CAUTION: Analysis for this report was completed prior to the issuance of Engineer Technical Letter (ETL) 1110-2-575, EVALUATION OF I-WALLS, dated 1 September 2011.

http://publications.usace.army.mil/publications/eng-tech-ltrs/ETL_1110-2-575/ETL_1110-2-575.pdf

The Corps is performing additional evaluation of the I-walls along the 17th, Orleans and London outfall canals to address the 2011 ETL.

As of June 11, 2013, the new evaluation reports have not been finalized.

Any reference to this report should include this notice.

US Army Corps
of Engineers ®

# LAKE PONTCHARTRAIN AND VICINITY HURRICANE PROTECTION PROJECT

## ORLEANS AVENUE CANAL
## ORLEANS PARISH, LOUISIANA

# MOWL for Orleans Avenue Canal

**Prepared for:**

**Hurricane Protection Office (HPO)**
**U.S. Army Corps of Engineers**

**Prepared by:**

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## 1.0 *EXECUTIVE SUMMARY*

Some of the most severe flooding in the City of New Orleans in the aftermath of Hurricane Katrina was caused by the failure of the parallel protection systems on two of the three major outfall canals that discharge the City's storm water. These open canals connect pump stations located several miles inland to Lake Pontchartrain to the north of the City. Because the outfall canals were open to Lake Pontchartrain, the design of the canals had to consider the water levels in the Lake. Each canal consists of a combination of earthen levees and/or floodwalls that rise above the surrounding "protected" ground surface to accommodate a high water level in the canal during pumping and during high-water events in the Lake. The storm surge from Hurricane Katrina moved up the canals and the resulting high water levels ultimately caused structural failure of the floodwalls on the 17th Street Canal and the London Avenue Canal. The Orleans Avenue outfall canal did not experience failure. Immediately following Katrina, the U.S. Army Corps of Engineers (Corps) commenced the design and construction of Interim Closure Structures at the mouths of each of the three outfall canals to essentially isolate water levels in the canals from water levels in the Lake. To permit the City's storm water removal system to continue to function, pumps were added at the interim closure structures to pump water from the canals into the Lake. The interim closure system, therefore, currently requires "double pumping" – storm water is pumped into the canals by the City's original pump stations and subsequently pumped from the canals into the Lake by the interim pump stations installed after Hurricane Katrina. Because it is believed that sustained high water levels in the canals ultimately contributed to the failure of the flood protection system, concerns by all stakeholders remained regarding the "safe water level" that the canal walls could sustain during interim pumping. As a result of preliminary technical analysis of the repaired floodwalls, the Corps established interim Maximum Operating Water Levels" (MOWLs) for each canal. For the Orleans Avenue Canal, the MOWL was established at El 8 North America Vertical Datum 1988 (NAVD88). It is generally believed that this elevation could be exceeded if the pump stations were operated at or near capacity. At the same time, it was recognized that if the pumping systems were not operated at full capacity, there was a distinct danger that the City would flood.

In response to this dilemma, the Corps New Orleans District, Hurricane Protection Office (HPO) requested a study for the Orleans Avenue Canal to determine a MOWL that could be sustained for the flood control levees/floodwalls along both sides of the canal from Drainage Pump Station 7 (DPS 7) north to the Interim Control Structure (ICS) near Lake Pontchartrain. This report was prepared using Corps design and analysis procedures, specifically those based on the gap stability analysis methodology titled, *Stability Analysis of I-walls Containing Gaps between the I-wall and Backfill Soils* [7], and the Hurricane and Storm Damage Risk Reduction System Design Guidelines (HSDRRSDG) [4].

The Orleans Avenue Canal parallel protection system consists of earthen levees with floodwalls to provide additional protection. Floodwalls consist of I-walls and T-walls along the reaches of the canal defined in Table 1-1. None of the levees, I-walls, or T-walls along the Orleans Avenue Canal failed during Hurricane Katrina.



**TABLE 1-1**
**LEVEE REACH LOCATIONS**

| WEST REACH | WALL TYPE OR LEVEE | WEST BASELINE APPROXIMATE STATION | EAST REACH | WALL TYPE OR LEVEE | EAST BASELINE APPROXIMATE STATION |
|---|---|---|---|---|---|
| 1A | I-wall | 2+45 to 7+00 | 12A | Wall | 2+45 to 3+70 |
| 1B | I-wall | 7+00 to 9+25 | 12B | Levee | 3+70 to 4+70 |
| 1C | I-wall | 9+25 to 11+00 | 13A | I-wall | 4+70 to 7+00 |
| 1D | I-wall | 11+00 to 14+20 | 13B | I-wall | 7+00 to 11+20 |
| 2 | I-wall | 14+20 to 21+75 | 14 | I-wall | 11+20 to 20+50 |
| 3 | T-wall | 21+75 to 24+87 | 15 | I-wall | 20+50 to 30+00 |
| 4 | I-wall | 24+87 to 29+16 | 16 | I-wall | 30+00 to 36+40 |
| 5 | T-wall | 29+16 to 36+26 | **Harrison Avenue** | | 36+40 to 37+44 |
| **Harrison Avenue** | | 36+26 to 37+27 | 17 | I-wall | 37+44 to 50+00 |
| 5 | T-wall | 37+27 to 42+00 | 18A | I-wall | 50+00 to 61+00 |
| 6 | T-wall | 42+00 to 50+00 | 18B | I-wall | 61+00 to 64+00 |
| 7 | T-wall | 50+00 to 59+00 | **Filmore Avenue** | | 64+00 to 65+00 |
| 8 | T-wall | 59+00 to 63+58 | 19 | I-wall | 65+00 to 90+62 |
| **Filmore Avenue** | | 63+58 to 65+00 | **Robert E. Lee Avenue** | | |
| 9 | T-wall | 65+00 to 90+27 | 20A | I-wall | 92+20 to 93+46 |
| **Robert E. Lee Avenue** | | 90+27 to 91+88 | 20B | Levee | 93+46 to 101+50 |
| 10A | I-wall | 91+88 to 93+53 | 21 | Levee | 101+50 to 113+05 |
| 10B | Levee | 93+53 to 98+70 | | | |
| 11 | Levee | 98+70 to 112+50 | | | |

The MOWL for each I-wall and levee reach is tabulated in Table 1-2 and is compared to design criteria using each of the following individual analysis protocols: 1) stability using Spencer's Method; 2) stability using the Method of Planes; 3) minimum sheet pile penetration; 4) sheet pile penetration ratio; 5) maximum water level on exposed wall; 6) sheet pile wall stability; and 7) seepage. The elevations in bold identify the controlling criteria in areas where the calculation results were below El 10 NAVD88.

Stability for the I-wall and levee reaches was the controlling condition for the lowest MOWL identified on both banks of the canal. The factor of safety (FOS) calculated by the Spencer's Method analysis for Reaches 10B, 17, 18A, and 20B are slightly less than the required 1.4 with the canal water level at El 1 NAVD88, the normal Lake level. This indicates an inadequate FOS without the influence of the canal water load. These low FOS values resulted from the low undrained shear strength values for the levee embankment and underlying marsh clay stratum. Reach 18A was designed with a protected side stabilization berm extending approximately 90 feet from the I-wall. The recent topographic survey performed in 2010 indicates the berm extends only about 30 feet beyond the I-wall. The planned berm extending approximately 90 feet from the I-wall apparently was never constructed. The MOWL values for Reaches 12A, 12B, 15, 16, 18B, and 19 varied from El 5.0 to 8.5 NAVD88 for the limiting 4-feet criteria.

Reaches 10A, 13A through 16 and 20A, do not meet the minimum sheet pile penetration requirement of 10 feet. The penetration ratio limits the MOWL of Reach 20A to El 9.2 NAVD88. Limiting the water level to 4 feet on the wall above the earthen levee crest limits the MOWL to below El 10 NAVD88 for Reaches 1A, 1B, 1C, 1D, 2 4, 17, 18A 18B, and 19. The lowest MOWL is El 7.2 NAVD88 in Reach 2. The crest of the levees in Reaches 12A and 12 B are El 7.6 and 8.5 NADV88, respectively. There is a concrete wall in Reach 12A with crest grade El 9.7 NAVD88.

Seepage was found to be a controlling condition for reaches 1A, 1B, 1C, 12A, 13A, 13B and 20A. For all except Reach 20A the seepage model assumed barrier beach sand in the bottom of the canal. For Reach 20A, the seepage model was based on a layer of hydraulic sand fill in the base of the canal. Reaches 12A and 12B were also assumed to have barrier beach sand in the bottom of the canal. Although there was sand in the bottom of the canal, the seepage FOS for Reaches 12A and 12B met the minimum requirement at canal water levels of El 9.7 and 8.5 NAVD88, respectively, the maximum possible MOWL values for these reaches based on crest grades.

The MOWL values for the T-walls were controlled by wall estimated deflections. Reaches 7 and 8 had MOWL values of El 8.0 and 7.0 NAVD88, respectively. For Reach 8, the deflection at a canal water elevation of El 8 NAVD88 was estimated at about 1.6 inches. The maximum allowable value is 0.75 inch. The magnitude is very sensitive to a 1-foot variation in water level. Therefore, more detailed finite element method (FEM) analysis of this reach will be conducted during the remediation phase with other deficient reaches to further define the deflection magnitude. A

MOWL was not provided for the bridges as the bridges are not part of this study and the local geometry at the bridges would not limit or constrain the MOWL.

Table 1-3 provides a summary of the FOS values and deflections for the T-wall and FOS values for DPS 7. Figures 7-3 through 7-7 in the body of the text provide the calculated MOWLs for each criterion along east bank of the canal. Similarly, Figures 7-8 through 7-14 in the body of the text provides calculated MOWLs for each criterion along the west bank of the canal.

## TABLE 1-2
## REACH MOWL VALUES FOR I-WALLS AND EARTH LEVEES

| REACH | STATION | SPENCER'S METHOD SLOPE STABILITY FOS >1.4 MOWL NAVD88 | MOP SLOPE STABILITY FOS >1.3 MOWL NAVD88 | MINIMUM SHEET PILE PENETRATION D>10 FEET | SHEET PILE PENETRATION RATIO $D/H_1 = 3/1$ MOWL (NAVD88) | MAXIMUM 4 FT WATER DEPTH ON I-WALL MOWL NAVD88 | CWALSHT MOWL NAVD88 | SEEPAGE MOWL NAVD88 |
|---|---|---|---|---|---|---|---|---|
| 1A * | 2+45 to 7+00 | 10.0 | 10.0 | Yes | 10.0 | 7.6 | 10.0 | **6.5** |
| 1B * | 7+00 to 9+25 | 10.0 | 10.0 | Yes | 10.0 | 7.6 | 10.0 | **5.0** |
| 1C * | 9+25 to 11+00 | 10.0 | 10.0 | Yes | 10.0 | 7.6 | 10.0 | **5.5** |
| 1D | 11+00 to 14+20 | 10.0 | 10.0 | Yes | 10.0 | **7.6** | 10.0 | 10.0 |
| 2 | 14+20 to 21+75 | 10.0 | 10.0 | Yes | 10.0 | **7.2** | 10.0 | 10.0 |
| 4 | 24+87 to 29+16 | 10.0 | 10.0 | Yes | 10.0 | **7.3** | 10.0 | 10.0 |
| 10A | 91+88 to 93+53 | 10.0 | 10.0 | **No** | 10.0 | 10.0 | 10.0 | 10.0 |
| 10B | 93+53 to 98+70 | **<1.0** | **1.0** | Levee - No Sheet Pile, Crest above El 10.0 NAVD88 | | | | 10.0 |
| 11 | 98+70 to 112+50 | 10.0 | 10.0 | Levee - No Sheet Pile, Crest above El 10.0 NAVD88 | | | | 10.0 |
| 12A * | 2+45 to 3+70 | **8.5** | **8.5** | Levee with wall Crest, El 9.7, Analysis, El 8.5 NAVD88 | | | | **8.5** |
| 12B * | 3+70 to 4+70 | **8.5** | **8.5** | Levee - No Sheet Pile, Crest, El 8.5 NAVD88 | | | | **8.5** |
| 13A * | 4+70 to 7+00 | 10.0 | 10.0 | **No** | 10.0 | 10.0 | 10.0 | **7.0** |
| 13B* | 7+00 to 11+20 | 10.0 | 10.0 | **No** | 10.0 | 10.0 | 10.0 | **6.5** |
| 14 | 11+20 to 20+50 | 10.0 | 10.0 | **No** | 10.0 | 10.0 | 10.0 | 10.0 |
| 15 | 20+50 to 30+00 | 10.0 | **8.5** | **No** | 10.0 | 10.0 | 10.0 | 10.0 |
| 16 | 30+00 to 36+40 | 10.0 | **<8.0** | **No** | 10.0 | 10.0 | 10.0 | 10.0 |
| 17 | 37+44 to 50+00 | **<1.0** | **1** | Yes | 10.0 | 9.5 | 10.0 | 10.0 |
| 18A | 50+00 to 61+00 | **<1.0** | **1** | Yes | 10.0 | 9.9 | 10.0 | 10.0 |
| 18B | 61+00 to 64+00 | 8.0 | **7.0** | Yes | 10.0 | 9.4 | 10.0 | 10.0 |
| 19 | 65+00 to 90+62 | **5.0** | **5.0** | Yes | 10.0 | 8.0 | 10.0 | 10.0 |
| 20A-E | 92+20 to 93+46 | 10.0 | 10.0 | **No** | 9.2 | 10.0 | 10.0 | **9.0** |
| 20B-E | 93+46 to 101+50 | **<1.0** | **<1.0** | Levee - No Sheet Pile, Crest above El 10.0 NAVD88 | | | | 10.0 |
| 21-E | 101+50 to113+05 | 10.0 | 10.0 | Levee - No Sheet Pile, Crest above El 10.0 NAVD88 | | | | 10.0 |

* Indicates reach with canal bottom assumed to be in barrier beach sand

## TABLE 1-3
## REACH MOWL VALUES FOR T-WALLS

| Reach | Station | Canal Water Elevation NAVD88 | Spencer's Method FOS W/O Unbalanced Loads | MOP FOS W/O Unbalanced Loads | Calculated Deflection with Unbalanced Loads (in) | Lane WCR MOWL NAVD88 |
|---|---|---|---|---|---|---|
| 3 | 21+75 to 24+87 | 10.0 | 1.85 | 1.63 | 0.04 | 10 |
| 5 | 29+16 to 42+00 | 10.0 | 1.30 | 1.12 | 0.39 | 10 |
| 6 | 42+00 to 50+00 | 10.0 | 1.25 | 1.15 | 0.61 | 10.0 |
| 7 | 50+00 to 59+00 | **8.0** | 1.30 | 1.22 | 0.46 | 10.0 |
| 8 | 59+00 to 63+58 | **7.0** | 1.34 | 1.26 | 0.46 | 10.0 |
| | | **8.0** | 1.26 | 1.18 | 1.54 | 10.0 |
| 9 | 65+00 to 90+27 | 10.0 | 1.37 | 1.39 | 0.07 | 10.0 |
| | | | **Spencer's Method FOS** | **MOP FOS** | | |
| DPS 7 | | 10.0 | 2.36 | 1.67 | | |

The analyses in this report indicate that some reaches along the Orleans Avenue Canal have MOWL values lower than the present MOWL of El 8 NAVD88. Any reach with a MOWL below El 8 NAVD88 will be remediated expeditiously based on the most stringent criteria and will follow rigorous methods of analysis. The remainder of this report goes into significant detail to explain the technical aspects of the analyses performed and how engineering judgment was applied as needed. In the next phase, the Corps will pursue further analyses to ensure that the solution selected for the improved levee section fully meets all necessary requirements.

# *2.0 INTRODUCTION*

## 2.1 HURRICANE KATRINA

Hurricane Katrina (Katrina) moved over the New Orleans (City) area in the early morning hours on Monday, August 29, 2005. The storm surge, in advance of the hurricane, caused the water level in Lake Pontchartrain (Lake) to ultimately rise 10 to 12 feet [1] above the normal level of El 1.0 NAVD88. All elevations in this report reference the North American Vertical Datum of 1988 (2004.65) (NAVD88) unless the National Geodetic Vertical Datum of 1929 (NVGD) is indicated. It is noted that El 0 NAVD88 is equivalent to El 1.5 NGVD. Prior to Katrina, the maximum surge level recorded on the south shore of the Lake was about El 4.0 NAVD88. The maximum rainfall from Katrina was 14 inches over a 24 hour period along the south shore of the Lake. The largest previously recorded rainfall during a 24 hour period was 7 inches [1]. References cited in this report are included in Section 9.0.

## 2.2 THE OUTFALL CANALS

Three outfall canals, the London Avenue Canal, the 17th Street Canal, and the Orleans Avenue Canal, provide discharge of surface water collected from the City storm-runoff systems. The City has been subsiding for many years and continues to subside due to: 1) confinement of the Mississippi River by levees, thus eliminating river sedimentation during high river flows; and 2) pumping of ground water. Since much of the City is now located below sea level, precipitation that falls on the City must be pumped up into the canals for discharge to the Lake. Flow of water from the City is initiated towards the Lake by gravity as the pumping causes the hydraulic grade line to rise. The canals were designed as open canals at the north end along the Lake at the time Katrina occurred. Because of the increase in Lake water level during Katrina, the fact that the canals were open allowed the storm surge to flow into the canals, causing the water levels to rise to levels that had not previously been experienced. The locations of the three outfall canals are shown on Figure 2-1. A general description of the outfall canals follows.


**FIGURE 2-1**
**LOCATION OF OUTFALL CANALS [1]**

- **17th Street Outfall Canal** – The 17th Street Canal is located in Jefferson Parish immediately west of the boundary with Orleans Parish. The canal extends north about 2.2 miles from Drainage Pump Station No. 6 (DPS 6), located near Interstate Highway I-10, to discharge at the Lake. The parallel protection system consists of a low levee and an I-wall on both sides of the canal. The I-wall that breached during Katrina was replaced with a T-wall.

- **Orleans Avenue Outfall Canal** – The Orleans Avenue Canal is located to the east of the 17th Street Outfall Canal in Orleans Parish. The canal extends north about 2.4 miles from Drainage Pump Station No.7 (DPS 7), located near I-610, to discharge at the Lake. The parallel protection system consists of a low levee and I-walls on both sides of the canal. In some reaches, T-walls were used to provide flood protection. No failures of the parallel protection system occurred along the Orleans Avenue Canal during Katrina.

- **London Avenue Outfall Canal** - The London Avenue Canal is located east of the Orleans Canal and west of the Inner Harbor Navigation Canal (IHNC). The canal extends about 2.6 miles from Drainage Pump Station No. 3 (DPS 3) to discharge at the Lake. The parallel protection system consists of a low levee and an I-wall on both sides of the canal. The I-walls that breached during Katrina were replaced with T-walls and the I-wall that failed as the result of excessive deflection was replaced with an L-wall.

## 2.3 PURPOSE OF REPORT

This report was prepared to reevaluate existing conditions and to identify areas in need of rehabilitation. This report is intended to provide a basis to pursue required improvements to the I-walls (or other components of the parallel protection system) along the Orleans Avenue Canal. The purpose of this report is to document the methodology and conclusions of actions taken to determine the Maximum Operating Water Level (MOWL) for the existing floodwalls and levees of the Orleans Avenue Canal in accordance with the criteria and methods of the guidance documents of the U.S. Army Corps of Engineers (Corps) developed specifically for the Hurricane and Storm Damage Risk Reduction System (HSDRRS). The MOWL was formerly termed the Safe Water Elevation (SWE) in other Corps documents. The MOWL is defined as the elevation of water in the canal where the canal levees and floodwalls meet the stability requirements, sheet pile penetration requirements, and seepage control requirements identified in the project criteria.

## 2.4 ENHANCED QA/QC OF SUPPORTING DATA AND PEER REVIEW OF THIS REPORT

In some cases, additional field and laboratory testing was performed to support the calculations presented in this report. Enhanced quality assurance and quality control (QA/QC) of field and laboratory test procedures were performed for the new data developed for this report. Rigorous internal and external peer review of analyses supporting this report and of the report text and appendices were performed by the Independent Technical Review (ITR) Team consisting of personnel from the following organizations.

- Geotechnical Engineers from the Mississippi Valley Division (MVD) including some members of the MVD Geotechnical Criteria Applications Team (GCAT);

- Geotechnical Engineers from the State of Louisiana Office of Coastal Restoration (OCPR); and
- Geotechnical Engineers representing the Southeast Louisiana Flood Protection Authority–East (SLFPA–E).

Most of the reviewers have been associated with the intensive investigations and evaluations in the aftermath of Katrina and brought significant experience and expertise to the review process.

This report and appendices were initially prepared for the Corps by ECM-GEC, a Joint Venture and subconsultant Black and Veatch Special Projects Corporation (B&V). The report was edited by ECM-GEC with the assistance of Ray Martin, Ph D., P.E., of Ray Martin, LLC and Robert Bachus, Ph.D., P.E., of Geosyntec Consultants for the HPO. The analyses performed by B&V, included in the Appendices of the edited report, were not reviewed in detail by Drs. Martin and Bachus and they are therefore not responsible for the content of these appendices except to the extent covered in peer review process by the ITR Team where spot checks of the data and analyses were performed.

# *3.0 HISTORY OF OUTFALL CANALS*

An 1878 map [15] of the City indicates all three canals were in existence by that time. In 1915 and 1947 the low levees along the canals were raised in response to overtopping by hurricanes and settlement of the canals [3]. The storm surge along the south shore of the Lake was estimated at El 4.0 NGVD88 for the 1947 hurricane. In 1955 the Congress authorized the Corps to study methods of containing hurricane storm surge such that it would not overtop the outfall canals and the Lake front levees. In 1960 the Corps proposed installing gates at the location of the discharge of each canal into the Lake. The Orleans and Jefferson Parish Levee Boards and the Sewerage & Water Board of New Orleans were partners with respect to funding of these projects and were also responsible for the operation of the canals. Opposition delayed this proposed modification [3]. In 1965 the Corps warned that the levees flanking the outfall canals were inadequate in terms of grade and stability. Finally, in 1985 the Corps was authorized to study two alternative approaches to provide hurricane storm surge protection for the outfall canals. The alternatives were to provide: 1) gated structures at the canal entrances; and 2) a parallel protection system consisting of flood walls. After an extended debate between the various parties to the project, Congress mandated construction of the parallel protection system alternative in 1992 [1].

## 3.1 STANDARD PROJECT HURRICANE AND DESIGN TOP OF FLOOD WALLS

The 1959 Standard Project Hurricane (SPH) [1] parameters, which were based on historic hurricanes covering a period of 57 years from 1900 to 1956, were used by the Corps to design the Lake Pontchartrain and Vicinity project including the outfall canals. This SPH was considered to have a recurrence interval of 100 years [1]. The Corps developed the criteria for design of the outfall canals after authorization by Congress in the Flood Control Act of 1965.

The design water surface for each canal was established based on the 1959 SPH. The SPH indicated that the Lake water surface on the south shore would be El 10.0 NAVD88. Beginning with this Lake water level, the Corps used the HEC-2 Water Surface Program [1] to calculate the water levels in the three outfall canals. Waves were not considered a significant issue due to the canal entrance conditions. The design tops of flood walls were set between El 11.5 and 13.5 NAVD88, based on this analysis [1]. After Katrina the top elevations of the I-walls were found to be up to 1 to 2 feet lower than the original elevations at which they were constructed, resulting in less protection than had been planned [1].

## 3.2 OUTFALL CANAL FAILURES

The storm surge from Katrina caused one failure along the 17th Street Canal and two failures along the London Avenue Canal. Figure 3-1 illustrates the locations of the outfall canal failures. The Orleans Avenue Canal levees and flood walls did not fail. The 17th Street Canal failed south of the Old Hammond Road Bridge near the north end of the canal between about 6:00 and 9:00 AM on August 29, 2005 [1]. A 400-ft long section of the east I-wall failed between Stations 560+50 and 564+50 when the water level in the canal was at about El 7 NAVD88, or about 5.5 feet below the top of the I-wall at the time of failure. The water level in the canal prior to Katrina was about El 3.0 NAVD88 and it ultimately rose to a maximum level of about El 9 NAVD88 during Katrina. It is believed that the failure occurred when a gap formed between the sheet pile wall, supporting the I-wall, and levee soil on the flood side of the I-wall. This gap allowed canal water to fill the space between the sheet pile and the levee soil down to the tip of the sheet pile. Ultimately, a shear failure developed below the tip of the I-wall in the soft clay foundation soils. Figure 3-1 illustrates the locations of the outfall canal failures.

The London Avenue Canal failed in two locations between 6 and 8 AM on August 29, 2005. The first failure occurred between 6 and 7 AM along the east I-wall north of Mirabeau Avenue and has been designated the south breach. This breach was about 60 feet long, but the I-wall deflected outward over a length of about 210 feet between Stations 70+40 and 72+50. Based on estimates of the storm surge, the water level in the canal was rising during the failure and ranged from about El 7 NAVD88 initially to about El 8 NAVD88 when this failure was complete. The second failure occurred between about 7 and 8 AM south of Robert E. Lee Avenue along the west I-wall and was designated the north breach. This breach was about 410 feet long and occurred between Stations 114+00 and 118+10. Based on estimates of the storm surge, the water level was at about El 8 NAVD88 when this failure initiated and was at about El 9.5 NAVD88 when the failure was complete. The east I-wall opposite the north breach tilted significantly but did not breach between about Stations 116+50 and 119+00. It is believed that these failures were also caused by the formation of a gap along the flood side of the sheet pile walls. The tips of the sheet pile walls along the London Avenue Canal were underlain by a sand layer. When the gap extended to the sand layer the water pressure from the canal caused uplift failure in the marsh layer overlying the sand layer beyond the levee and catastrophic failure ensued.


**FIGURE 3-1**
**LOCATIONS OF ORLEANS AVENUE OUTFALL CANAL**

During Katrina, the flood walls and earth levees along the Orleans Avenue Canal experienced a high water level of El 11.1 NAVD88 as noted the IPET report [1]. As mentioned previously, there were no failures at any location along the Orleans Avenue Canal during Katrina.

## 3.3 POST HURRICANE KATRINA ACTIONS

Following Katrina, the Chief of Engineers at the Corps created the Interagency Performance Evaluation Task Force (IPET) of "distinguished---government, academic, and private sector scientists and engineers who dedicated themselves solely to---understand the behavior of the New Orleans HPS in response to Hurricane Katrina and assist in the application of that knowledge to the reconstitution of a more resilient and capable system" [1]. The following paragraphs summarize the IPET activities and findings as they relate to the three outfall canals.

The IPET was established by the Corps in October 2005 and consisted of 150 world class engineers and scientists. The IPET conducted an intensive investigation that helped to understand the performance of the New Orleans levees, floodwalls, and other system components during Hurricane Katrina. The IPET helped identify lessons learned from the failures so that these lessons could be used in the rapid repairs to the system and the repairs included in the long-term improvements. These lessons are also being incorporated into Corps policy and guidance.

The IPET investigation is recorded in the IPET Final Report, Volumes I – IX which was issued June 1, 2007 [1]. The report was titled "Performance Evaluation of the New Orleans and Southeast Louisiana Hurricane Protection System." Volume V of the report was subtitled "The Performance - Levees and Floodwalls," and discusses the forensic investigations conducted following Katrina.

Two other panels were established to review the work of the IPET. The Corps requested that the American Society of Civil Engineers (ASCE) establish an External Review Panel of equally distinguished individuals to provide continuous peer review of the IPET work and to provide a summary report. The report of findings was published by ASCE [16, 17]. The second panel was requested by the Assistant Secretary of the Army for Civil Works and was established under the auspices of National Academy of Engineering - National Research Council (NRC). The NRC established the Committee on New Orleans Regional Hurricane

Protection Projects. The purpose was to "provide strategic oversight of the IPET and to make recommendations concerning hurricane protection in New Orleans." [1]

The ASCE published various papers authored by others in a special ASCE Geotechnical and Geoenvironmental Engineering Journal issue dedicated to the performance of the flood protection structures during Katrina [2]. Other professional groups, including the Independent Levee Investigation Team from the University of California at Berkeley (ILIT) [3], performed investigations and submitted reports to the Corps.

### *3.3.1 IPET Findings*

One of the most surprising elements of the failures along the 17th Street and London Avenue Canals was that they occurred before water overtopped the I-walls during the rise in canal water levels resulting from the hurricane surge on the Lake. Volume V of the Final IPET Report [1] dated June 1, 2006 discusses the investigations conducted following Katrina to develop an understanding of the failure mechanisms. The IPET attributed the failures along these canals to the following specific causes:

- As the water levels rose above the crest of the levees in the canals, gaps formed between the sheet piles supporting the I-walls and the soils on the flood side of the levee embankments. Water filled these gaps, increasing the water loads on the walls and reduced the stability factor of safety of the I-walls. The formation of the gap was observed in centrifuge model tests and finite element soil-structure interaction analyses.

