CHAPTER TEN: ENGINEERING OVERVIEW; EARTHEN LEVEES AND FLOODWALLS

10.1. Overview

The vast majority of flood protection for the greater New Orleans area is strongly dependent upon the presence and ability of earthen levees to separate large water bodies, such as Lake Pontchartrain, Lake Borgne, the Mississippi River, and the Gulf of Mexico, and appurtenant channels and canals, from inundating developed land areas and causing flooding of homes and businesses. Earthen levee flood protection systems not having redundancy can be viewed as series systems, where failure at one location, or failure of one component, can result in catastrophic failure of the entire flood protection system and result in tragic loss of life, damage to fundamental infrastructure (basic services such as water, sewage, and electricity), and substantial devastation and economic impact to the immediate and surrounding regions. These systems can be in place for a short duration (a few years) or for a very long duration (hundreds of years). In order to ensure the desired level of flood protection system performance, identification and mitigation of "weak links" in the system is crucial in order to maintain long-term system integrity.

The earthen levees are supplemented and extended at many locations by means of more "structural" components comprised of concrete and steel. Steel sheetpile curtains are routinely used either to extend a "cut off" barrier to retard underseepage flow beneath levees, or to provide support for reinforced concrete floodwalls at the crests of earthen levees. In some cases, the sheetpile curtains are extended vertically above the earthen crests without concrete to simply extend the crest elevation as an interim measure until a more permanent crest raising can be implemented. The concrete floodwalls are used to achieve increased crest height without the extra weight of additional earthen levee fill, and/or without the need to widen the earthen levee embankment section to accommodate additional earthen levee fill in situations where the available "footprint" is limited. Concrete walls are also employed to provide frames for gates (usually steel gates) that can be opened to allow traffic to pass through (e.g.: automobiles, trains, ships, etc.) and then closed when storms arrive.

Few studies have systematically analyzed actual long-term performance of earthen levees and/or composite levee-floodwall systems to confirm effective design parameters, assumed loading conditions, and actual performance after major flooding events. Additionally, evaluations of component transitions (i.e. earthen levee to concrete structure transitions), erodeability overtopping, wave-scour, and effective inspection programs have not been well documented and are critical components for high reliability flood protection systems.

Chapter 10 builds upon the technical lessons from the previous chapters, and establishes additional findings as well as background to facilitate the presentation of "lessons learned" and "recommendations" with regard to design and construction of these types of regional flood protection systems that will then be presented and discussed in Chapter 11.

The main goals of this Chapter are to: (1) provide a brief overview of some of the principal design procedures and standards employed in the development of pre-Katrina regional flood protection system, (2) identify several critical “weak link” features not adequately addressed in current design methods widely used in that region so that appropriate design modifications can be implemented to improve levee performance, (3) establish an erodeability testing methodology which can be used to assess existing earthen levees, (4) present some limited comments regarding “unwatering” (pumping), and (5) present comments and observations regarding emergency and interim levee and floodwall reconstruction efforts in the wake of hurricane Katrina.

10.2. Potential Levee Failure Mechanisms

There are numerous failure mechanisms that can result in the failure and breaching of earthen levees and/or floodwalls, and the resultant catastrophic flooding of protected areas. These failure mechanisms can occur as a single mode, or as a combination several different types of failure modes acting in unison. Levees can fail as a result of damage to the levee itself, if the foundation on which the levee is constructed fails, or as a result of failure of a floodwall for a composite levee/floodwall section. An abbreviated overview of many of the potential failure mechanisms of interest in the greater New Orleans area is presented here:

10.2.1. Structural Causes

This category includes potential failure mechanisms where the dominant issue is either the strength and stability of the levee embankment and/or foundation soils, or the structural capability of “structural” elements (e.g. sheetpile curtains, floodwalls, or gates) and/or their interaction with the levees and foundations soils that support them. Such mechanisms include:

<u>Slope Instability</u> – If the levee embankment soils and/or the underlying foundation materials that support the levee are weak, or become destabilized, a slope failure can develop and result in catastrophic failure of the levee. Slope failures can be minor or they can be significant enough to result in the catastrophic failure of the levee system. A number of catastrophic failures occurred during hurricane Katrina due to this mechanism. Slope failures can be sub-divided into separable classes as

(a) <u>Bearing capacity failure</u>; Failure of the weak foundation soils to vertically support the weight of the levee embankment. This is most common during construction (before the foundation soils have time to consolidate and gain strength under the embankment load.) A failure of this type just recently occurred on a section of levee under reconstruction in Plaquemines Parish on May 29, 2006.
(b) <u>Lateral translational stability failure</u>; Failure by sliding laterally, usually as a result of being “pushed” by elevated water pressures on the water (canal) side.
(c) <u>Deeper, rotational-type stability failure</u>; This can also be caused by the “push” of elevated water on the outboard (canal) side, but these types of failures can also occur due to undercutting of the canal side of the levee by dredging operations, or by storm surge scour or river flow.

<u>Structural Failures of Walls, Sheetpiles or Gates</u> – Simple structural failure as a result of structural elements inability to safely bear the forces and loads exerted against them. There appear to have been no structural failures of this type, except in cases where other types of

embankment or foundation failure, or overtopping erosion and resultant lateral unbracing of floodwalls, occurred.

Structural Impacts – Structural impacts occur when physical objects collide with the levee. This can occur during storm events when boats or barges become loose from their moorings and are driven into the levee by wind or water forces, or simply from accidental boat impacts due to operator error.

10.2.2. Causes due to Hydraulic Forces

This category includes failure mechanisms where the dominant parameters involve groundwater flow and pore pressure. Among these are:

Underseepage/Instability – As shown in Figure 10.1 (red lines), if the underlying foundation materials that support the levee are adequately permeable, water can quickly travel through these porous materials as the water head differential between the outboard and inboard sides of the levee increases. This underseepage raises the pore pressures within the soils at the inboard side, and this in turn reduces the shear strengths of these soils. This can result in catastrophic failure by means of resultant slope instability (as described in the previous section).

Bottom Heave or "Blowout" - This is a variant of underseepage, but involves the hydraulic pressure, rather than simple erosion. An increase in water pressure caused by a storm surge can travel through a permeable zone in a levee's foundation. If the water pressure exceeds the total overburden pressure at the landside toe of the levee, then the (impervious) soil overburden at that location can be displaced (heaved) by the water pressure, producing a large void into which subsequent flow will rush (rapidly exacerbating localized erosion and failure). This is often referred to as a "blowout" failure.

Erosion and Piping – As shown in Figure 10.1 (blue lines), erosion and piping occurs when the localized hydraulic gradient becomes large enough to "pull" soil grains from their location, and when there is no soil on that soil's down-flow side that can "filter" these soil grains and thus hold them in place. Erosion and piping can be sub-divided into two sub-classes as:

(a) Exit seepage erosion and piping; This is one of the most common causes of levee failures world-wide. If underseepage flow (and/or flow laterally through the levee embankment) becomes sufficient as to raise the exit gradient, then there is little or no resistance to erosion of soil particles at the point of water exit (either low on the inboard side levee slope face, or on the ground surface at and just inboard of the levee toe). Once the seepage gradient is able to exert enough "drag" on the soil grains to overcome the stability due to their self-weight, erosion begins. As soon as erosion begins, the local flow-net rapidly converges on the "hole" that begins to develop, as the water moves towards a preferential short-cut in its effort to escape. That, in turn, increases the local gradient and thus accelerates the erosion. The result is that erosion can rapidly 'eat back' a tunnel (or "pipe") beneath the levee [hence the name "piping".] Soils from above routinely slough and fall into this rapidly developing "pipe", but they are usually immediately washed out by the ever increasing flow until, finally, the embankment ruptures and erodes through catastrophically. An illustration of this was provided previously in Figure 8.105.

(b) Internal seepage erosion; This can occur internally within either the levee, or within the foundation soils. As water flows through these soils, smaller/finer soil particles can be "washed" out resulting in the internal erosion of the levee or foundation soil. Enough internal erosion can lead to the collapse and subsequent full-blown "wash-out" failure of the levee. For levees constructed of layers with significantly different permeabilities, the layer with the highest permeability becomes the main "conduit" by which the water flows through the levee. This concentrated flow can lead to higher water velocities through the levee and more rapid degradation. Appropriate control of soil gradation, and use of appropriate soil gradations within adjacent embankment sections (which is called "filtering"), are the keys to prevention of internal erosion.

The major design standards specifically address both internal erosion and piping as well as exit seepage erosion and piping, and levees are required to be engineered against these failure mechanisms. Mitigation strategies can include utilization of low permeability materials (such as clay), provision of underseepage "cut offs" (e.g. sheetpile curtains extending below the levees), internal drains or filters to safely "vent" pore pressures while filtering soil grains to hold them in place, widenend embankments and use of inboard-side stability berms to lengthen the flow path (and thus reduce exit gradients at the inboard side toe), control of soil gradation to prevent internal erosion, etc.

Effectively mitigating erosion and piping can be hampered by the presence of burrowing animals that can carve intricate tunnel networks within the earthen levees. Effective detection and corresponding correction of these animal-induced internal erosion channels is very challenging, and many levee failures throughout the world are a result of this failure mechanism. Exit erosion and piping can also be exacerbated by the presence of trees low on the inboard side levee faces and/or at the inboard toe. Trees that die can leave root hole paths that can exacerbate erosion and piping in this critical area. Trees that blow over during storm winds (and/or due to weakening of their roots' foundation soils due to wetting) can suddenly leave large voids that can serve as initiation points for rapid advancement of localized exit erosion and piping. As a result, it is common practice to prevent growth of trees (as a "maintenance" issue) in this critical inboard toe area.

10.2.3. Causes Involving Surficial Erosion

These include the various forms of surface erosion which can occur due to surface water flowing over (across) or against the exposed surface of the levee.

(a) Overtopping – As shown in Figure 10.2, overtopping occurs when the water level on the outboard side of the levee exceeds the crest elevation of the levee. The inboard side of the levee acts as a spillway for the overtopping water and damage is inflicted on the levee as a result of water scour. Levees are not generally designed for overtopping and as a result, if overtopping does occur, they are can be highly susceptible to catastrophic failure unless overtopping duration and intensity are limited and erosion-resistant materials are used to construct the levee (or unless erosion protection, in the form of "armoring" is applied to the exposed crest and faces of the levee.) As water passes over the top of the levee, its velocity increases as it runs down the back side, and the erosive shear force of the water increases with this increase in velocity.

Accordingly, overtopping erosion is usually most severe initially low on the back slope face. Eventually, as the crest is "notched", the resulting flow through the crest can also scour rapidly and further exacerbate the erosive process.

(b) Sharp Overtopping and Jetting – This is more likely to occur on levees with floodwalls. As shown in Figure 10.3, jetting occurs when the water level on the outboard side of the levee exceeds the top of wall elevation for structural walls that are founded within the earthen levee. Unlike overtopping of a conventional earthen levee, the floodwall acts as a weir and water falling over the wall impacts the levee in a concentrated stream that is much more energy intensive than conventional overtopping. For typical New Orleans floodwalls, the water impact velocities are on the order of 6 to 8 m/s. Levees are not generally designed for overtopping and jetting and as a result, if overtopping and jetting does occur, a deep scour trench can rapidly develop against the land-side face of the floodwall. This reduces the earth pressure providing lateral support for the wall, making the wall highly susceptible to potential catastrophic lateral failure (when pushed by the water pressures on its outboard side). This erosive trenching can be prevented by installing "splash-pads", coarse rip-rap, or other energy dissipating devices at the inboard side toe of the floodwall, and the use of highly erosion-resistant materials to construct the crown section of the levee is also advisable here.

(c) Lateral Surface Erosion – As shown in Figure 10.4, lateral surface erosion generally occurs on the outboard side of the levee and is the result of water flowing past the levee face, or against the face of the stream channel banks below. If the imposed shear stress from the water abrading against the soil face is high enough, soil scour occurs and the integrity of the overall levee is significantly reduced. Levees that are exposed to chronic water flow, such as river levees, are generally designed and constructed with armoring or erosion protection to minimize scour-induced surface erosion. In general, well-compacted levees constructed of high-plasticity clays are much more resistant to surface erosion than uncompacted cohesionless soils (e.g. "clean" sands) and silty sands. Surface protection such as rip-rap, concrete pads, soil-cement reinforcement, and select vegetation coverings are typical methods used to protect levee faces from surface erosion.

(d) Wave Impacts – As shown in Figure 10.5, wave impacts can cause significant erosion to levee faces. Wave-induced erosion consists of run-up (sloshing up and down of water as a result of staggered wave arrival) and "mini-jetting" when the crest of the waves breaks on the levee face. Levees that are anticipated to be impacted by waves are generally designed with armoring to prevent damage from wave impacts.

The aforementioned failure mechanisms are not intended to be an exhaustive list, but rather to highlight common potential failure modes of interest for the levees in the New Orleans region.

10.3. Design Standards

Design standards are not just the primary means by which earthen levees are designed; they are also the main metric by which proposed levee design and construction projects are assessed and critiqued by reviewers. Incomplete, inaccurate, or inappropriate design standards can lead to actual field performance which is less than desired. As part of this study, current earthen levee design standards from the United States Army Corps of Engineers and the United States Federal Emergency Management Administration were reviewed. A summary of the design guidelines for the USACE and FEMA are presented in Sections 10.4 and 10.5, respectively. This was not a fully exhaustive review (as that would have been beyond the scope of our current study), but this issue was studied as it provides important context for understanding some of the designs and decisions executed in the development of the pre-Katrina New Orleans regional flood protection system.

10.3.1. United States Army Corps of Engineers Design Standards

The primary manual and summary of design standards for earthen levees for the United States Army Corps of Engineers (USACE) is EM 1110-2-1913, "Engineering and Design – Design and Construction of Levees." This design manual covers the topics of: field investigations, laboratory testing, (fill) borrow areas, seepage control, slope design and settlement, levee construction, and special considerations (such as pipelines and other utility crossings, access roads and ramps, levee enlargements, junctions with concrete closure structures, and other special features such as landside ditch construction and levee vegetation management.)

10.3.1.1 Primary Design Procedure

The design procedure and requirements for levee design are established by EM 1110-2-1913. The outlined design procedure provides guidance from the initial preliminary evaluation through final design. These requirements (the principal "steps" in the design process) are summarized in Table 10.1.

Further engineering analysis guidance for the design of levees is provided in the following manuals:

- Slope Stability Analyses EM 1110-2-1902
- Settlement Analyses EM 1110-2-1904
- Levee/Structure Transitions EM 1110-2-2502

The punch list of design steps identified in Table 10.1 provides an overview of design parameters and principal steps for levee design. EM 1110-2-1913 prescribes required Factors of Safety for slope stability of newly designed levees, existing levees, and other embankments and dikes. These Factors of Safety vary from 1.0 for short-term loading conditions to 1.4 for long-term (steady state conditions). Specific design criteria are not provided for settlement and erosion-susceptibility.

In addition to these design parameters, material specifications and construction procedures, critical elements in the actual life-span performance of levees, have also been defined by the USACE. These components are described in more detail in the following sections.

10.3.1.1. Material Selection

Acceptable soils for the construction of levees (borrow materials) are defined by EM 1110-2-1913 as "any soil is suitable for constructing levees, except very wet, fine-grained soils or highly organic soils." Choosing a material type is generally a function of accessibility and proximity to the project area. The design guidelines emphasize that studies should be performed to ascertain the in-situ moisture contents of the borrow materials. It is noted that "the cost of drying borrow material to suitable water contents can be very high, in many cases exceeding the cost of longer haul distances to obtain material that can be placed without drying." Thus, any materials may be used in the construction of levees so long as they are not overly wet fine-grained soils or highly organic soils. As will be discussed later in this chapter, our field observations and laboratory testing clearly show that a high-performance levee must be constructed from superior materials, and that utilization of more marginal materials (as allowed by the design guidelines) can result in catastrophically poor performance.

10.3.1.2 Required Levee Soil Compaction

Three general types of engineered earthen levees are presented in the EM 1110-2-1913 design criteria. These are compacted, semicompacted, and uncompacted levees. The USACE notes that, traditionally, compacted levees are usually used for areas of high property values and/or high land use, high populations, and for steep-sloped embankments with controlled compaction during construction which are utilized on good foundation conditions. Areas of low values, poor foundations, or high rainfall during the construction season generally warrant specification of semicompacted or uncompacted levees.

According to the USACE design guidelines, compacted levees are required to be constructed in areas where strong embankments of low compressibility are needed adjacent to concrete structures or forming parts of highway systems. Compacted levees require specification of appropriate ranges of fill material water content during compaction (with respect to standard effort optimum water content), initial loose lift thickness of typically 6 to 9 inches, compaction equipment type (e.g. sheepsfoot rollers, rubber-tired rollers, etc.), and either the number of compaction passes to attain a given percent compaction or standard maximum density or specification of the minimum required density (relative compaction).

Semicompacted levees are recommended by the USACE to be constructed in areas where there are no space limitations and thus steep-sloped embankments are not required, where onsite foundation soil conditions are relatively weak and unable to support steep-sloped embankments, where underseepage conditions require a wide base, and/or where the water content of borrow materials or rainfall during construction does not allow for the proper compaction of levee fill material. Semicompacted levees require the specification of lift thickness (typically approximately 12 inches) and are compacted by the movement of hauling and spreading equipment, or by sheepsfoot or rubber-tired roller compaction equipment.

The USACE recommends uncompacted levees only to be used for temporary/emergency use. These levees are constructed by fill cast or dumped in place as thick layers with little or no spreading or compaction. Hydraulic fill by dredge, often from channel excavations, ~~is~~ a common fill borrow source for uncompacted levees. Hydraulic fills are known to be highly susceptible to erosion upon overtopping and are not recommended to be used in the normal construction of levees, except in locations where the levees are protecting agricultural areas whose failure would not endanger human life or for zoned embankments that include impervious seepage barriers.

10.3.1.3 Embankment Geometry

Embankment geometry specified by the USACE design guidelines is controlled either by material selection and compaction efforts during construction, or by the foundation soil conditions. Maximum side slopes for levees are 1V on 2H. These steep-sided levees are required to be constructed from high-grade borrow materials that are compacted near optimum moisture content and with appropriate compaction equipment. Levees with non-ideal borrow materials, such as sand levees, are required to have much shallower side slopes (on the order of 1V to 5H) to prevent damage from seepage and wave action. These design guidelines assume that this geometry and associated levee material will then be adequately resistant to scour and erosion, but as demonstrated by the numerous levee failures during Hurricane Katrina, this is not reliably always the case.

Final top of levee elevations must also account for future settlements, as determined by EM 1110-2-1904. In the past, the USACE specified a certain freeboard distance between the final top of levee elevation and the design storm water level to account for hydraulic, geotechnical, construction, operation, settlement and maintenance uncertainties. The updated design procedures set forth in EM 1110-2-1913 are risk-based, and are assumed to directly account for hydraulic uncertainties and establish a nominal level of protection.

10.3.1.4 Identified Potential Failure Modes for Design

The principal causes of potential levee failures, as identified by EM 1110-2-1913, consist of the following mechanisms (but see also Section 10.2 of this chapter):

- Overtopping;
- Surface erosion;
- Internal erosion (piping); and
- Slides within the levee embankment or the foundation soils.

Considerable discussion is presented in the design manual to mitigate effects of internal erosion/piping (see EM 1110-2-1913 Chapter 5 – *Seepage Control*). Guidance on overtopping, surface erosion, and slides within the levee embankment or the foundation soils is not well-developed in this design manual. However, guidance is provided for the augmentation of soil-cement protection applied to exposed slopes, susceptible to erosion.

10.3.1.5 Erosion Susceptibility

Although not directly addressed or identified in EM 1110-2-1913, general guidelines for erosion susceptibility of fine-grained cohesive soils are presented in EM 1110-2-1100 [Coastal Engineering Manual Part III], EM 1110-2-1100 [Coastal Engineering Manual Part VI], and "Channel Rehabilitation: Processes, Design, and Implementation," (1999). These manuals provide insights on erosion and critical values of average overtopping discharges. They provide valuable design information for levees situated in coastal areas, <u>and these design concepts should be applied to urban levees situated in coastal areas</u>. Levee stretches such as the MRGO and the

Orleans East southeastern levees abut Lake Borgne (an extension of the Gulf of Mexico) and are susceptible to wave attack during strong storms.

For coastal grass covered sea-dikes and protected embankment seawalls, EM 1110-2-1100 indicates that no damage occurs for overtopping discharges of less than about 0.15 ft^3/s per foot. Significant damage is expected for overtopping flows greater than 0.35 ft^3/s per foot. The overtopping discharge flows were based on wave run-up exceeding the crest of the embankment or floodwall crest, and include estimated impact forces associated with the wave action impacting the embankment. These values were based on field studies conducted both in the United States and in the Netherlands. Discharge flow values are not based on sustained overtopping discharge as a result of the mean storm water level rising above the crest of the embankment or floodwall.

Maximum permissible velocities for flow within river and stream channels are summarized in USACE (1999), and are based on field research from 1915 to about 1926. Permissible velocities (for a canal type section with an average depth of 3 feet) are presented in Table 10.2.

Equivalent shear stress in this Table was calculated using the following equation (Munson et al, 1990):

$$\tau_w = K\rho V^2/2 \qquad \text{[Equation 10.1]}$$

In this correlation, the shear stress (τ_w) imposed on the surface exposed to the water flow is a function of the surface roughness (K), the density of the fluid (ρ), and the velocity of the fluid (V). Based on this table, permissible water velocities vary between 1.5 ft/s for highly erosion susceptible materials to as much as 6 ft/s for highly erosion resistant materials. Correspondingly, allowable shear stresses vary from a low of 2.4 lb/ft^2 for highly erosion susceptible materials to as much as 36 lb/ft^2 for highly erosion resistant materials. Again, erosion plus jetting can lead to impact velocities with erosive potential about 3 to 4 times the maxima above. Although these design guidelines are available for use, they do not appear to have been incorporated into the design of the MRGO frontage and New Orleans East coastal levees fronting Lake Borgne.

10.3.2 United States Federal Emergency Management Agency (FEMA) Design Standards

Separate from the USACE levee design guidelines, design criteria for levee systems required by the United States Federal Emergency Management Agency (FEMA) are presented in the Title 44, Volume 1, Part 65 of the Code of Federal Regulations. These criteria establish the minimum standards to which levees must adhere in order to satisfy the 100-year level (referred to as the base flood) of protection mandated by FEMA. The main design criteria for FEMA approved levees are: freeboard, closures/transitions, embankment protection, embankment and foundation stability, settlement, interior drainage, and other specialty design criteria deemed appropriate by FEMA for unique situations.

10.3.2.1 Freeboard

Levees constructed adjacent to rivers are mandated to have a minimum freeboard of three feet above the water surface level of the base flood. In areas where the levee is constructed adjacent to structures, such as bridges, an additional one foot of freeboard is required extending 100 feet to either side of the structure. Levees constructed on the coast must have a minimum freeboard of one foot above the height of the calculated one percent wave or the maximum wave run-up (whichever is greater) associated with the 100 year still-water surge elevation. This category best fits some portions of the New Orleans Hurricane Protection System. Exceptions may be granted, based on site-specific engineering studies, but a freeboard of less than two feet is not deemed acceptable under any circumstance.

10.3.2.2 Closures

Closures refer to openings within the flood protection system. These closures can be for through traffic (such as railroad traffic which is frequently grade controlled and can not easily be diverted over levees), for pipeline crossings, or for maintenance purposes. FEMA requires all closures to be structural parts of the overall flood protection system during operation, and that they be designed in accordance with sound engineering practice.

10.3.2.3 Embankment Protection

Engineering analyses are required to be performed to demonstrate that no appreciable erosion of the levee embankment will occur during the base flood due to currents or waves, and that any anticipated erosion will not result in failure of the levee embankment or foundation either directly or indirectly through seepage or subsequent instability. Specific factors to be analyzed to determine the adequacy of embankment protection are: expected flow velocities (especially in constricted areas), expected wind and wave action, ice loading, impact of debris, slope protection techniques, duration of flooding at various stage and velocities, embankment and foundation materials, levee alignment, bends, transitions, and levee side slopes. The FEMA guidelines do not, however, provide guidance regarding acceptable performance criteria/standards for the identified embankment protection factors to be evaluated.

10.3.2.4 Embankment and Foundation Stability

Stability analyses for levee embankments are required to be submitted that demonstrate the adequacy of both short-term and long-term slope stability of flood protection levees. Stability analyses are required to include the expected seepage during the storm loading conditions, and to demonstrate that seepage into, beneath or through the embankment will not result in unacceptable stability performance. FEMA provides for the use of the USACE Case IV (as defined by EM 1110-2-1913, “Design and Construction of Levees”) as an additionally acceptable engineering analysis method. The required factors for evaluation include: depth of flooding, duration of flooding, embankment geometry and length of seepage path at critical locations, embankment and foundation materials, embankment compaction, penetrations, other design factors affecting seepage (such as drainage layers), and other design factors affecting embankment and foundation stability (such as interior berms). The FEMA guidelines do not, however, provide quantitative guidance regarding acceptable performance criteria/standards for the identified stability modes to be evaluated.

