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Threatened Levees on Sherman Island

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ABSTRACT

Severe levee cracking was observed along a section of a levee in the Sacramento Delta area of California. An extensive program of geotechnical investigation and treatment was performed. The results of the investigation point to the danger of considering only limiting equilibrium factors of safety for highly deformable peat soils, since large deformations may be initiated long before critical stability is indicated.

INTRODUCTION

Sherman Island lies in California at the western limit of the Sacramento-San Joaquin Delta. Like most of the delta islands, it is predominantly below sea level and protected by perimeter levees which were built over peaty soils. The levees were originally constructed in the 1860's, and have been enlarged periodically as the adjacent land subsided. In recent years, the levees have been maintained to the extent possible given the limited available funding, and they are regularly inspected by the local Reclamation District. The island was flooded in 1969 following a levee failure, but since that time it

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has been kept whole through a program of careful inspection, ongoing maintenance, and rapid response to problem situations.

In the summer of 1990, a sizeable array of cracks developed along a reach of the levee approximately 1000 ft long. The cracks were located in the landside berm of the levee, in the sides of the levee embankment, and on the top of the levee. They were typically aligned along the levee giving the general appearance of the toe of the levee berm having been pulled laterally outwards. Crack movements on the embankment progressed quite rapidly, with the cracks frequently opening to a width of one or two inches. Vertical offsets as much as 12 in. also developed across some of the cracks. However longitudinal displacement along the cracks was not observed. Figure 1 shows a typical photograph of a crack; Figure 2 presents a schematic section through the levee showing a characteristic crack array.

Figure 1 Photograph of Crack

The cracks extended across the levee crown into the waterside slope and, in addition to stability concerns, presented a very real threat that San Joaquin River water would flow relatively unimpeded through them into the island. Since the levee at this location is predominantly composed of sand, erosion and piping could then easily breach the levee and flood the island. The prospect of

an island of 9,900 acres being flooded to a depth of 10 to 15 ft was extremely serious. The cracking occurred during the period of low summer flows and following 4 years of drought. If flooding had resulted, it would have pulled saline water from San Francisco Bay far up into the Delta and could have required the temporary closure of major water supply projects for the State of California.

Treating the observed levee distress required resources beyond those of the Reclamation District. However, Sherman Island is one of the eight western islands nominated for the Threatened Levee Program, administered by the Department of Water Resources (DWR) for the State of California. Funding for remedial levee work was authorized directly.

During the months of August to December, 1990, remedial measures were applied at this main area of observed cracking, as well as at other areas of levee distress. This paper describes the investigation of factors affecting the levee cracking, the engineering of corrective measures, and the implementation of those measures. The paper also discusses the cracking phenomenon, its probable causes and typical treatments.

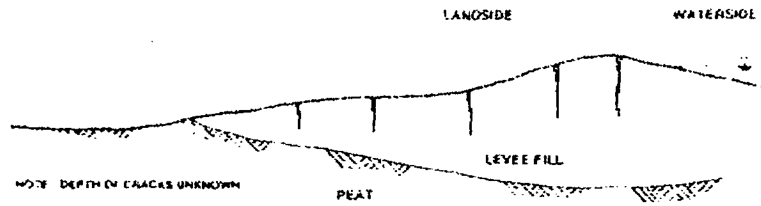


Figure 2 Schematic Section Showing Typical Crack Array

FIELD AND LABORATORY INVESTIGATION

The main site investigation techniques used for the program were borings. The borings provided information regarding the subsurface strata, plus samples for office inspection and laboratory testing. Inclinometers were installed in the borings to provide information regarding ongoing lateral movements in the ground. Most importantly, they also provided a means of monitoring deformations due to placement of levee fill and, thereby, a means for controlling the filling process. Additional site investigation work was performed by the DWR, including monitoring crack widths over time, performing a limited series of field vane strength tests, and surveys of key levee sections.

The boring samples were routinely subjected to a basic laboratory testing program of moisture/density determination and Torvane strength testing. In addition, selected samples were used for consolidation testing and Unconsolidated Undrained (UU) shear strength testing. Direct Simple Shear (DSS) testing was performed on one sample of peat and one sample of the silty clay. The field and laboratory data are introduced as appropriate in subsequent sections of the paper.

