

Field Performance of a Geogrid-Reinforced Embankment

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The field performance of a geogrid-reinforced levee test section built in Marrero, Louisiana, is discussed. The section was a full-scale model for a 1-mi-long levee built to protect a subdivision in Marrero from hurricane tidal waves. The section was built and monitored to assess the behavior of reinforced embankments on the extremely soft clays found in the Lower Mississippi Valley. The section was fully instrumented with inclinometers, settlement plates, and piezometers. Two rolls of geogrids were instrumented with strain gauges. Measurements were taken during and for a long period of time after construction. Field measurements included horizontal movements, vertical settlements, pore pressures, and strains in geogrids. Analyses of the data from the test section indicate that the geogrids allowed the use of smaller stabilizing berms. This conclusion was applied to the final design, resulting in appreciable savings. In addition, such large-scale tests are useful in developing design guidelines for the use of geosynthetics in protection levees in the Lower Mississippi Valley region.

Hurricane protection levees built in Louisiana must follow the guidelines of the Corps of Engineers (COE). These guidelines stipulate the height of the levee and the minimum factors of safety against stability failures. When a new levee was under consideration in Marrero, Louisiana, COE recommended that the crown of the levee be at least 3 ft above the standard project hurricane (SPH) tide level and that the slopes have a factor of safety of at least 1.3. Considering the extremely soft foundation soil at the site, these guidelines produced a levee ranging from 10 to 11 ft in height with rather flat slopes and large stabilizing berms.

To minimize the total cost of the project, the West Jefferson Levee District commissioned Eustis Engineering to examine three possible designs: (1) a standard COE uncompacted fill levee constructed from borrow cast from adjacent borrow pits, (2) a standard COE semicompacted fill levee constructed of hauled clay fill from an outside borrow source, and (3) a semicompacted reinforced fill. The three proposed cross sections are shown in Figure 1. The advantages of using as narrow a levee as possible are the lesser quantity of fill, the smaller base width of the levee, and the smaller amount of wetland destroyed by construction. The last two advantages stem from the location of the proposed levee with respect to an existing drainage canal (Figure 1), which will force the construction of the embankment in the marsh. The subdivision is situated to the east of the drainage canal and is to be protected from hurricane tidal wave coming from the marsh. The minimum distance between the centerline of the levee and the edge of

the canal, the quantity of fill material per foot of levee, and base widths for each design are summarized in Table 1.

In this particular case, the distance between the swamp and the subdivision is such that Design 1 would require reclaiming some swampland, at a considerable financial and ecological cost. It was felt that even with the additional expense of the reinforcement, the third option would be more economical and protect the environment. To assess the validity of the solution, it was decided to build a test section and monitor its behavior with a full array of instrumentation.

DESIGN OF THE TEST SECTION

The test section is now part of the north-south reach of the finished levee and was constructed in a marsh. It is located in the town of Marrero in Jefferson Parish (county) on the west bank of the Mississippi River. COE recommended using a SPH tide level of 7 ft National Geodetic Vertical Datum (NGVD) for that particular reach, yielding a crown elevation of 10 ft after all settlements have taken place.

Soil Conditions

The area of interest is located in the southeastern part of the State of Louisiana. It is on the central Gulf coastal plain on the modern delta of the Mississippi River, which projects into the Gulf of Mexico. The depositional history of interest for engineering application is from the Pleistocene to present time, since most of the sediments at the site were deposited during that period. The sediments are typically divided into natural levees, point bar, and backswamp deposits. The natural levees are the slightly elevated ridges that occur on both sides of the river. Point bar deposits are the direct results of the lateral migrations of the river. Over the past 5,000 years, seven major deltas are discernible in coastal Louisiana (1). During the migration process, erosion occurs from the banks and the coarsest materials are redeposited immediately downstream at the convex side of the river banks. Backswamp deposits are formed by the deposition of sediments in shallow ponded areas of overbank flows. They consist primarily of thinly laminated clays and silts that sometimes have a high organic content. Initial urbanization of the New Orleans region followed the natural levees and point bar deposits. However, as the city of New Orleans has expanded, it has become more and more common to build on marginal sites such as swampy and marshy deposits. The site of interest is located in a low-lying cypress swamp area subject to inundation from tidal flows.

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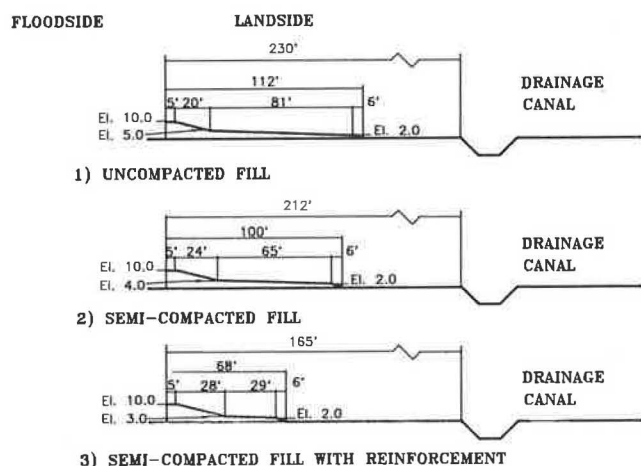


FIGURE 1 Cross sections of the alternative designs.

A total of 21 borings were drilled between January 9 and February 4, 1986, along the length of the proposed levee. The test section was constructed between Borings 12 and 13. The soil properties for the design of the test section were established from Borings 10 through 14. Boring 12 is a 5 in. diameter boring drilled to a depth of 100 ft. Borings 10, 11, 13, and 14 are 3 in. diameter holes drilled to a depth of 50 ft. Visual classification was performed at the site by a technician, and undisturbed samples were taken to the laboratory. The testing program consisted of Atterberg limits, unconfined compression tests, unconsolidated undrained triaxial tests, consolidated undrained triaxial tests, minivane tests, and consolidation tests.