10.3.2.5 Settlement

Once levees have been constructed to the specified crest elevation, their ability to provide the desired degree of flood protection against the base flood is often dependent to large extent on settlements due to time-dependent compression of the foundation materials beneath the levee. In order to demonstrate the adequacy of the crest elevation over the intended service life, FEMA requires that engineering analyses be submitted that assess the potential and magnitude of future losses of freeboard as a result of levee settlement, and demonstration that freeboard will be maintained within the minimum freeboard requirements for the duration of the intended levee service period. Detailed analysis procedures, such as those specified in the USACE EM 1110-2-1904, "Soil Mechanics Design – Settlement Analyses," are expected. The required factors for evaluation include: embankment loads, compressibility of embankment soils, compressibility of foundation soils, age of the levee system, and construction compaction methods. There are no specific provisions for regional subsidence, tectonic subsidence, nor for potential water level rise due to long-term climate change.

10.3.2.6 Interior Drainage

FEMA requires that the protected side of the flood protection system be capable of draining onsite water. An analysis is required to be submitted that identifies the source(s) of potential flooding, the extent of the flooded area, and, if the average depth of flooding is greater than one-foot, the water-surface elevation(s) of the base flood. The analysis is required to be based on the joint probability of interior and exterior flooding and the capacity of facilities (such as drainage lines and pumps) for draining interior floodwater.

10.3.2.7 Other Design Criteria

In areas where levee systems have relatively high vulnerabilities, or other unique situations, FEMA may require other design criteria and analyses be submitted for review and approval. The rationale for the requirement of additional analyses will be provided by FEMA. The review and subsequent evaluation standard of the analyses for the specified design criteria are to be based on "sound engineering practice."

10.3.2.8 Other FEMA Requirements

In order for the levee flood protection system to be recognized by FEMA as providing protection for the base flood, additional requirements, beyond the established design procedures and criteria, are required to be in place. Maintenance and operation plans are required to be submitted that detail how the flood protection system will be maintained and operated during its service period. In addition, FEMA has certification requirements which require that a registered professional engineer certify the levee design, and that certified as-built plans of the completed levee be submitted. Federal agencies with responsibility for levee design may also certify that the levee has been adequately designed and constructed to provide the desired degree of protection against the base flood.

10.4 Storm Surge and Wave Action During Hurricane Katrina

During Hurricane Katrina, the earthen levees were subjected to storm surges and wind generated wave action. Accurately determining the magnitude of these forces is reliant on numerical simulations and modeling with calibration from field data such as in-place instrumentation that recorded data during Hurricane Katrina as well as post-hurricane field assessments, such as high-water marks. The most reliable storm surge and wave action information collected and recorded during Hurricane Katrina was captured by instrumentation installed at select locations within the greater New Orleans area. However, the number of instrumentation locations was extremely limited, and as a result, little reliable storm surge and wave action information is available from this source. Many of the instruments were damaged during the storm and only partial records were collected.

Instruments that recorded useful data used to establish storm surge and wave action information were located at the following locations (IPET 2006):

- Lake Pontchartrain near 17th Street Canal (*hydrograph & wave characteristics*)
- Pump station #6 on the 17th Street Canal (*hydrograph*)
- Lake Pontchartrain at the Lakefront Airport (*camera-based hydrograph)*
- Inner Harbor Navigation Channel at I-10 (*hydrograph*)
- Inner Harbor Navigation Channel at the Lock (*hydrograph*)
- Gulf Intracoastal Waterway at I-510 (*hydrograph*)

A detailed review and reconstruction of the storm surge and wave action during Hurricane Katrina based on the data from the installed instruments, measured high water marks and interviews was completed by IPET (2006). The storm surge and wave action information presented by IPET was used in our performance evaluation of the levees. A discussion of the maximum storm surges is presented in the following section along with an overview of our field reconnaissance and levee condition survey and mapping.

10.5 Field Reconnaissance and Levee Condition Mapping with Regard to Levee Erosion

Field reconnaissance was a vital part to assessing and understanding the performance of the earthen levee flood protection systems. Multiple field visits were performed by the team to visually observe and evaluate the performance of the levee systems. The initial levee assessments occurred between September 29 and October 15, 2005. The principal purposes of these initial site investigations were to perform an initial survey of major damage areas, to perform initial forensic studies at the major failure (breach) sites and at nearby, more successful sites, and to note and record time sensitive data and observations before ongoing emergency repair operations obscured vital storm-related levee system performance information. The results of our initial observations and findings are presented in Seed et al., (2005).

Subsequent to the initial field reconnaissance and forensic studies, a series of field survey explorations have been performed to extend the initial condition surveys and to collect physical samples for testing to ascertain susceptibility to erosion. The variable performance of the earthen levee flood protection components during hurricane Katrina provide a unique learning opportunity in that many of the levee system elements were overtopped, impacted by moving objects and debris (such as steel barges and fishing boats), and/or attacked by wind-generated waves. Some sections performed extremely well, while other sections performed poorly. As a result, there is a valuable opportunity to draw empirical lessons regarding the interactions between water and wave loadings, embankment and foundation soils and geometry, and performance.

Figure 10.6 shows the extents of the formal visual reconnaissance (the dashed black line) that was completed as part of our follow-on study on this issue. Due to access, schedule, and funding limitations, the Independent Levee Investigation Team was not able to complete a full and comprehensive survey of the entire greater New Orleans area for this element of our studies. Locations of noteworthy performance have been identified in the numbered boxes on Figure 10.6, and these are discussed in further detail below (refer to Figure 2.6 in Chapter 2 for a summary of design elevations for the flood protection system). Please note that these locations are intended only to represent typical findings and are not intended to summarize the complete performance of the overall flood protection system. In addition, the specified design flood protection system component crest elevations may not be the actual crest elevations at the time of hurricane Katrina's arrival due to factors such as incomplete staged construction, consolidation and settlement, regional subsidence, difficulties in correlating datums and elevation benchmarks, etc.

It is important to emphasize that accurately determining elevations in the greater New Orleans area is extremely complicated. Factors that exacerbate the problem include regional subsidence, localized consolidation settlement, progressive settlement of benchmarks used to establish regional datums, and the temporal variation in time of completion of individual project sections. A tremendous effort has been undertaken by the IPET team to "equalize" all the locations that are part of the New Orleans Flood Defense System and to merge the many component-specific elevations to one common project-specific elevation datum.

Following is a summary and discussion of levee performance at select locations along the perimeter of the New Orleans Flood Defense System sections traversed in this section of our studies (as shown in Figure 10.6.) A detailed and more comprehensive assessment of the as-reconstructed levee system is recommended to be conducted upon completion of the ongoing repairs and upgrades.

10.5.1 Location 1 – Lakefront Airport

Location 1 is situated near the intersection of Downman Road and Hayne Boulevard, south of the old Lakefront Airport. At this location, an earthen levee connects to a railroad bridge and vehicular underpass and an adjoining concrete floodgate structure, and the levee is situated parallel to an active railroad line. This is a highly complex "penetration" (where several access ways pass through or across the federal levee; including the rail line and the roadway), and a complex set of "transitions" where disparate flood protection elements abut each other and must perform well in combination. Unfortunately, this is one of numerous sites where these elements were not adequately detailed and coordinated, and performance was unacceptably poor and a breach occurred at this location.

High water marks, as reported by IPET (2006), at this location reached approximately Elevation +12 feet (MSL). The design elevation of the levee system at this location was Elevation +13.5 feet (NGVD29). Exact datum conversions in this area are not clearly established and are still under review by the IPET team, but the design elevation has been identified as Elevation +11.8 feet (MSL), resulting in some degree of overtopping at this location.

Storm-surge induced overtopping traveled through the low spot at this complex transition/penetration, which was the granular gravel ballast for the railroad line, and this flow eroded the railroad line embankment, which served as a transition levee between the concrete floodwall (design Elevation + 13.5 feet MSL = +11.8 feet NAVD 88-2004.65) and the earthen levee (design Elevation +14.5 feet MSL = +12.8 feet NAVD88-2004.65) shown in Figure 10.7. Figure 10.8 shows the location where overtopping occurred resulting in significant scour around the floodwall and Figure 10.9 provides a view across the railroad line where the railroad line embankment was eroded allowing for the terminus of the earthen levee to be scoured. Note that at the time of our visit, the railroad embankment had already been repaired by railroad personnel.

Performance factors of the levee system that impacted the performance of the flood protection components included the following: (1) unprotected high-permeability ballast (gravel) which allowed high water levels to seep through the gravel ballast and erode the supporting railroad embankment, (2) inadequate transition details between the flood protection components which allowed for low points to be exploited, and (3) the presence of embankment and levee materials that were not erosion resistant, resulting in scour as a result of overtopping. Without redesigning this transition area, future performance at this location (under similar or more severe storm surge conditions) is anticipated to be poor, and it will likely breach again.

10.5.2 Location 2 – Jahncke Pump Station Outfall

Location 2 is situated near the intersection of Hayne Boulevard and Jahncke Road, near Lake Pontchartrain. At this location a concrete outfall structure protrudes through the flood protection levee. High water marks, as reported by IPET (2006), at this location reached a maximum Elevation of +12 feet (NAVD88-2004.65). The design elevation of the levee system at this location was Elevation +14.5 feet (NGVD29). Exact datum conversions in this area are not clearly established and are still under review by the IPET team, but the design elevation has been identified as Elevation +12.8 feet (NAVD88-2004.65), resulting in a minor degree of overtopping at this location. Our field reconnaissance verified that minor overtopping occurred at this location, as can be seen in Figures 10.10 and 10.11.

Figure 10.10 provides an eastward looking view. Small patches can be seen on the levee crest where minor erosion occurred. Figure 10.11 presents a view of scour-related erosion behind the concrete outfall structure transition.

Performance factors of the levee system that impacted the performance of the flood protection components included the following: (1) placement of rip rap boulders along the Lake Pontchartrain margin which aided in damping wind-driven waves approaching the levee, (2) the presence of the active railroad line which also aided in damping wind-waves, and (3) utilization of moderately erosion-resistant embankment materials (moderately compacted clayey soils and sandy clay soils.)

As shown in Figures 10.10 and 10.11, performance at this location was acceptable despite the moderate overtopping; only limited erosion occurred at select locations across the earthen levee crest and on the back face, and additional erosion occurred preferentially at several soil/structure interfaces. None of this erosion was sufficient as to result in a breach of the flood defenses at this location. The moderate erosion that did occur, however, suggests that more severe overtopping would likely be more problematic and might well cause full breaching if this section is not upgraded for erosion resistance.

10.5.3 Location 3 – Eastern Perimeter of New Orleans East

Location 3 is situated approximately 0.6 miles east of Highway 11 and approximately 1 mile north of Chef Menteur Highway (Hwy 90). In this vicinity, the flood protection system consists primarily of earthen levees that are protected by both low-lying swamplands and trees on both the outboard (water) side and the inboard (protected) side of the levee. High water estimates, as determined through numerical modeling by IPET (2006), suggest that the water at this location reached a maximum Elevation of approximately +16 feet (NAVD88-2004.65). The design elevation of the levee system at this location was Elevation +14.5 feet (MSL). Exact datum conversions in this area are not clearly established and are still under review by the IPET team, but the design elevation has been identified as Elevation +12.4 feet (NAVD88-2004.65), resulting in significant and relatively sustained overtopping at this location.

Figure 10.12 provides a southward looking view. The overall condition of the levees in this area is excellent and no observable damage or erosion was encountered. As part of local improvements after hurricane Katrina, an outfall access structure, near Hwy 90, was outfitted with a rock-gabion transition zone to minimize scour around the concrete access structure, as seen in Figure 10.13. This is an excellent idea, and should serve to further improve the performance of this detail in future events.

The excellent performance at this site was likely due to a number of factors, including: (1) the presence of low-lying swamp areas which aided in damping wind-waves approaching the levee, (2) the presence trees and shrubs outboard and inboard of the levee which also aided in damping wind-waves, and (3) utilization of moderately to highly erosion-resistant embankment materials.

10.5.4 Location 4 – Southeast Corner of New Orleans East

Location 4 is situated at the southeast corner of the New Orleans East polder. In this vicinity, the flood protection system consists primarily of earthen levees adjacent to the GIWW, fronting Lake Borgne. A small stretch of low-lying swamp protects the outboard side of the levees in this area, but it affords relatively little effective protection from wind-driven waves when “Lake” Borgne swells with storm surges from the Gulf.

High water marks, as determined by IPET (2006) using numerical simulations, suggest that water levels at this location reached a maximum Elevation of approximately +16 feet

(NAVD88-2004.65). The design elevation of the levee system at this location was Elevation +19 feet (MSL). Exact datum conversions in this area are not clearly established and are still under review by the IPET team. As discussed previously in Chapter 7, significant erosion and breaching occurred at this location, and this length of levee frontage was the single largest point of ingress for the floodwaters that eventually inundated the New Orleans East protected basin. The IPET studies have ascribed this massive erosion principally to overtopping, but it is the view of this investigation that considerable erosion also occurred due to wave action prior to full overtopping, and that through-levee seepage and underseepage may also have played a role at some locations (see Chapter 7).

Figure 10.14 shows one of a number of eroded zones or "slots" of the original levee that was breached and scoured as a result of storm-surge induced erosion during the hurricane. The levee embankment at this section was comprised largely of material dredged from the excavation of the adjacent GIWW channel, and large portions of this embankment appear to have had little resistance to erosion.

Figure 10.15 shows completed levee rehabilitation work at the southeast corner. At the time of our visit, construction activities had shifted approximately 1 mile west and consisted of belly-dump trucks placing borrow material which was being spread by bulldozers and track-walked. Dump trucks were also directed to travel over the newly placed levee, employing the semicompaction technique defined in EM 1110-2-1913.

Performance factors of the levee system that impacted the unacceptable performance of the earthen levees along this frontage included the following: (1) lack of slope protection (and crest protection) to minimize erosion due to both storm-driven waves and overtopping flow, (2) the site's location adjacent to the "open" and relatively deep waters of Lake Borgne allowing for significant wind-driven waves to form and scour the flood side of the levee (and to notch the crest), (3) the lack of useful protection from outboard side swamp and/or cypress groves, etc., to reduce the energy and intensity of wind-driven waves, and (4) the utilization of embankment fill soils of variable erosion resistance (and permeability) so that both wind-driven wave erosion and through-flow erosion, as well as overtopping (wave splashover) erosion are all likely to have been active at this location.

This levee frontage performed disastrously poorly, and these factors that contributed to that unacceptable performance must be eliminated in the future.

10.5.5 Location 5 – Entergy Michoud Generating Plant

Location 5 is situated along the GIWW/MRGO shared channel, immediately beneath the Hwy 47/Paris Road bridge. In this vicinity, the flood protection system consists primarily of earthen levees. High water marks, as reported by IPET (2006), at this location reached a maximum Elevation of +16.3 feet (NAVD88-2004.65). The design elevation of the levee system at this location was Elevation +15 feet (MSL). Exact datum conversions in this area are not clearly established and are still under review by the IPET team, but the design elevation has been identified as Elevation +13.2 feet (NAVD88-2004.65), resulting in moderate to significant overtopping at this location.. Our field reconnaissance verified that overtopping in fact occurred at this location.

Figure 10.16 shows actual overtopping of the levee as captured by a security video camera at the Entergy Michoud Generating Plant during Hurricane Katrina. Figure 10.17 presents a post-Hurricane Katrina view of the same section of levee shown in Figure 10.16. Only minor damage occurred on the protected side, with a majority of the damage appearing to have resulted from wave reflection from the adjacent bridge abutment. Excepting the section where some minor erosion occurred on the inboard-side slope of the levee, this levee frontage for many hundreds of feet in each direction showed little evidence of damage from relatively sustained, moderate to significant overtopping flows. The overall condition of the levees in this area was good and no major damage was encountered.

The good performance of this embankment in the face of sustained moderate overtopping was likely due to several factors including: (1) utilization of moderately to highly erosion-resistant embankment materials (clay), and (2) the small fetch of the GIWW/MRGO channel at this location which limited the height of the wind-generated waves.

10.5.6 Location 6 – GIWW/MRGO Southern Shoreline Levee

Location 6 is situated along the ICWW/MRGO interchange beneath the Hwy 47/Paris Road bridge, on the opposite side of the channel (on the south bank) from Location 5. In this vicinity, the flood protection system consists primarily of earthen levees with a concrete floodwall beneath the bridge that connects the eastern and western levee segments. High water marks, as reported by IPET (2006), at this location reached a maximum Elevation of +16.3 feet (NAVD88-2004.65). The design elevation of the levee system at this location was Elevation +14 feet (MSL). Exact datum conversions in this area are not clearly established and are still under review by the IPET team, but the design elevation has been identified as Elevation +13.2 feet (NAVD88-2004.65), resulting in significant and likely relatively sustained overtopping at this location. Our field reconnaissance verified that overtopping occurred at this location.

The overall condition of the levees in this area was good and no major damage was encountered. The concrete floodwall constructed beneath the Hwy 47/Paris Road bridge, due to its top of wall elevation being lower than that of the neighboring earthen levees, acted as a weir during the high water period and "sucked" in nearby steel barges, as shown in Figure 10.18. Despite the collision impact of the barges with the concrete wall, the system performed well. Some scour-related damage was observed at the transition between the concrete flood wall and the earthen levee. Figure 10.19 presents an eastward looking view of the levee, just west of the washed up barges. East of the Hwy 47/Paris Road bridge, Figure 10.20 shows a gas processing barge that collided with the earthen levee. The impact did not result in significant damage to the levee.

The good performance of the levee system at this location was likely due in large part to the utilization of moderately to highly erosion-resistant embankment materials; these erosion-resistant (clayey) materials were also capable of absorbing impact loads from the barges, allowing the barges to come to rest on the levee without breaching it. The small fetch of the GIWW /MRGO canal at this location may also have limited the height of the wind-generated waves, thereby minimizing wave-induced erosion of the levee materials.

During a subsequent visit to this site in March of 2006, we observed that a concrete apron was being installed by the USACE around the transition between the concrete floodwall and earthen levee. This transition detail is anticipated to minimize future erosion at this transition area during a future severe event and reduce the risk of an erosion-induced full breach of the levee.

10.5.7 Location 7 – Bayou Bienvenue Control Structure

As shown in Figure 10.6, Location 7 is situated along the northern end of the MRGO levee frontage, near the northeast corner of the St. Bernard Parish protected basin. In this vicinity, the flood protection system consists primarily of earthen levees that connect, with a concrete control structure (with steel floodgates) across Bayou Bienvenue. High water marks, as reported by IPET (2006), at this location reached a maximum Elevation of +18.4 feet (NAVD88-2004.65). The intended eventual design elevation of the levee system at this location was Elevation +17.5 feet (MSL). Exact datum conversions in this area are not clearly established and are still under review by the IPET team, but the design elevation has been identified as Elevation +13.2 feet (NAVD88-2004.65). As discussed in Chapter 6 (see Section 6.2), this levee frontage was incomplete at the time of hurricane Katrina's arrival, as a final crest raising to offset consolidation-induced settlements has not yet been implemented. As a result, major overtopping occurred at this location.

Figure 10.21 shows an aerial photograph of the Bayou Bienvenue control structure. The floodwall to the north of the control structure performed very well, withstanding an impact load from a steel barge which became lodged atop the concrete flood wingwall attached to the concrete control structure.

The southern side of the control structure did not perform well; massive erosion and scour produced a major breach at the contact between the concrete control structure and the adjacent earthen levee. The southern side of the control structure was built using mainly spoils from the excavation of the adjacent MRGO channel that are more erosion-susceptible than the clays on the northern side of the control structure. It is important to note that both sides of this control structure were subjected to similar loading conditions and overtopping occurred on both sides and as a result, this site offers a unique example of the importance of erosion-resistant soil materials. In addition, the southern portion of the control structure abuts the abandoned Bayou Bienvenue channel, as shown in Figure 10.21. It is not conclusive whether the backfill materials into the abandoned channel impacted the performance of the control structure, but further investigation should be employed to determine the performance factors for this side of the control structure.

Figure 10.22 shows a close up view of the flood control gate structure that acted as a weir as the water overtopped the flood protection system. Significant scour and erosion was observed around the structure. Upon a follow up visit in March of 2006, splashpads had been installed behind the flood gate structure. In addition, the steel barge had been removed from the concrete control structure and a vast sea of rip-rap protection installed around the control structure. Figure 10.23 presents a picture of the installed splash pads and Figure 10.24 presents a view of the control structure with the barge removed and placement of rip-rap. Figure 10.25 presents a schematic of the mapped scour around this concrete structure observed in October of 2005. Around the ends of the concrete wall, about 10 feet of soil have been eroded.

Factors of the levee system that impacted the performance of the flood protection components included the following: utilization of moderately to highly erosion-resistant embankment materials on the northern end of the control structure, utilization of moderately to highly erosion-susceptible embankment materials on the southern end of the control structure, and possible effects of the old Bayou Bienvenue channel abandonment backfill materials on the southern portion of the control structure. Future performance at this location under similar or more severe conditions is anticipated to be good for the northern half of the control structure and poor for the southern half of the control structure. Significant overtopping should be expected for larger storm surge events.

10.5.8 Location 8 – Mississippi River Gulf Outlet

Location 8 is situated along the western edge of the MRGO, south of the Bayou Bienvenue Control structure and north of the Bayou Dupre Control structure. In this vicinity, the flood protection system consists primarily of earthen levees constructed from excavated materials from the MRGO channel. High water marks, as reported by IPET (2006), at this location reached a maximum Elevation of approximately +18 feet (NAVD88-2004.65). The design elevation of the levee system at this location was Elevation +17.5 feet (MSL), however, reports indicated that these levees were not fully completed and had crest elevations that were 3 to 4 feet lower than the specified design elevation. In addition, exact datum conversions in this area are not clearly established and are still under review by the IPET team, but the design water level has been identified as Elevation +12.7 feet (NAVD88-2004.65). During Hurricane Katrina, moderate to major overtopping occurred at this location. Our field reconnaissance verified that moderate to major overtopping occurred.

Figure 10.26 shows a United States Geological Survey topographic map of the MRGO area. The identified "spoil area" corresponds to the zone of poor levee performance. Figure 10.27 shows aerial photography taken by NOAA in early September 2005 along the MRGO and shows severe erosion/breaches in the levee and barges that floated over the top of the levee and came to rest inside St. Bernard Parish after water elevations receded. Figure 10.28 shows close up aerial photographs of the severely eroded levees.

A bank erosion study was performed by the USACE (1988) that identified the presence of highly erosion-susceptible soils within the MRGO alignment. Merchant shipping traffic that traversed the MRGO created wake-induced waves and drawdown that were eroding the channel banks, resulting in the widening of the MRGO from an intended 650 feet to an actual average width of 1,500 feet, more than double the design width. Comments submitted from the Lower Mississippi Valley District on the report made the following comment in response to selecting the bank erosion mitigation alternative of decommissioning the MRGO:

> *The alternative to completely close the MRGO waterway should be evaluated….This alternative will control all future channel maintenance problems by controlling bank erosion, preventing the associated biological resource problems, preventing saltwater intrusion, and lessening the recreational losses.*

In addition to solving the aforementioned problems, it will also reduce the possibility of catastrophic damage to urban areas by a hurricane surge coming up this waterway and also greatly reduce the need to operate (and could possibly eliminate) the control structures at Bayous Dupre and Bienvenue

Slope protection measures were recommended to aid in stabilizing these highly erosive deposits against wave-induced erosion. At the time of Hurricane Katrina, slope protection measures along the flood side of the MRGO levee were not in place. As identified in the above comments, the Hurricane Katrina storm-surge massively eroded the levees and resulted in catastrophic failure.

Performance factors of the levee system that impacted the performance of the flood protection components included the following: utilization of highly erosive embankment materials, lack of appropriate surface slope protection to minimize erosion of the flood side of the levee during the storm-surge, and as-constructed crest elevations below design elevations allowing for significantly higher water overtopping heights. Future performance, based on prior performance, at this location under similar or more severe conditions is anticipated to be poor unless improved materials and construction methods are used. We were unable to sample this location and test the materials for erodibility, but we did perform a follow up visual reconnaissance in April of 2005, and at that time we were very favorably impressed at the calibre of the work, and of the materials, being placed and compacted. During that follow-on reconnaissance, we did not detect the use of unsuitable fills, but we would like to return to sample and test the soils used to construct the levee in this section of the MRGO.

10.5.9 Location 9 – Bayou Dupre Control Structure

Location 9 is situated approximately 6.5 miles southeast of the Bayou Bienvenue Control structure, on the west side of the MRGO. In this vicinity, the flood protection system consists primarily of earthen levees that connect via a concrete and steel flood access structure to a concrete control structure across Bayou Dupre. High water marks, as reported by IPET (2006), at this location reached a maximum Elevation of +17 to +22 feet (NAVD88-2004.65). The design elevation of the levee system at this location was Elevation +17.5 feet (MSL). Exact datum conversions in this area are not clearly established and are still under review by the IPET team, but the design water elevation has been identified as Elevation +12.7 feet (NAVD88-2004.65), resulting in moderate to major overtopping at this location. Our field reconnaissance verified that moderate to major overtopping occurred.