SUBSURFACE CONDITIONS

The levees of interest are constructed along the northern bank of the San Joaquin River, which flows in a deep channel on an alignment which has probably been relatively unchanged during the deposition of much of the Sherman Island soil profile. For present purposes, that profile can be considered as starting at depth with a sand stratum below approximate El -70 ft (NGVD). Pore water head elevations in that stratum appear to relate closely to the river surface elevations, suggesting that there is hydraulic connection between the two, perhaps by exposure of the sand stratum in the river bed.

Above the sand is a layer of silty clay, most probably Bay Mud laid down as the sea level rose following the last Ice Age. The clay stratum is on the order of 20 ft thick and overlain by peats which, in their natural state, are up to about 40 ft thick and extend to the island surface. Upward seepage occurs through the clays and peats from the artesian conditions in the underlying sand, with seep water typically drained off, collected, and pumped out of the island via a series of drainage ditches flowing to a pumping station.

Beneath the levees, the natural soil profile is modified by the considerable weight of levee fill, with resultant settlement (up to 20 to 30 ft) of the original peat surface, accompanied by peat strengthening. The levee fills are typically composed of peat, dredge materials (sand, silt and clay) and sandy fill, with the crown of the study levees usually consisting of relatively clean sand. In some locations the levees also appear to be located directly over natural levees of the San Joaquin River, which are indicated by layers of silty material within the peat stratum and by higher material strengths. These natural levees appear generally to be of very limited lateral extent, grading into the normal thick peat stratum beneath the levee berms.

The investigation and remediation concentrated on three instrumented sections across the levee. A site

plan is shown in Figure 3, with the location of sections A, C and D, borings, inclinometers and piezometers also shown. A database for Section A was developed first and used for analysis of the area, with data for Sections C and D developed thereafter. The three sections were found to be very similar, allowing data from all sections to be considered collectively.

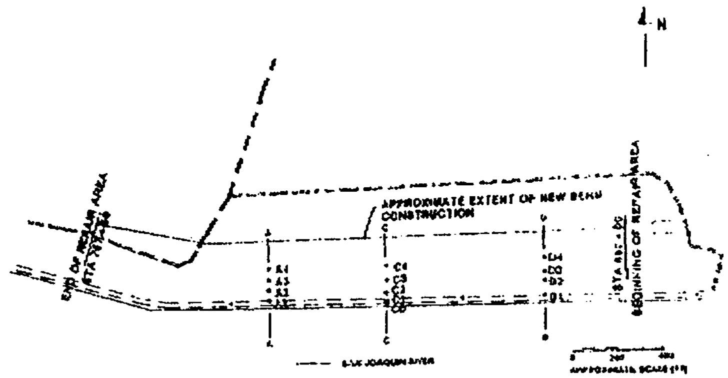


Figure 3 Site Plan of Main Crack Area

Figure 4 presents the profile through the levee at Section A, including the subsurface soil strata. A thick levee fill is apparent, overlying interbedded silts and peats beneath the levee crest and thought to be old natural levee deposits, which grade into thick peat deposits beneath and beyond the landside berm. A silty clay deposit beneath the peat then underlies the entire site, above a deeper stratum of sandy soils. A toe ditch is present outside the berm.

The increased depth of levee fill to landside of the levee crown is notable. This characteristic was even more apparent in Sections C and D. It is thought to reflect increased compressibility of the peaty soils as compared to the natural levee deposits. The likely cause of this situation is that the natural levee at the higher levels would typically have experienced air drying and stiffening during the deposition process (see discussion below). Thus levee filling on the landside of the natural levees would have caused greater compression of the subsoils and experienced more settlement than filling at the levee crown, which appears to be located directly over the highest area of the natural levee soils.