The soil profile at the test section consists of 15 ft of extremely soft to very soft brown and gray clay, organic clay, and humus. This deposit is underlain by strata of very soft to soft gray clay with silt pockets to a depth of approximately 55 ft below the existing ground surface. From that depth and continuing to a depth of approximately 76 ft is a stratum of medium stiff gray clay with sand pockets and shell fragments. Pleistocene deposits are encountered at a depth of 76 ft. Pleistocene deposits are strata of stiff to very stiff overconsolidated gray and tan clays with sand lenses and clayey sand layers extending to a depth of 87 ft. These deposits form the foundations of most heavy structures in the New Orleans area. Figure 2 shows the profile along Borings 10 through 14.

When not flooded, the depth of ground water is at or very near the ground surface. The natural water content and wet unit weights are plotted versus depth in Figure 3. The large variation in water content (30 to 450 percent) and wet unit weight (67 to 115 pcf) are typical of such organic and humus layers. These extremely soft soils are responsible for most of the settlement in the area. The water content in the clay

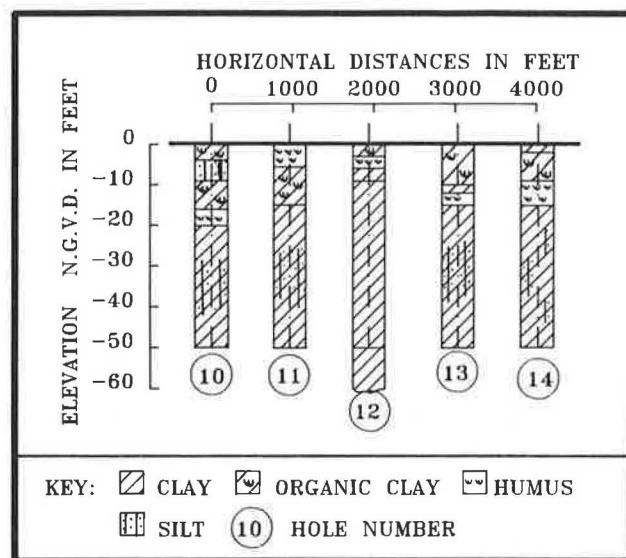


FIGURE 2 Soil profiles along Borings 10 through 14.

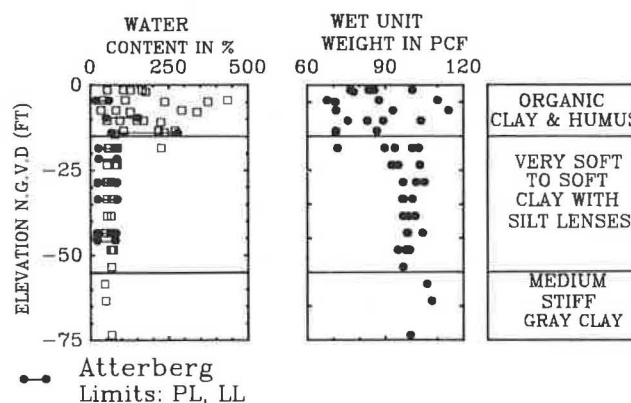


FIGURE 3 Natural water content and saturated unit weights.

stratum varies from about 40 to 80 percent. The Atterberg limits obtained on a few samples are also reported in Figure 3. Results from all shear strength tests are plotted in Figure 4. For design purposes, the 55 ft of soft material beneath the levee were divided into three sublayers:

1. a 15 ft thick layer of organic clay and humus with constant strength equal to 150 psf;
2. a 5 ft layer of very soft clay with constant undrained shear strength of 150 psf; and
3. a 35 ft soft to medium stiff clay with undrained shear strength increasing from 150 psf at elevation -20 ft to 400 psf at elevation -55 ft.

TABLE 1 CHARACTERISTICS OF THE THREE CROSS SECTIONS

Option Number	Levee Type	Distance from Drainage Canal (ft)	Quantity of Fill (yd ³ /ft)	Base Width (ft)
1	Uncompacted	230	36.7	224
2	Semicompacted	212	31.7	200
3	Reinforced	165	22.8	136

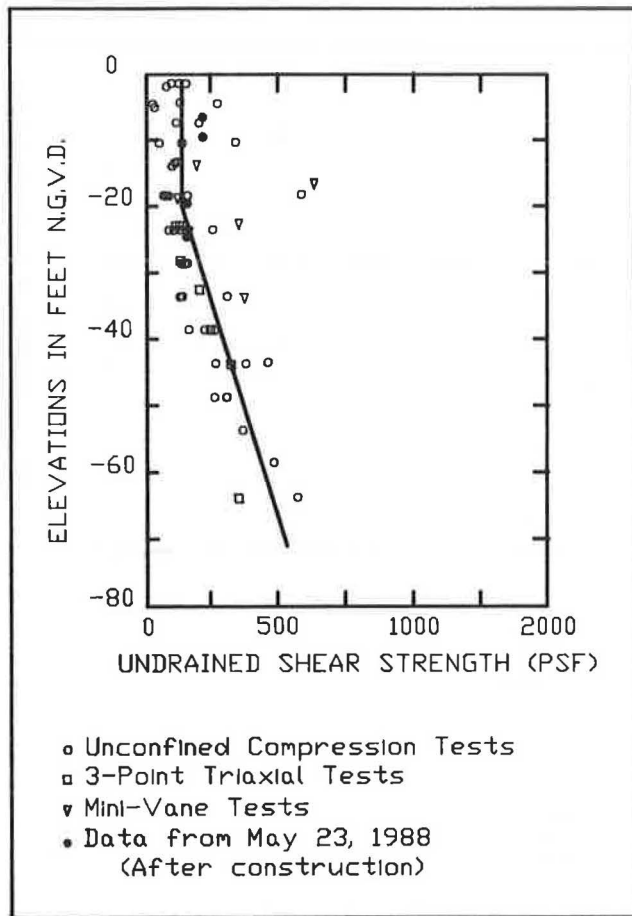


FIGURE 4 Shear strength profile.

