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
17TH STREET CANAL
ORLEANS PARISH, LOUISIANA

MOWL for 17th Street Canal

Prepared for:

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

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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 third outfall canal, the Orleans Avenue 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 17th Street Canal, the MOWL was established at El 6 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 17th Street 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 6 (DPS 6) 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 17th Street Canal parallel protection system consists of earthen levees with floodwalls to provide additional protection. Floodwalls consist of I-walls along the reaches of the canal defined in Table 1-1. One of the I-walls failed during Hurricane Katrina and was replaced with a T-wall. The location of the replacement wall is identified in Table 1-1.

The MOWL for each I-wall 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 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 of analysis for Reaches 3B, 6 and 29 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 even without the influence of the canal water which is located outside of the active zone below the crest. These low FOS values were the result of the low shear strengths of the embankment and foundation soils. In Reaches 22, 23, 25, 26 and 27, the MOP stability analysis controlled the MOWL.

In Reach 1 and in Reaches 9 through 15A the penetration of the sheet pile does not meet the minimum requirement of 10 feet. The depth to height ratio on the I-wall will limit the MOWL to below El 10 NAVD88 for Reaches 1 through 15A and 22 through 28. The MOWL for this criterion is above El 8 NAVD88 with the exception of Reach 9. 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 2 thru 12 and 21 thru 28.

Seepage was not found to be a concern on any reach of the 17th Street Canal for a MOWL below El 10 NAVD88. Reaches 15A, 15B, 16, 31, 32, and 33 were run with an open bottom; however, the SWEs were greater than +8.0 for these reaches. The FOS for a canal water elevation of +8 feet of 1.87 was calculated by HPO for Reach 16. (See Appendix D.8).

TABLE 1-1

LEVEE REACH LOCATIONS

WEST REACH	WALL TYPE	WEST BASELINE APPROXIMATE STATION	EAST REACH	WALL TYPE	EAST BASELINE APPROXIMATE STATION
1	I-wall	ICS to 552+22	19	I-wall	ICS to 552+17
Hammond Hwy. Bridge		552+22 to 553+70	**Hammond Hwy. Bridge**		552+17 to 553+58
2	I-wall	553+70 to 565+00	20	I-wall	553+58 to 560+10
3A	I-wall	565+00 to 570+00		T-wall	560+10 to 566+00
3B	I-wall	570+00 to 571+45	21	I-wall	566+00 to 570+73
4	I-wall	571+45 to 575+45	22	I-wall	570+73 to 581+50
5	I-wall	575+45 to 578+22	23	I-wall	581+50 to 588+67
6	I-wall	578+22 to 582+60	24	I-wall	588+67 to 598+24
7	I-wall	582+60 to 585+55	25	I-wall	598+24 to 608+00
8	I-wall	585+55 to 588+70	26	I-wall	608+00 to 612+92
9	I-wall	588+70 to 593+00	27	I-wall	612+92 to 615+03
10	I-wall	593+00 to 596+05	28	I-wall	615+03 to 620+30
11	I-wall	596+05 to 609+00	29	I-wall	620+30 to 624+88
12	I-wall	609+00 to 614+00	**Veterans Blvd. Bridge**		624+88 to 626+73
13	I-wall	614+00 to 617+00	30	I-wall	626+73 to 634+09
14	I-wall	617+00 to 625+00	31	I-wall	634+09 to 637+00
Veterans Blvd. Bridge		625+00 to 626+56	32	I-wall	637+00 to 638+44
15A	I-wall	626+56 to 635+00	**I-10 Bridge**		638+44 to 643+40
15B	I-wall	635+00 to 639+06	33	I-wall	643+40 to 658+00
I-10 Bridge		639+06 to 641+85	34	I-wall	658+00 to 662+87
16	I-wall	641+85 to 658+00	35	I-wall	662+87 to 670+63
17	I-wall	658+00 to 663+00			
18	I-wall	663+00 to 669+36			

The MOWL for the replacement T-wall was found to be El 10 NAVD88. Leakage through the replacement T-wall has occurred and has resulted in an elevated piezometric level in the granular fill on the protected side of the T-wall. This has caused a localized condition of excessive pressures below the thin clay fill blanket that was placed over the granular fill and some localized seeps through the clay blanket. This is not a wall stability issue. Recommendations from the Bachus and Martin [9] report identify potential methods to remediate the leakage

Table 1-3 provides a summary of the factors of safety and deflections for the T-wall and DPS 6. Figures 7-1 through 7-4 in the body of the text provide the calculated MOWLs for each criterion along east bank of the canal. Similarly, Figures 7-5 through 7-9 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

WEST 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[2]	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
1	Closure to 552+21.5	10.0	10.0	**NO**	**9.3**	10	10	10.0
2	553+70 to 565+00	**4.5**	6.5	YES	8.0	8.2	10	10.0
3A	565+00 to 570+00	8.0	**7.5**	YES	8.1	8.5	10	10.0
3B	570+00 to 571+45	**<1.0**[2]	1.0	YES	8.1	8.5	10	10.0
4	571+45 to 575+45	5.5	**4.5**	YES	8.2	8.5	10	10.0
5	575+45 to 578+22.5	**4.0**	**4.0**	YES	8.2	8.5	10	10.0
6	578+22.5 to 582+60	**<1.0**[2]	1.0	YES	8.2	8.5	10	10.0
7	582+60 to 585+55	8.5	8.5	YES	**8.1**	8.5	10	10.0
8	585+55 to 588+70	**6.5**	**6.5**	YES	8.2	8.5	10	10.0
9	588+70 to 593+00	**4.0**	**4.0**	**NO**	7.6	9.0	10	10.0
10	593+00 to 596+05	9.0	9.0	**NO**	**8.3**	9.5	10	10.0
11	596+05 to 609+00	9.0	9.0	**NO**	**8.3**	9.5	10	10.0
12	609+00 to 614+00	10.0	10.0	**NO**	**8.3**	9.5	10	10.0
13	614+00 to 617+00	10.0	10.0	**NO**	**9.1**	10.0	10	10.0
14	617+00 to 625+00	10.0	10.0	**NO**	**8.9**	10.0	10	10.0
15A	626+56 to 635+00	10.0	10.0	**NO**	**9.2**	10.0	10	10.0[4]
15B	635+00 to 639+06	10.0	10.0	YES	10.0	10.0	10	10.0[4]
16[3]	641+85 to 658+00	10.0	10.0	YES	10.0	10.0	10	8.0[4]
17	658+00 to 663+00	10.0	10.0	YES	10.0	10.0	10	10.0
18	663+00 to 669+36	10.0	10.0	YES	10.0	10.0	10	10.0

EAST 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/H1 = 3/1 MOWL NAVD88	MAXIMUM 4 FT WATER DEPTH ON I-WALL MOWL NAVD88	CWALSHT MOWL NAVD88	SEEPAGE MOWL NAVD88
19	Closure to 552+17	10.0	10.0	YES	10.0	10.0	10	10.0
20	553+58 to 560+10	**7.5**	**7.5**	YES	10.0	8.5	10	10.0
21	566+00 to 570+73	**4.5**	**4.5**	YES	10.0	8.0	10	10.0
22	570+73 to 581+50	4.5	**4.0**	YES	9.5	7.5	10	10.0
23	581+50 to 588+67	**5.5**	**5.5**	YES	9.4	7.2	10	10.0
24	588+67 to 598+24	**5.0**	**5.0**	YES	9.3	7.3	10	10.0
25	598+24 to 608+00	6.0	**5.0**	YES	9.6	7.5	10	10.0
26	608+00 to 612+92	**7.0**	**7.0**	YES	9.3	7.5	10	10.0
27	612+92 to 615+03	**7.5**	**7.5**	YES	9.9	7.8	10	10.0
28	615+03 to 620+30	**7.5**	**7.5**	YES	9.7	8.3	10	10.0
29	620+30 to 624+88	**<1.0**[2]	<1.0[2]8.0	YES	10.0	10.0	10	10.0
30	626+73 to 634+09	**8.5**	**8.0**	YES	10.0	10.0	10	10.0
31	634+09 to 637+00	**10.0**	**10.0**	YES	10.0	10.0	10	10.0[4]
32	637+00 to 638+44	**10.0**	**10.0**	YES	10.0	10.0	10	10.0[4]
33	643+40 to 658+00	**10.0**	**10.0**	YES	10.0	10.0	10	10.0[4]
34	658+00 to 662+87	**10.0**	**10.0**	YES	10.0	10.0	10	10.0
35	662+87 to 670+63	**10.0**	**10.0**	YES	10.0	10.0	10	10.0

Notes:

[1] H_1 = Height of water above the crest of the protected side earth embankment.

[2] FOS less than 1.4 for both Spencer's analyses at canal water levels equal El 1NAVD88

[3] The crest of flood side embankment is above El 10 NAVD88.

[4] Reach analyzed with an open connection between the canal and the beach sand

TABLE 1-3
REACH MOWL VALUES FOR T-WALLS AND DPS6

WALL TYPE	CANAL SIDE	STATION	MOWL NAVD88	SPENCER'S METHOD FOS	MOP FOS	DEFLECTION (IN)	SPENCER'S METHOD GAP FOS	MOP GAP FOS
T-Wall	East	560+10 to 566+00	10	1.76	1.96	<0.1	--	--
DPS 6	--	--	10	1.46	1.55	--	--	--
Training wall and embankment	West	--	10	3.24	3.20	--	3.07	2.94
Embankment and sheet pile wall	East	--	10	1.93	1.87	--	NA	NA

The analyses in this report indicate that some reaches along the 17th Street Canal have MOWL values lower than the present MOWL of El 6 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.

- **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.

FIGURE 2-1
LOCATION OF OUTFALL CANALS [1]

- **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 17th Street 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 17th Street 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 17TH STREET OUTFALL CANAL FAILURES

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 necessary to fully understand the failure mechanisms and address professional differences of opinion related to the 17th Street Canal I-wall failure.

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
- 17th Street Canal: El 6 NAVD88

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 ICSs for the 17th Street Canal was completed on June 1, 2009.

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 17th Street 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. 20 - General Design, 17th Street Outfall Canal (Metairie Relief)*, Volumes 1 and 2 [6] includes investigations performed through 1986;
- *Lake Pontchartrain, Louisiana and Vicinity, High Level Plan, Design Memorandum No. 20 - General Design, 17th Street Outfall Canal (Metairie Relief) General Design Supplement No. 1* [8]
- IPET Report, Volume 5 [1] includes data developed in vicinity of failure areas; and
- Additional investigations [10] performed by the Corps in 2005 through 2010 as described herein.

4.2 SURVEYS

Surveys of the canal were performed during June 2009 [12, 13]. These consisted of bathymetric and topographic surveys on the east and west sides of the canal from DPS 3 at the south end of the canal to the ICS at the north end of the canal.

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 17th Street Canal have MOWL values lower than the present MOWL of El 6 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.

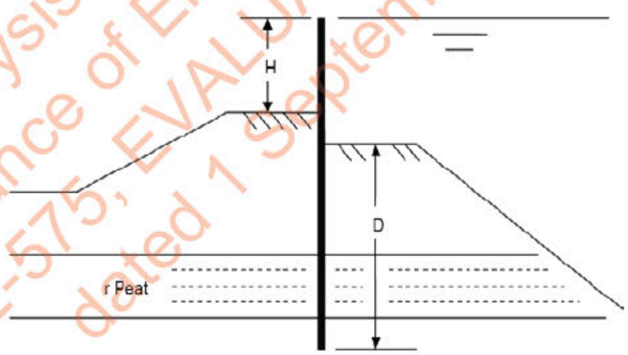

FIGURE 4-1
SHEET PILE PENETRATION CRITERIA DEFINITIONS

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 (H1), not the height to the top of the wall (H). The elevation where the canal water depth (H1) = 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 constructed after Katrina to replace failed I-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 17th Street 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 17th Street 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 17th Street 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 17th Street Canal, Holocene Lacustrine clays were deposited in a fresh water environment.

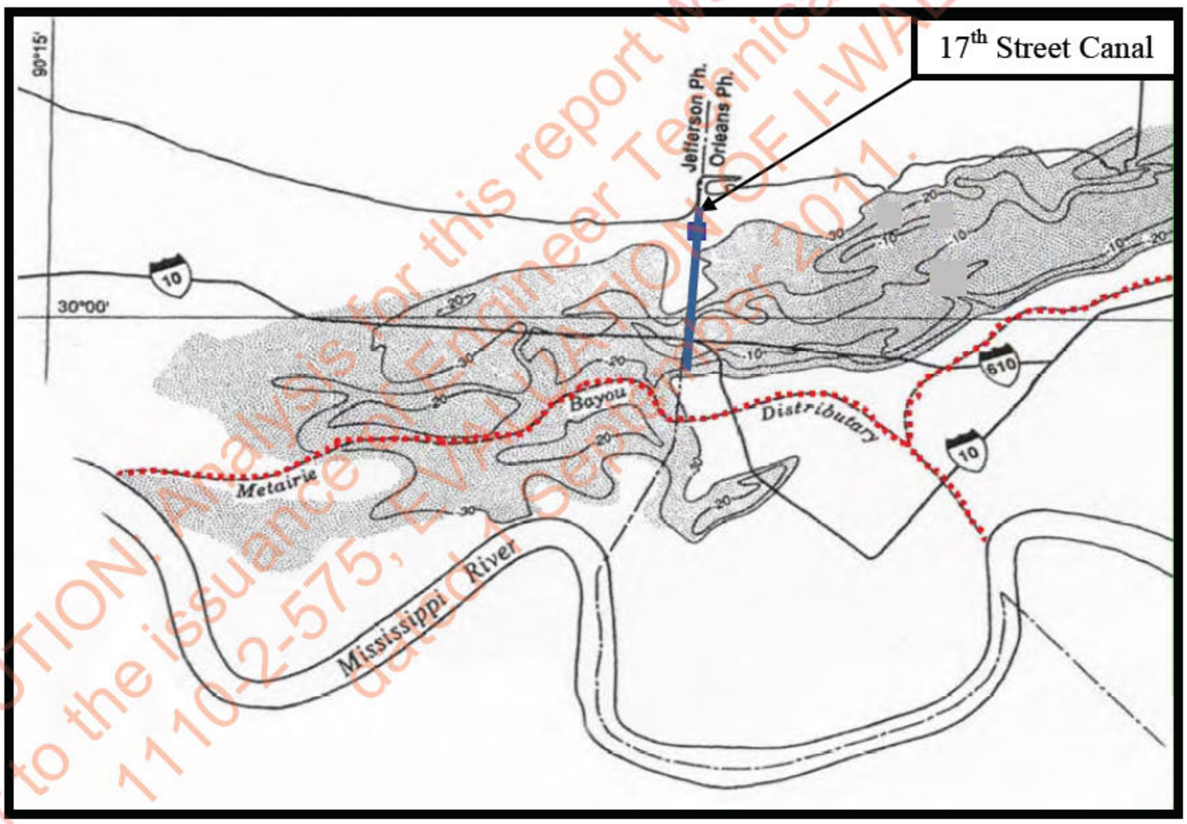

FIGURE 5-1
PINE ISLAND BARRIER BEACH AND BAYOU SAUVAGE (METAIRIE) DISTRIBUTARY CHANNEL WITH 17TH STREET CANAL FAILURE AREA SHOWN AS A BLUE BOX [1]

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.