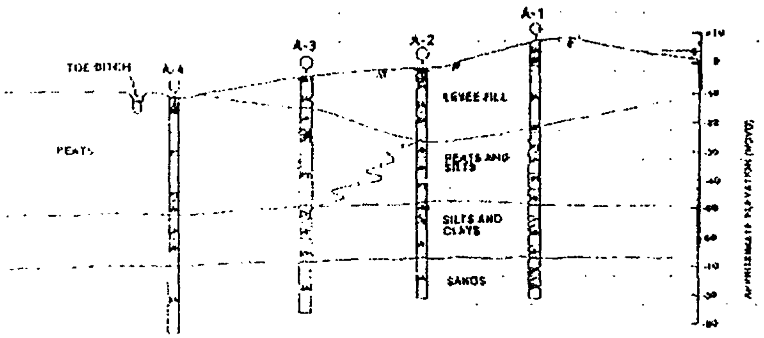


Figure 4 Soil Profile at Section A

STRESS HISTORY

Geotechnical analysis of the levee profile proceeded by assessing the stress history at various offsets from the levee crown using the results of consolidation tests. The stress history was assessed for the virgin conditions outside the original levee fill area (Borings A4, C4 and D4) and for the consolidated conditions beneath the levee embankment. Figure 5a presents the assessment for the virgin conditions, plotting the vertical effective stress profile and the measured maximum past pressure data for the virgin soil profile. The vertical effective stresses are low and the stress profile has a steep slope, reflecting the very low effective weight of the peat soils. The maximum past pressure data in the peat (PT) are seen to be equal to, or very slightly higher than, the effective stress profile, except near the surface where they lie significantly above the profile. The indication is that the peat profile has probably experienced a surface drying but is close to a normally consolidated condition at depth.

The underlying clayey (CH/MH) stratum has indication of a much more variable stress history, with some prestress values lying well above the effective stress profile and potentially corresponding to desiccated layers from the time of deposition. However some of the prestress values do fall near the effective stress profile, so the stratum can conservatively be considered as generally normally consolidated.

A comparison of vertical effective stress and maximum past pressure data beneath the levee embankment

(Borings A1, C1 and D1) is presented in Figure 5b. A much higher vertical effective stress profile applies at this location than in the virgin soils of Figure 9, reflecting the addition of about 30 ft of fill. Much higher prestress values are also evident, with some of the values falling very considerably above the effective stress line. These very high values are not consistent with virgin consolidation of weak peat and clay having occurred under the levee loads. Instead they suggest that some areas in the foundation soils had been heavily prestressed long before the levee fills were placed. A possible explanation for this situation lies in the presence of natural levee deposits below the present levee embankment. Natural levees are subject to drying and strengthening, and depending on the frequency of flooding and the height of the banks, layers of highly consolidated material may have formed in the study area.

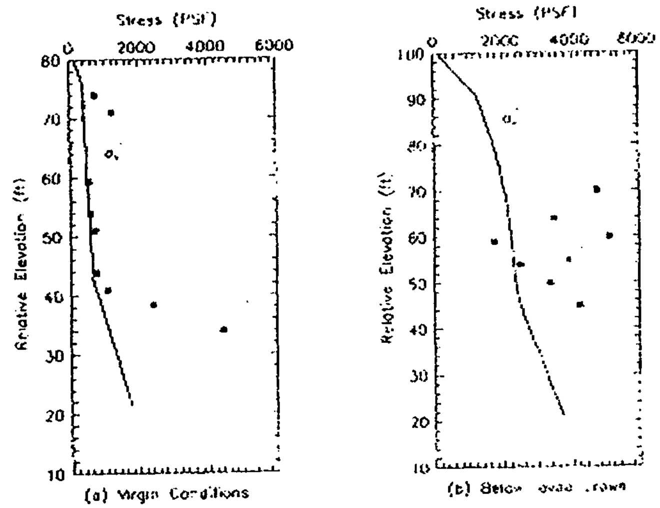


Figure 5: Stress History Profile

The conservative interpretation adopted for this situation was to assume that the soils beneath the levee had been consolidated to their existing vertical effective stresses only, and ignore the existence of higher strength zones in the levee foundation soils. This assumption was believed to yield lower bound assumptions regarding soil strengths, and upper bound assumptions regarding potential settlement.

SHEAR STRENGTH ASSESSMENT

An undrained strength model for the levee and foundation soils was developed as the starting point for stability analysis of the levee, using the SHANSEP process described by Ladd and Foott (1974). The SHANSEP process first establishes an appropriate normalized strength for application to each soil, using the fact that the undrained strength (s_u) of a normally consolidated cohesive soil very often bears a constant relationship to the vertical effective stress (σ_{vc}) of the soil. The relationship s_u/σ_{vc} is therefore a constant, called the "normalized strength." Normalized strengths obtained from DSS tests are often appropriate values for the stability evaluation of levees and embankments. For the present foundation soils they indicated $s_u/\sigma_{vc} = 0.37$ for the peat soils and $s_u/\sigma_{vc} = 0.26$ for the silty clays.