Design and Stability Analyses

The test section is 350 ft long, 10 ft high, 10 ft wide at the crown, and 136 ft wide at the base, including the two stabilizing berms. The levee is constructed with a central core of hauled semicompacted clay fill placed on a working pad of hauled sand fill. The stabilizing berms are constructed of hauled uncompacted clay fill placed from the sand pad. The slopes of the core are 1 vertical to 4 horizontal and that of the berms are about 1 vertical to 3 horizontal, as shown in Figures 5 and 6. Figure 5 is a cross section of the test section and shows the location of the reinforcement, the type of fill material used, and the stratification beneath it. Figure 6 is a plan view of the test section. The reinforcement consists of two layers of high-density polyethylene Tensar SR2 (UX1500) geogrids located at elevation +1.5 and +3.0 NGVD. The design parameters of the three fills used to build the test embankment, the foundation soils, and the reinforcements are summarized in Table 2.

The stability of the levee was analyzed using the COE Lower Mississippi Valley Division Method of Planes Analysis, also commonly known as the Wedge Method (2). The levee was designed for the end of construction case utilizing a factor of safety of 1.3 for the levee itself and a factor of safety of 1.5 for analyses extending to the adjacent canal. The method is a limit equilibrium analysis and expresses the factor of safety as the ratio of the sum of the resisting forces, R , over the sum of the driving forces, D . The reinforcement is included as a tensile force, T , added to the resisting forces to bring the factor of safety to the target value. Initial analyses were performed on the unreinforced section to determine the required tensile strength of the geogrids. The critical sections are shown on Figure 7 and the factors of safety listed in Table 3. The tensile

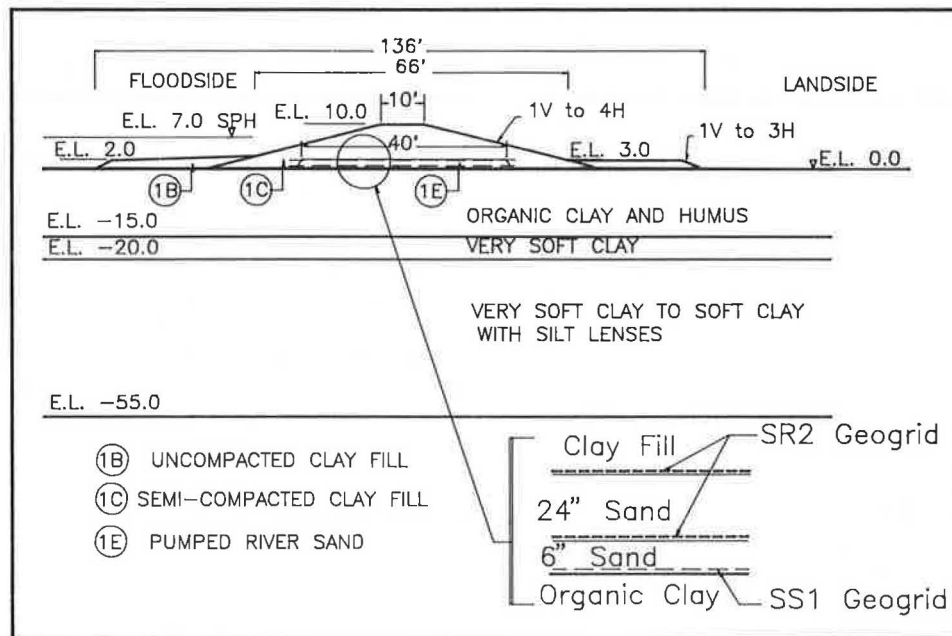


FIGURE 5 Cross section of test section.

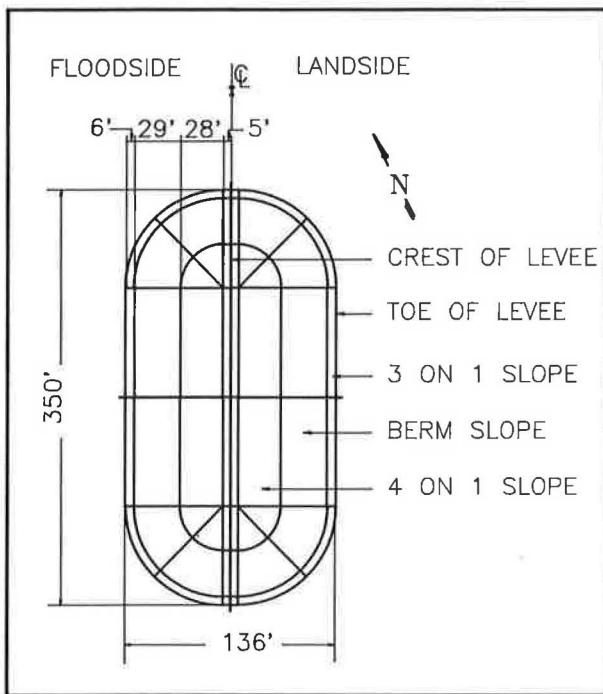


FIGURE 6 Plan view of the test section.

strength, T , needed to raise the factor of safety to 1.3 is calculated as follows:

$$T = 1.3 D - R \quad (1)$$

The length of embedment of the geogrids was computed to ensure adequate factor of safety against pullout. The factors of safety of the previous section computed with the reinforcement are listed in Table 3.

TABLE 2 MATERIAL DESIGN PARAMETERS

Stratum Number	Material	Saturated Unit Weight (pcf)	Cohesion (psf)		Friction Angle (degrees)
			Center	Bottom	
1B	Clay Fill A-6 or A-7 Hauled Uncompacted	100	200	200	0
1C	Clay Fill A-6 or A-7 Hauled Semicompacted	105	400	400	0
1E	Sand Fill A-3 Trucked Pumped River Sand	120	0	0	30
2	Humus/Organic Clay	74	150	150	0
3	Clay	95	150	150	0
4	Clay	98	275	400	0
—	Geogrid Tensar SR2 (UX1500)	Ultimate strength (at 15 percent strain):			6500 lbs/ft
		Tensile strength at 2 percent strain:			2055 lbs/ft
		Tensile strength at 5 percent strain:			3900 lbs/ft
		Long-term design load:			2350 lbs/ft
		Extension at 40 percent of maximal load:			3 percent
		Tensile modulus at 2 percent:			102,7506 lbs/ft
—	Geogrid Tensar SS1 (BX1100)	Ultimate strength (at 14 percent strain):			840 lbs/ft
		Ultimate strength at 2 percent strain:			280 lbs/ft
		Ultimate strength at 5 percent strain:			570 lbs/ft