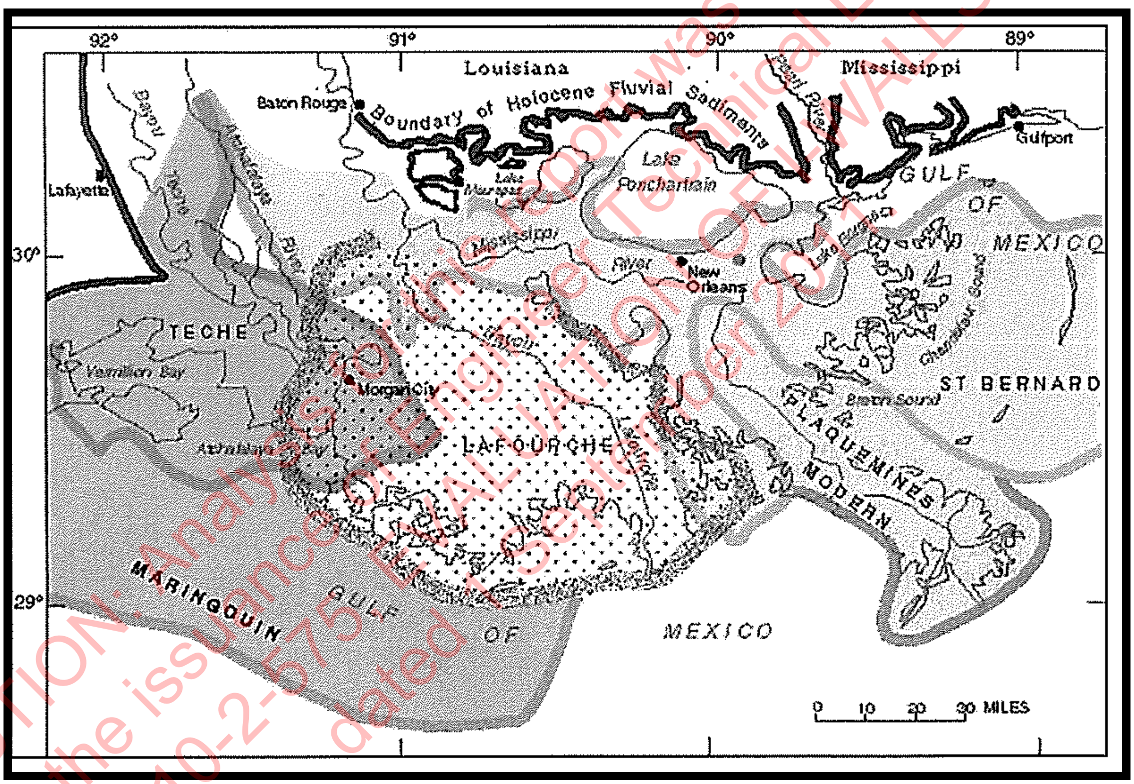

FIGURE 5-2
HOLOCENE DELTAS OF THE MISSISSIPPI RIVER (14)

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 10 through 54, illustrate the formations described above

6.0 GEOTECHNICAL CONSIDERATIONS

The geotechnical data used in this study were obtained from Design Memorandum No. 20 [6] (DM 20), Design Memorandum No. 20, General Design Supplement No.1 [8] (DM 20, Supplement 1) the IPET Report [1], and through additional investigations and laboratory testing performed during the period 2005 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 Next, a summary of the 17th Street Canal Full Scale Seepage Test is presented. 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 the various types of floodwalls, the tip elevations of the underlying sheet pile cutoff walls, a pump station and bridges. 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 Floodwalls

The existing I-walls along the levee crests were constructed in the early 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. After the I-wall failure occurred during Katrina, the failed I-wall section was replaced with a T-wall. The new pile supported T-wall was installed between Stations 560+00 and 566+00 for a total length of 600 feet. The top of the I-wall grades vary between El 12.1 and 13.5 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.

6.1.2 Sheet Pile Tip Elevations

The I-walls and T-wall 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 T-wall added after Katrina is not included in Table 6-1. The tip elevations of the existing I-wall sheet piles 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	SHEET PILE TIP ELEVATION. (FT) NAVD88	EAST BASELINE APPROXIMATE STATION	PROTECTED SIDE LEVEE CREST ELEVATION (FT) NAVD88	SHEET PILE TIP ELEVATION. (FT) NAVD88
663+30 to 669+17	9.0	-2.5			
662+91 to 663+30	Transition	-2.5	663+00 to 670+63	9.5	-1.5
660+40 to 662+91	10.5	-2.5	661+00 to 663+00	Transition	-1.5
659+66 to 660+40	12.0	0.0	643+00 to 661+00	10.5	-1.5
641+86 to 659+66	10.5	-1.5	**I-10**		
I-10			636+00 to 638+31	8.0	-1.5
636+06 to 639+06	10.0	-1.5	635+00 to 636+00	Transition	Transition
635+06 to 636+00	Transition	0.0	634+00 to 635+00	Transition	-6.5
626+44 to 635+06	6.5	-4.5	627+28 to 634+00	6.0	-6.5
Veterans Blvd.			**Veterans Blvd.**		
614+00 to 625+14	6.0	-4.5	615+00 to 624+27	5.0	-4.8
590+06 to 614+00	5.5	-6.0	614+00 to 615+00	Transition	Transition
588+00 to 590+06	Transition	-6.5	613+00 to 614+00	Transition	-8.3
554+05 to 588+00	4.5	-6.5	553+70 to 613+00	4.0	-8.3
Hammond Hwy.			**Hammond Hwy.**		
549+94 to 552+21	8.0	-1.5	550+22 to 552+10	8.5	-28.0

6.1.3 Pump Stations

Drainage Pump Station No. 6 (DPS 6) is located at the south end of the 17th Street Canal. The building was originally constructed in 1899 and has undergone several additions and modifications over the years. In the 1980s, a pile supported cantilevered floodwall monolith was constructed on the outlet side of the pump station, with a top grade of El 14 NAVD88. The wall is supported on top of a pile supported, concrete mat which forms the basin floor at El 2 NAVD88.

The flood walls on both sides of the discharge basin at DPS 6 are substantially different. The west side consists of a retaining wall with a 5.2H:1V slope above to the crest of the embankment. The retaining wall is located at the flood side toe of the embankment with a top of the I-wall grade of El 1.5 NAVD88. The embankment slopes up from the I-wall to the crest at El 11.5 NAVD88. The east side consists of an earth embankment with sheet pile wall at the crest. The sheet pile tip elevation for the west I-wall was based on the adjacent Reach 18 sheet pile tip elevation, since the "as-built" drawings for this I-wall did not indicate the sheet pile tip grade. A tip grade of El -4.0 NAVD88 was use in the analysis. The east embankment crest grade is El 8 2 NAVD88 and the sheet pile top grade is El 12.8 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 17th St. canal consists of thirteen 11 x 10.25' wide gates with a flow-rate capacity of 12,500 cubic feet per second. There are three stages of pumps used at the ICS; the phase 1 pumps consist of 12 MWI pumps with the power unit located on the engine platforms, phase 2 consists of 6 MWI pumps with the power units located on the pump platform, and phase 3 consists of 11 Fairbanks Morse and 14 MWI pumps with the power units located on the pump platform/closure platform.

6.1.4 Canal, Levees and Protected Side Grades

Surveys of the canal were performed during June 2009. Levee cross sections were taken approximately every 100 to 200 feet along the baselines on each side of the canal. 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. Cross sections were extended on to private property with reflectorless EDM devices. The survey report is included in Appendix C and the coordinates of the east and west canal baselines are included in Appendix G.

The average canal bottom width is about 90 feet and varies between about 70 and 110 feet. The top width of the canal averages about170 feet and varies between 150 and 210 feet. The canal bottom grade is relatively consistent across each section and ranges from about El -12 NAVD at the south end of the canal near DPS 6 to about El -18 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. 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. The survey cross sections are included in Appendix A.3 on Plates 55 through 74.

6.2 GEOTECHNICAL INVESTIGATIONS

The Corps initiated the field investigations along the 17th Street Canal during the period 1971 to 1973 with the completion of nine borings. From 1981 through 1986, a total of 110 borings were drilled for the development of DM 20 [6] which was issued in March 1990. These investigations were competed for I-wall design to increase the parallel protection along the canal levees. During 1995 through 1999 six additional borings were drilled for additional evaluations of the canal. Thus, a total of 125 borings were drilled along the canal prior to Katrina. Following the I-wall failures 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 117 test borings were drilled within reaches under consideration in this report for preparation of DM 20 and pre-and post DM 20 investigations prior to Katrina. The distribution of these borings along the canal is illustrated in Table 6-2. A total of 34 borings were drilled along the protected side of the west levee and two on the flood side. Four borings were drilled along the protected side of the east levee and three along the flood side. A total of 23 borings were drilled along the crest of the west levee and 38 along the crest of the east levee. A total of eight borings were drilled within the canal. An additional two borings were drilled upstream of DPS 6 in the canal. Three borings were drilled within the area of the DPS 6.

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 boring 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.

6.2 2 Post Katrina Investigations

Following the I-wall failure in August 2005, 100 test borings, 103 CPTs, and 17 VSTs were performed to evaluate the subsurface conditions along and within the canal.

6.2.2.1 Borings

A total of 12 borings were drilled in October 2005 at the request of the IPET investigators to fill in the data gaps for their analyses. Five borings were drilled in the canal, five on the protected side and two borings on the centerline.

An additional 88 borings were drilled during 2006 and 2007 beyond the IPET investigation area to evaluate the subsurface conditions and to obtain samples for laboratory testing. A total of 36 borings were drilled along the protected side toe of the west levee and 16 borings were drilled along the levee crest. Three borings were drilled along the west levee flood side. Along the east levee, 21 borings were drilled at the protected side toe and 10 borings were drilled along the levee crest. Ten additional borings were drilled along the east levee flood side toe. Four additional borings were also drilled within the canal.

TABLE 6-2
DISTRIBUTION OF TEST BORINGS

WEST AND EAST BASELINE STATIONS	INVESTIGATION LOCATIONS									
	WEST SIDE				CANAL		EAST SIDE			
	PROTECTED SIDE		CREST				CREST		PROTECTED SIDE	
	PRE-KATRINA	POST KATRINA	PRE-KATRINA	POST KATRINA	PRE-KATRINA	POST KATRINA	PRE-KATRINA	POST KATRINA	PRE-KATRINA	POST KATRINA
545+00 to 550+00	1[2]	3[2]	2	0	0	0	0	0	1[2]	4[2]
550+00 to 560+00	1+1[2]	3	3	2	2	1	3	0	3+2[2]	2
560+00 to 570+00 Breach	3	5	0	4	0	4	3	3	0	6
570+00 to 580+00	3	3	0	3	0	0	3	1	0	1
580+00 to 590+00	3	3	0	3	0	0	3	1	0	1
590+00 to 600+00	0	3	3	3	0	0	3	1	0	1
600+00 to 610+00	1	0	1	0	0	0	2	1	0	1
610+00 to 620+00	1	0	2	0	0	0	3	1	0	1
620+00 to 630+00	3	0	0	0	0	0	5	1	0	7+1[2]
630+00 to 640+00	3	0	0	0	0	0	3	0	0	0
640+00 to 650+00	9+1[2]	12	4	1	4	0	3	1	1	1
650+00 to 660+00	3	7	4	0	1	4	3	0	0	0
660+00 to 670+00	3	0	3	0	1	0	3	0	0	0
670+00 to DPS 6	2[3]	0	1	0	2[4]	0	1	0	1[3]	0

TOTALS	**34+ 2[2]+ 2[3]**	**36+3[2]**	**23**	**16**	**8+ 2[4]**	**9**	**38**	**10**	**4+ 3[2]+ 1[3]**	**21+5[2]**
Notes: [1] Borings drilled from 1995 through 1999 [2] Borings located on flood side [3] DPS 6 area across canal [4] Upstream of DPS 6										

The excavation required for construction of the canal removed a significant portion of the marsh clay deposits and in some areas exposed the underlying barrier beach sands Deposition of soils in the base of the canal and scour of the canal bottom likely caused changes to the conditions which prevailed upon completion of the improvements to the canal in the 1990s. Only nine borings were drilled in the canal bottom since completion of the improvements to the parallel protection system. These borings were drilled between Stations 655+00 and 658+00 and in the area of the breach between Stations 559+50 and 566+00. A complete list of the 217 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 9 of Appendix A4

6.2.2.2 Cone Penetration Tests

A total of 103 CPTs were performed between 2005 and 2010. Twenty-eight CPTs were completed for the IPET investigation in 2005 and 2006. Seventy five CPTs were completed during 2006, 2007, 2009 and 2010. A total of 19 CPTs were performed on the west levee protected side toe and 27 CPTs were performed on the levee crest. Two of the crest CPTs were performed for the IPET investigation. One CPT was performed along the flood side toe of the west levee. Along the protected side toe of the east levee 33 tests were advanced including 12 for the IPET investigation. Twenty two CPTs were performed along the crest of the east levee including 13 for the IPET investigation. One CPT was performed in the canal for the IPET investigation. 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 9 of Appendix A.4.

