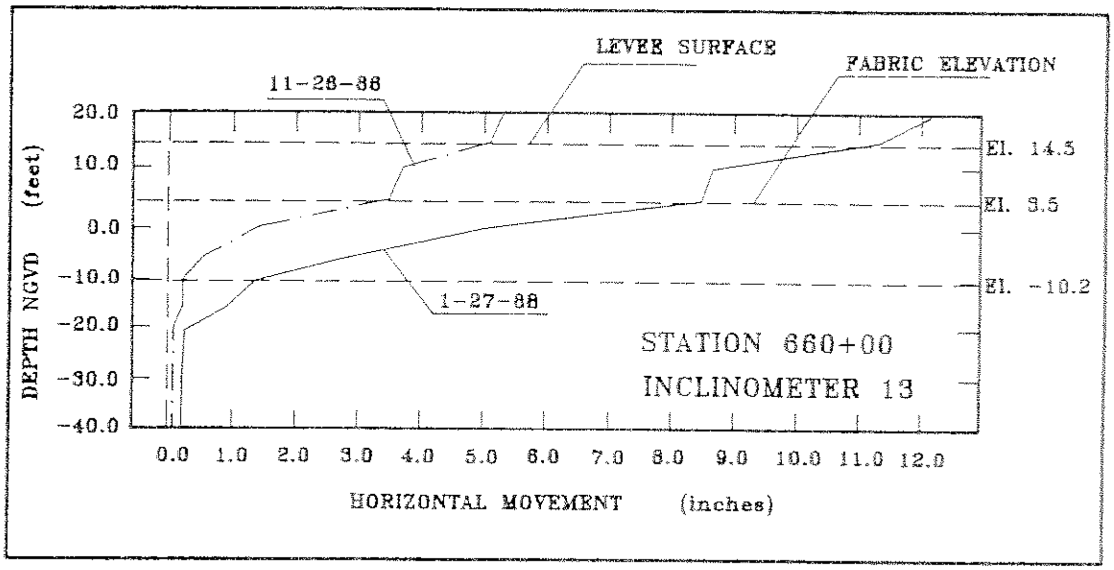

FIGURE 9 Horizontal movements below crown.

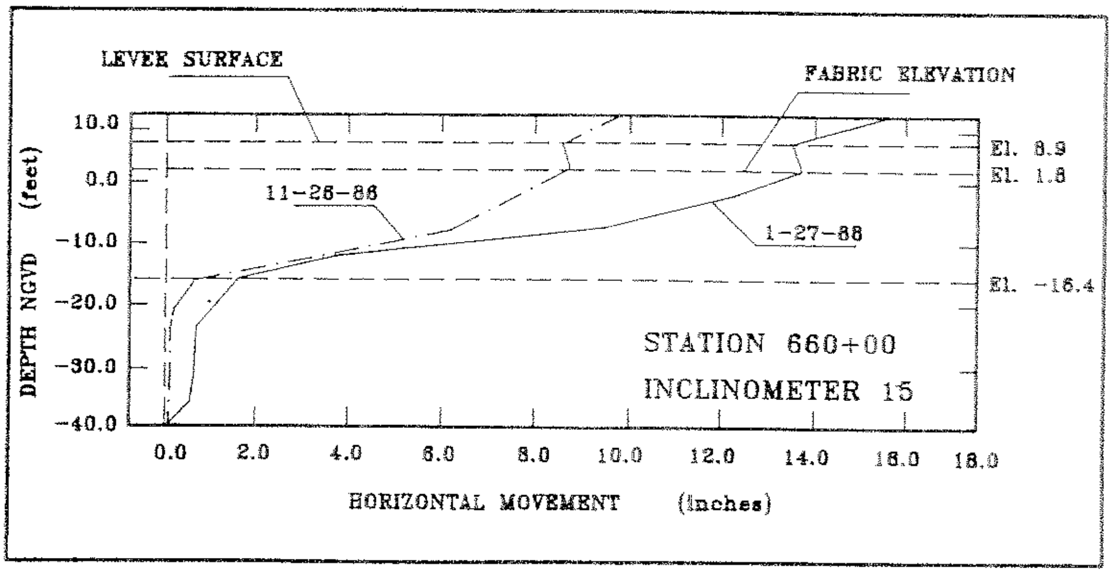

FIGURE 10 Horizontal movements at Gulf slope.

