Investigation into the cause of earthen embankment instability along the "V-line" artificial levee in Marrero, Louisiana, USA

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INVESTIGATION INTO THE CAUSE OF EARTHEN EMBANKMENT INSTABILITY ALONG THE “V-LINE” ARTIFICIAL LEVEE IN MARRERO, LOUISIANA, USA

A Thesis
Submitted to the Graduate Faculty of the Louisiana State University and Agricultural and Mechanical College in partial fulfillment of the requirements for the degree of Master of Science

In
The Department of Geology and Geophysics

by

Jason Matthew Hicks
Bachelor of Science in Physics, Dillard University, 2008
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The “V-line” levee, located in Marrero, LA, has a crack along its crest, measuring approximately 100 meters in length, 30 centimeters in depth, and 30 centimeters in width. This crack is a sign of levee instability. Seismic shear wave, CPT, and laboratory shear strength data were collected and processed to identify the cause of the instability. A zone of low seismic shear wave velocity was interpreted approximately 3 meters deep from the berm of the levee. Because of the large spacing between laboratory shear strength test sampling and CPT sites, a similar zone of low shear strength was not found in the CPT and laboratory shear strength data. High contents of peat and organic clay were interpreted within and beneath the levee fill using CPT data; the location of the highest contents of organic clay and peat correspond to the location of the levee instability. Organic material tends to have a lower shear strength and higher water retention which can cause lower shear strengths and higher shear stress after heavy rainfall. Based on the location of the crack at the crest of the levee and the zone of low seismic shear wave velocity, it is likely that the crack at the crest of the levee is a precursor to slope base failure.
CHAPTER 1. INTRODUCTION

Artificial levees are of great importance in populated coastal floodplains because they protect otherwise seasonally flooded areas (Saucier, 1994). Artificial levees, also called embankments or dikes, are built using earth or rock and are used primarily to protect inhabited areas from flooding caused by an increase in the water level of rivers, lakes, streams, etc. (Neuendorf et al., 2005). Artificial levees have been used for over 4,000 years, beginning with the Harappan people of the Indus valley in 2300 BC who used levees to control the Indus and as an irrigation system for their crops (Frazee, 1997).

The importance of artificial levees may increase over time especially if protected floodplains are actively drained. Normally, floodplain deposits have a high organic content. The high organic content of these soils makes them naturally compact over time. In addition, when water is drained from the area more oxygen enters the soils and they begin to decompose at a higher rate (Ingebritsen, 1999). Because levees prevent the influx of new sediments into the floodplain, subsidence in protected areas may increase and the potential for economic loss increases substantially.

One example of this type of subsidence is in the floodplains of the Sacramento-San Joaquin delta. Since the area was drained and levees built to protect it from flooding, in the late 1800s, the land has subsided up to 15 feet (Ingebritsen, 1999) (Fig. 1.1).

Organic material tends to have lower shear strength, the shear stress necessary to fail a material, than other material, so shear failure of slopes is not uncommon in floodplains (Mesri, 2007). An average field vane shear strength value of 10 kPa was reported by Yogeswaran (1995) for tropical peat. The floodplains of the Sacramento-San Joaquin delta are an example of this, with about 100 levee failures in the area since the early 1890s (Ingebritsen, 1999).

Another example of a floodplain area susceptible to artificial levee failure is the Greater New Orleans area, (Louisiana, USA). This area was built by floodplain deposition from the Mississippi River. In particular, the 17th St. canal levee was breached during hurricane Katrina on August 29, 2005 at 9:30 AM (Nelson, 2008) (Fig. 1.2). The breach occurred along an approximately 150m section of the eastern side of the floodwall. During the breach, the section of failed levee was translated, as a block, eastwardly. Chunks of peat were found in the debris after the failure. Peat and organic clay, which is clay with a high organic content, were located beneath the levee from directly below the artificial levee fill to 6m below the artificial levee fill according to U.S. Army Corps of Engineers design documents (1990) (Fig. 1.3).
Figure 1.1 Map of the Sacramento-San Joaquin Delta area with the amount of land subsidence since the area was settled indicated by shades of brown (Ingebritsen, 1999).
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Figure 1.3(b) Geological cross-section along the eastern side of the 17th St. Canal based on data in the U.S. Army Corps of Engineers design documents (1990). Notable in the cross-section is high content of both peat and organic clay in the area of the breach. (Nelson, 2008)
The highest content of organic material beneath the 17th St. canal floodwall is in the area of the breach; its low shear strength is likely a contributing factor to the failure of the floodwall.

1.1 Study Location

“V-line” levee, which is in Marrero, Louisiana, USA, (fig. 1.4 and fig. 1.5) and is approximately 13 km south of the New Orleans Central Business District, is the area of interest in this study.

1.2 The Problem

A section of the “V-line” levee is apparently unstable, evidenced by a crack along the crest of the levee approximately 1 km from the southern vertex of the levee (Figure 1.6), measuring approximately 100 m long (Fig. 1.7) and a maximum of 0.3 m deep and wide (Fig. 1.8). A crack near the crest of a slope is a common precursor to slope failure in the case of slopes composed of a cohesive soil, such as clay; cracks of this type are referred to as tension cracks (Bromhead, 1986).
Figure 1.5 Outline of the borders of Louisiana (left), the area near the “V-line” levee, with the “V-line” levee outlined in red (right) (U.S. Geological Survey, 2006; Google Earth 5, 2010)
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Figure 1.8 A photograph, taken on August 9th 2007, of the deepest and widest section of the crack in the crest of the “V-line” levee, measuring approximately 0.3 meters deep and 0.3 meters wide. Dr. Juan Lorenzo (left) and James Merritt (right) for scale.
1.2.1 Causes of Levee Instability

Slope failure occurs when the shear stress imposed on slope and the underlying material exceeds the shear strength of the materials in the subsurface. Shear strength is the maximum shear stress that a material can withstand before shear failure occurs. It is important to determine the shear strength of materials in the subsurface in order to locate areas of instability.

Earthen slope instability has four main causes which are: (1) the slope is too high or steep for the materials of which it is composed; (2) the materials beneath the slope are too weak to sustain the slope at its present profile; (3) pore pressures are too high, which causes a decrease in normal stress on the plane of failure and thus a decrease in apparent shear strength; or (4) some external load, such as load from structures built upon the slope, increases the shear stress imposed on the underlying material (Bromhead, 1986).

The instability of the “V-line” levee is likely a combination of several of the main causes of earthen slope instability. The weight of the levee and the low shear strength of the material beneath it are a likely contributing factor to the instability of the levee. No data has been collected along the levee that would indicate an increase in pore pressure, so this cause of levee instability cannot be confirmed. Because the organic material has the potential to have a high permeability and water retention, an external load from rainwater could contribute to the instability of the levee during and after heavy rainfall events.

The “V-Line” levee was built upon inland swamp and interdistributary deposits (Fig. 1.9), which are characterized by clay, organic clay, and peat (Saucier, 1994). The levee was likely built using nearby material, it is also primarily made up of clay, organic clay, and peat. Both clay and peat have low shear strengths, the shear strength of peat lower than that of clay (Fig. 1.10). Because of the low shear strength of these materials, the foundation on which the levee is built is weak, which increases the likelihood of failure.

Because organic material has a large range of hydraulic conductivity and water retention values, cause (4) could contribute to the instability of the levee during heavy rainfall events. A soil with a relatively high water retention and hydraulic conductivity will absorb and retain rainwater, thus increasing the soil’s weight and the stress on the underlying material. An organic material like undecomposed moss can absorb water because of its high hydraulic conductivity, approximately 3.810 (cm/sec)10^-5, but is unable to retain water because of its low water retention. An organic material like a well decomposed peat is less able to absorb water because of its low hydraulic conductivity, approximately 0.45 (cm/sec)10^-5. An intermediate organic material like a moderately decomposed woody peat can absorb water because of
its moderately high hydraulic conductivity, approximately 496 (cm/sec)10⁻⁵, and is able to retain water because of its moderately high water retention (Boelter, 1968) These physical properties of peat could lead to an increase in the stress imposed on the “V-line” levee and exacerbate levee instability.