6.2.2.3 Vane Shear Tests

Seventeen VSTs were also completed in 2006, 2008, 2009 and 2010. These tests were performed in the very soft to soft consistency marsh clays to estimate the undrained shear strength of these soils. Six tests were performed along the west levee: two along the protected side and four on the crest. Eleven tests were performed along the east levee: two on the crest and nine along the protected side toe. 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 9 of Appendix A.4. The field investigation logs, for the entire data set used in development of this study, are provided in Appendix E.

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
545+00 to 550+00	0	0	0	0	0	0	0	0	0	0
550+90 to 560+00	1	0	2	0	0	0	4	0	5	3
560+00 to 570+00 East Side Breach	2	0	5	1	1	0	11	2	10	1
570+00 to 580+00	3	0	5	0	0	0	1	0	3	0
580+00 to 590+00	3	1	4	0	0	0	1	0	1	1
590+00 to 600+00	3	1	2	0	0	0	1	0	3	1
600+00 to 610+00	1[1]	0	1	0	0	0	1	0	1	0
610+00 to 620+00	1	0	1	0	0	0	1	0	1	0
620+00 to 630+00	0	0	2	0	0	0	2	0	0	0
630+00 to 640+00	0	0	2	0	0	0	0	0	3	1
640+00 to 650+00	2	0	1	1	0	0	0	0	2	0
650+00 to 660+00	2	0	1	1	0	0	0	0	2	1
660+00 to 670+00	2	0	1	1	0	0	0	0	2	1

670+00 to DPS 6	0	0	0	0	0	0	0	0	0	0
TOTALS	**19+ 1[1]**	**2**	**27**	**4**	**1**	**0**	**22**	**2**	**33**	**9**
Note: [1] CPTs located on flood side										

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.

6.3.1 Recent Canal Sediments

Borings were drilled within the canal only between Stations 550+00 and 566+00 and between Stations 642+00 and DPS 6. The recent canal sediments consist of fat clays in the northern area of the canal including near the breach between Stations 559+50 and 566+00 and lean clays and poorly graded sands in the southern area of the canal. The thickness of these materials is difficult to assess. Borings performed in the canal bottom do not differentiate between recent canal sediments and older marsh clays. It is likely that the lean clays represent the recent canal sediments. The poorly graded sands likely represent barrier beach sands.

6.3.2 Fill Clays

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 16 feet in thickness along the crests of the levees but is generally 10 to 12 feet thick. Along the levee toes the fill ranges from 1 to 12 feet thick but is generally about 3 to 6 feet thick. Fill material consists of fat and lean clay with some organic matter and artificial fill materials.

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 2 to 17 feet, but typically thicknesses range from about 6 to 8 feet. The thinnest area of the marsh clay is between Stations 640+00 and 660+00, except along the west bank where the thin layer extends from Stations 625+00 to 660+00. The base of the marsh stratum varies from about El -7 NAVD88 near Station 640+00 and then declines to the north to between about El -10 NAVD88 to El -20 NAVD88 with the deepest area to about El -22 NAVD88 along the west bank centerline at Station 560+00. 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 Prodelta Clays

In the southern reaches of the canal, south of Station 660+00, prodelta soft to medium consistency fat clays underlie the marsh stratum where the surface of the barrier beach sands dips downward. These clays are generally less than about 10 feet thick.

6.3.5 Lacustrine Clays

These predominately soft to medium consistency fat clays of lacustrine origin underlie the marsh clays north of Station 617+00. The stratum varies in thickness from about 18 to 28 feet at the north end on the canal and averages about 20 feet thick until it thins out south of Station 610+00.

6.3 6 Barrier Beach Sands

The barrier beach sand stratum underlies the marsh clay stratum from Station 617+00 to 660+00. 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 stratum extends beneath the prodelta clays to the south and the lacustrine clays to the north. The beach sand varies in thickness from about 4 to 8 feet at the north end of the canal. It then thickens southward to about 8 to 14 feet until the stratum increases to about 20 to 25 feet thick at Station 617+00 where the lacustrine clay stratum terminates. The maximum thickness is about 40 feet where the marsh clay is thinnest between about Stations 640+00 and 660+00. The stratum thins to the south of Station 660+00 where the prodelta clays separate this stratum from the overlying marsh clays. The base of the stratum varies from about El -40 to -50 NAVD88.

6.3.7 Bay Sound Clays

The bay sound clay stratum underlies the barrier beach sands and varies from about 15 to 40 feet in thickness. The stratum consists of medium 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 through the north and mid sections of the canal to about -90 NAVD88 near the south end of the canal.

6.3.8 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 10 through 18 - East Bank Centerline Soil and Geologic Profiles;
- Plates 19 through 27 - East Bank Toe Soil and Geologic Profiles;
- Plates 27 through 36 - West Bank Centerline Soil and Geologic Profiles;
- Plates 37 through 45 - West Bank Toe Soil and Geologic Profiles; and
- Plates 46 through 54 – Soil and Geologic Cross Sections A-A’ through I-I’.

The cross section locations are shown on Plates 1 through 9 in Appendix A.4. 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 tip elevations of the original I-wall sheet piles and replacement T-wall are plotted on Plates 10 through 18 and 27 through 36 in Appendix A.4.

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 poorly graded sand;
- Fill - Fat and lean clay with some organic matter and artificial fill materials;
- 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;
- Prodelta – Soft to medium consistency fat clay;
- 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 fat 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 20 [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 20

During preparation of DM 20 [6] laboratory testing was performed on selected samples obtained along the 17th Street 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 20 are included in Appendix E. The shear strength versus depth plots used in the design are included on Plates 56 and 57 of DM 20. The shear strength versus depth properties were estimated to be similar within the following four canal reaches:

- DPS 6 to Station 670+00;
- Station 670+00 to Station 635+00;
- Station 635+00 to Station 552+70 and
- Station 552+70 to 545+00.

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 20 [6]. Unit weight data obtained from laboratory testing of samples were supplemented by the unit weight data included in DM 20. 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 17th Street 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 17TH STREET 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. The permeability values for the non granular fill placed as a thin cap on the protected side of the replacement T-wall and the granular fill placed below the cap and T-wall base slab to a as deep as El -17 NAVD88 were estimated by Martin and Bachus [9]. These values were used in their study of the seepage in the T-wall area. These values were: k = 3x10-4 (1x10-5 ft/sec) and k = 3x10-3 cm/sec (1x10-4 ft/sec), respectively, and were assumed for use in this study.

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 17th Street 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 [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 17th Street Canal.

6.6.1.1 17th Street Canal Pump Test Permeability Data for Poorly Graded Sand

A pump test [25] was performed adjacent to the 17th Street Canal by the Corps in 2006 to evaluate the permeability of the barrier beach poorly graded sand stratum. The test site was located on the east side of the canal, centered on the I-10I-610 right of way just east of Bellaire Drive. 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 1.6 to 5.6 percent and averaged 3.1 percent. 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 1.1 x 10-2 cm/sec to 1.9 x 10-

2 cm/sec, with an average of about 1.5 x 10-2 cm/sec. These values are similar to the values measured in the same formation at London Avenue Canal Pump Test [20].

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

The permeability of the barrier beach poorly graded 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 gra n size correlation analyses presented above for the 17th Street 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 x 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 17th Street pump test, the ERDC recommended value, k = 1.5 x 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 x 10-3 to 0.27 x 10-3 cm/sec and the average value was 1.59 x 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 5.78x10-4 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 17th Street 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 17th Street 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 17th Street 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 (protected side toe only). 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. 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:

Su = qc/Nc; where Nc = 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 undrained shear strengths of the marsh and lacustrine clays under the centerline of the levees were estimated from data included in DM 20 [6], more recent CPT [10] data obtained along the crest of the levees and from testing of undisturbed samples from borings obtained in 2006.

The original design did not take into consideration that the undrained shear strength of the marsh and lacustrine clays under the crest were higher than strengths at the toe of the levee due to the consolidation of these soils under the weight of the levee fills. Since the undrained shear strength testing performed during the 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 CPT and VST data [10] and testing of undisturbed samples from borings obtained after Katrina. Undrained shear strength values at the toe were generally selected such that they were not greater than 95 percent of the centerline undrained shear strength values, except in the case of fill where some undrained strengths were higher at the toe then at the centerline. If only DM 20 [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, except for fill soils as noted above. 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 shear strength properties of the beach sand stratum were estimated based on review of available data [10].

The undrained shear strength of the bay sound clays were obtained from DM 20 [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, lucustine clay and bay sound clay strata were obtained from DM 20 [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 20. The unit weight of the underlying beach sand stratum was estimated based on review of available data [10].

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 variations with depth is include 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 use MOP as a check. It was also agreed to use finite element seepage analysis when specific projects dictate this level of analysis.

6.10 17TH STREET CANAL FULL SCALE CANAL SEEPAGE TEST

A full scale seepage test was performed in the canal between about Stations 642+50 and 643+50 under the direction of Eustis Engineering Company (Eustis) in December 1983. A report of the test, dated January 12, 1984, is included in DM 20 [6]. The test was performed to evaluate the planned dredging in the canal in conjunction with planned improvements to the parallel protection system. The selected site was located at a high point in the underlying barrier beach sand stratum so that the excavation would penetrate the sand layer. A 50 foot by 100 foot area was excavated into the beach sand deposits in the center of the canal between Reaches 16 and 33. The excavation was approximately 10 feet below canal bottom grade, or about El ~ -9 NAVD88. This elevation and other discussed below were convert from the Cairo Datum used in the Eustis Report to NAVD88 based on the known elevation of the ground surface at Piezometer P-1. The El ~ -9 NAVD88 is lower than the present canal base grade. Figure 6-1 provides a plan of the test location. The test location is shown on Plate 7. Piezometers were located as shown on Figure 6-1 and Plate 7 and are designated P-1 (1983) through P-6 (1983).

The observations and conclusions of the 1984 Eustis report were:

1) *"All six (6) piezometers functioned throughout the test period.*
2) *Variations of the water elevation in the piezometers before, during and after excavation did not respond to the variations of the water elevation in the canal but, instead, responded to the amount of rainfall in the area.*
3) *The underlying sand stratum was exposed over some portion if not over the entire bottom area of the test section on 16 December* [1983].
4) *During the period when the underlying sand stratum was exposed on 16 December* [1983], *the water elevation in the canal rose 0.41 of a foot but the water elevation in the piezometers fell slightly or remained unchanged.*
5) *Sedimentation deposits covered the bottom of the test section in a relatively short period of time.*

Based on the foregoing observations, the following conclusions may be reasonable.

1) *The water elevation in the piezometers was not affected by the water level in the canal because the surface of the underlying sand has become intermixed with fines to some depth below design grade* [El ~ -18 NAVD88]. *This layer of contaminated sand acts as a seal preventing the water in the canal from influencing the hydrostatic head at and beyond the levee toe.* [The design graded referenced was specified by the New Orleans Sewerage and Water Board for the proposed dredging project.]
2) *Upon completion of the proposed dredging to design grade in the canal, sedimentation will probably deposit on the bottom in a relatively short period of time further sealing off the water pressure in the canal from the surrounding ground water."*

Subsequent to excavation of the test section on December 16, 1983, three borings, E-1 (1983) through E-3 (1983) were drilled within the test section on December 19, 1983.

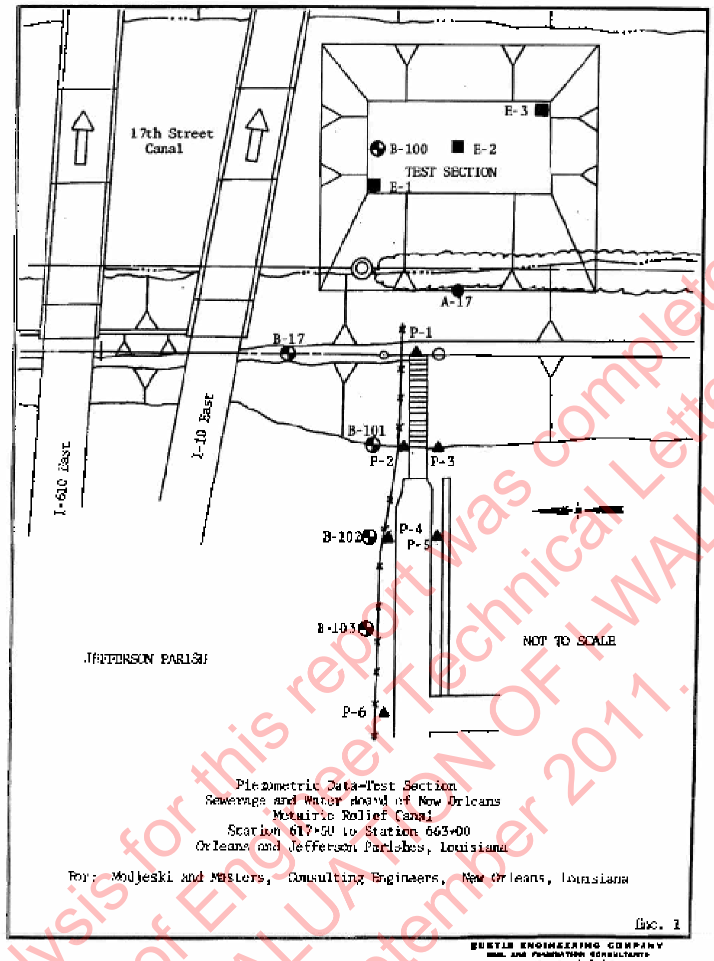

FIGURE 6-1
LOCATION OF CANAL TEST SECTION

These borings indicated that 2.7 to 4 feet of sediment was present within the test section overlying the beach sand. No laboratory testing was performed to verify the classification of the sediment. Eustis further concluded:

"However, sound engineering judgment would indicate that piezometers should be installed along the entire reach in which the sand stratum may be exposed at the bottom of the canal. Readings should be taken during and subsequent to excavation operations to more definitively define the reaction of the sand strata to the water level in the canal."