- The marsh clay foundation soils were essentially normally consolidated beneath the levee slopes and beyond the toes of the levees. In these areas, the undrained shear strength of the clays was lower than under the crest of the levee which had been loaded to higher effective stresses as the result of the levee embankment fill. This variation in undrained shear strength was found to be an important factor in the evaluation of the stability of the levees. Failure to account for this shear strength variation in the marsh clays likely resulted in the failure of the I-wall along the 17th Street Canal.

- Where the I-wall sheet pile penetrated through the marsh clays into the sands, the open gap on the canal side of the sheet pile allowed the full hydrostatic head of the canal water to pressurize the sands. This resulted in high uplift pressures, increased hydraulic exit gradients at the ground surface, and the potential for piping at the toe of the levees on the protected side. Failure to account for this pressurizing of the sand layer likely resulted in the failures and tilt of the I-walls on the London Avenue Canal.

Following Katrina, the Corps took several actions to protect the outfall canals against future storm surges until a final plan could be developed to correct any remaining deficiencies of the HPS. These measures are described in the following paragraphs.

### *3.3.2 Interim Safe Water Elevations*

Following the failures along the 17th Street Canal and the London Avenue Canal, the Corps established interim MOWL for each of the three outfall canals:

- London Avenue Canal: El 5 NAVD88;
- Orleans Avenue Canal: El 8 NAVD88; and
- $17^{th}$ Street Canal: El 6NAVD88

These restrictions were intended to limit canal operating water elevations on the parallel protection structures (i.e. levees and I-walls) until further engineering studies could be completed to establish the MOWL for each canal

### *3.3.3 Interim Closure Structures*

The Corps also decided to construct Interim Closure Structures (ICSs) on the outfall canals at their confluence with the Lake to protect the canals against storm surges during tropical and extra-tropical events. Each ICS included gates and pump stations. The interim pump stations were sized with sufficient capacity to provide continuity of operations with the interior drainage pump stations for each canal. The ICS for the Orleans Avenue Canal was completed on June 1, 2006.

### *3.3.4 Design of Outfall Canals to Withstand a Maximum Operating Water Level of El 8 NAVD88*

In 2010 the MVN Corps made the decision that the I-wall levee parallel protection systems along each of the canals would be remediated to withstand a MOWL of El 8 NAVD88. This is a much more desirable MOWL from an operational perspective than the interim safe water levels on the London Avenue and the 17th Street Canals. This decision was made given that permanent closure structures and pump stations are planned to replace the existing ICS at the mouths of the canals. The permanent pump stations will operate in tandem with the existing local drainage pump stations. The closure structures will remain open under normal weather conditions; however, during significant tropical and extra-tropical events the gates will be closed, and the canals will function as conduits for the flow of runoff pumped from the city. Design of the improvements to the parallel protection systems for all canals to achieve a MOWL of El 8 NAVD88 is presently underway.

# 4.0 PROJECT GUIDELINES AND METHODOLOGY

The changes incorporated into the analyses of the parallel protection systems for each canal have been modified since Katrina, based on lessons learned from the canal failures. Concurrent with the IPET investigation, and assisted by several IPET members, the Corps developed a series of design guidelines [4] to: 1) provide consistency for the new designs, 2) enhance the current engineering criteria, and 3) incorporate the most current engineering standards and analysis guidelines related to use of state-of-the-practice methods of analysis. Spencer's Method for slope stability analyses and finite element seepage analyses are now routinely used by the Corps as a result of the IPET findings and recommendations. The required FOS for use with Spencer's Method was also increased from 1.3 to 1.4. The new guidelines are intended to be integrated into process that will result in parallel protection systems that are both resilient and robust.

Evaluations of the current MOWL of the Orleans Avenue Canal I-wall levee and T-wall levee parallel protection systems utilized the methodologies specified in the Hurricane and Storm Damage Risk Reduction System Design Guidelines (HSDRRSDG) [4]. A second document titled Stability Analysis of I-Walls Containing Gaps between the I-Wall and Backfill Soils [7] modifies the method previously specified in the Interim HSDRRSDG for: 1) determining the I-wall gap depth; and 2) performing the Spencer's Method stability analysis.

The application of the guidance documents to analysis of the I-walls and T-walls for this project were reviewed at various meetings attended by B&V, the ITR Team and the Corps during 2007 through 2010. These meetings were held to refine the guidance to this specific project, to reconcile differences in the application of the guidance to analyses performed and to review comments on draft reports. Specific parts of the recently revised guidelines identified, discussed, and agreed to by the Corps related to the gap propagation, piping analyses and modification of the heave analysis when finite element seepage analyses are performed. A detailed description of each guideline and how it was applied to this project is discussed in subsequent sections of this report.

## 4.1 SHEAR STRENGTH VERSUS DEPTH RELATIONSHIPS

For the purpose of this report, shear strength versus depth relationships are termed "strengthlines." These relationships are used for the analysis of individual reaches. The data used to develop strengthlines were obtained from the following references.

- Lake Pontchartrain, Louisiana and Vicinity, High Level Plan, Design Memorandum No. 19 - General Design, Orleans Avenue Outfall Canal, Volumes 1, 2 and 3 [6] includes investigations performed through 1985;

- IPET Report, Volume 5 [1]; and

- Additional investigations [10] performed by the Corps in 2006 and 2010 as described herein.

## 4.2 SURVEYS

Surveys of the canal were performed during June 2010 [12]. These consisted of bathymetric and topographic surveys on the east and west sides of the canal from DPS 7 at the south end of the canal to Reach 4 on the west side of the canal and Reach 18B on the east side of the canal. The cross sections for the remainder of the reaches were obtained from "as built" drawings [11].

## 4.3 MAXIMUM SAFE WATER ELEVATIONS

### *4.3.1 Guideline*

It was agreed during a meeting with the Corps on May 4, 2009 that MOWLs up to El 10 NAVD88 were to be evaluated. As referenced previously, the term MOWL is intended to replace the Safe Water Elevation (SWE).

### *4.3.2 Methodology*

Where analysis results for existing I-walls meet or exceed the El 10 NAVD88 criteria, no additional effort was to be made to determine the MOWL. Where analysis results for the existing I-walls indicate that a reach does not meet the El 10 NAVD88 criterion, the critical MOWL for that reach was reported along with the controlling criteria (e.g., stability, sheet pile penetration, seepage, etc.) that resulted in the lowest calculated MOWL. The maximum water level in the canal will be controlled by the operation of the pump stations and gates. The analysis results presented in this report indicate that some reaches along the Orleans Avenue Canal have MOWL values lower than the present MOWL of El 8 NAVD88. Any reach with a MOWL below El 8 NAVD88 will be remediated.

## 4.4 I-WALLS - HEIGHT, MINIMUM SHEET PILE PENETRATION, AND MINIMUM SHEET PILE PENETRATION RATIO

### *4.4.1 Guidelines*

The design and configuration of I-walls is defined in the HSDRRSDG [4]. Article 3.2.1 indicates that I-walls are limited to a total height above grade on the protected side (H) of 4 feet (Figure 4-1). The height H is measured from the protected side levee crest. The guidelines provide additional requirements for a minimum sheet pile penetration (D) of 10 feet. The depth D is measured from the lowest crest grade, either on the flood side or on the protected side of the levee. The guidelines also indicate a minimum penetration ratio (D/H) of 3. The Corps' extensive experience with I-walls indicates that they perform well if they meet these criteria.



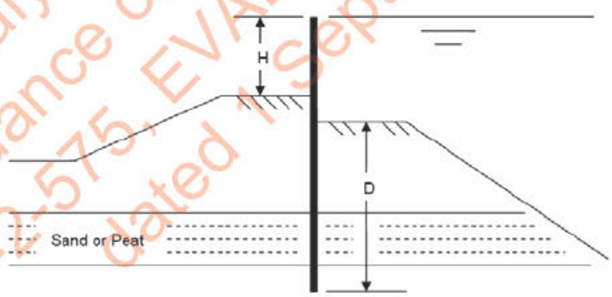


**FIGURE 4-1**
**SHEET PILE PENETRATION CRITERIA DEFINITIONS**

### *4.4.2 Methodology*

For the purposes of this report, existing I-walls were analyzed to a maximum canal water level of El 10 NAVD88, in lieu of the HSDRRSDG [4] requirement of the top of structure. The minimum sheet pile penetration ratio was checked using the height from the protected side levee crest to the water level on the wall ($H_1$), not the height to the top of the wall (H). The elevation where the canal water depth ($H_1$) = 4 feet is reported for reaches where this elevation is below El 10NAVD88.

## 4.5 I-WALLS - GAP ANALYSIS

### *4.5.1 Guidelines*

The GCAT document *Stability Analysis of I-Walls Containing Gaps between the I-Wall and Backfill Soils* [7] provides a methodology for the determination of the gap depth. This new method supersedes the methodology described in the HSDRRSDG. The depth of the gap determined using this methodology is relatively insensitive to the elevation of the water in the canal. The full potential gap depth was assumed to develop for both seepage and slope stability analyses when the canal water level exceeded the flood side levee crest by any amount.

The GCAT methodology does not provide guidance on the condition where the calculated gap depth approaches the top of the beach sand layer. The HSDRRSDG [4], Article 3.2.2.3, recommends the following:

*"If the computed gap is within 5 feet of the aquifer* [e.g., beach sand layer], *the crack shall be assumed to extend to the aquifer For specific cases where the geology of the foundation is well known and the designer is confident that the strata is more than 2.0 feet below the tip of the sheet pile, the crack shall extend only to the depth calculated. A well known geology shall have field investigations spaced closer than 100 feet."*

The GCAT guidelines suggest that the piezometric surface be determined from a finite element analysis assuming the maximum depth of the gap.

### *4.5.2 Methodology*

Discussions were held between the Corps and the ITR team at a meeting on October 7, 2009 to define the procedure to be used when the calculated gap depth approaches the top of the beach sand layer. Based on the results of that meeting it was decided to extend the calculated gap depth to the top of the beach sand layer if the calculated gap depth was within

3 feet of the top of the beach sand layer and is, therefore, more conservative than recommendations made by the GCAT.

## 4.6 I-WALLS - GLOBAL STABILITY

### *4.6.1 Guidelines*

Table 3.1, Article 3.1.2.2 of the HSDRRSDG [4] provides guidelines for the stability of I-walls. This table provides a requirement that Spencer's Method [5] of analysis is to be used as the primary analysis method and that the MOP [35] is to be used as a check. The HSDRRSDG assumes that the water level is at the top of the I wall.

### *4.6.2 Methodology*

The Corps required that the existing I-wall levee parallel protection system for each reach be analyzed using both Spencer's Method and the MOP during a meeting held on May 4, 2009. The GEO-SLOPE program SLOPE/W, Version 7.16 [34] was used to perform the Spencer's Method of analysis. The minimum factor of safety (FOS) for Spencer's Method was established as 1.4 and for the MOP as 1.3. For the analyses presented herein, the maximum canal water surface elevation will be limited to El 10 NAVD88, not top of the wall as stated in the HSDRRSDG guidelines

## 4.7 I-WALLS - FAILURE PLANE THROUGH SHEET PILE

### *4.7.1 Guidelines*

No guidelines were provided in the HSDRRSDG [4] as to where, or if, potential failure surfaces in a stability analysis can pass through the sheet pile. The GCAT guidelines do not allow penetration of a potential failure surface through the sheet pile for the gap analysis.

### *4.7.2 Methodology*

During a meeting held with the Corps on May 4, 2009 it was agreed that penetration of a potential failure surface through the sheet pile would not be permitted in the gap analyses. All potential failure surfaces in the gap analysis will be initiated at the sheet pile tip. To be consistent with the gap analyses, the sheet pile will be included in the global analyses. However, the Corps required that potential failure surfaces in the global analyses be allowed to penetrate through the bottom 5 feet of the sheet pile. While these two requirements are inconsistent, it is conservative to allow potential failure surfaces in the global analyses to penetrate through the bottom 5 feet of the sheet pile and both criteria were used for the analyses of the canal.

## 4.8 I-WALLS – WALL STABILITY

### *4.8.1 Guidelines*

Article 3.2.2.2 of the HSDRRSDG specifies the use of the Corps software CWALSHT to determine the required sheet pile tip penetration. Two cases using "Q" shear strengths are required: Case "a" cantilever wall and Case "b" bulkhead wall. One "S" shear strength case is required, and this is for the Case "b" bulkhead wall. This case is only performed on I-walls with differential fill depths on either side of the I-wall of greater than 2 feet.

### *4.8.2 Methodology*

Cases "a" and "b" were performed using the CWALSHT. Case "a" was evaluated using the MOWL of El 10 NAVD88 for deflection away from the canal, and case "b" was performed using the low water level of El -1 NAVD88 for deflection towards the canal. In all cases the analyses were performed by applying a FOS of 1.5 to the active and passive soil strengths. In accordance with Corps instructions, the CWALSHT analysis was performed using the "design" mode. Analyses were performed using the Fixed Surface Wedge Method and Sweep Search Wedge Method. The method producing the deeper design tip was then compared to the as-built tip elevations to evaluate suitability of the sheet pile penetrations.

## 4.9 I-WALLS - PIEZOMETRIC SURFACE

### *4.9.1 Guidelines*

The HSDRRSDG [4] require that the piezometric surface used in the stability calculation be in accordance with Corps Publications EM-1110-2-1913 [28] and DIVR 1110-2-400 [31]. The GCAT guidelines suggest that the piezometric surface be determined from a finite element analysis considering the maximum calculated depth of the gap.

### *4.9.2 Methodology*

The seepage analyses were performed using the GEO-SLOPE program SEEP/W, Version 7.16 [34]. The piezometric surface is critical to the stability analysis, especially in areas where a shallow sand layer may be exposed at the base of the canal on the flood side or when a gap is introduced. Piezometric surfaces obtained from these analyses were used for both the global and gap stability analyses and conservatively included the presence of a gap for both cases.

## 4.10 T-WALLS – EMBANKMENT STABILITY

### *4.10.1 Guidelines*

Table 3.1, Article 3.1.2.2, of the Interim HSDRRSDG [4] provides a methodology for the analysis of T-wall stability. The procedures require that the analyses consider two water levels in the canal: the design water surface elevation and water at the top of the T-wall. This methodology uses a Spencer's Method [5] of analysis and the transfer of unbalanced loads onto support piles.

### *4.10.2 Methodology*

The existing T-walls were not designed using the new T-wall criteria. The analyses included herein used the new T-wall criteria. The as-built drawings of the new walls were provided by the Corps. The as built pile configuration was analyzed using ENSOFT Group 7 Software [36], a program for the analysis of piles in a group.

The unbalanced load was determined using Spencer's Method of analysis utilizing the GEO-SLOPE program SLOPE/W, Version 7.16 [34]. The guidance document specifies that a global stability analysis be performed on the T-wall cross-section, with the assumption that the horizontal water load on the concrete portion of the T-wall be assumed to be supported by the T-wall foundation piles and not be part of the stability analysis. According to the HSDRRSDG [4] a FOS greater than 1.5 will not apply any soil loads to the T-wall foundation piles. T-walls were evaluated for a MOWL up to El 10 NAVD88.

## 4.11 PIPING ANALYSIS

### *4.11.1 Guidelines*

The piezometric surface used in piping analyses will be determined from a finite element analysis that is based on the gap analysis. The FOS to be used for underseepage/piping will be 1.6, in accordance with Article 3.1.4.3, Table 3.5(a) of the HSDRRSDG [4]. In discussions with the IRT team at a May 2010 meeting, it was agreed that the analysis for heave in accordance with Article 3.2.2.4 of the HSDRRSDG was no longer required, based on guidance developed by GCAT and approved by the Corp.

### *4.11.2 Methodology*

The seepage analyses were performed using the GEO-SLOPE program SEEP/W, Version 7.16 [34].

# *5.0 GEOLOGY*

The geology of the Orleans Avenue Canal area is very complex [1, 6, 14]. The near surface soils were deposited during Holocene time as the ocean rose after the last ice age. The following paragraphs present a brief description of regional and local geology.

## 5.1 PHYSIOGRAPHY

The Orleans Avenue Canal is located on the Mississippi River Delta Alluvial Plain which is the southernmost part of the Mississippi River Alluvial Plain. Specifically, the project is located on the southern edge of the Lake Pontchartrain Basin and east of the Mississippi River. The highest ground surface elevations in the area are located along the natural levees adjacent to Bayou Sauvage (also described as Bayous Metairie and Gentilly) which crosses the south end of the canal and along the Mississippi River. Elevations along the Bayou Sauvage natural levees are near -1.5 NAVD88 and along the Mississippi River natural levees vary from approximately El 8.5 to 13.5 feet NAVD88. In the lowest swamp and marsh areas the ground surface is as low as El -8.5 NAVD88. The lowest area along the canal is -7.4 NAVD88.

## 5.2 REGIONAL AND LOCAL GEOLOGY

At the close of the Pleistocene epoch, about 15,000 to 12,000 years before present, the sea level was approximately 360 to 400 feet below present sea level and the Mississippi River was entrenched into the old Pleistocene sediments that underlie the coastal Louisiana area. The elevation of the Pleistocene surface under the London Avenue Canal varies from about El -60 to -70 NAVD88. At the end of the Pleistocene epoch the ancestral Mississippi River valley was to the west of New Orleans in the area of Morgan City, LA and the Gulf of Mexico shoreline was located much farther to the south than it is today. Massive deposition of fluvial sediments occurred during the Holocene sea level rise in the broad alluvial valley of the ancestral Mississippi River. The local sediment deposition process included the following specific stages. The Holocene bay sound clays were deposited on top of the old Pleistocene surface as the sea level began to rise rapidly and inundated the New Orleans area. The Pine Island barrier beach sand formation was deposited above the bay sound clays about 4,000 to 5,000 years before present when the sea level was about 10 to 15 feet below current elevations. Figure 5-1 illustrates the estimated surface contours of the barrier beach in the area of the Orleans Avenue Canal. Note the surface of this barrier beach sand deposit is about El -10 NAVD88 at its highest elevation. Contours shown on Figure 5-1 are difficult to read, but are all below current sea level. The barrier beach formed a shoreline before the various Mississippi River deltas advanced toward the Gulf of Mexico. In some areas to the north of the barrier beach along the Orleans Avenue Canal, Holocene Lacustrine clays were deposited in a fresh water environment.

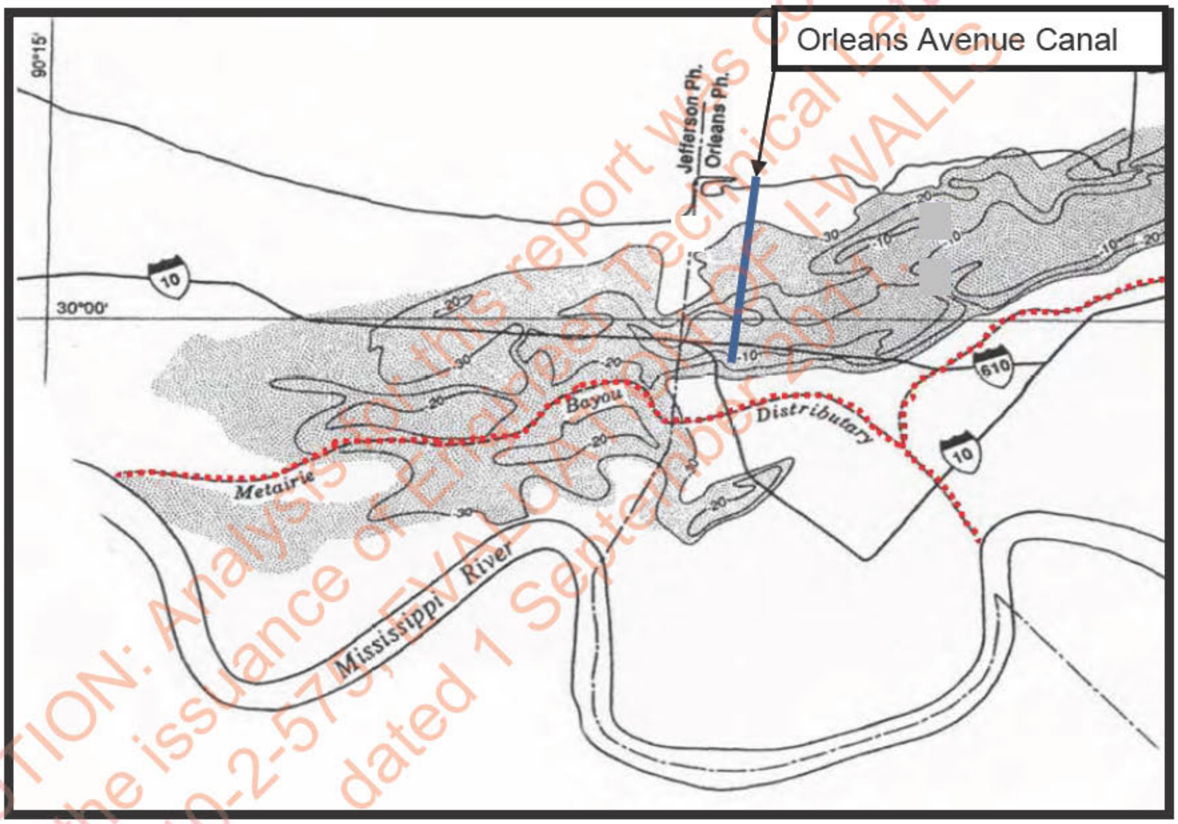


**FIGURE 5-1**
**PINE ISLAND BARRIER BEACH AND BAYOU SAUVAGE (METAIRIE) DISTRIBUTARY CHANNEL [1]**



Present day coastal Louisiana is the product of numerous, but generally short lived, delta systems that have been built seaward by deposition of Mississippi River fluvial sediments. Five major deltaic systems have built seaward during the past 7,000 years as the Mississippi River changed its course in the southern Louisiana area as shown in Figure 5-2. The Plaquemines/modern delta complex is the most recent. The next most recent was the

LaFourche delta complex which developed south and west of New Orleans. The St Bernard delta complex developed prior to the LaFourche delta complex and contained the Mississippi River and its distributary channels, which were responsible for depositing sediments in New Orleans area. The restriction of the Mississippi River sediment laden floodwaters to the river channel in the New Orleans area has resulted in the gradual degradation of the study area through subsidence.

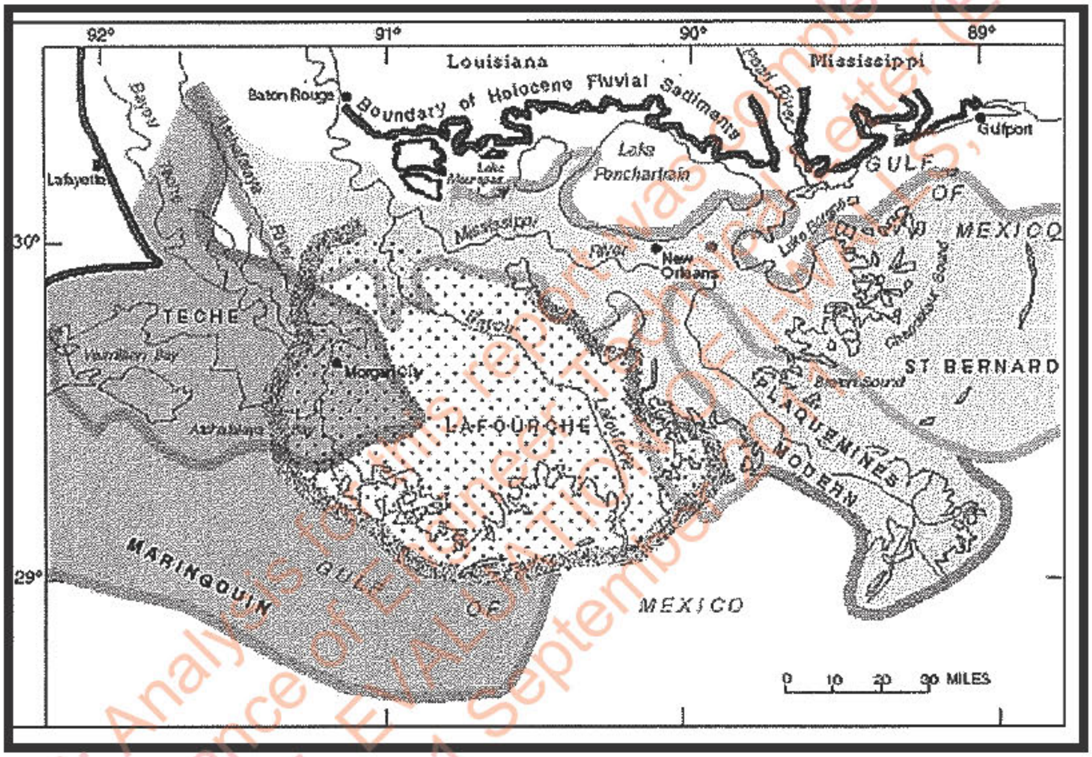


**FIGURE 5-2**
**HOLOCENE DELTAS OF THE MISSISSIPPI RIVER (14)**

The surficial clays and peat that make up the marsh and swamp deposits which overlie the Pine Island barrier beach sands and the older intradelta and prodelta deposits are part of the St Bernard delta complex. These sediments were deposited as recently as 800 years [23] ago mostly by the Bayou Sauvage distributary channel. A distributary channel originates from the main river channel and distributes water and sediment to the delta area thus expanding the delta. This distributary channel was located along the southern edge of the old Pine Island Barrier Beach. Natural levees developed on both sides of Bayou Sauvage as water flowed over the banks of the distributary channel during flooding. The natural levees in the Bayou Sauvage area consist of silts and lean and fat clays. Finer grained sediments were deposits beyond the natural levees in the marsh areas and are termed interdistributary deposits. Below the marsh deposits and natural levees are older intradelta and prodelta deposits. Intradelta deposits are typically more coarse grained higher energy deposits that formed when the distributary system was young. The prodelta deposits formed at the delta front and were laid down beneath the water surface before the distributary system fully developed. The stratigraphy shown on the Soil and Geologic Profiles and Cross Sections included in Appendix A.3, Plates 9 through 49, illustrate the formations described above.

# *6.0 GEOTECHNICAL CONSIDERATIONS*

The geotechnical data used in this study were obtained from Design Memorandum No. 19 [6] (DM 19), the IPET Report [1], and through additional investigations and laboratory testing performed during the period 2006 to 2010 [10]. The existing structures are presented first followed by a discussion of the geotechnical investigations. The subsurface conditions are then presented along with development of soil and geologic profiles and cross sections. This is followed by discussion of laboratory and in situ testing data, design permeability values, and design shear strength and unit weight values. The results of the London Avenue Canal I-wall Load Test (London Load Test) are presented in summary form as the findings from this study are relevant to the analyses performed for the Orleans Avenue Canal. Finally, the levee reaches developed from assessment of these data conclude this section.

## 6.1 EXISTING STRUCTURES AND GROUND SURFACE GRADES

The existing structures under consideration in this study include I-walls, T-walls, the tip elevations of the underlying sheet pile cutoff walls, a pump station, bridges and earthen levees. The existing ground surface grades of the canal levees and canal bottom and of the adjacent protected areas on both sides of the canal levees are also an integral part of the project. The following paragraphs briefly describe these features.

### *6.1.1 Floodwall Top Grades and Levee Crest Grades*

The existing I walls and T-walls along the levee crests were constructed in the 1990's to improve the parallel protection system and reduce the potential for flooding during hurricane events which cause the level of the water in the Lake to rise. No I-walls or T-walls along the Orleans Avenue Canal failed during Katrina. The top of I-wall and T-wall grades vary between El 12.5 and 14.4 NAVD88 throughout the length of the canal. These walls were analyzed for MOWL of El 10 NAVD 88, the maximum MOWL considered in this study. The earth levees without I-walls have crest grades ranging from 8.5 to 9.7 NAVD88 and evaluated for MOWL values equal to these elevations.

### *6.1.2 Sheet Pile Tip Elevations*

The I-walls and T-walls are each connected to subsurface sheet pile cutoff walls which are embedded in the base of the walls. The tip elevations of these sheet pile walls vary along the length of the canal due to variations in subsurface conditions. The sheet pile tip elevations and locations where they apply were obtained from "as-built" drawings [11] of the canal provided in Corps documents. Table 6-1 provides a summary of the original sheet pile tip elevations for the west and east sides of the canal. The table is arranged according to the original reaches defined in the "as built" drawings based on variations in sheet pile tip elevations. The tip elevations of the existing sheet piles for the I walls and T-walls are plotted on the centerline soil and geologic profiles provided in Appendix A.3.