Figures 10.29 and 10.30 show aerial photographs of the Bayou Dupree control structure. The area to the south of the control structure performed very well, while the northern side of the control structure did not perform well, with significant erosion and scour as a result of the overtopping. The northern portion of the control structure abuts the abandoned Bayou Dupre channel, as shown in Figure 10.31. It is not conclusive whether the backfill materials into the abandoned channel impacted the performance of the control structure, but further investigation should be employed to determine the performance factors for this side of the control structure.

Figures 10.32 and 10.33 show aerial photographs of repair operations underway at Bayou Dupre in January 2006. As can be seen in Figure 10.33, sand borrow material has been imported to be used in the backfilling repair operations in the deep scour pools on the north side of the control structure.

Performance factors that impacted the performance of the flood protection components at this location included the following: utilization of highly erosive embankment materials, lack of appropriate surface slope protection to minimize erosion of the flood side of the levee during the storm-surge, as-constructed crest elevations below design elevations allowing for significantly higher water overtopping heights, and possible effects of the old Bayou Dupre channel abandonment backfill materials on the northern side of the concrete control structure.

10.5.10 Location 10 – St. Bernard Parish Interior Levee (Forty Arpent Levee)

Location 10 is situated north of the Corinne Canal and approximately ¾ miles east of Hwy 47/Paris Road. In this vicinity, the flood protection system was designed to be a secondary containment system for potential overtopping-related flooding behind the MRGO frontage levees and to act as a barrier against rainwater that is discharged into the swamp area. This part of the flood protection system consists primarily of earthen levees with a design Elevation of +8.0 to 9.0 feet (MSL). The actual elevation of this system during Hurricane Katrina was on the order of 5 to 8 feet (MSL) - (note that IPET did not establish NAVD88-2004.65 elevations at this location). High water marks were not reported by IPET at this location. Based on our field reconnaissance, it was apparent that major overtopping occurred at this location.

Figure 10.34 shows an eastward looking view of the earthen levee. Although this levee was significantly overtopped, it did not experience significant damage or erosion. Figure 10.35 shows a fishing boat that was washed over the levee shown previously in Figure 10.34 and came to rest in a residential neighborhood. The clearly excellent erosion resistance of this secondary levee, despite significant overtopping, was the result of use of cohesive, clayey soils as the levee fill material. The ability of these soils to sustain significant overtopping without catastrophic erosion is an important lesson.

Upon a subsequent visit to this location in March of 2006, we observed that the levee had been improved and raised by several feet to a new Elevation of +10 feet (MSL). Figure 10.36 presents the same view as in Figure 10.34, but 5 months later. Based on our observations, it appeared that cohesive soils and semi-compaction construction methods were used for this improvement. These soils appeared to be in a relatively erodeable condition as initially placed, but it also appeared likely that significant wetting from rainfall would improve their resistance to erosion from overtopping.

The performance factor that most significantly influenced the observed surprisingly good performance of the flood protection levee embankment at this site was the utilization of moderately to highly erosion-resistant embankment materials. Future performance of the levee embankment at this location under similar or more severe conditions is also anticipated to be good, however, significant overtopping should be expected for larger storm surge events which may cause the MRGO levees to breach, as this secondary levee has a crest elevation well below that of the main MRGO frontage levees.

10.5.11 Summary of Observed Performance Factors with Regard to Erosion Due to Overtopping

Based on observations from our field reconnaissance and review of aerial photographs, it is apparent that the performance and post Hurricane Katrina conditions of the earthen levee systems varied significantly, from good performance in areas with major overtopping to poor performance in areas with minor degrees of overtopping.

Table 10.3 presents a summary of the 10 locations evaluated as part of this study. Most of the levees studied in this pilot study were overtopped as a result of the large storm surge that rushed onshore. The magnitude of the storm surge and resulting overtopping did not, however, singularly dominate the observed performance of the levees. Additional factors were the degree of outboard side protection afforded by cypress swamps (which diminished wave energies), and the intrinsic resistance of the levee embankment and foundation soils to erosion.

Table 10.3 lists these factors, as well as the observed performance of the sections studied. As with the ILIT team's overall observations and studies of failures and successful performances throughout the New Orleans Flood Defense System, it is apparent that the use of highly erodeable soils such as cohesionless (sandy) soils represents a potentially unacceptably hazardous condition, and that the use of suitably compacted cohesive, clayey soils with relatively high intrinsic resistance to erosion can provide a measure of ductility and resilience to an otherwise brittle system (levees that can overtop for some number of hours, without catastrophically eroding and breaching.)

10.6 Erosion Susceptibility Evaluation

To date, the field of scour and erosion has not been well characterized and there is a paucity of well-defined field case studies that relate actual performance to design parameters. Both the USACE and FEMA design guidelines do not specifically provide acceptability criteria for erosion susceptibility due to the lack of comprehensive knowledge in this area. As a result of the NOFDS being "loaded to failure," it has provided an unfortunate opportunity to recognize lessons learned and improve our body of knowledge for the performance of levee flood protection devices.

It is important to note the magnitude of devastation caused to the regional flood protection system as a result of erosion. The levee system along the MRGO, which is the main protection mechanism for the 100,000+ citizens of St. Bernard Parish (and the Lower 9th Ward) against storm surges from the Gulf of Mexico, was catastrophically degraded as a result of erosion during Hurricane Katrina. Similarly catastrophic erosion occurred at the "sister" section of levee at the southeast corner of the New Orleans East protected basin. Figure 10.37 shows a comparison of two LIDAR surveys of the MRGO levee system at the Bayou Bienvenue control structure, near the intersection of the GIWW at the northeast corner of St. Bernard Parish. Effects of subsidence can be clearly seen in the elevation differences on the north side of the control structure and the control structure itself between 2000 and immediately following Hurricane Katrina in 2005. The levee on the north side of the Bayou Bienvenue control structure was largely undamaged. The levee on the south side of the control structure was catastrophically damaged. The magnitude of the erosion has been highlighted and white "splotches" of displaced levee materials can be seen in the aerial photograph.

There are many factors that influence the erosion susceptibility of soils, and a more comprehensive discussion of these was provided in Chapter 9. Fundamentally, soil erosion is controlled by the resisting characteristic of the soil (including soil type and character, soil fabric and structure, in-situ density, etc.) and eroding forces (including the magnitude and duration of the shear stress applied to the soil, impact or jetting pressures, etc.) from the contacting (eroding) fluid.

A sampling and laboratory testing program was devised upon completion of our field levee condition survey and mapping in October of 2005, to try to understand and characterize the properties associated with the levees that performed well and the characteristics of the levees that did not perform well during Hurricane Katrina. The intent of this study was to better understand the nature of the levee materials that performed well during the extreme conditions in order to provide recommendations on how to improve the sections of earthen levees that did not perform well.

In-situ samples were collected and from select levee sites during January and February of 2006. The selected sampling sites included levees that performed very well during Hurricane Katrina, levees that performed moderately well, and levees that did not perform satisfactorily. Figures 10.38 and 10.39 identify the locations where samples were collected for laboratory analyses.

Erosion susceptibilities of the soils were characterized using a state of the art erosion index testing method, developed by Dr. Jean-Louis Briaud at Texas A & M University, known as an Erosion Function Apparatus (EFA). This test method required undisturbed samples to be sampled from the field and be carefully transported back to Texas A&M University for analyses.

As described in Chapter 9, the EFA is a test that determines the shear stress and velocity of flowing water required to erode soil from a cylindrical tube that is slowly advanced into a rectangular pipe of flowing water. The more erosion resistant the soil, the faster the water (and the higher the shear stress) is required to flow in the rectangular pipe in order to erode the soil sample. A diagram of the EFA is presented in Figure 10.40. The measured shear stress at the point at which the soil begins to erode is defined as the critical shear stress. Shear stress less than the critical shear stress will not result in erosion, whereas applied shear stresses in excess of the critical shear stress will result in erosion. Determination of the erodibility index is useful in completing analyses for overtopping and surface erosion.

Upon completion of the test, the erodibility index of the soil was defined and the rate of erosion as a function of applied shear stress (or velocity) established. This relationship can then be compared with anticipated shear stresses the soil will experience in the field. If the estimated field shear stresses are less than the shear stress required to erode the soil, no erosion is anticipated to occur. If the field shear stresses exceed the laboratory determined critical shear stress, the erodibility index provides a means by which to estimate the magnitude of the overall erosion.

In addition to the erosion testing itself, additional engineering characteristics of the earthen levee sections under study were also characterized. These characteristics included the following:

- Gradation, including passing the Number 200 sieve (ASTM D422)
- Hydrometer (ASTM D422)
- Atterberg Limits Determination (ASTM D4318)
- Unconfined Compression (ASTM D2166)
- Dry density and moisture content determination (ASTM D4937/2216)
- Maximum dry density determination (ASTM D1557)

Table 10.4 presents a summary of the locations where samples were collected for analyses, and these locations are shown in Figures 10.38 and 10.39.

The samples were collected by pushing an approximately 3-inch diameter steel (Shelby) tube into the ground to retrieve soil samples using a geotechnical testing drill rig. Sites 4, 5, and 6 were located along the MRGO section of levee that suffered severe overtopping and erosion. At these sites the levee materials were collected in a soil sample bag and reconstituted back in the laboratory due to the highly disturbed nature of the levee materials.

The erosion susceptibilities of all the samples collected are presented in Figure 10.41. The test result designations are based on the Site Number, the boring number at the site, the depth interval over which the sample was collected, and additional sample notes. Thus, a sample marked as S1-B1-(0-2ft)-TW indicates that the sample came from Boring 1 at Site No. 1 (Levee east of HWY 11 and North of HWY 90) from a depth of 0 to 2 feet below the crest of the levee.

The results of the EFA test results are also presented in Table 10.4. The EFA test results matched very well with the observed performance in the field. Areas where the levee performance was observed to be good generally had low to very low erosion susceptibilities. In areas where the levee performance was poor, the materials had a high to very high erosion susceptibility.

The effects of material compaction were also evaluated. Previous work has been performed in this area and a design guideline prepared by FHWA (1988). Figure 10.42 shows that for a material of a given plasticity index, the permissible shear stress increases nearly ten-fold when the material is properly compacted.

Figure 10.43 shows the dramatic impact proper compaction can have on the erodibility of some types of soils. Materials sampled from the MRGO levee were tested at two compaction levels: low compactive effort and high compaction effort. The corresponding results speak volumes to the importance of compaction in earthen levees. The low-compaction sample was found to be very highly erodible, whereas the high-compaction sample exhibited low erodibility characteristics. High compaction effort is based on 90-95% relative compaction per the Modified Proctor test (ASTM D1557).

Figure 10.44 provides a summary from the pioneering erodibility work performed by Dr. Briaud et al. at Texas A & M University as part of these studies by (see Chapter 9). Soils that fell within the very high to high erodibility categories are prone to failure by overtopping. Soils that fell within the medium erodibility catergory fell in a transition zone, and soils that fell within the low to very low erodibility categories were shown to be resistant to erosion induced failure as a result of overtopping. These laboratory test results were well-validated by actual levee performance during Hurricane Katrina.

10.7 Observed Failure Modes During Hurricane Katrina

Table 10.5 presents a summary of the types of failure mechanisms of earthen levees that were observed in the greater New Orleans area flood defense system, including the ten locations identified above. In addition, the required design evaluations per the USACE and FEMA guidelines have also been summarized in this table.

Overtopping, jetting, internal erosion and piping, underseepage and piping, underseepage-induced instability, and lateral and semi-rotational foundation instability failure mechanisms were all observed in the greater New Orleans region during Hurricane Katrina. There were multiple locations where trees that were rooted within the levee zone had fallen over and may have contributed to the failure of the levee. Liquefaction may have been a partial contributor to some of the failures along the MRGO levee system. The forces associated with the breaking waves impacting the MRGO levee may have been sufficient to induce liquefaction in the relatively weak foundation materials.

Some notable attributes were observed to be associated with levees that performed well during the hurricane. These attributes included:

- Utilization of erosion-resistant soils for levee construction;
- Gradual soil/structure transition zones (rock gabions around concrete structures);
- Presence of low-lying swamp and vegetation on the outboard sides to dampen wind-waves; and
- Presence of rip-rap protection.

Some notable attributes were observed to be associated with a number of levees that performed poorly during the hurricane. These attributes included:

- Utilization of low erosion-resistant construction material;
- Transitions between different flood protection component types;
- Lack of surface slope protection for erosion-susceptible soil levees; and
- Abandoned channel backfills underlying levees.

10.8 Establishment of Design Criteria and Acceptable Performance

Varying degrees of levee reliability and performance are required based on the assets that the levees are protecting. Early in the last century, most U.S. levees protected agricultural farm lands, where the consequences of levee breaching and subsequent flooding resulted primarily in the loss of crops. With the growth of urban areas into lowlands adjacent to rivers and coastlines over the past two to three hundred years (the first urban levee was constructed in New Orleans in 1718), and especially over the past century, levees have become increasingly important defense mechanisms for industry, vital infrastructure elements such as drinking water, sewage, and electricity transmission, and, most importantly, large populous regions (cities and towns) and thus human life. Little guidance is provided by either the USACE or FEMA design criteria as to what are acceptable design standards for high-consequence urbanized areas vs. low-consequence agricultural areas.

The current approach to establishing design standards utilized by the USACE is conducting cost-benefit analyses. This financial/risk evaluation procedure analyzes the cost of achieving a certain level of protection and compares it with the recognized benefit associated with that level of protection. Unfortunately, the cost/benefit model used by the USACE for levee systems does not account for: (1) the loss of human life, (2) economic losses to cities, counties, and states as a result of a non-operational and non-functional revenue base (e.g.: businesses shut down due to damage and lack of utilities, lack of a work force to operate businesses, lack of a tax base due to the displaced residents, and lack of tourists, etc.), and (3) numerous other costs and losses known collectively as "secondary" and "tertiary" costs/losses associated with economic ripples that spread farther a field from the immediate locale of the disaster in question (as opposed to the more easily quantifiable primary or "direct" losses associated with system failure and consequent flooding.) These secondary and tertiary losses become increasingly important as the scope of a disastrous failure increases; these uncounted losses are most pronounced for full-blown "catastrophes."

This is unfortunate, as it results in systematic under-valuation of the likely benefits of investing effort and resources to prevent disasters before they occur. In the case of hurricane Katrina, estimates of "losses" due to the catastrophic flooding of approximately 85% of the greater New Orleans region vary significantly at the time of this writing, but most independent estimates are on the order of ~$100 to $200 billion, and some estimates range as high as $400 billion. A clear outlier is the recent estimate of damages proffered by the IPET study's Draft Final Report (IPET; June 1, 2006) which estimated these damages at approximately $25 billion. This was largely a function of the procedures which the USACE is required to employ in making potential loss projections; these systematically undercount expected losses, and they also undercounted actual losses in the wake of Katrina. This systematic under-evaluation of projected (and real) losses provides a poor basis for subsequent decision-makers (e.g. federal and local government) to base decisions regarding appropriate allocation of resources to defend against risk and threats.

10.8.1 USACE Risk Management Approach

In response to budget constraints, increased situations requiring cost-sharing, and general public concern for the performance and reliability of completed projects, the USACE has evaluated the use and methodology of risk-based analyses (especially as related to the geotechnical components of these projects). A seminar was convened in 1983 by the USACE in order to "incorporate more information into the safety assessment [of projects] than [traditional] factor of safety methods." A more recent evaluation was undertaken, and the results of this effort are presented in Engineering Technical Letter (ETL) 1110-2-556, published on May 28,

1999. This study recognized that there are inherent uncertainties associated with infrastructure problems and that the total effect of risk and uncertainty on a project's economic viability should be examined in order for "conscious decisions" to be made reflecting "explicit trade-offs between risk and cost."

According to ETL 1110-2-556, major sources of uncertainty that require evaluation include the following:

- Uncertainty in loadings;
- Uncertainty in engineering analysis parameters;
- Uncertainty in analytical models (model bias);
- Uncertainty in performance;
- Conversion of empirically-derived performance modes;
- Frequency and magnitude of physical changes or failure events; and
- Conditions of unseen features.

ETL 1110-2-556 identifies special situations uniquely applicable to geotechnical problems that result in uncertainties with large magnitudes:

- Natural earthen materials generally exhibit high variability in composition and engineering properties;
- Engineering characteristics of soils can exhibit high variability due to composition, deposition, sampling, and field & laboratory testing procedures;
- Engineering analyses can be performed assuming total stress (excluding the effects of groundwater) or effective stress (including the effects of groundwater). As a result, groundwater uncertainties may either be included or excluded in the analyses;
- Consideration of spatial correlation of soil properties is required due to the variability of deposition history; and
- The spatial scale of the project (as much as tens of miles long for levees) requires "sectioning" of the system into subcomponents.

These uncertainty factors can result in very large ranges and broad distributions for parameter bounds. For example, a mean soil shear strength value, determined based on a subsurface field sampling and laboratory testing, which is used to evaluate the stability of levee slopes may naturally vary between ± 30% of the mean value to as much as ± 75% of the mean value. These broad distributions significantly impact the reliability of the resulting calculated answer.

The report summarized recommended target reliabilities for expected performance levels. These target reliabilities are presented in Table 10.6.

The approximate median Factors of Safety associated with the established expected performance levels were added to the target reliability indices presented in ETL 1110-2-556 according to the following formula:

$$F.S._{50} = e^{(\beta \sigma \ln RS)} \qquad \text{[Equation 10.2]}$$

In this formulation, β is the safety index, R is the capacity, and S is the loading/demand. A fairly typical coefficient of variation of 30% is assumed in both the loading and capacities, and a lognormal distribution for the system and the loads are assumed.

It is interesting to note that this approach had not yet been applied to the New Orleans regional flood defense system. Instead, the system had been designed using more "traditional" approaches, including the common use of a required Factor of Safety of FS $\geq$ 1.3 for stability analyses associated with the short-term ("transient") conditions produced by hurricane-induced storm surge, winds and waves.

If this is then compared with the projections of Table 10.6, the expected performance for these design criteria would be anticipated to be "unsatisfactory", with an approximately estimated 7% probability of failure. It is, of course, a matter of judgement as to how many individual segments and intersections comprise the overall New Orleans regional flood protection system, and also how accurately the "typical" coefficients of variation for both loadings and resistances characterize each of these, but the overall accuracy of the projection of expected performance based on the simplified estimates of Table 10.6 could be argued to be well-justified by the multiple and catastrophic failures of the flood protection system that occurred during hurricane Katrina.

10.8.2 Other Risk-Based Approaches

Alternative approaches to establishing design levels exist. John Christian (2004) summarized studies that back-calculated the annual probability of failure based on failures of actual engineered systems by Baecher et al. The lives lost or financial losses associated with the failures were plotted against the back-calculated annual probabilities of failure. This plot provides a mechanism by which to ascertain the "targeted" level of performance (the targeted level of reliability) of a given engineered structure, based on historic practice in a number of diverse fields. The resulting plot is presented in Figure 10.45.

Additional examples include both the Netherlands and Hong Kong, which have risk-based decision making tools (Figure 10.46) as part of their planning process to establish acceptable levels of safety for engineered systems protecting significant populations based on the expected number of fatalities, as well as expected financial "losses" (Christian 2004). Failures that might impact large populations and may result in large numbers of fatalities (e.g.: greater than 1,000) are required to have very low annual probabilities of failure.

Using the risk management planning relationship (Figure 10.46) developed by the Hong Kong Government Planning Department as an example, a proposed engineered system that has the potential to result in 1,000 fatalities would have an acceptable risk (based on an annual frequency of occurrence) of 10^{-8}, the range over which the principle "As Low As Reasonably Prudent" (ALARP) reliability level is recommended varies from annualized Pf_{pa} of 10^{-8} to 10^{-6}, and a risk with a Pf_{pa} of less than 10^{-6} is considered unacceptable for any case.

Table 10.7 presents a summary of calculated Annual Return Periods (in years), based on the probability of occurrence limits recommended by the Hong Kong Government Planning Department. This example highlights the sociological decision made in Hong Kong that high consequence events should occur very infrequently, with an annual return period of 1 million years! Although this may not be realistic due to natural and anthropogenic uncertainties, the premise of varying acceptable risk as a function of consequences is rational and feasible.

The Dutch have well-developed risk-based approaches for targeting the reliability of flood protection systems. Like the southern Louisiana region, the Dutch face two distinct types of flooding risk; river floods and flooding from catastrophic North Sea storms and their associated storm surge, waves and winds. The Dutch lost a large fraction of their nation to ocean storm flooding in the mid 1950s, and determined to develop rationally risk-based flood defense systems to prevent recurrence of similar catastrophic flooding in the future. The levels of targeted flood defense levels, in terms of return periods, are interesting in contrast to the approximately 200 to 300-year recurrence level that was nominally targeted for New Orleans. The Dutch use a recurrence level of flood loading on the order of 1,000-10,000 years for river floods of populous areas (major towns and cities), and 10,000 years for ocean storms. For less populated (largely agricultural) areas, recurrence levels targeted for flood protection design are on the order of 500-1,000 years.

All of these targeted levels of protection greatly exceed the targeted levels for the New Orleans regional flood protection systems.

Another good way to look at targeted levels of reliability for flood protection is to re-examine Figure 10.45. This figure is re-plotted as Figure 10.47, with a red cross-hatched region showing approximately the level of reliability associated with the New Orleans regional flood protection system at the time of Katrina's arrival. Based on the studies of the ILIT team, it is inferred that the New Orleans flood defense system would have been expected to fail catastrophically about once every 50 to 100 years. The consequences of the failures during Katrina were on the order of 1,300 lives lost, and/or about \$100 to \$200 billion in economic losses.

A red dashed line indicating the approximate levels of reliability targeted by current U.S. practice with regard to dams is also shown in Figure 10.47. It is interesting to note that current U.S. practice for dams would have called for approximately three orders of magnitude higher level of reliability than that which was apparently provided by the New Orleans regional flood protection system (a factor of approximately 1,000 times higher reliability or assured safety.) That is largely because "dams" are generally assumed to be associated with large consequences in the event of failure. Few U.S. dams, however, have likely consequences of failure larger than those associated with the failure of the New Orleans regional flood defense systems during hurricane Katrina. This would appear to suggest that the flood defense systems for major metropolitan areas should be engineered to have targeted levels of reliability on a par with those commonly targeted for U.S. dams.

10.9 Brief Comments on Drainage and Pumping

As discussed in Chapter 4, the New Orleans area is situated below sea level and significantly below the river level in the Mississippi River. The proper functioning of the drainage and pumping system is critical to ensure the population of New Orleans is not inundated by groundwater, rainwater, or floodwater. Essentially every drop of rain that falls into New Orleans has to be pumped out, because most of the developed metropolitan and suburban regions are below mean sea level. In addition, underseepage constantly passes in beneath the perimeter levee systems, and has to be pumped out as well.

Accordingly, despite the potential critical flood-fighting contribution of the drainage and pumping systems, "the pumping stations have not been considered to be part of the hurricane protection system except in a few instances where the buildings are structural part of a levee or floodwall" (IPET, 2006). Based on the observed poor performance of the pumping systems during Hurricanes Katrina and Rita, this perception of the drainage and pumping system must be changed in order to provide more reliable system performance in the future.

There are about 80 pumping stations (IPET, 2006) in the four study areas, and a majority of these pumping stations are more than over 50 to 100 years old. The pump stations are powered by an assortment of electrical supply systems, and most of the older pump stations are powered by antiquated 25HZ power generation facilities constructed in the late 1910's and early 1920's. As a result of Hurricane Katrina, approximately one-third of the total system pumping capacity within the study areas was lost after the passing of the hurricane. Only 16% of the pumping stations were fully operational during the hurricane.

An extensive review of the performance of the pumping and drainage systems in the Jefferson, Orleans, Plaquemines, and St. Bernard parishes was beyond the scope of our ILIT studies. Such an evaluation was performed by IPET, however, and the results are presented in their draft final report (IPET; June 1, 2006). An overview map of the parishes and regions studied by this element of the IPET studies is presented in Figure 10.48.

Figure 10.49 presents a detailed view of pump stations within Jefferson parish. The maximum pumping capacity within Jefferson parish is 48,460 cubic feet per second (cfs) by a total of 27 pumping stations that drain an area of 73,500 acres (IPET, 2006). Figure 10.50 presents a detailed map of pump stations within Orleans parish. The maximum pumping capacity within Orleans parish is 48,900 cfs from a total of 23 pumping stations that drain an area of 60,000 acres (IPET, 2006). Figure 10.51 presents a detailed map of pump stations within Plaquemines parish. The maximum pumping capacity within Plaquemines parish is 12,065 cfs from a total of 21 pump stations that drain an area of 55,000 acres. Figure 10.52 presents a detailed map of pump stations within St. Bernard parish. The maximum pumping capacity within St. Bernard parish is 7,000 cfs from a total of 8 pump stations that drain an area of 17,620 acres.

