TWITCHELL ISLAND LEVEE IMPROVEMENTS, NORTHERN CALIFORNIA DELTA AREA

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Abstract

The California Delta area, located westerly and southerly of Sacramento, California, has peat and organic soil deposits ranging in thickness from a few metres to about 15 m. The peat and organic soils were deposited in Holocene time and are typically weak and highly compressible. The area includes a system of islands and tracts, protected by perimeter levees, with ground elevations about 3 to 4 m below sea level. Typically, the levees exhibit lower stability due to weak foundation soils.

The stability of about 6 km of the Twitchell Island levees along the San Joaquin deep water shipping channel was improved in 1992 by constructing a landward toe-berm over approximately 7 m of soft and compressible fibrous peat and organic soils underlain by stronger clays and sands. The project required about 300,000 m^3 of mostly fine sand fill and was completed over a period of 6 months. The sand fill was dredged from a water storage reservoir, barged to the site, unloaded with a clam shell crane and bulldozed in place.

For design, soil strengths were estimated using stress history and normalized engineering property methods (Ladd and Foott, 1974). Undrained shear strengths of the peat ranged from about 15 to 40 kPa. Toe berm widths ranged from about 20 m to 35 m. Slope stability factors of safety were improved from between 1.1 to 1.2 to about 1.3. The toe berms were constructed in two stages to avoid overstressing the peat foundation soils; the thickness of each stage was about 120 cm. Ground settlements in the range of 10% to 20% of the peat thickness were measured. Some ground cracking, fissuring, sinkhole formation and localized slope movements occurred during the construction. Cracking and movements usually occurred soon after toe berm loads were applied and then decreased with increasing time.

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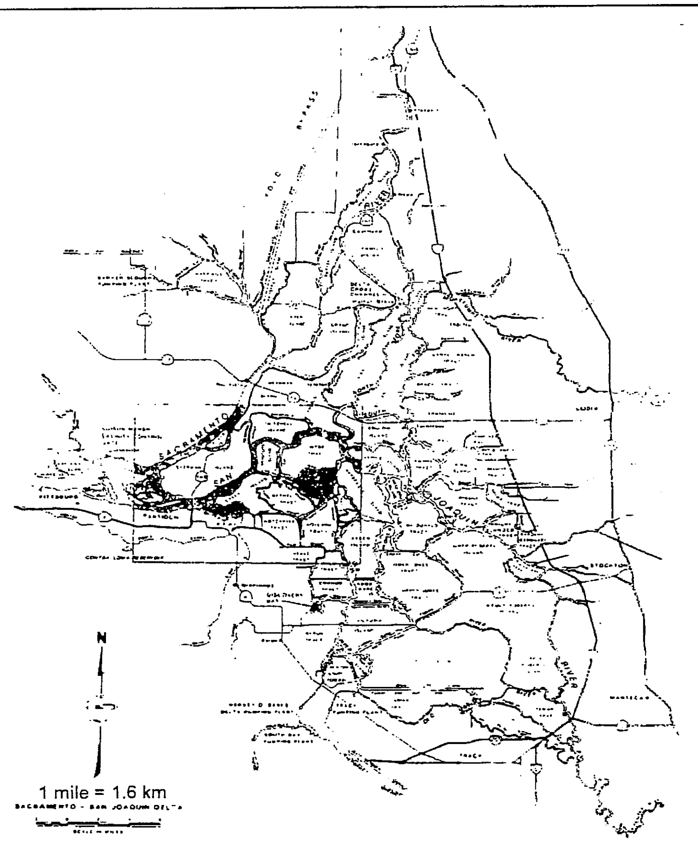
Introduction

Twitchell Island is located in the delta of the Sacramento-San Joaquin Rivers, immediately east of San Francisco Bay as shown in Figure 1. Twitchell Island is one of many islands in the delta that is protected by levees from surrounding rivers and sloughs. In the late 19th century, these areas were marshlands, flooded during high tides and high river flows. Recognizing the agricultural value of the rich peat soils, farmers reclaimed the land for agriculture by constructing systems of small dikes and drainage ditches. By draining and cultivating the land, the peat soils have been exposed to accelerated oxidation, shrinkage due to drying and dewatering, wind erosion, and compaction by farm equipment. As a result, the landward side of the levees has gradually subsided below river level, until now the land is often 3.5 to 5 m below the river.

The levees were typically not compacted, and are constructed on highly compressible peat foundations. The peat is highly deformable and creates levee stability problems. This problem has become more severe as the land subsided, due to the increased hydraulic gradient across the levees. In addition, erosion has created steep waterside slopes. The calculated safety factors against both landside or waterside slope instability are frequently below 1.2. Furthermore, the weight of the levee embankment has significantly compressed the peat, often distorting and cracking the fill and the peat. This cracking, combined with burrowing of rodents and other levee defects, has lead to seepage that periodically results in boils and sinkholes on the levee landside slope and at the landside. As a result, levee failures are not uncommon. According to the California Department of Water Resources (1989), three levee failures that have occurred in the western region of the delta have resulted in emergency expenditures in excess of $39 million (U.S.).

Figure 1 - Location Map



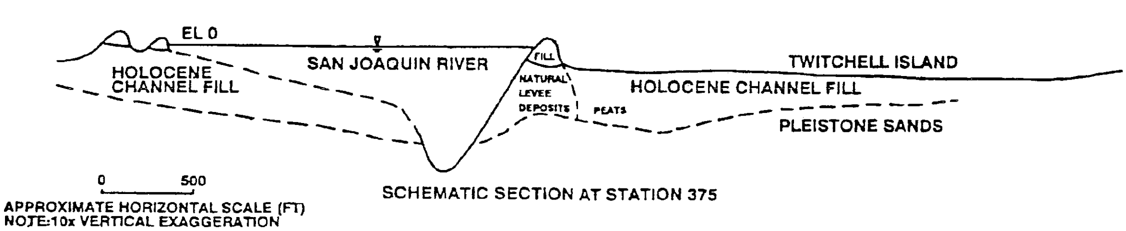
The Delta is an important link in the water supply system for the State of California, serving as a source of drinking water for more than 16 million people. Twitchell Island is one of eight western islands that is considered critical, because according to the DWR, flooding of the islands after a levee breach would have both short and long-term effects on Delta water quality. The islands are adjacent to major Delta channels where fresh and salt waters mix. If an island failed and was not reclaimed, saltwater from San Francisco Bay would move further into the Delta. Short term effects are also severe. In one previous island flooding incident, the inrush of salt water to fill the island increased the chloride concentration above drinking water standards, interrupting delivery of water to Southern California by the State Water project.