**TABLE 6-1**
**ORIGINAL "AS-BUILT" REACHES [11]**

| WEST BASELINE APPROXIMATE STATION | PROTECTED SIDE LEVEE CREST ELEVATION (FT) NAVD88/WALL TYPE | SHEET PILE TIP ELEVATION (FT) NAVD88 | EAST BASELINE APPROXIMATE STATION | PROTECTED SIDE LEVEE CREST ELEVATION (FT) NAVD88/WALL TYPE | SHEET PILE TIP ELEVATION. (FT) NAVD88 |
|---|---|---|---|---|---|
| 2+45 to 21+44 | 3.5/I-wall | -28.5 | 2+45 to 4+50 | Levee | No piles |
| 21+44 to 24+83 | 3.3/T-wall | -21.5 | 4+50 to 36+18 | 8.0/I-wall | -1.3 |
| 24+83 to 29+12 | 3.5/I-wall | -28.5 | **Harrison Ave.** | | |
| 29+12 to 36+14 | 2.8/T-wall | -26.5 | 37+29 to 63+62 | 5.5/I-wall | -9.8 |
| **Harrison Ave.** | | | **Filmore Ave.** | | |
| 37+13 to 50+11 | 2.8/T-wall | -27.5 | 64+68 to 90+18 | 4.0/I-wall | -15.3 |
| 50+11 to 63+66 | 2.8/T-wall | -35.5 | **Robert E. Lee Ave.** | | |
| **Filmore Ave.** | | | 91+52 to 92+52 | 7.5/I-wall | -1.5 |
| 64+52 to 90+27 | 3.0/T-wall | -40.0 | 92+52 to 113+05 | Levee | No piles |
| **Robert E. Lee Ave.** | | | | | |
| 91+42 to 92+85 | 7.5/I-wall | -9.5 | | | |
| 92+85 to 112+50 | **Levee** | **No piles** | | | |

### *6.1.3 Pump Stations*

Drainage Pump Station No. 7 (DPS 7) is located at the south end of the Orleans Avenue Canal. The building was originally constructed in the late 1890s on essentially a mass brick foundation. The pump station has undergone several additions and modifications over the years. The discharge pipes for this pump station empty into the Orleans Avenue Canal in between reinforced concrete retaining walls that connect to the outside face of the building wall. These retaining walls were not considered in the structural stability analysis since there will be opposing loads on each side of the wall at the SWE. The exterior pump station mass masonry foundation wall was evaluated. Since original construction, various openings in the wall have been plugged with concrete. The wall was evaluated for a SWE of El 8.0 NAVD88.

The ICS consists of gated structures that are used to block surge from tropical storms and hurricanes, as well as other events that cause the level of Lake Pontchartrain to rise, from the canals and pumps that allow the S&WB to continue to pump water from the city from the rain event that will likely accompany a surge event. These structures were constructed to prevent failures of the floodwalls similar to those that occurred on the 17th Street and London Avenue Canals during Katrina. The ICS and pump station in the Orleans Avenue canal consists of five 11 x 10.25' wide gates with a flow-rate capacity of 12,500 cubic feet per second. The pumps used at the ICS consist of 10 MWI pumps with the power unit located on the engine platforms.

### *6.1.4 Canal, Levees and Protected Side Grades*

Surveys [12] of the canal were performed during June 2010 for west Reaches 1 through 4 and east Reaches 12 through 18B. The survey locations were based on initial plans to perform remediation work in these reaches. Levee cross sections were taken approximately every 200 feet along the baselines on each side of the canal within these reaches. The cross sections for the remaining reaches to the ICS were obtained from the "as-built' drawings [11]. Ground surface elevations were obtained along each cross-section at approximately 10-foot intervals and at all abrupt changes in grade. The cross-sections were generally extended 50 feet beyond the protected side toe of the levees on each side of the canal. Soundings were recorded at 20 foot intervals along each section within the canal. The survey was performed using a combination of geodetic levels and Real-Time-Kinematic (RTK) Global Positioning System (GPS). The survey report is included in Appendix C and the coordinates of the east and west canal baselines are included in Appendix E.

The average canal bottom width is about 100 feet and varies between about 80 and 120 feet. The top width of the canal averages about140 feet and varies between 120 and 160 feet. The canal bottom grade is relatively consistent across each section and ranges from about El -5 NAVD at the south end of the canal near DPS 7 to about El -12 NAVD near the Lake.

The topographic and hydrographic data were analyzed by grouping the levee cross sections based on similar topography. The analyses cross-section grades were created by using the lowest elevations on the protected side and the average elevations on the flood side, except for Reaches 1A, 12A and 13B. This resulted in more soil mass on the flood side and less soil mass on the protected side to make the slope stability analysis conservative for failures propagating from the flood side to the protected side. Within Reaches 1A, 12A and 13B, due to the potential for the barrier beach sand to be present in the bottom of the canal, the lowest elevation profile on the flood side was used for the analyses. The survey cross sections are included in Appendix A.3 on Plates 49 through 61. The plates note whether the levee cross-sections were developed from the survey [12] or from "as-built" drawings [11].

## 6.2 GEOTECHNICAL INVESTIGATIONS

The Corps initiated the field investigations along the Orleans Avenue Canal during the period 1970 to 1973 with the completion of 16 borings. During 1984 and 1985, a total of 61 borings were drilled for the development of DM 19 [6] which was issued in August 1988. Thus, a total of 77 borings were drilled along the canal prior to Katrina. These investigations were competed for design of the I-walls and T-walls to increase the parallel protection along the canal levees. Following Katrina in August 2005 additional borings, cone penetration tests (CPTs), vane shear tests (VSTs), and laboratory tests were performed for: 1) evaluation of the failures; 2) determination of MOWL and reaches in need of repair; and 3) design of remedial repairs. The following paragraphs describe these investigations.

### *6.2.1 Pre-Katrina Investigations*

A total of 64 test borings were drilled within reaches under consideration in this report for preparation of DM 19 prior to Katrina. The distribution of these borings along the canal is illustrated in Table 6-2. Twenty three borings were drilled along the protected side of the west levee and 14 borings were drilled along the protected side of the east level. Fourteen borings were drilled along the crest of the west levee and 26 along the crest of the east levee. Only three borings were drilled within the canal, all at the north end between the Robert E. Lee Bridge and the ICS. The borings are reasonably well distributed along the west levee at a rate of about one every 475 feet on the protected side and one every 800 feet along the crest. Along the east levee, the distribution along the crest was about one boring every 425 feet and on the protected side one boring every 800 feet.

The ground surface elevations shown on the boring logs for the older borings may not agree with current ground surface elevations due to subsidence and/or grading work that has occurred at the borings locations. The ground surface elevations at the locations of the recent borings discussed below generally agree with the ground surface elevations obtained during the recent survey performed for this study.



**TABLE 6-2**
**DISTRIBUTION OF TEST BORINGS**

| WEST AND EAST BASELINE STATIONS | WEST SIDE – PROTECTED SIDE – PRE-KATRINA | WEST SIDE – PROTECTED SIDE – POST-KATRINA | WEST SIDE – CREST – PRE-KATRINA | WEST SIDE – CREST – POST KATRINA | CANAL – PRE-KATRINA | CANAL – POST KATRINA | EAST SIDE – CREST – PRE-KATRINA | EAST SIDE – CREST – POST KATRINA | EAST SIDE – PROTECTED SIDE – PRE-KATRINA | EAST SIDE – PROTECTED SIDE – POST KATRINA |
|---|---|---|---|---|---|---|---|---|---|---|
| 2+45 to 5+00 | 1 | 0 | 1 | 0 | 0 | 0 | 1 | 0 | 1 | 0 |
| 5+00 to 15+00 | 1 | 4 | 1 | 1 | 0 | 0 | 3 | 0 | 4 | 4 |

| | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|
| 15+00 to 25+00 | 1 | 1 | 2 | 2 | 0 | 0 | 2 | 0 | 0 | 0 |
| 25+00 to 35+00 | 2 | 0 | 0 | 0 | 0 | 0 | 2 | 0 | 0 | 0 |
| 35+00 to 45+00 | 2 | 2 | 1 | 0 | 0 | 0 | 2 | 0 | 1 | 0 |
| 45+00 to 55+00 | 2 | 0 | 0 | 0 | 0 | 0 | 2 | 0 | 0 | 0 |
| 55+00 to 65+00 | 2 | 0 | 0 | 0 | 0 | 0 | 4 | 0 | 0 | 0 |
| 65+00 to 75+00 | 2 | 0 | 0 | 0 | 0 | 0 | 1 | 0 | 0 | 2 |
| 75+00 to 85+00 | 1 | 0 | 1 | 0 | 0 | 0 | 2 | 0 | 1 | 0 |
| 85+00 to 95+00 | 1 | 0 | 1 | 0 | 0 | 0 | 3 | 0 | 1 | 0 |
| 95+00 to 105+00 | 0 | 0 | 2 | 0 | 2 | 0 | 2 | 0 | 0 | 0 |
| 105+00 to 113+00 | 1 | 0 | 2 | 0 | 1 | 0 | 2 | 0 | 0 | 0 |
| **TOTALS** | **16** | **7** | **11** | **3** | **3** | **0** | **26** | **0** | **8** | **6** |

### *6.2.2 Post Katrina Investigations*

Following Katrina in August 2005, 16 test borings, 58 CPTs, and 3 VSTs were performed to evaluate the subsurface conditions along and within the canal.

***6.2.2.1** Borings*

Thirteen borings were drilled in 2006 and three direct push tubes were advanced in 2010. Seven borings were drilled along the west levee protected side toe and the three direct push borings were drilled along the crest to fill in data gaps. Six borings were drilled along the east levee protected side to fill in data gaps. No borings were drilled within the canal. A complete list of the 80 borings considered in this MOWL study is included in Appendix A.1, Table A.1-1. The boring locations are also plotted on Plates 1 through 8 of Appendix A3.

***6.2.2.2** Cone Penetration Tests*

A total of 58 CPTs were performed, 13 in 2006 and 45 in 2010. Seventeen CPTs were performed on the west levee centerline to fill in data gaps in the test borings advanced previously. Six CPTs were performed along the west levee protected side toe. Twenty one CPTs were advanced along the protected side toe of the east levee and 14 CPTs were performed along the crest of the east levee. The distribution of these CPT locations is summarized in Table 6-3. A complete list of CPT locations is included in Appendix A.1, Table A.1-2. The CPT locations are also plotted on Plates 1 through 8 of Appendix A.3.

#### *6.2.2.3 Vane Shear Tests*

Three VSTs were also completed in 2010, two along the protected side toe of the east levee and one along the centerline on the east levee. These tests were performed in the very soft to soft consistency levee fill soils and the marsh clays to estimate the undrained shear strength of these soils. The distribution of these VST locations is summarized below in Table 6-3. A complete list of VST locations is included in Appendix A.1, Table A.1-3. The VST locations are also plotted on Plates 1 through 8 of Appendix A.3. The field investigation logs, for the entire data set used in development of this study, are provided in Appendix E.

## 6.3 SUBSURFACE CONDITIONS

The following paragraphs provide a discussion of the subsurface conditions found throughout the length of the canal under consideration in this study. The information is presented beginning with the youngest and progressing to the oldest strata.



**TABLE 6–3**
**DISTRIBUTION OF CONE PENETRATION AND VANE SHEAR TESTS**

| WEST AND EAST BASELINE STATIONS | INVESTIGATION LOCATIONS | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|
| | WEST SIDE | | | | CANAL | | EAST SIDE | | | |
| | PROTECTED SIDE | | CREST | | | | CREST | | PROTECTED SIDE | |
| | CPTs | VSTs | CPTs | VSTs | CPTs | VSTs | CPTs | VSTs | CPTs | VSTs |
| 2+45 to 5+00 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 |
| 5+00 to 15+00 | 0 | 0 | 2 | 0 | 0 | 0 | 5 | 0 | 4 | 1 |
| 15+00 to 25+00 | 0 | 0 | 2 | 0 | 0 | 0 | 1 | 0 | 3 | 0 |
| 25+00 to 35+00 | 0 | 0 | 2 | 0 | 0 | 0 | 1 | 0 | 2 | 0 |
| 35+00 to 45+00 | 0 | 0 | 2 | 0 | 0 | 0 | 1 | 0 | 2 | 0 |

| | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|
| 45+00 to 55+00 | 0 | 0 | 2 | 0 | 0 | 0 | 0 | 0 | 1 | 0 |
| 55+00 to 65+00 | 1 | 0 | 2 | 0 | 0 | 0 | 0 | 0 | 2 | 0 |
| 65+00 to 75+00 | 0 | 0 | 2 | 0 | 0 | 0 | 3 | 1 | 2 | 1 |
| 75+00 to 85+00 | 0 | 0 | 2 | 0 | 0 | 0 | 2 | 0 | 0 | 0 |
| 85+00 to 95+00 | 1 | 0 | 1 | 0 | 0 | 0 | 0 | 0 | 1 | 0 |
| 95+00 to 105+00 | 2 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 2 | 0 |
| 105+00 to 113+00 | 2 | 0 | 0 | 0 | 0 | 0 | 1 | 0 | 2 | 0 |
| Totals | **6** | **0** | **17** | **0** | **0** | **0** | **14** | **1** | **21** | **2** |

### *6.3.1 Recent Canal Sediments*

Only three borings were drilled with the canal. These borings were located in the north end of the canal between Stations 101+00 and 106+00 considering both the east and west base lines. The recent canal sediments consisted of very soft consistency fat and lean clays and silts to depths of greater than 4.5 feet in these three borings.

### *6.3.2 Fill Clays and Hydraulic Fill Clays, Silts and Silty Sands*

Fill materials are present on both sides of the canal including the constructed levees and beyond the protected side toes. The fill varies from about 4 to 13 feet in thickness along the crests of the levees south of the Robert E. Lee Bridge. North of the Robert E. Lee Bridge a zone of hydraulic fill extends to much greater depths and the total thickness of fill varies from about 26 to 34 feet in thickness. Along the toe of the west levee the fill ranges from 1 to 5 feet thick south of the Robert E. Lee Bridge. Along the east levee fill was present at the levee toe from DPS 7 to about Station 30+00. Between Station 30+00 and the Robert E. Lee Bridge no fill was present. From the Robert E. Lee Bridge to the ICS both fill and hydraulic fill were present. The fill depths ranged from about 3 to 8 feet in thickness and the hydraulic fill ranged up to about 22 feet in thickness. Fill material consists of fat and lean clay with some organic matter and artificial fill materials. The hydraulic fill consists of lean clay, silt and silty sand with organic matter.

### *6.3.3 Marsh Clays*

Underlying the fill materials are swamp and marsh deposits. These materials have been identified herein as the marsh clay stratum. The marsh thickness varies from about 3 to 20 feet, but typically thicknesses range from about 10 to 14 feet. The thinnest area of the marsh clay is located at the southern end of the canal south of Stations 15+00. The base of the marsh stratum varies from about El -8 NAVD88 near DPS7 and declines to about El -20 NAVD88 near the Robert E. Lee Bridge. From the Robert E. Lee Bridge to the ICS the marsh clays are absent and hydraulic fill underlies the levee fills soils. The marsh clays have been compressed by the weight of the fill material used to construct the levees. Thus, they typically have a reduced thickness under the crests of the levees and tend to be thicker at the levee toes, assuming the cross section had a uniform marsh thickness prior to levee construction. The marsh clays are very soft to medium consistency fat clays with high moisture contents and occasional interbedded lenses of soft to very soft consistency lean clay, with occasional sand and silt layers, peat and wood.

### *6.3.4 Lacustrine Clays*

These predominately soft to medium consistency fat clays of lacustrine origin underlie the marsh clays north of Stations 45+00 to 50+00. The stratum varies in thickness from about 10 to 16 feet and slopes gradually downward from about El -20 NAVD88 near Stations 45+00 to 50+00 to about El -35 NAVD88 near the ICS.

### *6.3.5 Barrier Beach Sands*

The barrier beach sand stratum underlies the marsh clay stratum from DPS 7 to Stations 45+00 to 50+00 North of Station 45+00 to 50+00 these beach sands underlie the Lacustrine clays. This sand is typically loose to very dense poorly graded sand but at some locations a layer of silty sand has been identified at the top of the beach sand. The beach sand varies in thickness from about 40 to 20 feet from DPS 7 to Station 40+00 and then thins to about 20 to 6 feet north of Station 60+00 to the ICS at the north end of the canal. The base of the stratum varies from about El -40 to -50 NAVD88.

### *6.3.6 Bay Sound Clays*

The bay sound clay stratum underlies the barrier beach sands and varies from about 12 to 20 feet in thickness. The stratum consists of soft to stiff consistency fat clays and lean clays with some silt and silty sand layers and shells. The base elevation of the bay sound clays varies from about El -60 NAVD88 to about -70 NAVD88.

### *6.3.7 Pleistocene Clays*

The older Pleistocene stratum underlies the younger bay sound clays. This stratum consists of stiff to very stiff consistency oxidized fat to lean clays interbedded with layers of dense to very dense sands. This is the bearing material for deep foundations in the New Orleans area and the formation extends to about El -500 to -600 NAVD88.

## 6.4 SOIL AND GEOLOGIC PROFILES AND CROSS SECTIONS

Soil and geologic profiles and cross sections have been developed from the subsurface investigation data set described previously and are included in Appendix A.4 [21]. Profiles were developed parallel to the direction of the canal at the toe and center line of the levees. Cross sections were developed perpendicular to the direction of the canal to represent the various subsurface conditions along the canal. These profiles and cross sections are provided on the following plates:

- Plates 9 through 16 – West Bank Centerline Soil and Geologic Profiles;
- Plates 17 through 24 – West Bank Toe Soil and Geologic Profiles;
- Plates 25 through 33 - East Bank Centerline Soil and Geologic Profiles;
- Plates 33 through 40 – East Bank Toe Soil and Geologic Profiles; and
- Plates 41 through 48 – Soil and Geologic Cross Sections A-A' through H-H'.

The cross section locations are shown on Plates 1 through 8 in Appendix A.3. The elevation of the top of the boring on the individual plates may not coincide with the levee section shown as the levee elevations vary within the reaches. The cross sections were developed using the protected side elevations for the various strata on the flood side since few borings were available on the flood side of the levees. In Reaches 11 and 21 the actual flood side strata elevations were used. The tip elevations of the original I-wall and T-wall sheet piles are plotted on Plates 9 through 16 and 25 through 33 in Appendix A.3.

The strata descriptions used on these plates, ordered from the youngest to oldest deposits, are presented below.

- Recent Canal Sediments – Fat clay, lean clay and silt;
- Fill - Fat and lean clay with some organic matter and artificial fill materials;
- Hydraulic Fill - Lean clay, silt and silty sand with organic matter;
- Marsh – Very soft to medium consistency fat clays with occasional interbeds of very soft to medium consistency lean clay and with occasional sand and silt layers, peat and wood;
- Lacustrine - Soft to medium consistency fat clays;
- Barrier Beach - Loose to very dense sands and silty sands;
- Bay Sound – Medium to stiff consistency fat clay and lean clay with some silt and silty sand layers and shells; and
- Pleistocene – Stiff to very stiff consistency oxidized clays interbedded with layers and lenses of dense to very dense sands.

## 6.5 LABORATORY AND IN-SITU TESTING

Laboratory testing data were obtained from DM 19 [6], the IPET Report [1], and recent testing performed for this study [10]. The following paragraphs summarize the information reported in these data sources.

### *6.5.1 Design Memorandum 19*

During preparation of DM 19 [6] laboratory testing was performed on selected samples obtained along the Orleans Avenue Canal. All collected samples were visually classified. Laboratory tests performed included the following:

- Visual classifications;
- Moisture content;
- Atterberg limits;
- Grain size distribution;
- Unconfined compression tests;
- Unconsolidated undrained compression tests;
- Consolidated undrained compression tests with pore pressure measurements;
- Consolidated drained compression tests; and
- Consolidation tests.

The results of laboratory testing varied substantially by soil type, location along the canal, and the depth. The values reported in DM 19 are included in Appendix E. The shear strength versus depth plots used in the design are included on Plates 39 and 40 of DM 19. The shear strength versus depth properties were defined for the following canal reaches:

- Station 0+00 to Station 90+50 – east and west side toe;
- Station 0+00 to Station 90+50 – center line of west levee;
- Station 0+00 to Station 90+50 - center line of east levee; and
- Station 90+50 to Lake.

The shear strength versus depth reaches were modified based on recent laboratory and in-situ testing and analyses.

#### *6.5.2 Recent Laboratory and In-situ Testing*

***6.5.2.1** Grain Size*

No grain size testing was performed for this MOWL study.

***6.5.2.2** Permeability*

No laboratory permeability testing was performed for this MOWL study

***6.5.2.3** Shear Strengths and Unit Weights*

Undrained shear strength data were obtained from: 1) laboratory testing of undisturbed samples performed during this study; 2) CPT and VST in-situ testing performed during this study; and 3) data presented in DM 19 [6]. Unit weight data obtained from laboratory testing of samples were supplemented by the unit weight data included in DM 19. The results of the laboratory testing are provided in Appendix E.

## 6.6 DESIGN PERMEABILITY VALUES

The permeability of the barrier beach sands and canal bottom sediments were recognized to be critical parameters that needed to be accurately estimated in order for the seepage analyses of the various reaches of the canal to represent the in-situ conditions. Although no laboratory testing was performed for this MOWL study, a pump test was conducted in 2006, to assess the permeability of the underlying beach sand stratum. Recommended permeability values to be used in this study were provided in a Memorandum [24] dated July 19 2009 and authored by Noah Vroman of the Corps Engineering Research and Development Center (ERDC). This memorandum was authored for the London Avenue Canal site, but the values were considered generally applicable to the Orleans Avenue Canal site. These estimated values are presented in Table 6-4. The recommendations include permeability values for the barrier beach sands and canal bottom sediments and the less critical marsh clay and bay sound clay strata, all of which are required for the seepage analyses of the various canal reaches.

**TABLE 6-4**
**ERDC RECOMMENDED LONDON AVENUE CANAL SITE MATERIAL PERMEABILITIES CONSIDERED APPLICABLE TO THE ORLEANS AVENUE CANAL SITE**

| STRATUM | SOIL CLASSIFICATION (USCS) | PERME-ABILITY (KX) (CM/SEC) | PERME-ABILITY (KX) (FT/SEC) | PERME-ABILITY RATIO (KV/KH) |
|---|---|---|---|---|
| Fill clay (levee) | CH, CL | $1x10^{-6}$ | $3.28x10^{-8}$ | 1 |
| Marsh clay | CH with roots, wood | $1x10^{-5}$ | $3.28x10^{-7}$ | 1 |
| Beach silty sand | SP-SM (10% to 15% fines) | $7x10^{-4}$ | $2.30x10^{-5}$ | 1 |
| Beach sand | SP (5% or less fines) | $1.5x10^{-2}$ | $4.92x10^{-4}$ | 1 |
| Bay sound clay | CH, CL | $1x10^{-6}$ | $3.28x10^{-8}$ | 1 |
| Canal sediments ( if present) | SM,ML | $1x10^{-5}$ | $3.28x10^{-7}$ | 1 |
| **Note:** Soil classifications are in accordance with the Unified Soil Classification System [26] | | | | |

The sheet pile permeability was assumed set at $3x10^{-9}$ cm/sec ($1x10^{-10}$ ft/sec) to represent a relatively impermeable condition.

### *6.6.1 Validation of ERDC Permeability Recommendations*

The ERDC recommended permeability values were validated based on the following data. The permeability of poorly graded barrier beach sand stratum was estimated from the results of a pump test performed near the Orleans Avenue Canal. These results were checked using correlations with grain size data developed by Batool and Brandon [27] for the London Load Test and for samples collected during the London Avenue Canal MOWL study [13, 19]. The permeability of the silty sand layer, which sometimes is present at the top of the poorly graded barrier beach sand stratum, was evaluated by in situ falling head tests performed at the site of the London Load Test site by Batool and Brandon [27] and during the London Avenue Canal MOWL study. These results were also checked using correlations with grain size data developed by Batool and Brandon [27] for the London Load Test. Finally, the permeability of the canal bottom sediments were estimated during the London Avenue Canal MOWL study based on correlations with grain size of samples obtained from the canal bottom. The following paragraphs discuss these various studies and how they relate to the Orleans Avenue Canal.

#### *6.6.1.1 Orleans Avenue Canal Pump Test Permeability Data for Poorly Graded Sand*

A pump test [21] was performed adjacent to the Orleans Avenue Canal by the Corps in 2006 to evaluate the permeability of the poorly graded barrier beach sand stratum. The test site was located on the east side of the canal, just south of Harrison Avenue. The screened zone for the test was within sands described as poorly graded sand (SP) or poorly graded sand with silt (SP-SM) according to the Unified Soil Classification System (USCS) [26]. The fines content of the samples obtained within the screened zone ranged from 2.2 to 7.5 percent for an average of 4.8 percent fines. This is slightly higher than the average of 4.4 percent fines at the London Avenue Pump Test [20]. The USCS defines poorly graded sands as material with 5 percent or less fines and poorly graded sand with silt as material with a fines content of 5 to 12 percent. The permeability values of the barrier beach sand measured in this test ranged from 0.7 x $10^{-2}$ cm/sec to 1.6 x $10^{-2}$ cm/sec, with an average of about 1.1 x $10^{-2}$ cm/sec. These permeability values are slightly lower, but similar to the values measured in the same formation at London Avenue Canal Pump Test [20] and at the 17$^{th}$ Street Canal Pump Test [25] which both averaged 1.5 x $10^{-2}$ cm/sec.

#### *6.6.1.2 London Avenue Canal Permeability of Poorly Graded Sand Based on Correlations with Grain Size Data*

The permeability of the poorly graded barrier beach sand stratum at the London Load Test location was also estimated by Batool and Brandon [27] using correlations with grain size data. Samples of the sand were obtained from borings in the area of the load test and grain size analyzes were performed. Both the Hazen's Formula and the Kozeny-Carman relationship were used to estimate the permeability with the following results.

- Hazen's Formula – 1.16 x $10^{-2}$ cm/sec; and
- Kozeny-Carman relationship - 1.46 x $10^{-2}$ cm/sec.

These values compare favorably with the pump test results described above. The ERDC recommended permeability value of the poorly graded beach sand presented in Table 6-4 was consistent with the results of the pump test and grain size correlation analyses presented above for Orleans Avenue Canal.

During the London Avenue Canal MOWL study [19] the permeability of the poorly graded sands were further evaluated using the results of the grain size analyses. The permeability of these materials was estimated using the following two methods:

- Hazen's Formula; and
- Figure 17 from Corps Technical Memorandum 3-424 (TM) [32].

The results of the analyses for the poorly graded beach sand samples obtained from the borings along the levees and from below the canal bottom sediments. The Hazen formula and the TM generally predict permeabilities that are similar to the previous studies discussed above and cluster around the permeability value, $k = 1.5 \times 10^{-2}$ cm/sec, recommended by ERDC [24] in Table 6-4. Based on these results from the London Avenue Canal MOWL study [19], and the results discussed above for the Orleans Avenue pump test, the ERDC recommended value, $k = 1.5 \times 10^{-2}$ cm/sec, was deemed reasonable and conservative and was used in this study.



***6.6.1.3*** *2006 London Avenue Canal In Situ Falling Head Permeability Tests for Silty Sand*

The permeability of the silty sand layer was estimated by performing a series of in-situ falling head or slug tests in piezometers installed for the London Load Test and were evaluated by Batool and Brandon [27]. When the silty sand layer is present it significantly reduces the flow from an I wall gap to the underlying poorly graded sands. The silty sand provides greater head loss which reduces the uplift forces on the base of the protected side marsh clay stratum. This improves the stability of the I-wall levee embankment and foundation soils and the potential for excessively high ground surface exit gradients at the toe of the levee. The results of nine tests ranged from $2.68 \times 10^{-3}$ to $0.27 \times 10^{-3}$ cm/sec and the average value was $1.59 \times 10^{-3}$ cm/sec or about an order of magnitude lower than for the poorly graded sand stratum located below this silty sand layer.

***6.6.1.4*** *2010 London Avenue Canal In Situ Falling Head Permeability Tests for Silty Sand*

Additional in-situ falling head tests were performed in piezometers installed along the London Avenue Canal within the upper silty sand stratum in 2010 during the London Avenue Canal MOWL study [19]. Six of seven tests resulted in a range of permeability values from 2.42 x $10^{-3}$ to 3.46 x $10^{-3}$ cm/sec and appear to support the previous results from the London Load Test where the average permeability value was 1.59 x $10^{-3}$ cm/sec. The seventh test value of 5.78x$10^{-4}$ cm/sec was similar to the value recommended by ERDC was 7 x $10^{-4}$ cm/sec.

#### ***6.6.1.5*** *London Avenue Canal Permeability of Silty Sand Based on Correlations with Grain Size Data*

The permeability of this layer was also estimated by Batool and Brandon [27] on the basis of grain size data from samples obtained in borings in the area of the London Load Test with the following results.

- Hazen's Formula – 2.79 x $10^{-3}$ cm/sec; and
- Kozeny-Carman relationship - 1.51 x $10^{-3}$ cm/sec.

These values compare favorably with results obtained from the in-situ falling head tests.

Although the permeability value recommended by ERDC, 7 x 10-4 cm/sec, is about 50 percent lower than the in-situ testing data and the values obtained by Batool and Brandon [27] through correlation with grain size for the London Avenue Canal site, it was assumed this was a reasonable estimate for the silty sand permeability and this value was used in this study of the Orleans Avenue Canal.