IPET identified four major failure modes for pump station malfunctions during Hurricane Katrina. These consisted of (1) loss of operational staff as a result of evacuation orders at pumping stations that required manual operation, (2) loss of potable water to lubricate and cool pumps during operation as a result of municipal water distribution system malfunction, (3) loss of electricity to power the pumping station, and (4) shut-down or disruption of the pump facilities as a result of flooding. Pumping failure from evacuations, flooding, loss of electricity, and loss of lubrication (potable water) accounted for 46%, 26%, 8%, and 4% loss of total pumping capacity, respectively (Figure 10.53). These failures and breakdowns do not reflect multiple failure modes, such as a pump station being evacuated, only later to be overwhelmed as a result of breaches in the flood defense system (i.e. a break in the drainage canal wall such as at 17^{th} Street Canal).

In addition to these four failure modes, there are three other significant pump and drainage system-based failure modes that impacted the proper performance of the pump and drainage system during Hurricane Katrina. These failure modes are (1) reverse flow (Figure 10.54) through the pumping stations due to inadequate pump discharge elevation clearance (and a lack of internal reverse-flow protection), (2) loss of drainage capabilities as a result of breach of the drainage canal [such as within the 17^{th} Street Drainage Canal, the Orleans Canal, and the London Avenue Drainage Canal as shown in Figure 10.55], and (3) lack of sufficient temporary storage capacity [such as within St. Bernard parish between the interior levee and the MRGO exterior levee], where discharged water is ponded behind the MRGO levee until the gates of the two control structures can be opened to allow the stored water to drain into Lake Borgne. It is interesting to note that water discharged from the Lower 9^{th} Ward must drain through approximately 10.5 miles of bayou (Figure 10.55) until it is finally discharged from the protected area through the control structure gates, which are not configured to allow discharge of water during storm events, instead of having the pump station discharge directly into the much closer IHNC about 650 feet from the pump station.

Mitigation of these systemic flaws and performance inhibitors will require significant effort, and should prompt a full re-evaluation of the pumping and unwatering system configuration and details in order to develop an improved system that can function reliably both during and after major storm events.

10.10 Conclusions

Hurricane Katrina resulted in the catastrophic flooding of the greater New Orleans area. Although the magnitude of the storm surge that overwhelmed the levee flood defense system was greater than the capacity of the system, the extent of the devastating damage could have been greatly minimized if the system had been robustly designed. There were many miles of earthen levees that were significantly overtopped, but did not breach catastrophically. These levees that did not breach were only overtopped for a few hours' duration and the quantity of water that did flow over the levees could have been pumped out of the protected area utilizing the existing drainage network and pump infrastructure (see Chapter 4). The levees that were not able to withstand overtopping breached catastrophically, allowing the full magnitude of the storm surge to overwhelm the protected area.

Design guidelines need to be updated to ensure the design and construction of robust levee systems. All failure mechanisms must be acknowledged and included in the design evaluation. All levees should be designed to withstand overtopping. Material selection and compaction are critical components to ensure adequate performance and appropriate specifications for material selection and compaction should be developed and should be incorporated into the design guidelines.

The current design guidelines sponsored by both the USACE and FEMA assume that overtopping does not occur and does not require safety in the event of overtopping of the levees.

A probabilistic approach should be utilized to determine the appropriate factor of safety for the design of these levee systems. Accounting for uncertainties in demands on the system (height of storm surges, wave impacts, etc.) as well as uncertainties in the capacity of the levee system (erosion resistance, foundation stability, etc.) must be included in the safety evaluation of the levee system.

The current design guidelines sponsored by both the USACE and FEMA are based on deterministic factor of safety levels that do not account for a broad range of uncertainties nor do they account for mechanisms to ensure an appropriate level of safety based on the consequences of failure.

10.11 References

Christian, John T. (2004), “Geotechnical Engineering Reliability: How well do we know what we are doing?” *J. of Geotechnical and Geoenvironmental Engineering*, Vol. 130, No. 10, October 1, 2004.

Google Earth, (2005), earth.google.com, <accessed: March 21, 2006>

Hong Kong Government Planning Department. (1994). “Potentially hazardous installations.” *Hong Kong planning standards and guidelines,* Chapter 11, Hong Kong.

Interagency Performance Evaluation Task Force, (2006), “Performance Evaluation, Status and Interim Results, Report 2 of a Series, Performance Evaluation of the New Orleans and Southeast Louisiana Hurricane Protection System,” March 10, 2006.

IPET, (2006), “Performance Evaluation of the New Orleans and Southwest Louisiana Hurricane Protection System, Draft Final Report of the Interagency Performance Evaluation Task Force, Volume VI – The Performance – Interior Drainage and Pumping,” available online: https://ipet.wes.army.mil/, date accessed: June 1, 2006.

Munson, Bruce R., Donald F. Young, and Theodore H. Okiishi, (1990), “Fundamentals of Fluid Dynamics – Third Edition,” John Wiley and Sons, Toronto, Canada, ISBN 0-471-17024-0.

Seed, Raymond B., et al., “Preliminary Report on the Performance of the New Orleans Levee Systems in Hurricane Katrina on August 29, 2005,” Report No. UCB/CITRIS - 05/01, November 17, 2005.

Topozone.com, (2006), www.topozone.com, <accessed March 21, 2006>

United States Army Corps of Engineers, (2006), MRGO Levee Reconnaissance Handout, “MRGO Center Profile Line,” April 8, 2006.

United States Army Corps of Engineers, (2002), EM 1110-2-1100 (Part III), “Coastal Engineering Manual Part III,” April 30, 2002.

United States Army Corps of Engineers, (2002), EM 1110-2-1100 (Part VI), "Coastal Engineering Manual Part VI," February 28, 2005.

United States Army Corps of Engineers, (2003), EM 1110-2-1902, "Engineering and Design – Slope Stability," October 31, 2003.

United States Army Corps of Engineers, (1999), "Risk-Based Analysis in Geotechnical Engineering for Support of Planning Studies," ETL 1110-2-556, May 28, 1999.

United States Army Corps of Engineers, (1999), "Channel Rehabilitation: Processes, Design and Implementation," July 1999.

United States Army Corps of Engineers, (1990), EM 1110-2-1904, "Engineering and Design – Settlement Analyses," September 30, 1990.

United States Army Corps of Engineers, (2000), EM 1110-2-1913, "Engineering and Design – Design and Construction of Levees," April 30, 2000.

United States Army Corps of Engineers, (1989), EM 1110-2-2502, "Engineering and Design – Retaining and Floodwalls," September 29, 1989.

United States Army Corps of Engineers, (1988), "Mississippi River-Gulf Outlet, St. Bernard Parish, La., Bank Erosion, Reconnaissance Report," February 1988.

United States Army Corps of Engineers, (1983), "Proceedings, Seminar on Probabilistic Methods in Geotechnical Engineering," Miscellaneous Paper GL-83-26, September 1983.

United States Army Corps of Engineers, (1966), "Lake Pontchartrain, LA. and Vicinity, Chalmette Area Plan, Design Memorandum No. 3, General Design," November, 1966.

United States Code of Federal Regulations, 44CFR65.10, October 1, 2005.

Versteeg, M. (1987) "External Safety Policy in the Netherlands: An approach to risk management," *J. Hazard. Mater.*, 17, 215-221.

Table 10.1 Summary of Major Levee Design Steps

Step	Procedure
1	Conduct geologic study based on a thorough review of available data including analysis of aerial photographs. Initiate preliminary subsurface explorations.
2	Analyze preliminary exploration data and from this analysis establish preliminary soil profiles, borrow locations, and embankment sections.
3	Initiate final exploration to provide: a. Additional information on soil profiles b. Undisturbed strengths on foundation materials c. More detailed information on borrow areas and other required excavations
4	Using the information obtained in Step 3: a. Determine both embankment and foundation soil parameters and refine preliminary sections where needed, noting all possible problem areas. b. Compute rough quantities of suitable material and refine borrow area locations.
5	Divide the entire levee into reaches of similar foundation conditions, embankment height, and fill material and assign a typical trial section to each reach.
6	Analyze each trial section as needed for: a. Underseepage and through seepage. b. Slope stability. c. Settlement. d. Trafficability of the levee surface.
7	Design special treatment to preclude any problems as determined from Step 6. Determine surfacing requirements for the levee based on its expected future use.
8	Based on the results of Step 7, establish final sections for each reach.
9	Compute final fill quantities needed; determine final borrow area locations.
10	Design embankment slope protection.

Table 10.2 Permissible canal velocities with average flow depth of 3 feet

Material	Clear water, no detrius (ft/s)	Water transporting colloidal silts (ft/s)	Equivalent Shear Stress[1] (lb/ft^2)
Fine sand (noncolloidal)	1.5	2.5	2.4 - 6.3
Sandy loam (noncolloidal)	1.75	2.5	3.1 - 6.3
Silt loam (noncolloidal)	2	3	4.0 - 9.0
Alluvial silt (noncolloidal)	2	3.5	4.0 - 12.3
Ordinary firm loam	2.5	3.5	6.3 - 12.3
Fine gravel	2.5	5	6.3 - 25.0
Stiff clay	3.75	5	14.1 - 25.0
Alluvial silt (colloidal)	3.75	5	14.1 - 25.0
Coarse gravel (noncolloidal)	4	6	16.0 - 36.0
Shales and hardpans	6	6	36.0

[1]Assuming a roughness constant equal to 1 and fluid consisting of seawater.

Table 10.3: Performance summary for selected levee locations.

Location	Design Water Elevation (ft)[1]	Maximum Storm Surge Elevation (ft)[2]	Overtopping	Immediate Post-Hurricane Levee Condition[2]
1 - Lakefront Airport	[13.5] 11.8	12	Minor	Poor
2 - Jahncke Pump Station Outfall	[14.5] 12.8	12	Minor	Adequate
3 - Eastern Perimeter of New Orleans East	[14.5] 12.4	~18?	Moderate to Major	Very Good
4 - Southeast Corner of New Orleans East	[19.0] 13.0	~18?	Moderate to Major	Poor
5 - Entergy Michoud Generating Plant	[15.0] 13.2	16	Moderate to Major	Good
6 - IWW/MRGO Southern Levee	[14.0] 13.2	16	Moderate to Major	Good
7 - Bayou Bienvenue Control Structure	[17.5] 13.2	18	Moderate to Major	Good/Poor
8 - Mississippi River Gulf Outlet	[17.5] 12.7 (~10[3])	17-22	Major	Poor
9 - Bayou Dupre Control Structure	[17.5] 12.7 (~10[3])	17-22	Major	Poor
10 - St. Bernard Parish Interior Levee (Forty Arpent Levee)	[8] ~6[4] (~3[3])	Not Established	Major	Very Good

[1]Elevations converted from NGVD29 elevation (in brackets) to equivalent NAVD88(2004.65) elevation, from IPET (2006)

[2]Based on NAVD88(2004.65) vertical datum. From IPET (2006)

[3]Elevation at the time of Hurricane Katrina was below the design elevation

[4]A conversion between the original design elevation from the NGVD29 to the new NAVD88(2004.65) elevation was not available from IPET

[2]This is an assessment of conditions immediately after the hurricane, before significant repair and reconstruction.

Table 10.4 Summary of sampling locations for laboratory testing

Site No.	Description	Latitude (°N)	Longitude (°W)	Erosion Performance
1	Levee east of Hwy 11 and North of Hwy 90	30.0895	89.8587	Good
2	Entergy Powerplant	30.0065	89.9389	Good
3	MRGO North Control Structure (North)	29.9996	89.9170	Good
4	MRGO Levee (northern section)	Not Established		Poor
5	MRGO Levee (middle section)	Not Established		Poor
6	MRGO Levee (southern section)	Not Established		Poor
7	St. Bernard Parish South	29.8769	89.7818	Good
8	St. Bernard Parish North	29.9558	89.9466	Good
9	Lakefront Airport Transition Levee	30.03344	90.026	Moderate
10	Hayne Blvd	30.05908	89.96697	Good
11	Hayne Blvd and Paris Road (Beach)	30.07577	89.94467	Not Applicable
12	Orleans East Southeast RR Transition	30.06156	89.83352	Poor
13	Orleans East Southeast Corner	30.04481	89.83089	Poor
14	Intracoastal Waterway North (New Levee)	30.03542	89.85399	Unknown
15	Intracoastal Waterway North (Remaining Levee)	30.02707	89.87448	Poor
16	Levee west of Entergy Plant	30.00465	89.95062	Good
17	St. Bernard Parish (Middle)	29.92541	89.8948	Good

Note: Geographical coordinates based on WGS84 datum.

(Table 10.4, Continued) Summary of sampling locations for laboratory testing

Site No.	Description	Performance	EFA Erosion Susceptibility Determination
1	Levee east of Hwy 11 and North of Hwy 90	Good	Low (IV)
2	Entergy Powerplant	Good	Low to Very Low (IV-V)
3	MRGO North Control Structure (North)	Good	Low to Very Low (IV-V)
4	MRGO Levee (northern section)	Poor	High (II)
5	MRGO Levee (middle section)	Poor	High (II)
6	MRGO Levee (southern section)	Poor	High (II)
7	St. Bernard Parish South	Good	Medium (III)
8	St. Bernard Parish North	Good	Medium (III)
9	*Lakefront Airport Transition Levee*	*Moderate*	*Not Tested*
10	*Hayne Blvd*	*Good*	*Not Tested*
11	Hayne Blvd and Paris Road (Beach)	Not Applicable	Not Applicable
12	Orleans East Southeast RR Transition	Poor	High to Medium (II-III)
13	*Orleans East Southeast Corner*	*Poor*	*Not Tested*
14	*Intracoastal Waterway North (New Levee)*	*Unknown*	*Not Tested*
15	Intracoastal Waterway North (Remaining Levee)	Poor	Very High to High (I-II)
16	*Levee west of Entergy Plant*	*Good*	*Not Tested*
17	*St. Bernard Parish (Middle)*	*Good*	*Not Tested*

Table 10.5 Levee Failure Mechanisms

Failure Mechanism	USACE Guidelines	FEMA Guidelines	Observed in Greater New Orleans Area
Overtopping	Not allowed	Not allowed	Yes
Jetting	Not allowed	Not allowed	Yes
Internal Erosion and Piping	Design criteria provided	Analyses required	Yes
Lateral Surface Erosion	Protection required	Protection required	Possibly
Wave Impacts	Protection required	Protection required	Yes
Structural Impacts	Not addressed	Not addressed	Yes
Slope Failures	Design criteria provided	Analyses required	Yes
Sliding	Design criteria provided	Analyses required	Yes
Underseepage	Design criteria provided	Analyses required	Yes
Liquefaction	Not directly addressed	Not directly addressed	Possibly
Bottom Heave/Blowout	Not directly addressed	Not directly addressed	Yes

Table 10.6 Target Reliability Indices

Expected Performance Level	Beta (β)	Probability of Unsatisfactory Performance	*Approximate Median Factor of Safety[1] ($F.S._{50}$)*
High	5.0	0.0000003	*2.5*
Good	4.0	0.00003	*2.1*
Above average	3.0	0.001	*1.7*
Below average	2.5	0.006	*1.6*
Poor	2.0	0.023	*1.4*
Unsatisfactory	1.5	0.07	*1.3*
Hazardous	1.0	0.16	*1.2*

Table 10.7 Risk Levels for a System with the Potential for 1,000 Fatalities

Risk Level	Pf_{pa}	Annual Return Period (yrs)
Acceptable	$<10^{-8}$	>100,000,000
ALARP	10^{-6} to 10^{-8}	1,000,000 to 100,000,000
Unacceptable	$>10^{-6}$	< 1,000,000

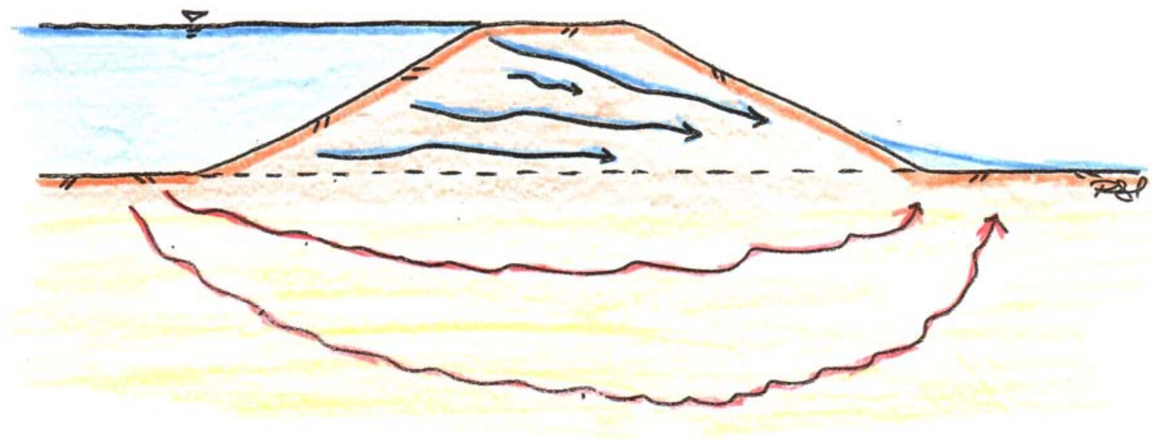


Figure 10.1: Underseepage in highly permeable underlying foundation materials (red lines) can result in the catastrophic failure of the levee in that once the foundation materials have been eroded, the levee (which may be completely undamaged) has no underlying support and falls into the resulting void and essentially washes away.

Internal erosion and piping (blue lines) occurs in levee materials that have high permeabilities (such as sand and gravel) and allow for water to rapidly flow from high pressure areas to low pressure areas. As the water flows through the levee, smaller/finer soil particles are "washed" out of the levee resulting in the internal erosion of the levee. Enough internal erosion of the levee can lead to the collapse and subsequent "wash-out" of the levee.

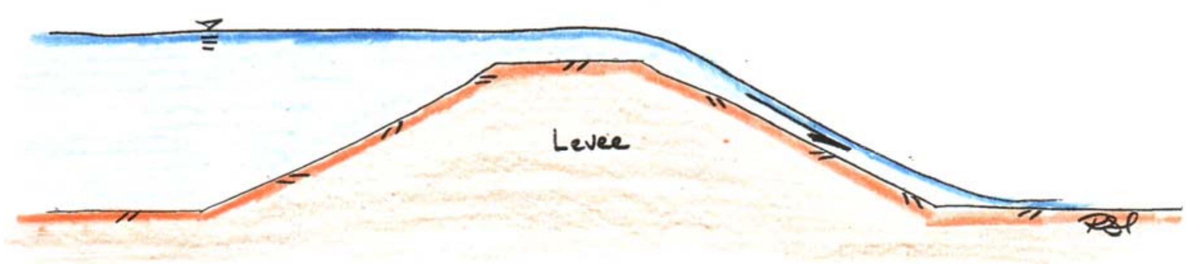


Figure 10.2: Overtopping occurs when the water level on the outboard side of the levee exceeds the crest elevation of the levee. The inboard side of the levee acts as a spillway for the overtopping water and damage is inflicted on the levee as a result of water scour.

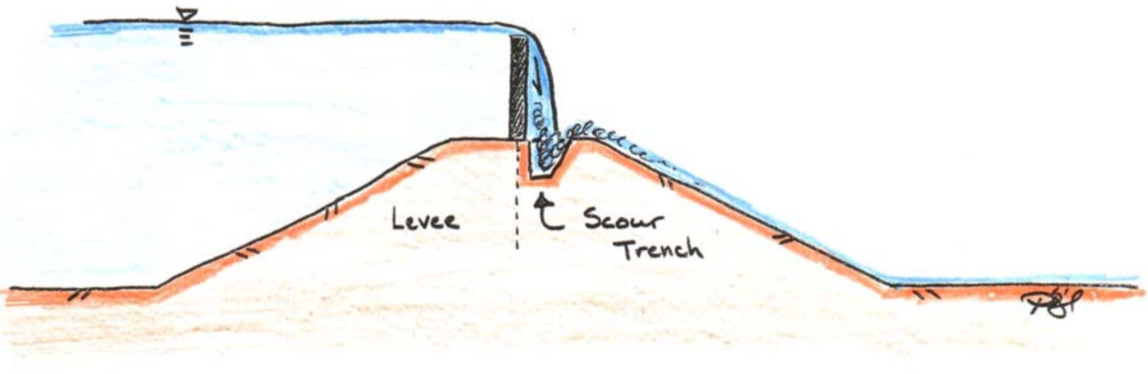


Figure 10.3: Jetting occurs when the water level on the outboard side of the levee exceeds the top of wall elevation for structural walls that are founded within the earthen levee. Unlike overtopping of a conventional earthen levee, the floodwall acts as a weir and water impacts the levee in a concentrated stream that is much more energy intensive than conventional overtopping.

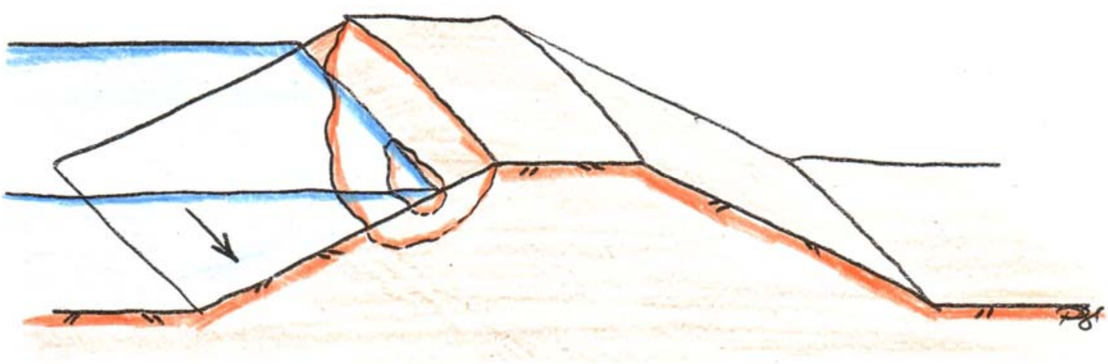
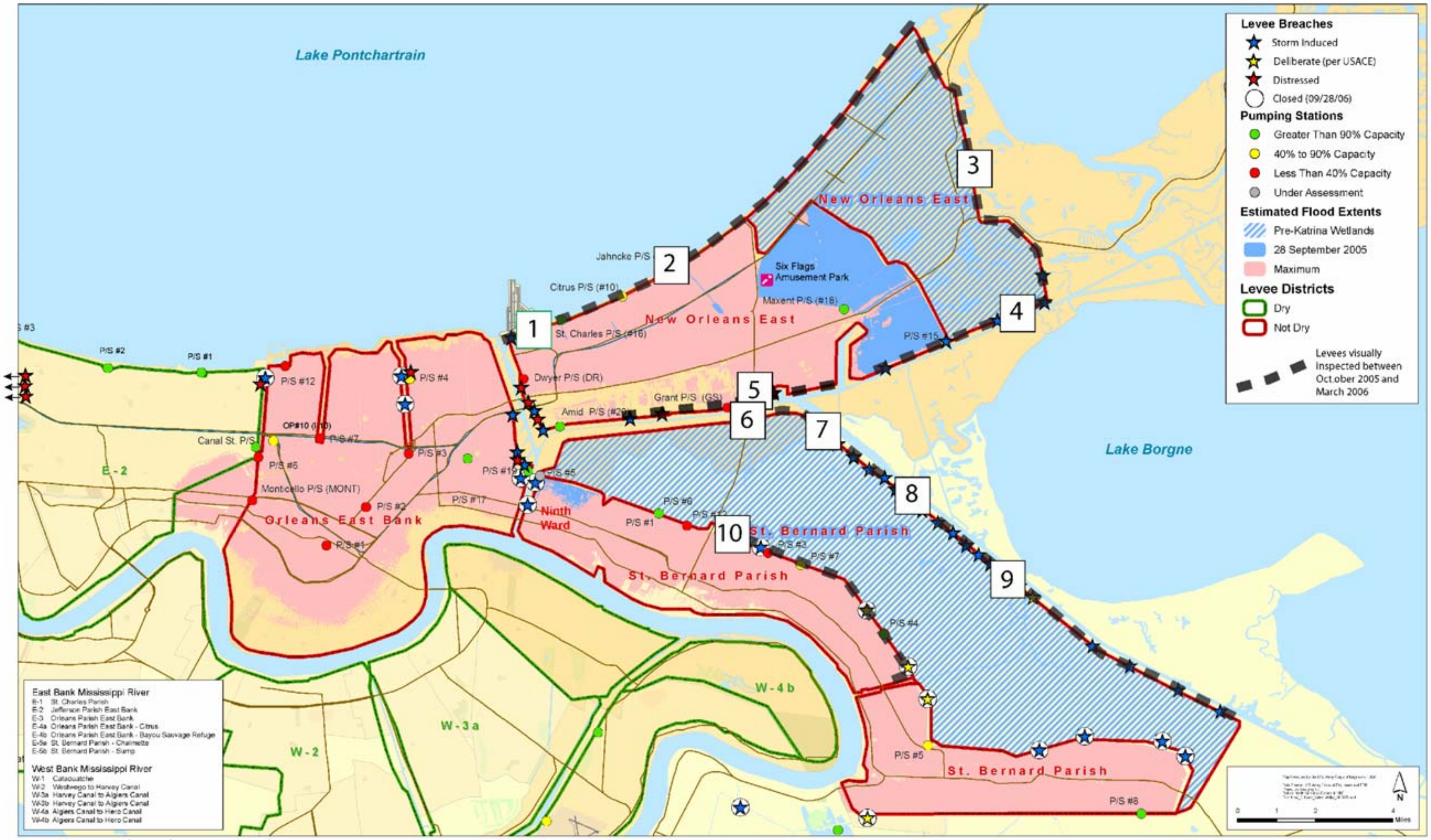


Figure 10.4: Surface erosion generally occurs on the outboard side of the levee and is the result of water flowing past the levee face. If the imposed shear stress from the water abrading against the soil levee face is high enough, soil scour occurs and the integrity of the overall levee is significantly reduced.