In 1988, the California Legislature appropriated $12 million (U.S.) annually for Delta Flood protection. With these funds, investigation and design of an upgrade for Twitchell Island were performed in 1991, with construction in 1992 through 1993. This paper describes the design and construction of that upgrade.

Geology

The Sacramento-San Joaquin Delta has been an area of general subsidence and deposition for over 140 million years. During that time, up to 10,000 m of sediments in thickness were deposited in marine, brackish, and freshwater conditions. The sediments of primary importance for evaluating Delta levees were deposited in the last 70,000 years. During glacial periods, the delta was characterized by rivers systems, with fast-flowing rivers typically depositing sand in alluvial fans and channels. At the end of the glacial period about 10,000 years ago, sea level rose and progressively flooded San Francisco Bay and the Delta. In shallow bays where conditions were conducive to plant growth, peat accumulated. Once the plants were established, their growth led to peat deposition at a rate which kept pace with the rising sea level. The process of peat formation led to the development of peat islands, with river channels and sloughs established around them and within some of the larger islands. During floods, the rivers would overflow their banks to form natural levees of sand and silt along the edges of the islands. Many of the current levees are founded on these natural levees. A schematic geologic cross-section depicting this general model for subsurface conditions at the Twitchell Island Levees is presented in Figure 2.

Figure 2 - Schematic Geologic Cross-Section



1 ft = 0.3048 m

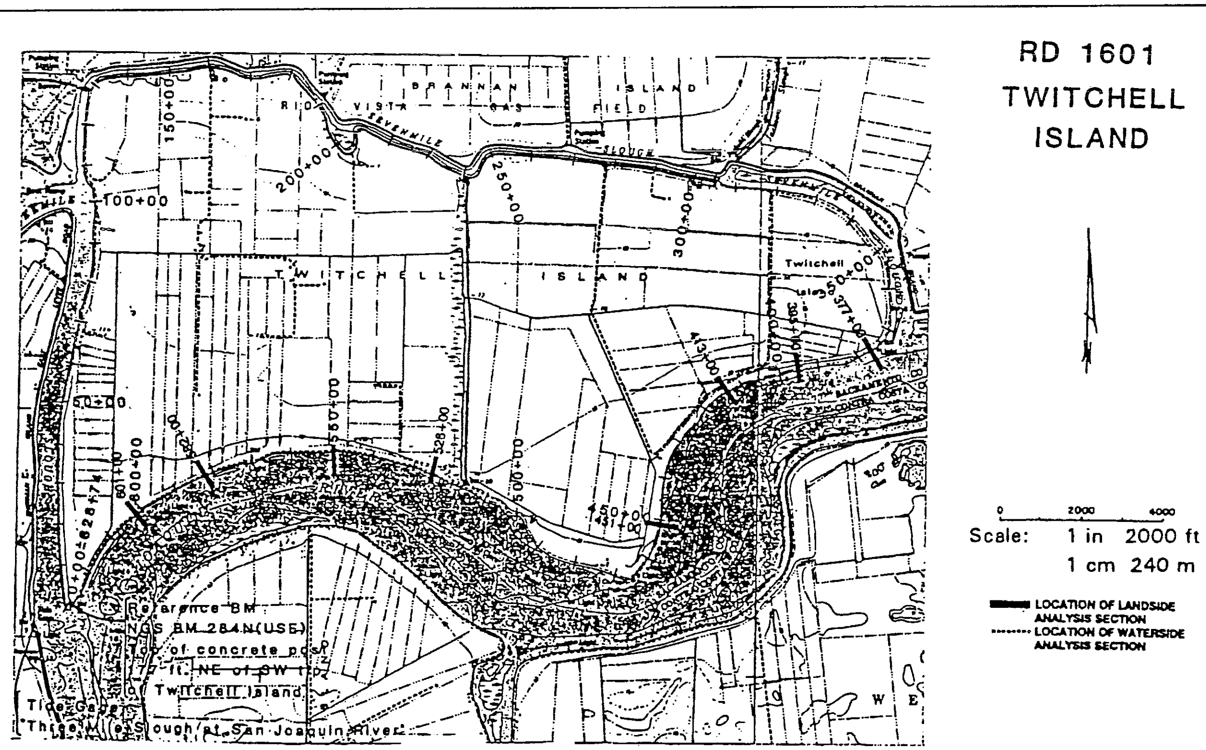
Levee Conditions

Twitchell Island, shown in Figure 3, is formed by the San Joaquin River on the south, Threemile Slough on the west, and Sevenmile Slough on the north and east. The San Joaquin river channel is substantially deeper than the sloughs, and performance of the levee along the river has been poorer than along the sloughs. Thus, the upgrade was focused on an almost 8 km length of levee along the San Joaquin River.

Numerous borings indicated that the base of the peat was typically at about el. -10.5 m (Natl. Geodetic Vertical Datum). The average peat unit density was 10 kN/m^3, with a range of 8.5 to 12 kN/m^3. Water contents in peat beneath the berm ranged between 170% and 900%, with an average of about 540%. Higher densities and lower water contents were typically measured in the more consolidated peat beneath the levee crown.

Survey data showed that typical landside ground elevation was -3.0 to -3.5 m, with a crest elevation of about +2.3 to 2.9 m. The 100 year flood elevation varied from +2.0 to +2.1 m along the levee. The levee crown width varied considerably, typically ranging between about 5 to 8 m. Landside slopes normally ranged between 2H:1V and 3H:1V, with some as flat as 6H:1V. Waterside slopes were typically 2H:1V, with some

Figure 3 - Twitchell Island



locations 1.5H:1V and even steeper locally. The waterside slope is covered by large rip-rap for wave protection.

Upgrade Design

To analyze levee stability, undrained strength analyses (USA) were performed. Ladd (1991) demonstrated that a primary advantage of the USA over an effective stress analysis (ESA) is that shear-induced pore pressures are considered. An ESA typically uses hydrostatic pore pressures because no reliable method exists for predicting shear induced pore pressures. Ladd calculated that the Factor of Safety from an ESA is typically over twice that computed using an USA. Thus, Ladd concludes that ESA will generally give unsafe estimates of the factor of safety, for materials where positive pressure is developed during shearing.

As outlined by Ladd, the steps for an USA are:

1.) Establish the initial stress history of the deposit by high quality laboratory tests

2.) Establish changes in the vertical stress history during construction

3.) Develop the ratio of undrained strength to effective vertical stress (s_u/σ'_{vc}) versus Overconsolidation ratio (OCR) relationships for the foundation soils

4.) Use these relationships and the stress history profiles to compute undrained strength values for the USA.