#### ***6.6.1.6*** *London Avenue Canal Estimated Permeability Data for Canal Bottom Sediments*

The permeability results for canal bottom sediments were estimated during the London Avenue Canal MOWL study [19] based on the Hazen formula. The results indicated a ranged from about k = 1 x $10^{-2}$ to 1x $10^{-6}$ cm/sec for sampled collected from the canal bottom. It was concluded that the value recommended for silty sand (SM) and sandy silt (ML) by ERDC, k = 1 x $10^{-5}$ cm/sec, would be used in this study of the Orleans Avenue Canal to represent the canal bottom sediments.

## 6.7 DESIGN SHEAR STRENGTH AND UNIT WEIGHT VALUES

The shear strength versus depth relationships for the various reaches of the Orleans Avenue Canal were developed based on guidance provided in the HSDRRSDG, Subsection 3.1.2.1 Strengthlines [4], which states that the selected shear strength relationship with depth should be drawn where approximately one-third of the test values fall below the line and two-thirds of the test values fall above the line. The design shear strengths were selected using unconsolidated undrained triaxial tests (Q-tests), unconfined compression tests (UCTs), CPTs and VSTs. A shear strength relationship with depth was also plotted from the ratio c/p where c represents the undrained shear strength, or cohesion, at a specific depth and p represents the effective overburden pressure at that depth. A c/p ratio of 0.22 was selected for use in the marsh clays and lower bay sound clays based on guidance from the Corps. This relationship was used as a guide in developing a shear strength with depth relationship in reaches where laboratory and in situ test data were inadequate.

The 0.22 c/p line was calculated for both the protected side toe and the embankment centerline for this Orleans Avenue Canal MOWL study. Effective overburden stresses at the toe of the embankment were estimated by conservatively ignoring additional stress increase from the embankment. For the centerline, the effect of the embankment on the vertical stress was considered. Due to the embankment geometry, the vertical stress increase with depth beneath the embankment is not a linear function. The SIGMA/W software was used to estimate vertical stress under the embankment using the same model geometry as in the SLOPE/W analysis. The bottom of all stability models was set to El -70 NAVD88, but the bottom of the stress model was deepened to El -120 to -150 NAVD88 to decrease boundary effects. Poisson's ratio for the sand was 0.3 and for all other cohesive layers was 0.47 based on London Avenue Site Specific Load Test Report, Appendix D, Table 1 [9]. Groundwater conditions for the in situ model were based on normal tidal water level of El 1.0 NAVD88 which represents the long-term water level in the canal. The vertical effective stress at the bottom of each layer was exported from the SIGMA/W model and input into the shear strength versus depth relationships calculation. The results of the SIGMA/W model, shown as a plot of vertical effective stress contours, are presented in Appendix D.3 along with the SEEP/W and SLOPE/W results.

In accordance with the above referenced HSDRRSDG guidance, Q-tests, as well as CPTs and VSTs, were given more weight than UCTs when estimating shear strengths. Q-tests are typically performed at three different confining pressures and are more representative of in-situ undrained strengths whereas UCTs are not confined and typically exhibited lower strength values than the Q-tests. VSTs represent in situ undrained strengths.

Shear strengths were developed from CPT data based on the following relationship:

$S_u = q_c/N_c$; where $N_c = 20$.

The Nc value was assumed based on the Corps historical knowledge of the soils in the New Orleans area. Typically the Corps has found that undrained shear strengths obtained from this relationship are equivalent to or lower than undrained shear strengths obtained from VSTs.

The hydraulic fill material was assigned both cohesive and cohesionless shear strengths. Borings B-41 and B-45 indicate that the material is a silty sand. The unit weights from laboratory testing support this classification. CPT correlation data from OWCPT-21 thru 23 and OECPT-21 thru 23 indicate both cohesive and cohesionless material.

The undrained shear strengths of the marsh and lacustrine clays under the centerline of the levees were estimated from data included in DM 19 [6] or more recent CPT [10] data obtained along the crest of the levees.

The original design did take into consideration that the undrained shear strength of the marsh clays and lacustrine clays under the crest of the levee were higher than the strengths at the toe of the levee due to the consolidation of these soils under the weight of the levee fills. Since some of the undrained shear strength testing performed during the original design was completed on samples obtained from under the crests of the levees, the results represented higher strengths than were available at and beyond the levee toes. During this MOWL study, lower undrained strengths were used for the marsh and lacustrine clays at and beyond the levee toes as recommended by the IPET Report [1]. The undrained shear strength of the marsh and lacustrine clays at the toes of the levees was based on DM 19 [6] data where available, and CPT and VST data [10] for tests performed at the levee toes where data were available. In no case were undrained shear strength values selected that were greater than 95 percent of the centerline undrained shear strength values. If only DM 19 [6] data were available from the centerline, the toe shear strengths values were reduced 5 percent to account for reduced vertical stress at the toes of the levees. Where there were no laboratory, CPT, or VST data available for evaluation of the undrained shear strengths of the marsh and lacustrine clays on the flood side toes of the levees, the undrained strengths of these soils were assumed to be the same as for the protected side toes. These strengths had little effect on the global stability analyses and they did not impact the gap analyses.

The assumed shear strength properties of the beach sand stratum were in agreement with Table 3.2 of the HSDRRSDG [4].

The undrained shear strength of the bay sound clays were obtained from DM 1 [6] and post Katrina borings and CPT testing [10]. If no undrained shear strength data were available, the undrained shear strength versus depth relationship was estimated by the c/p ratio discussed above.

The averages of unit weights for the marsh clay, lacustrine clay and bay sound clay strata were obtained from DM 19 [6] and post Katrina laboratory testing [10]. Average unit weight values for these strata along the protected side toes and flood side toes of the levees were assumed to be the same as reported for the centerline unless data were available for the toe in DM 19. The assumed unit weight values of the underlying beach sand stratum were in agreement with Table 3.2 of the HSDRRSDG [4].

Graphs summarizing the water contents, unit weights and shear strengths versus depth for each canal reach were plotted to evaluate the properties. The selected design relationship between soil strength and depth and unit weight and depth for each reach are included on these graphs which may be found in Appendix B. A summary of the canal reach data including shear strength and unit weight variations with depth is included in Appendix A.2.



## 6.8 RESULTS OF LONDON AVENUE CANAL I-WALL LOAD TEST

A full-scale I-wall load test was conducted on the London Avenue Canal (London Load Test) in the summer of 2007 to evaluate the MOWL at a specific location along the 3.2-mile long canal. The test simulated two canal bottom conditions. The first condition assumed that the recent canal sediments and possibly a thin marsh clay layer were present, overlying the beach sand. The second assumed that the beach sand was present at the base of the canal. The test was performed in two stages within a cofferdam attached to the I-wall. During the first stage, simulating marsh clay overlying the beach sand in the bottom of the canal the water level was raised from El 0.0 to El 7.0 NADV88 in increments of 0 5 feet. Each increment of load was held until the instrumentation indicated that equilibrium had been reached with respect to pore pressure response on the protected side and wall deflection had ceased. During the second stage, water in the cofferdam was allowed to flow down through wells into the sand layer simulating beach sand present at the base of the canal underlying the marsh deposit. Thus, the piezometric pressure in the sand was directly impacted by the water level in the cofferdam. The same sequence of loading was performed for the second stage as was used in the first stage of the test.

The instrumentation systems were continuously monitored to assure that instability conditions did not occur. The test results indicated that at this specific site, under this specific set of subsurface and structural conditions, the maximum measured top-of-wall movements increased from approximately 0 5 inch with 4 feet of water depth loading the I-wall to 1.5 inches at 6 feet of water depth. In addition to the measured top-of-wall movement, conditions that could have led to seepage instability were not detected until the final load of the second stage of the test when the water level reached El 7.0 NAVD88. After readings stabilized, the water levels were reduced and the test terminated. A more complete description of the test may be found in the London Avenue Canal MOWL study [19].

## 6.9 POST KATRINA STABILITY AND SEEPAGE ANALYSES PROCEDURES

Prior to Hurricane Katrina, MVN utilized the Method of Planes (MOP) stability analysis method [35] to design the original I-wall levee parallel protection systems. This stability analysis method is a wedge method which only satisfies horizontal equilibrium. It considers the soil mass above a slip surface and consists of three wedges, the active, the neutral and the passive. It has been demonstrated [22] that the MOP is generally conservative and that the factors of safety it produces are lower than more modern analysis methods that do satisfy all conditions of static equilibrium. Following Hurricane Katrina, it was agreed to use the universally accepted Spencer's Method [5], which satisfies all of the conditions of equilibrium for future stability analyses as the primary method of analysis and MOP used as a check. It was also agreed to use finite element seepage analysis when specific projects dictate this level of analysis.

## 6.10 LEVEE REACHES

The canal was originally divided into several reaches along both the east and west levees in DM No. 19 [6] and was modified during construction as indicated by the "as built" drawings [11] provided by the Corps. The "as built" reaches were identified in Table 6-1. Extensive additional subsurface investigations and topographic and bathymetric surveys have provided additional information to characterize in greater detail the conditions along the canal. This information was used during this study to further divide the east and west floodwalls into a larger number of reaches than originally existed.

### *6.10.1 Reach Definition*

The canal was subdivided into the reaches initially based on I-wall sheet pile cutoff wall tip elevations. The geotechnical properties, ground surface grades of the embankment and canal, and the possibility that there was a direct hydraulic connection between the bottom of the canal and the underlying beach sand stratum were used to further subdivide the canal and additional reaches were added. Specifically, the canal reaches referenced in this study were developed based on the following four criteria.

- Barrier Type - There are three types of flood protection barriers along Orleans Avenue Canal: earth levees; I-Walls atop earth levees; and T-Walls atop earth levees. Reaches were differentiated by barrier type.

- I-wall and T-wall Sheet Pile Tip Elevations - The tip elevations of the sheet pile cut off walls below the I-walls and T-walls vary along the canal alignment on both banks. The reaches were selected such that the sheet pile tip elevations are consistent throughout an individual reach.

- Stratigraphy, Soil Strength, and Unit Weights – The reaches were selected such that the undrained shear strengths and unit weights of the clays, thickness of the marsh clays, the top of the beach sand, and other stratagraphic characteristics were relatively consistent throughout an individual reach.

- Ground Surface Elevations - The cross section of the levees vary along the canal alignment. The lowest protected side crest elevation and the average elevations on the flood side, except for Reaches 1A, 12A and 13B were used throughout an individual reach. Within Reaches 1A, 12A and 13B, due to the potential for the barrier beach sand to be present in the bottom of the canal, the lowest elevation profile on the flood side was used for the analyses. Reaches were then selected based on similar ground surface elevations.

- Direct Connections between the Canal Water and Beach Sand Deposit - The areas along the canal where a direct hydraulic connection to the beach sand was estimated to exist were designated separate reaches

The canal was divided into 21 reaches, 11 on the west bank and 10 on the east bank, based on these criteria as shown in Table 6-5. The protected side and flood side levee embankment crest elevations and sheet pile tip elevations are also included in Table 6-5. The reach locations are shown on Plates 1 through 8 included in Appendix A.3.

The bridges were also excluded from the reaches. The footprint width of the bridge abutment embankment is at least 2 to 3 times the I-wall levee embankment footprint, and therefore, seepage and stability is not an issue at these locations. The formation of gaps between the flood side soils and the sheet pile cutoff walls below the bridge abutments are precluded from occurring since they are pile supported. Any remediation that is ultimately recommended adjacent to a bridge abutment must be analyzed for wrap-around underseepage if the sheet pile cutoff wall under the abutment has a higher tip elevation than the proposed remediation sheet pile cut-off wall.

### *6.10.2 Reach Geometry and Geotechnical Properties*

A summary of the design data used to evaluate each reach is included in Appendix A.2. This summary provides a brief description of the following items for each reach.

- How the station limits were established for each reach;
- How the field investigation data were used to develop the stratigraphy for the reach; and
- The elevations of the following critical components within each reach;
  - Top of floodwall;
  - Flood side levee crest;
  - Protected side levee crest;
  - Protected side levee toe; and
  - Sheet pile cutoff wall tip.

The existing elevations of the tops of the floodwalls and the other features were obtained from the recent surveys. The cross sections developed from these survey data that were used to evaluate each reach are included in Appendix A.3 on Plates 49 through 61. The survey cross sections include the original design ground surface cross sections and the design ground surface cross sections used in this MOWL Study. Plates 1 through 8 in Appendix A.3 provide an aerial view of the canal alignment. The reach locations are indicated on these plates.

**TABLE 6-5**
**LEVEE REACH LOCATIONS FOR THE ORLEANS AVENUE CANAL**

| WEST REACH | BASELINE APPROXIMATE STATION | WALL TYPE | EMBANK-MENT CREST ELEVATION NAVD88 | | SHEET PILE TIP ELEVATION. (FT) NAVD88 | EAST REACH | BASELINE APPROXIMATE STATION | WALL TYPE | EMBANK-MENT CREST ELEVATION NAVD88 | | SHEET PILE TIP ELEVATION. (FT) NAVD88 |
|---|---|---|---|---|---|---|---|---|---|---|---|
| | | | PROTECTED SIDE | FLOOD SIDE | | | | | PROTECTED SIDE | FLOOD SIDE | |
| 1A | 2+45 to 7+00 | I-wall | 3.60 | -1.60 | -28.5 | 12A | 2+45 to 3+70 | Levee w/wall | 7.6 | 7.6 | -- |
| 1B | 7+00 to 9+25 | I-wall | 3.60 | -1.60 | -28.5 | 12B | 3+70 to 4+70 | Levee | 8.5 | 8.5 | -- |
| 1C | 9+25 to 11+00 | I-wall | 3.60 | -1.60 | -28.5 | 13A | 4+70 to 7+00 | I-wall | 8.00 | 8.00 | -1.3 |
| 1D | 11+00 to 14+20 | I-wall | 3.60 | -1.60 | -28.5 | 13B | 7+00 to 11+20 | I-wall | 8.00 | 8.00 | -1.3 |
| 2 | 14+20 to 21+75 | I-wall | 3.20 | -2.30 | -28.5 | 14 | 11+20 to 20+50 | I-wall | 8.00 | 8.00 | -1.3 |
| 3 | 21+75 to 24+87 | T-wall | 3.40 | -1.20 | -21.5 | 15 | 20+50 to 30+00 | I-wall | 8.00 | 8.00 | -1.3 |
| 4 | 24+87 to 29+16 | I-wall | 3.30 | -1.80 | -28.5 | 16 | 30+00 to 36+40 | I-wall | 8.00 | 8.00 | -1.3 |
| 5 | 29+16 to 36+26 | T-wall | 2.80 | 0.50 | -26.5 | **Harrison Ave.** | | | | | |
| **Harrison Ave.** | | | | | | 17 | 37+44 to 50+00 | I-wall | 5.50 | 5.50 | -9.8 |
| 5 | 37+27 to 42+00 | T-wall | 2.80 | 0.50 | -27.5 | 18A | 50+00 to 61+00 | I-wall | 5.90 | 6.10 | -9.8 |
| 6 | 42+00 to 50+00 | T-wall | 2.80 | 0.50 | -27.5 | 18B | 61+00 to 64+00 | I-wall | 5.40 | 5.40 | -9.8 |
| 7 | 50+00 to 59+00 | T-wall | 2.80 | 0.50 | -35.5 | **Filmore Ave.** | | | | | |
| 8 | 59+00 to 63+58 | T-wall | 2.80 | 0.50 | -35.5 | 19 | 65+00 to 90+62 | I-wall | 4.00 | 4.00 | -15.3 |
| **Filmore Ave.** | | | | | | **Robert E. Lee Ave.** | | | | | |

| WEST REACH | BASELINE APPROXIMATE STATION | WALL TYPE | EMBANK-MENT CREST ELEVATION NAVD88 | | SHEET PILE TIP ELEVATION, (FT) NAVD88 | EAST REACH | BASELINE APPROXIMATE STATION | WALL TYPE | EMBANK-MENT CREST ELEVATION NAVD88 | | SHEET PILE TIP ELEVATION, (FT) NAVD88 |
|---|---|---|---|---|---|---|---|---|---|---|---|
| | | | PROTECTED SIDE | FLOOD SIDE | | | | | PROTECTED SIDE | FLOOD SIDE | |
| 9 | 65+00 to 90+27 | T-wall | 3.00 | 0.50 | -40.0 | 20A | 92+20 to 93+46 | I-wall | 6.50 | 6.50 | -1.5 |
| **Robert E. Lee Ave.** | | | | | | 20B | 93+46 to 101+50 | Levee | 13.20 | 13.2 | -- |
| 10 | 91+88 to 93+53 | I-wall | 7.50 | 7.50 | -9.5 | 21 | 101+50 to 113+05 | Levee | 13.20 | 13.2 | -- |
| 10 | 93+53 to 98+70 | Levee | 13.20 | 13.20 | -- | | | | | | |
| 11 | 98+70 to 112+50 | Levee | 13.20 | 13.20 | -- | | | | | | |

## *7.0 EXISTING SAFE WATER CONDITIONS*

The east bank levee and flood wall system of the Orleans Avenue Canal is located adjacent to City Park for most of the length of the canal. The west bank of the canal is a combination of parkland and primarily residential development. As the city has grown, single and multi-unit homes, apartments, condominiums, businesses, infrastructure, roads, bridges, and other urban developments have been constructed in proximity to the canal and, in some cases, have encroached nearly to the toes of the levees. This development has the potential to adversely impact the MOWL due to the conditions on the protected side of the levee. The following section discusses the analysis procedures and results used to evaluate the existing MOWL along the canal.

### 7.1 EXISTING SAFE WATER CONDITIONS ANALYSIS

The existing MOWL along the Orleans Avenue Canal were evaluated. The following four potential failure modes were analyzed for each I-wall reach:

- Global stability;
- Gap analysis - only applicable to I-walls;
- Wall rotation; and Seepage

The stability of the T-walls, pump station walls and the pump station was also evaluated.

Global stability is the overall stability of the levee and floodwall at high water with no formation of a gap on the flood side face of the I-wall. The critical failure surfaces for global stability are deep-seated, where the entire levee and floodwall system slides in the landside direction. The pore pressures from the gap analyses were used in the global stability analyses as recommended by the Technical Review Team (TRT).

Both the Spencer's Method [5] and the Method of Planes (MOP) [35] analyses were used to evaluate slope stability in accordance with the methodology identified in Section 4.6 of this report. The program SLOPE/W Version 7.16 [34] was used in the analyses. The subsurface conditions at each reach of the Orleans Avenue Canal were evaluated for both a block and a circular failure. The critical failure surface identified was further optimized by the internal methodology included in the SLOPE/W software.

The gap analysis was based on the formation of a gap on the flood side of the I-wall. A gap condition does not occur for a T-wall because it is supported by batter piles to substantially reduce deflection during loading. The formation of a gap results in several major impacts on the MOWL evaluation.

- The full hydrostatic pressure is introduced to the base of the gap;
- The length of the critical failure surface is reduced; and
- The length of the seepage path is potentially reduced.

By introducing hydrostatic head from the canal to a point below the top of the marsh clay stratum in the barrier beach sands causes a reduction in the length of the seepage path. The reduced head loss due to a reduced seepage path length also increases uplift pressures below the marsh clay stratum which could result in rupture. The increase in pore pressures in the sand also reduces the shear strength of the sand and increases the exit gradient at the toe of the levee.

The depth of the gap was estimated in accordance with the methodology identified in Section 4.5 of this report. This procedure was used to calculate the maximum gap that could develop based on the undrained shear strength of the levee clay and marsh clay. The calculated maximum gaps were used in the stability and seepage analyses. During the computation of the gap depths, it was determined that the methodology was relatively insensitive to the water height on the flood side of the floodwall. Based on this methodology, any water height on the I-wall above the levee crest will result in the same calculated gap depth. The piezometric surface for each reach was developed using the SEEP/W Version 7.16 [34], which allows direct transfer of soil pore water pressures into SLOPE/W.

Wall rotation is controlled by the ability of the floodwall system to resist movement toward the protected side. The potential for movement is controlled by the depth of sheet pile penetration, the deformation properties of the supporting soil on the protected side, and the stiffness of the wall member. The embedded I-wall sheet pile sections, as indicated on the “as built” drawings [11] are PZ 22 and PZ27. The potential for wall rotation was estimated based on sheet pile penetration and penetration ratio.

The potential seepage failure mode involves active seepage forces that are capable of displacing and transporting subsurface material due to high ground surface exit gradients. The erosion occurs from the ground surface back towards the source of seepage. This type of erosion is called “piping” and it can result in ultimate failure of the levee embankment. Three conditions are required to achieve a piping failure mode:

- Sufficient exit gradient;
- Unfiltered exit; and
- Erodible material.

At the Orleans Avenue Canal, all three conditions exist for a potential piping seepage failure. The exit gradient is increased by formation of a gap adjacent to the I-wall and the ground surface along the canal levees where piping could initiate is unfiltered. The marsh clays are not particularly erodible but the beach sand below the clay is erodible. In locations where the marsh clays are thin, or lenses of sand exist within the clays, the potential for piping is increased. Where the marsh clays are thin, the potential for soil rupture due to the high uplift pressures at the base of the clay could also facilitate piping. An additional concern is a direct seepage path from the base of the canal under the sheet pile tips within the beach sands. This can occur when the bottom of the canal penetrates the top of the beach sand stratum.

For T-walls, an additional condition that may occur is “roofing” caused by settlement of the soil below the pile-supported wall base slab. This condition is mitigated by the continuous sheet pile anchored in the base slab that will cut off any void below the base slab. The minimum embedment of the sheet pile into the concrete base slab is 9 inches. A steel reinforcement bar is also required to be placed through the sheet pile and then anchored into the concrete base slab.

Because the MOWL is controlled by specific failure modes, the FOS for each failure mode is reported for each reach.

### *7.1.1 Global Stability*

The global stability analyses were performed under the condition potential failure surfaces could penetrate up to 5 feet above the tip of the I-wall sheet pile. The sheet pile was assigned a high shear strength above 5 ft from the sheet pile tip to restrict the SLOPE/W program from identifying a controlling failure surface from penetrating the sheet pile above this level. This requirement is conservative compared to the guidelines discussed in Section 4.8 of this report for the I-wall gap analysis where potential failure surfaces are required to pass below the sheet pile tip. The effect is to cause the global stability analyses to yield lower factors of safety than would be the case if the potential failure surfaces were restricted to below the sheet pile tips.

The piezometric surfaces determined from the gap analyses were used in the global stability analyses as recommended by the TRT.

The MOWL was first determined by the Spencer's Method [5] of analysis and was checked using the MOP [35] methodology. The MOP analysis is performed in two steps. In the first step the MOP program was allowed to identify the most critical active wedge. If the critical active wedge did not intercept the sheet pile at a height greater than 5 feet above the sheet pile tip, the analysis was continued using this active wedge location. If the critical active wedge found in this first step intercepted the sheet pile at a height greater than 5 feet above the sheet pile tip, the active wedge was restrained at the most critical active wedge that penetrated the bottom 5 feet of the sheet pile.

The results of the global stability analysis, including the global MOWLs and FOSs are presented in Table 7-1.

Hydraulic fill is located below the levee fill and beyond the toes of the levees in Reach 10B and 11 on the west side of the canal and Reaches 20A, 20B, and 21 on the east side of the canal. The hydraulic fill in Reach 10B was classified similar to the overlying levee fill and was evaluated as clay. Reaches 11, 20A and 20B were evaluated assuming two conditions for the hydraulic fill. In the first case, designated 11-clay, 20A-clay and 20B-clay in Table 7-1, the hydraulic fill layers classified clay were modeled as clay and the layers classified sand were modeled as sand both in the seepage and stability analyses. In the second case, designated 11-sand, 20A-sand and 20B-sand in Table 7-1, the entire hydraulic fill stratum was modeled as sand. The purpose was to evaluate which subsurface condition produced the lowest FOS. The results from both analyses are included in Table 7-1 and varied by reach. The variation in material type did not affect the MOWL for any reach where it was evaluated. The hydraulic fill in Reach 21 was generally classified as sand and was modeled as sand.

The FOS calculated by the Spencer's Method of analysis for Reaches 10B, 17, 18A, and 20B are slightly less than the required 1.4 with the water level in the canal at El 1.0 NAVD88. This water level corresponds with the normal Lake water level. This indicates an inadequate FOS without the influence of the canal water load. These low FOS were the result of the low shear strengths identified in the 2010 CPTs advanced at the toe of these levees. Reach 18A was designed with a protected side stabilization berm extending approximately 90 feet from the I-Wall. This berm is shown in DM-19 [6] and on the "as-built" drawings [11]. The topographic survey performed in 2010 [12] indicated that the berm width was about 30 feet in this reach.

## TABLE 7-1
## GLOBAL STABILITY MOWLS AND FACTORS OF SAFETY FOR I-WALLS WITHIN LEVEES AND FOR LEVEES WITHOUT I-WALLS

| WEST REACH | SPENCER'S METHOD | | MOP | | EAST REACH | SPENCER'S METHOD | | MOP | |
|---|---|---|---|---|---|---|---|---|---|
| | MOWL NAVD88 | FOS | MOWL NAVD88 | FOS | | MOWL NAVD88 | FOS | MOWL NAVD88 | FOS |
| 1A | 10.0 | 3.16 | 10.0 | 2.38 | 12A[3] | **8.5** | 2.27 | **8.5** | 2.29 |
| 1B | 10.0 | 3.02 | 10.0 | 2.51 | 12B[2] | **8.5** | 2.40 | **8.5** | 2.32 |
| 1C | 10.0 | 3.21 | 10.0 | 2.78 | 13A | 10.0 | 2.14 | 10.0 | 2.25 |
| 1D | 10.0 | 4.21 | 10.0 | 3.33 | 13B | 10.0 | 2.09 | 10.0 | 2.25 |
| 2 | 10.0 | 3.01 | 10.0 | 2.33 | 14 | 10.0 | 1.74 | 10.0 | 1.77 |
| 3 | T-wall | | | | 15 | 10.0 | 1.51 | 10.0 | 1.49 |
| 4 | 10.0 | 3.27 | 10.0 | 2.52 | 16 | 10.0 | 1.41 | 10.0 | 1.40 |
| 5 | T-wall | | | | **Harrison Avenue** | | | | |
| **Harrison Avenue** | | | | | 17 | **<1.0** | 1.34[1] | **1.0** | 1.62 |
| 5 | T-wall | | | | 18A | **<1.0** | 1.28[1] | **1.0** | 1.35 |
| 6 | T-wall | | | | 18B | **8.5** | 1.44 | **8.5** | 1.46 |
| 7 | T-wall | | | | Filmore Avenue | | | | |
| 8 | T-wall | | | | 19 | **5.0** | 1.48 | **5.0** | 1.49 |
| **Filmore Avenue** | | | | | **Robert E. Lee Avenue** | | | | |
| 9 | T-wall | | | | 20A-clay | 10.0 | 1.94 | 10.0 | 1.91 |
| Robert E. Lee Avenue | | | | | 20A-sand | 10.0 | 1.90 | 10.0 | 1.72 |
| 10A | 10.0 | 2.77 | 10.0 | 2.61 | 20B-clay | <1.0 | 1.28[1] | <1.0 | 1.29[1] |
| 10B | **<1.0** | 1.33[1] | **1.0** | 1.49 | 20B-sand | <1.0 | 1.33[1] | <1.0 | 1.27[1] |
| 11-clay | 10.0 | 1.68 | 10.0 | 1.80 | 21 | 10.0 | 1.50 | 10.0 | 1.75 |
| 11-sand | 10.0 | 1.70 | 10.0 | 1.95 | | | | | |

**Notes:**
[1] FOS less than 1.4 for Spencer's and/or 1.3 for MOP analyses at canal water levels equal to El 1 NAVD88, the normal Lake level
[2] MOWL limited to top of earth embankment
[3] MOWL assumed the same as for Reach 12B although concrete wall crest grade is El 9.7 NAVD88. The levee crest grade is El 7.6 NAVD88



Reaches 12A and 12B on the east side of the canal just north of DPS 7 consist of earthen levees. The crest of the levee in Reach 12A is El 7.6 NAVD88. The protected side and flood side slopes are armored with a concrete slab and a 2-foot wide by 2.1-foot high concrete wall extends above the concrete slab to El 9.7 NAVD88. In Reach 12B the crest of the levee is El 8.5 NAVD88. Both reaches were evaluated for a MOWL of El 8.5 NAVD88 independent of the wall in Reach 12A. The results are shown in Table 7-1. The concrete wall atop Reach 12A was also evaluated for stability under the canal water level of El 8.5 NAVD88. The wall was assumed to be a monolithic concrete block and was evaluated considering the following cases. The factors of safety are listed for each case:

- Sliding at the base of the block, FOS = 4.8; and
- Overturning at the toe of the block, FOS = 5.6.

It was verified that the resultant eccentricity was within the middle third of the block so that there was no tension on the base of the block.

The MOP input, output, and plots of each reach are presented in Appendix D.1. The Spencer's Method analyses are located in Appendix D.3 along with input and output reports. The results of the wall calculations are presented in Appendix D.8. Executable input files are located in Appendix E.