Figure 10.5: Wave impacts can cause significant erosion to levee faces. Wave-induced erosion consists of run-up (sloshing up and down of water as a result of staggered wave arrival) and "mini-jetting" when the crest of the waves breaks on the levee face.



Source: Modified after USACE, 2005

Figure 10.6: Map showing the extents of the visual reconnaissance (dashed black line) of the earthen levee systems performed between October 2005 and March 2006. Locations of notable performance are identified in the numbered boxes.

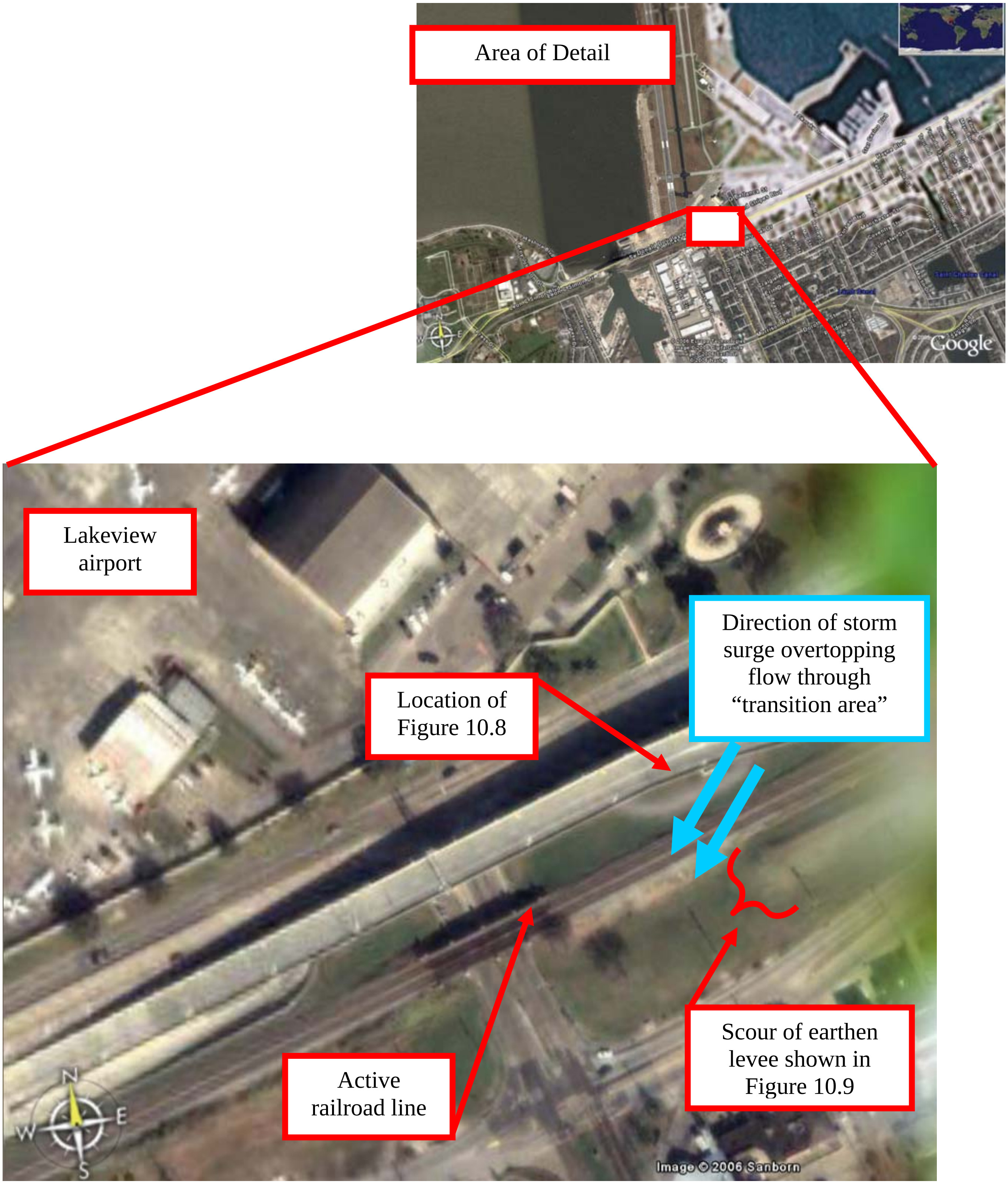
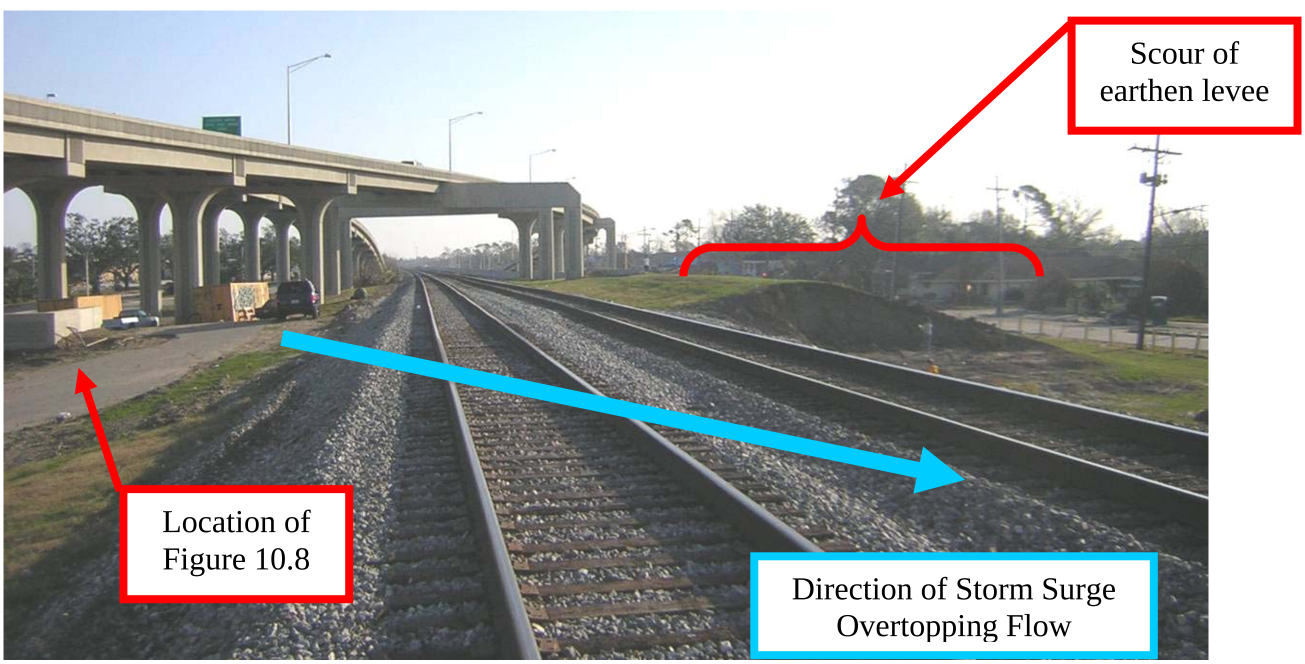


Image from Google Earth, 2006

Figure 10.7: Storm-surge induced overtopping traveled through the granular gravel ballast for the railroad line and eroded the railroad line embankment, which served as a transition levee between the concrete floodwall and the earthen levee shown in Figure 10.8.

Photograph by Rune Storesund

Figure 10.8: Lakefront levee near the Lakefront Airport at location 1 (as indicated on Figure 10.7) where overtopping occurred and significant scour around the floodwall was observed.



Photograph by Rune Storesund

Figure 10.9: Significant erosion was observed on the levee behind the floodwall shown in Figure 10.8. The storm surge overtopped the floodwall and railroad ballast and failed the earthen levee.

Photograph by Rune Storesund

Figure 10.10: Lakefront levee at location 2 (as indicated on Figure 10.6) where minor overtopping occurred. These levees performed well and only minor, surficial damage was observed.

Photograph by Rune Storesund

Figure 10.11: Observed scour at the Jahncke Pump Station outfall structure (location 2 as indicated on Figure 10.6). Scour was limited to areas of soil-structure interfaces, and no full breach occurred.

Photograph by Rune Storesund

Figure 10.12: Condition of levees east of HWY 11 (location 3 on Figure 10.6) in October 2005. These levees performed exceptionally well and were not eroded during Hurricanes Katrina or Rita.

Photograph by Rune Storesund

Figure 10.13: Levee rehabilitation work (near location 3 on Figure 10.6) after Hurricane Katrina included reinforcement and protection of soil-structure interactions with rock-gabion transition zones.

Photograph by Rune Storesund

Figure 10.14: Zones of earthen levees at the southeast corner of the New Orleans East polder (location 4 on Figure 10.6) were washed out during Hurricane Katrina allowing water to rush in.

Photograph by Rune Storesund

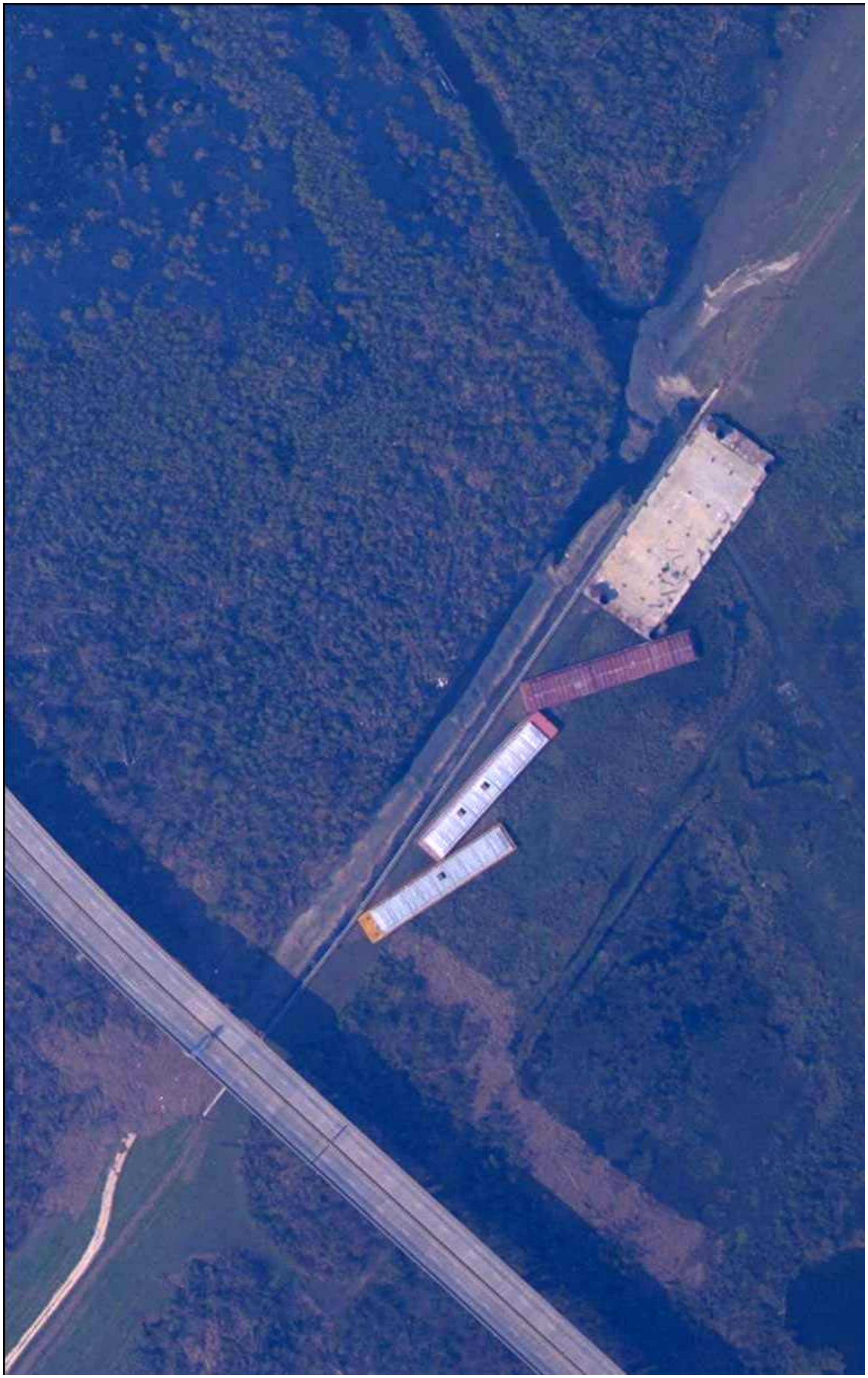
Figure 10.15: Levee rehabilitation work (location 4 on Figure 10.6) post Hurricane Katrina. The "semicompaction" construction approach was employed, utilizing earth moving equipment to also compact the placed material as it transported borrow materials to the construction site.

Photograph courtesy the Entergy Corporation

Figure 10.16: Photographs (location 5 on Figure 10.6) captured by a security camera at the Entergy Michoud Generating Plant beneath Route 47, on the IWW/MRGO show active overtopping during Hurricane Katrina.

Photograph by Rune Storesund

Figure 10.17: Post Hurricane Katrina view of the earthen levee shown in Figure 10.16. Only minor damage on the protected side was observed, with the majority of the damage a result of wave reflection from the bridge abutment at the right-hand side of the picture.



Photograph courtesy ngs.woc.noaa.gov/Katrina/KATRINA0000.HTM

Figure 10.18: The presence of a concrete floodwall beneath the Hwy 47/Parish Road Bridge acted as a weir during the overtopping stages of the storm and "sucked" in nearby barges. Scour can be observed behind the concrete floodwall, but there is only minor scour damage visible at the earthen levee/concrete floodwall transition. Overall, this system performed well (also considering the impact associated with the barges).

Photograph courtesy Lee Wooten

Figure 10.19: Woody debris and steel barges "washed up" on the southern ICWW/MRGO levee just west of Route 47 (location 6 on Figure 10.6). This is a side view from the aerial photograph presented in Figure 9.24.

Photograph courtesy Francisco Silva-Tulla

Figure 10.20: A gas processing barge "washed up" on the southern IWW/MRGO levee just east of Route 47 (location 6 on Figure 10.6). This levee was also overtopped and was not significantly damaged by the barge impact.

Photograph courtesy Les Harder

Figure 10.21: Aerial photograph of the Bayou Bienvenue Control Structure (location 7 on Figure 10.6). The northwestern half of the control structure levee system performed extremely well (withstanding significant impact loads from the steel barge), while the southeastern portion suffered severe erosion.

Photograph by Rune Storesund

Figure 10.22: This flood control gate acted like a weir as water overtopped the structure during the storm surge. Significant scour and erosion was observed around the flood control gate structure.

Photograph by Rune Storesund

Figure10.23: By March of 2006, after Hurricane Katrina, splash pads had been installed at this flood control gate to mitigate erosion impacts if any future overtopping of the flood protection system occurs. This photograph was taken at the same location as Figure 10.22.

Photograph by Rune Storesund

Figure 10.24: The northern side of the Bayou Bienvenue control structure has beenrepaired, and then heavily reinforced with new rip-rap transported to New Orleans from Kentucky. This photograph was taken about 6 months after Hurricane Katrina and the barge, which can be seen in Figure 10.21, has been removed.

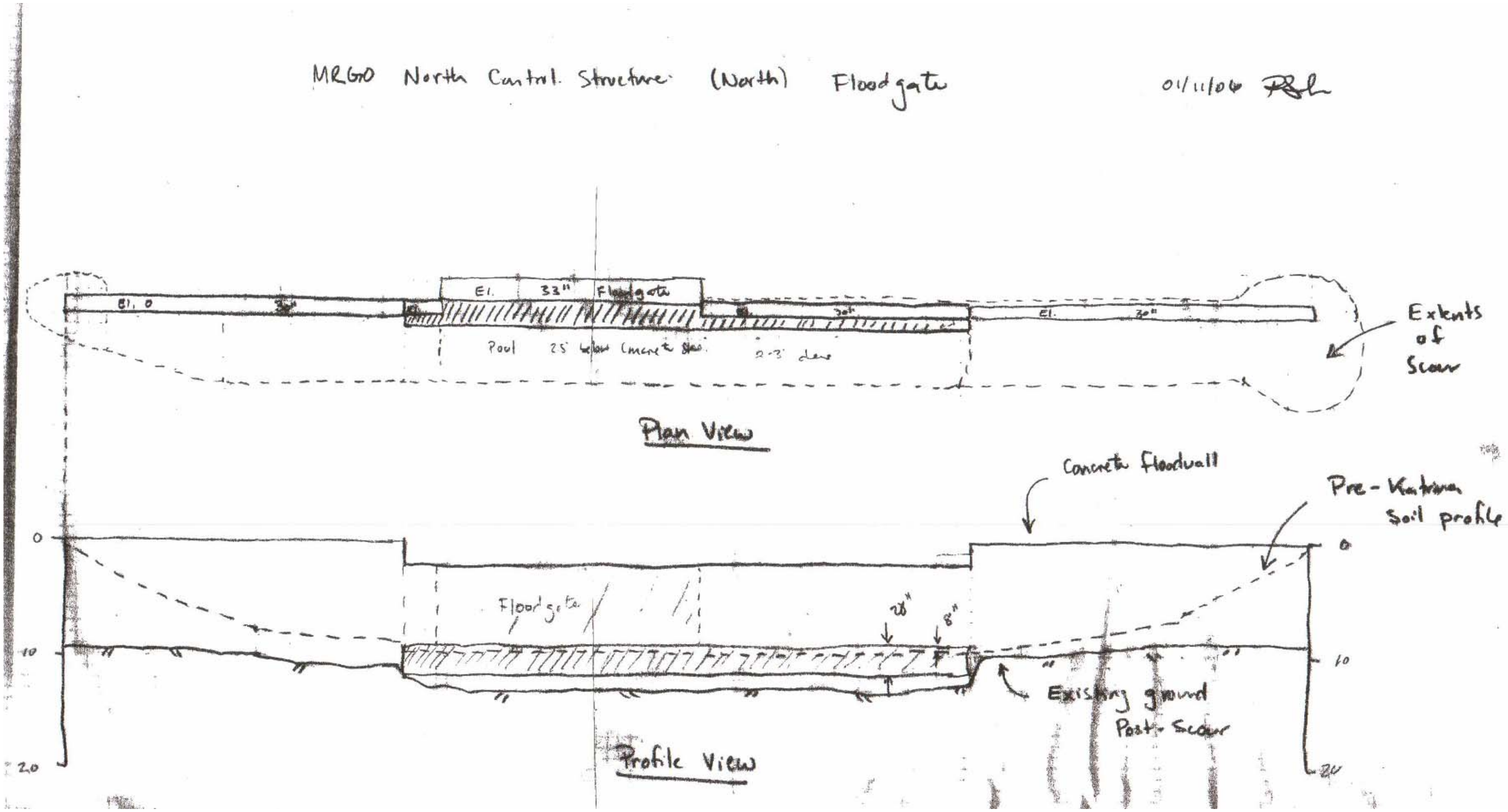
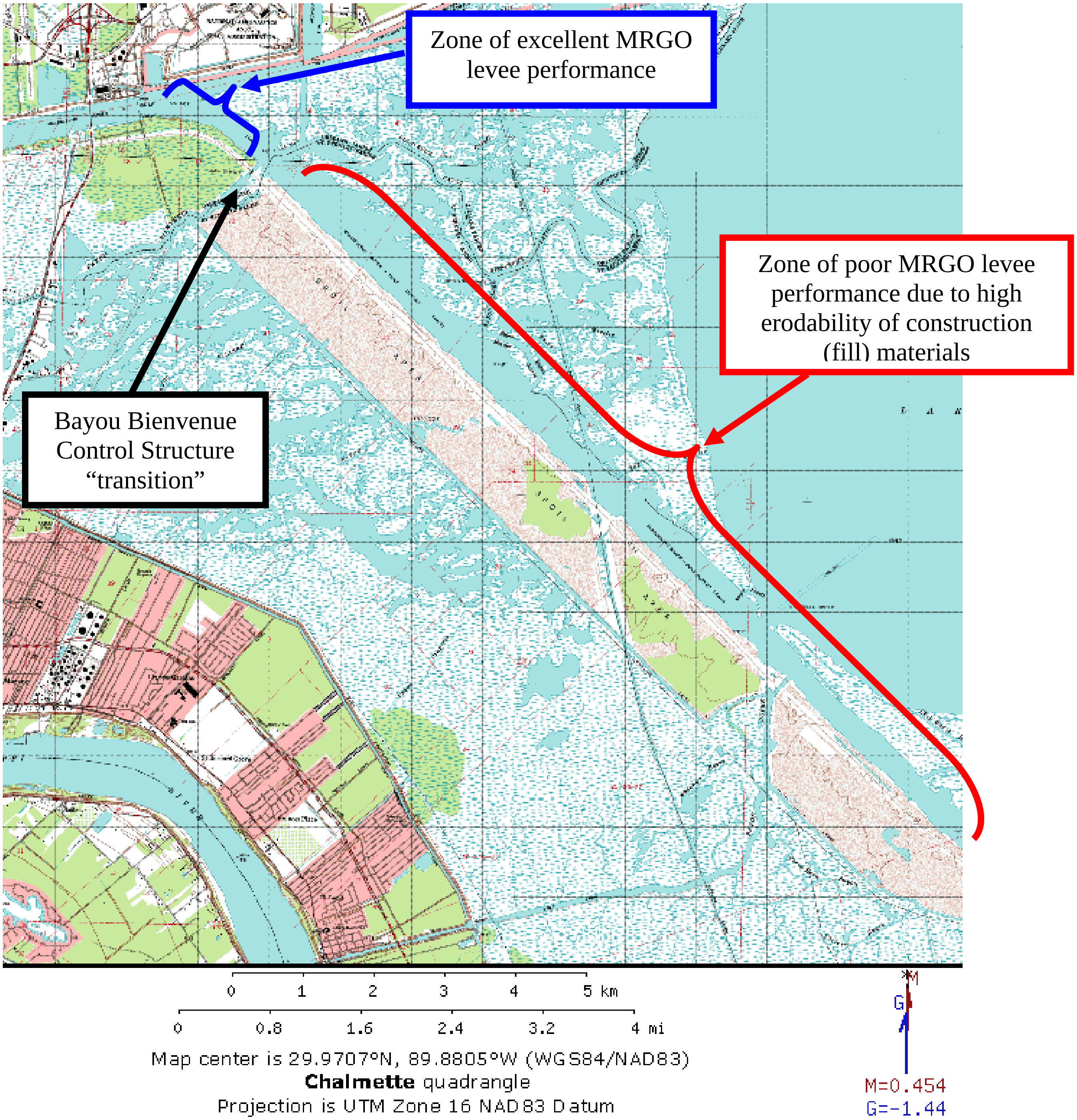
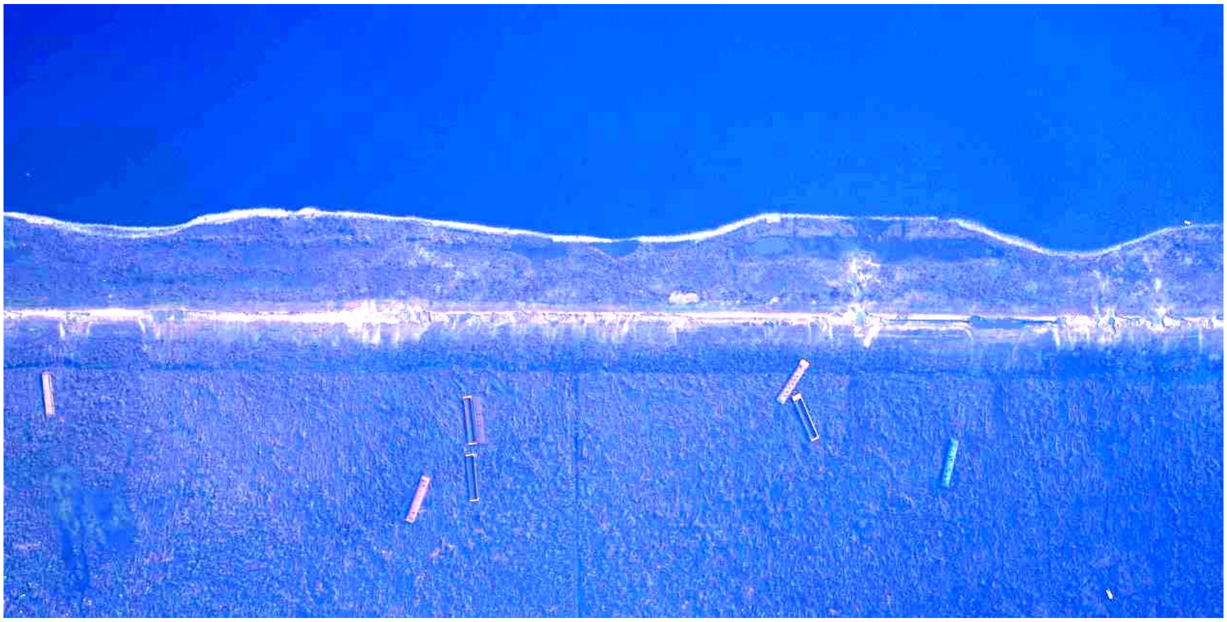


Figure 10.25: Scour pattern behind around the floodgate structure at the Bayou Bienvenue Control Structure shown in Figure 10.22.