6.11 LEVEE REACHES

The canal was originally divided into several reaches along both the east and west levees in DM No. 20 [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.

6.11.1 Reach Definition

The canal was initially subdivided into the reaches 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.

- I-wall Sheet Pile Tip Elevations - The tip elevations of the sheet pile cut off walls below the I-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 and the top of the beach sand are 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 and toe ground surface grades were selected for each reach and these grades were used throughout an individual reach. 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 35 reaches, 18 on the west bank and 17 on the east bank, based on these criteria as shown in Table 6-5. One reach contains a T-wall and it was excluded from the numbering system in this analysis. The reach locations are shown on Plates 1 through 9 included in Appendix A.4.

The bridges were also excluded from the reaches. 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.

6.11.2 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.4 on Plates 55 through 74. The survey cross sections include the revised design ground surface cross sections used in this MOWL Study. Plates 1 through 9 in Appendix A.4 provide an aerial view of the canal alignment. The reach locations are indicated on these plates.

TABLE 6-5
LEVEE REACH LOCATIONS

WEST REACH	WALL TYPE	WEST BASELINE APPROXIMATE STATION	EAST REACH	WALL TYPE	EAST BASELINE APPROXIMATE STATION
1	I-wall	ICS to 552+22	19	I-wall	Closure to 552+17
Hammond Hwy. Bridge		552+22 to 553+70	**Hammond Hwy. Bridge**		552+17 to 553+58
2	I-wall	553+70 to 565+00	20	I-wall	553+58 to 560+10
3A	I-wall	565+00 to 570+00		T wall	560+10 to 566+00
3B	I-wall	570+00 to 571+45	21	I-wall	566+00 to 570+73
4	I-wall	571+45 to 575+45	22	I-wall	570+73 to 581+50
5	I-wall	575+45 to 578+22	23	I-wall	581+50 to 588+67
6	I-wall	578+22 to 582+60	24	I-wall	588+67 to 598+24
7	I-wall	582+60 to 585+55	25	I-wall	598+24 to 608+00
8	I-wall	585+55 to 588+70	26	I-wall	608+00 to 612+92
9	I-wall	588+70 to 593+00	27	I-wall	612+92 to 615+03
10	I-wall	593+00 to 596+05	28	I-wall	615+03 to 620+30
11	I-wall	596+05 to 609+00	29	I-wall	620+30 to 624+88
12	I-wall	609+00 to 614+00	**Veterans Blvd. Bridge**		624+88 to 626+73
13	I-wall	614+00 to 617+00	30	I-wall	626+73 to 634+09
14	I-Wall	617+00 to 625+00	31	I-wall	634+09 to 637+00
Veterans Blvd. Bridge		625+00 to 626+56	32	I-wall	637+00 to 638+44
15A	I-wall	626+56 to 635+00	**I-10 Bridge**		638+44 to 643+40
15B	I-wall	635+00 to 639+06	33	I-wall	643+40 to 658+00
I-10 Bridge		639+06 to 641+85	34	I-wall	658+00 to 662+87
16	I-wall	641+85 to 658+00	35	I-wall	662+87 to 670+63
17	I-wall	658+00 to 663+00			
18	I-wall	663+00 to 669+36			

7.0 EXISTING SAFE WATER CONDITIONS

The majority of the reaches along the east and west banks of the 17th Street Canal are adjacent to residential neighborhoods. 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 17th Street 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-wall, 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 17th Street 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, PMA22, Casteel CZ-113, and ARBED BZ12.1. 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 17th Street 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. 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. Executable input files are located in Appendix F.

The FOS calculated by the Spencer's Method of analysis for Reaches 3B, 6 and 29 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 of the embankment and foundation soils. In Reaches 22, 23, and 25 through 27, the MOP stability analysis controlled the MOWL.

7.1.2 Gap Analysis

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.

Table 7-1
Global Stability MOWLs and Factors of Safety for I-walls within Levees

WEST REACH	SPENCER'S METHOD		MOP		EAST REACH	SPENCER'S METHOD		MOP	
	MOWL NAVD88	FOS	MOWL NAVD88	FOS		MOWL NAVD88	FOS	MOWL NAVD88	FOS
1	10.0	1.55	10.0	1.43	19	10.0	1.89	10.0	2.00
2	7.0	1.41	7.0	1.42	20	7.5	1.41	7.5	1.30
3A	10.0	1.40	10.0	1.33	21	5.0	1.45	5.0	1.43
3B[1]	<1.0	1.38	1.0	1.55	22[2]	4.5	1.44	**4.0**	1.32
4	8.0	1.45	8.0	1.42	23[2]	7.0	1.45	**6.5**	1.33
5	6.0	1.40	6.0	1.34	24	5.0	1.43	5.0	1.37
6[1]	<1.0	1.38	1.0	1.85	25[2]	6.0	1.40	**5.0**	1.32
7	10.0	1.53	10.0	1.53	26	9.0	1.40	**8.5**	1.34
8	8.0	1.42	8.0	1.36	27[2]	9.0	1.43	**8.5**	1.32
9	7.0	1.41	7.0	1.37	28	9.5	1.47	9.5	1.36
10	10.0	1.56	10.0	1.48	29[1]	<1.0	1.31	8.0	1.43
11	10.0	1.55	10.0	1.51	30	9.0	1.42	9.0	1.40
12	10.0	2.02	10.0	1.91	31	10.0	2.55	10.0	2.37
13	10.0	2.05	10.0	2.01	32	10.0	2.10	10.0	1.73
14	10.0	1.65	10.0	1.57	33	10.0	1.81	10.0	1.90
15A	10.0	2.18	10.0	2.06	34	10.0	2.09	10.0	2.18
15B	10.0	1.83	10.0	1.80	35	10.0	2.61	10.0	2.46
16	10.0	1.71	10.0	1.87					
17	10.0	2.03	10.0	2.09					

WEST REACH	SPENCER'S METHOD		MOP		EAST REACH	SPENCER'S METHOD		MOP	
	MOWL NAVD88	FOS	MOWL NAVD88	FOS		MOWL NAVD88	FOS	MOWL NAVD88	FOS
18	10.0	2.17	10.0	2.51					

[1] FOS less than 1.4 for Spencer's analyses at canal water levels equal to El 1 NAVD88, the normal Lake level
[2] MOP MOWL controls (result is **BOLD**)

The MOWL identified in the Spencer's analysis was checked using the MOP methodology. The MOP analysis was again 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 above the sheet pile tip, the analysis was continued using this active wedge location. If the critical active wedge determined in this first step was found to intercept the sheet pile above the sheet pile tip, the active wedge was restrained at the most critical active wedge that did not penetrate the sheet pile.

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. This was the case for Reaches 1 through 18, Reaches 26 through 28 and Reaches 31, 33 and 35.

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 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 F.

A Gap analysis was not performed for Reach 16 because the flood side crest of the earth levee was El 10.2 NAVD88. The Spencer's Method of analysis for Reach 29 resulted in a calculated FOS or 1.21 with water in the canal at the crest of the flood side earth levee, El 8.3 NAVD88. The FOS for the global stability analysis was 1.31 with water in the canal at El 1 NAVD88, the normal Lake water surface level. This low FOS resulted from the low shear strength values used in the analysis for this reach. The MOP analysis was not performed for this reach. The MOP analysis controlled in Reaches 3A, 4, 22 and 25. In Reach 30 in order to achieve a FOS of 1.3 for the MOP analysis the water level in the canal falls below the top of the top of the flood side earth levee of +8.2 feet. The SWE for the gap analysis does not apply.

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
1	-1.5	10.0	1.83	10.0	1.69	19	-15.1	10.0	2.64	10.0	2.85
2	-9.5	4.5	1.41	6.5	1.36	20	-15.0	7.5	1.42	7.5	1.56
3A[3]	-9.5	8.0	1.40	**7.5**	1.31	21	-10.8	4.5	1.59	4.5	1.47
3B	-9.5	1.5	1.42	1.5	1.35	22[3]	-7.7	5.5	1.47	**5.0**	1.32
4[3]	-9.5	5.5	1.43	**4.5**	1.34	23	-16.0	5.5	1.48	5.5	1.35
5	-9.5	4.0	1.41	4.0	1.32	24	-13.5	5.0	1.48	5.0	1.42
6	-9.4	1.5	1.41	1.5	1.57	25[3]	-16.4	6.0	1.40	**5.0**	1.31
7	-9.5	8.5	1.44	8.5	1.44	26	-17.7	7.0	1.41	7.0	1.40

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
8	-9.5	6.5	1.40	6.5	1.32	27	-17.8	7.5	1.43	7.5	1.35
9	-6.0	4.0	1.41	4.0	1.37	28	-14.3	7.5	1.44	7.5	1.32
10	-6.0	9.0	1.43	9.0	1.36	29	-4.1	<1.0	1.31[2]	--[4]	--[4]
11	-6.0	9.0	1.42	9.0	1.34	30	-3.3	8.5	1.46	8.0	NA[2,5]
12	-6.0	10.0	2.03	10.0	1.81	31	-5.4	10.0	2.27	10.0	2.26
13	-6.0	10.0	1.89	10.0	1.89	32	-4.1	10.0	2.13	10.0	1.68
14	-4.5	10.0	1.50	10.0	1.39	33	-3.4	10.0	1.79	10.0	1.78
15A	-4.5	10.0	1.98	10.0	1.81	34	-2.8	10.0	2.16	10.0	2.21
15B	-4.5	10.0	1.63	10.0	1.52	35	-4.5	10.0	2.63	10.0	2.45
16[1]	Gap Analysis not Performed										
17	-2.5	10.0	1.85	10.0	1.68						
18	-2.5	10.0	2.00	10.0	2.18						

[1] The crest of the flood side embankment is above Elevation +10 ft.
[2] FOS less than 1.4 for Spencer's Method analysis at canal water level equal to El 1 NAVD88, the normal Lake level, MOWL below crest of flood side earth levee
[3] MOP MOWL controls (result is **BOLD**)
[4] MOP analysis not performed
[5] SWE below crest of flood side earth levee

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 criterion limits the water height (H1) 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/H1 is required to be at least 3. Table 7-3 provides a summary of the I-wall stability for each canal reach.

TABLE 7-3
SEVENTEENTH STREET CANAL WALL STABILITY

WEST 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	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
1	6.6	6.8	-1.5	**8.1**	**9.3**	10.0	19	7.2	4.9	-33.8	38.7	10.0	10.0
2	4.2	1.8	-9.5	11.3	**8.0**	**8.2**	20	4.5	1.0	-18.9	19.9	10.0	**8.5**
3A	4.5	1.4	-9.5	10.9	**8.1**	**8.5**	21	4.0	1.5	-18.8	20.3	10.0	**8.0**
3B	4.5	1.4	-9.5	10.9	**8.1**	**8.5**	22	3.5	-0.9	-18.8	17.9	**9.5**	**7.5**
4	4.5	1.6	-9.5	11.1	**8.2**	**8.5**	23	3.2	-0.5	-19.0	18.5	**9.4**	**7.2**
5	4.5	1.5	-9.5	11.0	**8.2**	**8.5**	24	3.3	-1.0	-19.0	18.0	**9.3**	**7.3**
6	4.5	1.5	-9.5	11.0	**8.2**	**8.5**	25	3.5	-0.6	-18.8	18.2	**9.6**	**7.5**
7	4.5	1.2	-9.5	10.7	**8.1**	**8.5**	26	3.5	-0.2	-17.7	17.5	**9.3**	**7.5**
8	4.5	1.7	-9.5	11.2	**8.2**	**8.5**	27	3.8	0.5	-17.8	18.3	**9.9**	**7.8**
9	5.0	1.8	-6.0	**7.8**	**7.6**	**9.0**	28	4.3	1.8	-14.3	16.1	**9.7**	**8.3**
10	5.5	2.5	-6.0	**8.5**	**8.3**	**9.5**	29	8.0	8.3	-14.5	22.5	10.0	10.0
11	5.5	2.5	-6.0	**8.5**	**8.3**	**9.5**	30	8.2	8.2	-12.4	20.6	10.0	10.0
12	5.5	2.4	-6.0	**8.4**	**8.3**	**9.5**	31	8.1	8.5	-5.4	13.5	10.0	10.0
13	6.0	3.3	-6.0	**9.3**	**9.1**	10.0	32	8.5	8.5	-5.4	13.9	10.0	10.0
14	6.0	4.2	-4.5	**8.7**	**8.9**	10.0	33	9.7	9.8	-3.4	13.1	10.0	10.0
15A	6.5	3.6	-4.5	**8.1**	**9.2**	10.0	34	9.7	9.9	-3.4	13.1	10.0	10.0
15B	7.3	7.5	-4.5	11.8	10.0	10.0	35	8.1	8.5	-4.5	12.6	10.0	10.0
16	9.9	10.2	-1.5	11.4	10.0	10.0							
17	10.0	9.9	-2.5	12.4	10.0	10.0							
18	8.5	8.6	-2.5	11.0	10.0	10.0							

Reach 1 and Reaches 9 through 15A do not meet the minimum sheet pile penetration of 10 feet as shown in bold face type in Table 7-3. The D/H1 ratio limits the MOWL to slightly below El 10 NAVD88 for Reaches 1 through 15A and 22 through 28. The MOWL for this criterion is above Elevation +8 feet with the exception of Reach 9.

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 2 thru 12 and 20 thru 28. 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 still 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.

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 20 [6] (see Appendix E).

7.1.4 T-Wall Stability

The original construction of the 17th Street Canal parallel protection system did not include T-walls. The I-wall breach that occurred during Katrina was replaced with a T-wall in 2006. This pile supported T-wall was 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 4.9 of this report. "As-built" cross sections of the T-wall section are included in Appendix E.