### *7.1.2 Gap Analysis*

In contrast to the global stability analyses, all potential failure surfaces for the gap analyses were initiated at the I wall sheet pile tip. For the SLOPE/W analyses, the full length of the sheet pile was assigned a high shear strength to restrict the program from identifying a controlling failure surface through the sheet pile. The piezometric surfaces determined from the Seep/W seepage analyses that considered a gap were used in the gap stability analyses.

In several reaches, sheet piles from previous floodwalls remained in place after the I-walls and T-walls were constructed. The location of the sheet piles and the tip elevations were determined from the "as-built" drawings [11]. They were removed from the models since they were located on the flood side of the existing I-walls.

The MOWL identified in the Spencer's analysis was checked using the MOP methodology. The MOP analysis was again performed in two steps.

When the MOP stability analysis indicated that the gap penetrated to the tip of a sheet pile, the fully penetrating gap case, the stability analysis was performed with the soil load removed and a hydrostatic water load equivalent to that used in the Spencer's Method analysis applied to the tip of the sheet pile. Below the sheet pile tip, the water pressure previously calculated from the Seep/W analysis, was added for the MOP analysis. In Reaches 10A, 13 through 17 and Reach 20A the gap was fully penetrating.

When the analysis indicated that the gap only penetrated a portion of the distance to the tip of the sheet pile, the partially penetrating gap case, a force was added to the sheet pile to account for the lateral earth pressure. The stability analysis was performed with the soil removed to the sheet pile tip and a hydrostatic water load, equivalent to that used in the Spencer's Method analysis was applied to the depth of gap penetration. Below this level the water pressure previously calculated from the Seep/W analysis was used in the MOP analysis. The modifications to the MOP analysis required for the gap analysis and to calculate the required force to accommodate the partially penetrating gap case are included in Appendix D.2.

The results of the gap stability analyses, including the gap MOWLs and FOSs are presented in Table 7-2. The gap analysis does not apply to Reaches 3 and 5 through 9, as they contain T-Walls or Reaches 10B, 11, 12A, 12B, 20B and 21 which are solely earthen levees.

The results of the gap stability analyses are provided in Appendix D.2 for the MOP methodology and Appendix D.3 for the Spencer's Method analysis along with input and output reports. Executable input files are included for review in Appendix E.

Hydraulic fill is located below the levee fill and beyond the toe of the levee in Reach 20A on the east side of the canal. This reach was evaluated assuming two conditions for the hydraulic fill. In the first case, designated 20A-clay in Table 7-2, the hydraulic fill layers classified clay were modeled as clay and the layers classified sand were modeled as sand both in the seepage and gap stability analyses. In the second case, designated 20A-sand in Table 7-2, the entire hydraulic fill stratum was modeled as sand. The purpose was to evaluate which subsurface condition produced the lowest FOS. The results from both analyses were identical and are included in Table 7-2.

The Spencer's Method of analysis for Reaches 17 and 18A resulted in a calculated FOS less than 1.4 with water at the crest of the flood side earth levee. In this case no gap

**TABLE 7-2**
**GAP STABILITY MOWLS AND FACTORS OF SAFETY FOR I-WALLS**

| WEST REACH | BASE ELEVATION GAP NAVD88 | SPENCER'S METHOD | | ADJUSTED MOP | | EAST REACH | BASE ELEVATION GAP NAVD88 | SPENCER'S METHOD | | MOP | |
|---|---|---|---|---|---|---|---|---|---|---|---|
| | | MOWL NAVD88 | FOS | MOWL NAVD88 | FOS | | | MOWL NAVD88 | FOS | MOWL NAVD88 | FOS |
| 1A | -7 | 10.0 | 5.35 | 10.0 | 4.49 | 12A | Levee | | | | |
| 1B | -7 | 10.0 | 4.94 | 10.0 | 5.09 | 12B | Levee | | | | |
| 1C | -7 | 10.0 | 5.81 | 10.0 | 5.89 | 13A | -1.3 | 10.0 | 1.72 | 10.0 | 1.59 |
| 1D | -7 | 10.0 | 10.00 | 10.0 | 8.24 | 13B | -1.3 | 10.0 | 1.63 | 10.0 | 1.65 |
| 2 | -13 | 10.0 | 4.54 | 10.0 | 4.42 | 14 | -1.3 | 10.0 | 1.52 | 10.0 | 1.40 |
| 3 | T-wall | | | | | 15 | -1.3 | 10.0 | 1.70 | **8.5**[2] | 1.30 |
| 4- | -13 | 10.0 | 5.05 | 10.0 | 4.59 | 16 | -1.3 | 10.0 | 1.75 | **<8.0**[2] | 1.26[1] |
| 5 | T-wall | | | | | Harrison Avenue | | | | | |
| Harrison Avenue | | | | | | 17 | -9.8 | **<5.5** | 1.33[1] | **5.5** | 1.30 |
| 5 | T-wall | | | | | 18A | -6.6 | **<6.1** | 1.26[1] | **<6.1** | 1.23[1] |
| 6 | T-wall | | | | | 18B | -7.4 | **8.0** | 1.42 | **7.0**[2] | 1.41 |
| 7 | T-wall | | | | | Filmore Avenue | | | | | |
| 8 | T-wall | | | | | 19 | -6.5 | **5.5** | 1.48 | **5.5** | 1.48 |
| Filmore Avenue | | | | | | Robert E. Lee Avenue | | | | | |
| 9 | T-wall | | | | | 20A-clay | -1.5 | 10.0 | 2.09 | 10.0 | 1.86 |
| Robert E. Lee Avenue | | | | | | 20A-sand | -1.5 | 10.0 | 2.09 | 10.0 | 1.86 |
| 10 | -1.5 | 10.0 | 2.91 | 10.0 | 2.57 | 20B | Levee | | | | |
| 10 | Levee | | | | | 21 | Levee | | | | |
| 11 | Levee | | | | | | | | | | |

[1] Analysis was performed at MOWL equal to flood side crest of levee (lowest elevation that would generate a gap) and did not meet FOS criteria, thus the MOWL is below crest of flood side earth levee.
[2] MOP MOWL controls (result is **BOLD**)



would form for a lower water level and the MOWL is below crest of flood side earth levee. The FOS values for the gap stability analysis were 1.33 and 1.26, with water in the canal at

El 5.5 and 6.1 NAVD88, respectively. These low FOS values resulted from the low shear strength values used in the analysis for this reach. In Reaches 16 and 18A the MOP analysis did not achieve a FOS of 1.3 with the water level in the canal at the crest of the flood side earth levee. The FOS values for the gap stability analysis were 1.26 and 1.23, respectively, with water in the canal at El 8.0 and 6.1 NAVD88, respectively. The MOP analysis controlled in Reaches 15, 16, and 18B.

### *7.1.3 I-Wall Rotation*

These analyses provided a check of the I-wall sheet pile against minimum criteria presented in Section 4.4. The analysis did not apply to the T-walls and earth levees in Reaches 3, 5 through 9, and 10B and 11 on the west bank and Reaches 12A, 12B, 20B and 21 on the east bank of the canal. The criterion limits the water height ($H_1$) on the I-wall to 4 feet or less above the protected side levee crest. The minimum penetration depth (D) criterion for the sheet pile wall is 10 feet below the lowest levee crest. This is a straightforward check that does not relate to the water level in the canal. The penetration ratio $D/H_1$ is required to be at least 3. Table 7-3 provides a summary of the I-wall stability for each canal reach.

Reaches 10A, 13 through 16 and 20A did not meet the minimum sheet pile penetration of 10 feet as shown in bold face type in Table 7-3. The $D/H_1$ ratio limits the MOWL to slightly below El 10 NAVD88 for Reach 20A. Limiting the water depth on the I-walls to 4 feet above the levee crests reduces the MOWL to below El 10 NAVD88 for Reaches 1A, 1B, 1C, 1D, 2, 4, and 17 through 19. The lowest MOWL, based on this criterion is El 7.2 NAVD88.

The stability of the I-walls was also evaluated by the CWALSHT program [33] for a MOWL of El 10 NAVD88. All analyses were performed by applying a FOS = 1.5 to the active and passive soil strengths. In accordance with MVN Corps requirements, the CWALSHT runs were made in design mode. Two cases were evaluated. In case "a" the canal water level was set at El 10 NAVD88 and the analysis considered wall rotation away from canal. In case "b" the canal water level was set at El -1 NAVD88 and the analysis considered wall rotation toward canal. This is termed the bulkhead case. Every reach was run using both the Fixed Surface Wedge Method and Sweep Search Wedge Method. In order for CWALSHT to generate a solution for case "a", the strength of the topmost soil stratum (the embankment) was reduced until a successful run could be made.

In all cases the reductions are quite large and in every case, the design sheet pile tip was above the actual installed tip. Case "a" results are reported in Table 7-4. In every reach, the resulting sheet pile tip elevation was higher than the actual installed sheet pile tip elevation. Therefore, all reaches have a MOWL greater than El 10 NAVD88 according to the CWALSHT analyses. This analysis is very conservative.

**TABLE 7-3**
**LONDON AVENUE CANAL WALL STABILITY**

| West Reacch | Protect Side Crest Elevation NAVD88 | Flood Side Crest Elevation NAVD88 | Sheet Pile Tip Elevation NAVD88 | Sheet Pile Penetration (D) (ft) | Maximum MOWL $D/H_1 = 3/1$ NAVD88 | Maximum MOWL - 4 Ft Water on Wall NAVD88 | East Reach | Protect Side Crest Elevation NAVD88 | Flood Side Crest Elevation NAVD88 | Sheet Pile Tip Elevation NAVD88 | Sheet Pile Penetration (D) (ft) | Maximum MOWL $D/H_1 = 3/1$ NAVD88 | Maximum MOWL - 4 Ft Water on Wall NAVD88 |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| 1A | 3.6 | -1.6 | -28.5 | 26.9 | 10.0 | 7.6 | 12A | Levee | | | | | |
| 1B | 3.6 | -1.6 | -28.5 | 26.9 | 10.0 | 7.6 | 12B | Levee | | | | | |
| 1C | 3.6 | -1.6 | -28.5 | 26.9 | 10.0 | 7.6 | 13A | 8.0 | 8.0 | -1.3 | **9.3** | 10.0 | 10.0 |
| 1D | 3.6 | -1.6 | -28.5 | 26.9 | 10.0 | 7.6 | 13B | 8.0 | 8.0 | -1.3 | **9.3** | 10.0 | 10.0 |
| 2 | 3.2 | -2.3 | -28.5 | 26.2 | 10.0 | 7.2 | 14 | 8.0 | 8.0 | -1.3 | **9.3** | 10.0 | 10.0 |
| 3 | T-wall | | | | | | 15 | 8.0 | 8.0 | -1.3 | **9.3** | 10.0 | 10.0 |
| 4 | 3.3 | -1.8 | -28.5 | 26.7 | 10.0 | 7.3 | 16 | 8.0 | 8.0 | -1.3 | **9.3** | 10.0 | 10.0 |
| 5 | T-wall | | | | | | Harrison Avenue | | | | | | |
| Harrison Avenue | | | | | | | 17 | 5.5 | 5.5 | -9.8 | 15.3 | 10.0 | 9.5 |
| 5 | T-wall | | | | | | 18A | 5.9 | 6.1 | -9.8 | 15.7 | 10.0 | 9.9 |
| 6 | T-wall | | | | | | 18B | 5.4 | 5.4 | -9.8 | 15.2 | 10.0 | 9.4 |
| 7 | T-wall | | | | | | Filmore Avenue | | | | | | |
| 8 | T-wall | | | | | | 19 | 4.0 | 4.0 | -15.3 | 19.3 | 10.0 | 8.0 |
| Filmore Avenue | | | | | | | Robert E. Lee | | | | | | |
| 9 | T-wall | | | | | | 20A | 6.5 | 6.5 | -1.5 | **8.0** | 9.2 | 10.0 |
| Robert E. Lee Avenue | | | | | | | 20B | Levee | | | | | |
| 10A | 7.5 | 7.5 | -1.5 | **9.0** | 10.0 | 10.0 | 21 | Levee | | | | | |
| 10B | Levee | | | | | | | | | | | | |
| 11 | Levee | | | | | | | | | | | | |

## TABLE 7-4
## CWALSHT STABILITY ANALYSIS OF I-WALLS, CASE "A"

| WEST REACH | LOWEST CALCULATED SHEET PILE TIP GRADE NAVD88 | MODE SWEEP OR FIXED | AS-BUILT SHEET PILETIP GRADE NAVD88 | STRENGTH REDUCTION (PSF) | EAST REACH | LOWEST CALCU-LATED SHEET PILE TIP GRADE NAVD88 | MODE SWEEP OR FIXED | AS-BUILT SHEET PILETIP GRADE NAVD88 | STRENGTH REDUC-TION (PSF) |
|---|---|---|---|---|---|---|---|---|---|
| 1A[1] | -11.69 | Sweep | -28.5 | 480 | 12A | Levee | | | |
| 1B[1] | -11.69 | Sweep | -28.5 | 480 | 12B | Levee | | | |
| 1C[1] | -11.69 | Sweep | -28.5 | 480 | 13A | 5.18 | Sweep | -1.3 | 450 |
| 1D[1] | -11.69 | Sweep | -28.5 | 480 | 13B | 5.20 | Sweep | -1.3 | 450 |
| 2 | -13.33 | Sweep | -28.5 | 600 | 14 | 5.22 | Sweep | -1.3 | 440 |
| 3 | T-wall | | | | 15 | 5.24 | Sweep | -1.3 | 500 |
| 4 | -10.51 | Sweep | -28.5 | 425 | 16 | 5.13 | Sweep | -1.3 | 400 |
| 5 | T-wall | | | | Harrison Avenue | | | | |
| Harrison Avenue | | | | | 17 | 2.99 | Sweep | -9.8 | 0 |
| 5 | T-wall | | | | 18A | 2.75 | Sweep | -9.8 | 0 |
| 6 | T-wall | | | | 18B | 2.21 | Sweep | -9.8 | 0 |
| 7 | T-wall | | | | Filmore Avenue | | | | |
| 8 | T-wall | | | | 19 | -1.61 | Sweep | -15.3 | |
| Filmore Avenue | | | | | Robert E. Lee Avenue | | | | |
| 9 | T-wall | | | | 20A | 4.13 | Sweep | -1.5 | 100 |
| Robert E. Lee | | | | | 20B | Levee | | | |
| 10A | 5.38 | Sweep | -1.5 | 800 | 21 | Levee | | | |
| 10B | Levee | | | | | | | | |
| 11 | Levee | | | | | | | | |
| [1] Reaches 1A-1D are the same CWALSHT analysis since the flood side and protected side embankment crests are the same. | | | | | | | | | |



For case "b" the CWALSHT program was not able to generate a meaningful solution for any of the analyzed reaches because the active soil pressures were less than the passive soil pressures and the protected side water level was always less than the canal water level. The results of the CWALSHT analyses are included in Appendix D.7. The structural analysis of the sheet piles was performed during the original design and is included in DM 19 [6] (see Appendix E).

### *7.1.4 T-Wall Stability*

The original construction of the Orleans Avenue Canal parallel protection system included T-walls in Reaches 3 and 5 through 9 on the West side of the canal. These pile supported T-walls were designed in accordance with the Corps guidelines current at the time of their design. An analysis of the "as-built" [11] wall sections was performed in accordance with the guidelines of Section 4.10 of this report. "As-built" cross sections of the T-wall sections are included in Appendix E.

The subsurface profiles and shear strength and unit weight parameters used in the analyses are defined in Appendix A.2. According to the "as built" cross sections, the sheet pile cutoff walls beneath the T-walls, extend to the tip grades ranging from El -21.5 to El -40 NAVD88 as shown in Table 6-5. These grades depend on the subsurface conditions within the reaches. The sheet piles penetrate into, but not through, the barrier beach sand stratum. The sheet pile walls were assumed impervious for the seepage analysis as recommended by the TRT. The T-wall sections were analyzed in accordance with the guidelines of Section 4.10 of this report.

The limit equilibrium analysis was performed using the Spencer's Method [5] of analysis with the canal water surface at El 10 NAVD88 and using only a block search routine beneath the T-wall. The analyses were performed assuming that the various T-wall pile foundations were present. The FOS values for the T-wall sections were calculated and if found greater than 1.5 for the Spencer's Method [5] of analysis and 1.3 for the MOP analysis, it was assumed that there are no unbalanced soil loads acting on the walls and no distributed loads acting on the foundation pile systems. The calculations were initially performed for a MOWL of El 10 NAVD88. Where a FOS of 1.5 could not be achieved for the Spencer's Method of analysis the canal water level was reduced and unbalanced loads were developed for the reach. Table 7-5 provides a summary of the estimated unbalanced loads and the resulting calculated values of FOS for the various T-Wall reaches. The slope stability and T-wall calculations and output files are included in Appendix D.4.

The ENSOFT program, Group 7 [36], was used to analyze the pile groups for the T-wall reaches. A typical pile group layout for each reach was used in the analysis, based on the "as-built" drawings [11]. Since there was no unbalanced soil load for Reach 3, only the water load acting on the T-wall was applied to the pile group. For Reaches 5 through 9, the water loads and unbalanced soil loads were applied to the pile groups. Two analyses were performed for each reach. The total unbalanced load was applied to the first row of each pile group and also to both rows of each pile group. The calculated deflection was similar for both cases. The water load calculations are included in Appendix D.4.

**TABLE 7-5**
**ESTIMATED UNBALANCED T-WALL LOADS**

| WEST REACH | WATER LEVEL NAVD88 | BASE ELEVATION T-WALL NAVD88 | BASE ELEVATION FAILURE PLANE SPENCER'S NAVD88 | SPENCER'S FOS W/O UNBALANCED LOADS | SPENCER'S UNBALANCED LOAD (LBS/FT OF WALL) | MOP FOS W/O UNBALANCED LOADS | MOP UNBALANCED LOAD (LBS/FT OF WALL) |
|---|---|---|---|---|---|---|---|
| 3 | 10.0 | -5.8 | -12.8 | 1.85 | 0 | 1.63 | 0 |
| 5[1] | 10.0 | -5.5 | -16.3 | 1.30 | 3300 | 1.12 | 1969 |
| 6 | 10.0 | -5.5 | -17.5 | 1.25 | 3700 | 1.15 | 2059 |
| 7 | 8.0 | -5.5 | -21.0 | 1.30 | 3900 | 1.10 | 1050 |
| 8 | 7.0 | -5.5 | -21.3 | 1.34 | 3950 | 1.26 | 526 |
| 8 | 8.0 | -5.5 | -21.3 | 1.26 | 5350 | 1.18 | 1789 |
| 9 | 10.0 | -5.5 | -26.2 | 1.37 | 2500 | 1.39 | 0 |

**Note:**
[1] Harrison Avenue Bridge is located within this reach between Stations 36+26 to 37+27.

The piles were assumed to be pinned and not fixed in the pile cap. This assumption was conservative and resulted in larger pile head deflections. The "S" and "Q" cases of pile capacity analysis relate to the use of S or Q strengths in the analysis [30]. The S-case strength values were obtained from HSDRRSDG table 3.9 [4]. The Q strength is obtained from unconsolidated undrained tests. It was determined that the "Q" case produced more conservative end bearing and side friction values. Table 7-6 provides the pile head deflections and the pile stresses for each T-Wall reach.

The guidelines of Section 4.10 of this report indicate that deflections are required to be less than 0.75 inch at the MOWL. The pile head deflection in Reach 8 was estimated to be 1.54 inches for a MOWL of El 8.0 NAVD88. The MOWL was reduced to El 7.0 NAVD88 and the pile head deflection was reduced to 0.46 inch as shown in Table 7-6. The magnitude is very sensitive to a 1-foot variation in water level. Therefore, more detailed finite element method (FEM) analysis of this reach will be conducted during the remediation phase with other deficient reaches to further define the deflection magnitude. The actual deflection and pile stresses will be lower than indicated in Table 7-6 under the actual loading condition. The unbalanced soil load is not applied to the piles until stability failure is imminent when the FOS approaches one. During Katrina the walls withstood a canal water level to El 11.1 NAVD88 with no noted distress.

When all of the various analysis results were considered, the MOWL for the T-walls in Reaches 3, 5, 6, and 9 are greater than El 10 NAVD88 and the MOWL for the T-walls in Reaches 7 and 8 are estimated to be El 8 NAVD88.

Structural analyses of the T-walls, performed for an MOWL of El 10 NAVD88, for all reaches except Reach 8 indicated that the amount of reinforcement in the walls and the base slabs satisfies both the current HSDRRSDG [4] and *EM 1110-2-2104, Strength Design for Reinforced Concrete Hydraulic Structures* [29]. Structural analysis performed at a water level of El 8 NAVD88 for Reach 8 indicated that the amount of reinforcement in both the walls and the footings is sufficient. The T-walls are tapered to only 12 inches wide at the top of the stem. This width does not meet current structural criteria for T-Walls.



**TABLE 7-6 PILE**
**DEFLECTIONS AND MAXIMUM PILE STRESS UNDER UNBALANCED AND WATER LOADS**

| WEST REACH | MOWL NAVD88 | CALCULATED DEFLECTION WITH WATER AND UNBALANCED LOADS APPLIED TO ROW(S) 1/ 1 & 2 (IN) | MAX STRESS LOAD ON FIRST PILE ROW (PSI) | | MAX STRESS LOAD ON BOTH PILE ROWS (PSI) | |
|---|---|---|---|---|---|---|
| | | | COMPRESSION | TENSION | COMPRESSION | TENSION |
| 3 | 10.0 | 0.03/0.04[1] | 969 | 222 | 441 | 262 |
| 5 | 10.0 | 0.39/0.39 | 919 | 702 | 899 | 690 |
| 6 | 10.0 | 0.61/0.61 | 1210 | 866 | 990 | 831 |
| 7 | 8.0 | 0.46/0.46 | 1220 | 756 | 785 | 657 |
| 8 | 7.0 | ---[2]/0.46 | 910 | 738 | 583 | 406 |
| 8 | 8.0 | ---[2]/1.54 | 1410 | 1390 | 1260 | 1040 |
| 9 | 10.0 | 0.07/0.10 | 390 | 457 | 260 | 323 |

[1]Water load only
[2] Not calculated for Row 1

### *7.1.5 Pump Station Wall Structural Stability*

The DPS 7 exterior pump station brick foundation wall was evaluated for an MOWL of El 8 NAVD88. This brick wall was constructed in the early 1930s and the mortar used likely equivalent to "Type N" mortar listed in Table 2.2.3.2 of ACI 530-08 [37]. The code lists an allowable stress of 30 psi for this type mortar. The calculated maximum flexural tensile stress in the wall for an MOWL of El 8 NAVD88 is 36.4 psi. This indicates an overstress of about 21%. However, historically, the wall has withstood water levels above El 8 NAVD88. If it can be shown that a stronger mortar was used during construction, such as Type M or Type S, which have allowable stresses of 40 psi, then the analysis indicates a satisfactory design for an MOWL of El 8 NADV88. Structural calculations are included in Appendix D.5.

### *7.1.6 Pump Station Sliding Stability*

The overall sliding stability of DPS 7 was evaluated using the Spencer's Method [5] in SLOPE/W [34] and the MOP [35]. The pile foundations were not included for this analysis, which is conservative. Gap analysis was not used for this evaluation since the structure is pile supported. Two sections were evaluated for the pump station due to the significant differences in the structure cross sections from east to west. A west side section and an east side section were evaluated. The soil parameters from Reach 1 adjacent to the pump station were used for the west side analysis. The soil parameters from Reach 12 adjacent to the pump station were used for the east side analysis.

The Spencer's Method and MOP FOS values for global stability were evaluated for a MOWL of El 10 NAVD88. The lowest computed Spencer's Method FOS was 2.36 and the lowest MOP FOS was 1.67. These results indicate there are no unbalanced loads to be applied to the foundation piles. The analyses are provided in Appendix D.6.

### *7.1.7 Seepage Analysis*

The seepage analyses performed for this study assumed that a gap forms along the flood side of the I-wall when the water level in the canal is equal to the embankment crest elevation. If the canal water level was below the crest of the levee, no gap was considered. A constant head boundary was established at a distance of 110 feet from the I-walls and T-walls based on discussions with the TRT. This constant head boundary was set at 2 feet below ground surface grade. In addition, the sheet pile was considered impermeable for all analyses.

#### ***7.1.7.1*** *Canal Bottom Sediments Analysis*

The bathymetric survey indicated that in Reach 1 and opposite Reaches12 and 13, the base of the canal was near the elevation of the top of the barrier beach sand stratum. This assessment was based on field investigation data along both sides of the canal. The possibility exists that a potential direct connection could occur between the canal water and the barrier beach sand stratum. This could result in elevated piezometric pressures at the bottom of the marsh clay stratum on the protected sides of the canal. Reaches 1A, 1B, 1C, 12A, 12B, 13A and 13B were analyzed assuming beach sand in the base of the canal as described in the Section 7.1.7.3. See Soil and Geologic Cross Sections A-A' shown on Plates 41 in Appendix A.3 show the variation in the canal bottom elevation relative to the top elevation of the beach sand stratum.

As part of this study, no borings were taken in the canal so the thickness classification of the canal sediments overlying the barrier beach sand stratum could not be evaluated. It was assumed that these reaches contained semi-impervious canal sediments. These sediments cause a reduction in the head due to seepage and reduced piezometric pressures below the protected side marsh clay stratum relative to the condition of direct contact with the beach sand. Verification sampling in the canal can be undertaken to evaluate whether these assumed conditions exist

#### ***7.1 7.2*** *Canal Piezometer Seepage Analysis*

A series of seepage analyses were performed and the results were compared to measured piezometric readings to evaluate the potential for a direct connection between the canal and the barrier beach sand below the canal. The analysis was performed considering the head loss from the bottom of the canal to piezometers located along the canal. The piezometers are sealed within the barrier beach sand stratum. Piezometers OP-7 and OP-9 are located within Reaches 1C and 5 on the west side of the canal and piezometer OP-8 is located with Reach 15 on the east side of the canal. These piezometers are located along the protected side toe of the levees. Piezometer OP-12 is also located on the east side of the canal, but is about 700 feet away from the canal. This was considered to provide "background" groundwater fluctuation in the area. The locations of these piezometers are plotted on Plates 1 through 3 of Appendix A.3.

The canal water levels were obtained from www.rivergauges.com from a gage located at the Harrison Avenue Bridge (approximate Station 37+00). Daily water levels were obtained for the canal for the period January 1, 2008, to February 1, 2010. These fluctuations are shown on Figure 7-1. The highest measured canal water level was El 4.8 NAVD88 which occurred on September 13, 2008 and was attributed to Hurricane Ike. A second high water level El 4.5 NAVD88, occurred nine days earlier on September 4, 2008 and was attributed to Hurricane Gustav. The piezometers readings are also plotted on Figure 7-1 for the same period as the canal water levels. Figure 7-1 indicates that Piezometer OP-9 reported several isolated spikes in water levels which did not coincide with the changes in the canal water levels. The durations of the spikes are very short in duration, on the order of 1 to 2 hours. Based on review of 24-hour rainfall [8], these spikes correspond to rainfall events and appear to be due to surface water infiltration into the piezometer as shown by Figure 7-2. The surface seal and piezometer cap are missing.

Review of the piezometer data indicates that the readings from Piezometers OP-7 and OP-8 are generally 1.5 to 2.0 feet higher than the readings from Piezometers OP-9 and OP-12. Piezometers OP 7 and OP-8 are located closer to the Lake. These data suggest a general trend of higher groundwater levels as the Lake is approached. This trend was confirmed by readings from piezometers closer to the Lake. Review of the data in Figure 7-1 suggests that a direct connection of the canal water to the beach sand deposit does not exist north of Harrison Ave.

As an additional check of the piezometer observations, seepage analyses were performed using SEEP/W [34]. The stratigraphy used in the analysis was developed from field investigations data in the area of the piezometers. The canal bottom was modeled for two cases. In Case 1 the canal bottom was assumed to be beach sand. In Case 2, the bottom of the canal was assumed to consist of marsh clay or semi-impervious canal sediments.