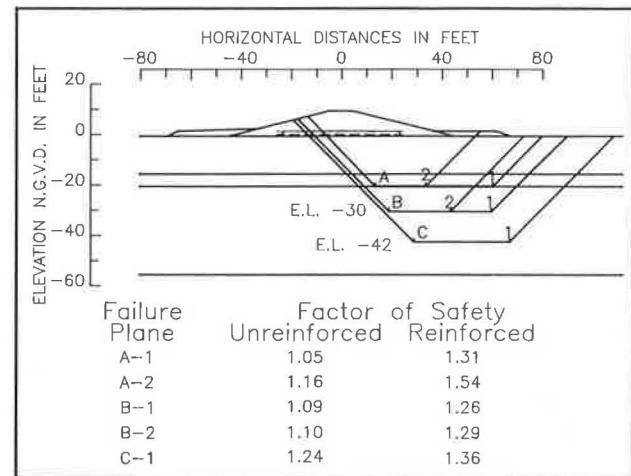


FIGURE 7 Critical failure surfaces.

Because the failure masses are expected to move towards the landside (east) of the levee, both layers of geogrid were placed asymmetrically to ensure proper anchoring. The bottom layer is 49.2 ft long and starts 25 ft left of the section centerline, whereas the top layer is 32.8 ft long and starts 18.4 ft left of the centerline.

Construction

Construction began on October 26, 1987. With the water table at ground surface elevation, the layer of organic clay and humus was extremely soft and workmen would sink to their knees while walking around. A base reinforcement of Tensar SS1 (BX1100) geogrid was laid on the ground surface to improve working conditions. The SS1 was rolled parallel to the test section centerline under the core and the access road centerline. The base reinforcement was rolled perpendicular to the levee centerline beneath the uncompacted stability berms. A 3 ft overlap was used for adjacent panels of SS1. The SS1 geogrids were not included in the stability analyses of the levee section.

A layer of sand about 6 in. thick was laid on top of the SS1 grid to form a base for the first layer of SR2 geogrid. The sand was trucked to the site, dumped and dozed initially with a small track dozer (John Deere 450). However, this dozer tended to sink and create a large mud wave; it was eventually replaced by a larger dozer (Case 850C). An additional 2 ft of sand was then hauled over the geogrid layer to form the access road for construction of the berms and slopes of the levee.

The second layer of SR2 was laid on the sand, and construction proceeded with hauling of the clay fill. The SR2 grids were rolled perpendicular to the centerline of the levee in the main part of the section and parallel to the centerline at both ends. Adjacent strips of SR2 were rolled butt to butt and not overlapped. The clay was hauled from the stockpile using five dump trucks and pushed in place using two dozers (Case 1150C and 450C). The only compaction effort applied to the clay fill was that of the earthmoving equipment. This is common practice for the levees built under COE supervision. Experience has shown that adequate compaction is obtained. The construction history of the test section is summarized in Table 4.

Settlement Analyses

Most of the settlement of the test section will be caused by compression of the soft foundation soils. Three consolidation tests were performed by Eustis Engineering on samples recovered from depths of 14, 23.5, and 43.5 ft in Boring 14. The first sample was from the highly organic clay layer with initial water content of 258 percent, void ratio of 6.43, and saturated unit weight of 74 pcf, and which showed tremendous compressibility. The other two samples were from the underlying soft clay and exhibited less compressibility. A fourth consolidation test was performed by COE on a sample recovered from the hole drilled for inclinometer I-1 (see Figure 8 for location).

Settlement of the levee was estimated using Terzaghi's one dimensional consolidation theory and the results of the consolidation tests. It was computed that about 6 ft of settlement would take place under the crown of the levee and about 1.6 ft at the toe of the berms. About 4.5 ft of the total crown settlement is expected to occur in the organic layer. Further, given the extreme softness of the organic layer, it was anticipated that up to 2 ft of settlement will occur very rapidly, some of it during construction. Consequently, a new lift of fill will have to be added to bring the levee up to the required

TABLE 4 CONSTRUCTION STAGES

Date (1987)	Construction Activity
10-26	Access road graded
10-27	SS1 Roll placed at elevation 0.0 ft
10-28	Sand hauled on top of SS1
10-29 to 10-30	1.5 ft sand blanket built
10-31	First layer of SR2 installed at elevation +1.5 ft LADOTD installs the two settlement plates
11-1 to 11-2	Sand hauled to elevation +2.0 ft Clay hauled to build sides of levee
11-3 to 11-4	Sand blanket at elevation +3.0 ft
11-5 to 11-7	Clay hauled for construction of levee and berms
11-8 to 11-11	No construction work
11-12 to 11-13	Construction of berms
11-14 to 11-18	No construction work
11-16	COE installs the four piezometers
11-17	COE installs the three inclinometers
11-19	Second layer of SR2 installed at elevation +3.0 ft
11-20	Clay hauled to build levee
11-21 to 11-22	No construction work
11-23	Levee brought up to elevation +5.0 ft
11-24 to 11-25	Levee brought up to elevation +7.0 ft
11-26 to 11-29	No construction work
11-30	Levee brought up to elevation +9.5 ft
12-1	No construction work
12-2	Levee brought up to grade and shaped (end of construction)

10 ft. This aspect of the project is discussed further in a later section of this paper. The coefficient of consolidation of the soft clay was computed to be 10.1 ft²/yr.

INSTRUMENTATION

Rationale

The purpose of the test section was to investigate the feasibility of using geogrids for the Louisiana Department of Transportation and Development (LADOTD) and the West Jefferson Levee District. An ancillary purpose was to demonstrate compliance with applicable COE design criteria. The concern of these agencies is the stability of the constructed levee and its ability to perform adequately if a hurricane hits the area. Stability means that no failure mechanism such as those examined above should develop below ground surface and that the geogrids should not be stressed near their ultimate capacities. Consequently, it was decided to monitor the lateral movement below ground surface and the strains in the geogrids. Ability to perform adequately means that the crown of the levee should be at the prescribed SPH tide level. Consequently, it was decided to monitor vertical movements of the levee and pore pressure throughout the foundation to predict future settlement and estimate gain in shear strength with time.