Stress History

Preconsolidation stresses were established beneath various areas of the levee using consolidation test results. Overconsolidation ratios for the peat samples from under the levee crown and from under the levee toe are plotted against depth in Figures 4 and 5. The peat foundation soils have OCRs near 1, and are therefore very close to being normally consolidated, which is consistent with the geologic history of the peat deposits. Slightly higher OCRs were measured for some samples in the top 2 to 3 m under the toe, probably due to desiccation resulting from drainage for farming. The one high value under the levee crest most probably relates to desiccation of a peat layer shortly after deposition. However, based on the preponderance of data indicating normally consolidated conditions, an OCR of 1 was selected for design.

Normalized Strength Parameters

Direct simple shear tests (DSS) were performed on samples from under the levee toe. Only relatively small effective vertical stresses could be applied to the very compressible sample without exceeding the maximum vertical deformation allowed by the test apparatus. The tests were therefore performed with relatively low effective vertical stress of about 25 kPa. Normalized strength parameters of s_u/σ'_{vc}=0.47 to 0.58 were obtained, thought to be high, even for a peat soil. These high values may be realistic for the peat , reflecting a combination of fiber reinforcement and behavior at low effective stresses, but it was not considered prudent to rely on them for determining strength under the higher vertical stress levels existing beneath the levee crown.

Normalized strength parameters for organic soils from three islands in the Delta, including the Twitchell Island data, were plotted against effective stress as shown in Figure 6. A value of s_u/σ'_{vc} = 0.40 was selected to determine peat strengths, subject to a minimum strength of 12 kPa which would apply in the low overburden stress areas,

Figure 4 - OCR under levee toe

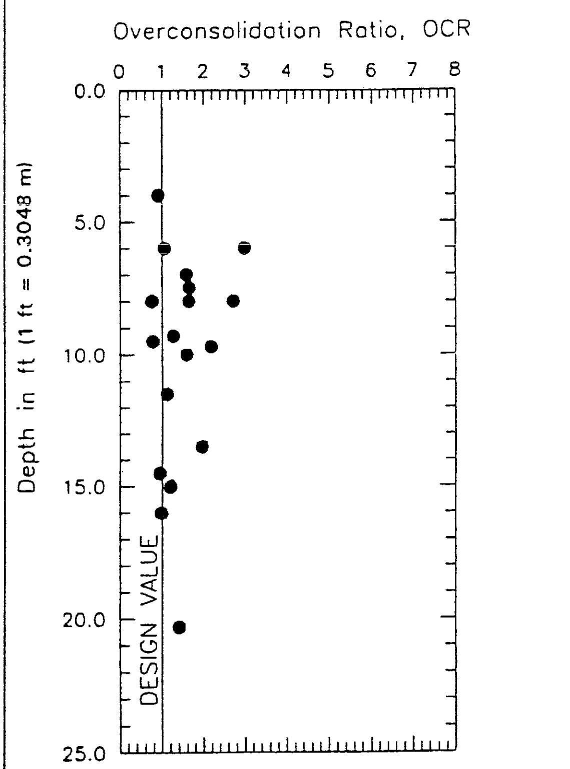
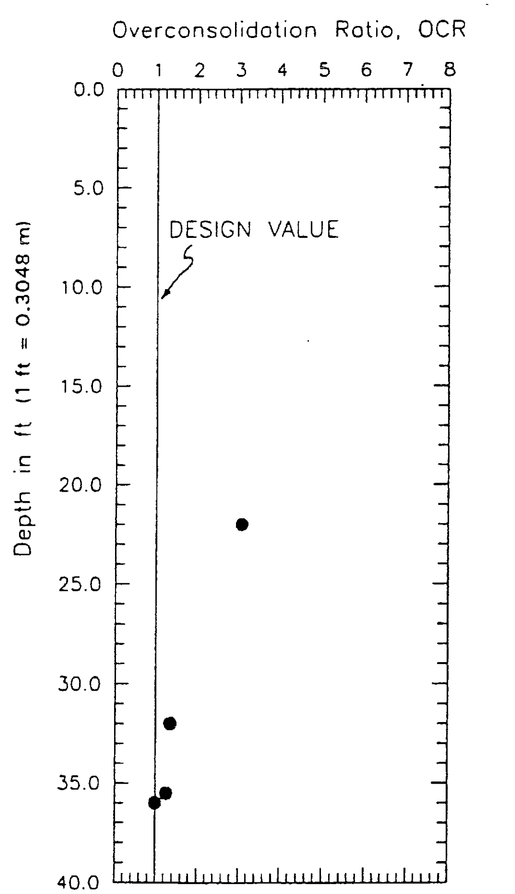


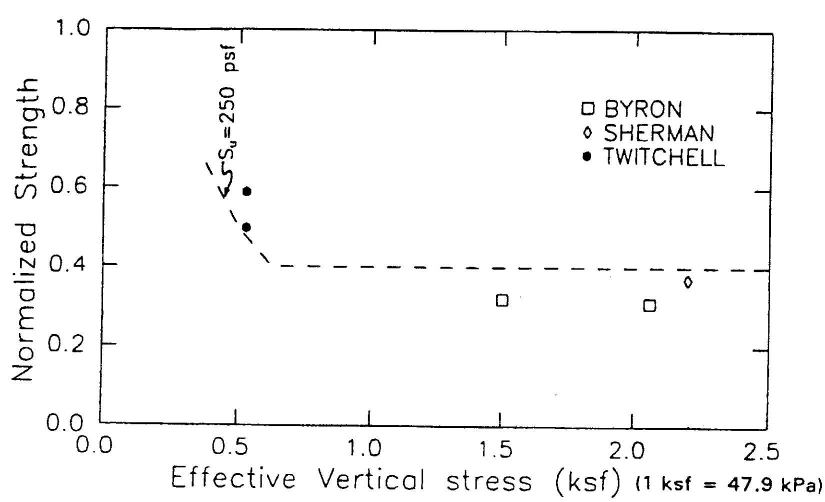
Figure 5 OCR under Crown



effectively representing an increase in s_u/σ'_{vc} as indicated in the figure.