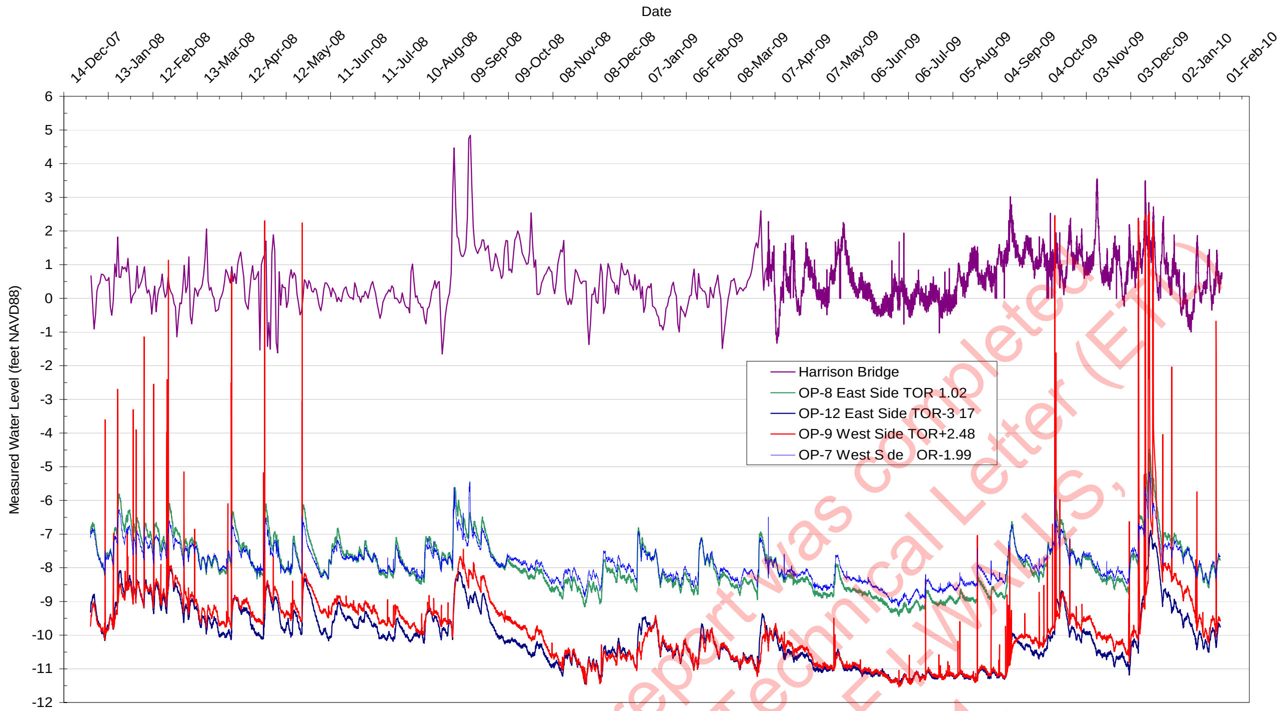


**FIGURE 7-1**
**CANAL AND PIEZOMETER READINGS 1/1/2008 TO 2/1/2010**

**FIGURE 7-2**
**PIEZOMETER OP-09 WITHOUT CAP OR SURFACE SEAL**

In both cases the canal water level was El 4.8 NAVD88, which corresponds to the high canal water level on September 13, 2008. At this canal level, the water surface is below the crest of the flood side levee embankment for the east canal reaches under consideration, but above the flood and protected side crest of the levee embankment on the west side of the canal. Therefore, a gap could form along the west canal reaches represented by piezometers OP-7 and OP-9. Thus, a second seepage path to the beach sands could exist during the time the canal water level was evaluated. The modeled hydraulic heads at the piezometer locations were compared to the actual recorded piezometer responses. The results of the seepage analyses are shown in Table 7.7. These model results are included in Appendix D.4.

**TABLE 7-7**
**COMPARISON OF PIEZOMETER READINGS AND CALCULATED TOTAL HEADS FOR A CANAL WATER LEVEL OF EL 4.8 NAVD88**

| PIEZOMETER NUMBER | REACH | PIEZOMETER WATER LEVEL (NAVD88) | CALCULATED PIEZOMETER WATER LEVEL (FT NAVD88) | |
|---|---|---|---|---|
| | | | Case 1 | Case 2 |
| OP-7 | 5 | 5.5 | -1.2 | -7.8 |
| OP-8 | 15 | 6.5 | -0.8 | -3.9 |
| OP-9 (gap) | 1C | -8.1 | -1.1 | -2.9 |
| OP-9 (no gap) | 1C | -8.1 | --- | -5.5 |

The analyses indicate that the calculated piezometer water levels are significantly higher than the measured piezometer readings for Case 1 where the canal bottom was assumed be the barrier beach sand. When Case 2 was evaluated, where the bottom of the canal was assumed to consist of marsh clay or semi-impervious canal sediments, the calculated water levels were slightly higher than the measured readings. The influence of the formation of a gap compared to no gap formation was evaluated for Reach 1C using piezometer OP-9 data for Case 2. When no gap was assumed with marsh clay or semi-impervious canal sediments in the bottom of the canal the calculated piezometer water level was lower at El -5.5 NAVD88. These results support the conclusion that a direct connection of the canal water to the barrier beach sand deposit does not exist north of Harrison Ave.

#### *7.1.7.3 Canal Seepage Analysis*

The overall canal seepage analysis was performed using SEEP/W [34]. The exit gradients at the ground surface on the protected side were calculated at three locations: 1) at the protected side of the sheet pile, 2) at the protected side mid-slope, and 3) at the protected side toe. In all cases, the toe location controlled. The minimum calculated seepage FOS, as indicated by the guidelines of Section 4.11, is 1.6. The seepage FOS is defined as the critical exit gradient divided by calculated exit gradient. The uplift pressures below the marsh clay were also calculated for each reach, but a heave analysis was not required for this study due to the use of finite element seepage analyses. The results of the seepage analysis are presented in Table 7-8. The seepage analyses for the I-walls and earth levees using SEEP/W are located in Appendix D.3 and the input and output files are located in Appendix E. The seepage analyses for the T-walls using SEEP/W are located in Appendix D.4 and the input and output files are located in Appendix E.



**TABLE 7-8**
**SEEPAGE ANALYSIS RESULTS**

| West Reach | Gap Bottom Elevation NAVD88 | Canal Bottom Assumed Poorly Graded Sand | Under-seepage MOWL FOS ≥ 1.6 at Levee Toe NAVD88 | East Reach | Gap Bottom Elevation NAVD88 | Canal Bottom Assumed Poorly Graded Sand | Under-seepage MOWL FOS ≥ 1.6 at Levee Toe NAVD88 |
|---|---|---|---|---|---|---|---|
| 1A | -7 | Yes | **6.5** | 12A | Levee | Yes | **8.5**[1] |
| 1B | -7 | Yes | **5.0** | 12B | Levee | Yes | **8.5**[1] |
| 1C | -7 | Yes | **5.5** | 13A | -1.3 | Yes | **7.0** |
| 1D | -7 | No | 10.0 | 13B | -1.3 | Yes | **6.5** |
| 2 | -13 | No | 10.0 | 14 | -1.3 | No | 10.0 |
| 3 | T-wall | | 10.0 | 15 | -1.3 | No | 10.0 |
| 4 | -13 | No | 10.0 | 16 | -1.3 | No | 10.0 |
| 5 | T-wall | | 10.0 | Harrison Avenue | | | |
| Harrison Avenue | | | | 17 | -9.8 | No | 10.0 |
| 5 | T-wall | | 10.0 | 18A | -6.6 | No | 10.0 |
| 6 | T-wall | | 10.0 | 18B | -7.4 | No | 10.0 |
| 7 | T-wall | | 10.0 | Filmore Avenue | | | |
| 8 | T-wall | | 10.0 | 19 | -6.5 | No | 10.0 |
| Filmore Avenue | | | | Robert E. Lee Avenue | | | |

| West Reach | Gap Bottom Elevation NAVD88 | Canal Bottom Assumed Poorly Graded Sand | Under-seepage MOWL FOS ≥ 1.6 at Levee Toe NAVD88 | East Reach | Gap Bottom Elevation NAVD88 | Canal Bottom Assumed Poorly Graded Sand | Under-seepage MOWL FOS ≥ 1.6 at Levee Toe NAVD88 |
|---|---|---|---|---|---|---|---|
| 9 | T-wall | | 10.0 | 20A-clay | -1.5 | No | 10.0 |
| Robert E. Lee Avenue | | | | 20A-sand | -1.5 | No | **9.0** |
| 10A | -1.5 | No | 10.0 | 20B-clay | Levee | No | 10.0 |
| 10B | Levee | No | 10.0 | 20B-sand | Levee | No | 10.0 |
| 11-clay | Levee | No | 10.0 | 21 | Levee | No | 10.0 |
| 11-sand | Levee | No | 10.0 | | | | |
| [1]Seepage analysis performed at maximum water level for stability for these reaches | | | | | | | |

The results of the seepage analysis were significantly affected by the following.

- Thickness of the marsh clay stratum;
- Propagation of a full potential gap when the canal water level reaches the crest of the flood side levee embankment;
- Propagation of the gap through the marsh clay stratum;
- Low ground surface elevation of the protected side levee toe;
- Presence or absence of a continuous silty sand layer below the marsh clay stratum at the top of the barrier beach sand stratum; and
- Presence or absence of semi-impervious canal bottom sediment blanket.

The lowest MOWL values were identified in Reaches 1A, 1B, 1C, 13A and 13B where the natural semi-impervious canal bottom sediments were assumed to be absent and the bottom of the canal was assumed to consist of the barrier beach sand stratum. These MOWL values ranged from a low of El 5.0 NAVD88 to a high of 7.0 NAVD88. Six reaches containing hydraulic fill were evaluated as either: 1) a stratum consisting of layers of sand and clay; or 2) a stratum consisting of sand. Reaches 11 and 20B indicated the same MOWL for both subsurface conditions, El 10 NAVD88. For Reach 20A, the sand assumption produced a slightly lower MOWL of El 9 NAVD88.

The T-walls were designed with sheet piles tip grades in the barrier beach sand stratum. The suitability of the lengths of the sheet pile for the T-walls was checked using the Lane Weighted Creep Ratio (LWCR) [28]. The sheet pile walls are considered impermeable and the maximum differential head is the difference between the elevation of the exit point of the potential seepage path and the canal water elevation. The differential pressure is dissipated in the clay fill, marsh clay and lacustrine strata. Three seepage paths were considered:

- Case 1 - Seepage at the toe of the T-Wall exiting vertically at the top of the protected side earth levee; and
- Case 2-Seepage exiting at the toe of the protected side earth levee;
- Case 3 - Seepage through the beach sand exiting at the toe of the protected side levee.

The base width of the T-wall for each reach was determined from the “as-built” drawings [11] included in Appendix E. The calculation was based on assuming the canal water level was at El 10 NAVD88.

The LWCR is defined as

$$L_w > CH$$

where $L_w$ = weighted seepage length N/3+V + 2N’: N = horizontal seepage length at interface below foundation (assumed zero to account for roofing at the base of the foundation); N’ = horizontal seepage length through soil; V = vertical seepage length; and C = factor depending on soil type. The recommended creep ratio for the soft clays in the marsh and lacustrine layers is 3 for the beach sand is 7.

The allowable creep ratio is based on a weighted average of the soil types along the seepage path for each seepage path analyzed. The results indicate all reaches meet the required LWCR for all case evaluated for a MOWL of El 10 NAVD88. The calculated LWCR values are shown in Table 7-9. Calculations are provided in Appendix D.4.

**TABLE 7-9**
**LANE WEIGHTED CREEP RATIO FOR T-WALLS**

| REACH NUMBER | MAXIMUM DIFFERENTIAL HEAD AT MOWL EL 10 NAVD88 (FT) | $L_W$ (FT) | CASE | REQUIRED LWCR | CALCULATED LWCR | LWCR MOWL NAVD88 |
|---|---|---|---|---|---|---|
| 3 | 17.5 | 44.7 | 1 | 4.3 | 6.6 | 10.0 |
| | | 92.8 | 2 | 3.9 | 6.9 | 10.0 |
| | | 110.5 | 3 | 5.8 | 8.2 | 10.0 |
| 5 | 18.0 | 57.5 | 1 | 4.3 | 7.8 | 10.0 |
| | | 83.5 | 2 | 4.1 | 5.8 | 10.0 |
| | | 101.5 | 3 | 5.5 | 7.0 | 10.0 |
| 6 | 18.1 | 57.4 | 1 | 3.9 | 7.8 | 10.0 |
| | | 84.6 | 2 | 3.8 | 6.2 | 10.0 |
| | | 102.6 | 3 | 5.1 | 7.5 | 10.0 |
| 7 | 19.2 | 73.2 | 1 | 4.3 | 9.9 | 10.0 |
| | | 103.2 | 2 | 4.2 | 6.4 | 10.0 |
| | | 121.3 | 3 | 5.4 | 7.5 | 10.0 |
| 8 | 18.5 | 73.3 | 1 | 3.6 | 10.0 | 10.0 |
| | | 101.1 | 2 | 3.4 | 6.1 | 10.0 |
| | | 119.1 | 3 | 4.6 | 7.2 | 10.0 |
| 9 | 16.5 | 80.1 | 1 | 4.1 | 10.5 | 10.0 |
| | | 107.6 | 2 | 3.5 | 7.4 | 10.0 |
| | | 131.6 | 3 | 4.7 | 9.1 | 10.0 |

## 7.2 SUMMARY OF MOWL

Stability was the controlling condition for the lowest MOWL identified on both banks of the canal. The FOS calculated by the Spencer's Method analysis for Reaches 10B, 17, 18A, and 20B are slightly less than the required 1.4 with the canal water level at El 1NAVD88, the normal Lake level. This indicated an inadequate FOS without the influence of the canal water load. These low FOS values resulted from the low undrained shear strength values for the levee embankment and underlying marsh clay stratum. Reach 18A was designed with a protected side stabilization berm extending approximately 90 feet from the I-wall. The recent topographic survey performed in 2010 indicates the berm extends only about 30 feet beyond the I-wall. The MOWL values for Reaches 12A, 12B, 15, 16, 18B, and 19 varied from El 5.0 to 8.5 NAVD88.

Reaches 10A, 13A through 16 and 20A, do not meet the minimum sheet pile penetration requirement of 10 feet. The penetration ratio will limit the MOWL of Reach 20A to El 9.2 NAVD88. Limiting the water level to 4 feet on the wall above the earthen levee crest will limit the MOWL to below El 10 NAVD88 for Reaches 1A, 1B, 1C, 1D, 2, 4, 17, 18A, 18B, and 19. The lowest MOWL for this criterion is El 7.2 NAVD88 in Reach 2.

Seepage was found to be a controlling condition for reaches 1A, 1B, 1C, 13A, 13B and 20A. For all of these six reaches except Reach 20 the seepage model assumed barrier beach sand in the bottom of the canal. For Reach 20A, the seepage model was based on a layer of hydraulic sand fill. Verification sampling in the canal can be required to evaluate whether these assumed conditions exist. Reaches 12A and 12B were also assumed to have barrier beach sand in the bottom of the canal. Both reaches were analyzed for an MOWL of 8.5 NAVD88 as this was the crest of the levee for Reach 12B. The crest of the concrete wall for Reach 12A was El 9.7 NAVD88. The seepage FOS met the minimum requirement at a canal water level of El 8.5 NAVD88 for both reaches.

The MOWL values for the T-walls were controlled by wall estimated deflections. Reaches 7 and 8 had MOWL values of El 8.0 and 7.0 NAVD88, respectively. For Reach 8, the deflection at a canal water elevation of El 8 NAVD88 was estimated at about 1.6 inches. The maximum allowable value is 0.75 inch. The guidelines allow a variance on pile head deflection and an MOWL value of El 8.0 NAVD88 can be considered. A MOWL was not provided for the bridges as the bridges are not part of this study and the local geometry at the bridges would not limit or constrain the MOWL.

The MOWL for each reach is tabulated versus each of the individual design criteria in Table 7-10. The elevations in bold identify the controlling criteria below a MOWL of El 10 NAVD88. Table 7-11 provides a summary of the FOS values and deflections for the T-walls and FOS values for DPS 7. Figures 7-3 through 7-7 provides the MOWL for each criterion along east bank of the canal. Figure 7-8 through 7-14 provides the MOWL for each criterion along west bank of the canal.

**TABLE 7-10**
**REACH MOWL VALUES FOR I-WALLS AND EARTH LEVEES**

| REACH | STATION | SPENCER'S METHOD SLOPE STABILITY FOS >1.4 MOWL NAVD88 | MOP SLOPE STABILITY FOS >1.3 MOWL NAVD88 | MINIMUM SHEET PILE PENETRATION D>10 FEET | SHEET PILE PENETRATION RATIO $D/H_1$ = 3/1 MOWL NAVD88 | MAXIMUM 4 FT WATER DEPTH ON I-WALL MOWL NAVD88 | CWALSHT MOWL NAVD88 | SEEPAGE MOWL NAVD88 |
|---|---|---|---|---|---|---|---|---|
| 1A * | 2+45 to 7+00 | 10.0 | 10.0 | Yes | 10.0 | 7.6 | 10.0 | **6.5** |
| 1B * | 7+00 to 9+25 | 10.0 | 10.0 | Yes | 10.0 | 7.6 | 10.0 | **5.0** |
| 1C * | 9+25 to 11+00 | 10.0 | 10.0 | Yes | 10.0 | 7.6 | 10.0 | **5.5** |
| 1D | 11+00 to 14+20 | 10.0 | 10.0 | Yes | 10.0 | **7.6** | 10.0 | 10.0 |
| 2 | 14+20 to 21+75 | 10.0 | 10.0 | Yes | 10.0 | **7.2** | 10.0 | 10.0 |
| 4 | 24+87 to 29+16 | 10.0 | 10.0 | Yes | 10.0 | **7.3** | 10.0 | 10.0 |
| 10A | 91+88 to 93+53 | 10.0 | 10.0 | **No** | 10.0 | 10.0 | 10.0 | 10.0 |
| 10B | 93+53 to 98+70 | **<1.0** | **1.0** | Levee - No Sheet Pile, Crest above El 10.0 NAVD88 | | | | 10.0 |
| 11 | 98+70 to 112+50 | 10.0 | 10.0 | Levee - No Sheet Pile, Crest above El 10.0 NAVD88 | | | | 10.0 |
| 12A * | 2+45 to 3+70 | **8.5** | **8.5** | Levee with wall, Crest, El 9.7, Analysis, El 8.5 NAVD88 | | | | **8.5** |
| 12B * | 3+70 to 4+70 | **8.5** | **8.5** | Levee - No Sheet Pile, Crest, El 8.5 NAVD88 | | | | **8.5** |
| 13A * | 4+70 to 7+00 | 10.0 | 10.0 | **No** | 10.0 | 10.0 | 10.0 | **7.0** |
| 13B* | 7+00 to 11+20 | 10.0 | 10.0 | **No** | 10.0 | 10.0 | 10.0 | **6.5** |
| 14 | 11+20 to 20+50 | 10.0 | 10.0 | **No** | 10.0 | 10.0 | 10.0 | 10.0 |
| 15 | 20+50 to 30+00 | 10.0 | **8.5** | **No** | 10.0 | 10.0 | 10.0 | 10.0 |
| 16 | 30+00 to 36+40 | 10.0 | **<8.0** | **No** | 10.0 | 10.0 | 10.0 | 10.0 |
| 17 | 37+44 to 50+00 | **<1.0** | **1** | Yes | 10.0 | 9.5 | 10.0 | 10.0 |
| 18A | 50+00 to 61+00 | **<1.0** | **1** | Yes | 10.0 | 9.9 | 10.0 | 10.0 |
| 18B | 61+00 to 64+00 | 8.0 | **7.0** | Yes | 10.0 | 9.4 | 10.0 | 10.0 |
| 19 | 65+00 to 90+62 | **5.0** | **5.0** | Yes | 10.0 | 8.0 | 10.0 | 10.0 |
| 20A-E | 92+20 to 93+46 | 10.0 | 10.0 | **No** | 9.2 | 10.0 | 10.0 | **9.0** |
| 20B-E | 93+46 to 101+50 | **<1.0** | **<1.0** | Levee - No Sheet Pile, Crest above El 10.0 NAVD88 | | | | 10.0 |
| 21-E | 101+50 to113+05 | 10.0 | 10.0 | Levee - No Sheet Pile, Crest above El 10.0 NAVD88 | | | | 10.0 |

* Indicates reach with canal bottom assumed to be in barrier beach sand

## TABLE 7-11
## REACH MOWL VALUES FOR T-WALLS AND DPS 7

| REACH | STATION | CANAL WATER ELEVATION NAVD88 | SPENCER'S METHOD FOS W/O UNBALANCED LOADS | MOP FOS W/O UNBALANCED LOADS | CALCULATED DEFLECTION WITH UNBALANCED LOADS (IN) | LWCR MOWL NAVD88 |
|---|---|---|---|---|---|---|
| 3 | 21+75 to 24+87 | 10.0 | 1.85 | 1.63 | 0.04 | 10.0 |
| 5 | 29+16 to 42+00 | 10.0 | 1.30 | 1.12 | 0.39 | 10.0 |
| 6 | 42+00 to 50+00 | 10.0 | 1.25 | 1.15 | 0.61 | 10.0 |
| 7 | 50+00 to 59+00 | **8.0** | 1.30 | 1.22 | 0.46 | 10.0 |
| 8 | 59+00 to 63+58 | **7.0** | 1.34 | 1.26 | 0.46 | 10.0 |
| | | **8.0** | 1.26 | 1.18 | 1.54 | 10.0 |
| 9 | 65+00 to 90+27 | 10.0 | 1.37 | 1.39 | 0.07 | 10.0 |
| | | | **Spencer's Method FOS** | **MOP FOS** | | |
| DPS 7 | | 10.0 | 2.36 | 1.67 | | |

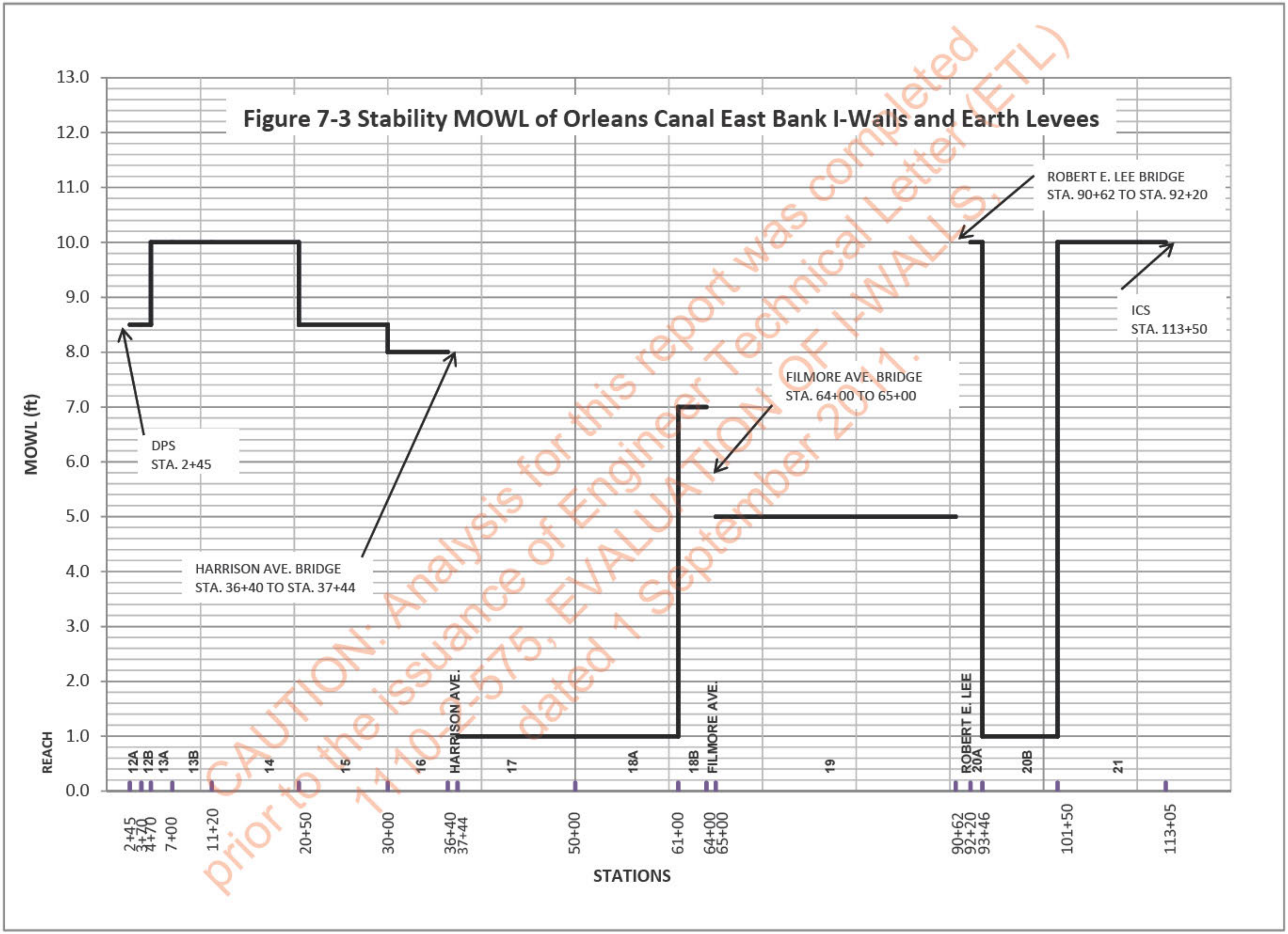
Figure 7-3 Stability MOWL of Orleans Canal East Bank I-Walls and Earth Levees
MOWL (ft)
13.0
12.0
11.0
10.0
9.0
8.0
7.0
6.0
5.0
4.0
3.0
2.0
1.0
0.0
REACH
DPS
STA. 2+45
HARRISON AVE. BRIDGE
STA. 36+40 TO STA. 37+44
FILMORE AVE. BRIDGE
STA. 64+00 TO 65+00
ROBERT E. LEE BRIDGE
STA. 90+62 TO STA. 92+20
ICS
STA. 113+50
12A
12B
13A
13B
14
15
16
HARRISON AVE.
17
18A
18B
FILMORE AVE.
19
ROBERT E. LEE
20A
20B
21
2+45
3+70
4+70
7+00
11+20
20+50
30+00
36+40
37+44
50+00
61+00
64+00
65+00
90+62
92+20
93+46
101+50
113+05
STATIONS

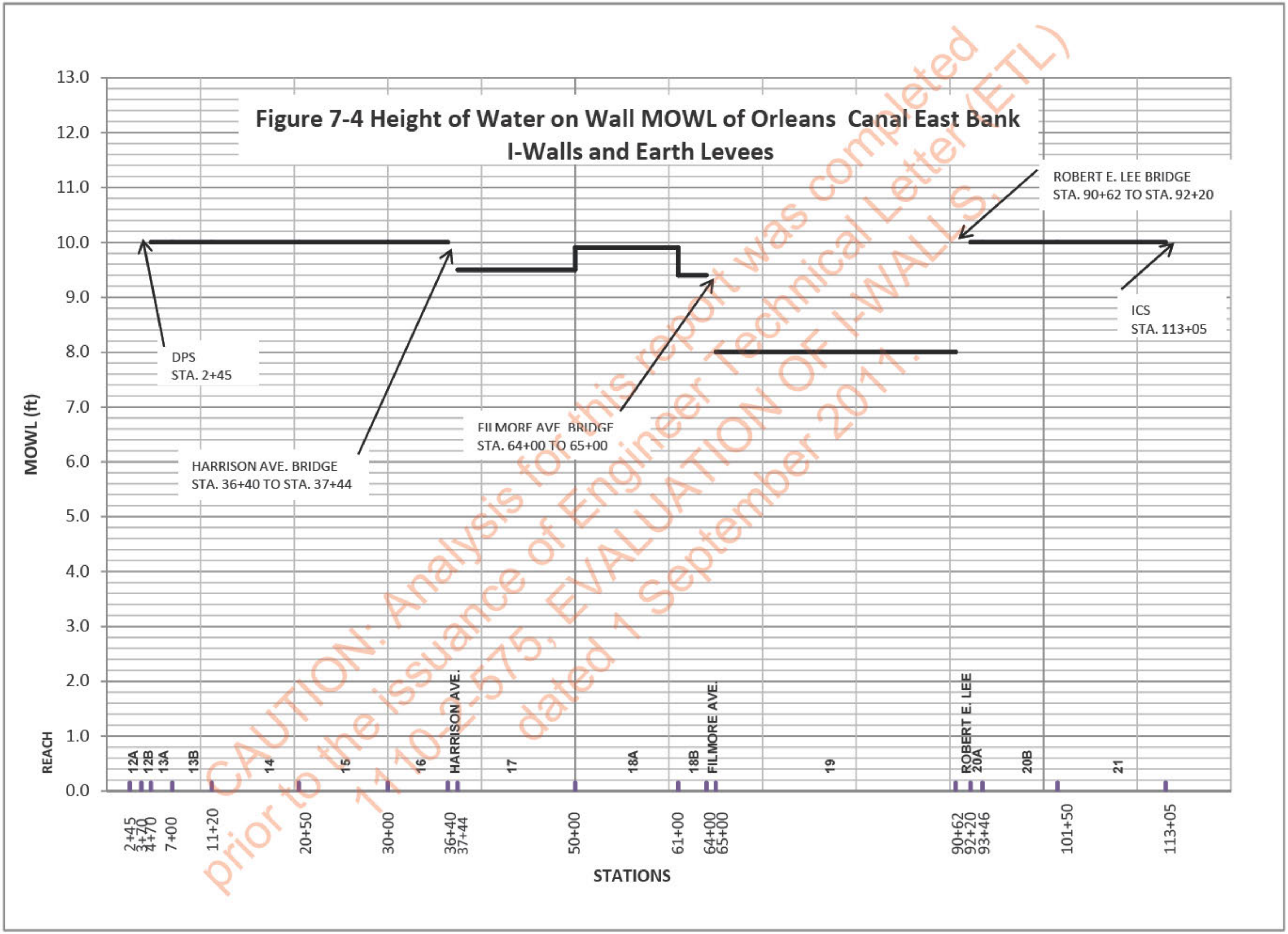

Figure 7-4 Height of Water on Wall MOWL of Orleans Canal East Bank
I-Walls and Earth Levees
MOWL (ft)
REACH
STATIONS
DPS
STA. 2+45
HARRISON AVE. BRIDGE
STA. 36+40 TO STA. 37+44
FILMORE AVE. BRIDGE
STA. 64+00 TO 65+00
ROBERT E. LEE BRIDGE
STA. 90+62 TO STA. 92+20
ICS
STA. 113+05
12A
12B
13A
13B
14
15
16
HARRISON AVE.
17
18A
18B
FILMORE AVE.
19
ROBERT E. LEE
20A
20B
21
2+45
3+70
4+70
7+00
11+20
20+50
30+00
36+40
37+44
50+00
61+00
64+00
65+00
90+62
92+20
93+46
101+50
113+05