The crack that has formed near the crest of the “V-line” levee can contribute to the further weakening of the levee. The crack introduces a secondary permeability and makes it easier for water to infiltrate the levee.

1.2.2 “V-line” Levee Crack Formation

The crack at the crest of the “V-line” levee is likely a tension crack. Tension cracks form at the onset of levee instability as a failure surface forms, the failing block of material pulls away from the levee, and tension is generated. According to Rankine active earth pressure theory (Rankine, 1857) (Fig. 1.12), because the materials at shallow depths have little to no lateral pressure resisting the tension caused by the failing block, a tension crack can form only at shallow depths.

Figure 1.9 Color coded map illustrating the modern deltaic facies at the ground surface in the area surrounding the “V-line” levee (Saucier, 1994)
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Figure 1.11 Water retention curves for several northern Minnesota peat materials (Boelter, 1968)
Figure 1.12 Horizontal pressure as defined by Rankine active earth pressure theory given a cohesive. $Z_t$ is the maximum depth to which tensile stress is present (Duncan and Wright, 2005)

$$Z_t = \frac{2c}{\gamma \tan(45 - \frac{\phi}{2})}$$

$c =$ cohesion

$\gamma =$ unit weight

$Z_t =$ maximum depth of tension
1.2.3 Soil Sensitivity

Lorenzo (2010) interprets a zone of low shear wave velocity beneath the “V-line” levee near the crack at the crest of the levee which may be explained by soil deformation near the plane of failure. After clay undergoes strain it goes from an undisturbed state to a remoulded state and its shear strength may be reduced. The ratio between the undisturbed and remoulded shear strength or the clay is the sensitivity of the clay. The values of sensitivity measured in clay samples can be classified as follows:

~1.0: insensitive clays
1-2: clays of low sensitivity
2-4: clays of medium sensitivity
4-8: sensitive clays
>8: extra-sensitive clays
>16: quick-clays

(Skempton and Northey, 1952)

If a zone of shearing has formed beneath the levee and some shear has occurred, the sensitivity of the clay would account for the zone of low shear wave velocity interpreted beneath the levee.

1.3 Geologic Background

1.3.1 Mississippi River Deltaic Environment

The “V-line” levee is made up of marsh and swamp deposits, has marsh and swamp deposits several meters beneath it, and other deltaic facies at further depths. The sediment in the shallow subsurface beneath the “V-line” levee was laid down during the formation of the St. Bernard and Plaquemines-Modern Delta Complexes (Frazier, 1967). A delta is a body of sediment deposited in the zone of interaction between a river, or other fluvial system, and an ocean or other standing body of water (Nichols, 1999). The Mississippi river delta complexes formed as a result the multiple progradations of the Mississippi river into the Gulf of Mexico (Frazier, 1967).

A delta lobe is a portion of a delta complex; this feature forms during a relatively short period of time and is the result of a single or a discrete set of distributaries (Saucier, 1994). The formation of a delta complex is a cyclic process which involves phases of progradation, aggradation, and transgression. The
Mississippi river delta in the area of the “V-line” levee has gone through progradation and is currently in the aggradational phase. It has not reached the transgressive phase, so no transgressive phase facies are expected.

- **Delta Progradation**

  As interpreted from CPT data (See chapter 2) there is a layer of clay beneath “V-line” levee from approximately 14 to 22 meters from the crest of the levee; this is likely the result of prodelta sediment deposition. The prodelta is a thick layer of fine-grained sediments located between the delta front and marine environments stratigraphically. The prodelta is formed as fine-grained sediment, carried ahead of the delta front, drops from suspension and is dispersed by waves and currents. Owing to currents and wave action, prodelta deposits are laid down in a broad fan and are the most homogeneous, widespread, and continuous of all deltaic environments (Saucier, 1994).

  As interpreted from CPT data (See chapter 2) there is a layer of intermixed silt and sand beneath the “V-line” levee from approximately 8 to 14 meters from the crest of the levee; this is likely the result of delta front sediment deposition. The delta front forms as water from a deltaic distributary advances into a body of water and its velocity of abruptly reduced. Sand and silt are deposited ahead of the distributary mouth and are redistributed laterally by waves; the redistribution of sediment by waves generates a broader body of sediment called the delta front (Boggs, 1995; Nichols, 1999).

- **Delta Aggradation**

  After the prodelta and delta front sediments are laid down, upstream of the mouth of the delta sediment is built upward in a delta building phase called aggradation. All of the sediment deposited above the delta front deposits beneath the “V-line” levee is likely the result of aggradation. During the phase of aggradation, natural levees and the fine-grained sediment and organic material of the deltaic plain are deposited (Boggs, 1995).

- **Natural Levee Formation**

  As the Mississippi river floods, sediment held in suspension is deposited along the banks of the river. During each period of flooding, sediment builds upward and outward to form a broad ridge composed of silts and sands that slopes gently away from the parent channel called a natural levee (Frazier, 1967; Saucier, 1994). Natural levees formed by the southern reaches of the Mississippi river are composed of more fine-grained material, primarily silty clay, because the grain size of particles held in suspension decreases downriver.
Figure 1.13 Deposits resulting from the Mississippi River Delta formation (Boggs, 1995)

- **Deltaic Plain**

  The deltaic plain environment found outside the reaches of the natural levees is determined by the amount of sediment provided by river flood deposits. Early in the development of natural levee, flooding is frequent and vegetation is unable to grow due to high inorganic sedimentation rates, therefore inorganic clay dominates. As levees build, they confine progressively higher water stages, flooding becomes less frequent, and inorganic sedimentation rates decrease; as sedimentation rates decrease vegetation is no longer “choked out” by inorganic sedimentation and marsh environments can take hold. The dominance of inorganic clay or organic deposition is also controlled by an environment’s distance from the river system. Close to the river system, sedimentation rates are higher and inorganic clay dominates, while further from the river marsh vegetation dominates because flood-borne sediments are unable to extend to that distance. (Frazier, 1967)

1.3.2 Mississippi River Deltaic Sediments

  There a number of different sediment types found beneath the “V-line” levee, including clay, sand, and peat, each with unique physical characteristics. The physical characteristic of each sediment
type control the stability of the foundation beneath the levee, for example sand is permeable, allowing flow through or below the levee, and peat has low shear strength and high compressibility which lead to shear failure and subsidence. Therefore, it is necessary to understand the physical characteristics of each sediment type in order to explain the cause of levee instability.

- **Sand vs Clay**

  An artificial levee built on floodplain deposits is prone to be less stable than one built on an existing natural levee of equal thickness because of the differences in the shear strength behaviors of clay and sand. The shear strength (τ) of a soil is a combination of cohesion (c) and angle of internal friction (Φ), expressed by the Coulomb failure envelope. The c of a soil is its shear strength under no confining stress and the Φ of a soil is its increase in shear strength with increase in normal stress (σn) on the plane of failure. The relationship between τ, c, Φ, and σn can be described with Mohr-Coulomb failure criterion (Equation 1.2).

  Clay has significant c and low Φ relative to sand, while clean sand has negligible c and high Φ relative to clay. An average Φ of clay and sand are 20° and 35° respectively. An average c of clay is 65 kPa (Cokca, 2004). The τ of clay and sand with increasing σ is illustrated below:

  Although clay has cohesion and thus a higher shear strength than sand in shallow environments, because the Φ of clay is less than that of sand, its shear strength increases less with depth, and foundations of clay are more prone to deep-seated slope failure.

