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Meindert A. Van
Deltares, Delft, The Netherlands

Cor Zwanenburg
Deltares, Delft, The Netherlands

John M. van Esch
Deltares and Delft University of Technology, Delft, The Netherlands

Michael K. Sharp
Engineering Research and Development Center, Vicksburg, Mississippi

Reed L. Mosher
Engineering Research and Development Center, Vicksburg, Mississippi

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Dr. Meindert A. Van
Deltares
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John M. van Esch
Deltares and Delft University of Technology
Delft, Netherlands

Michael K. Sharp, PhD, PE
Engineer Research and Development Center
Vicksburg, Mississippi, USA

Reed L. Mosher, PhD
Engineering Research and Development Center
Vicksburg, Mississippi

ABSTRACT

A peat levee at Wilnis in The Netherlands suddenly failed at the end of the relatively dry summer of 2003. On Monday, 29 August 2005, Hurricane Katrina struck the U.S. gulf coast and breached, among other failures, the 17th Street Canal. These failures triggered large research programs. In the Wilnis case, it was eventually deduced that the 5-m horizontal translation of the levee was triggered by a combination of reduced weight by evaporation, shrinkage and cracking of the peat material, and an increased head in the sand layer under the dike. A key factor in the 17th Street Canal failure was the formation of a gap between the wall and the levee fill on the canal side of the fill. Due to climate change, more extreme dry and wet periods, land subsidence, and increasing sea and river levels, the horizontal shifting due to cracking is becoming more significant in the safety assessments of levees. In this paper, aspects of horizontal failures during extreme dry or wet periods are elaborated. First, a geo-hydrologic design procedure to assess the consequences of droughts for cracked peat levees is presented. The design procedure is then validated with measurements of a peat levee, the Middelburgsekade, and extreme water table positions that are likely to occur once in a period of 400 years that have been predicted for this levee. Furthermore, the most dangerous cracks for the Wilnis case are indicated. Next, the performance of levees and floodwalls during Hurricane Katrina are presented. Finally, the failure of the 17th Street Canal breach in New Orleans is described in detail. Conclusions are drawn related to horizontal failures and location of cracks during extreme weather conditions.

INTRODUCTION

In the summer of 2003, a peat dyke at Wilnis in the Netherlands failed during a very dry period. The failure mode was not a Bishop type circular failure used in routine dike stability analyses in The Netherlands, see figure 1.

The failed dike segment was displaced horizontally over 6 m. The failure plane was found to lie at the boundary between peat and the underlying sand. It was concluded that a chain of events including weight loss and shrinkage of the peat, due to the dry weather conditions, are considered important factors causing failure. Analyses of peat behaviour made it clear that several processes, controlled by hydrological conditions in the unsaturated and saturated zones in the embankment, resulted in fracturing of the peat. The fracturing, along with a very high-strength anisotropy of the peat, resulted in a connection between the water in the canal and the water in the sand, thereby raising the piezometric head in the sand by several meters, allowing the dike to simply ‘float’ away for over 5 m, pushed by the water in the canal. A detailed description of this failure can be found in Bezuijen et al. (2005).

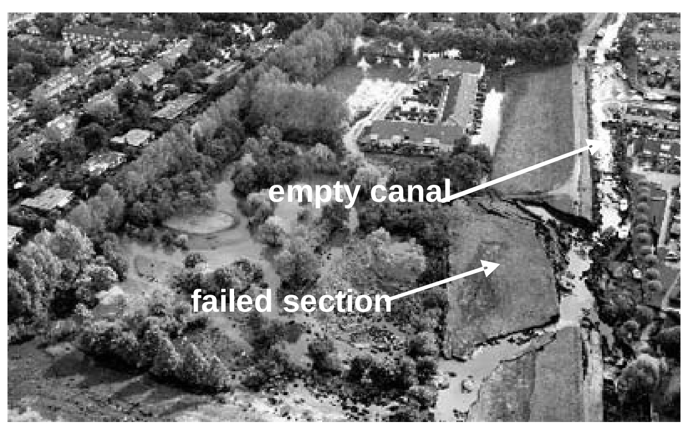

Fig. 1. Failed canal dike at Wilnis, August 2003.