Layout

The instrumentation package consists of 3 inclinometers, 2 settlement plates, 4 piezometers, and 34 strain gauges installed on the geogrids. Figure 8 shows the layout of the geotechnical

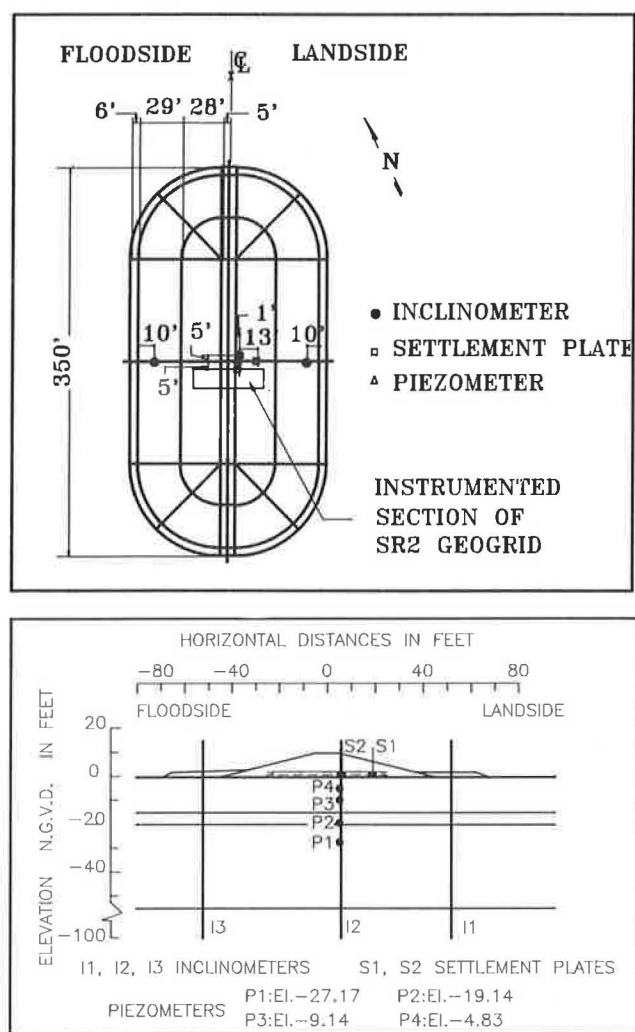


FIGURE 8 Layout of the instrumentation: (top) plane view, (bottom) cross section.

instrumentation installed at the test section. Figure 9 shows the position of the strain gauges on the two layers of Tensar SR2 geogrid. The installation and monitoring of the instruments was a collaborative effort of four agencies: COE, the Louisiana Transportation Research Center (LTRC), LADOTD, and Eustis Engineering (EE), as summarized in Table 5.

Inclinometers

The inclinometers (SINCO Model 50320 probes) were installed by COE at the locations shown on Figure 8. EE monitored and reduced the data from the inclinometers using a magnetic tape recorder and reduction program. The three 3.14 in. tubes are located on a line across the test section and extend 100 ft below the original ground surface. Inclinometer I-1 is located on the landside (east side) 52 ft away from the centerline of the levee; inclinometer I-3 is placed symmetrically on the floodside (west side) of the levee; and inclinometer I-2 is

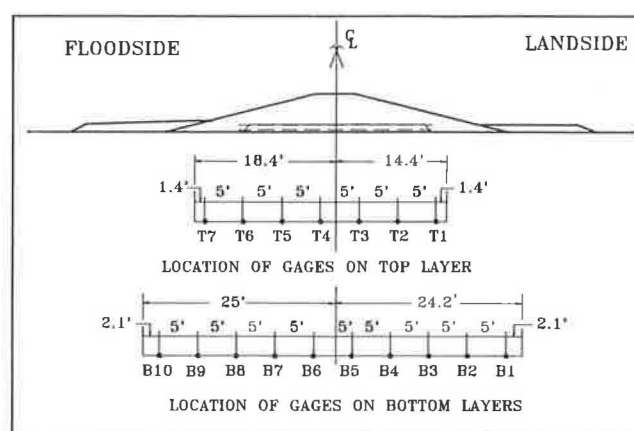


FIGURE 9 Location of the strain gauges on the geogrid.

TABLE 5 SUMMARY OF INSTRUMENTATION INSTALLATION AND MONITORING

Instrument	Installed by	Monitored by
Inclinometers	COE	EE
Piezometers	COE	LTRC
Strain gauges	LTRC	LTRC
Settlement plates	LADOTD	LTRC

located 6 ft east of the centerline of the levee and 5 ft north of the center of the test section.

The specifications called for the grooves in tubes to be oriented parallel and perpendicular to the long axis of the levee. However, the tubes for inclinometers I-1 and I-2 were not installed properly, and the direction of the grooves are at angles of 12 and 20 degrees counterclockwise, respectively. Inclinometer I-2 was hit during construction and the deflections recorded in the first 10 ft may not be representative of actual displacements.

Piezometers

The four piezometers were 18 in. long, with 2 in. diameter slotted (0.01 in.) screens and 0.75 in. diameter risers. They were installed by COE and monitored by LTRC. They are set 6 ft east of the crown centerline on the middle section of the levee at elevations -27.17 (P-1), -19.14 (P-2), -9.64 (P-3), and -4.83 (P-4) ft NGVD. The piezometers tips were placed in a 6 in. diameter hole and backfilled with 4 ft of concrete sand.

Settlement Plates

The settlement plates consisted of 2 ft by 2 ft by 0.25 in. wrought iron plates with 2.5 in. riser pipes. Two plates were installed on a sand blanket at elevation +1 ft NGVD by the LADOTD and monitored by LTRC. Plate S-1 is set 19 ft east of the levee crown along the short axis. Plate S-2 is located 5 ft south of the short axis and 6 ft east of the levee crown centerline.

Strain Gauges

The Louisiana Transportation Research Center installed and monitored the strain gauges on the geogrids. The gauges (Micro Measurement Model No. CEA-13-250UM-350, using M-Bond GA-7 adhesive and 326-D-SV wire leads) were installed on panels of primary reinforcement (SR2). The exact locations of the gauges are shown in Figure 9. Note that there is a top gauge and a bottom gauge at each location.