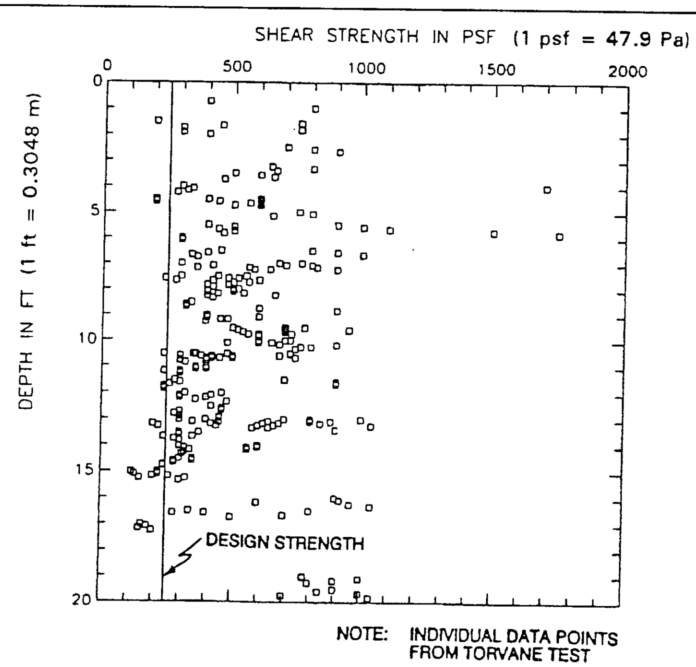
The design shear strength profiles were developed using the normalized strength parameters described above and the SHANSEP procedure described by Ladd and Foott (1974). In-situ effective overburden stresses were calculated corresponding to variations in the thickness of levee fill. Calculated results for average conditions beneath the levee crown and outside the levee toe are shown in Figures 7 and 8.

Figure 6 - Design Strength Envelope



Shear strengths were also assessed using the results of Torvane tests, and the values are also shown in Figures 7 and 8. The SHANSEP and Torvane data generally agree beneath the crown. At the levee toe, the minimum design strength of 12 kPa is near the lower bound for the Torvane data. This situation is thought to reflect the sensitivity of the Torvane test to the high fiber strength within much of the peat materials, as noted by Landva (1986) for in-situ vane testing.

Figure 7 - Strength under Crown



Stability Analyses

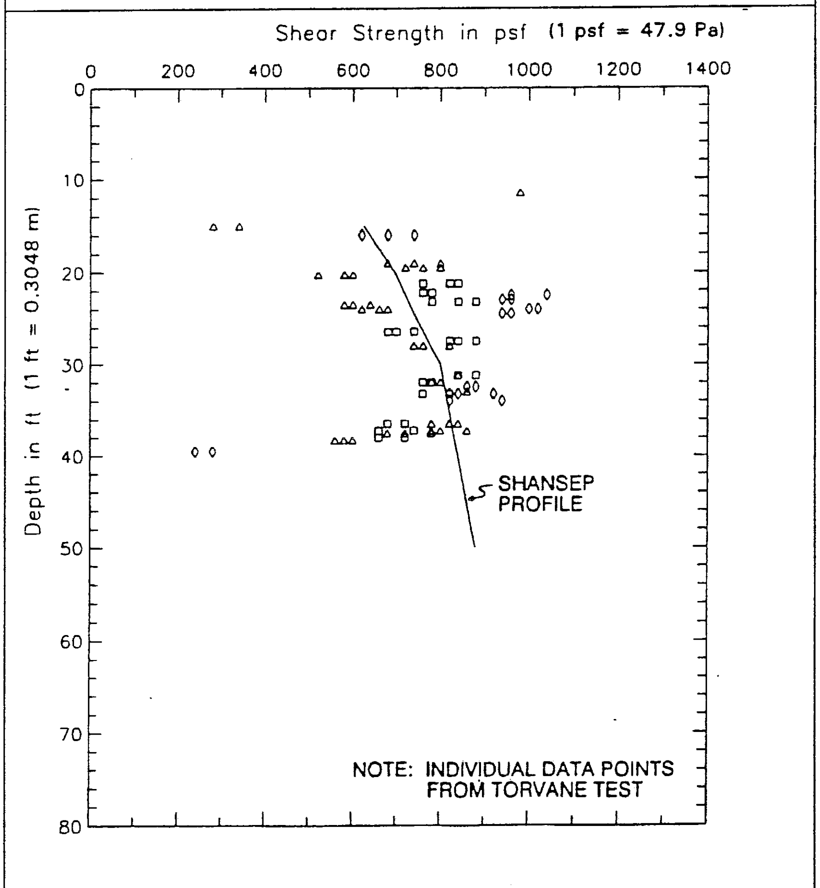
Eight sections were chosen for analyses, representing levee conditions over the typical range of levee height, embankment slope, berm geometry, and peat thickness. For each section, the shear strength variation in the peat across the width of the levee was calculated using the

SHANSEP method described above. A typical cross-sections showing the parameters used for analysis is shown in Figure 9.

The factors of safety determined in the analyses ranged from 1.06 to 1.40. The factor of safety correlated with the size of the toe berm and the elevation of the landside toe. When the importance of the toe berm configuration became apparent, a parametric study was performed to investigate the effect of berm size on levee stability. Several thicknesses and widths of berm were studied, with the potential strength gain and predicted settlement taken into account.

An important behavioral aspect of levees over peat is that berm placement consolidates the peat foundation and results in significant settlement. As the berm settles, it is partially submerged below the water table, which reduces the effective stress on the underlying peat because of buoyancy effects. Since peat strength is related to the effective vertical stress, submergence of the berm therefore reduces its effectiveness in increasing the peat strength. The decreased strength gain in the peat is further reduced because the vertical stress must be increased above a threshold value corresponding to the minimum design strength, before peat strength gain occurs. For a minimum design strength of 12 kPa and s_u/σ'_{vc}=0.40, the vertical stress must be increased above about 30 kPa to cause strength increase. This corresponds to a berm approximately 2.5 m thick including allowance for settlement under the water table. Therefore, until berm thicknesses exceed 2.5 m, their contribution to stability is due primarily to their dead weight, with little or no contribution from increase peat strength.