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

WEST REACH	LOWEST CALCU-LATED SHEET PILE TIP GRADE NAVD88	MODE SWEEP OR FIXED	AS-BUILT SHEET PILETIP GRADE NAVD88	STRENGTH REDUC-TION (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)
1	4.25	Fixed	-1.5	165	19	0.93	Sweep	-33.8	950
2	-5.70	Sweep	-9.5	550	20	-5.22	Fixed	18.9	525
3A	-5.77	Fixed	-9.5	575	21	-4.03	Fixed	-18.8	250
3B	-5.77	Fixed	-9.5	575	22	-16.50	Sweep	-18.8	150
4	-5.02	Fixed	-9.5	650	23	-9.38	Sweep	-19	300
5	-4.21	Fixed	-9.5	300	24	-11.92	Sweep	-19	160
6	-3.80	Fixed	-9.5	650	25	-10.86	Sweep	-18.8	280
7	-5.56	Sweep	-9.5	525	26	7.01	Fixed	-17.7	500
8	-4.02	Sweep	-9.5	550	27	-5.50	Fixed	-17.8	500
9	-5.58	Sweep	-6	510	28	-4.40	Fixed	-14.3	500
10	-2.37	Sweep	-6	500	29	4.16	Sweep	-14.5	575
11	-3.29	Sweep	-6	450	30	7.31	Sweep	-12.4	525
12	-2.74	Sweep	-6	300	31	7.25	Sweep	-5.4	0
13	-2.59	Sweep	-6	600	32	6.77	Sweep	-5.4	300
14	-0.82	Sweep	4.5	710	33	8.34	Sweep	-3.4	745
15A	-0.75	Sweep	-4.5	700	34	8.37	Sweep	-3.4	695
15A	2.33	Sweep	-4.5	700	35	7.44	Fixed	-4.5	75
16[1]	NA	Fixed	-1.5	785					
17[1]	NA	Fixed	-2.5	800					
18	6.98	Fixed	-2.5	700					
[1] Reach 16 and 17 flood embankment approximately +10.0 ft; therefore there is no water load on the I-wall in these reaches.									

According to the "as built" cross sections, the sheet pile cutoff wall beneath the T-wall, extends to El -55 or -67.25 NAVD88, depending on the subsurface conditions. The sheet piles penetrate through the barrier beach sand stratum and into the underlying bay sound clay stratum. The "as-built" cross sections [11] also indicate that granular fill was placed in and under the levee embankment and on the protected side of the levee embankment to about El -17 NAVD88. This replacement is described in the Bachus and Martin report [9] (see Appendix E). The friction angle of the granular fill was reduced, in this MOWL analysis, from the 35 degrees assumed in reference 9, to 32 degrees because of the heterogeneous mix of materials in the fill.

Four survey cross sections of the T-wall were performed during the June 2009 survey of site conditions [12]. The data from these survey cross sections as well as the design cross-section are presented on Plate 74 of Appendix A.4. Review of the survey cross-section and the "as-built" sections indicated an inconsistency with respect to the levee embankment elevations beyond the flood side heel of the T-wall. The flood side ground surface was assumed to slope from the top of T-wall heel concrete, at El 0 NAVD88 to meet the existing cross sections on a 3H:1V slope for the MOWL analysis. This slope is shown on Plate 74.

The T-wall section was analyzed in accordance with the guidelines of Section 4.9 of this report. A summary of the shear strength and unit weight values with depth for the flood-side, centerline, and protected side are included in Appendix A.2 between data for Reaches 20 and 21. These data are plotted on the graphs of Appendix D.4.

The replacement T-wall has visible seepage on the protected side of the flood wall. Piezometers SPP-1 through SSP-17 were installed in this area to monitor the piezometric level on the protected side of the T-wall. Piezometers SSP-1a and SSP1b are located south of the failure on the east bank. SSP2a and 2b are located on the west bank across from the failure area. The monitoring results from these piezometers are provided in the Bachus and Martin report [9] provided in Appendix E. Leakage through the replacement T-wall has resulted in an elevated piezometric level in the granular fill on the protected side of the T-wall. This has caused a localized condition of excessive pressures below the thin clay fill blanket that was placed over the granular fill and some localized seeps through the clay blanket. This is not a wall stability issue. The leakage raises the concern of high exit gradients that could lead to internal erosion and ponding of water on the surface. Recommendations from the Bachus and Martin report identify potential methods to remediate the leakage concerns including the installation of a collection trench near the toe of the levee to collect leakage and convey collected water to the storm water sewer or active dewatering/pumping back to the canal. A second alternative is to place additional fill soil to improve the uplift stability and eliminate the visible evidence of seepage in the area.

The sheet pile was assumed impervious for the seepage analysis as recommended by the TRT. 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 T-wall pile foundations were present. The minimum FOS was 1.76. The MOP FOS was 1.96. Therefore, no unbalanced load exists on the wall and no distributed load exists on the foundation pile system. The slope stability calculations are included in Appendix D.4.

The ENSOFT program, Group 7 [36], was used to analyze the pile groups for the T-wall. The piles supporting the T-Wall are HP 14 by 89 H. A typical pile group layout for one T-Wall monolith was used in the analysis, based on the "as-built" drawings [11]. Since there was no unbalanced load, only the water load acting on the T-wall was applied to the pile group. The water load calculations are included in Appendix D.4.

The "as-built" cross sections [11] indicated that the protected side piles were driven to El -17 NAVD88 through the granular fill that was placed during reconstruction. However, records indicate that the flood side piles did not encounter this material. Soil types cannot be varied horizontally in the ENSOFT program [36], so two different soil profiles were considered. One analysis considered the centerline soil profile and the other included granular fill to El -17 NAVD88. An analysis was performed for each soil profile, and the results were similar with the granular fill producing slightly higher capacities. The centerline soil profile was used for the final analysis. These output files are included in Appendix D.4

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. The elevations of the current levee embankment are lower than they were pre Katrina, therefore, downdrag was not included in the vertical pile capacity analysis. It was determined that the "Q" case produced more conservative end bearing and side friction values.

Structural analyses indicated that the amount of reinforcement in both the walls and the footings was sufficient based on both current HSDRRSDG and EM 1110-2-2104, Strength Design for Reinforced Concrete Hydraulic Structures [29].

The pile deflections at the top of the pile were less than 0.1 inch for both soil profiles analyzed. The moment and shear forces generated in the piles for both soil profiles were within the required limits for the pile capacities considered. When all of the various analysis results were considered, the MOWL for the T wall is greater than El 10 NAVD88. The T-wall calculations are provided in Appendix D.4.

7.1.5 Pump Station Wall Structural Stability

The DPS 6 cantilevered floodwall was evaluated for an MOWL of El 10 NAVD88. This wall has a top of wall grade of El 14 NAAVD88 and a base grade of El 2 NAVD88 at the top of a pile supported concrete mat. A structural analysis of the wall indicates that the strength and stability of the wall is sufficient for a MOWL of El 10 NAVD88. Structural calculations are included in Appendix D.5.

7.1.6 Pump Station Sliding Stability

The overall sliding stability of DPS 6 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 and the analysis cross section was through the intake of the pump station indicating that there is limited soil on the protected side of the pump station. The soil parameters from Reach 18, adjacent to the pump station, were used for the analysis with the exception that the fill properties were adjusted. The undrained shear strength of the clay fill was 800 psf for Reach 18 and this was considered too high. It was reduced to 400 psf with a unit weight of 100 pcf for this analysis based on recommendations in HSDRRDG Section 3 for compacted clay fill [4]. The Spencer's

Method and MOP FOS for global stability were evaluated for a MOWL of El 10 NAVD88. The analyses are provided in Appendix D.6.

Stability analyses were also performed for the DPS 6 discharge basin west side training wall and embankment and the east side embankment and sheet pile wall. Soil parameters for the west I-wall and embankment were estimated from the adjacent Reach 18 parameters as no other data were available. For the east embankment and sheet pile wall the adjacent Reach 35 soil parameters were used. The results of the stability analyses for a MOWL of El 10 NAVD88 are shown in Table 7-5. All FOS were above required minimum values.

The minimum seepage FOS value for the west wall was 2.06. There was no excess head on the east wall. The analyses are provided in Appendix D.6.

TABLE 7-5
STABILITY FACTORS OF SAFETY FOR DPS 6

LOCATION	MOWL NAVD88	SPENCER'S METHOD GLOBAL FOS	MOP GLOBAL FOS	SPENCER'S METHOD GAP FOS	MOP GAP FOS
DPS 6	10	1.46	1.55	NA	NA
West side training wall and embankment	10	3.24	3.20	3.07	2.94
East side embankment and sheet pile wall	10	1.93	1.87	NA	NA

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-wall 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 several reaches the base of the canal was below the elevation of the top of the marsh clay stratum based. This assessment was based on field investigation data along both sides of the canal. This level of excavation could permit a potential direct connection 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 15A, 15B, 16, 31, 32, and 33 were analyzed assuming beach sand in the base of the canal.

Several other reaches have potentially thin canal sediments deposits overlying the beach sand stratum in the canal based on the Soil and Geologic Cross Sections of Plates 46 through 54. These were Reaches 14, 17, 18, 29, 30 and 34. It was assumed based of the canal seepage test described in Section 6.10 of this report, that these reaches contained semi-impervious canal sediments. These sediments cause a reduction in the head due to seepage and a reduced piezometric pressures below the protected side marsh clay stratum relative to the condition of direct contact with the beach sand

7.1.7.2 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 4.10, 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 7-6. The calculation output for the seepage analyses are presented in Appendix D.3. The input and output files are located in Appendix E.

Only in Reach 30 did the sheet piling penetrate through the marsh clay stratum. This is a significant benefit as the marsh clay below the tips of the sheet pile provides head loss, from a potential fully penetrating gap, and thus reduced piezometric pressures in the underlying beach sand stratum. This reduces the uplift pressures at the toe of the protected side of the levee.

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 (only in Reach 30);
- 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 was identified in Reach 16 where the natural semi-impervious canal bottom sediments were assumed to be absent and the bottom of the canal as assumed to consist of the barrier beach sand stratum. The FOS for a canal water elevation of +8 feet of 1.87 was calculated by HPO for Reach 16. (See Appendix D.8)

TABLE 7-6
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
1	-1.5	No	10.0	19	-15.1	No	10.0
2	-9.5	No	10.0	20	-15.0	No	10.0
3A	-9.5	No	10.0	21	-10.8	No	10.0
3B	-9.5	No	10.0	22	-7.7	No	10.0
4	-9.5	No	10.0	23	-16.0	No	10.0
5	-9.5	No	10.0	24	-13.5	No	10.0
6	-9.4	No	10.0	25	-16.4	No	10.0
7	-9.5	No	10.0	26	-17.7	No	10.0
8	-9.5	No	10.0	27	-17.8	No	10.0
9	-6.0	No	10.0	28	-14.3	No	10.0
10	-6.0	No	10.0	29	-4.1	No	10.0
11	-6.0	No	10.0	30	-3.3	No	10.0
12	-6.0	No	10.0	31	5.4	Yes	10.0
13	-6.0	No	10.0	32	-4.1	Yes	10.0
14	-4.5	No	10.0	33	-3.4	Yes	10.0
15A	-4.5	Yes	10.0	34	-2.8	No	10.0
15B	-4.5	Yes	10.0	35	-4.5	No	10.0
16	NA[1]	Yes	8.0				
17	-2.5	No	10.0				
18	-2.5	No	10.0				

[1]Canal water does not reach the I-wall; therefore no gap is formed.

The SWE for piping for Reach 16 based on an open bottom is Elevation +8.0 feet with a FOS of 1.87. This analysis was performed by the HPO based on the individual points of the survey data instead of the design profile due the previous concerns raised in this Reach.

The replacement T-wall was designed with fully penetrating sheet piles through the barrier beach sand stratum into the bay sound clay stratum. The suitability of the length of the sheet pile for the T-wall was checked using the Lane Weighted Creep Ratio

(LWCR) [28]. Since the sheet pile is considered impermeable and fully penetrates into the bay sound clay stratum, the maximum hydraulic head difference (H) between the canal water level and water level on the protected side of the sheet piling was estimated in the analysis. Two 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, El 3 NAVD88; and
- Case 2-Seepage exiting at the toe of the protected side earth levee, El -3 NAVD88.

The calculation was based on assuming the canal water level was at El 10 NAVD88.

The LWCR is defined as

C= L_w/H

where Lw = weighted seepage length N/3+V: N = horizontal seepage length; and V = vertical seepage length. The value of N for the first case was 19 ft and for the second case, 53.5 ft. The calculate LWCR values are shown in Table 7-7. Calculations are provided in Appendix D.4. These values are 17 to 9.6. Since the sheet piles penetrate through the beach sand deposit and the creep ratio for fine sand is 7 [28], the calculated LWCRs are acceptable.

TABLE 7-7
LANE WEIGHTED CREEP RATIO FOR T-WALL

LOCATION	SHEET PILE LENGTH FLOOD SIDE (FT)	SHEET PILE LENGTH PROTECTED SIDE (FT)	FOOTING WIDTH OR HORIZONTAL DISTANCE, N (FT)	CHANGE IN HEAD, H (FT)	L_w (FT)	CALCULATED LWCR
Case 1, El 2.6 NAVD88	55	58	19	7	118.9	17
Case 2, El -3 NAVD88	55	52	53.5	13	124.8	9.6

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 3B, 6 and 29 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 1 and Reaches 9 through 15A do not meet the minimum sheet pile penetration requirement of 10 feet. The penetration ratio on the I-wall will limit the MOWL to below El 10 NAVD88 for Reaches 1 through 15A and 22 through 28. Only Reach 9 has a MOWL below El 8 NAVD88 for this criterion. 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 2 thru 12 and 21 thru 28.

Seepage was not found to be a concern on any reach for a MOWL below El 8.0 NAVD88. Reaches 15A, 15B, 16, 31, 32, and 33 were evaluated with beach sand in the base of the canal; however, the MOWL values were greater than El 8 NAVD88.

The MOWL for the replacement T-wall is El 10 NAVD88. 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 7-8. The elevations in bold identify the controlling criteria below a MOWL of El 10 NAVD88. Table 7-9 provides a summary of the FOS and deflections for the T-Wall and DPS 6. Figures 7-1 through 7-4 provides the MOWL for each criterion along east bank of the canal. Figure 7-5 through 7-9 provides the MOWL for each criterion along west bank of the canal.