The SHANSEP procedure then applies the normalized strength values to the vertical effective stress (σ_{vc}) profiles at appropriate locations, to yield undrained strength (s_u) profiles. This procedure was applied for each boring offset in the levee section. The SHANSEP strength profiles were then compared with other strength data to assess the reasonableness of the values obtained.

The resulting comparisons are shown in Figure 6. Figure 6a relates to the levee crown (Borings A1, C1, D1) and shows the SHANSEP profile plus Torvane data and UU data. The Torvane was used to obtain a quick indication of undrained strength in cohesive soils, and assemble a sizeable database of strength values relatively inexpensively. Landva (1986) identified limitations of the test in peats, so definitive strength values would generally not be expected, but the test does indicate approximate strengths and their variability. Figure 6a shows that the SHANSEP data fall at the upper end of the range of Torvane data in the peats and silts and usually within the body of the Torvane data in the silty clays. The UU data are all much higher than the SHANSEP values, perhaps indicating that better quality cohesive and peat samples were selected for laboratory testing (in order to avoid tests on silt samples which have little value). Figures 6b and 6c show a comparison of SHANSEP and Torvane data at the A2, C2, D2 and the A3, C3, D3 offsets, respectively. Figure 6b again shows Torvane data lying below the SHANSEP strengths in the peats and above and below the SHANSEP values in the clays, while Figure 6c shows the Torvane values to be generally above the

SHANSEP strength profile. The effective stress values are progressively reduced from Figure 6a through 6c, reflecting reduced thickness of levee fill. Figure 6d presents the same comparison for the unloaded virgin soil profile (Borings A4, C4, D4) and also shows the DWR field vane shear test results. The Torvane data are now usually much higher than the SHANSEP profile. The field vane data are also much higher, although they would typically require "correction" to yield appropriate design values. A correction factor of about 2/3 would be required to yield the SHANSEP strengths, within the expected range for the present soils. In this case, the SHANSEP data have been estimated using an assumed uniform vertical effective stress profile of about 600 psf throughout the peat, to include the effect of surface drying (see Figure 5b).

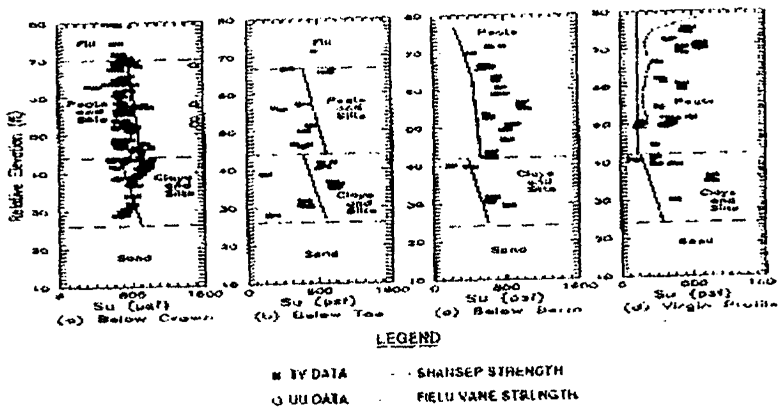
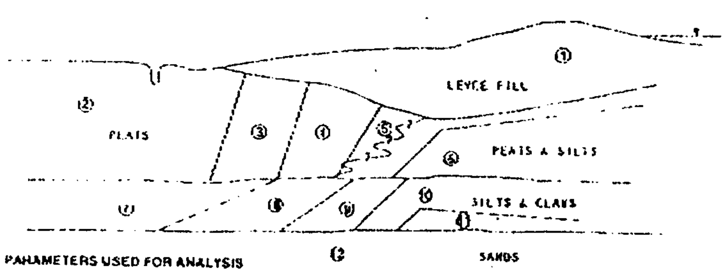


Figure 6 Shear Strength Profile

Based on the above, the SHANSEP strength values were judged to represent a reasonable basis for design. It seems likely that the Torvane data overestimate strengths in the very weak, virgin profile, probably in part due to fibrous reinforcing of the soil by organic material. It then seems probable that this Torvane tendency for strength overestimation gives way to a slight tendency for underestimation of strengths at the higher stress levels beneath the levee embankment, possibly reflecting an increased sample disturbance effect with the Torvane, plus the increased presence of silt within the peaty layer. However, to avoid the possibility of overestimating strengths below the levee embankment, the values used in the peat for stability analyses were reduced slightly from the SHANSEP values.