- **Peat**

  Peat deposits are the partially decomposed remains of plants. Early in the process of decomposition peat is fibrous. Peat deposits display very low shear strengths; with typical values for cohesion and angle of internal friction of 10 kPa and 10° respectively (Figure 1.10). When its shear strength is exceeded, peat creeps and spreads; so high settlements are typical (Waltham, 2001). Fibrous peat particles are highly perforated and have a hollow cellular structure, which makes them very permeable and compressible. Fibrous peat deposits also display high void ratios. Fibrous peat’s hollow cellular structure and high void ratio allows for a potentially high water content, which lowers the shear strength of the material. After significant decomposition, peat is referred to as amorphous peat. Amorphous peat typically display a lower void ratio, permeability, and compressibility, because the particles decrease significantly in size through decomposition (Mesri and Aljouni, 2007).
Apart from its moisture content and dry density, the shear strength of a peat deposit appears to be influenced, firstly, by its degree of humification and, secondly by its mineral content. As both these factors increase so does the shear strength. Conversely, the higher the moisture content of the peat the lower is its shear strength. Peat is especially prone to rotational failure of failure by spreading, particularly under the action of horizontal seepage forces. (Bell, 2000)

**1.4 The Formation of a Slope Failure Surface**

Soil tends to fail at an angle (\( \alpha \)) defined by its angle of internal friction (\( \Phi \)) (Equation 1.3).

\[
\alpha = 45 + \frac{\Phi}{2}
\]

(Equation 1.3) (Sowers, 1970)

Failure begins at some depth in the slope at the angle \( \alpha \) and progresses upslope and/or downslope until the failure surface is expressed at the surface. Cohesive soil, like clay, tends to fail deeper within the slope than non-cohesive soil (Terzaghi and Peck 1967), like sand and silt, because relative to non-cohesive soil it has a low \( \Phi \) and high cohesion, which makes it relatively strong at shallow depths and weak at deep depths. Because of this soil property, the failure surface beneath the “V-line” levee is likely deep-seated as opposed to a shallow feature.
CHAPTER 2. METHODS

2.1 Cone Penetration Test (CPT)

The Cone Penetration Test is used extensively in this study, so information regarding how the test is conducted and how the results of the test are interpreted are explained. Cone Penetration Test data, tip resistance ($q_c$) and sleeve friction ($f_s$), were collected along the crest and toes of the “V-line” levee at depths up to 27 meters in order to determine the Soil Behavior Type (SBT) and undrained shear strength ($S_u$) of the sediments composing and underlying the levee by FFEB Geotechnical Consultants and submitted to the United States Army Corps of Engineers, New Orleans District, during the months of February and March of 2007. Each CPT site is assigned a number, indicating its location along the levee, and letters, CL, FT, or PT abbreviating Center Line, Flood Toe, and Protected Toe respectively, which further describe the CPT site location (Fig. 2.1). How CPT data is collected and used to determine the SBT and $S_u$ can be found in the following sections.

The cone penetration test is a fast, continuous, and economical means of determining subsurface soil type. The test is conducted by hydraulically pushing a steel cone, called a cone penetrometer, into the soil at a constant rate of approximately two cm/sec (Fellenius and Eslami, 2000). As the penetrometer is pushed through the ground, tip resistance ($q_c$) and sleeve friction ($f_s$) are measured. Tip resistance is the force necessary to push the cone through the ground, and sleeve friction is the frictional force measured along the friction sleeve. As established by the International Society for Soil Mechanics and Foundation Engineering (ISSMFE) a reference cone consists of a cone with an apex angle of 60° and a basal area of 10 cm², and a friction sleeve with a surface area of 150 cm² (Lunne et al., 1997). Tip resistance and sleeve friction can be used along with an interpretation chart, e.g. Robertson et al. (1986), to determine the soil behavior type of a soil, which is a prediction of soil type/grain size in the subsurface using CPT data (Mayne, 2007).

The piezocone, introduced in 1974, collects pore water pressure measurements along with tip resistance and sleeve friction (Fig. 2.2) (Lunne et al., 1997) to develop a more accurate picture of SBT. The pore pressure can be measured at three locations which are, the cone tip ($u_1$), behind the cone tip ($u_2$), and behind the friction sleeve ($u_3$). Higher values of $q_c$ reflect more coarse-grained material and higher values of $f_s$ and $u$ both reflect less coarse-grained material. Measured values of $q_c$ and $f_s$ are used together to determine the SBT of soil, with $u$ increasing the accuracy of SBT predictions.
Fig. 2.1 A satellite image of the “V-line” levee (Google Earth) with the location of each CPT site labeled.
Fig. 2.2 Diagram of a Cone Penetrometer (Lunne et al., 1997)

Fig. 2.3 Diagram of a typical Cone Penetration test rig and penetrometer (Mayne, 2007)
The cone penetrometer is pushed through the ground by a thrust machine commonly referred to as a rig. To resist the upward force generated as a result of the downward thrust of the cone penetrometer, the thrusting mechanism of the rig is, generally, mounted inside of a heavy truck and thrust through the center and bottom of the truck (Fig. 2.3).

The Cone Penetration Test (CPT) has a number of advantages over conventional boring and laboratory testing. The CPT provides a continuous or virtually continuous record of ground conditions, causes less ground disturbance, the ground disturbance between tests is consistent between one test and another, and is more economical than conventional boring and laboratory testing, costing approximately $20/ft (Abu-Farsakh and Titi, 2004).

2.1.1 Robertson et al. (1986) SBT Chart

The Robertson et al (1986) SBT chart (Fig. 2.4) is chosen for this study because SBT can be determined without the inclusion of pore-pressure data and it is commonly used by civil engineers. In this study an adaption of the Robertson et al. 1986 SBT chart (Fig. 3.2) is used to determine the type of soil composing and beneath the “V-line” levee. This SBT chart was chosen amongst all of the SBT charts currently used by engineers because the ranges of \( q_c \) and \( R_f \) defining each SBT are well defined and pore pressure measurements are unnecessary.

2.2 Laboratory Shear Strength Testing

Shear strength of the material beneath crest of the “V-line” levee was directly determined by collecting samples (Fig. 2.5) and using the unconfined compression and triaxial shear tests in the laboratory on those samples (Fig. 2.6). The SSW side of the laboratory shear strength profile was generated using 14 shear strength data point, the NNE side of the laboratory shear strength profile was generated using 12 shear strength data points, and data between each data point and between the two sampling sites was interpolated using Matlab (MathWorks, 2010), which is a high-level commercial numerical calculation programming language. Although the shear strength data set is generated using two different types of compression test, unconfined compression test and triaxial shear test, it appears to be consistent and the differences between tests do not appear to be significant.

2.2.1 Triaxial Shear Test

The triaxial shear test is used to determine the drained or undrained shear strength, in units of pressure (Tsf or kPa), of a cylindrical soil sample, whose height should be approximately twice its
Fig. 2.4 Robertson et al (1986) SBT chart which is used to determine the SBT of the soil beneath the levee. The number in the center of each zone bounded by black lines corresponds to a SBT defined in the legend to the right of the chart.
Fig. 2.5 A satellite image of the “V-line” levee (Google Earth, 2010) with the location of each laboratory shear strength sample collection site labeled and indicated by yellow circles.
Fig. 2.6 Shear strength from laboratory sampling sites WWHC-68UFT (left) and WWHC-70UFT (right) with the data in between the two sites and between data points interpolated.

Fig 2.7 (Left) Principal stresses on the soil sample in the triaxial shear apparatus, (Right) Diagram of the triaxial shear apparatus with the key features labeled.
diameter, by means of vertical compression until shear failure (Chen, 2000). The sample is enclosed in a rubber membrane, to control drainage, put into a cell, which is filled with water which is used to apply a confining normal stress \( \sigma_3 \) to the specimen, as a ram applies a vertical stress \( \sigma_1 \) (Figure 2.7) (Smoltczyk, 2002). When the triaxial shear is used to determine the drained shear strength of a soil sample a duct passing through the bases of the apparatus allows drainage. Alternatively, when the triaxial shear test is used to determine the undrained shear strength of a soil sample, drainage is prevented and pore pressure is measured.