Visual inspection of drying peat levees shows a cracked levee surface, including large and small cracks. Not all the observed cracks impose a threat to levee stability. However, from visual inspection alone, it is hard to distinguish between the dangerous and non-dangerous cracks. The Wilnis failure motivated a research program. Development of cracks and the influence of cracks on levee stability were important topics in this study. The study resulted in an evaluation tool for geo-hydrological boundary conditions for stability analysis and the results provided insight into crack development that will improve visual inspection during future extreme conditions.

GEO-HYDROLOGIC BOUNDARY CONDITIONS

The first part of the research project from the Wilnis case focused on a geo-hydrologic design procedure that used the unsaturated flow module PLAXIS (Brinkgreve and van Esch 2003). In addition to the unsaturated flow, an expression for saturation and relative permeability was derived. Furthermore, the influence of cracks is implemented in the model. An agro meteorological model that simulates evapo-transpiration and is coupled to the groundwater flow model was developed. This model also includes hysteresis in drying and wetting of the soil. In van Esch et al. (2007), this complex model and its implementation and validation in Plaxflow has been described.

For the validation of this model, the consequences of droughts for an existing peat dike, the 'Midedelburgsekade' near Boskoop in the Netherlands, were studied for successive periods of drought and precipitation over a period of three years starting 1 January 2004.

Based on cone penetration tests and Begemann borings, a ground profile was composed and a Plaxis model was generated. Figures 2 and 3 show the layering of peat and clay and displays the points where the groundwater heads were measured during a period of three years.

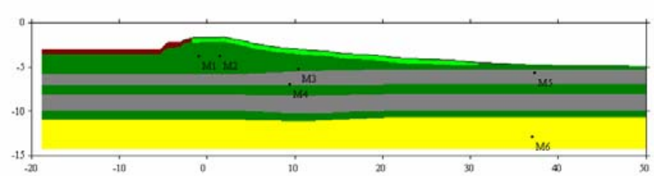

Fig. 2. Ground water flow model.

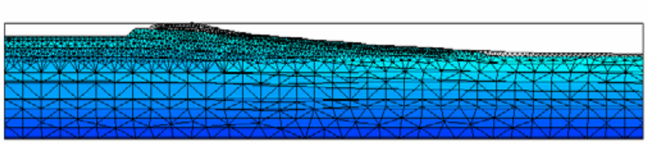

Fig. 3. Position water table in August 2005.

The water pressure field obtained for the summer of 2005 is shown in Figure 3 as an example. Measured heads in the observation wells and calculated heads in corresponding nodes are given over a period of three years in Figure 4, starting (= day 0) on 1 January 2004. Both signals show a seasonal variation due to evapo-transpiration and a high-frequency response mainly due to precipitation.