OBSERVED PERFORMANCE

In the following discussion the reader is referred to Figure 8 for the direction of the readings. It is assumed that the levee runs north-south, and consequently the floodside is located on the west side of the levee and the protected or landside is on the east side of the levee.

Lateral Movement

The observed lateral displacements for the three inclinometers are given in Figures 10 to 12. The first measurements were taken on the morning of November 23, 1987, when there was already about 4.5 ft of fill placed at the site. At this time there was a small but measurable lateral displacement away from vertical. The second measurements, except inclinometer I-3, were taken on the afternoon of November 23, when an additional foot of fill had been hauled into place. There is a very small increase in lateral movement in inclinometers I-1 and I-2. The next measurements were taken on November 26, 1987, when the impact of the 7.5 ft of fill was being felt. The recorded displacements show a logical movement pattern within the foundation. Inclinometer I-1, located on the east

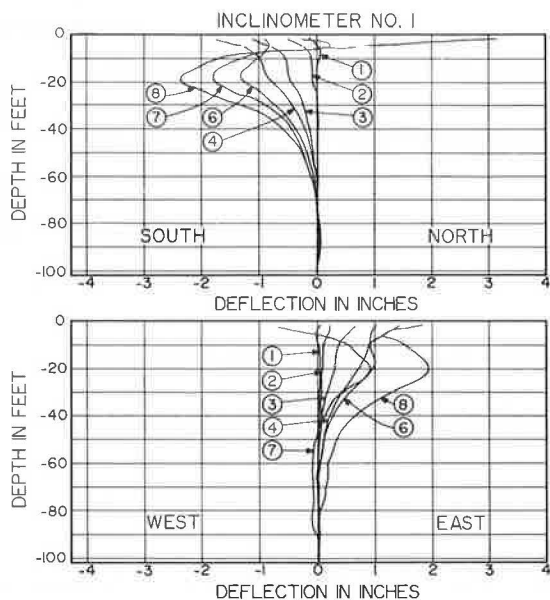


FIGURE 10 Lateral movement at inclinometer I-1. 1, November 23, 1987; 2, November 23, 1987; 3, November 24, 1987; 4, November 25, 1987; 5, January 4, 1988; 6, February 23, 1988; 7, February 24, 1989; 8, April 28, 1989.

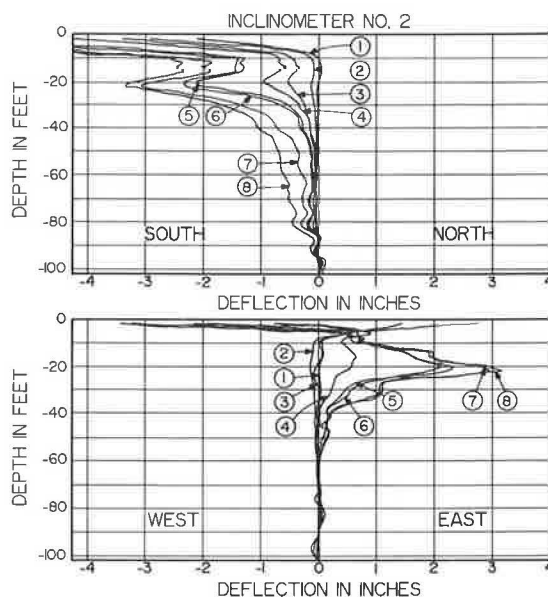


FIGURE 11 Lateral movement at inclinometer I-2. 1, November 23, 1987; 2, November 23, 1987; 3, November 24, 1987; 4, November 25, 1987; 5, January 4, 1988; 6, February 23, 1988; 7, February 24, 1989; 8, April 28, 1989.

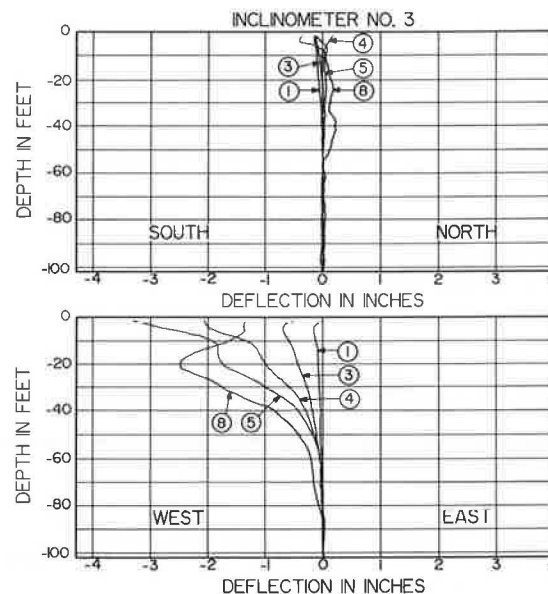


FIGURE 12 Lateral movement at inclinometer I-3. 1, November 23, 1987; 2, November 23, 1987; 3, November 24, 1987; 4, November 25, 1987; 5, January 4, 1988; 6, February 23, 1988; 7, February 24, 1989; 8, April 28, 1989.

side of the levee, shows eastward (landside) movement, and I-3, located on the west side of the levee, shows westward (floodside) movement. Note that due to the improper orientation of I-1 and I-2, a rather large deflection is reported in the north-south direction. I-3, which was correctly placed, shows very little deflection along the north-south axis. This expresses the near-plane strain behavior of the central section of the embankment.

The fourth set of readings were taken when the levee was completed. At that time maximum deflections reached 0.33 in. towards the landside at I-1, 0.6 in. towards the landside at I-2, and 1.15 in. towards the floodside at I-3. However, I-1 and I-2 also show 0.9 in. and 1.0 in. of southward deflection, mainly because of the misplacement of the tubes. The movements in the direction perpendicular to the levee (east-west or landside-floodside) should be slightly greater than indicated and can be computed from simple vector analysis. Corrected readings are 0.51 in. landside at I-1 and 0.9 in. landside at I-2.