TABLE 7-8
REACH MOWL VALUES FOR I-WALLS AND EARTH LEVEES

WEST 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[2]	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
1	Closure to 552+21.5	10.0	10.0	**NO**	**9.3**	10	10	10.0
2	553+70 to 565+00	**4.5**	6.5	YES	8.0	8.2	10	10.0
3A	565+00 to 570+00	8.0	**7.5**	YES	8.1	8.5	10	10.0
3B	570+00 to 571+45	**<1.0**[2]	1.0	YES	8.1	8.5	10	10.0
4	571+45 to 575+45	5.5	**4.5**	YES	8.2	8.5	10	10.0
5	575+45 to 578+22.5	**4.0**	**4.0**	YES	8.2	8.5	10	10.0
6	578+22.5 to 582+60	**<1.0**[2]	**1.0**	YES	8.2	8.5	10	10.0
7	582+60 to 585+55	8.5	8.5	YES	**8.1**	8.5	10	10.0
8	585+55 to 588+70	**6.5**	**6.5**	YES	8.2	8.5	10	10.0
9	588+70 to 593+00	**4.0**	**4.0**	**NO**	7.6	9.0	10	10.0
10	593+00 to 596+05	9.0	9.0	**NO**	**8.3**	9.5	10	10.0
11	596+05 to 609+00	9.0	9.0	**NO**	**8.3**	9.5	10	10.0
12	609+00 to 614+00	10.0	10.0	**NO**	**8.3**	9.5	10	10.0
13	614+00 to 617+00	10.0	10.0	**NO**	**9.1**	10.0	10	10.0
14	617+00 to 625+00	10.0	10.0	**NO**	**8.9**	10.0	10	10.0
15A	626+56 to 635+00	10.0	10.0	**NO**	**9.2**	10.0	10	10.0[4]
15B	635+00 to 639+06	10.0	10.0	YES	10.0	10.0	10	10.0[4]
16[3]	641+85 to 658+00	10.0	10.0	YES	10.0	10.0	10	**8.0**[4]
17	658+00 to 663+00	10.0	10.0	YES	10.0	10.0	10	10.0
18	663+00 to 669+36	10.0	10.0	YES	10.0	10.0	10	10.0

EAST 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/H1 = 3/1 MOWL NAVD88	MAXIMUM 4 FT WATER DEPTH ON I-WALL MOWL NAVD88	CWALSHT MOWL NAVD88	SEEPAGE MOWL NAVD88
19	Closure to 552+17	10.0	10.0	YES	10.0	10.0	10	10.0
20	553+58 to 560+10	**7.5**	**7.5**	YES	10.0	8.5	10	10.0
21	566+00 to 570+73	**4.5**	**4.5**	YES	10.0	8.0	10	10.0
22	570+73 to 581+50	4.5	**4.0**	YES	9.5	7.5	10	10.0
23	581+50 to 588+67	**5.5**	**5.5**	YES	9.4	7.2	10	10.0
24	588+67 to 598+24	**5.0**	**5.0**	YES	9.3	7.3	10	10.0
25	598+24 to 608+00	6.0	**5.0**	YES	9.6	7.5	10	10.0
26	608+00 to 612+92	**7.0**	**7.0**	YES	9.3	7.5	10	10.0
27	612+92 to 615+03	**7.5**	**7.5**	YES	9.9	7.8	10	10.0
28	615+03 to 620+30	**7.5**	**7.5**	YES	9.7	8.3	10	10.0
29	620+30 to 624+88	**<1.0**[2]	**<1.0**[2]	YES	10.0	10.0	10	10.0
30	626+73 to 634+09	**8.5**	**8.0**	YES	10.0	10.0	10	10.0
31	634+09 to 637+00	**10.0**	**10.0**	YES	10.0	10.0	10	10.0[4]
32	637+00 to 638+44	**10.0**	**10.0**	YES	10.0	10.0	10	10.0[4]
33	643+40 to 658+00	**10.0**	**10.0**	YES	10.0	10.0	10	10.0[4]
34	658+00 to 662+87	**10.0**	**10.0**	YES	10.0	10.0	10	10.0
35	662+87 to 670+63	**10.0**	**10.0**	YES	10.0	10.0	10	10.0

Notes:

[1] H_1 = Height of water above the crest of the protected side earth embankment.

[2] FOS less than 1.4 for both Spencer s analyses at canal water levels equal El 1NAVD88

[3] The crest of flood side embankment is above El 10 NAVD88.

[4] Reach analyzed with an open connection between the canal and the beach sand

TABLE 7-9
REACH MOWL VALUES FOR T-WALLS AND DPS6

WALL TYPE	CANAL SIDE	STATION	MOWL NAVD88	SPENCER'S METHOD FOS	MOP FOS	DEFLECTION (IN)	SPENCER'S METHOD GAP FOS	MOP GAP FOS
T-WALL	EAST	560+10 TO 566+00	10	1.76	1.96	<0.1	--	--
DPS 6	--	--	10	1.46	1.55	--	--	--
TRAINING WALL AND EMBANKMENT	WEST	--	10	3.24	3.20	--	3.07	2.94
EMBANKMENT AND SHEET PILE WALL	EAST	--	10	1.93	1.87	--	NA	NA

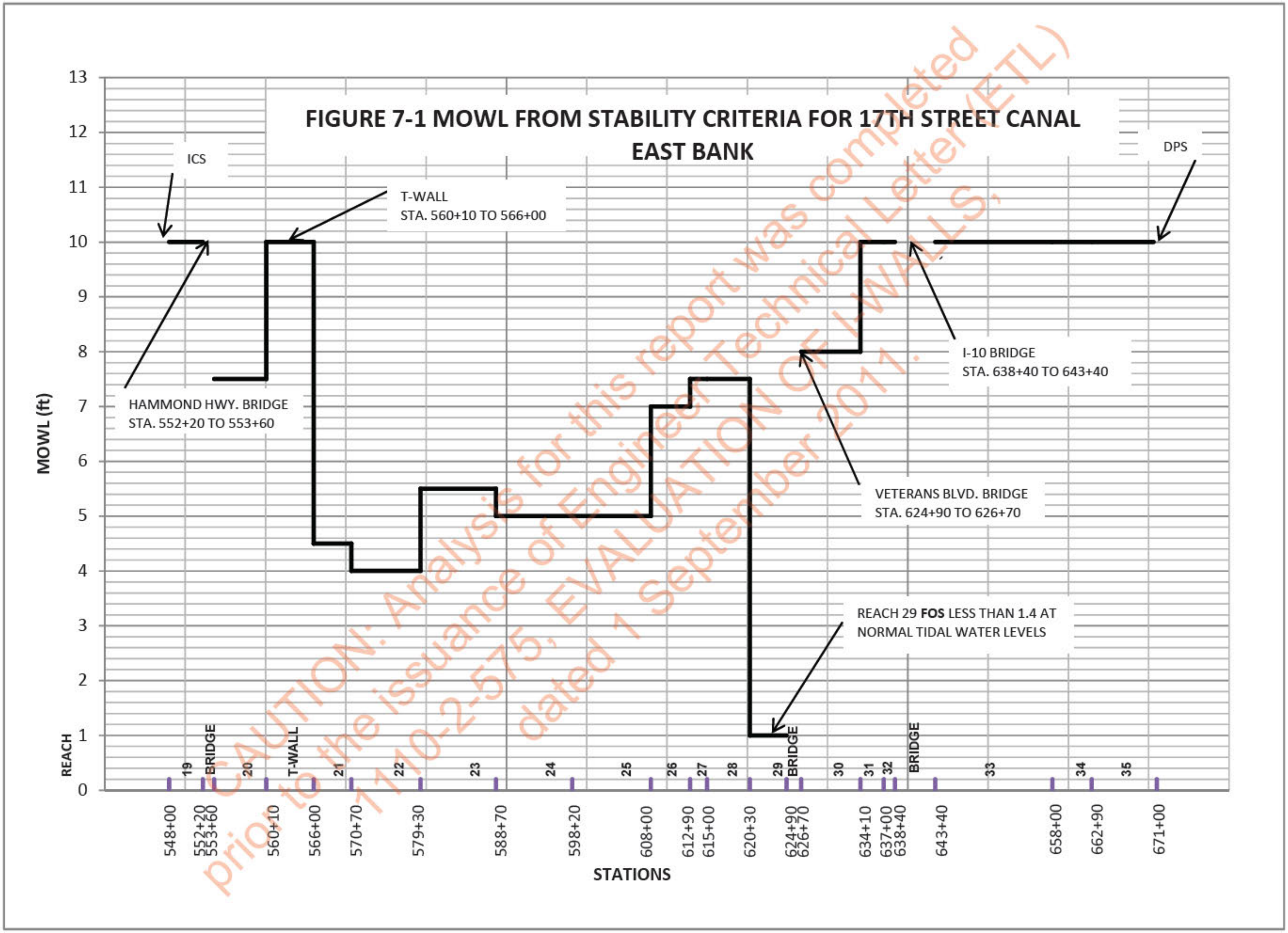
FIGURE 7-1 MOWL FROM STABILITY CRITERIA FOR 17TH STREET CANAL EAST BANK
ICS
DPS
T-WALL
STA. 560+10 TO 566+00
HAMMOND HWY. BRIDGE
STA. 552+20 TO 553+60
I-10 BRIDGE
STA. 638+40 TO 643+40
VETERANS BLVD. BRIDGE
STA. 624+90 TO 626+70
REACH 29 FOS LESS THAN 1.4 AT
NORMAL TIDAL WATER LEVELS
MOWL (ft)
REACH
STATIONS
548+00
552+20
553+60
560+10
566+00
570+70
579+30
588+70
598+20
608+00
612+90
615+00
620+30
624+90
626+70
634+10
637+00
638+40
643+40
658+00
662+90
671+00
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.

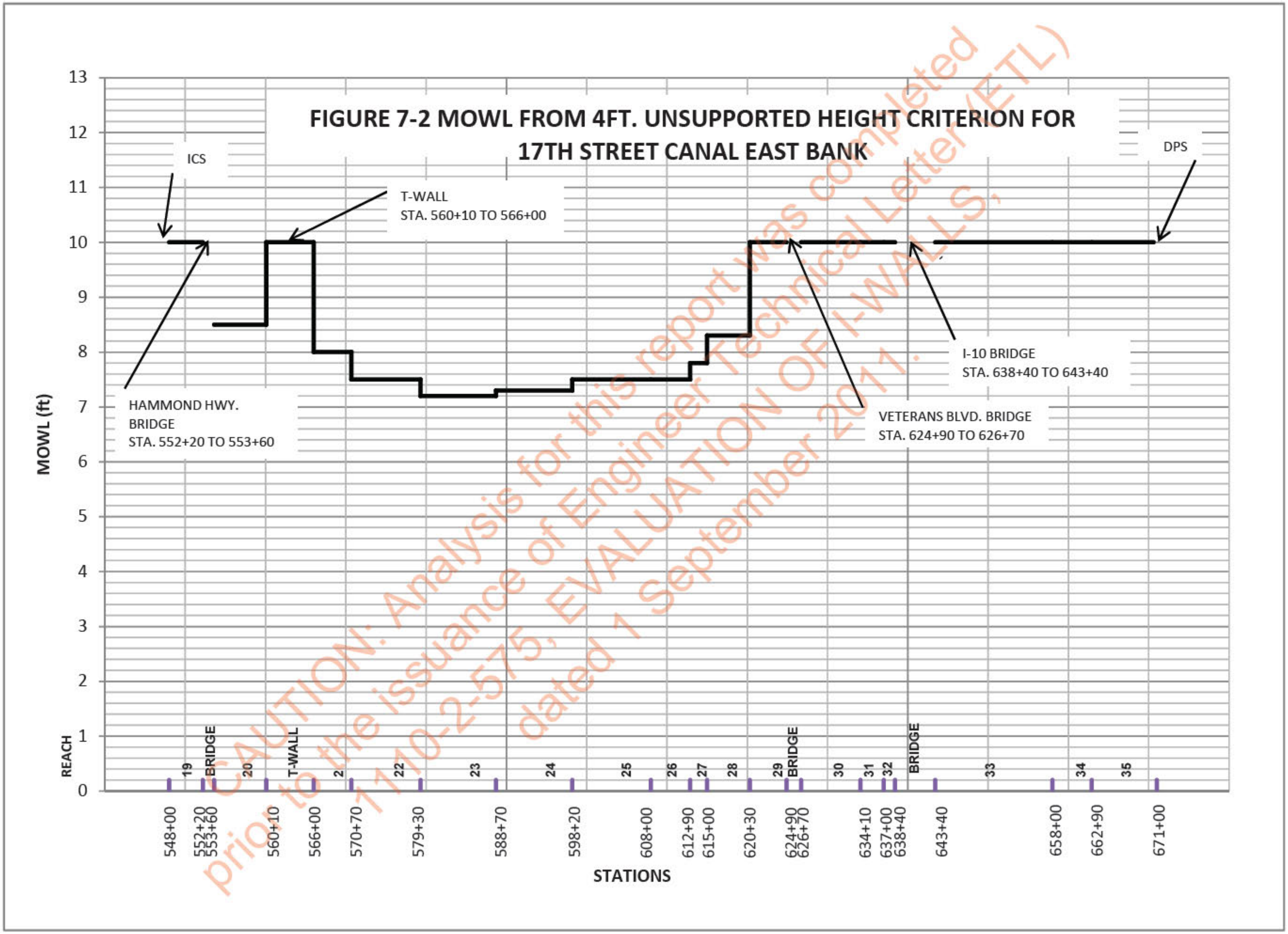
FIGURE 7-2 MOWL FROM 4FT. UNSUPPORTED HEIGHT CRITERION FOR
17TH STREET CANAL EAST BANK
ICS
T-WALL
STA. 560+10 TO 566+00
DPS
HAMMOND HWY.
BRIDGE
STA. 552+20 TO 553+60
I-10 BRIDGE
STA. 638+40 TO 643+40
VETERANS BLVD. BRIDGE
STA. 624+90 TO 626+70
MOWL (ft)
REACH
19
BRIDGE
20
T-WALL
2
22
23
24
25
26
27
28
29
BRIDGE
30
31
32
BRIDGE
33
34
35
548+00
552+20
553+60
560+10
566+00
570+70
579+30
588+70
598+20
608+00
612+90
615+00
620+30
624+90
626+70
634+10
637+00
638+40
643+40
658+00
662+90
671+00
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.