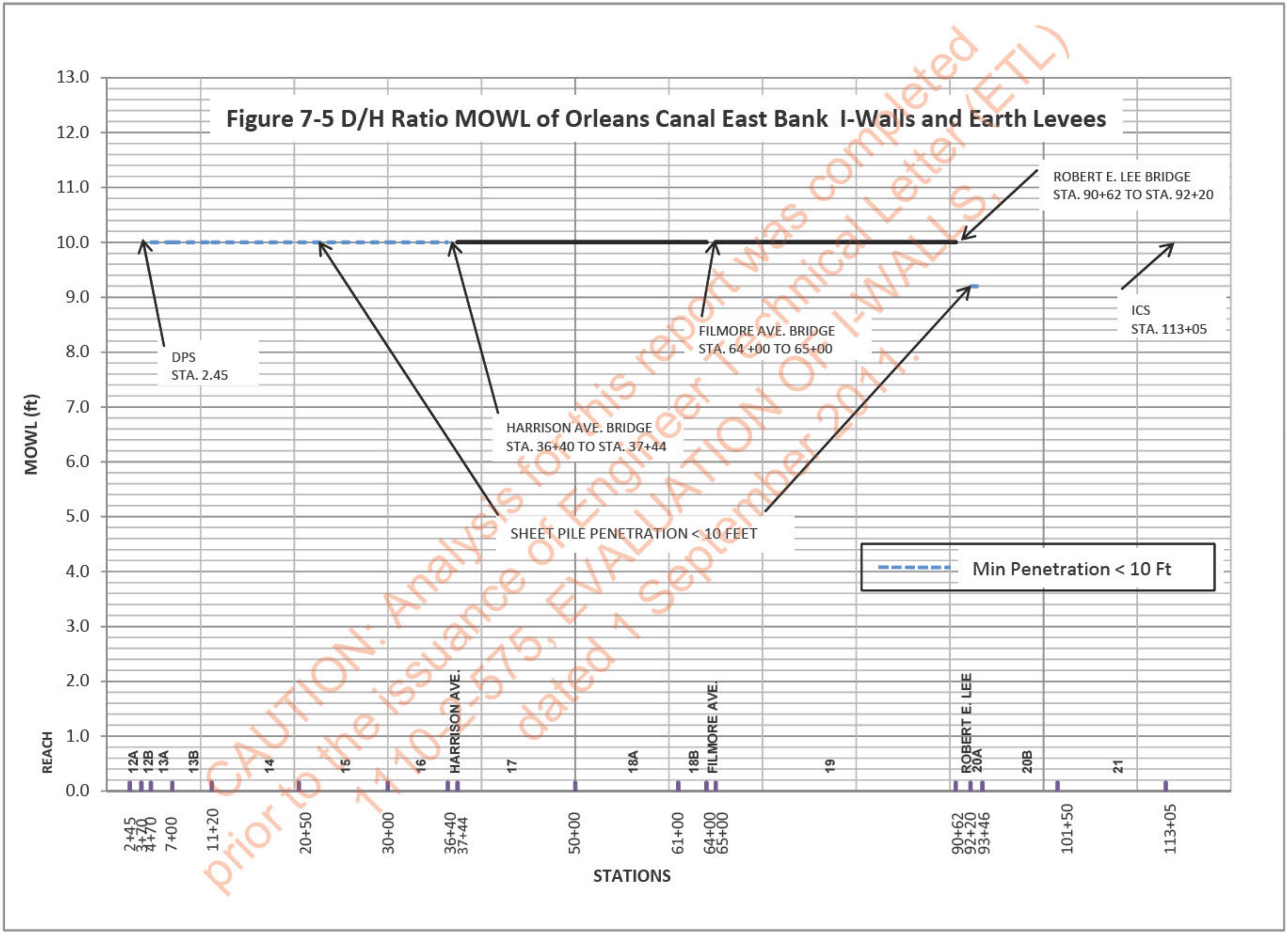


**Figure 7-5 D/H Ratio MOWL of Orleans Canal East Bank I-Walls and Earth Levees**

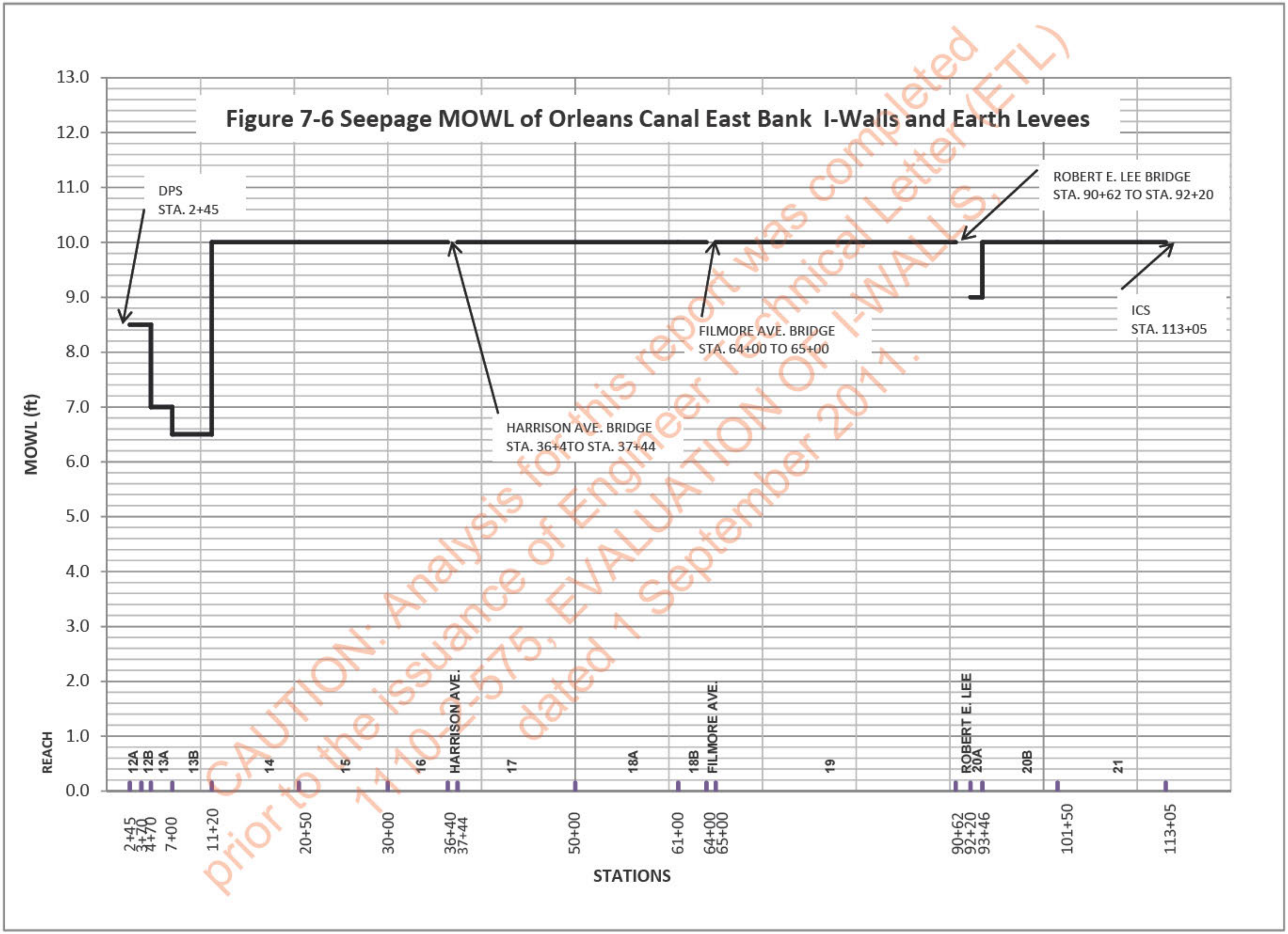
Figure 7-6 Seepage MOWL of Orleans Canal East Bank I-Walls and Earth Levees
MOWL (ft)
REACH
STATIONS
DPS
STA. 2+45
HARRISON AVE. BRIDGE
STA. 36+4TO STA. 37+44
FILMORE AVE. BRIDGE
STA. 64+00 TO 65+00
ROBERT E. LEE BRIDGE
STA. 90+62 TO STA. 92+20
ICS
STA. 113+05
12A
12B
13A
13B
14
15
16
HARRISON AVE.
17
18A
18B
FILMORE AVE.
19
ROBERT E. LEE
20A
20B
21
2+45
3+70
4+70
7+00
11+20
20+50
30+00
36+40
37+44
50+00
61+00
64+00
65+00
90+62
92+20
93+46
101+50
113+05

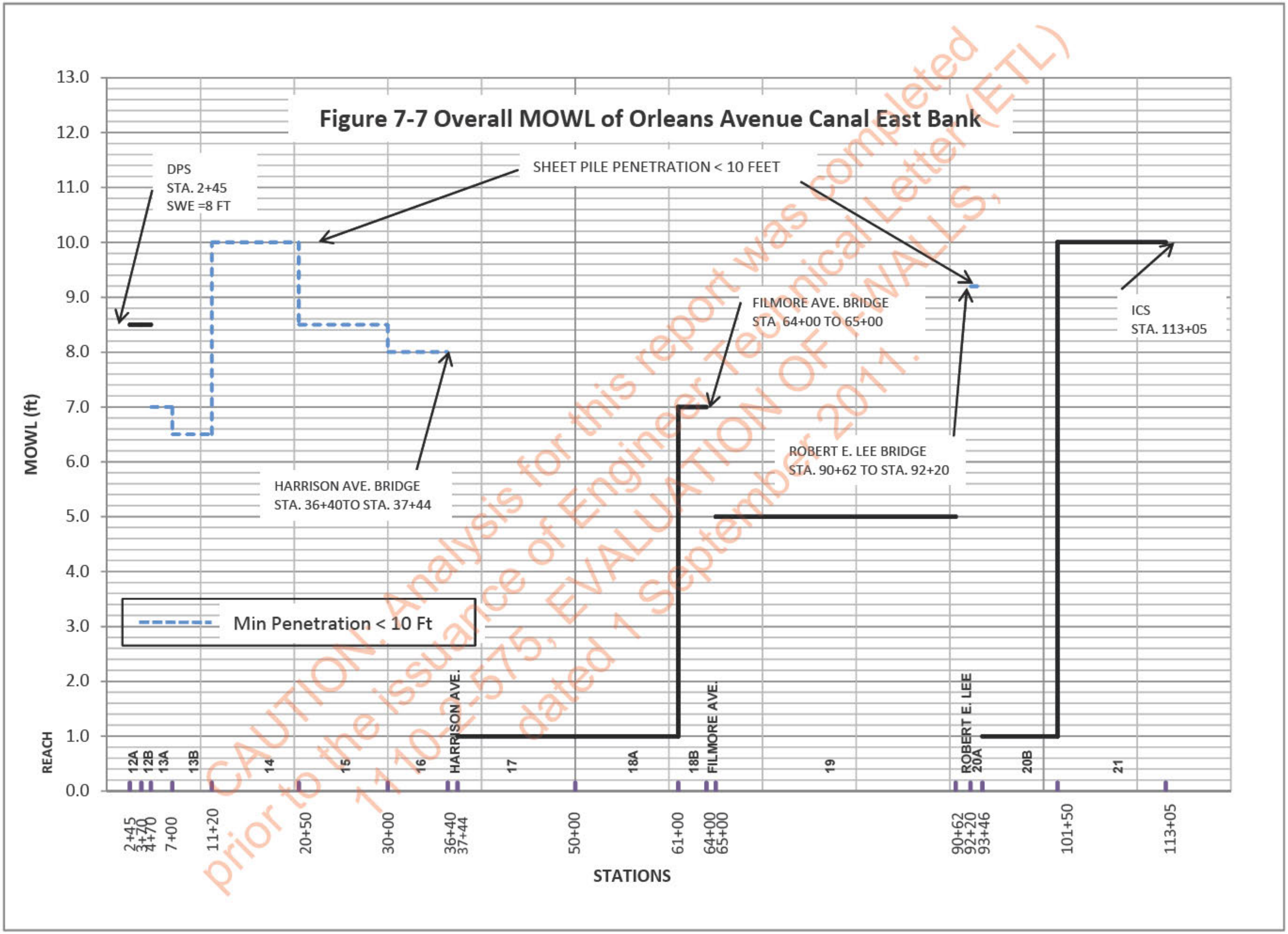

Figure 7-7 Overall MOWL of Orleans Avenue Canal East Bank
MOWL (ft)
STATIONS
REACH
DPS
STA. 2+45
SWE =8 FT
SHEET PILE PENETRATION < 10 FEET
FILMORE AVE. BRIDGE
STA. 64+00 TO 65+00
ICS
STA. 113+05
HARRISON AVE. BRIDGE
STA. 36+40TO STA. 37+44
ROBERT E. LEE BRIDGE
STA. 90+62 TO STA. 92+20
Min Penetration < 10 Ft
HARRISON AVE.
FILMORE AVE.
ROBERT E. LEE
12A 12B 13A 13B 14 15 16 17 18A 18B 19 20A 20B 21
2+45 3+70 4+70 7+00 11+20 20+50 30+00 36+40 37+44 50+00 61+00 64+00 65+00 90+62 92+20 93+46 101+50 113+05
CAUTION: Analysis for this report was completed prior to the issuance of Engineer Technical Letter (ETL) 1110-2-575, EVALUATION OF I-WALLS, dated 1 September 2011.

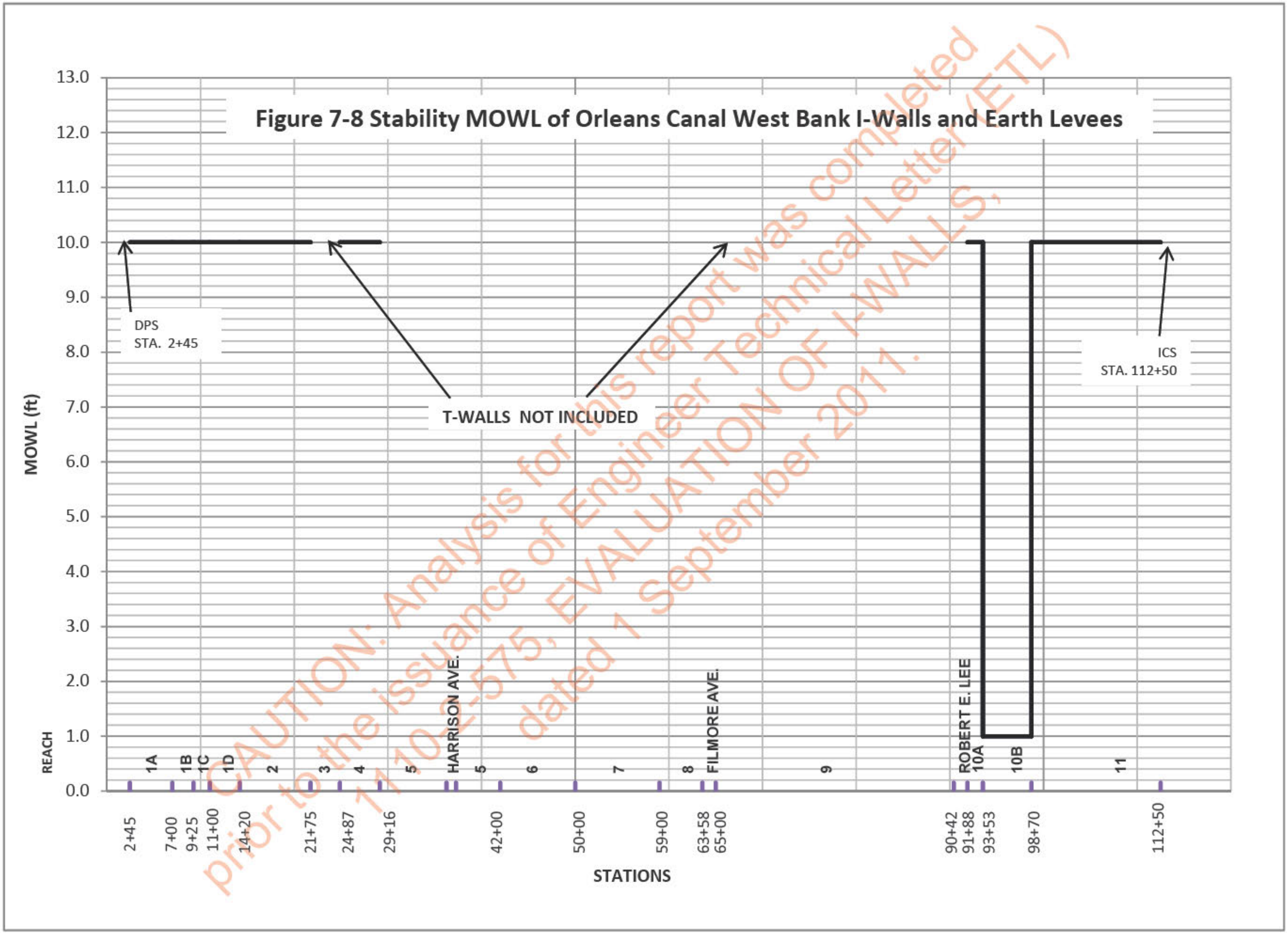

Figure 7-8 Stability MOWL of Orleans Canal West Bank I-Walls and Earth Levees
MOWL (ft)
13.0
12.0
11.0
10.0
9.0
8.0
7.0
6.0
5.0
4.0
3.0
2.0
1.0
0.0
DPS
STA. 2+45
T-WALLS NOT INCLUDED
ICS
STA. 112+50
REACH
1A
1B
1C
1D
2
3
4
5
HARRISON AVE.
5
6
7
8
FILMORE AVE.
9
ROBERT E. LEE
10A
10B
11
2+45
7+00
9+25
11+00
14+20
21+75
24+87
29+16
42+00
50+00
59+00
63+58
65+00
90+42
91+88
93+53
98+70
112+50
STATIONS

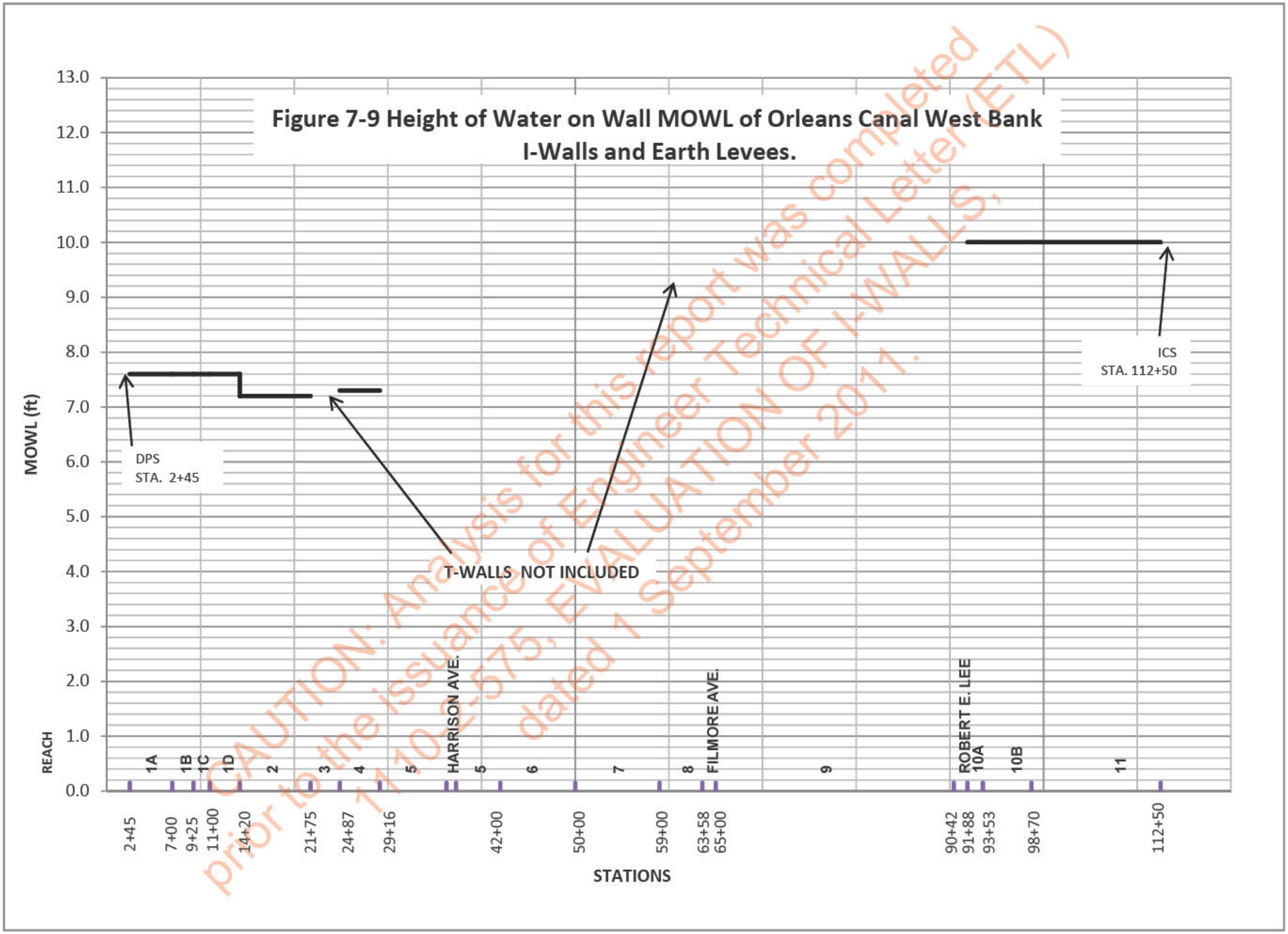


**Figure 7-9 Height of Water on Wall MOWL of Orleans Canal West Bank I-Walls and Earth Levees.**

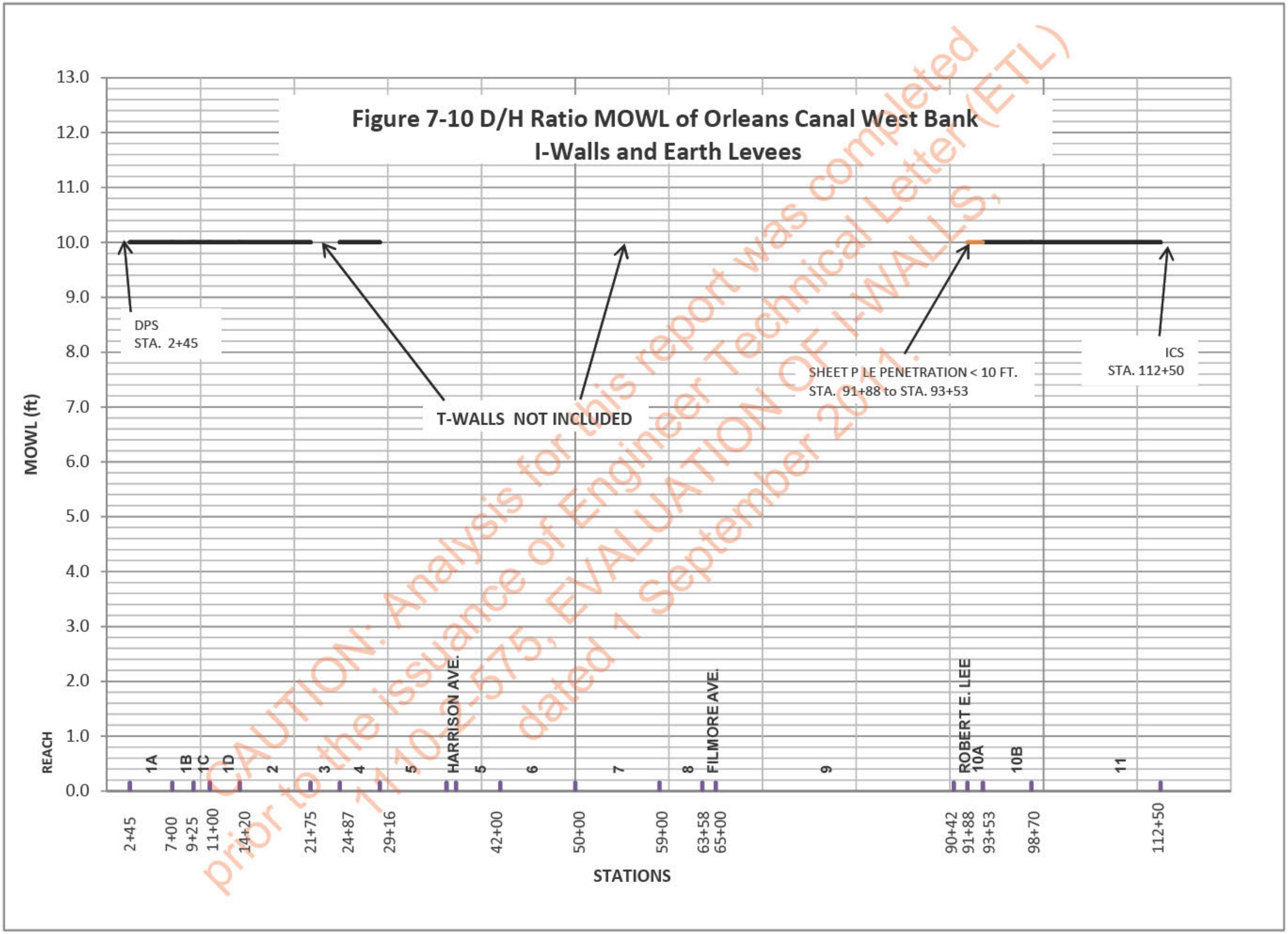
Figure 7-10 D/H Ratio MOWL of Orleans Canal West Bank
I-Walls and Earth Levees
MOWL (ft)
DPS
STA. 2+45
T-WALLS NOT INCLUDED
SHEET P LE PENETRATION < 10 FT.
STA. 91+88 to STA. 93+53
ICS
STA. 112+50
REACH
1A
1B
1C
1D
2
3
4
5
HARRISON AVE.
5
6
7
8
FILMORE AVE.
9
ROBERT E. LEE
10A
10B
11
2+45
7+00
9+25
11+00
14+20
21+75
24+87
29+16
42+00
50+00
59+00
63+58
65+00
90+42
91+88
93+53
98+70
112+50
STATIONS
CAUTION: Analysis for this report was completed prior to the issuance of Engineer Technical Letter (ETL) 1110-2-575, EVALUATION OF I-WALLS, dated 1 September 2011.