The triaxial shear test has both advantages and disadvantages. Use of the triaxial shear test as a means of determining the shear strength of a soil sample is advantageous because confining pressure can be controlled, which allows angle of internal friction and cohesion to be determined, the shear strength of any soil type can be tested, i.e. sand, silt, clay, peat, etc., and drainage is controlled, allowing for the measurement of both drained and undrained shear strength. The disadvantage of the use of the TST is its cost, approximately $500 per test (Tashjian Towers Corporation, 2010). There are other means of determining the shear strength of a soil sample, like the unconfined compression, direct shear, and vane shear tests, which are less reliable but will still return usable information at a lower cost.

### 2.2.2 Unconfined Compression Test

The unconfined compression test is used to determine the undrained shear strength of a cohesive, laterally unconfined, undisturbed, cylindrical clay sample, whose height is between 2 and 2.5 times its diameter, by means of vertical compression until shear failure, at a loading rate between 0.5 and 2% of the initial sample height per minute. The vertical compressive pressure that causes shear failure of the sample is called the unconfined compressive strength \( q_u \). In the case of a highly plastic sample that does not shear, but instead deforms and bulges, the unconfined compressive strength is defined as the vertical compressive pressure at 15% axial strain. The undrained shear strength \( S_u \) of a sample tested using the unconfined compression test is defined as one-half the \( q_u \). (Day, 1999)

Results from the unconfined compression test can be unreliable for at least two reasons. During the unconfined compression test, samples are tested only at room pressure conditions. Because there is no additional confining pressure, the effective normal stress \( \sigma' \) is zero and the effect of the effective internal angle of friction \( \Phi' \) is negated, according to Mohr-Coulomb theory for predicting the shear strength of saturated soils with the additional effective normal stress variable proposed by Terzaghi (1936) (eq. 2.1). The negation of the internal angle of friction makes the results less reflective of the
strength of the material while in the subsurface because the measured shear strength is cohesion. Because clay has higher cohesion than sand, it will display higher apparent shear strength.

\[ \tau = c' + \sigma' \tan \Phi' \]  

(Equation 2.1)

Where,

\[ \tau = \text{soil shear strength (Ts'f)} \]
\[ c' = \text{effective cohesion (Ts'f)} \]
\[ \sigma' = \text{effective normal stress (Ts'f)} \]
\[ \Phi' = \text{internal angle of friction (°)} \]

Another source of unreliability is the lack of control of internal soil conditions such as, degree of saturation and pore water pressure. As a soil’s degree of saturation increases, its pore water pressure tends to increase; thus causing a decrease in apparent shear strength. Although the Unconfined Compression Test has shortcoming it is less costly, at approximately $100 per test (Tashjian Towers Corporation, 2010), than the triaxial shear test.

### 2.3 Seismic Shear Wave Survey

Seismic shear wave data are collected along the “V-line” levee to determine subsurface conditions. Horizontally polarized seismic shear wave data are collected by Lorenzo (2010) between September 2007 and February 2008 along protected side of the “V-line” levee, within 30 m of its crest, in pseudo-walkaway tests. The data are collected using 72 14 Hz horizontal geophones, with a 1m spacing between each geophone and a 1m spacing between the first geophone and the seismic source. Shear waves are generated by hitting the side of an I-beam which is dug into the ground approximately 2cm to allow proper coupling with the earth. Three profiles are collected, one parallel to the damaged levee crest, and, for reference, two near undamaged sections of artificial levee (Fig. 2.8)

Seismic shear wave data can be used to determine the maximum shear modulus \( G_{\text{max}} \) of a medium. Shear Modulus is an elastic modulus that measures the rigidity of a material. It is the ratio between the shear stress imparted on a material and the shear strain caused by the shear stress; therefore for high values of shear modulus it takes a larger stress to cause a given strain. Shear Modulus estimated by collecting shear-type seismic data through the following relationship:
where $G \ (Pa)$ is shear modulus, $V_s \ (m / s)$ is shear wave velocity and $\rho \ (kg / m^3)$ is density.

A material’s resistance to shearing can be determined by stressing the material until failure, and is termed the shear strength of the material, or by cyclically shearing a material and measuring the speed at which the resulting shear waves travel through the medium, and is called the shear modulus of the material. The shear strength and shear modulus of a material are difficult to compare because during seismic experiments the material is deformed at small strains elastically, while when determining the shear strength of a material through laboratory shear strength tests or from CPT results the material is deformed at larger strains plastically.

Figure 2.8 A satellite image of the “V-line” levee (Google Earth, 2010) with the central location of each seismic shear wave acquisition line indicated by a yellow circle.
CHAPTER 3. ANALYSIS OF DATA

3.1 CPT Data Analysis

3.1.1 Stage one (Digitizing Data)

CPT logs from along the crest, flood toe, and protected toe of the “V-line” levee are available only in paper form, so it is necessary to digitize the CPT logs (See Appendix A) to determine where each data point lies on the Robertson et al (1986) SBT chart. These CPT logs provided both tip resistance ($q_c$) and sleeve friction ($s_f$) data. The paper CPT logs were digitized using Didger (Golden Software, 2011), a program that can be used to convert and image of a data plot into data points. In order to accurately reflect the variability of $q_c$ and $s_f$ in each CPT log between 200 and 300 points were selected for digitization. In order to have the same number of points on each plot, the data were interpolated to one point every 0.25 ft using Matlab (MathWorks, 2010), a programming language used for numerical calculation. The interpolated data was then converted from units of feet and tons per square foot (Tsf) to meters and bars, respectively, so they could be used in the Robertson et al. 1986 chart (Fig. 2.4).

3.1.2 Stage two (Well Locations)

In order to generate a cross-section that accurately reflected the distance between each well the location of each well is needed. The location of the CPT wells is determined using given latitudes and longitudes of sampling sites (WWHC62CL-WWHC72CL from Fig. 3.1) for calibration in Didger, and a diagram (fig. 2.2) showing a map view of each CPT and sampling site.

3.1.3 Stage three (Interpretation)

The Robertson et al. 1986 interpretation chart is used to determine the soil behavior type (SBT) of materials beneath the “v-line” levee at each CPT site. The chart uses $q_c$ and Friction ratio ($R_f$), which equals $s_f / q_c$, to determine soil behavior type.

3.1.4 Adjusted Robertson et al. (1986) Interpretation Chart

The Robertson et al. 1986 interpretation chart (Fig. 2.4) is adjusted in 3 ways to make it more appropriate for my study. The original Robertson et al. 1986 interpretation chart included 8 grain
Fig. 3.1 Map view of CPT and sampling sites

size divisions from clay to sand. Because the physical behavior many of the 8 grain size divisions is similar, the range from clay to sand was divided into 3 sections, clay, silt, and sand. Because of many of the calculated values of friction ratio ($R_f$) exceeded the maximum value for $R_f$ in the original Robertson et al. 1986 chart, the clay and organic material divisions were extrapolated to a higher value to include all calculated values of $R_f$. The organic content of clay beneath the levee is of great significance because increased organic content reduces the shear strength of soil by increasing moisture retention and reducing density. To differentiate between clay and “organic clay”, a new division near the border between clay
and peat on the Robertson et al. 1986 interpretation chart called organic clay was made. The resulting chart can be found below (Fig. 3.2).

![Modified Robertson et al. (1986) SBT chart](image)

**Fig. 3.2** Modified from the Robertson et al. (1986) SBT chart, decreasing the number of zones, extrapolated the clay and peat zones to include higher values of $R_f$, and dividing the clay zone into clay and organic clay zones.