Figure 8 - Strength Profile below Berm



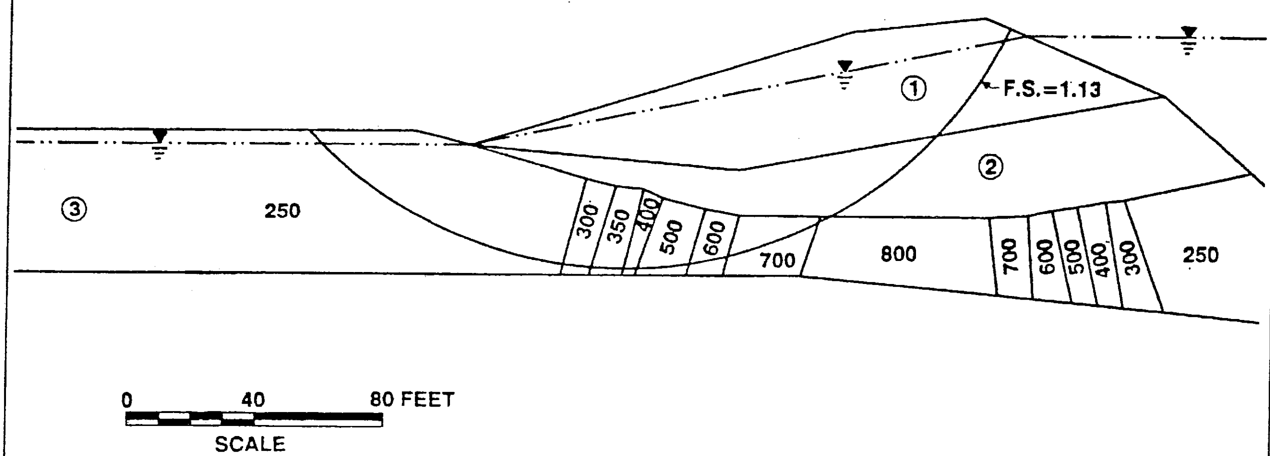
Settlement Analyses

Berm settlement was estimated using consolidation test results. Settlements depend on the assumed preconsolidation stress in the peat and the peat thickness, so a range of settlements was calculated as shown in Figure 10. Also shown on the figure are measured settlements of berms placed during an

Figure 9 Typical Section for Analysis

PARAMETERS USED FOR ANALYSIS

ZONE	SOIL TYPE	DENSITY (PCF)	FRICTION ANGLE	COHESION (PSF)
1	FILL	115	30	0
2	FILL	90	0	500
3	PEAT	70	0	AS INDICATED BELOW
4	SAND	125	35	0



earlier stabilization project. The data, taken when the berms were still settling, indicate that these berms had reached the lower bound of the calculated settlement range.

Design Geometry

Once the effects of settlement were quantified, berms of varying widths and thicknesses were analyzed to determine a configuration resulting in a factor of safety of 1.3. This factor of safety is often used as a reasonable design objective for improvement of Delta levees. The analyses indicated that this objective could not be achieved with a 12 m wide berm because the critical circle extended outside the berm, but that a 18 m wide, 2.5 m thick berm was adequate to achieve the desired safety factor. Therefore, this geometry was selected for most sections along the river.

In some locations where waterside slopes were steep and the river channel was deep, waterside levee stability was calculated to be marginal. In such locations, a wider 36 m berm was selected to allow extending the levee to the landside in the future.

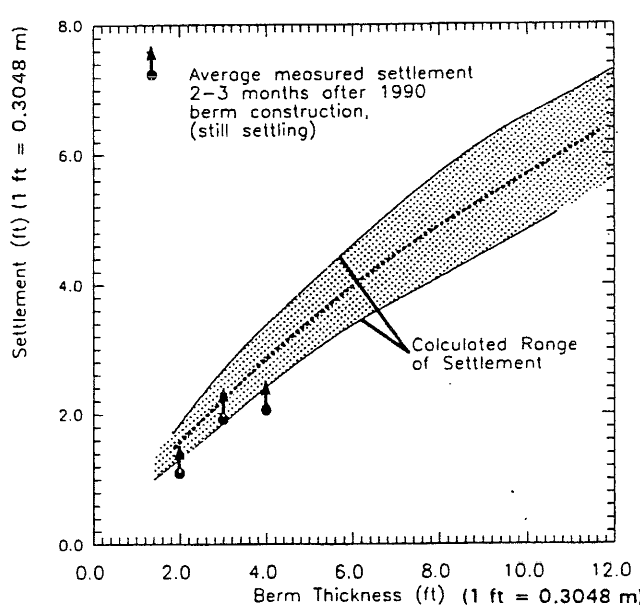
Some berm was already in place along much of the levee, and the upgrade was designed to consider this material. In locations where fill increments would be greater than 1 m, staged berm construction was adopted to reduce the likelihood of a foundation failure due to the additional weight of berm material.

Construction

The upgrade construction began by clearing brushy vegetation, such as berry bushes and small trees before placing the berm. Grass and other low vegetation was not cleared. Before placing the toe berm, an array of two to three settlement plates was installed at approximately 30 m intervals along the berm. One plate was set approximately 9 m landward of the levee toe, and the second was set about 17 m from the levee toe.. The settlement plates are identified by their station location and the letter A or B, with the A plates closer to the levee. A third plate was placed along the array where the berm was to be 36.5 m wide. This third plate was typically 24 m from the levee toe, and was called the C plate.

Approximately 400,000 m^3 of silty sand fill was imported to Twitchell Island to construct the berm, raise the levee crest, and flatten the levee slopes. Over 300,000 m^3 of this was obtained from maintenance dredging of an intake forebay for the California water project. The material was excavated using a suction dredge, barged in scows to Twitchell Island, and off loaded using a crane with a 9.4 m^3 clamshell bucket. The material was saturated and fluid when it was placed. After a few weeks the excess water drained, and then the berm was fine-graded and track-walked with a LGP-D6 dozer.

Figure 10 - Berm Settlement Characteristics



The berm was placed in two lifts beginning at the west end of the levee. The first lift was 900 to 1200 mm thick, and was placed over a three month period in 1992. Once the first lift had been placed, settlement plates showed that the settlement rate had decreased sufficiently at the west end of the levee for placement of the second berm lift to begin immediately. The second lift was typically 600 to 900 mm thick. After the second lift was placed, material was pushed up against the levee to obtain a 3:1 slope on the landside. As the last stage in the levee upgrading, fill was placed on the levee crown to raise it to the design elevation.