The next stage in the stability assessment was the development of strength contours within the foundation soils for the Section A levee cross section. Based on standard correlations, fill material strength properties of $C=0$, $\phi=30^{o}$ were also selected. By using the strength contours to divide the foundation soil into zones of assumed uniform conditions, the stability model of Figure 7 was then obtained.



PARAMETERS USED FOR ANALYSIS

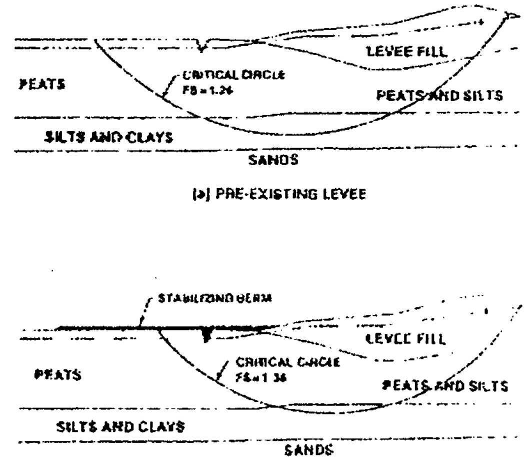
ZONE	SOIL TYPE	COHESION (PSF)	FRICTION ANGLE	DENSITY (PCF)
1	FILL	0	30	[illegible]
2	PEAT	200	0	70
3		400	0	75
4		600	0	80
5		700	0	85
6		800	0	90
7	CLAY	300	0	110
8		400	0	[illegible]
9		600	0	[illegible]
10		[illegible]	0	[illegible]
11		[illegible]	0	110
12	SAND	0	35	135

Figure 7 Stability Model

STABILITY ANALYSIS

Stability was analyzed for a number of conditions using the above described strength profiles, which are considered to be generally conservative. The factor of safety for the pre-existing levee configuration was calculated as 1.26. The increase in stability resulting from placing an 80 ft wide, 2 ft thick berm outside the toe ditch, plus a fill connecting this new berm to the original berm was investigated. It resulted in a factor of safety of 1.36. These analyses and the critical stability surfaces are summarized in Figure 8. Addition of a further 2 ft of fill over a width of 16 ft at the levee

crown reduced the safety factor to 1.35. None of the indicated factors of safety are such that major instability would be expected, particularly in view of the conservative approach used to select shear strengths. Factors of safety close to unity would generally be expected before gross shear failure will occur.



(a) PRE-EXISTING LEVEE

(b) STABILITY WITH TOE BERM AND CONNECTING FILL

Figure 8 Results of Stability Analyses

CORRECTIVE ACTION

Corrective action in the main area of cracking commenced effectively on an emergency basis in August, 1990, with the placement of an 80 ft wide, 2 ft thick, silty sand berm to the landside of the toe ditch below the levee. The toe ditch was then cleared and a toe drain formed in it. A connecting fill was next placed over the drain to connect the new berm to the original berm, with a thin dressing of material carried onto the original berm to fill in low spots. Wet spots on the existing berm due to seepage through the embankment were also individually drained. Other than the use of clean sand in the toe drain, plus localized use of drain rock wrapped in filter fabric at localized seepage areas, the fill material was all a silty sand obtained from dredged material previously stockpiled on the island

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While these construction activities were proceeding, the site investigation, testing and analyses described earlier were underway and the inclinometers were installed and monitored. The inclinometers indicated lateral deformations on the order of one or two inches in the virgin peat under the new berm, but no ongoing movements were measured in the levee crest and original berm. Crack monitoring along the levee embankment also indicated that movements had slowed very substantially.