### 3.2 Shear Strength from Laboratory, CPT, and Seismic Shear Wave Velocity

The cone penetration test, triaxial shear test, and shear wave seismic surveys can all be used to determine a material’s resistance to shearing. Each of these test generate different forms of data and to be compared, must all be converted to shear strength. The triaxial shear test directly measures shear strength so no conversion equation is necessary. The cone penetration test measures tip resistance ($q_c$), sleeve friction ($f_s$), and sometime pore pressure ($u$) and therefore requires equations to convert the data generated by this test to shear strength. Shear wave seismic surveys measure shear wave velocity in the medium ($V_s$), which can easily be converted to shear modulus using the equation, $G = \rho * V_s$ but converting shear wave velocity into shear strength is more complex.

#### 3.2.1 Shear Strength from CPT Data

The tip resistance parameter from the cone penetration test can be used to determine the shear strength of a material (Fig. 3.3) using the equation:
\[ S_u = \frac{q_c - \sigma_{ov}}{N_k} \]  
(Equation 3.1)

Figure 3.3 Illustration of shear strength derived from CPT data collected along the “V-line” levee

Where \( N_k \) is cone factor, \( S_u \) is undrained shear strength, and \( \sigma \) is overburden pressure. The cone factor is a factor that varies between sites and material being tested, but a value of 14±5 is applicable to clays with any PI value (Duncan and Wright, 2005). The CPT displays a higher apparent shear strength and a larger range of values because the strength measured is in-situ and therefore under confining pressure which causes a higher apparent shear strength.

Determining \( S_u \) using \( q_c \) has both advantages and disadvantages. Determining \( S_u \) from \( q_c \) can be advantageous because the cone penetration test measures data continuously or nearly continuously, shear strength can be determined at a high vertical resolution. Because \( q_c \) is measured in-situ, the
material being tested is subject to $\sigma_{vo}$, the calculated $S_u$ may be more reflective of the material actual shear strength when compared to unconfined compression test results, where the material is under atmospheric pressure. The use of $q_c$ to determine $S_u$ can be disadvantageous because the equation requires the constant $N_k$ which cannot be calculated and therefore makes the calculated $S_u$ less precise. When using the above equation, in order to calculate $S_u$, it is also necessary that the $\sigma_{vo}$ to which the sample is being subjected be known. Although this parameter can be determined, it is not typically measured during cone penetration tests.

3.2.2 Laboratory/CPT Derived Shear Strength Comparison

Shear strength derived from Laboratory Shear Strength Testing, Unconfined Compression Test and Triaxial Shear Strength Test, are smaller than those derived from the CPT because the testing is done under no confining stress and because of this, has a lower apparent shear strength (Fig. 3.4). The average shear strength derived from CPT site WWHCCPT-65CL is higher than the surrounding area because it has the highest content of sand and thus the highest average shear strength.

3.2.3 Shear Strength from Seismic Shear Wave Velocity

Seismic shear wave data is collected at strains small enough to be in the elastic regime of deformation. Because seismic deformation is in the elastic regime it is difficult to compare seismic shear wave data to shear strength data where material is deformed plastically. Seismic shear wave data collected along the “V-line” levee is converted to tip resistance ($q_c$) using equation 3.5, then to shear strength ($S_u$) using equation 3.1; the resulting $S_u$ value was negative at a range of density and $N_k$ values; therefore, $S_u$ could not be determined using seismic shear wave data.

3.3 Seismic Shear Wave Velocity from CPT Data

Several empirical relationships have been developed for the conversion of CPT data into shear wave velocity. Using these empirical relationships, CPT and shear wave seismic survey data can be directly compared and used together. Much vertical detail can be derived from CPT data, but data is only acquired where drilling is done, which limits horizontal resolution to the distance between drilling sites. Shear wave seismic surveys are less vertically detailed than the CPT but, because at the speed at which a survey can be conducted and the lack of damage to ground, are nearly continuous horizontally. The following empirical relationships can be used to correlate common points between CPT and
Fig. 3.4 Chart displaying the average shear strength (blue asterisk) and range of shear strength values (red line) of CPT (WWHCCPT-63CL through WWHCCPT-66CL) and Laboratory Shear Strength test (WWHC-68UFT and WWHC-70UFT) sites, which are near one another.
shear wave seismic survey data sets, which make correlation between CPT sites easier and add detail to the seismic data set.

\[ V_s = 3.18q_c^{0.549}f_s^{0.025} \quad (n = 229 \quad r^2 = 0.778) \quad \text{Clay} \quad (\text{Equation 3.2}) \]

\[ V_s = 12.02q_c^{0.319}f_s^{-0.0466} \quad (n = 92 \quad r^2 = 0.574) \quad \text{Sand} \quad (\text{Equation 3.3}) \]

\[ V_s = (10.1\log q_c - 11.4)^{1.67}\left(\frac{f_s}{q_c}\right)^{0.3} \quad (n = 323 \quad r^2 = 0.695) \quad \text{All} \quad (\text{Equation 3.4}) \]

(Hegazy and Mayne, 1995)

\[ V_s = 1.75q_c^{0.627} \quad (n = 481 \quad r^2 = 0.736) \quad \text{Clay} \quad (\text{Equation 3.5}) \]

(Mayne and Rix, 1995)

Figure 3.5 Seismic shear wave velocities of material beneath the “V-line” levee as predicted by the northernmost shear wave experimentation site, indicated by red circles, and CPT data converted to shear wave velocities using equations 3.3(black), 3.5(magenta), and a combination of the two equations (green) based soil type as predicted by CPT data and the Robertson et al. 1986 Soil Behavior Type Chart
The empirical relationships between $V_s$, $q_c$, and $f_s$, equations 3.2-3.5, are used to compare the northernmost seismic shear wave data collected and processed by Lorenzo (2010) and the CPT site WWHCCPT-65CL (Fig. 3.5), which is the nearest CPT site to the seismic shear wave data collection site. The equations that most closely predict the seismic shear wave velocity at depth based upon CPT data are equations 3.5 for clay and 3.3 for sand. Equation 3.5 overestimates shear wave velocity for sand and equation 3.3 underestimates shear wave velocity for clay. To account for this, the Robertson et al. (1986) Soil Behavior Type chart was used in conjunction with the CPT data to predict the soil type at depth and the more appropriate equation of the two. Equations 3.2 and 3.4 are also used to predict shear wave velocities, but are found to predict the measured shear wave velocities less reliably than equations 3.3 and 3.5.

Because the $V_s$ predicted using CPT data is determined using empirical relationships, it is not expected that it would reflect the data perfectly, but equations 3.3 and 3.5 do provide reasonable approximations of $V_s$. 
CHAPTER 4. RESULTS

4.1 Seismic Shear Wave Survey Results

Horizontally polarized shear wave data was collected along protected side of the “V-line” levee, within 30 m of its crest, in pseudo-walkaway tests (Lorenzo, 2010). Three profiles were collected, one parallel to the damaged levee crest, and, two near undamaged sections of artificial levee (Figure 2.8). Data is collected along the crack in order to find anomalies in the subsurface, near the crack to get information about the area just outside the area of the crack, and far from the tension crack to get information about a section of levee unaffected by the crack. A zone of low shear wave velocity was interpreted using the seismic shear wave data by Lorenzo (2010) between CPT sites 63CL and 64CL at a depth of approximately 7 meters (Fig. 4.1).