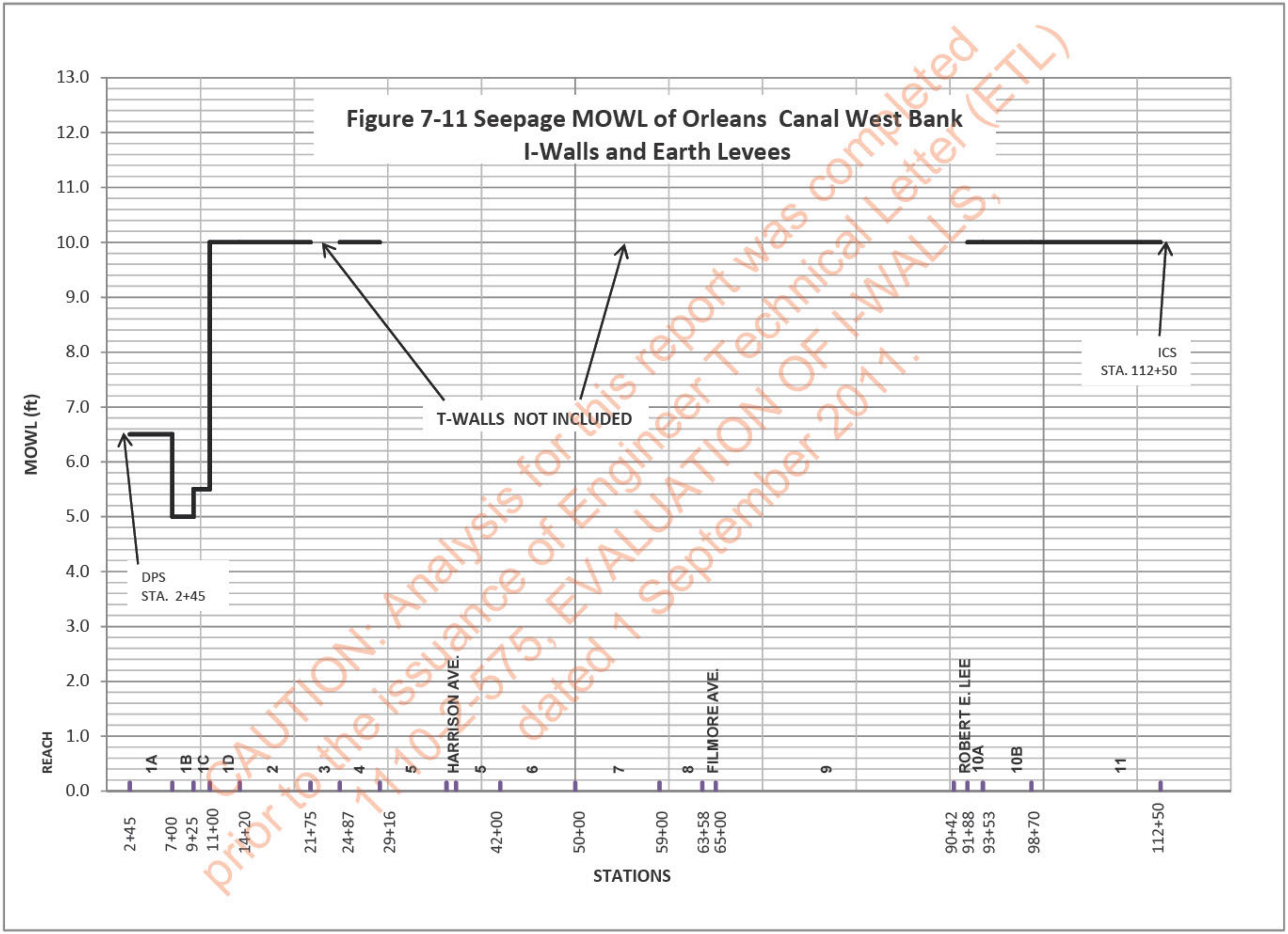
Figure 7-11 Seepage MOWL of Orleans Canal West Bank
I-Walls and Earth Levees
MOWL (ft)
13.0
12.0
11.0
10.0
9.0
8.0
7.0
6.0
5.0
4.0
3.0
2.0
1.0
0.0
REACH
T-WALLS NOT INCLUDED
ICS
STA. 112+50
DPS
STA. 2+45
1A
1B
1C
1D
2
3
4
5
HARRISON AVE.
5
6
7
8
FILMORE AVE.
9
ROBERT E. LEE
10A
10B
11
2+45
7+00
9+25
11+00
14+20
21+75
24+87
29+16
42+00
50+00
59+00
63+58
65+00
90+42
91+88
93+53
98+70
112+50
STATIONS

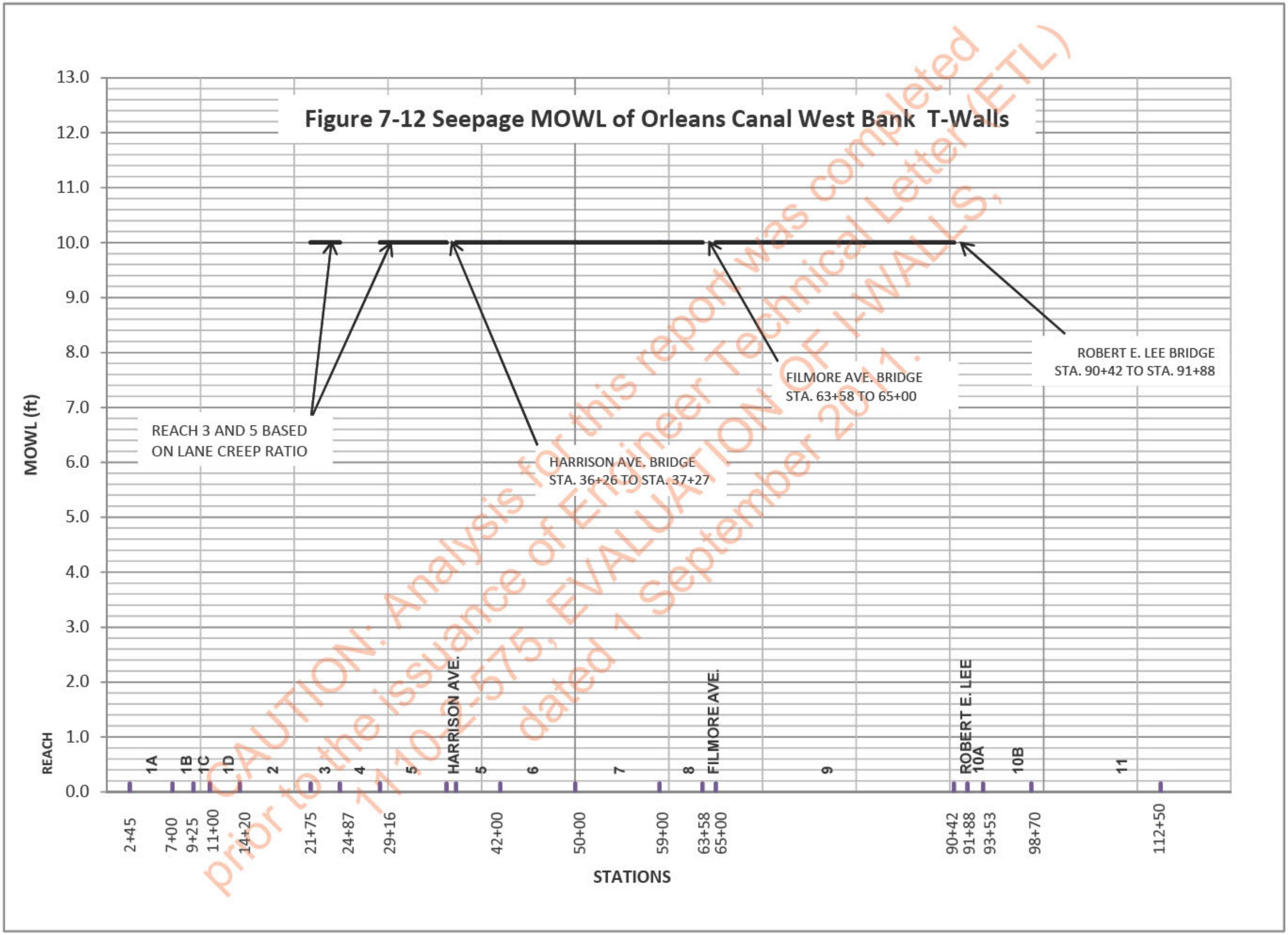
Figure 7-12 Seepage MOWL of Orleans Canal West Bank T-Walls
MOWL (ft)
13.0
12.0
11.0
10.0
9.0
8.0
7.0
6.0
5.0
4.0
3.0
2.0
1.0
0.0
REACH 3 AND 5 BASED ON LANE CREEP RATIO
HARRISON AVE. BRIDGE STA. 36+26 TO STA. 37+27
FILMORE AVE. BRIDGE STA. 63+58 TO 65+00
ROBERT E. LEE BRIDGE STA. 90+42 TO STA. 91+88
REACH
1A
1B
1C
1D
2
3
4
5
HARRISON AVE.
5
6
7
8
FILMORE AVE.
9
ROBERT E. LEE
10A
10B
11
2+45
7+00
9+25
11+00
14+20
21+75
24+87
29+16
42+00
50+00
59+00
63+58
65+00
90+42
91+88
93+53
98+70
112+50
STATIONS

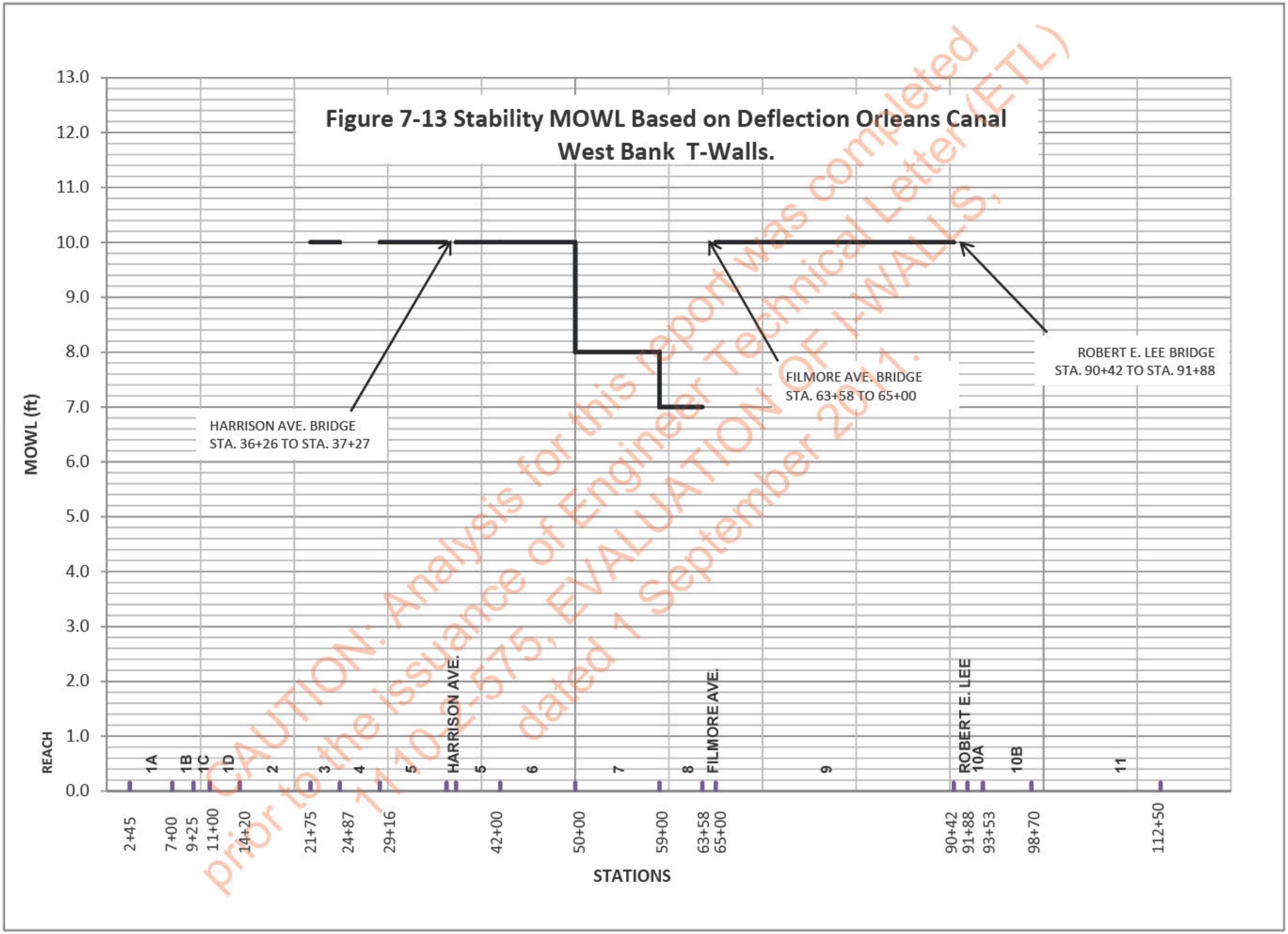
Figure 7-13 Stability MOWL Based on Deflection Orleans Canal West Bank T-Walls.
MOWL (ft)
STATIONS
REACH
HARRISON AVE. BRIDGE STA. 36+26 TO STA. 37+27
FILMORE AVE. BRIDGE STA. 63+58 TO 65+00
ROBERT E. LEE BRIDGE STA. 90+42 TO STA. 91+88
HARRISON AVE.
FILMORE AVE.
ROBERT E. LEE
1A
1B
1C
1D
2
3
4
5
5
6
7
8
9
10A
10B
11
2+45
7+00
9+25
11+00
14+20
21+75
24+87
29+16
42+00
50+00
59+00
63+58
65+00
90+42
91+88
93+53
98+70
112+50
0.0
1.0
2.0
3.0
4.0
5.0
6.0
7.0
8.0
9.0
10.0
11.0
12.0
13.0

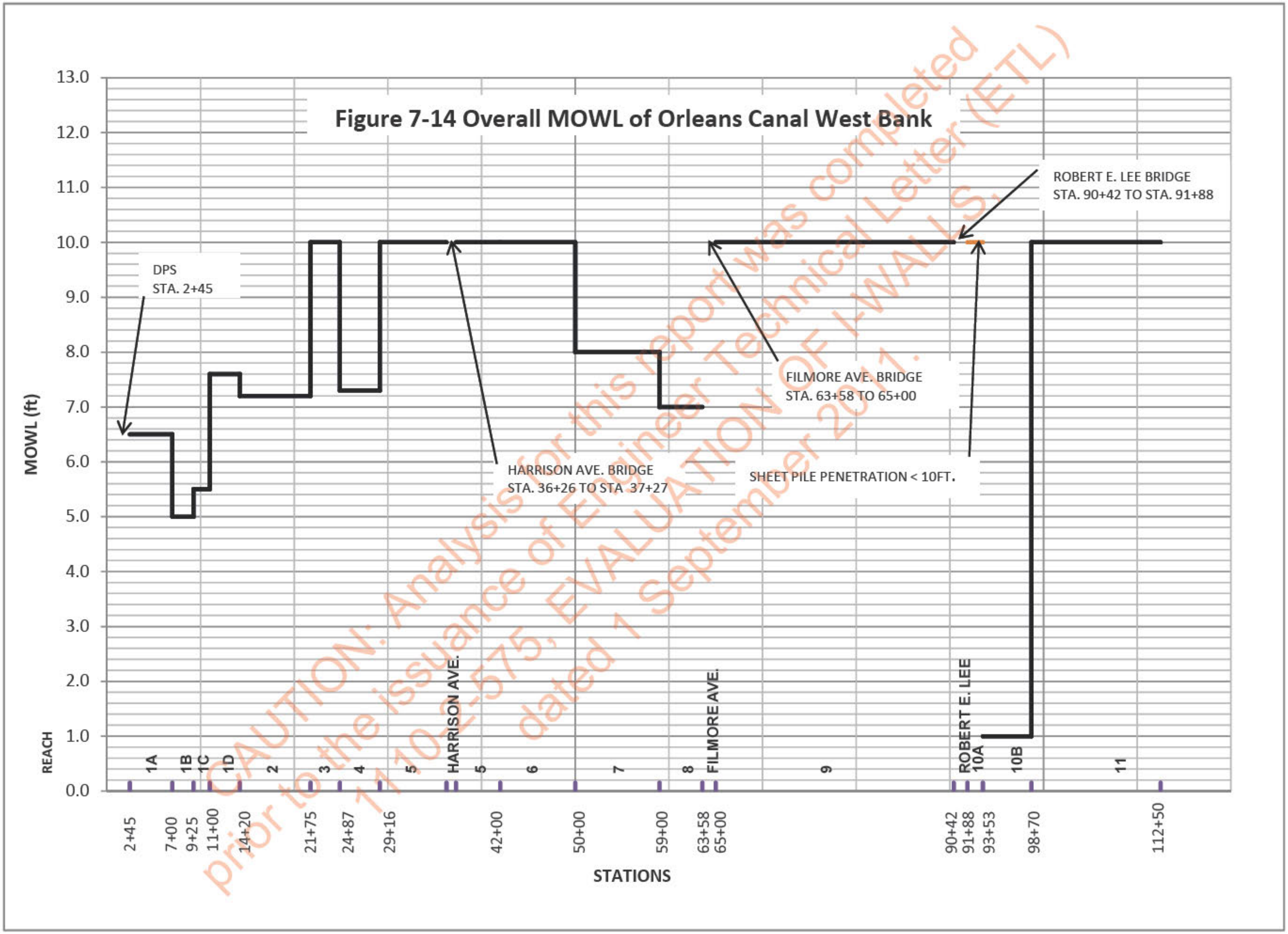

Figure 7-14 Overall MOWL of Orleans Canal West Bank
MOWL (ft)
13.0
12.0
11.0
10.0
9.0
8.0
7.0
6.0
5.0
4.0
3.0
2.0
1.0
0.0
REACH
DPS
STA. 2+45
HARRISON AVE. BRIDGE
STA. 36+26 TO STA 37+27
FILMORE AVE. BRIDGE
STA. 63+58 TO 65+00
ROBERT E. LEE BRIDGE
STA. 90+42 TO STA. 91+88
SHEET PILE PENETRATION < 10FT.
1A
1B
1C
1D
2
3
4
5
HARRISON AVE.
5
6
7
8
FILMORE AVE.
9
ROBERT E. LEE
10A
10B
11
2+45
7+00
9+25
11+00
14+20
21+75
24+87
29+16
42+00
50+00
59+00
63+58
65+00
90+42
91+88
93+53
98+70
112+50
STATIONS

# 8.0 IMPACT TO CURRENT OPERATIONS

The analyses confirm that most problems along the Orleans Avenue Canal are related to stability. Based on the analyses tabulated above, some critical reaches along the canal need improvements to achieve the requisite stability under the normal Lake level. Other reaches need improvements to sustain the selected operational MOWL of El 8 NAVD88. Likewise, a few reaches fail to meet the stringent requirements demanded by the new criteria and methods of analysis for the current MOWL of El 8 NADV 88. For this reason, the Corps will move expeditiously and prioritize the implementation of the rehabilitation design and construction to ensure that all requirements are met.

Several factors temper the results of the analyses developed in this study and the prioritization of required improvements to the I-wall parallel protection system.

- First, all I-walls and T-walls experienced significantly higher hydraulic loading during Katrina than the current MOWL; during Katrina the canal water level was reported to reach approximately El 11 NAVD88. The I-walls and T-walls did not exhibit signs of distress under those high water loads. They also have not shown any distress under the water loads resulting from the current operating protocol under which the canal has been operated since Katrina. Also, since Katrina, the outfall canal experienced two significant tropical events, Hurricanes Gustav and Ike, where the water levels in the canal were at or above El 4 NAVD88 and an extra-tropical event where the water level reached slightly above El 4.95 NADV88.

- Second, the stability of the I-walls was based on very conservative estimates of undrained shear strength of the soils on the protected side of the levees as indicated by the fact the MOWLs developed in this study were in many cases below the water levels experienced during Katrina. In some cases the MOWLs were El 1NAVD88. These levels are for a FOS of 1.4 and do not indicate failure, but the I-walls experienced no distress at much higher water levels, and these water levels would have indicated failure according to these analyses.

- Third, the seepage stability of the I-walls is a function of the connectivity of the water in the canal to the barrier beach sands. The lack of seepage problems from Katrina due to the other hurricane and extra-tropical events support a position that there are semi-impervious canal sediments and marsh clays overlying the beach sand stratum at the bottom of much of the canal that affords dissipation of the canal hydraulic head and which improves safety. The analyses are based on the most conservative assumption regarding the continuity of these sediments, i.e., if the blanket is less than 2.0 ft thick, the blanket is assumed not to be present.

- Fourth, the seepage analysis was based on a conservative methodology, developed by GCAT, to estimate the gap formation between the I-wall and the soil on the flood side of the canal when the canal water level exceeds the crest of the levee embankment. This methodology is based on the analyses and evaluations performed after Katrina by IPET, and it is consistent with the centrifuge testing at ERDC. However, it is deemed to be conservative because it assumes that the gap will form, to the maximum depth possible, at very modest canal water levels. The methodology in its current version does not consider the stiffness afforded by the soil on the protected side of the wall or the stiffness of the wall itself. Therefore the gradual progression of the gap with increasing water level is not modeled. The methodology has not been peer reviewed yet and some enhancements may emerge from this process, once completed.

- Fifth, the I-walls are being analyzed based on the most stringent HSDRRSDG criteria for all design aspects. These criteria require higher FOS than the criteria that are normally used for interior protection features. The I-walls were part of the perimeter system but with the change to add a permanent closure structure at the mouth of the outfall canal, the I-walls are now an interior feature. Interior features are designed with less stringent criteria. This adds to the conservatism used in analyzing the I-walls and in designing I-wall improvements.

These factors point to the conservatism inherent in the selected analysis methodologies, especially at low canal water elevations. Since the construction of the canal and up to the time of Katrina, the canal was open to the Lake. As such, it was exposed to uncontrolled water level fluctuations as a function of surges from the Lake. During this loading history, the I-walls did not experience any observable damage or permanent deformation that may have raised concerns regarding the stability of the walls. Katrina demonstrated that in the other two outfall canals the I-walls were not as reliable during high canal water levels. To permanently address this situation, one of the many steps taken by the Corps has been to close the outfall canals to the Lake during tropical and extra-tropical events. The long term solution will be to build permanent closure structures and pump stations at the mouth of the outfall canals thereby preventing storm surge from entering the canals. This Corps decision significantly reduces the potential risk of the I-walls malfunctioning or failing during loading and the consequences thereof. Currently, water level in the canal is controlled through the use of an interim gated closure structure and a temporary pump station at the mouth of the canal which pumps runoff concurrently with the interior permanent pump stations. Under this condition, the consequences of failure would be limited.

The above rationale is not totally true for the higher water levels necessary to operate the canal in an efficient and safe manner for the selected operational plans for the system. Although the consequence effects would be similar, the probability of failure of the I-walls goes up with increasing water levels and the amount of water released would be higher producing more damages. For this reason the parallel protection system must be improved, expeditiously, to the selected MOWL of El 8 NAVD88. This MOWL is also necessary for the future development plans of the City of New Orleans, as the city-owned pump stations are improved in the future to be capable of pumping water in the canal up to the proposed MOWL of El 8 NAVD88.

In summary, the Corps remains confident in the continued operation of the canal following the current water management protocols that prevents encroaching on the MOWL of El 8 NADV88. At the same time, the Corps recognizes that several reaches of the I-walls must be improved and is committed to move expeditiously to implement the required improvements based on the most stringent criteria and following rigorous methods of analysis. In the next phase, the Corps will pursue further analyses to ensure that the solution selected for the improved parallel protection system fully meet all necessary requirements.

# *9.0 REFERENCES*

## 9.1 NEW ORLEANS DISTRICT CRITERIA


**1.** Interagency Performance Evaluation Task Force (IPET), *Performance Evaluation of the New Orleans and Southeast Louisiana Hurricane Protection System*, US Army Corps of Engineers, June 1, 2007

**2.** Special Issue on the Performance of Geo-Systems During Hurricane Katrina, ASCE Journal of Geotechnical and Geoenvironmental Engineering, Vol. 134, No. 5, May 2008

**3.** Independent Levee Investigation Team (ILIT), Investigation of the Performance of the New Orleans Flood Protection Systems in Hurricane Katrina on August 29, 2005, Final Report, National Science Foundation, July 31, 2006

**4.** Hurricane and Storm Damage Risk Reduction System Design Guidelines-(HSDRRSDG), New Orleans District Engineering Division, US Army Corps of Engineers, October 2007 with June 12, 2008 revisions

**5.** Spencer, E., "A Method of Analysis of the Stability of Embankments Assuming Parallel Inter-slice Forces" Gèotechnique, 17:11-16, 1967

**6** Lake Pontchartrain, Louisiana and Vicinity, High Level Plan, Design Memorandum No. 19 - General Design, Orleans Avenue Outfall Canal, Volumes 1, 2 and 3, US Army Corps of Engineers, New Orleans District, New Orleans, Louisiana, August 1988

**7.** Geotechnical Criteria Applications Team (GCAT), Stability Analysis of I-Walls Containing Gaps between the I-Wall and Backfill Soils, Version 6, Hurricane Protection Office, US Army Corps of Engineers, 14 September 2009

**8.** National Climatic Data Center for the New Orleans International Airport, Station 166660, January 29, 2008 through October 12, 2009

**9.** Hurricane Protection Office, US Army Corps of Engineers, and St. Louis District Corps of Engineers London Avenue Site Specific Load Test, Final Version, August 2008

**10.** Numerous investigations and laboratory testing programs performed along 17th Street Canal, New Orleans District, US Army Corps of Engineers, 1970-2010. All available borings and test data are provided in electronic format in Appendix E.

**11.** "As-built" drawing of construction along the levees of the Orleans Avenue Canal, New Orleans District, US Army Corps of Engineers, 1970-2010. All available as-built drawings are provided in electronic format in Appendix E.

**12.** Survey Report for the Remediation to Raise the Safe Water Elevation for the Orleans Avenue Canal, Chustz Surveying Inc., May 2010

**13.** 17th Street Canal MOWL Report, MVN, New Orleans, LA, 2011

**14.** Geomorphology and Quaternary Geologic History of the Lower Mississippi Valley, Roger T. Saucier, U.S. Army Engineering Waterways Experiment Station, December 1994

**15.** 1878 Hardee Map of New Orleans, The Historical New Orleans Collection, New Orleans, LA

**16.** Ten Calls To Action, A Report of the ASCE Hurricane Katrina External Review Panel, August 2006, Reston, VA

**17.** The New Orleans Hurricane Protection System - What Went Wrong and Why, A Report of the ASCE Hurricane Katrina External Review Panel, August 2006, Reston, VA

**18.** London Avenue Canal Load Test – Small Load Test Site Selection, Noah Vroman, ERDC Corps, Tom Brandon, Virginia Tech, Neal Schwanz, MVP Corps, 2006

19. London Avenue Canal MOWL Report, MVN, New Orleans, LA, 2011

20. London Avenue Canal Pump Test, MVN Corps, New Orleans, LA, 2006

21. Orleans Avenue Canal Pump Test, MVN Corps, New Orleans, LA, 2006

22. An Examination of the Method of Planes for Slope Stability Analyses, Report by Steven G. Wright, Austin, TX, April 1, 2006

23. Recent Deltaic Deposits of the Mississippi River: Their Development and Chronology, Frazier, D. E., Transactions of the Gulf Coast Association of Geological Societies, Vol. 17, 1967, p. 287-315

24. Memorandum - London Avenue Canal – Recommended Seepage Analysis Permeabilities, Noah Vroman, ERDC Corps, July 19, 2009

25. 17th Street Canal Pump Test, MVN Corps, New Orleans, LA, 2006

26. Classification of Soils for Engineering Purposes, ASTM D2487, ASTM, West Conshohocken, PA, 2010

27. Finite Element Seepage Analysis of London Avenue Canal Load Test, Abeera Batool and Thomas L. Brandon, Dept. of Civil and Environmental Engineering, Virginia Tech, Blacksburg, VA November 7, 2010

28. Security from Under-seepage – Masonry Dams on Earth Foundations, Lane, E. W., Transactions ASCE 100, pages 1235-1351

## 9.2 US ARMY CORPS OF ENGINEERS DESIGN CRITERIA

1. EM 1110-2-1913 Design and Construction of Levees, April 30, 2000

2. EM 1110-2-2104 Strength Design for Reinforced Concrete Hydraulic Structures, August 2003

3. EM 1110-2-2906 Design of Pile Foundations, January 1991

4. DIVR 1110-1-400 Landside Seepage Berms for Mississippi River Levees, December, 1998

5. TM 3-424 Investigation of Underseepage and Its Control, Lower Mississippi River, October, 1956

## 9.3 SOFTWARE

1. Design and Analysis of Sheet-Pile Walls by Classical Methods (CWALSHT), X0031, US Army Corps of Engineers Waterways Experiment Station Information Technology Laboratory (USAEWES), Vicksburg, Mississippi, 2001

2. GeoStudio 2007 (includes packages SLOPE/W and SEEP/W), Version 7.16, GEO-SLOPE International, Calgary, Canada, 2010

3. Stability With Uplift (also known as the Method of Planes), FS004, New Orleans District, New Orleans, Louisiana, 1982

4. Group for Windows (formally Group 7), version 7.0.22, ENSOFT, INC, Austin, Texas, 2006

5. ACI 530 Building Code Requirements for Masonry Structures. (TMS 402-08/ACI 530-08/ASCE 5-08), 2008

## 9.4 OTHER REFERENCES NOT CITED IN TEXT

1 EM 1110-2-1901 Seepage Analysis and Control for Dams, September 30, 1986 (Original), April 30, 1993 (Change 1)

2. EM 1110-2-1914 Design, Construction and Maintenance of Relief Wells, May 29, 1992

3. EM 1110-2-2502 Retaining and Flood Walls, September 29, 1989

4. EM 1110-2-2504 Design of Sheet Pile Walls, March 31, 1994

5. ETL 1110-2-569 Design Guidance for Levee Underseepage, May 01, 2005

**6.** Microsoft® Office Excel 2003, Part of Microsoft Office Professional Edition 2003, Microsoft Corporation, 2003

Only software products used directly in analyses are listed above. Numerous other software products supporting office and production functions have been used in various stages of producing this report but are not listed here.