At that stage the levee was judged to be relatively stable, based on both the stability analyses and field observations. It was therefore decided to restore the levee embankment. This process involved two steps, the first being to compact the embankment back together at the crack locations, after which the full freeboard and levee crown width were restored. In performing this work, it was recognized that the peat soils could deform excessively, resulting in levee cracking, long before critical limiting equilibrium stability conditions (i.e. safety factor = 1.0) would be developed. Accordingly, the inclinometers were carefully monitored throughout the restoration work for any evidence that lateral deformations might be developing. Such evidence would have been evaluated with a view to ceasing filling activities or removing fill already placed.

The levee embankment was compacted using an Essick VF-S4T vibrating sheepsfoot roller towed behind a D6 crawler tractor, which provided a substantial vibratory compactive effort with excellent maneuverability. It was found that when the vibrator was applied with its longitudinal axis above and parallel to a crack, the crack would close and heal in a very satisfactory manner. By this process the levee cracks were sealed and the integrity of the levee embankment restored.

The levees were next surveyed and areas and amounts of freeboard deficiency noted. Starting first at an area extending 200 ft on each side of Section C, the freeboard was restored by placing up to 2 ft of material on the levee crown, which was also widened significantly to provide the standard 16 ft crown width. The Section C inclinometers were monitored carefully during this fill placement, but no subsurface movements were observed beneath the crown or the original berm. The crown was thereupon restored to full design grade through the entire area, with no significant movements observed in any of the crown or berm inclinometers.

DEFORMATION ANALYSES

The stability analyses and observed field behavior strongly suggested that the levee distress was not the result of a classical limiting equilibrium slope stability failure. The problem appeared instead to relate to deformation of the peaty foundation soils, which are known to be highly deformable and creep susceptible. Figure 9 demonstrates this characteristic by comparing the deformation modulus values measured in the DSS tests on the peat and the silty clay, with modulus values for a variety of cohesive soils. Generally, soils with lower moduli are more deformable (and probably more creep susceptible, see Foott and Ladd, 1981). The peat has by far the lowest modulus of all the soils (note the logarithmic vertical axis in Figure 9).

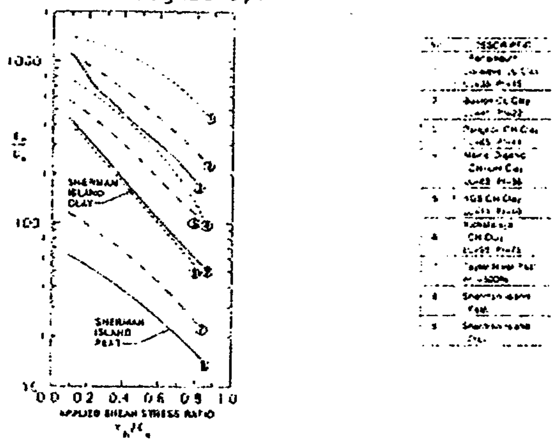

Figure 9 Direct Simple Shear Modulus Values

Accordingly, the levee behavior was analyzed using a deformation model to assess possible causes of the observed cracking and the effectiveness of the correction. The deformation analysis used a finite difference computer program, FLAC (Itasca Consulting Group, 1989), and a simplified model of the foundation conditions beneath the levee. The foundation soil strengths were similar to those developed for the stability analyses, and bilinear stress-strain relationships were used. With this relationship, the soil becomes very "soft" (i.e. extremely low modulus) when its shear stress exceeds its shear strength. The important initial modulus values for the soil deformations were selected from the DSS data, to give peat modulus values significantly lower than the silty clay, which in turn had lower values than the levee fill. Figure 10 shows the model and parameters used for the analysis. Undrained modulus values were used to assess the short-term, undrained response to load application.

The analytical procedure commenced with an assessment of the initial stress conditions under the existing levee, obtained by "turning on" the self weight of the levee and allowing the foundation soil stresses to equilibriate under those conditions. The deformations predicted due to application of the 80 ft wide berm outside the toe were then computed, to which were added the deformations resulting from application of the toe drain fill and connecting berm. Finally, the incremental deformations resulting from placement of the levee crown fill were computed.