The fifth set of measurements shown for the inclinometers were taken 40 days after completion of the test section. Maximum deflection in the east-west direction occurred at depth 20 ft and are 1 in. landside at I-1, 2.2 in. landside at I-2, and 1.9 in. floodside at I-3. Landside movement at I-1 and I-2 should be larger, considering the twisted position mentioned previously and the large deflection picked up in the north-south direction. The deflections recorded 60 days after construction are nearly equal to those measured at 40 days and follow the same pattern.

The seventh set of readings were taken 435 days after completion of the test section, just before the final lift was hauled into place to bring the crown of the test section back to 10 ft. This means that the whole levee was completed and tied to the test section, which at that time, because of settlement, was about 2.5 ft lower than the required 10 ft. The last set of deflection curves, shown in Figures 10–12, was recorded on April 28, 1989, or 2 months after construction of the final lift. Appreciable additional horizontal movement occurred near elevation –20 ft, which corresponds to the anticipated most critical failure surface A-1 shown in Figure 7. The deflections corrected for skewness of the tubes are 2.41 in. landside at I-1, 4.05 in. landside at I-2, and 2.47 in. floodside at I-3.

Settlement

The settlement plates were not monitored until 20 days after their installation. At that time about 4.5 ft of fill had been hauled into place, and it is reasonable to assume that some settlement has already taken place. Settlement and height of fill at the location of the plate are plotted versus time on Figure 13 for plates S-1 and S-2. At 140 days after the initial readings, the recorded settlements were 2.17 ft at S-1 and 3.1 ft at S-2. It is noteworthy that about half of the total settlement occurred during the construction period of about 15 days.

Pore Pressure

Water elevation in the piezometers was monitored during construction of the embankment and for 5 months following completion of the test section. The data are reported on Figure 14 as the water rise in each of the tubes plotted versus time. To facilitate the discussion, the height of the embankment at the location of the piezometers is also reported in Figure 14. The direct relationship between embankment height and excess pore pressure is easily seen: first, a rapid rise in pore pressure with construction; and then a time-dependent

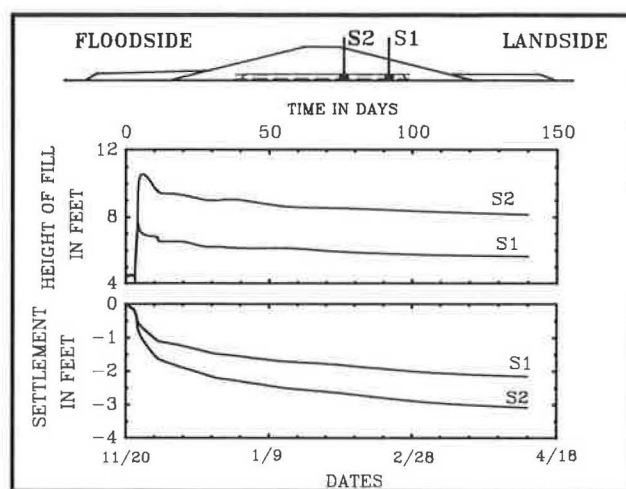


FIGURE 13 Settlement versus time at plates S-1 and S-2.

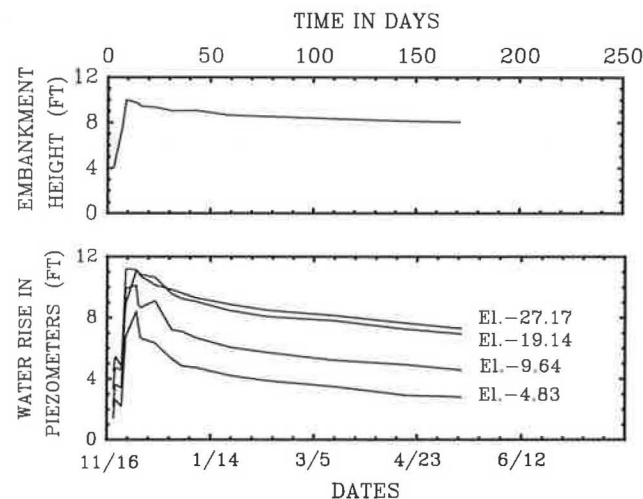


FIGURE 14 Piezometer readings versus time.

dissipation due to horizontal and vertical seepage as the soft layers consolidate. Figure 15 presents the same data for the first 20 days following installation of the piezometers (November 16 through December 6, 1987). The initial readings were taken on November 19 and the excess pore pressure reported represents the 4 ft of material already in place. The graphs in Figure 16 show clearly the effect of the interruption in construction from November 20 to November 22 and the rapid rate of construction from November 23 to November 26. Piezometers P-3 and P-4, which are the shallowest and are located in the organic layer, show the most sensitive response to construction activities. They show drops of 1.6 ft and 1.8 ft between November 30 and December 2, when the levee was shaped to grade. This normally requires the scraping of excess fill that was hauled and compacted into place. It is noteworthy that after 30 days the rates of dissipation are fairly similar for all piezometers.

The pore pressure dissipation curves were used to compute the coefficients of consolidation in the two layers using the procedure described in Navy Manual DM-7.1 (3). In the organic

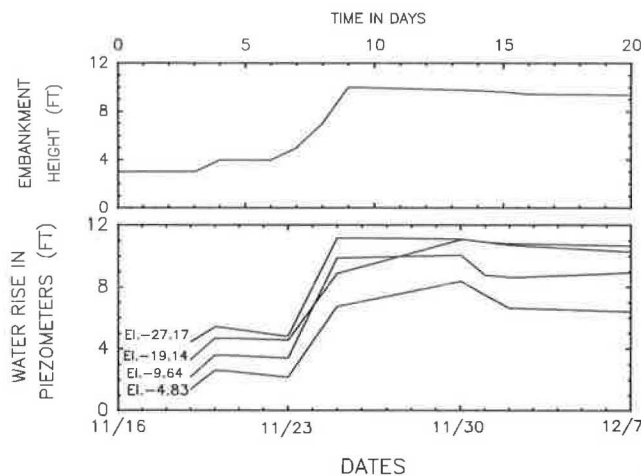


FIGURE 15 Piezometer readings versus time (reduced scale).