Topographic map from Topozone.com

Figure 10.26: A U.S.G.S. topographic map showing the presence of "spoils" along the MRGO. As seen in Figure 10.28, the levee materials to the southeast of the Bayou Bienvenue Control Structure performed poorly.



Photograph courtesy ngs.woc.noaa.gov/Katrina/KATRINA0000.HTM

Figure 10.27: Aerial photograph taken by NOAA in early September 2005 along the MRGO showing severe erosion/breaches of the earthen levee and transport and deposition of large barges over the levee as a result of the storm surge (location 8 on Figure 10.6).

Photograph courtesy Les Harder

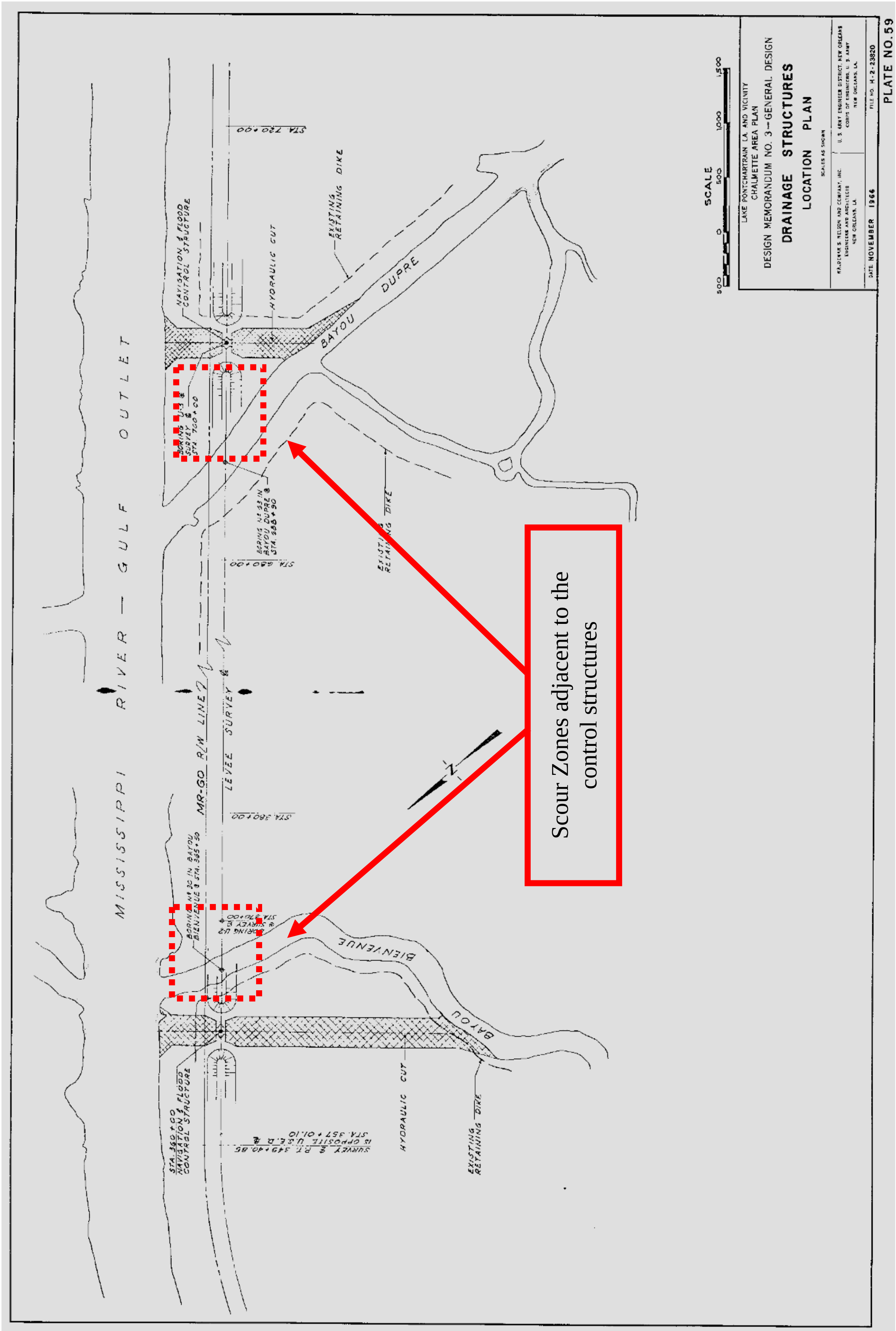
Figure 10.28: Close up aerial photograph of severely eroded MRGO levees at location 8 on Figure 10.6.

Photograph courtesy Les Harder

Figure 10.29: Control structure at Bayou Dupre (location 9 on Figure 10.6) suffered extensive scour and erosion during the storm surge and overtopping conditions associated with Hurricane Katrina.

Photograph courtesy Les Harder

Figure 10.30: Another aerial photograph of the scour and erosion damage to the Bayou Dupre control structure.



Source: USACE (1966)

Figure 10.31: Areas of severe scour and erosion damage to the two control structures appear to correspond to the alignment of the abandoned original channel.

Photograph courtesy Robert Bea

Figure 10.32: Aerial photograph taken of the repair operations at Bayou Dupre in January 2006.

Photograph courtesy Robert Bea

Figure 10.33: Close up aerial photograph showing backfilling operations at Bayou Dupre.

Photograph courtesy Rune Storesund

Figure 10.34: Eastward looking view of the secondary earthen levee (constructed to approximately Elevation +6.0 feet MSL) immediately to the north of the Corinne Canal, east of Paris Road (location 10 on Figure 10.6). This levee was significantly overtopped and did not experience significant damage.

Photograph courtesy Rune Storesund

Figure 10.35: A fishing boat was washed over the levee shown in Figure 10.34 and landed in this Chalmette residential neighborhood within St. Bernard Parish.

Photograph courtesy Rune Storesund

Figure 10.36: The same levee shown in Figure 10.34, where a fishing boat washed over the levee during Hurricane Katrina, has now been raised from an elevation of approximately +6 feet MSL to an elevation of approximately +10 feet MSL by March of 2006.

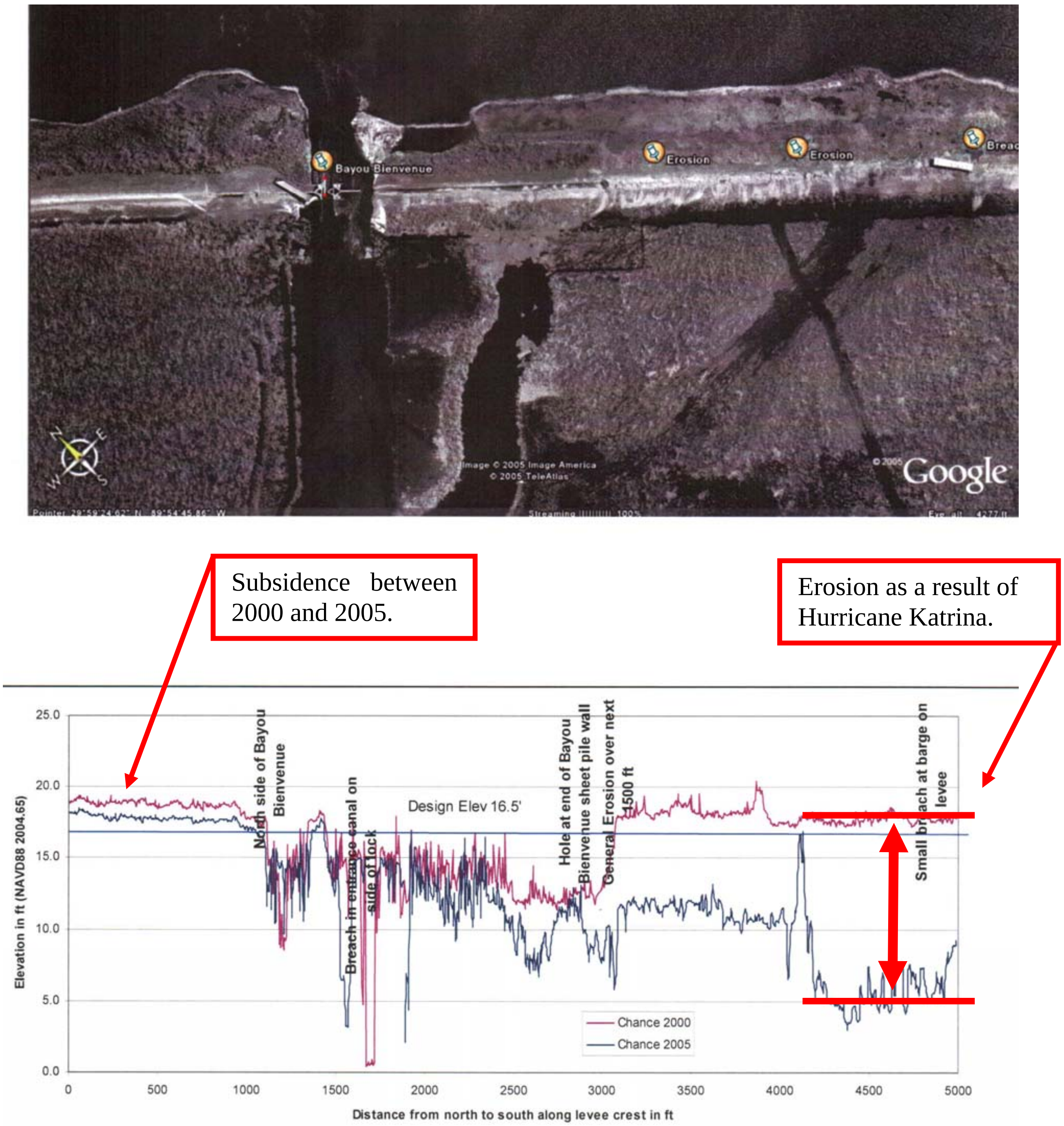


Figure 10.37: This is a comparison of two LiDAR surveys of the MRGO levee system at the Bayou Bienvenue control structure, near the intersection of the GIWW at the northeast corner of St. Bernard Parish. Effects of subsidence can be clearly seen in the elevation differences on the north side of the control structure and the control structure itself between 2000 and immediately following Hurricane Katrina in 2005. The levee on the north side of the Bayou Bienvenue control structure was largely undamaged. The levee on the south side of the control structure was catastrophically damaged. The magnitude of the erosion has been highlighted and white "splotches" of displaced levee materials can be seen in the aerial photograph (USACE 2006).

Figure 10.38: Levee sample sites within the Orleans East Protected Area.

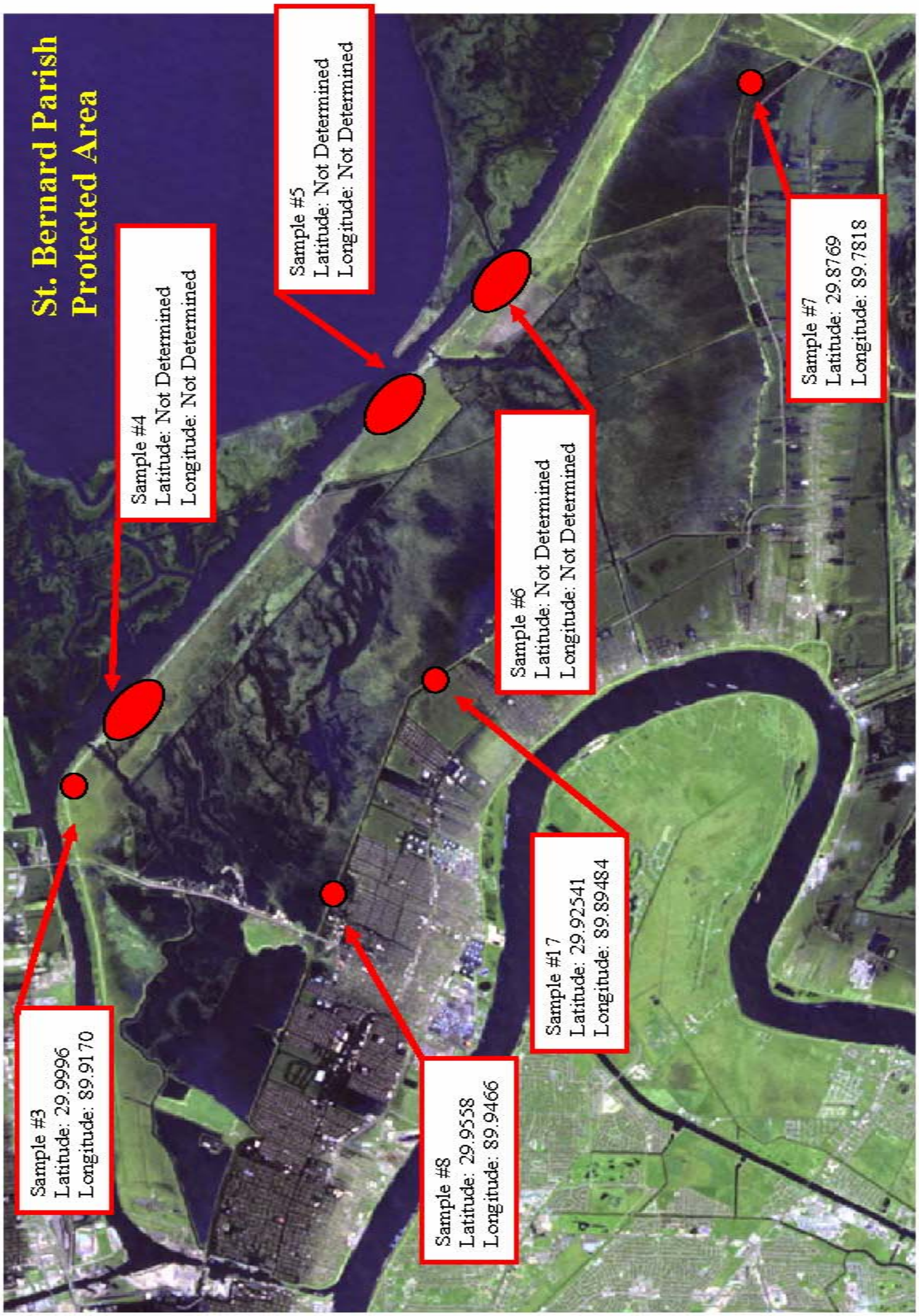


Figure 10.39: Levee sample sites within the St. Bernard Parish Protected Area.

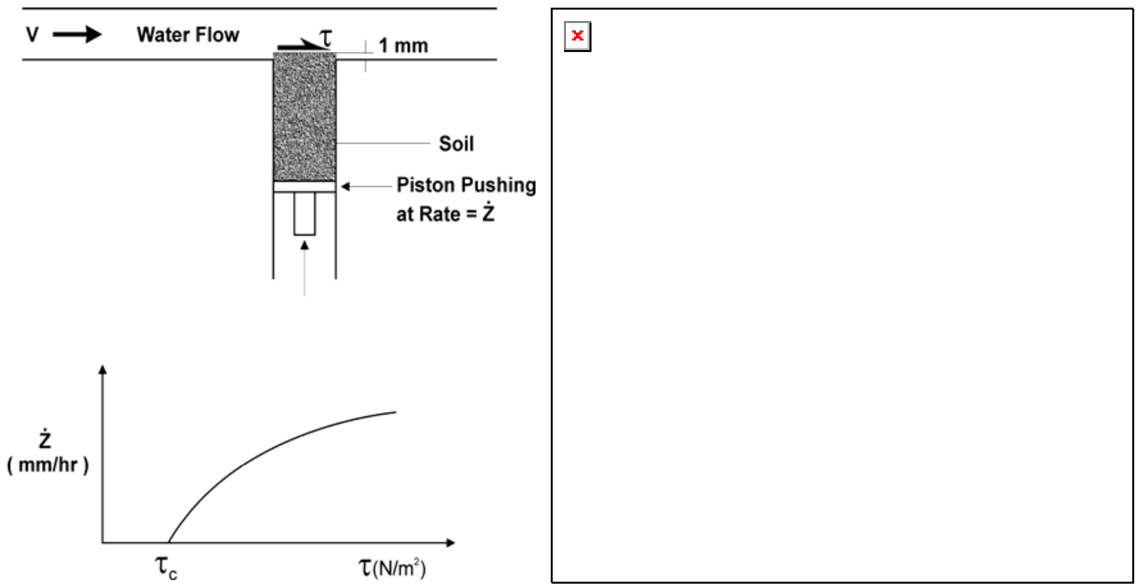


Figure 10.40: The EFA (Erosion Function Apparatus) as developed by Briaud et al. Soil samples are advanced into a rectangular tube of flowing water, creating a shear stress on top of the inserted soil sample. The velocity of the water is increased until the critical shear stress is achieved, where the soil begins to actively erode.

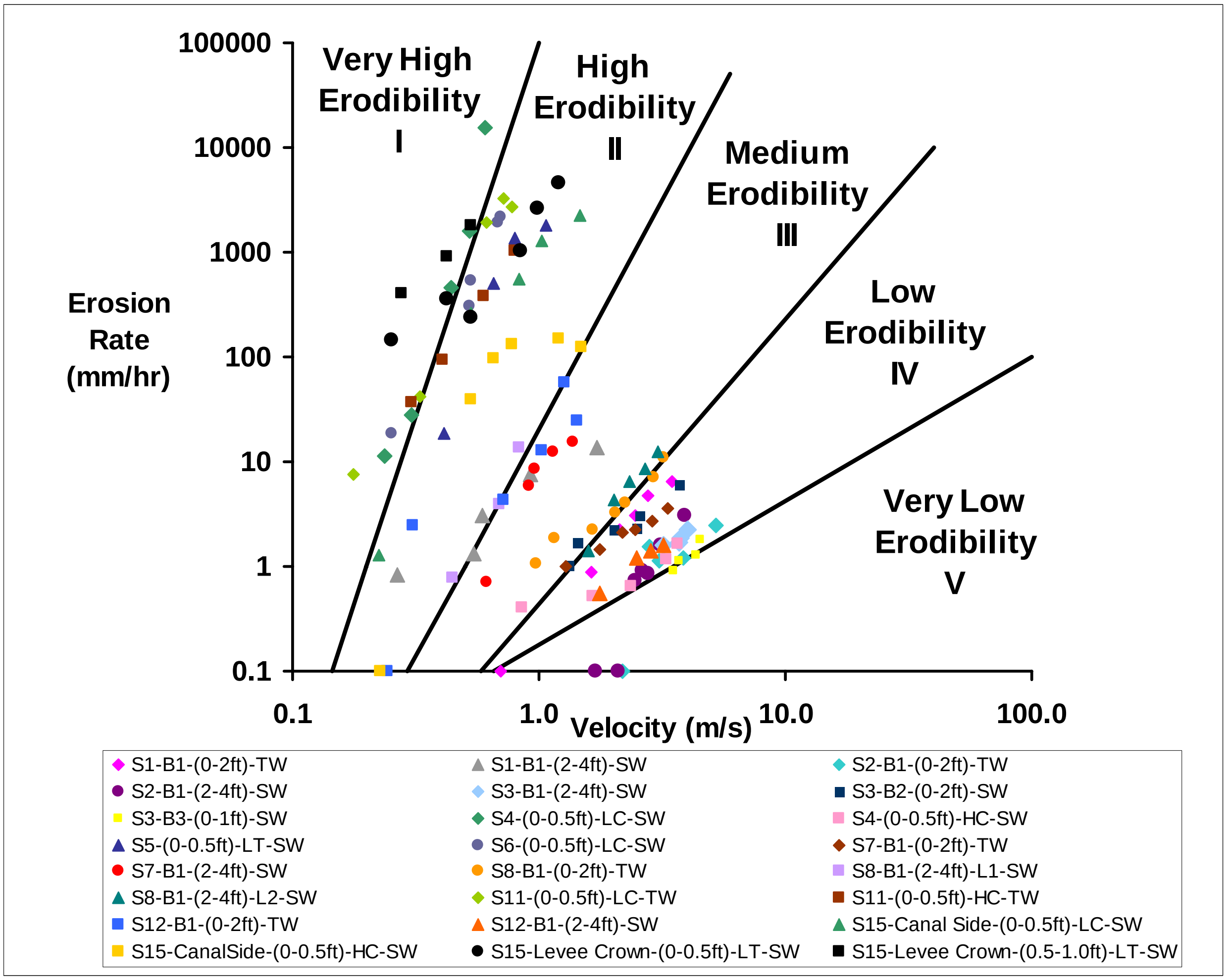
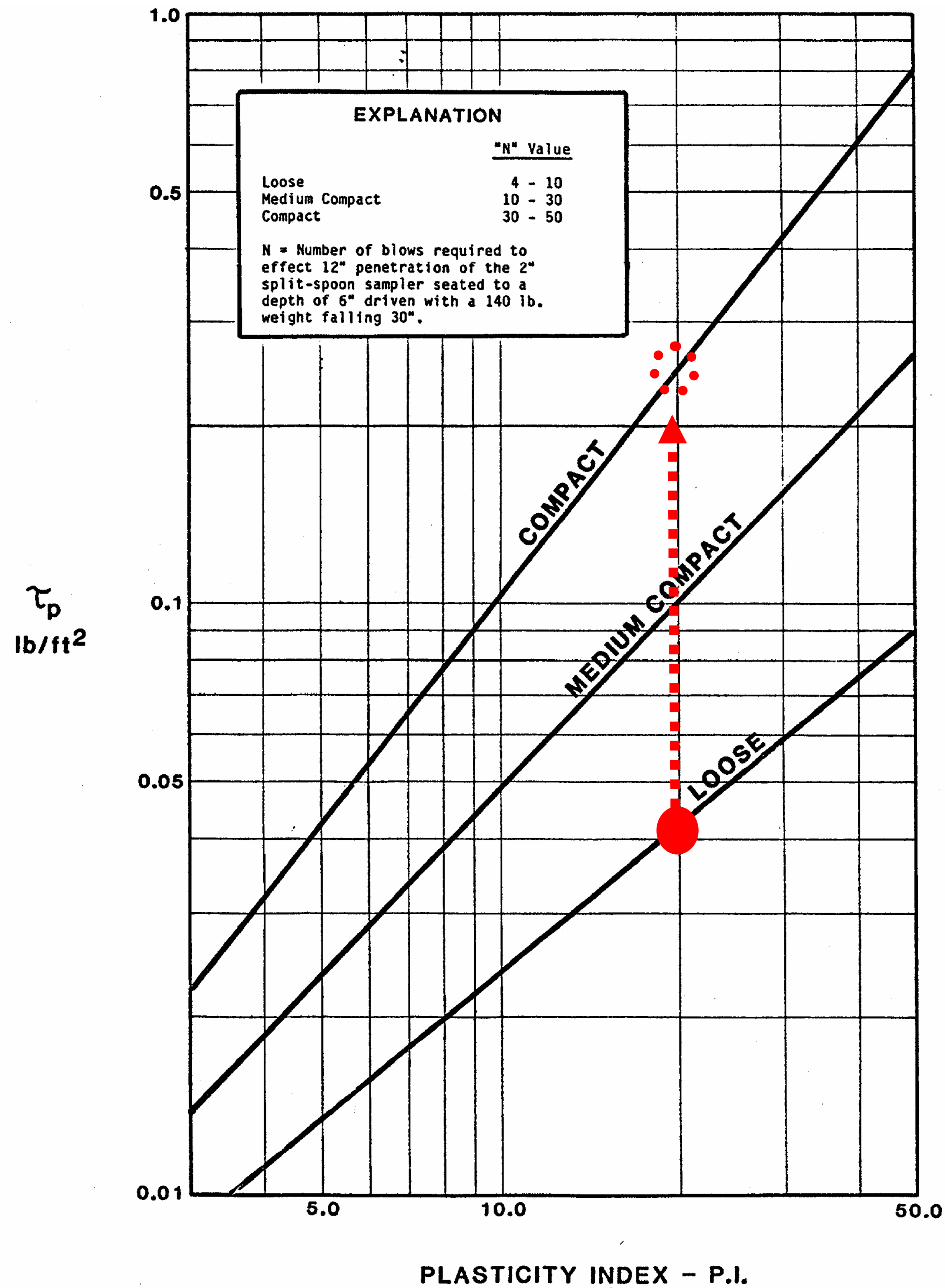


Figure 10.41: Summary of all EFA test results. The test result designations are based on the Site Number, the boring number at the site, the depth interval over which the sample was collected and sample notes. The EFA test results indicate that the materials used in levee construction varied from very high to very low erodibility, which matched the observed performance of these levees.