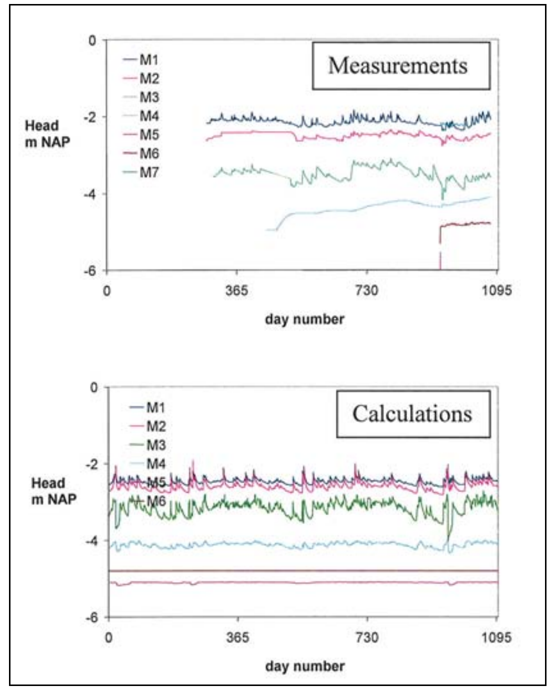

Fig. 4. Measured and calculated groundwater levels

From these results, it was concluded that calculated data resembled the measured data quite well although it was hard to reproduce measured head data exactly because the measurement data already showed a wide variability due to the natural spatial variability of peat material and the three-dimensional shape of the levee surface.

Precipitation and evapo-transpiration data were analyzed over a period of 40 years. For each year, maximum net precipitation and maximum net evapo-transpiration data were assembled in a histogram. Figure 5 shows the cumulative frequency distribution of maximum values of daily outflux data per year and the fitted cumulative Gumbel distribution.

The same procedure was repeated for weekly averaged and monthly averaged data. Extreme precipitation events that will take place once in a period of 400 years are: 101 mm/d during a day, 21 mm/d during a week, and 8 mm/d during a month. Calculations give an extreme evapo-transpiration over the same period as 10 mm/d during a day, 8 mm/d during a week, and 6 mm/d during a month. Calculations pointed out that loading the dyke with an extreme precipitation (extreme rainfall) over a week represents the upper bound water table for an extreme wet period. Loading the dyke with an extreme evapo-transpiration over a month gave the lower bound (i.e., dry period) for the position of the water table.

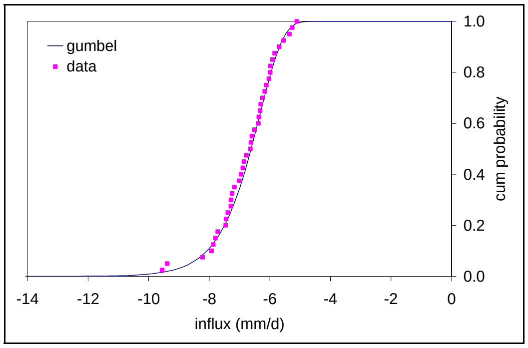

Fig. 5. Extreme daily inflow data

The maximum difference in position for extreme loading occuring only once every 400 years gives a difference in the position of the water table of 1.3 m.

Boundary conditions for a range of loading periods have been calculated for peat dykes in the Netherlands. These conditions can be used in groundwater flow calculations. The obtained water pressure fields for each loading case are prescribed in stability calculations that assess the safety of the peat dyke for the extreme loading period. This methodology is now proposed as a design procedure. Furthermore, the model now can predict drying and wetting of the levee due to climate conditions, which is the first step to predict cracking of levees.

INFLUENCE OF CRACKS IN THE WILNIS CASE

An inventory of cracks in peat levees during the dry period of 2003 shows that in most cases cracks occur longitudinally in the crown of the levee (Lubking and Van 2007). In some cases, cracks in the slope or an angle of 45 degrees or perpendicular to the levee are observed. Due to decreasing water content, the peat material will shrink. The deformations then will be downward of the slope and cracks will occur in the crest due to tensile forces exceeding the tensile strength. If these cracks occur without vertical displacements, shrinking of the levee is assumed the cause; if vertical deformation is observed, a stability problem may be the cause.

The question remains whether or not a shrinking crack in a longitudinal direction can initiate a stability problem. Therefore, for the Wilnis case, a number of FEM calculations have been made with cracks from 2 to 4 m deep (see figure 6); the cracks are numbered with letters A to G.

In figure 7 the shear strain is shown for the situation without cracks. Cracks A and B give a small reduction of 2 to 3% in the safety factor; for Cracks C and D the reduction is 1% or less. Furthermore, the calculations show that Cracks C and D will close due to deformations. This means that during a real stability problem in the levee, Cracks C and D will also close. Cracks E, F, and G reduce the initial stability; but due to the deformations, these cracks will close with little influence on the final stability.