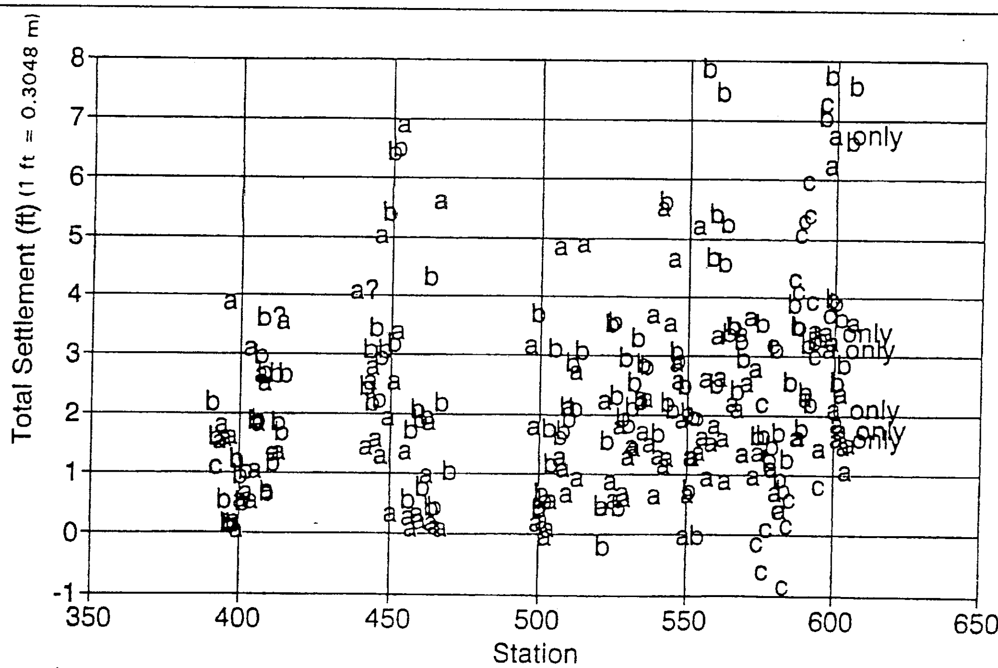
Levee Behavior

During construction, portions of the levee and some adjacent areas settled significantly and moved laterally, which caused localized cracks, sinkholes, and linear subsidence features. The settlement and lateral deformations probably resulted from large compression and shearing in the highly compressible and relatively weak peat.

Settlement

Settlements under the berm result from both consolidation and secondary compression. Settlements along the toe berm were relatively large. A plot of total settlements recorded along the berm is shown in Figure 11. The values range as high as 2.1 m. The average settlement measured at the A plates, nearest the levee, was 580 mm. The average settlement at the B plates was 790 mm, and the average settlement at the C plate was 730 mm.

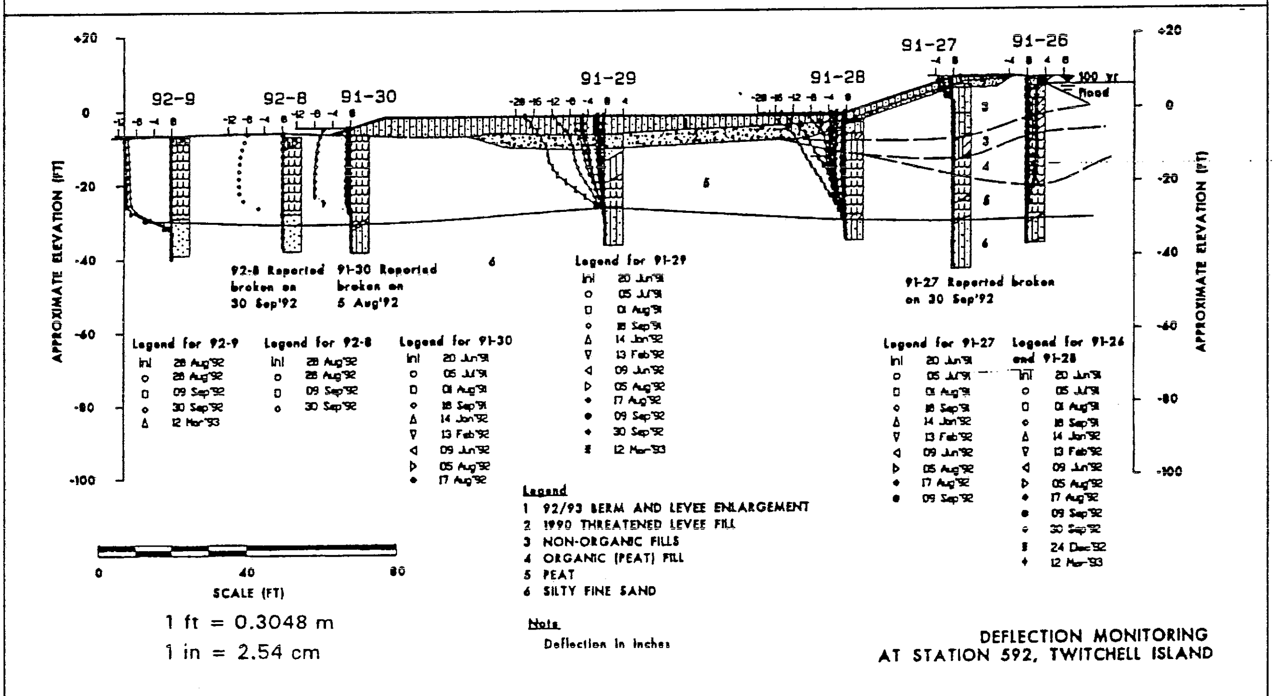
Figure 11 - Settlement Monitoring Results



Lateral Deformations

Inclinometers were used to measure lateral deformation at the project site. The results of monitoring at one of the most extensive arrays is shown on Figure 12. The inclinometers indicate relatively small deformations (less than 100 mm) beneath the crown, with deformations so large (in excess of 300 mm) that inclinometers at the toe of the berm were rendered inoperable by the deformation. In addition, deformation patterns at the toe of the berm typically showed concentrated displacement at the base

Figure 12 - Inclinometer Results



of the peat, while deformations patterns under the middle of the berm indicated relatively uniform strain increasing with time.

Cracking

Both vertical and lateral movements described above resulted in differential movements large enough to tear the peat, either in shear or in tension. Cracks were observed in several different locations along the levee cross-section, as discussed below.

Levee Crown: Single cracks were observed in various localized areas on the crown, often near the landside hinge point. The cracks ran parallel to the levee, and were as wide as 25 to 50 mm. They primarily showed lateral offset, with little vertical offset. They may have been related to differential horizontal movement between the levee and the berm.

Levee Slope: Some cracks were observed on the upper portion of the newly filled backslope, but cracks more often were observed along the toe of the slope. These cracks frequently occurred as multiple parallel cracks, usually 12 mm or less in width. They are likely a result of differential vertical movement between the levee and the berm.

Berm: In some locations, cracks were observed along the new berm. These cracks often exhibited several inches of vertical offset (see Figure 13). Frequently, the cracks were parallel with opposing vertical offset leaving grabens between the cracks. These cracks may be caused by lateral displacement of peat below the berm, or may be effects of old drainage ditches.

Area Adjacent to the Berm Toe: Cracks were noted in a gravel road immediately adjacent to one section of berm. These cracks appeared to be tension cracks, possibly resulting from stretching as the new berm settled downward. These cracks were notable primarily because they indicate tension at the toe of the fill, an area often in compression. However, the settlement of the berm fill may dominate the behavior, resulting in a zone of tension at the edge of the settlement bowl.