Graphic after FHWA 1988

Figure 10.42: The effects of compaction are clearly evident in this figure from this FHWA design guideline. For the same material (with a plasticity index of 20) a ten-fold increase in shear stress capacity can be achieved by properly compacting the material.

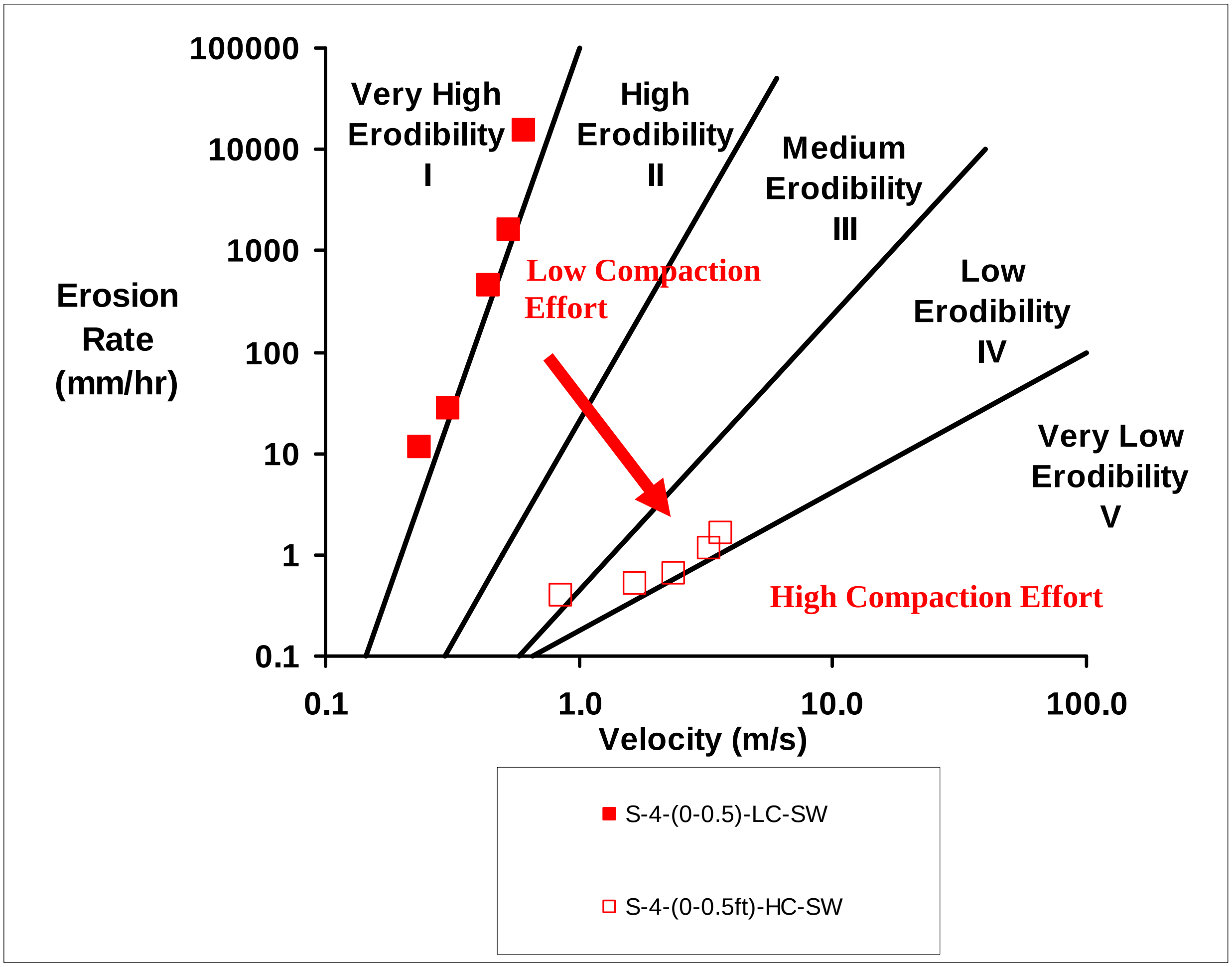


Figure 10.43: Here materials sampled from the MRGO were tested at two levels of compaction. This figure shows the dramatic impact proper compaction has on the erodibility of soils. Materials sampled from the MRGO levee were tested at two compaction levels: low compactive effort and high compaction effort. The corresponding results speak volumes to the importance of compaction in earthen levees. The low-compaction sample was found to be very highly erodible, whereas the high-compaction sample exhibited low erodibility characteristics.

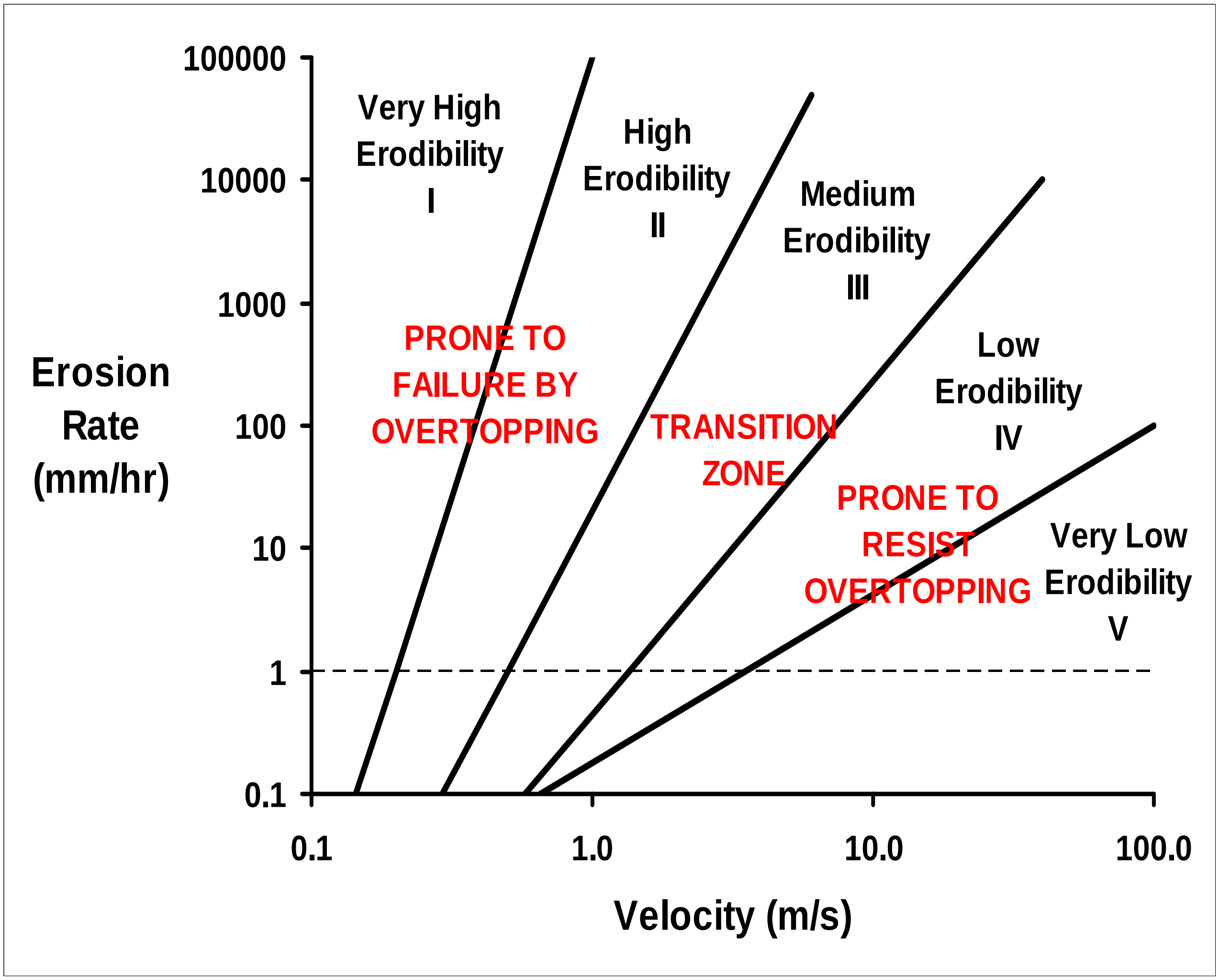


Figure 10.44: Resulting guideline table for evaluating erosion susceptibility of soils used for levee construction developed by Dr. Briaud from Texas A & M University.

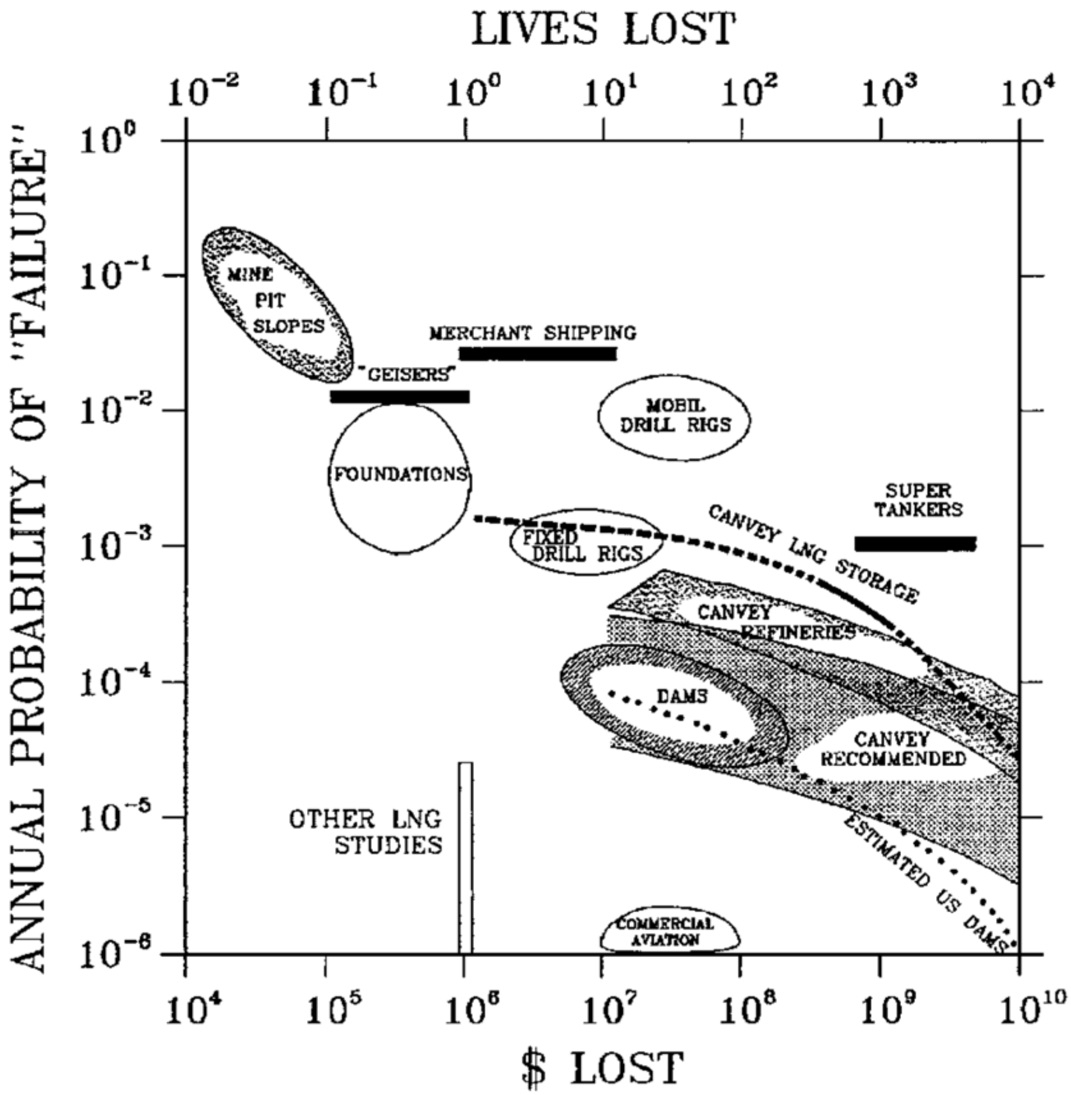


Figure 10.45: Plot of back-calculated annual probability of failure vs. lives lost and $ lost (note that either the "lives lost" or "$ lost" axes are used, they are not intended to be used in conjunction). From Christian, 2004.

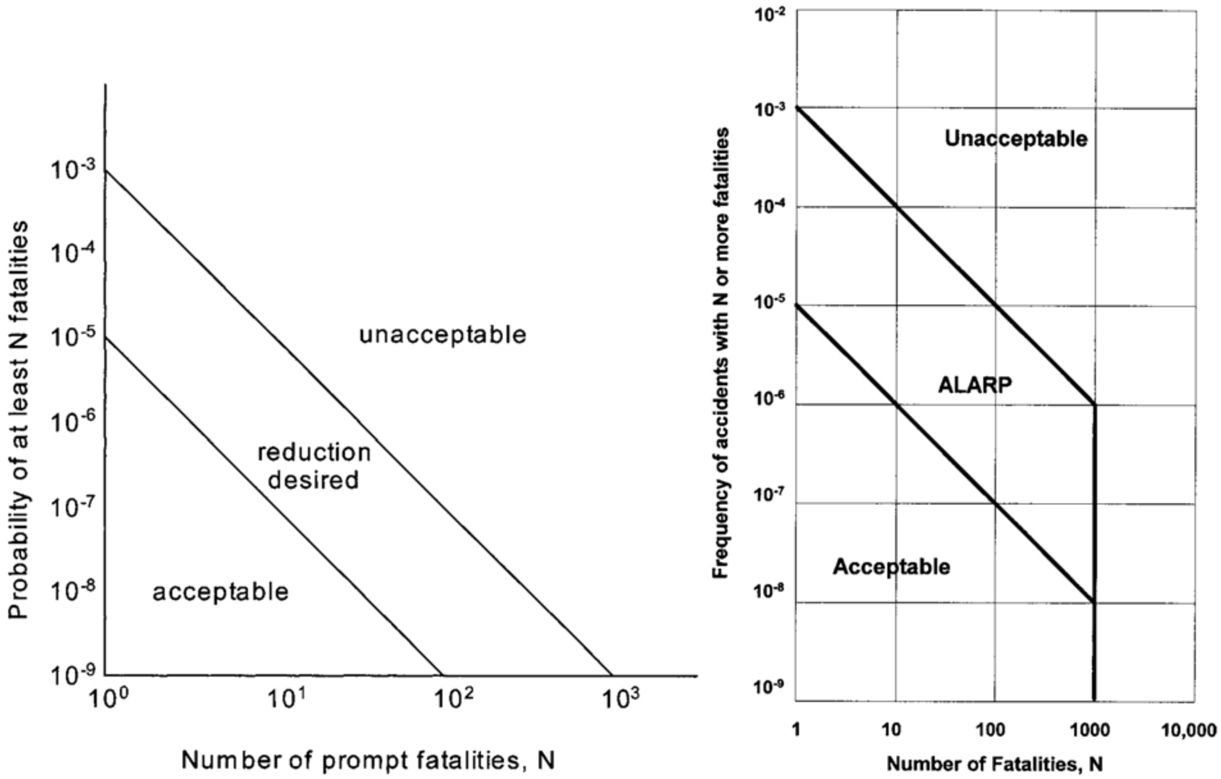


Figure 10.46: F-N diagrams adopted by the Hong Kong Planning Department (left) and F-N diagram as proposed for planning and design use in the Netherlands (right). From Christian, 2004.

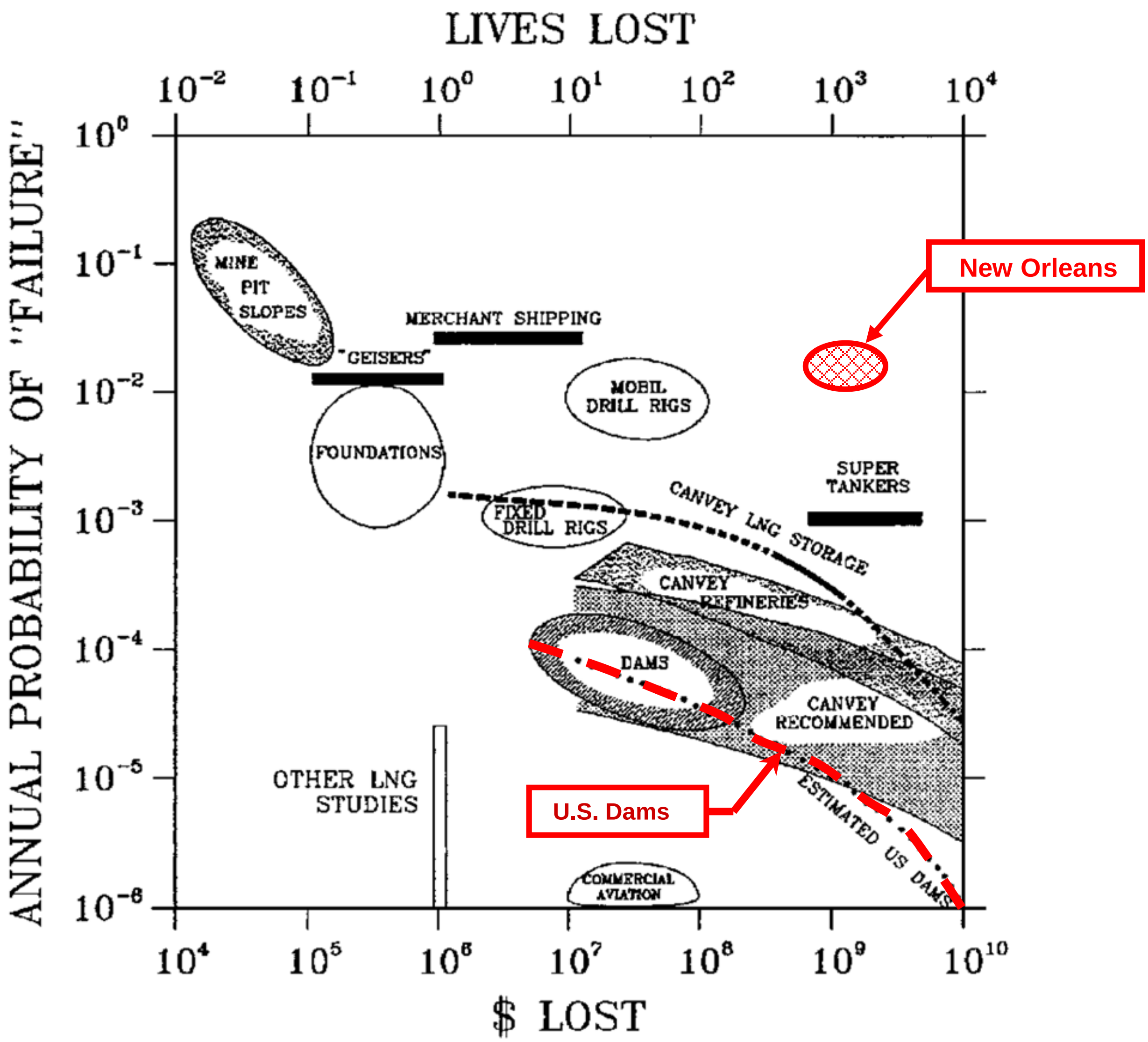


Figure 10.47: Figure 10.45 repeated, with approximate level of reliability and consequences for the New Orleans regional flood protection systems indicated, and with current U.S. practice for dams highlighted (red dashed line.)

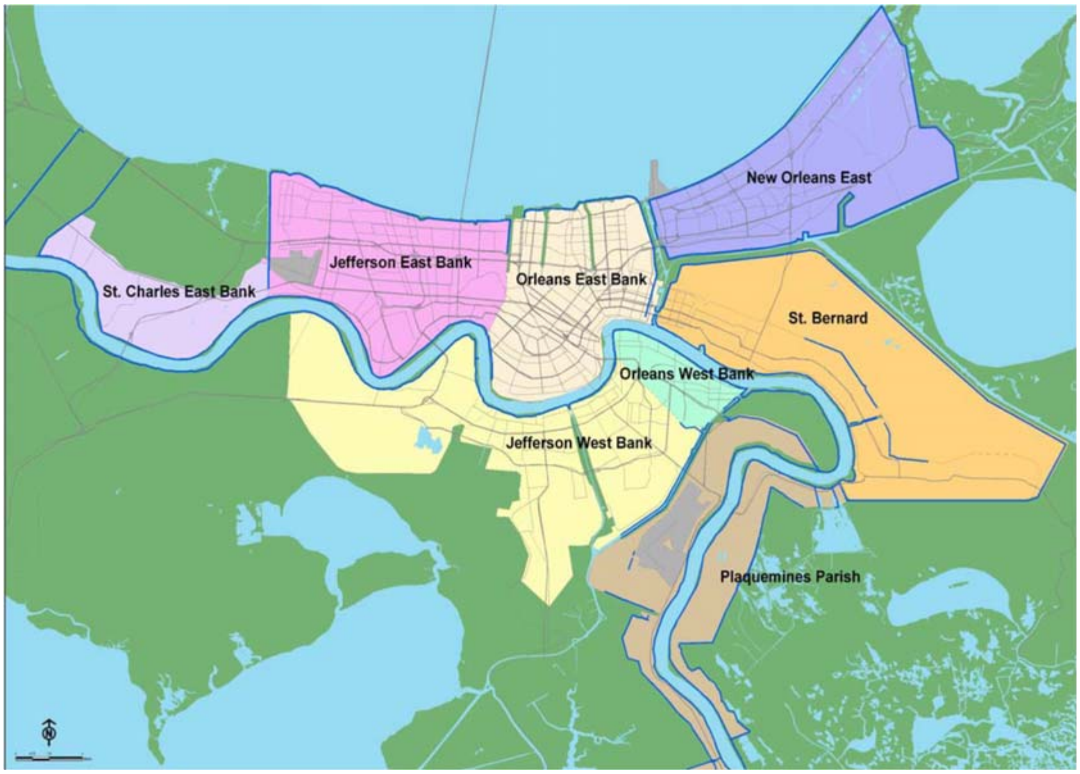


Figure 10.48: Overview of the drainage and pumping parishes studied as part of the IPET (2006) performance evaluation. The primary study areas were Jefferson, Orleans, Plaquemines, and St. Bernard parishes. [IPET; June 1, 2006]

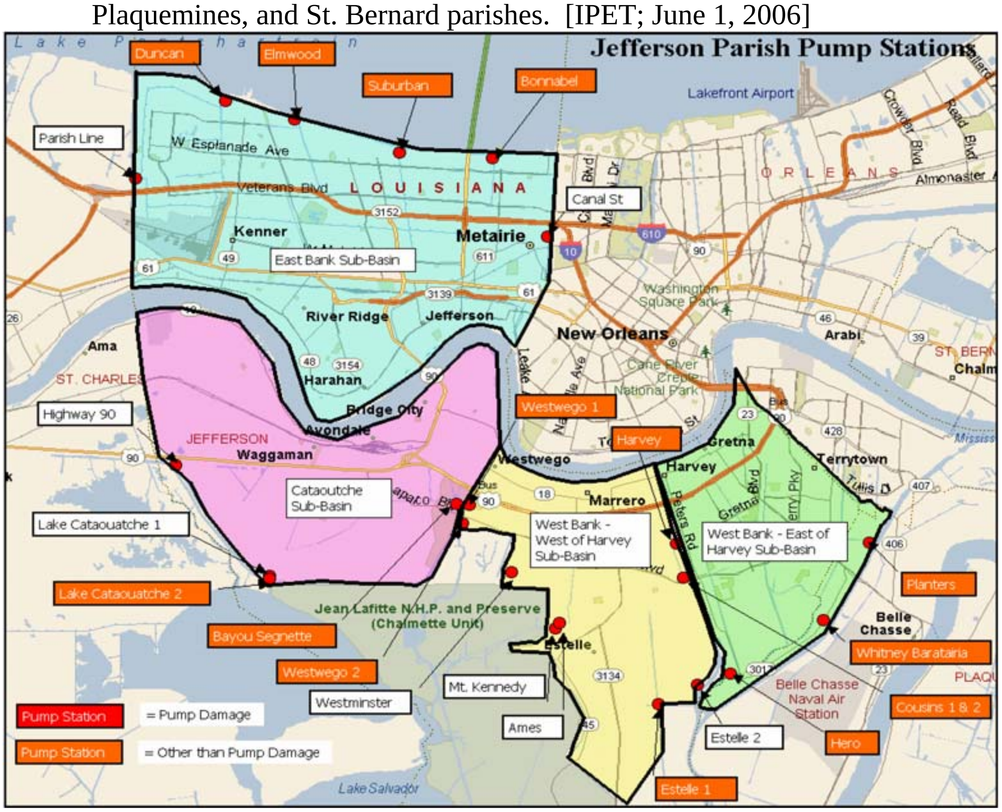


Figure 10.49: Detailed map of pump stations within Jefferson parish. [IPET; June 1, 2006]