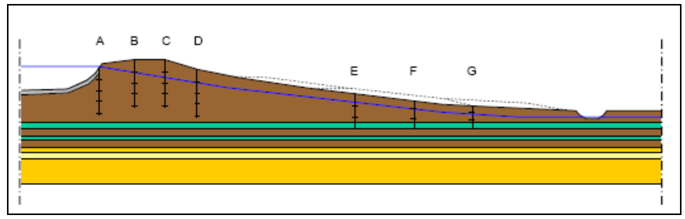

Fig. 6. FEM calculation, A to G represent cracks in geometry.

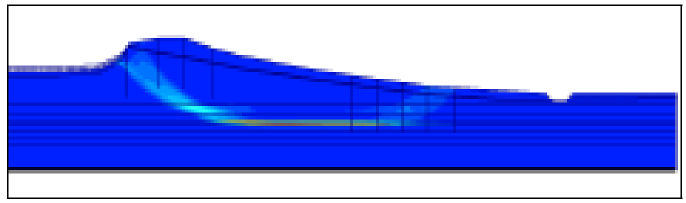

Fig. 7. Calculated shear strain.

The influence of water in Crack B is studied. If the crack is fully filled with water, the influence on the calculated safety factor is a reduction of 16%, which is a substantial reduction.

It is concluded that the most dangerous cracks in the longitudinal direction are those on the outer site of the levee crown that are or can become filled with water.

NEW ORLEANS FLOODWALL AND LEVEE PERFORMANCE ANALYSIS

On Monday, 29 August 2005, Hurricane Katrina struck the U.S. gulf coast. The effects of the storm were felt in the New Orleans area during the early morning hours. The storm produced a massive surge of water on the coastal regions that overtopped and eroded away levees and floodwalls along the lower Mississippi River in Plaquemines Parish, along the eastern side of St. Bernard Parish, along the eastern side of New Orleans East, and in locations along the Gulf Intracoastal Waterway (GIWW) and the Inner Harbor Navigation Canal (IHNC). Surge water elevated the level of Lake Pontchartrain, and shifting storm winds forced the lake water against the levees and floodwalls along its southern shores and New Orleans outfall canals resulting in high surge levels in the IHNC, the Mississippi River Gulf Outlet (MRGO), the GIWW, and the Mississippi River.

The performance of levees and floodwalls varied significantly throughout the New Orleans area. The investigation conducted post-Katrina indicates that the two main causes of breaches in the floodwall and levee system were erosion due to overtopping and instability due to soil foundation failure. Breaches due to instability of floodwalls occurred at one location on the 17th Street Canal, two locations on the London Avenue Canal, and one location on the IHNC, as shown in Figure 8.