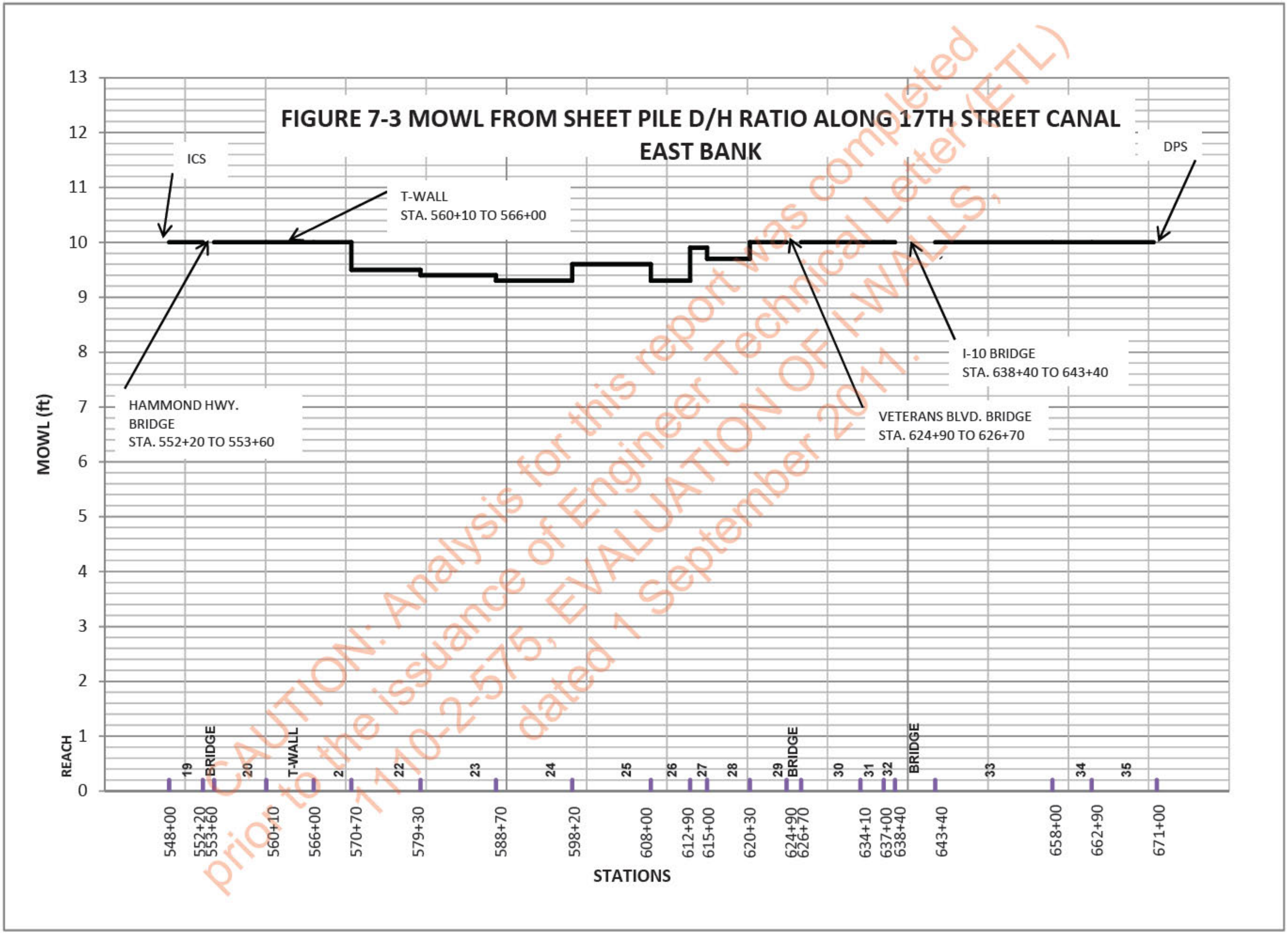
FIGURE 7-3 MOWL FROM SHEET PILE D/H RATIO ALONG 17TH STREET CANAL
EAST BANK
ICS
DPS
T-WALL
STA. 560+10 TO 566+00
HAMMOND HWY.
BRIDGE
STA. 552+20 TO 553+60
I-10 BRIDGE
STA. 638+40 TO 643+40
VETERANS BLVD. BRIDGE
STA. 624+90 TO 626+70
MOWL (ft)
REACH
STATIONS
13
12
11
10
9
8
7
6
5
4
3
2
1
0
19
BRIDGE
20
T-WALL
21
22
23
24
25
26
27
28
29
BRIDGE
30
31
32
BRIDGE
33
34
35
548+00
552+20
553+60
560+10
566+00
570+70
579+30
588+70
598+20
608+00
612+90
615+00
620+30
624+90
626+70
634+10
637+00
638+40
643+40
658+00
662+90
671+00
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.

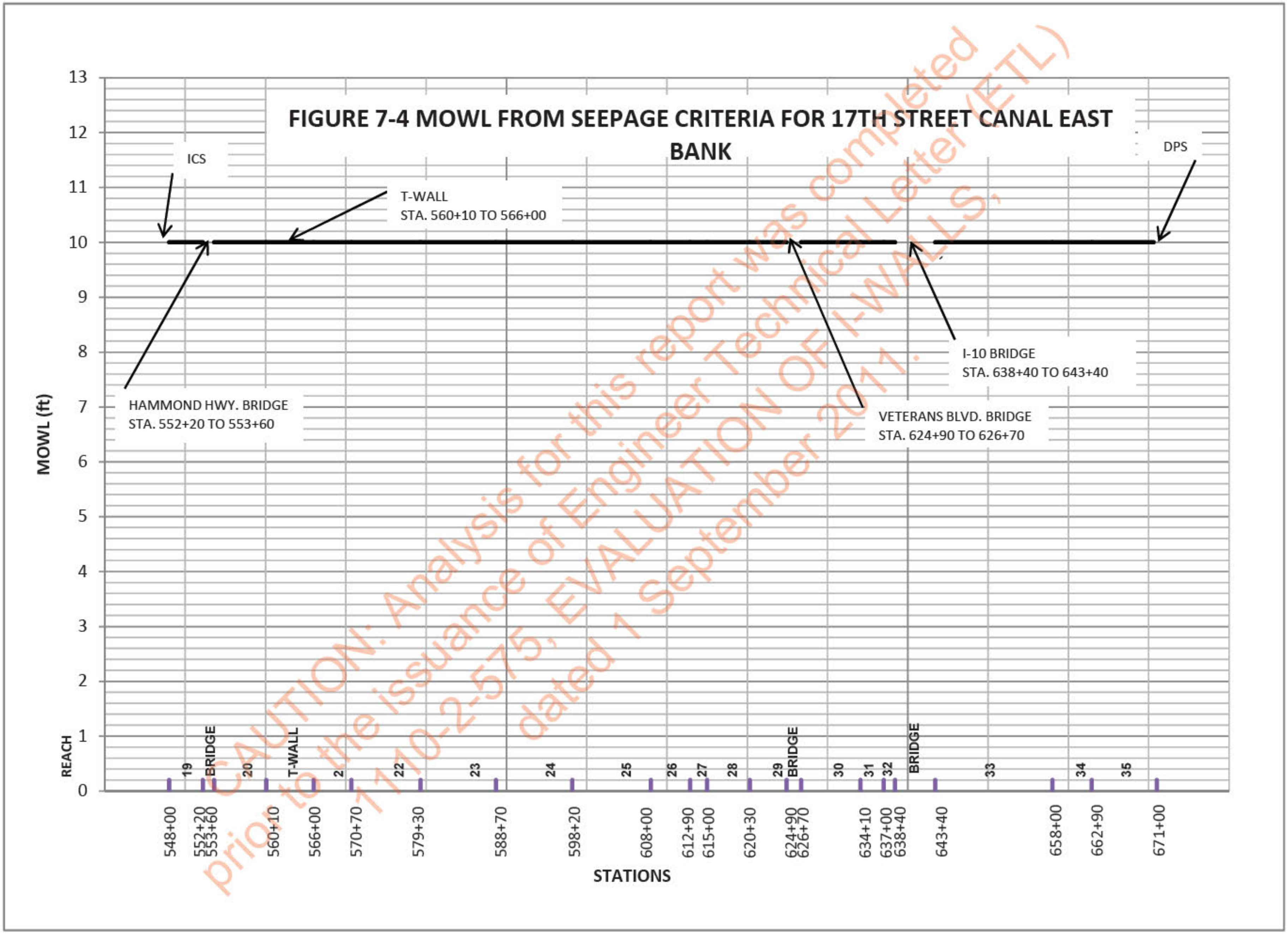
FIGURE 7-4 MOWL FROM SEEPAGE CRITERIA FOR 17TH STREET CANAL EAST BANK
MOWL (ft)
ICS
T-WALL
STA. 560+10 TO 566+00
DPS
I-10 BRIDGE
STA. 638+40 TO 643+40
HAMMOND HWY. BRIDGE
STA. 552+20 TO 553+60
VETERANS BLVD. BRIDGE
STA. 624+90 TO 626+70
REACH
19
BRIDGE
20
T-WALL
21
22
23
24
25
26
27
28
29
BRIDGE
30
31
32
BRIDGE
33
34
35
548+00
552+20
553+60
560+10
566+00
570+70
579+30
588+70
598+20
608+00
612+90
615+00
620+30
624+90
626+70
634+10
637+00
638+40
643+40
658+00
662+90
671+00
STATIONS

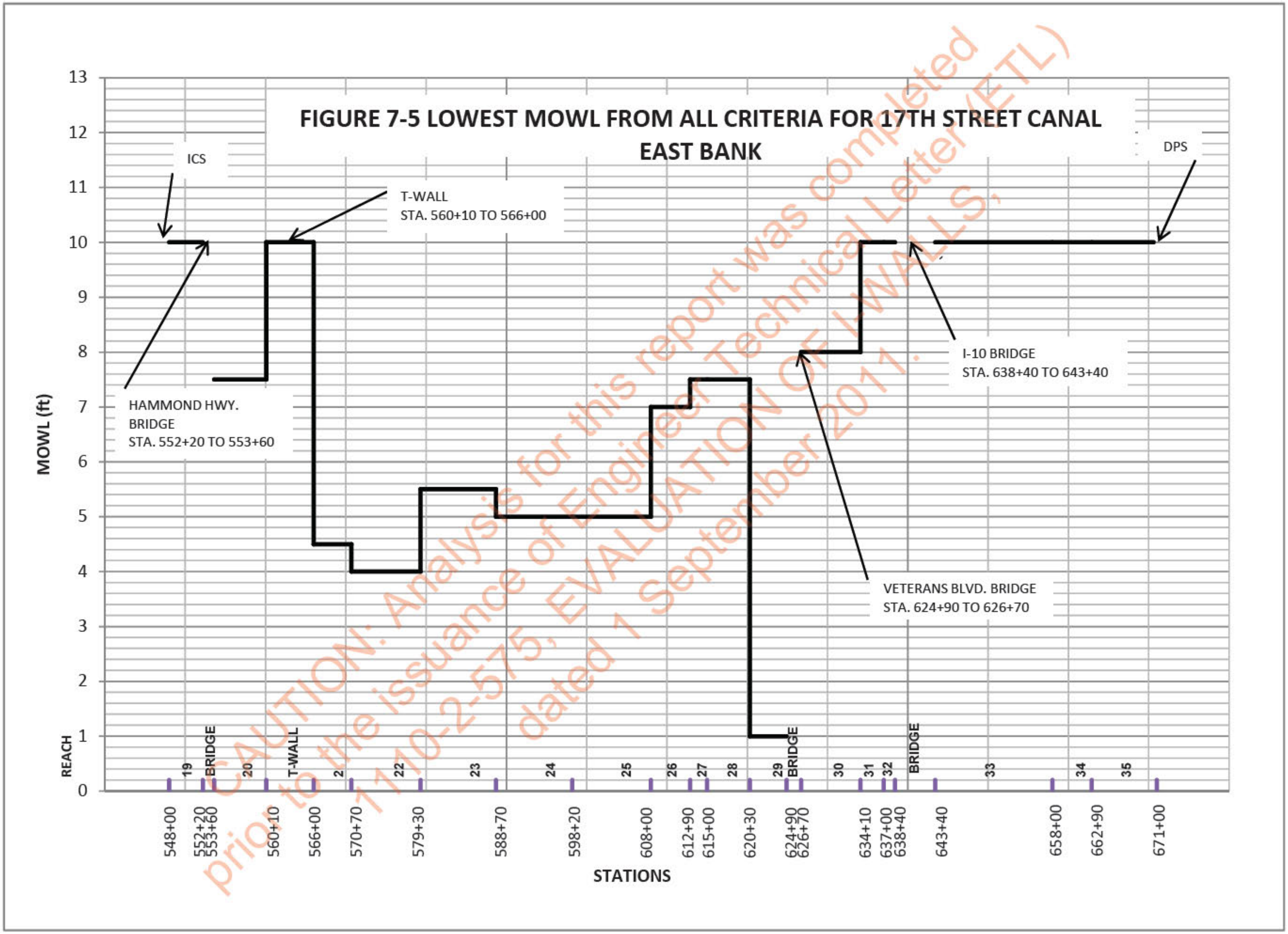

FIGURE 7-5 LOWEST MOWL FROM ALL CRITERIA FOR 17TH STREET CANAL EAST BANK
ICS
T-WALL
STA. 560+10 TO 566+00
DPS
HAMMOND HWY.
BRIDGE
STA. 552+20 TO 553+60
I-10 BRIDGE
STA. 638+40 TO 643+40
VETERANS BLVD. BRIDGE
STA. 624+90 TO 626+70
MOWL (ft)
REACH
STATIONS
0
1
2
3
4
5
6
7
8
9
10
11
12
13
19
BRIDGE
20
T-WALL
2
22
23
24
25
26
27
28
29
BRIDGE
30
31
32
BRIDGE
33
34
35
548+00
552+20
553+60
560+10
566+00
570+70
579+30
588+70
598+20
608+00
612+90
615+00
620+30
624+90
626+70
634+10
637+00
638+40
643+40
658+00
662+90
671+00