TABLE 5 HORIZONTAL MOVEMENTS IN THE SECTION

	HORIZONTAL MOVEMENT (inches)			
	LEVEE SURFACE	FABRIC	GROUND	SUBSOIL
Incl 13	13.00	7.30	5.1	1.30
CROWN	(El 14.5)[1]	(El 3.5)	(El 0.0)	(El -10.0)
Incl 14	8.50	6.80	4.0	0.250
CROWN	(El 14.5)	(El 3.5)	(El 0.0)	(El -10.0)
AVERAGE (CROWN)	10.75	7.90	5.0	0.525
Incl 15	14.30	14.40	13.0	4.00
GULF SLOPE	(El 8.9)	(El 1.8)	(El 0.0)	(El -11.0)
Incl 16	10.50	9.50	8.0	1.00
GULF SLOPE	(El 8.9)	(El 3.5)	(El 0.0)	(El -11.0)
AVERAGE (SLOPE)	12.40	11.95	10.5	2.50

[1] El. elevation in feet N.G.V.D.

showed movement toward the canal below elevation −30 ft NGVD (−9.1 m).

Fill hauling was completed on November 20, 1986, and design grade was reached one day later. No piezometer readings were taken until November 26 because of the extremely wet conditions at the site due to heavy rainfall. Table 6 shows the initial, peak, 300 days, and 400 days readings of the piezometers at the two stations. Figure 11 shows plots of the pore pressure distribution at station 660+00. Pore pressure increased gradually with the progress of filling and peaked when the height of the fill reached elevation 13.5 ft (4.1 m), then dissipated rapidly to small residual values above the initial readings. There was very slow change in the pore pressure after 80 days from the completion of construction, which reflects the low permeability of the subsoils.

The maximum average increases in the pore pressure at piezometers A, B, C, and D were 4.1, 9.03, 3.68, and 7.1 ft (1.25, 2.75, 1.12, and 2.17 m), respectively. A reduction in the pore pressure of 28 percent occurred in the four days after reaching the design grade of the levee. This rapid reduction is indicative of the quick process of consolidation in these clays. The maximum average increase in pore pressure of 9.03 ft (2.77 m) is equivalent to about 64 percent of the added fill of 8.5 ft (2.9 m) between elevations 5 and 13.5 ft (1.53 to 4.11 m) when the last reading was taken during construction. It should be noted that removal of the top soil of the existing levee may have resulted in negative initial pressures, and no readings were taken just after completion, which allowed pore pressures to dissipate somewhat. The highest residual pressure of 5 ft (1.53 m) was measured by the deepest piezometers, D, located in the bottom thick clay layer of very low permeability and poor drainage conditions. On the other hand, the quick dissipation of the pressure in piezometers B and C is due to their location between relatively highly permeable lay-

TABLE 6 PORE PRESSURES IN SUBSOIL

PIEZ No.	SOIL TYPE	TIP ELEVATION (ft)	PIEZOMETER READING (ft) INITIAL	13.5' FILL	300 DAYS	400 DAYS
1A	CH	-4.8	0.91	5.11	2.31	1.74
2A	CH	-5.0	0.91	4.91	2.21	2.38
1B	CH/ML[1]	-10.8	0.49	9.84	0.84	0.69
2B	CH/ML	-11.0	0.55	9.25	1.15	1.01
1C	ML	-20.8	-1.70	2.45	-0.45	-0.26
2C	ML	-20.0	-1.75	1.45	-0.75	-0.66
1D	CH	-30.3	0.06	7.36	3.26	2.43
2D	CH	-31.3	0.73	7.63	4.53	3.66