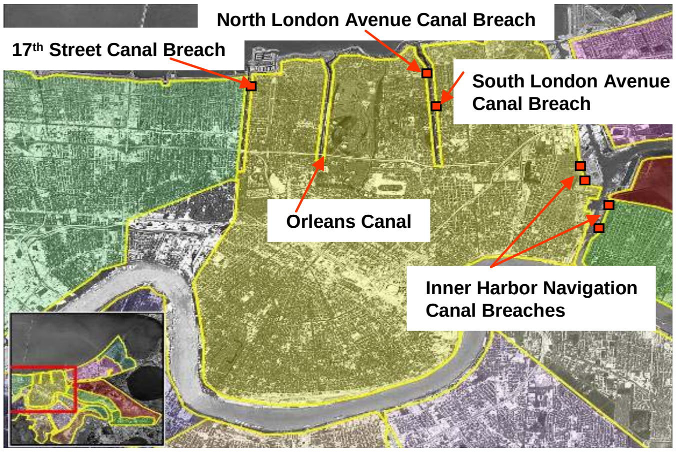

Fig. 8. Location of breaches in Orleans Parish, East Bank

Soil conditions in the area of the New Orleans outfall canals have been determined through evaluation of existing and recently drilled engineering borings, earlier geologic mapping studies of the area (Dunbar et al. 1994, 1995; Dunbar, Torrey, and Wakeley 1999; Saucier 1994), and new studies performed since August 2005. Geologic mapping of the surface and subsurface in the vicinity of the canal failures identifies distinct depositional environments related to Holocene (less than 10,000 years old) sea level rise and deposition of sediment by Mississippi River distributary channels during this period. Overlying the Pleistocene surface beneath the 17th Street Canal are approximately 50 to 60 ft (15 to 18 m) of shallow water, fine-grained sediments consisting of bay sound or estuarine, beach, and lacustrine deposits as indicated in the cross section shown in Figure 9. Overlying this shallow water sequence are approximately 10 to 20 ft (3 to 6 m) of marsh and swamp deposits that correspond to the late stages of deltaic sedimentation as these deltaic deposits became sub aerial. A buried barrier beach ridge extends in a southwest to northeast direction in the subsurface along the southern shore of Lake Pontchartrain. A stable sea level 10 to 15 ft (3 to 4.5 m) lower than current levels permitted sandy sediments from the Pearl River to the east to be concentrated by longshore drift and formed a sandy spit or barrier beach complex in the New Orleans area (Saucier 1994).

The site of the levee breach at the 17th Street Canal is located on the northern side of the beach ridge where the sand ridge is thinner. There is a layer of clay between the sand and the marsh layer; whereas both of the London Canal breaches are located over the thickest part of the barrier beach ridge complex, the sand deposit lies directly beneath the marsh layer, as shown in Figure 9.

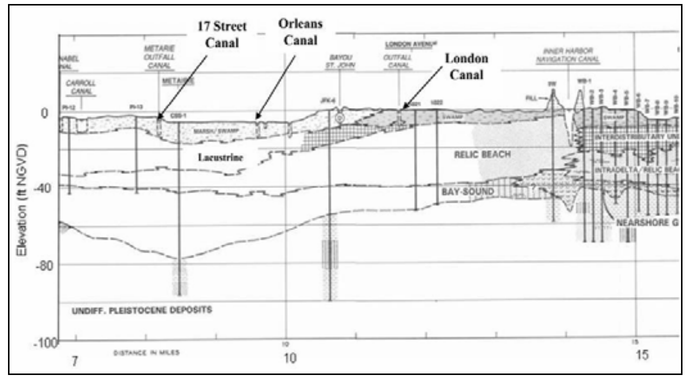

Fig. 9. Geological cross section extending west to east across eastern Jefferson Parish and into western Orleans Parish. Section runs from near the 17th Street Canal to the Inner Harbor Navigation Canal. Major outfall canals in Orleans Parish are noted on the section.

BREACH AT 17TH STREET CANAL

Observations made of the breach at the 17th Street Canal show that the most likely cause of the breach is a soil foundation failure. Figure 10a shows an aerial photo of an approximately 450-ft (140-m) breach in the floodwall along the east side of the 17th Street Outfall Canal south of the old Hammond Road Bridge. Figure 10b shows that a section of levee has moved more than 40 ft (12 m) inward to the landside. It appears that the water flowing through the breach washed away the remaining levee section making up the breach. The top of the I-wall section of the floodwall in the breach can be seen adjacent to the levee section that moved into the landside.