Figure 4.1 Seismic shear wave profile interpreted from seismic shear wave data collected in the area of the damaged levee and to the north and south of the damaged levee, with the zone of low shear wave velocity surrounded by a dashed line; see figure 2.8 for location (Lorenzo, 2010).
Figure 4.2 Cone Penetration Test Cross – Section generated using a modified Robertson et al. 1986 Interpretation Chart (Figure 3.2), with colors representing SBT. See figure 2.1 for CPT site locations.
4.2 Cone Penetration Test Results

The CPT was used to determine the Soil Behavior Type and shear strength of the materials composing and beneath the levee. Large amounts of peat and organic clay were found in the levee fill in the area of the tension crack on the levee (Fig. 4.2)

The tip resistance ($q_t$) from the Cone Penetration Test was used to determine shear strength of the materials composing and beneath the levee. The absence of an apparent weak zone in the CPT data is likely because the weak zone is approximately 100m while the spacing between CPT sites is 300m. The weak zone could be easily missed because of the short length of the feature for is smaller than the sampling interval. The tension crack is located between CPT sites 64CL and 63CL and cannot be clearly seen in the cone penetration test data.
CHAPTER 5. DISCUSSION

5.1 Resistance to Shearing Measured at Various Strain Levels

The measured resistance to shearing of a material will vary based upon the magnitude strain imposed. When a material is minimally elastically strained, i.e. seismically, maximum shear modulus, the material’s highest resistance to shearing, is used to represent shearing resistance (Figure 5.1). As strain increases within the elastic regime, the apparent shear modulus decreases and a range of secant shear modulus values are used to represent shearing resistance. When a material is strained to failure, shear strength, the material’s lowest resistance to shearing, is measured (Figure 5.1). Because collecting shear strength data involves coring and expensive laboratory testing, determining shear strength using seismic shear wave data would be cost-effective and quicker.

![Ideal elastic-plastic stress-strain behavior](image)

Fig. 5.1 Ideal elastic-plastic stress-strain behavior of a material with the proportional relationship between stress and strain before failure indicated in green and the relatively stable stress and increasing strain seen post-failure in red (Sowers, 1970).

Although a theoretical relationship between shear strength and shear wave velocity does not exist, empirical relationships do exist between plasticity index (PI), OverConsolidation Ratio (OCR), maximum shear modulus ($G_{max}$), shear modulus at larger strain (secant shear modulus, $G$), and shear strength ($S_u$). Vucetic and Dobry (1991) developed a chart, using empirical data, that illustrates the change in $G / G_{max}$ with increasing strain at a range of values for OCR and PI (Fig. 5.2). PI of a soil is its liquid limit (LL)
minus its plastic limit (PL); Soils with high PI tend to be less stable with high swelling potential. OCR of a soil is the ratio between its preconsolidation pressure, which is the highest overburden pressure with which it has been subjected to, and its present overburden pressure. Empirical relationships have also been developed by Weiler (1988), between $G_{\text{max}}$ and $S_u$ at a range of values for OCR and PI (Figure 5.3). In the relationship developed by Weiler (1988), as PI increases, the difference between $G$ or $S_u$ and $G_{\text{max}}$ decreases; as OCR increases, the difference between $G$ or $S_u$ and $G_{\text{max}}$ decreases. These relationships show that although $S_u$ cannot be predicted solely based on a value of $G_{\text{max}}$ or $V_s$, with the addition of OCR and PI values a prediction can be made, although PI and OCR data is not available for the “V-line” levee soil so making use of these relationships is beyond the scope of my research.

5.2 Data Lateral Resolution

CPT sites and cores taken for shear strength testing in the laboratory obtain data along the column where data was collected. All information between wells must be inferred from the available data. Because the area beneath the levee that exhibits low shear strength is between CPT and coring sites, and

![Chart illustrating the change in $G / G_{\text{max}}$ of normally and overconsolidated clay with increasing strain at a range of values for OCR and PI (Vucetic and Dobry 1991)](chart.png)
Fig. 5.3 Chart illustrating values of $G_{\text{max}} / S_u$ for a range of OCR and PI values with the data points used to generate the chart indicated by black circles (Weiler, 1988) the feature is smaller than the site spacing, it could not be resolved. A zone of weakness is seen in the shear wave data because data was collected in the area near the tension crack and therefore, near the zone of weakness.

5.3 CPT Data Interpretation Program Comparison

To test the validity of the CPT data SBT interpretation done using a Matlab program written for this study, the CPT data from one site was interpreted using both the Matlab program for this study and a program written by the Louisiana Transportation Research Center (Farsakh et al., 2003) (Fig. 5.4). The interpretations using software written by the LTRC and the Matlab program written for this study were found to be nearly identical. There are some small differences between the two SBT interpretations. Because the SBT zones for my Matlab program were defined digitizing an image from the Robertson et al. 1986 paper, what data points define the zones for the two programs may be different. I wrote my own program instead of using the one available from the LTRC so I would have the freedom to change how each SBT is defined. Although there are slight differences between the two interpretations, none of these differences are so significant that they would change my hypothesis and/or interpretation.
Fig. 5.4 SBT interpretation from the same CPT site (WWHCCPT-65CL) using software written by the Louisiana Transportation Research Center (LTRC) (Farsakh, 2003) (Left) and a Matlab program written for this study for comparison.

5.4 Faults Causing Slope Failure

Natural fault zones may also contribute to slope failure. As faults move they can remould soils and reduce their shear strength (Neuendorf, 2005). A reduction in shear strength leads to a less stable foundation and possibly slope failure.

It is unlikely that this is the cause of the instability of the “V-line” levee because there are no faults near the “V-line” levee and the nearest ones are perpendicular to the trend of the “V-line” levee and so it is unlikely that they would cause the apparent instability (Fig. 5.5). It was also found by Dunbar (2008) that the 17th St. Canal levee failure, which appears to be similar to the “V-line” levee instability, was not the result of natural faulting.
Figure 5.5 Map of the area near the “V-line” levee illustrating the locations of the known faults (Wallace, 1966 in Dunbar, 2008)

5.5 Weak Zone Location

The shape of the hypothesized failure surface developed before catastrophic slope failure will depend upon the materials composing and beneath the levee (Fig. 5.6). In the case of a homogeneous material beneath the levee, the failure surface is approximately cylindrical (Williams, 1982) (Fig. 5.7).
There are three main ways in which a slope can fail, which are, base failure, toe failure, and face failure (Chen, 2000) (Fig. 5.10).

When there is a layer of stronger material beneath the levee, the shape of the failure surface will form a cylinder that is tangential to the surface of the stronger material beneath it, in the case of the “V-line” levee the stronger material is a sand/silt mixture beneath clay. This type of failure is referred to as a base failure (Williams, 1982). The “V-line” levee is likely failing toward the western side of the levee, this is evidenced by the western side being slightly down-dropped in relation to the eastern side (figure 5.8).

The shear movements involved in soil slope failure occur across an area of appreciable thickness, so instead of a plane of failure forming, a zone of failure forms. Because after clay is disturbed its shear strength decreases to a lower residual value (USACE, 2003), clays in the zone of shearing are weaker than the surrounding material (figure 5.8).
Figure 5.7 Hypothesized slope failure surface beneath the crest of the “V-line” levee, given a homogeneous slope and slope foundation. The direction of soil motion is indicated by red arrows. See figure 5.6 for layer legend.
Figure 5.8 Close-up view of the crack along the crest of the “V-line” levee with the crack outlined in white on the right and visible on the left.
Fig. 5.9 Weak zone formation beneath the crest of the “V-line” levee as a result of the disturbance of clay, indicated by diagonal blue lines, of the “V-line” levee. The direction of soil motion is indicated by red arrows. See figure 5.6 for layer legend.

Figure 5.10 Most common types of slope failure (Chen, 2000)
A hypothesized failure surface is generated using the crack at the crest of the “V-line” levee, the zone of low shear wave velocity beneath the berm on the protected side of the levee, the assumption that the instability of the “V-line” levee will lead to a base failure, and that the failure surface will take the
same shape as that of a base failure (Fig. 5.10). The profile of the “V-line” was generated using LIDAR data (Louisiana State University, 2011), and a computer program called Global Mapper (Global Mapper Software LLC, 2009).

5.6 Location of Instability Along the “V-line” Levee

The unstable section of the “V-line” is likely so due to the higher content of organic material in it when compared to the surrounding sections of the “V-line” levee (Fig. 5.12).

Figure 5.12 The number of instances organic material (stippled), organic clay (dashed), and the addition of the two sediment types (solid) beneath the “V-line” levee.