Zone	Soil Type	Undrained Strength (psf)	Undrained Modulus (psf)	Density (pcf)
1	PEAT	200	1000	71
2		500	2500	77
3		600	4500	64
4	CLAY	300	6500	109
5		500	11000	108
6		600	17000	104
7	FILL		20000	106

Figure 10 Deformation Model

The movement data was recognized to be very approximate, almost conceptual in nature. Their value was to show how the foundation soils would respond to the corrective measures, in order that these measures could be suitably selected, applied and controlled. Placing the new toe berm on virgin soil outside the original levee section was calculated to result in sizeable deformations, as the peat tries to squeeze out from beneath the fill. This process moves soil towards the levee, creating an increased lateral resistance at the levee toe, which is further increased by placement of the toe drain and connecting berm fill. The result of this increased toe resistance is that negligible movements are indicated when the new crown fill is placed.

The response of the foundation soils to reinforcing of the levee toe is further demonstrated by the analyti-

cal results of Figure 11, which compares the zones of highly stressed soil in the levee foundation following crown restoration without the increased toe berms (Figure 11a) and with them (Figure 11b). The zone of highly stressed soil with the enlarged berms is much smaller than without them, indicating enhanced stability and reduced tendency for long-term creep deformations.

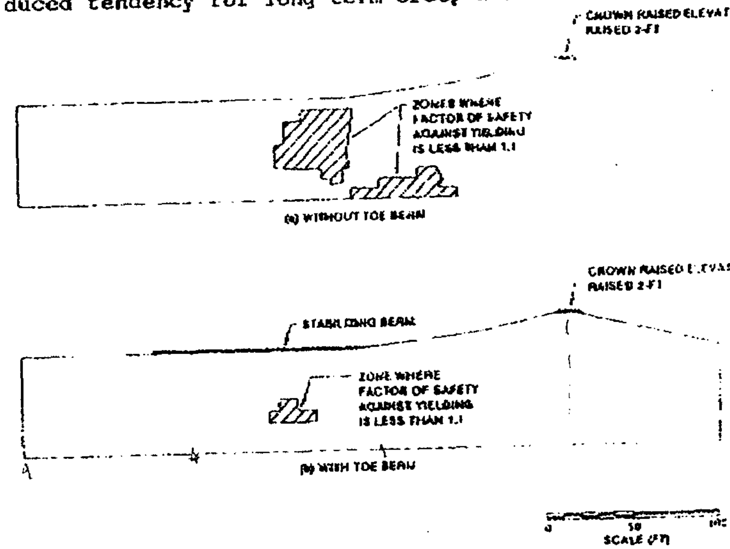

Figure 11 Deformation Analysis Stress Levels

MONITORING

At the time of writing this paper, nearly one yea[r] had elapsed after the corrective work was performed, including a period of abnormally high tides. Th[e] inclinometers were monitored on a monthly basis for th[e] first few months after construction, and approximatel[y] every other month thereafter. The levee surface has bee[n] inspected regularly by Reclamation District Personnel fo[r] cracking, wet areas, or any other abnormalities or sign[s] of distress. The inclinometers are showing no ongoin[g] movement near the levee embankment, although ongoin[g] movements of up to 4 in. have occurred under the new ber[m] fills. The deformation pattern is shown on Figure 12. The broad zone of shearing is suggestive of creep typ[e] movement in the newly loaded peat soils, and there is n[o] evidence of cracking or distress on the new fill surface and relatively few wet areas. The levee therefore appears to be stable and performing well.

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DISCUSSION OF LEVEE CRACKING PHENOMENA

The cracking problem addressed in the Sherman Island Threatened Levee work is an important phenomena which may occur widely, both on the island and in general where embankments are constructed on soft, compressible foundations. In this section, the potential causes and appropriate correction are discussed. The purpose is to summarize the information learned on this project so it will be readily available for future applications.