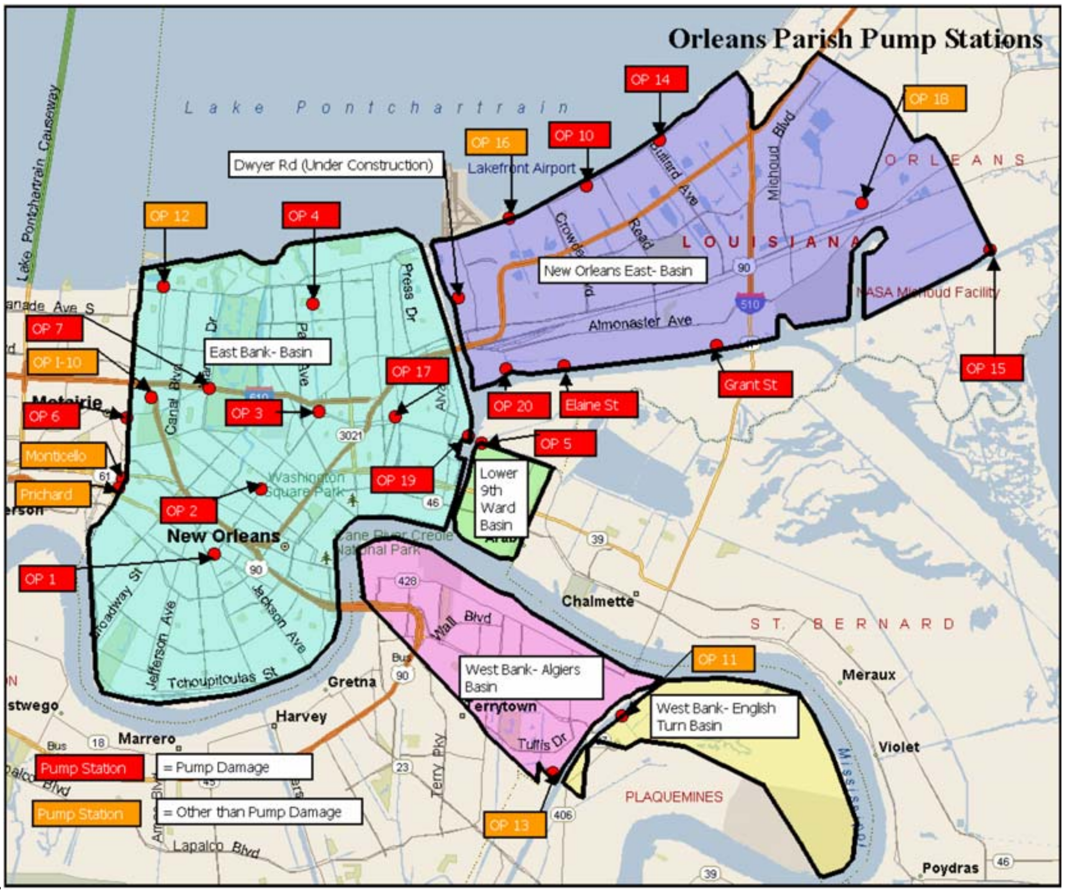


Figure 10.50: Detailed map of pump stations within Orleans parish. [IPET; June 1,2006]

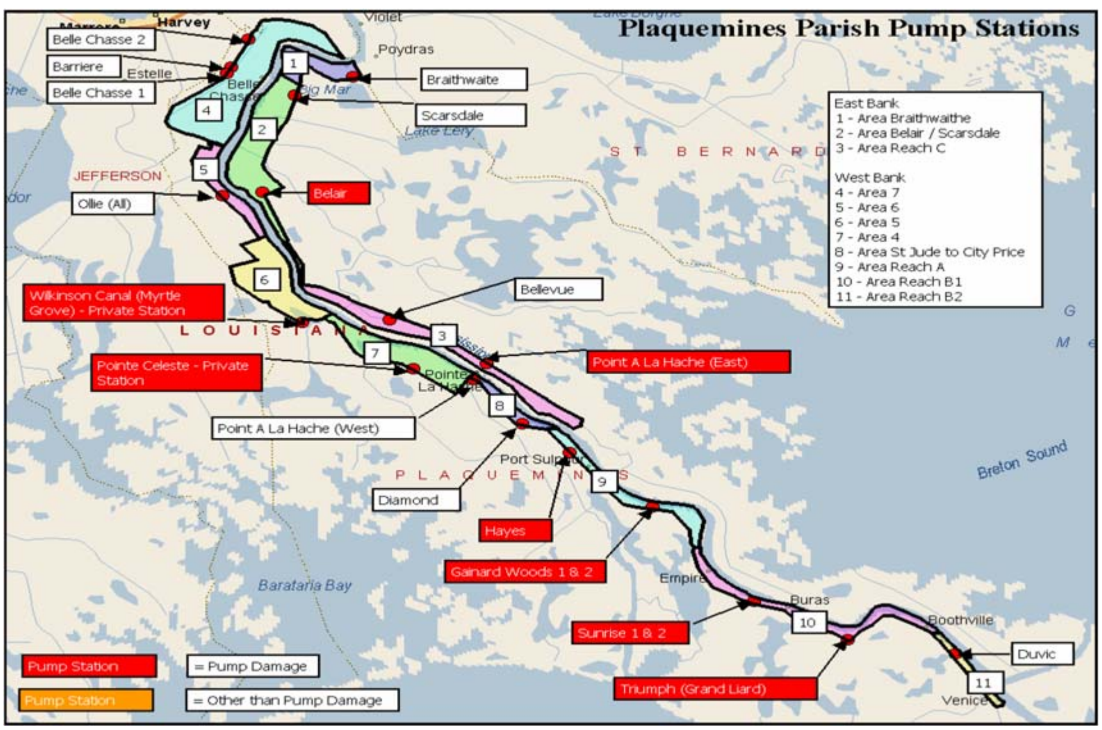


Figure 10.51: Detailed map of pump stations within Plaquemines parish. [IPET; June 1, 2006]

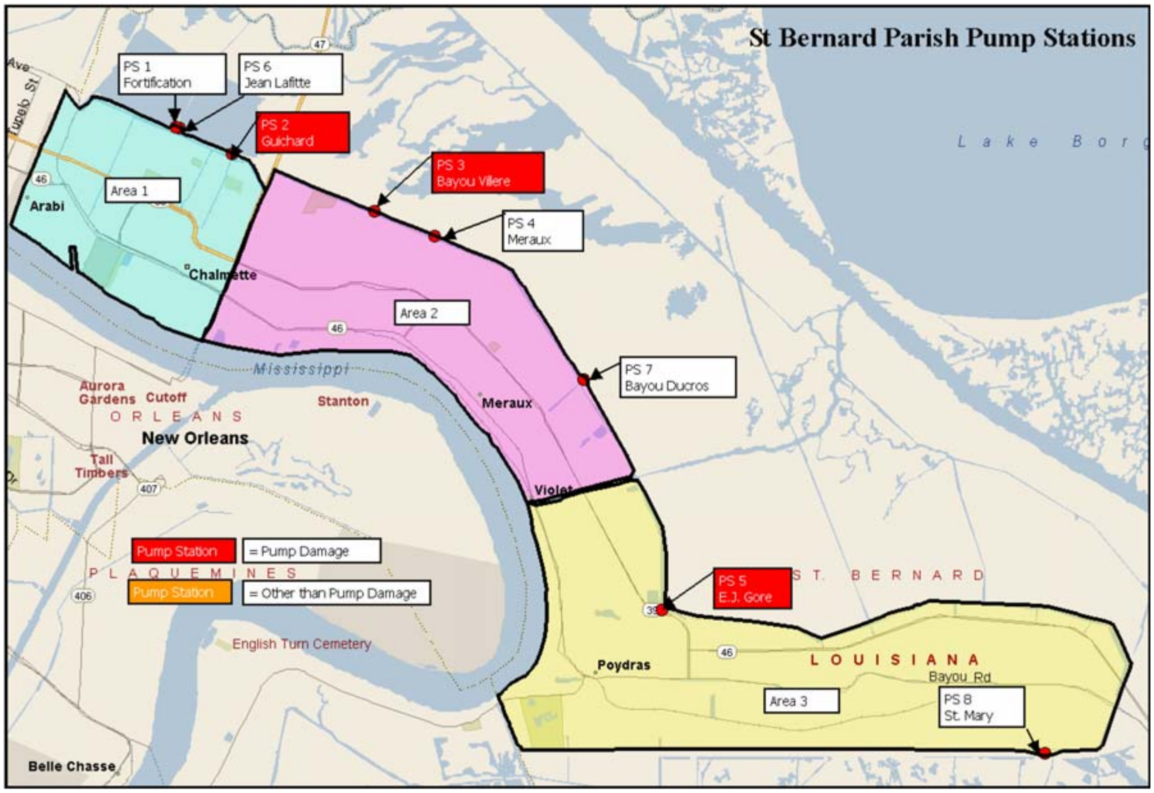
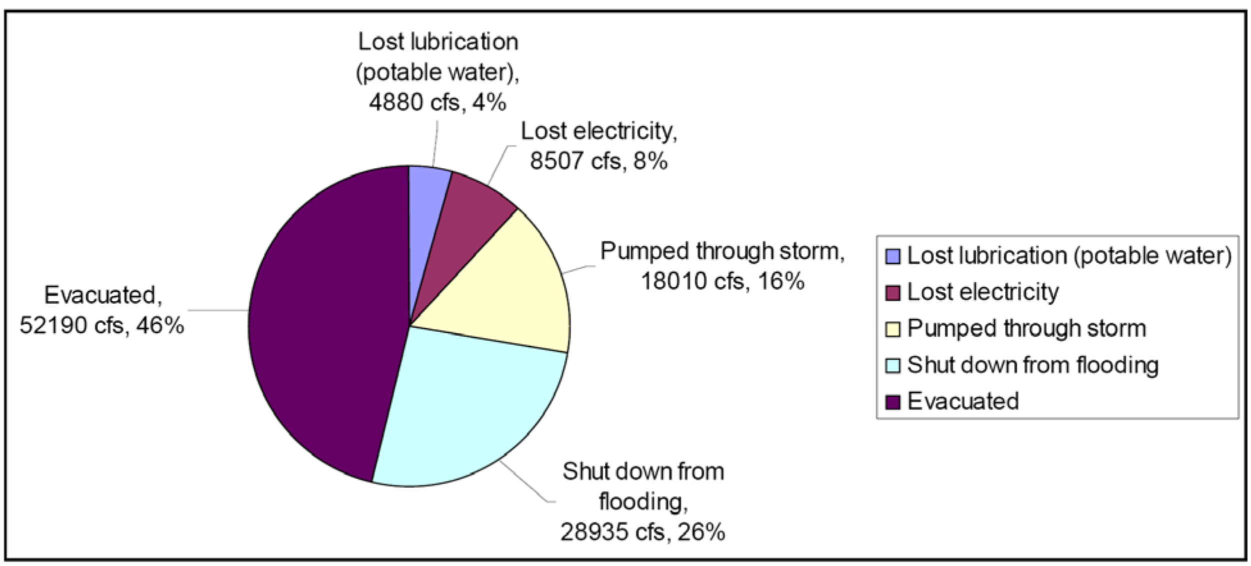


Figure 10.52: Detailed map of pump stations within St. Bernard parish. [IPET, June 1, 2006]



Source: IPET, 2006

Figure 10.53: Performance of the pumping system (and causes of pumping capacity loss) during and after hurricane Katrina.

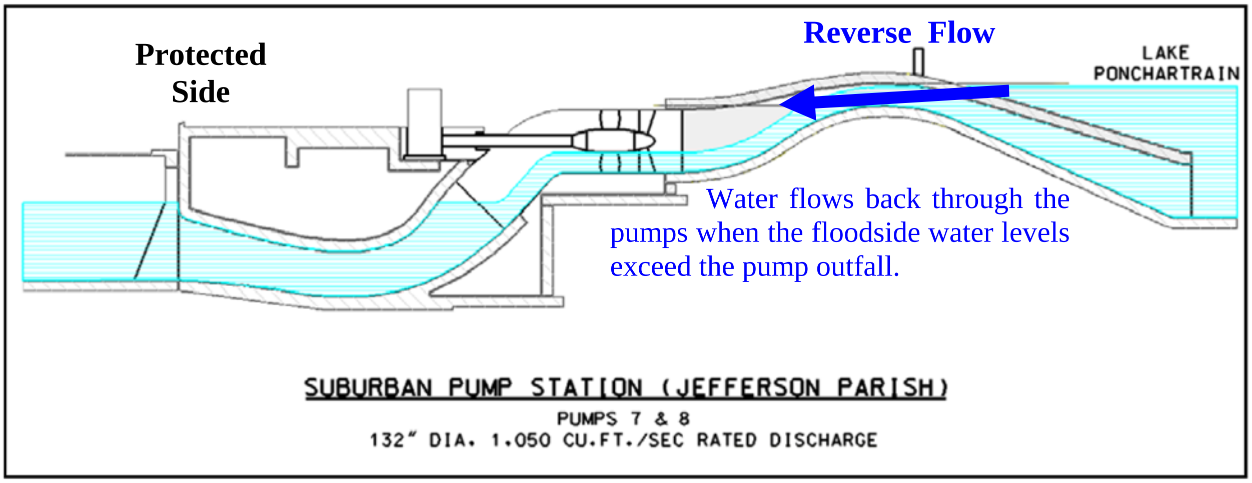


Figure 10.54: Conditions where the water level on the outboard side of the pumping station is higher than the pump infrastructure, resulting in reverse flow from the flood side back into the "dry" side. [IPET; June 1, 2006]

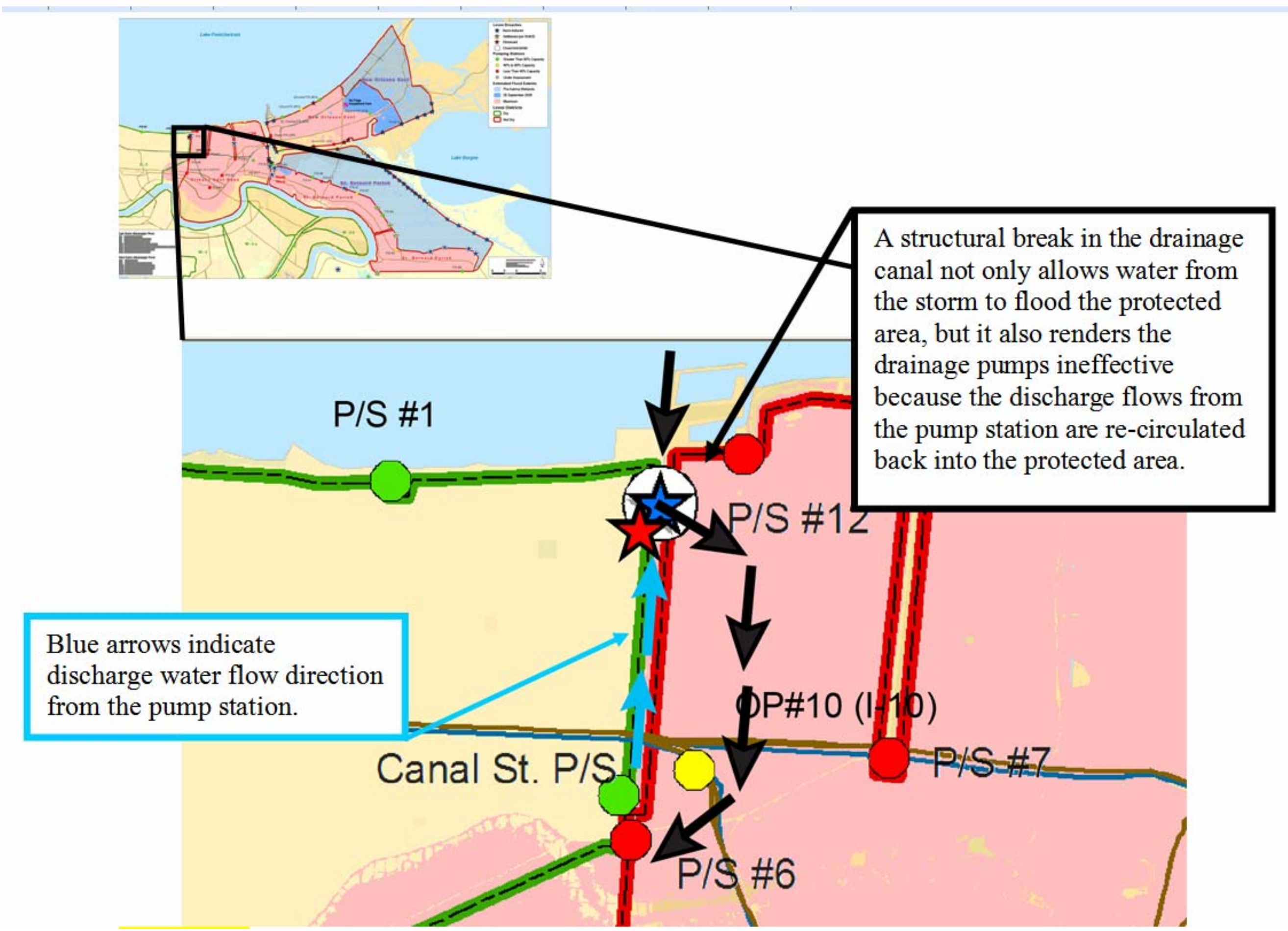


Figure 10.55: Breakdown of the pumping and drainage system as a result of structural failures within the drainage canals. Flood waters and discharge water from the pumping station flow back into the protected area where structural failures have occurred.

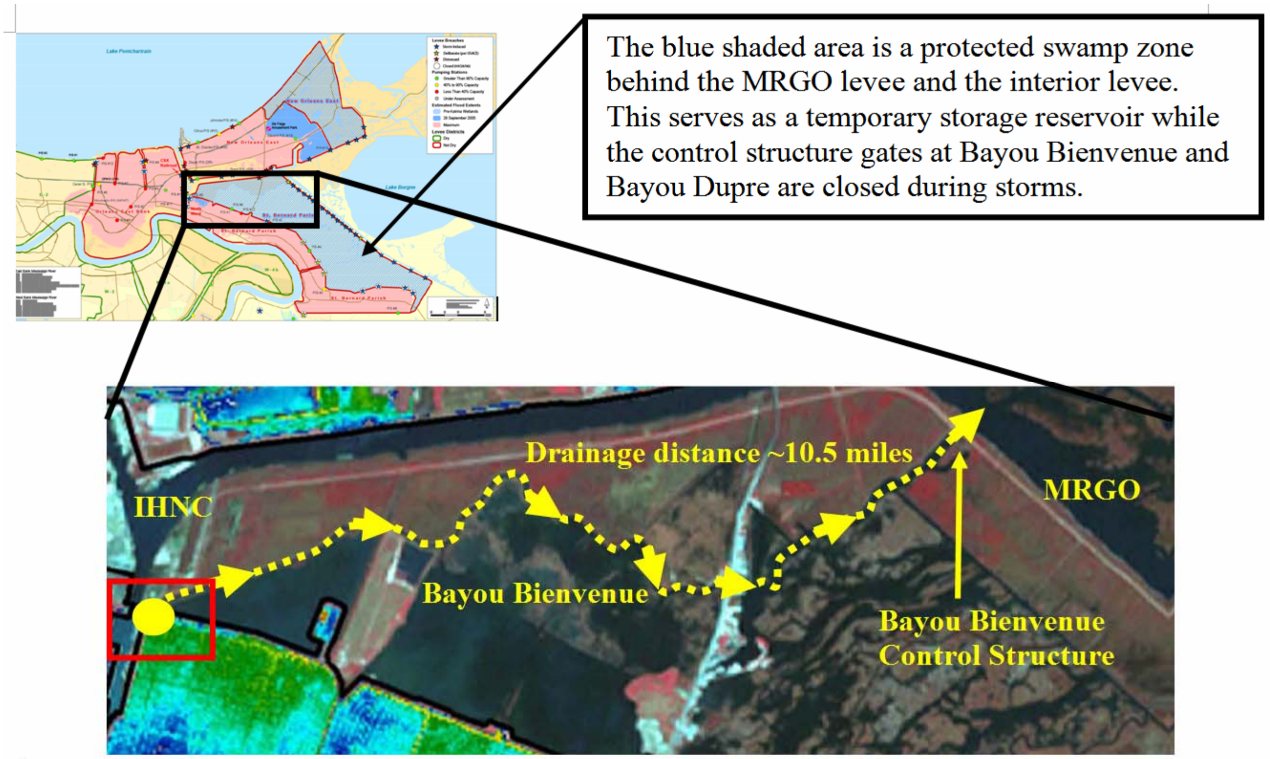


Figure 10.56(a): The current drainage configuration for the lower 9th Ward in St. Bernard parish starts at the Florida Street pump stations and flows approximately 10.5 miles through Bayou Bienveue, then through a control structure to the MRGO.

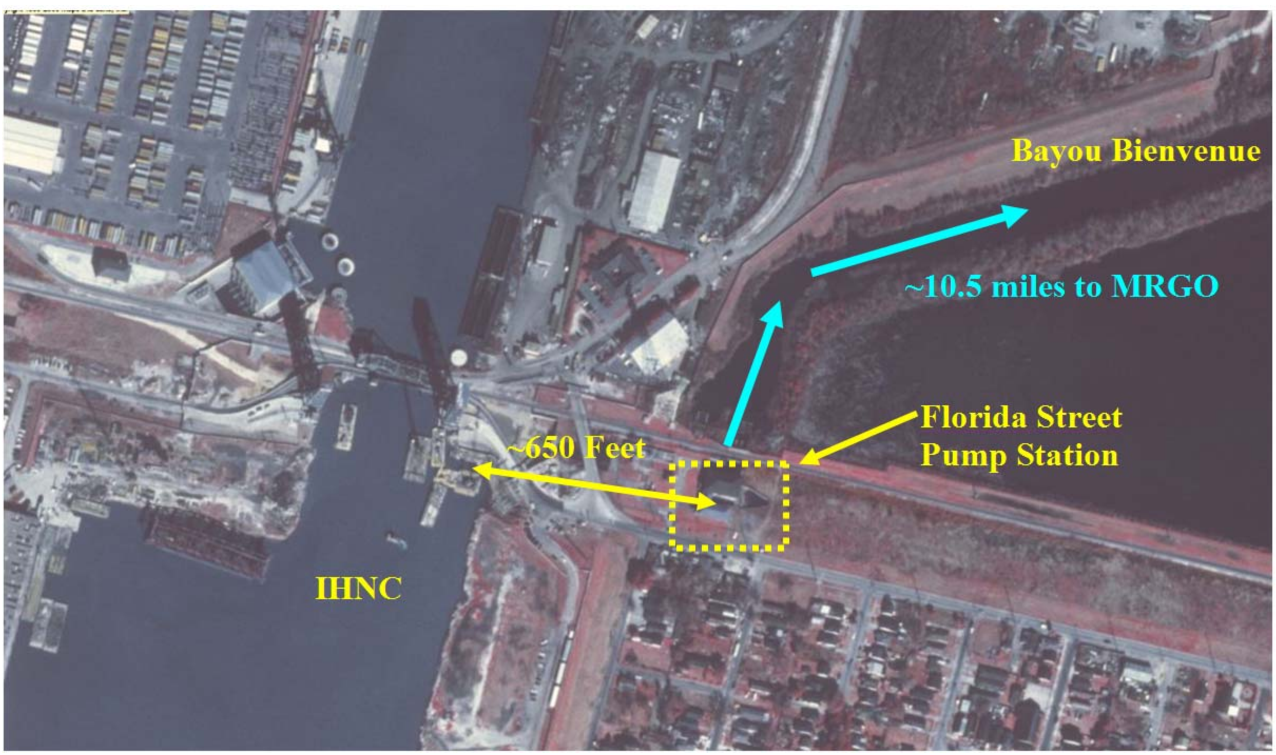


Figure 10.56(b): The alternative; pumping into the IHNC 650 feet to the west.