Although the combined organic material and organic clay is highest at CPT site 66, this high organic content quickly decreases to the north and south of this site. The highest laterally extensive content of organic material appears in the area of CPT site 63. This high content of organic material may lead to low shear strength and easier propagation of a failure surface and increased water retention which, during heavy raining events, would increase the load on the material underlying the levee and increase the likelihood of failure. The section of the 17th St. Canal floodwall that failed during hurricane Katrina had a similar distribution of organic material in the subsurface, with high contents of organic material beneath the levee and the highest beneath the breached floodwall (Fig. 1.14).

5.7 Slope Stability Calculation

Factor of Safety (FoS) of the “V-line” levee is calculated in the zone of instability to
quantitatively evaluate field stability conditions. FoS is the ratio between the forces resisting movement and those driving movement within a slope. A FoS below 1 implies unstable conditions and those near 1 imply potentially unstable conditions. Building codes typically define a stable slope as one with a minimum FoS of 1.5 under static loading conditions (Neuendorf et al., 2005). Xslope (Balaam, 2001) is free software that uses a common method, Bishop’s (1955) simplified method for circular failure surfaces, to calculate the stability of an earthen slope by predicting the FoS of the most likely failure surface. FoS is calculated on both the western and eastern sides of the “V-line” levee using shear strength and density data from a nearby boring (WWHC-68UFT); because the western side is steeper than the eastern side, a lower FoS, of 1.25, is predicted (Fig. 5.13). The fact that shear strength values from a nearby boring are used and the heterogeneity of the soil beneath the “V-line” levee are source of possible error. The effects of natural subsidence causing a steepened slope on the western side of the levee and the movement of soil beneath the levee causing strain weakening are not accounted for; and are additional sources of error. Without more information the FoS cannot be determined with certainty, and caution should be used.

Figure 5.13 Factor of Safety of the “V-line” levee in the area of instability, calculated using Xslope.

5.8 Further Testing

Because I propose that the instability of the “V-line” levee is partially caused by the an increase in water content of the materials overlying the zone of weakness, inferred from shear wave data (Lorenzo, 2010), data regarding the water content of the materials composing and beneath the levee would be beneficial. Some methods of determining water content include the neutron probe and electrical resistivity sensors.

As the tip resistance of materials is measured during the Cone Penetration Test, porewater pressure fluctuations cause changes in tip resistance. Future cone penetration testing should include porewater pressure measurements in order to make corrections to cone tip resistance (Fig 5.15).

Because limited seismic data was collected, the lateral extent of the zone of low shear strength is difficult to determine. If more detailed seismic data is collected, the lateral extent of the zone of low shear strength can be better constrained.
Figure 5.14 Correction to $q_c$ to obtain $q_t$ which takes the measured pore-water pressure ($u_2$) into account. $a_n$ is the correction factor determined for the cone penetrometer used in this example. (Mayne, 2007)
CHAPTER 6. CONCLUSION

There are several causes of the instability of the “V-line” levee. These causes are, the increase in the steepness of the slope of the levee due to subsidence, weak material composing and under the levee, and water being retained within the levee which increased the stress on the underlying material. A high content of clay and peat in the levee fill and the natural material beneath the levee is inferred from CPT data and maps of the surface deltaic environments by Saucier (1994). These materials are a likely cause of the instability of the “V-line” levee because of the low shear strength of clay and peat, and the high water retention capabilities of peat, which increase the stress on the material underlying the levee. Low shear wave velocities are measured in the subsurface near the crack in the crest of the “V-line” level, which are evidence that the damage to the levee is more than superficial. These low shear wave velocities are likely caused by a small amount of movement of the clay in the subsurface; because clay is sensitive, when it is remolded its shear strength is reduced. This reduction in shear strength can contribute to further levee instability. Although individual factors may not be the cause of the levee instability, together they increase the likelihood of failure.
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APPENDIX A (CPT DATA)

Center Line Wells

WWHCCPT-60CL

Sleeve Friction (kPa) | Tip Resistance (kPa) | Soil Type-Robertson '86

Depth (meters)
Protected Side Wells

WWHCCPT-60PT
Flood Side Wells

WWHCCPT-61FT

Depth (meters)

Tip Resistance (kPa)

Grain Size

Tip Resistance (kPa)
APPENDIX B (CPT SOIL BEHAVIOR TYPE MATLAB PROGRAM)

clear

%%% Load Zones
load zone_1_adjusted.dat
load zone_2_adjusted.dat
load zone_3_adjusted.dat
load zone_4_adjusted.dat
load zone_5_adjusted.dat
load zone_6_adjusted.dat
load zone_7_adjusted.dat
load zone_8_adjusted.dat
load zone_9_adjusted.dat
load organic_clay.dat

%%% Load Sleeve Friction Data
load WWHCCPT_60CL_friction.dat
load WWHCCPT_61CL_friction.dat
load WWHCCPT_62CL_friction.dat
load WWHCCPT_63CL_friction.dat
load WWHCCPT_64CL_friction.dat
load WWHCCPT_65CL_friction.dat
load WWHCCPT_66CL_friction.dat
load WWHCCPT_67CL_friction.dat
load WWHCCPT_68CL_friction.dat
load WWHCCPT_69CL_friction.dat
load WWHCCPT_70CL_friction.dat
load WWHCCPT_71CL_friction.dat

%%% Load Tip Resistance Data
load WWHCCPT_60CL_resistance.dat
load WWHCCPT_61CL_resistance.dat
load WWHCCPT_62CL_resistance.dat
load WWHCCPT_63CL_resistance.dat
load WWHCCPT_64CL_resistance.dat
load WWHCCPT_65CL_resistance.dat
load WWHCCPT_66CL_resistance.dat
load WWHCCPT_67CL_resistance.dat
load WWHCCPT_68CL_resistance.dat
load WWHCCPT_69CL_resistance.dat
load WWHCCPT_70CL_resistance.dat
load WWHCCPT_71CL_resistance.dat
%% Define Sleeve Friction Variables

fx0 = WWHCCPT_60CL_friction(:,1);
fy0 = WWHCCPT_60CL_friction(:,2);
fx1 = WWHCCPT_61CL_friction(:,1);
fy1 = WWHCCPT_61CL_friction(:,2);
fx2 = WWHCCPT_62CL_friction(:,1);
fy2 = WWHCCPT_62CL_friction(:,2);
fx3 = WWHCCPT_63CL_friction(:,1);
fy3 = WWHCCPT_63CL_friction(:,2);
fx4 = WWHCCPT_64CL_friction(:,1);
fy4 = WWHCCPT_64CL_friction(:,2);
fx5 = WWHCCPT_65CL_friction(:,1);
fy5 = WWHCCPT_65CL_friction(:,2);
fx6 = WWHCCPT_66CL_friction(:,1);
fy6 = WWHCCPT_66CL_friction(:,2);
fx7 = WWHCCPT_67CL_friction(:,1);
fy7 = WWHCCPT_67CL_friction(:,2);
fx8 = WWHCCPT_68CL_friction(:,1);
fy8 = WWHCCPT_68CL_friction(:,2);
fx9 = WWHCCPT_69CL_friction(:,1);
fy9 = WWHCCPT_69CL_friction(:,2);
fx10 = WWHCCPT_70CL_friction(:,1);
fy10 = WWHCCPT_70CL_friction(:,2);
fx11 = WWHCCPT_71CL_friction(:,1);
fy11 = WWHCCPT_71CL_friction(:,2);