Island Areas Beyond Berm: In two instances, tensile cracks were observed to

Figure 13 -Cracking on Berm

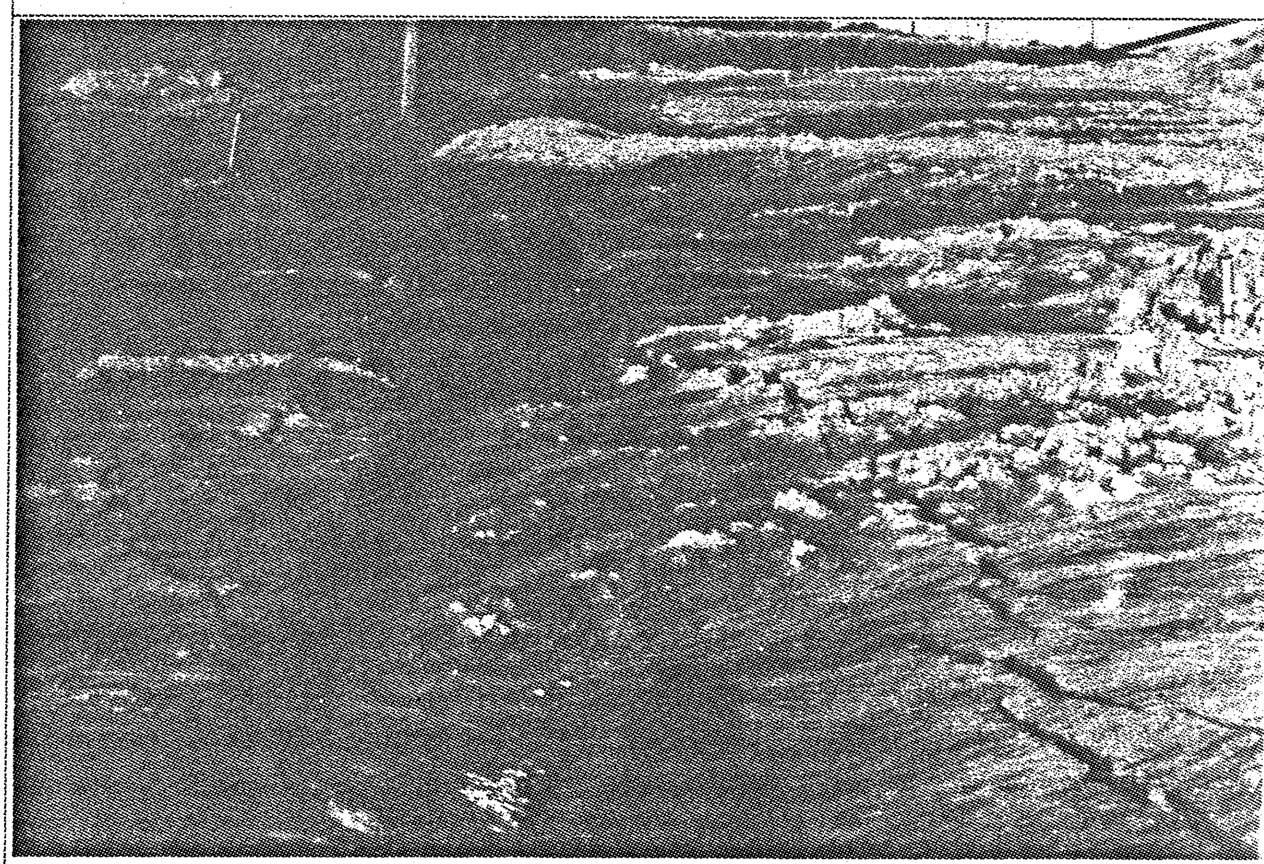
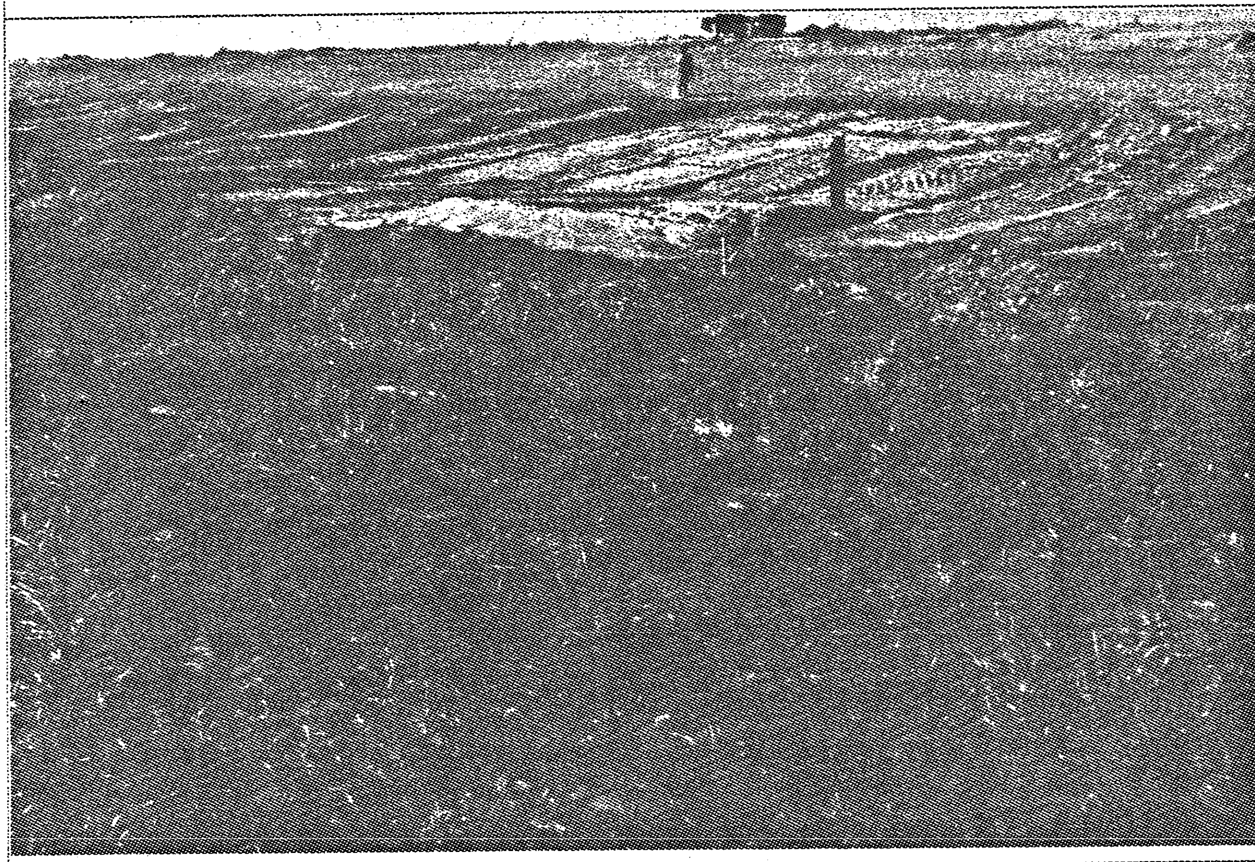