[1] Tip at interface of two layers

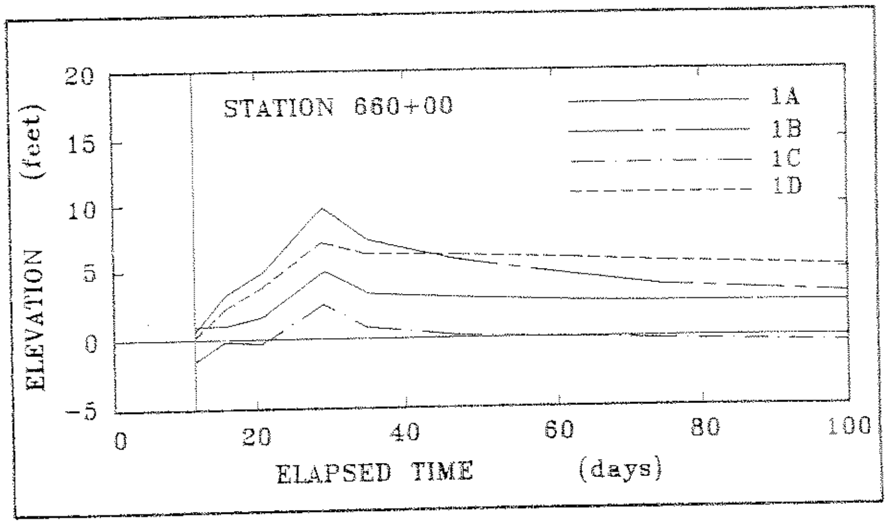

FIGURE 11 Pore pressures in subsoils.

ers of sand or silt. Piezometers A, by contrast, are located in the middle of the top CH layer, just below the clay levee, and accordingly measured much slower decays.

As expected, the pattern of settlement at the fabric level followed that of the pore pressures, with measurable increase in settlement recorded during the construction phase of lifts 2 through 8 (refer to Figures 2, 11, and 12). The rate of settlement decreased considerably after 150 days, and no measurable increase in the settlement was recorded after 300 days. The two settlement plates on the side slope, S-3 and S-4, showed identical readings, but an average difference of about 2 in. (50 mm) was observed between the readings of plates S-1 and S-2, located below the crown. The difference may be attributed to the variation in the thickness of the clay layers at the two stations or their drainage conditions. The maximum average settlement was 2.56 ft (0.78 m) under the slope on the Gulf side and 2.04 ft (0.62 m) below the crown. The settlement at the crown and the slope of the levee is equivalent to a loss of 18 and 38 percent of the added height of fill above each settlement plate, respectively. The net loss due to settlement at the crown as a ratio of the total height of the levee is 14 percent.

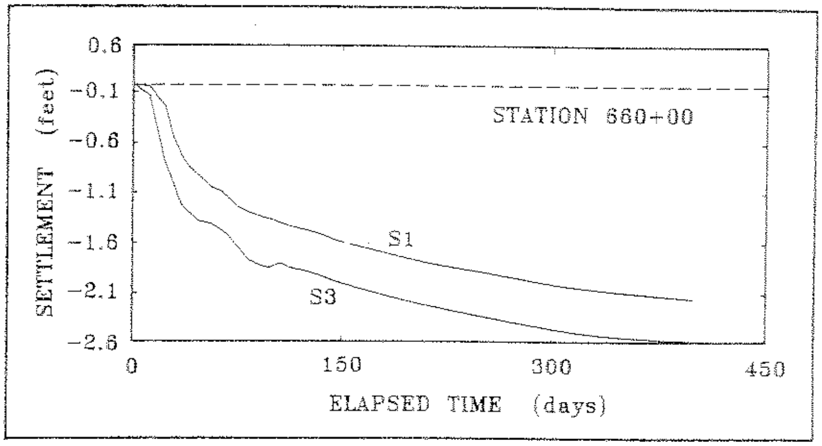

FIGURE 12 Measured settlement in the test section.

DISCUSSION OF THE RESULTS

The performance of most of the equipment was satisfactory. Readings were consistent and agreed well with the design and expected theoretical response. Further examination of the results yields the following remarks:

1. The goals of the project were achieved, considering the limited budget of $200,000 available for the field test. This budget represents a small fraction of the overall construction cost of the actual levee. The results of the test showed the feasibility of the design and construction, and accordingly it was decided to use geotextiles along the 13 mile stretch of levee. Use of geotextiles reduced the overall cost by 35 percent, required less than half the time to complete, and reduced the loss of marsh by 90 percent compared with a conventional levee. The reduction in the required construction materials for the levee is estimated to be about 60 percent.

2. The banks of the canal experienced very small movement due to the enlargement activities, which was originally a major concern for COE during the design phase. The data obtained from the test section were used to evaluate the design and to set guidelines for future designs. It is currently used in a research project to verify a new finite element computer program for the analysis of reinforced embankments.

3. Performance of the fabric was satisfactory, because the measured strains were much smaller than the long term design value of 5 percent. The other positive contribution of the fabric, besides providing a tensile resistance to the soil mass, was the reduction in size of the levee and accordingly the driving forces and settlements. The longitudinal strains in the fabric were relatively high for a plain strain type problem, as usually assumed in conventional or finite element analysis.