%% Define Tip Resistance Variables
qx0 = WWHCCPT_60CL_resistance(:,1);
qy0 = WWHCCPT_60CL_resistance(:,2);
qx1 = WWHCCPT_61CL_resistance(:,1);
qy1 = WWHCCPT_61CL_resistance(:,2);
qx2 = WWHCCPT_62CL_resistance(:,1);
qy2 = WWHCCPT_62CL_resistance(:,2);
qx3 = WWHCCPT_63CL_resistance(:,1);
qy3 = WWHCCPT_63CL_resistance(:,2);
qx4 = WWHCCPT_64CL_resistance(:,1);
qy4 = WWHCCPT_64CL_resistance(:,2);
qx5 = WWHCCPT_65CL_resistance(:,1);
qy5 = WWHCCPT_65CL_resistance(:,2);
qx6 = WWHCCPT_66CL_resistance(:,1);
qy6 = WWHCCPT_66CL_resistance(:,2);
qx7 = WWHCCPT_67CL_resistance(:,1);
qy7 = WWHCCPT_67CL_resistance(:,2);
qx8 = WWHCCPT_68CL_resistance(:,1);
qy8 = WWHCCPT_68CL_resistance(:,2);
qx9 = WWHCCPT_69CL_resistance(:,1);
qy9 = WWHCCPT_69CL_resistance(:,2);
qx10 = WWHCCPT_70CL_resistance(:,1);
qy10 = WWHCCPT_70CL_resistance(:,2);
qx11 = WWHCCPT_71CL_resistance(:,1);
qy11 = WWHCCPT_71CL_resistance(:,2);

%% Define Depth

d = 0:.25:75;

%% Define zone 1 (Robertson et al., 1986 SBT Chart)

zx1 = zone_1_adjusted(:,1);
zy1 = zone_1_adjusted(:,2);

%% Define zone 2 (Robertson et al., 1986 SBT Chart)

zx2 = zone_2_adjusted(:,1);
zy2 = zone_2_adjusted(:,2);

%% Define zone 3 (Robertson et al., 1986 SBT Chart)

zx3 = zone_3_adjusted(:,1);
zy3 = zone_3_adjusted(:,2);
%%% Define zone 4 (Robertson et al., 1986 SBT Chart)

\[zx4 = \text{zone}_4\_\text{adjusted}(:,1);\]
\[zy4 = \text{zone}_4\_\text{adjusted}(:,2);\]

%%% Define zone 5 (Robertson et al., 1986 SBT Chart)

\[zx5 = \text{zone}_5\_\text{adjusted}(:,1);\]
\[zy5 = \text{zone}_5\_\text{adjusted}(:,2);\]

%%% Define zone 6 (Robertson et al., 1986 SBT Chart)

\[zx6 = \text{zone}_6\_\text{adjusted}(:,1);\]
\[zy6 = \text{zone}_6\_\text{adjusted}(:,2);\]

%%% Define zone 7 (Robertson et al., 1986 SBT Chart)

\[zx7 = \text{zone}_7\_\text{adjusted}(:,1);\]
\[zy7 = \text{zone}_7\_\text{adjusted}(:,2);\]

%%% Define zone 8 (Robertson et al., 1986 SBT Chart)

\[zx8 = \text{zone}_8\_\text{adjusted}(:,1);\]
\[zy8 = \text{zone}_8\_\text{adjusted}(:,2);\]

%%% Define zone 9 (Robertson et al., 1986 SBT Chart)
zx9 = zone_9_adjusted(:,1);
zy9 = zone_9_adjusted(:,2);

%%% Define organic clay zone (Robertson et al., 1986 SBT Chart)
zx10 = organic_clay(:,1);
zy10 = organic_clay(:,2);

%%% Interpolate tip resistance and sleeve friction data to a new interval
d = 0:.25:75;

fs(:,1) = interp1(fy0, fx0, d);
qc(:,1) = interp1(qy0, qx0, d);

fs(:,2) = interp1(fy1, fx1, d);
qc(:,2) = interp1(qy1, qx1, d);

fs(:,3) = interp1(fy2, fx2, d);
qc(:,3) = interp1(qy2, qx2, d);

fs(:,4) = interp1(fy3, fx3, d);
qc(:,4) = interp1(qy3, qx3, d);
fs(:,5) = interp1(fy4, fx4, d);
qc(:,5) = interp1(qy4, qx4, d);

fs(:,6) = interp1(fy5, fx5, d);
qc(:,6) = interp1(qy5, qx5, d);

fs(:,7) = interp1(fy6, fx6, d);
qc(:,7) = interp1(qy6, qx6, d);

fs(:,8) = interp1(fy7, fx7, d);
qc(:,8) = interp1(qy7, qx7, d);

fs(:,9) = interp1(fy8, fx8, d);
qc(:,9) = interp1(qy8, qx8, d);

fs(:,10) = interp1(fy9, fx9, d);
qc(:,10) = interp1(qy9, qx9, d);

fs(:,11) = interp1(fy10, fx10, d);
qc(:,11) = interp1(qy10, qx10, d);

fs(:,12) = interp1(fy11, fx11, d);
qc(:,12) = interp1(qy11, qx11, d);

%%% Convert Tsf to Bars and Feet to Meters

for i=1:12

qc(:,i) = 0.95761 * qc(:,i);
fs(:,i) = 0.95761 * fs(:,i);

end

d  = 0.3048 * d;

%%% Calculate Friction Ratio (Rf)

for i=1:12

rf(:,i) = (fs(:,i)./qc(:,i))*100;

end

%%% Determine Data Point Location on Robertson et al. 1986 SBT Chart

for i=1:12

in1(:,i) = inpolygon(rf(:,i),qc(:,i),zx1, zy1);

end
in2(:,i) = inpolygon(rf(:,i),qc(:,i),zx2, zy2);
in3(:,i) = inpolygon(rf(:,i),qc(:,i),zx3, zy3);
in4(:,i) = inpolygon(rf(:,i),qc(:,i),zx4, zy4);
in5(:,i) = inpolygon(rf(:,i),qc(:,i),zx5, zy5);
in6(:,i) = inpolygon(rf(:,i),qc(:,i),zx6, zy6);
in7(:,i) = inpolygon(rf(:,i),qc(:,i),zx7, zy7);
in8(:,i) = inpolygon(rf(:,i),qc(:,i),zx8, zy8);
in9(:,i) = inpolygon(rf(:,i),qc(:,i),zx9, zy9);
in10(:,i)= inpolygon(rf(:,i),qc(:,i),zx10,zy10);
end

%% Define a variable (Interp) that describes the polygon in which each data point exists
for i=1:12
    for j=1:301
        if (in1(j,i)==1)
            interp(j,i) = 1;
        else
            interp(j,i) = 0;
        end
    end
end

for j=1:301
if (in2(j,i)==1)
    interp(j,i) = 2;
end
end

for j=1:301
    if (in3(j,i)==1)
        interp(j,i) = 3;
    end
end

for j=1:301
    if (in4(j,i)==1)
        interp(j,i) = 4;
    end
end

for j=1:301
    if (in5(j,i)==1)
        interp(j,i) = 5;
    end
end

end
for j=1:301
    if (in6(j,i)==1)
        interp(j,i) = 6;
    end
end

for j=1:301
    if (in7(j,i)==1)
        interp(j,i) = 7;
    end
end

for j=1:301
    if (in8(j,i)==1)
        interp(j,i) = 8;
    end
end

for j=1:301
    if (in9(j,i)==1)
        interp(j,i) = 9;
    end
end
end

end

for j=1:301

    if (in10(j,i)==1)

        interp(j,i) = 10;

    end

end

end

hold off

%%% Plot Data

stairs(interp(:,1),d)

set(gca,'FontSize',8)

axis([0 160 0 25])

axis ij

ylabel('Depth (m)')

title('60CL  61CL  62CL  63CL  64CL  65CL  66CL  67CL  68CL  69CL  70CL  71CL')

hold on

91
stairs(interp(:,2) + 10.26588692,d)

stairs(interp(:,3) + 24.93757648,d)

stairs(interp(:,4) + 39.44296619,d)

stairs(interp(:,5) + 53.92057687,d)

stairs(interp(:,6) + 68.52281887,d)

stairs(interp(:,7) + 82.91725883,d)

stairs(interp(:,8) + 97.32554672,d)

stairs(interp(:,9) + 112.0527943,d)

stairs(interp(:,10) + 124.9880456,d)

stairs(interp(:,11) + 136.9880456,d)

stairs(interp(:,12) + 148.9880456,d)
VITA

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