Figure 14 - Cracking Landward of Berm



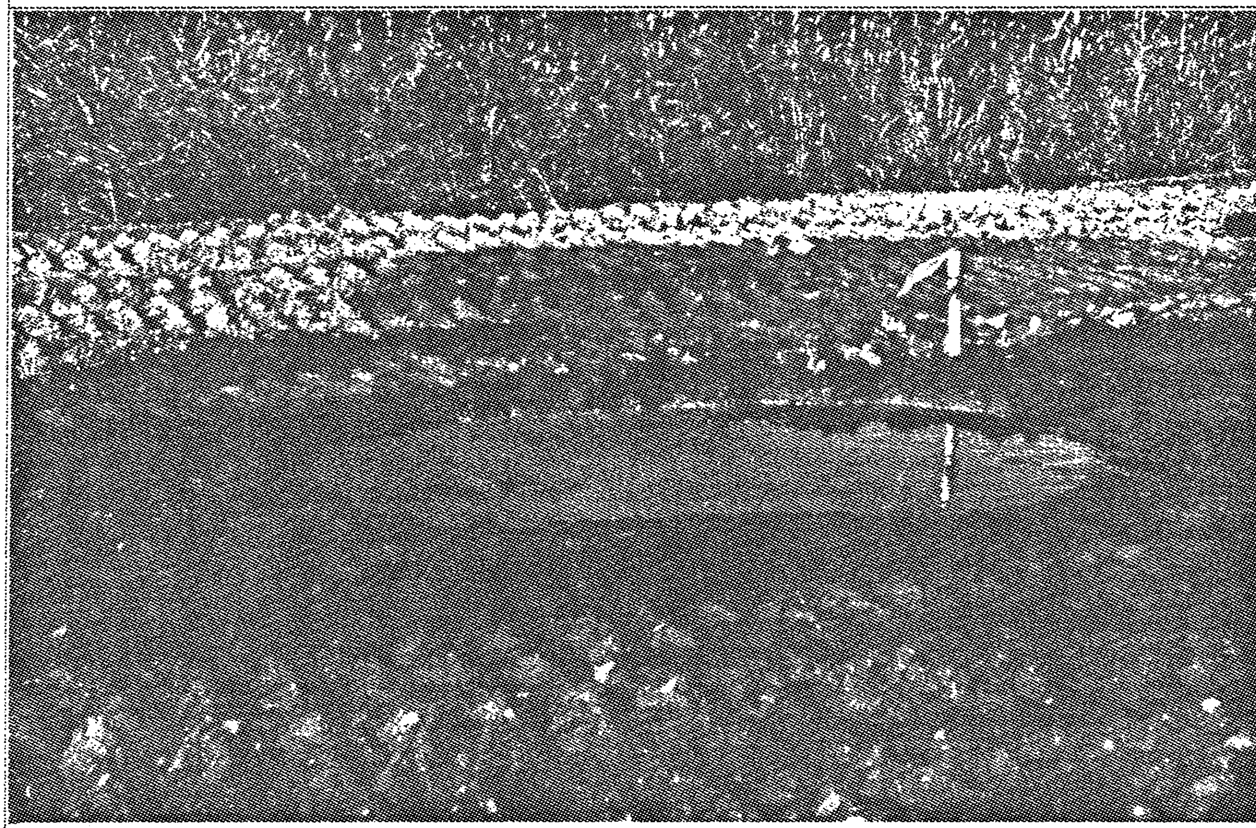
propagate more than 10 m landward of the berm at an angle of about 45 degrees to the berm alignment (See Figure 14). The cracks were on the order of 300 mm wide, and depths of nearly 3 m were measured. Soil blocks in the middle of the cracks were down-thrown, similar to the grabens described previously. The contractor reported observing one crack propagate rapidly, about 30 m ahead of the fill placement.

These cracks are believed to represent three dimensional effects of berm deformations. The cracks may be the lateral boundary of a soil block landward of the berm that experiences large, rapid lateral displacement during berm placement. The large vertical displacements occurring beneath the berm may have also contributed to the cracking. The cracks did not appear to threaten the levee or berm, but were backfilled for safety reasons.

Subsidence Features

Subsidence features are defined in this paper as deformations that have large vertical components relative to lateral components. They appeared on the site as either deep circular depressions (sink holes) or linear depressions as long as 300 m (see figure 15). Soundings indicated that these depressions were over 3 m deep. It is believed that the depressions result from an upper layer of fill flowing into underlying cracks in

Figure 15 - Subsidence Feature



the peat. Such cracks could result from lateral displacement at the base of the peat. Some subsidence features could be the result of other phenomena, but most features at Twitchell Island would fit this hypothesis.

Levee Repairs

In most instances, the most serious risk from cracks that occurred are believed to be the possibility of water flowing into the cracks and resulting in further deformation due to increased hydrostatic pressure. Cracks in the berm often were regraded after the fill settled, or they were covered with the second lift of fill. Cracks on the levee backslope were fine graded and track-walked. Some cracked areas on the levee crown were overexcavated and backfilled to reduce the potential for the cracks acting as a preferential seepage path.

Subsidence features were filled with either sand or gravel fill. Following the lateral spreading fissure theory, placing fill in these features could cause the fissure to widen. In fact, small features often required several truck loads of material to fill them. In some cases, it appeared helpful to wait several days prior to backfilling or to place the backfill in stages. It may be that this allowed dissipation of excess pore pressures.

However, these depressions sometimes reappeared, particularly when additional fill for the second berm lift was placed.

Conclusions

A SHANSEP type undrained strength analyses along with limit equilibrium stability calculations proved to be an adequate tool to design the upgrade construction for the Twitchell Island levees. The method provides the advantage that it considers shear induced pore pressures.

Although limit equilibrium stability analyses provide an adequate basis for design, deformation of the peat required careful observation and treatment during construction. Both shearing and compression appear to be important modes of deformation. The peat soils appear to have low tensile strength and a tendency to rip or tear due to differential movement. Fortunately, the levee backslopes and toe berms supported by soft peat and organic soils were able to tolerate large strain and deformations.

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