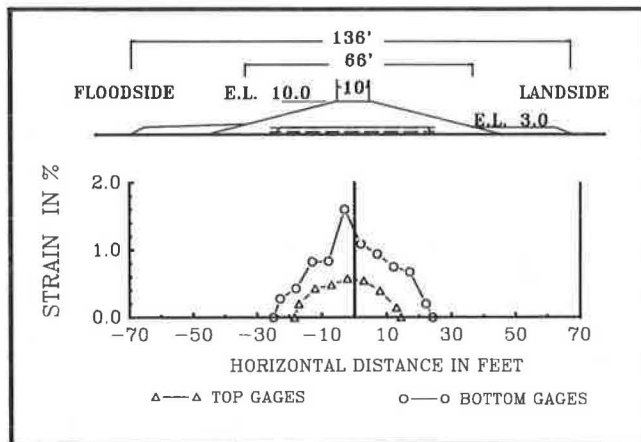


FIGURE 16 Strain in top and bottom layers of geogrids.

clay, C_v was calculated to be about 263 ft²/year. In the soft clay layer, C_v was computed to be about 58 ft²/year.

Strain Gauges

The initial readings were taken on November 19, 1987, and followed the same schedule as the piezometers. The strains recorded on April 29, 1988, in the top and bottom layers are plotted on Figure 16. The maximum strains were recorded at the gauges located near the centerline of the levee. The maximal total elongation of the bottom layer can be estimated to be about 5.0 in. and that of the top layer to be about 1.6 in. Strain gauge B-5 on the bottom layer recorded 1.669 percent and gauge T-4 on the top layer recorded 0.574 percent. The deformation with time of the geogrid follows closely the construction sequence. Figure 17 shows the increase in strain versus time for gauges B-5 and T-4. The gauges performed very well and were still operating at time of writing (November 1989). Note that each reading plotted is the average of the top and bottom reading at each location.

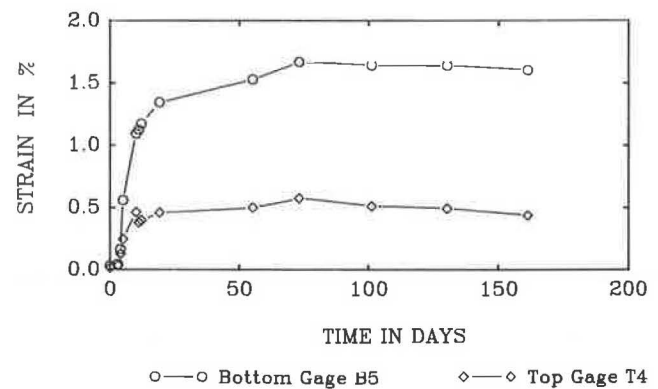


FIGURE 17 Strain in geogrids versus time.

DISCUSSION OF RESULTS

The stability analyses pointed out a potential critical sliding zone between elevation -20.0 and -30.0 ft. The inclinometer readings corroborated the prediction by indicating the largest horizontal movement at elevation -20.0 ft. The total movement of 2 in. eastward at I-1 and 2.5 in. westward at I-3 are not surprising in the type of soils underlying the levee. In addition, the total extension of about 5 in. measured in the bottom layer of geogrid agrees with the inclinometers, indicating a spreading of the base of the embankment.

The settlement analyses predicted a total compression of 5.6 ft under plate S-1, with about 4.2 ft to occur in the organic. At the end of the monitoring program (140 days), the maximum settlement under plate S-1 was 3.17 ft. Based on the laboratory C_v value, 140 days correspond to about 6 percent consolidation in the clay layer or 0.072 ft. Based on the field values of C_v , the 140 days correspond to 16 percent consolidation in the clay and 45 percent in the organic, for a total settlement of about 2.1 ft. It is important to note that the field values of C_v do not account for the effect of pore pressure dissipation during the construction stages and work stoppage. The measured 3.17 ft do not account for the 20 days of construction or for the settlement that occurred under the 4 ft of fill in place at the time the first readings were taken. The very soft organic deposit will start to experience compression as soon as earthmoving equipment rolls over the site and will compress as construction proceeds. This is further illustrated by the maximum pore pressure measured at the piezometer, which is lower than the applied overburden pressure of 10 ft of fill. In view of the working conditions at the Westminster levee, it is very likely that more than 4 ft of fill were placed to reach elevation 4.0 ft. The factor of five between the laboratory and field value of C_v for the clay is probably caused by three dimensional dissipation and by the presence of the silt lenses and seams within the layer, which increases permeability.

The average strains measured in the Tensar SR2 grids convert to about 1,700 lbs/ft in the lower layer and 600 lbs/ft in the top layer, for a total of 2,300 lbs/ft. This total tensile force is approximately the same as the allowable long-term stress in a single layer of SR2 geogrid (Table 2).

On May 23, 1988, or close to 6 months after the end of construction, two borings were drilled along the centerline of

the test section; the first boring 20 ft north of the central section and the second boring 30 ft south. The results of the shear strength tests are plotted in Figure 4 and clearly indicate a gain of strength in the organic layer due to consolidation. The logs of the borings also indicate that the bottom of the sand blanket is now around elevation -3.0 ft, further evidence of the total settlement measured by the plates. The gain of strength is on the order of 75 percent and was combined with the findings about the stresses in the geogrid to come up with a design for the full levee. Since the top layer of geogrid was only slightly stressed and the bottom layer stressed to a level less than the allowable long-term stress, the final specifications called for the use of the same section shown in Figure 7 but with only the bottom layer of geogrid. It was felt that the time-dependent strength gain of soil would not produce any load above the long-term limit of the geogrid.

CONCLUSIONS

The test section discussed in this paper was a full-scale model for the proposed one mile long Westminster hurricane protection levee. The section was fully instrumented with strain gauges, inclinometers, settlement plates, and piezometers.

The main benefits of the instrumentation program conducted at the test section of the Westminster levee are of economical and environmental nature. Results of the instrumentation program indicated that the design was stable. The

movements measured are compatible with what is expected in similar soil in southwest Louisiana. The mile long levee was built using the semicompacted fill with one layer of geogrid. The time dependent gain of strength was accounted for in the analyses. The use of smaller than usual stabilizing berms resulted in appreciable savings, despite the additional cost of the reinforcement. The design selected resulted in less destruction of ecologically important marshland.

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