4. A fabric may stabilize an embankment, but the possibility of a deep-seated or shallow failure in the subsoil may still exist. The field test indicated a possible block failure along the interface surface between the relatively soft top CH layer and the much stiffer ML layer. This failure surface was examined to reassess the tensile requirements of the fabric computed by the wedge and circular arc analysis (*1-3*). Slope stability analysis was performed on the unreinforced and reinforced embankments using the wedge method and program UTEXAS-2 (*5*) based on the Spencer and Simplified Bishop methods. It was found that, to maintain a safety factor of unity, a fabric would require a tensile strength of 670 lb/in. (117 kN/m), based on the results of the wedge analysis, or 550 lb/in. (96.3 kN/m), based on the arc analysis. The required tensile strengths determined by the wedge and circular arc analyses were much larger than the actual stresses in the fabric, computed from the measured strains in the field. Hence, it was concluded that the fabric used in the field test was adequate.

5. Existing field conditions affected, to some extent, the movement pattern and performance of the test section. The subsoils under the canal slope and the crown of the enlarged levee were preconsolidated under the weight of the existing levee, whereas the Gulf slope was situated in the marsh over relatively virgin soils. Accordingly, smaller horizontal and vertical movements were recorded at the crown of the enlarged levee than were recorded at the slope on the Gulf side. This pattern of deformation indicated a possible general movement of the enlarged levee toward the Gulf over the degraded stiff surface of the existing levee.

6. The results of the project demonstrated the necessity of installing a backup system of measurement in a field test. In spite of the malfunction and erratic readings recorded by the WES strain gauges, sufficient measurements of the strains in the fabric were obtained from the backup system of foil gauges. Considering the high price tag and the elaborate work associated with such a massive project, the cost of a backup system becomes trivial.

7. The wide scatter in the data obtained from the soil investigation is evidence of the need for an extensive soil investigation in any major project. Laboratory tests on the clay samples from the top layer showed cohesion values (c) ranging from 90 to 490 psf (4.31 to 23.46 kN/m^2), with an average value of 290 psf (13.88 kN/m^2). A design based on an average value of a soil parameter may be a dangerous practice. For example, underestimating a strength parameter and overestimating a unit weight in a slope stability analysis may result in an overly conservative design, whereas the opposite may produce an unsafe design.

8. The recorded change in pore pressure and settlement with time were consistent with the sequence of construction. Pore pressure and settlement increased with the increase of the fill height up to the design grade. The residual values

measured by the piezometers after 400 days were indicative of the low permeability of the bottom thick clay layers. Faster dissipation was observed in the thin clay layers surrounded by more permeable layers of sand or silt. The measured settlements agreed reasonably well with the values calculated from consolidation theory.

9. Creep deformation should be accounted for in the design of earth structures founded on highly organic soils. Large time-dependent or creep deformation, accompanied by little or no dissipation of pore pressures, was observed in the test section. Pore pressure dissipated rapidly after completion of the section, then continued to dissipate at a much slower rate. Meanwhile, the horizontal and vertical deformations of the levee continued to increase with time at a higher rate than the dissipation of the pore pressure, as shown in Figures 10 through 13. Between 10 and 20 percent of the total settlement and 8 percent of the horizontal displacement of the levee occurred without any measurable change in the pore pressure.

10. Even though it was possible to obtain all necessary information from the field test, it is advisable for future tests to lay out the instruments in a symmetrical arrangement about the centerline of the embankment. Using more settlement plates distributed on both sides of an embankment would allow for plotting the displacement profiles under the embankment. Using inclinometers on both sides of an embankment would also provide a complete view of the overall deformed shape of the section. Using additional instruments in this project was not feasible due to budget constraints.

CONCLUSIONS

The results of the field test indicated the feasibility and benefits of using geotextiles in the construction of levees on soft soils. Using a fabric reinforcement reduced the estimated construction cost of the 13 miles of levee from $85 million to $54.2 million. The expected construction time was reduced from 13 years to 6 years, and only 100 acres of marshes will be used instead of 4,000 acres. Most of the instruments performed well throughout the duration of the project in spite of the harsh field conditions. Readings of the instruments indicated that the section has performed according to design. Displacements and settlements were within the expected range for this size levee and soil conditions. The strains in the geotextile were much smaller than the design value of 5 percent that was assumed in the limit equilibrium analysis. The results of the test will be used in the future to calibrate finite element analysis of geotextile reinforced embankments. Parametric studies will be performed using the finite element method to develop new guidelines for future designs.

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