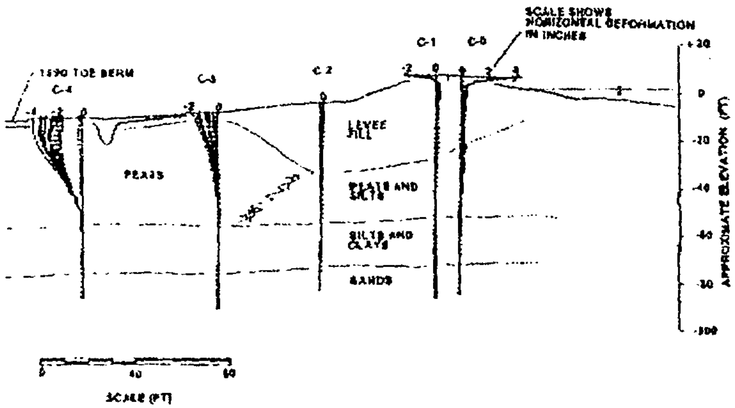

Figure 12 Deformations at the Crack Area

One hypothesis for the observed levee cracking was that it represented the initiation of a limiting equilibrium shear failure. The rapid decision to construct a new and wider levee berm was made to stabilize the levee against such a failure, recognizing that a series of anticipated high tides would apply additional destabilizing load on the levee. Assessing levee stability against a limiting equilibrium failure was therefore one of the first steps in the geotechnical engineering work. Parameters were conservatively selected for these materials. Nevertheless, the analyses showed that the initial levee configuration did not have stability factors of safety low enough to explain the observed cracking as being a failure condition. A stability failure would furthermore tend to "compress" the levee, whereas the observed behavior involved "stretching" the levee to produce the observed series of tension cracks. Stability failure was therefore not judged to be a realistic explanation for the observed cracking.

The observed cracking phenomenon may be more reasonably explained by considering the highly deformable and creep susceptible nature of the peat soils present beneath and to the landside of the levee embankment. The very low modulus values of the peat mean that it will typically deform considerably at applied loads significantly less than those required to cause failure. Thus conventionally adequate stability factors of safety are not a guarantee against excessive deformations and lateral movements of the peat material, with resultant cracking of the overlying fill. This situation is particularly acute when fill is placed over virgin peat, which may be very highly deformable until it has had time to consolidate and strengthen.

A likely explanation of the observed cracking is therefore that the pre-existing levee had suffered settlement and lateral movements of the underlying peat, particularly beneath its sizeable berms. These movements were probably related to a lowering of the water table on the landside of the levee, which by removing buoyancy effects has a net result similar to adding levee load. (Water table lowering was particularly likely to have occurred near the main area of cracking, where ongoing drought conditions were combined with a discontinuation of periodic flooding of the adjacent field to cause unusually dry conditions.) The peat deformations could then cause cracking of the levee which might typically start at the levee toe and progress into the embankment area. Reports of cracking of the landside slope of levees at time of drought are not uncommon and probably are frequently due to this cause.

Once cracked, the levee fill would tend to act as a series of adjacent blocks of soil on a soft base, and relative movements (e.g. as a heavy block settles and heaves up a lighter adjacent block) could be expected. Additional external loading could also trigger relative movements, which might explain why discovery of significant cracking followed a period of high tides and the placement of additional fill on the levee crown at the main area of cracking.

The greatest immediate danger following cracking may be the possibility of river water penetrating the crack system to flow freely through it, potentially eroding channels and ultimately washing out the levee. Sealing the cracks is therefore a priority. With a sandy levee, this can be done effectively by vibratory compaction, although it should first be ascertained that both overall and local levee stabilities are adequate and that localized liquefaction under vibratory loads is not a problem or can be managed. Treating a tendency for recurrence of

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the cracking then requires that the tendency for lateral deformations of the peat be reduced. This objective can be achieved by reducing the shear stress levels in the peat (i.e. by increasing slope stability factors of safety) and/or by stiffening the peat. Both these processes typically require adding more load outside the levee toe, which will typically result in additional settlement of the peat (which may itself be a problem) and may trigger additional lateral movement. The most cost-effective solution should therefore be established on a case-by-case basis. Once adequate stability is established, the best course of action if further cracking occurs may be compaction to seal the cracks, followed by local levee reshaping, if necessary.

SUMMARY

This paper has described how a major problem involving levee cracking on Sherman Island developed, was investigated, and corrective actions were taken to mitigate its effect. The cracking was addressed by widening the berms to increase levee stability, closing the cracks using vibratory compaction, and re-establishing the levee crown. The cause of the cracking is thought to be deformations of peaty foundation soils, probably related to extended drying of the island interior due to the drought and changes in farming practices.

ACKNOWLEDGEMENTS

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APPENDIX - UNITS CONVERSION

1 in. = 2.54 cm
1 ft = 0.3048 m
1 Acre = 4046.9 m^2 = 0.40469 hectares
1 psf = 0.04788 kPa

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