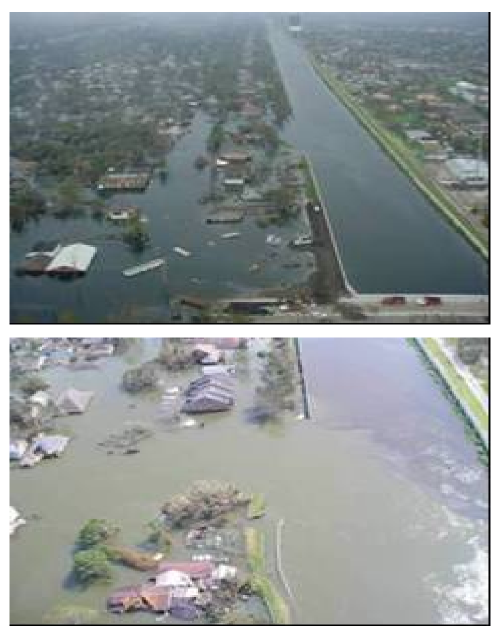

Fig. 10. (a) aerial view of breach, (b) aerial view showing I-wall and embankment translation.

After the emergency closure was complete and the water levels were drawn down, large blocks of the marsh were found strewn in neighborhoods surrounding the breach. A close examination of the marsh blocks revealed that an approximately 1-ft (0.3-m) clay layer was attached to the bottom of the marsh blocks. In order to inspect the failure plane or zone, a backhoe trench was excavated to expose a vertical surface through the slide block. Observations made in the trench confirmed that a portion of the clay layer, which was initially located beneath the marsh layer, was displaced upward and over a portion of the marsh layer by the lateral displacement that occurred during the failure. The failure plane of the slide block was within the clay under the levee, and the failure plane came upward through the marsh layer further landward.

A considerable number of borings had been made in the breach area and in neighboring areas before the failure. Additional borings have been drilled, cone penetration tests have been performed, and test pits have been excavated since the failure. Several hundred unconfined compression tests and unconsolidated-undrained (UU) tests have been conducted on the soils at the site. As shown in Table 1, the subsurface in the breach area was divided into six soil types over the depth of the investigation.

Table 1. Major Soil Groups at the 17th Street Outfall Canal Breach Site

Layer	Elev. Top of Layer, ft (m)	Elev. Bottom of Layer, ft (m)	Soil Type	Consistency
Embankment	5 (1.5)	-11.5 (-3.5)	Clayey (CL and CH	Stiff
Marsh	-11.5 (-3.5)	-16.5 (-5.0)	Organic/ Peat	Very Soft
Lacustrine	-16.5 (-5.0)	-36.5 (-11.1)	Clays (CH)	Very Soft
Bach Sand	-36.5 (-11.1)	-45 (-13.7)	Sand	
Bay sound/ Estuarine	-45 (-13.7)	-75 (-2.9)	Clayey (CH)	Stiff to Very Stiff
Pleistocene (Undiff.) Prairie Formation	-75 (-22.9)		Clays Generally CH with some sand	Stiff

The levee fill is compacted CL or CH material with an average liquid limit of about 45%. Beneath the fill is a layer of marsh 5 to 10 ft (1.5 to 3.0 m) thick. The marsh is composed of organic material from the cypress swamp that occupied the area and silt and clay deposited in the marsh. The average moist unit weight of the marsh is about 80 pcf (pounds per cubic feet) (12.57 kN/m^3). Beneath the marsh is a lacustrine clay layer with an average liquid limit of about 95%.

Having been covered and kept wet by the overlying layer of marsh, the clay is normally consolidated throughout its depth. The measured shear strengths of the levee fill scatter very widely, from about 120 psf (5.7 kN/m^2) to more than 5,000 psf (240 kN/m^2), and cannot be interpreted without applying judgment. Emphasis was placed on data from UU tests on 5-in.- (0.13-m-) diameter samples, which appear to be the best quality data available. The value s_u = 900 psf (43 kN/m^2) is a reasonable value to represent the levee fill.

The marsh (or peat) deposit is stronger beneath the levee crest where it was consolidated under the weight of the levee and weaker at the toe of the levee and beyond where it was less compressed. The measured shear strengths of the marsh scatter very widely, from about 50 psf (24 kN/m^2) to about 920 psf (44 kN/m^2). Values of s_u = 400 psf (19.2 kN/m^2) beneath the levee crest and s_u = 300 psf (14.4 kN/m^2) beneath the levee toe appear to be representative of the measured values. The clay, which has been found to be the most important material with respect to stability of the I-wall and levee, is normally consolidated. Its undrained shear strength increases with depth at a rate of 11 psf (0.5 kN/m^2) per foot (0.3 m) of depth. This rate of increase of strength with depth corresponds to a value of s_u /p' = 0.24. There is very little scatter in the results of the Cone Penetration Test with pore water pressure measurements—piezocone test (CPTU), and these values provide a good basis for establishing undrained strength profiles in the clay. The undrained strength at the top of the clay is equal to 0.24 times the effective overburden pressure at the top of the clay. With this model, the undrained shear strength of the clay varies with lateral position and is greatest beneath the levee crest where the effective overburden pressure is greatest. The undrained shear strength is the least at the levee toe and beyond where the pressure is lowest, and varying with depth, increasing at a rate of 11 psf per foot (1.7 kN/m^2/m) at all locations. Strengths from this analysis are compared to the design strengths in Table 2.

Table 2. Comparison of Strengths Used in Design with Those Determined in This Analysis

Material	Strength Used for Design	Strength Model Based on Data from This Analysis
Levee Fill	s_u = 500 psf, (23.9 kN/m^2) $\varphi = 0$	s_u = 900 psf, (43.1 kN/m^2) $\varphi = 0$
Marsh	s_u = 280 psf, (13.45 kN/m^2) $\varphi = 0$	s_u = 400 psf, (19.2 kN/m^2) $\varphi = 0$ beneath levee crest s_u = 300 psf, (14.4 kN/m^2) $\varphi = 0$ beneath levee toe

Limit equilibrium analyses were used to examine stability of the levee and I-wall. The results of these analyses are interpreted in terms of factors of safety and probabilities of failure. Stability analyses were performed for three cross sections within the breach area using the shear strength model in column three of Table 2. The results of these analyses were compared with the results of the analyses on which the design of the I-wall was based; additional analyses were performed for the design cross-section geometry and shear strengths using Spencer's method and the computer program SLIDE. The SLIDE analysis results were checked using UTEXAS4. The calculated factors of safety decreased as the elevation of the water level on the canal side of the wall increased. Smaller factors of safety were calculated when it was assumed that a gap existed between the wall and the soil on the canal side of the wall, with hydrostatic water pressures acting within this gap, increasing the load on the wall. The factors of safety calculated in the design analyses were higher than the factors of safety calculated for the conditions that are believed to best represent the actual shear strengths, geometrical conditions, and loading at the time of failure. The principal differences between the design analyses and the conditions described in this report relate to (1) the assumption that a gap formed between the wall and the levee soil on the canal side of the wall, and (2) the fact that the design analyses used the same strength for the clay beneath the levee slopes and the area beyond the levee toe as was used for the zone beneath the crest of the levee. The strength model used in this analysis has lower strengths beneath the levee slopes and beyond the toe.

Centrifuge physical modeling of the 17th Street East Bank levee was conducted. The centrifuge model results revealed that the gap formation had a major contribution to all of the breaches on the outfall canals. In all of the scale models where the toe of the sheet-pile wall terminated in the clay layer, such as the 17th Street Canal breach location, a translational failure occurred through the clay when the gap opened and filled with water. This was clearly seen in the instruments recording movement of the wall and in the video imagery. More details can be found in IPET 2007.

Two-dimensional finite element soil-structure interaction analyses with PLAXIS and FLAC using a nonlinear hyperbolic soil stress-strain model were conducted to provide a third approach to the development of a complete understanding of the 17th Street Canal breach mechanism. The finite element analyses show that a gap formed as the water rose on the canal side of the wall. The criterion for gap formation was earth pressure against the wall was less than the hydrostatic water pressure at that depth. When that condition was reached, the finite element mesh was adjusted to separate the soil elements from the wall elements with a gap. The gap began to open when the water in the canal rose to elevation 6.5 ft (2.0 m). Eventually, the gap extended to the tip of the sheet pile, which was at the lacustrine clay-marsh interface. Factors of safety were computed using the strength-reduction method. The strength-reduction method involves performing a series of finite element analyses using values of the strength parameters c and φ (or c' and φ') that are reduced by dividing them by assumed values of factor of safety. The correct factor of safety, as determined by this method, is the smallest value that results in unstable conditions in the analysis. When the gap developed and filled with water, the factor of safety decreased suddenly by about 25%, from 1.45 to 1.16. The development of a gap, which immediately filled with water, resulted in a marked increase in calculated displacements. As the water in the canal rose from 6.5 to 9.0 ft (2.0 to 2.7 m) NGVD, the maximum lateral deformation increased from 1.6 to 5.3 ft (0.5 to 1.6 m) and the factor of safety decreased from 1.16 to 0.98. A figure representing the analysis is shown in Figure 11. More details can be found in IPET 2007.

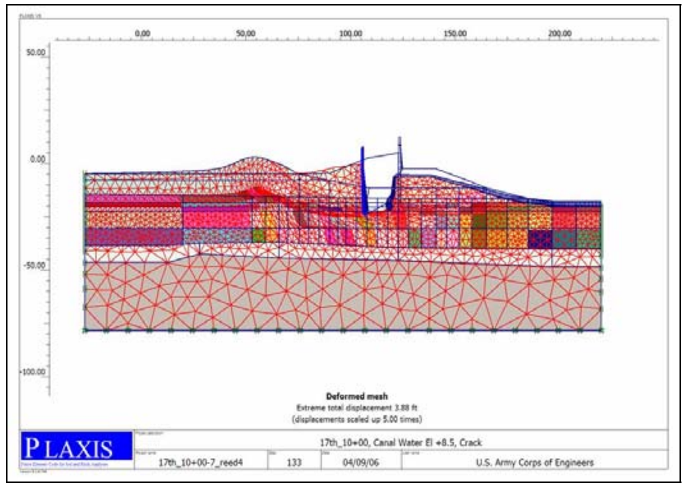

Fig. 11. Deformed mesh for canal water elevation 9.0 ft (2.7 m) (canal water elevation not to scale in figure).

Eyewitness reports indicate that the breach began to develop about 6:00 a.m. on Monday, 29 August 2005, and was fully developed before 9:00 a.m. Field evidence, analyses, and physical model tests show that the breach was due to instability caused by shear failure within the clay in the foundation beneath the levee and the I-wall with a rupture surface that extended laterally beneath the levee and exited upward through the marsh layer. A key factor in the failure was the formation of a gap between the wall and the levee fill on the canal side of the wall, which allowed water pressure to act on the wall below the surface of the levee. Another important factor was the low shear strength of the foundation clay beneath the outer parts of the levee and beyond the toe of the levee.

CONCLUSIONS

This paper presents a geo-hydrologic design procedure for extreme drying or wetting boundary conditions. Extreme wetting boundary conditions for the Dutch situation should be applied over a week to produce the highest position of the water table; for dry conditions, the one-month period is indicative of the lowest water table. For the Middelburgsekade, it is concluded that the maximum difference in position for extreme wetting or drying as it occurs once every 400 years gives a difference in the position of the water table of 1.3 m.

A study on the location of the most dangerous cracks on the levee shows that the most dangerous cracks in the longitudinal direction are those on the outer side of the levee crown that are or can become filled with water.

From the 17th Street Canal breach study, it was concluded that a key factor in the failure was the formation of a gap between the wall and the levee fill on the canal side of the wall, allowing water pressure to act on the wall below the surface of the levee.

The overall conclusion is that cracks close to the outer side of the crown, which can be filled with water, are extremely important to be recognized and dealt with, especially when observed by visual inspection during extreme conditions.

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