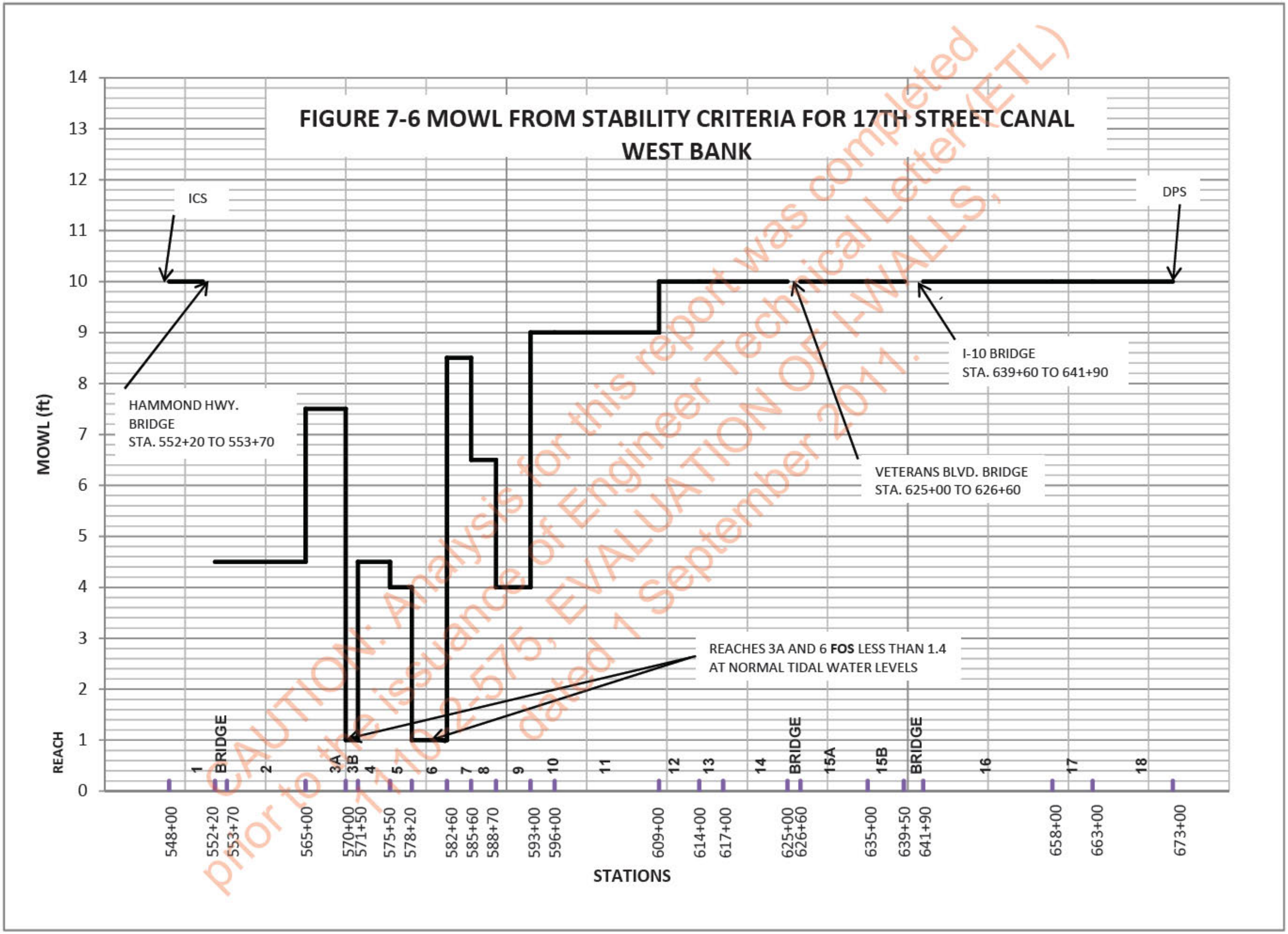
FIGURE 7-6 MOWL FROM STABILITY CRITERIA FOR 17TH STREET CANAL WEST BANK
ICS
DPS
HAMMOND HWY. BRIDGE STA. 552+20 TO 553+70
I-10 BRIDGE STA. 639+60 TO 641+90
VETERANS BLVD. BRIDGE STA. 625+00 TO 626+60
REACHES 3A AND 6 **FOS** LESS THAN 1.4 AT NORMAL TIDAL WATER LEVELS
MOWL (ft)
REACH
STATIONS
0
1
2
3
4
5
6
7
8
9
10
11
12
13
14
1
BRIDGE
2
3A
3B
4
5
6
7
8
9
10
11
12
13
14
BRIDGE
15A
15B
BRIDGE
16
17
18
548+00
552+20
553+70
565+00
570+00
571+50
575+50
578+20
582+60
585+60
588+70
593+00
596+00
609+00
614+00
617+00
625+00
626+60
635+00
639+50
641+90
658+00
663+00
673+00

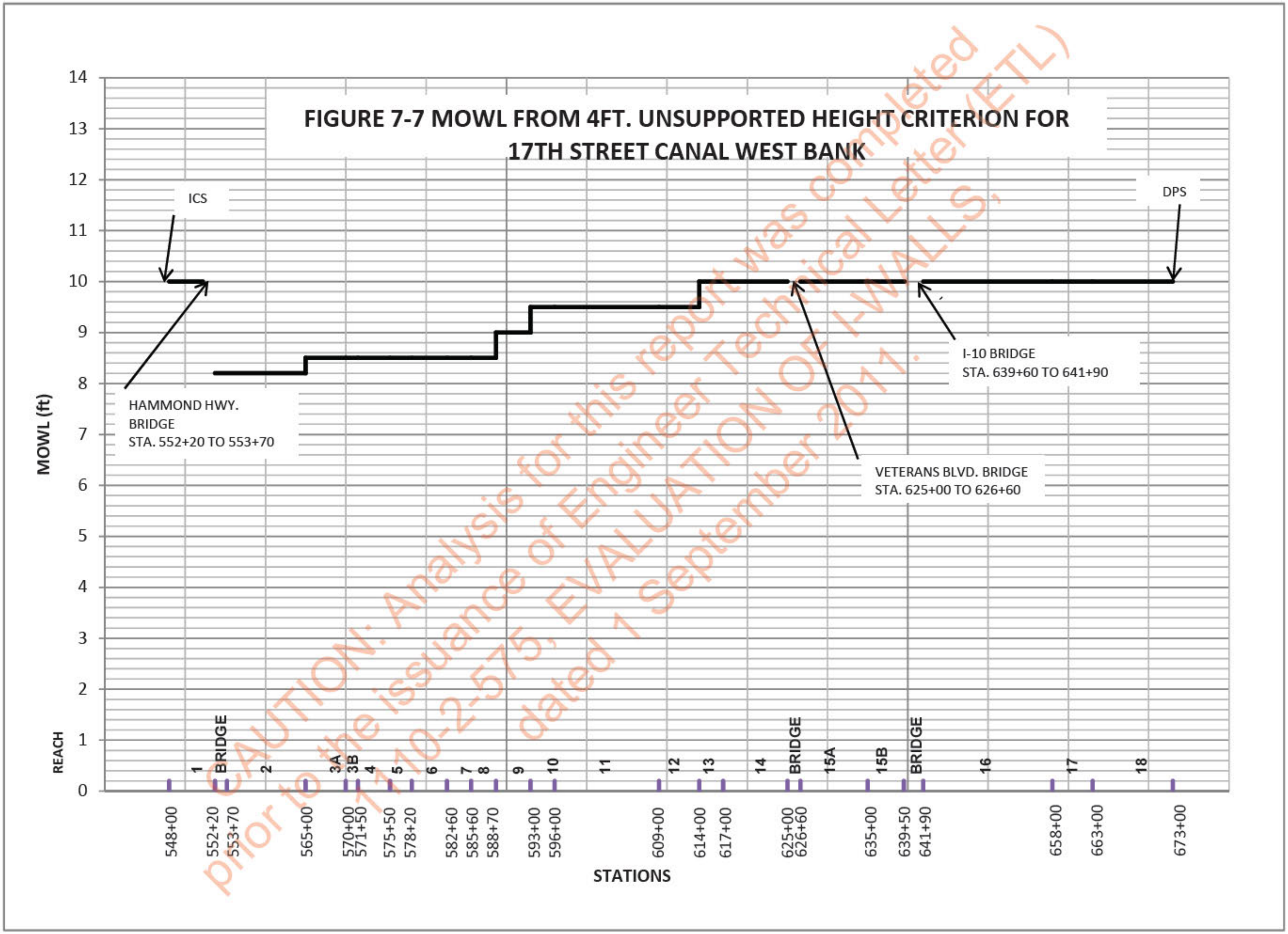
FIGURE 7-7 MOWL FROM 4FT. UNSUPPORTED HEIGHT CRITERION FOR 17TH STREET CANAL WEST BANK
ICS
DPS
HAMMOND HWY. BRIDGE STA. 552+20 TO 553+70
I-10 BRIDGE STA. 639+60 TO 641+90
VETERANS BLVD. BRIDGE STA. 625+00 TO 626+60
MOWL (ft)
REACH
STATIONS
1 BRIDGE 2 3A 3B 4 5 6 7 8 9 10 11 12 13 14 BRIDGE 15A 15B BRIDGE 16 17 18
548+00 552+20 553+70 565+00 570+00 571+50 575+50 578+20 582+60 585+60 588+70 593+00 596+00 609+00 614+00 617+00 625+00 626+60 635+00 639+50 641+90 658+00 663+00 673+00
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.

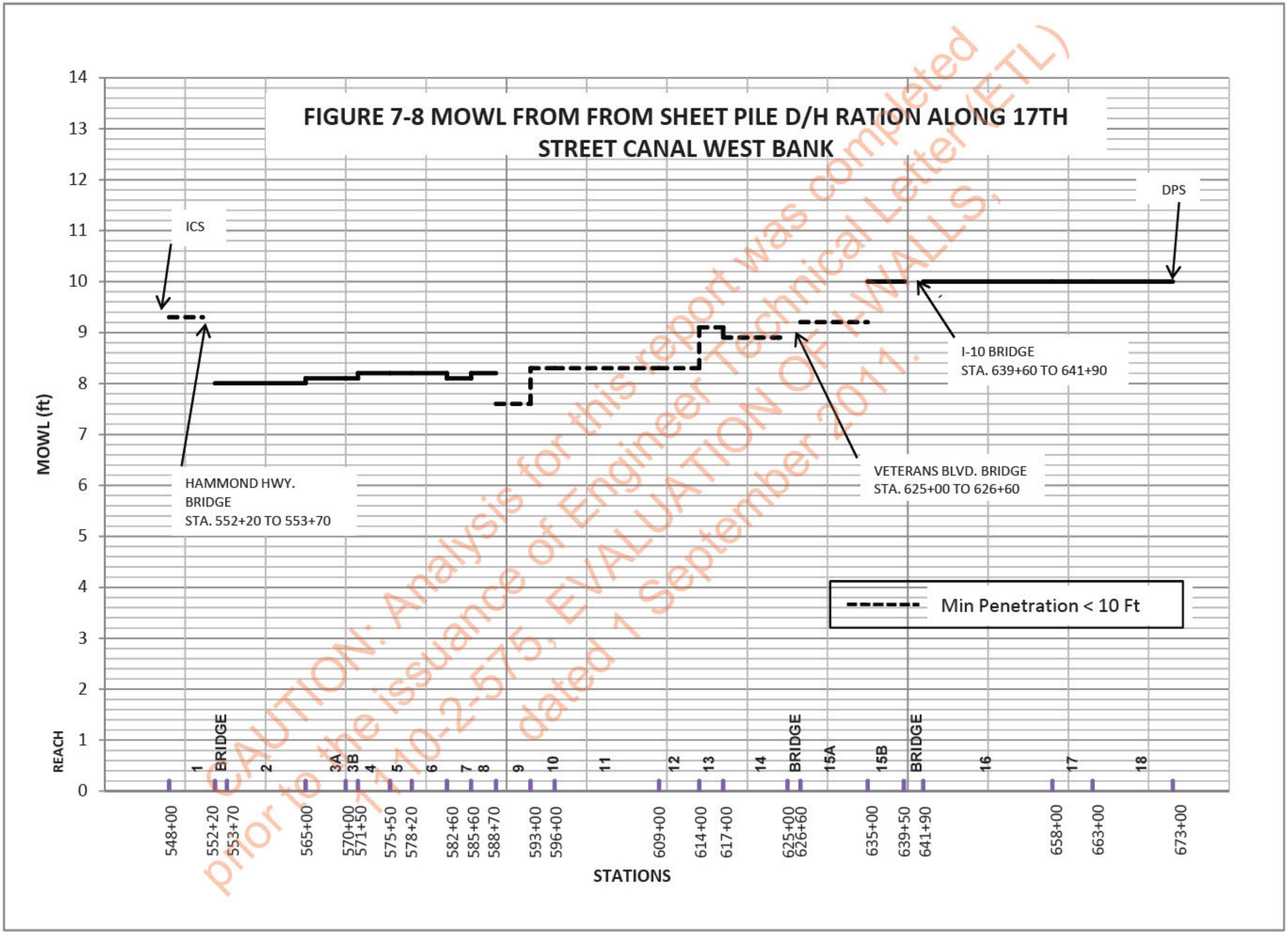

FIGURE 7-8 MOWL FROM FROM SHEET PILE D/H RATION ALONG 17TH STREET CANAL WEST BANK
MOWL (ft)
REACH
STATIONS
ICS
DPS
HAMMOND HWY. BRIDGE STA. 552+20 TO 553+70
VETERANS BLVD. BRIDGE STA. 625+00 TO 626+60
I-10 BRIDGE STA. 639+60 TO 641+90
Min Penetration < 10 Ft
1 BRIDGE 2 3A 3B 4 5 6 7 8 9 10 11 12 13 14 BRIDGE 15A 15B BRIDGE 16 17 18
548+00 552+20 553+70 565+00 570+00 571+50 575+50 578+20 582+60 585+60 588+70 593+00 596+00 609+00 614+00 617+00 625+00 626+60 635+00 639+50 641+90 658+00 663+00 673+00

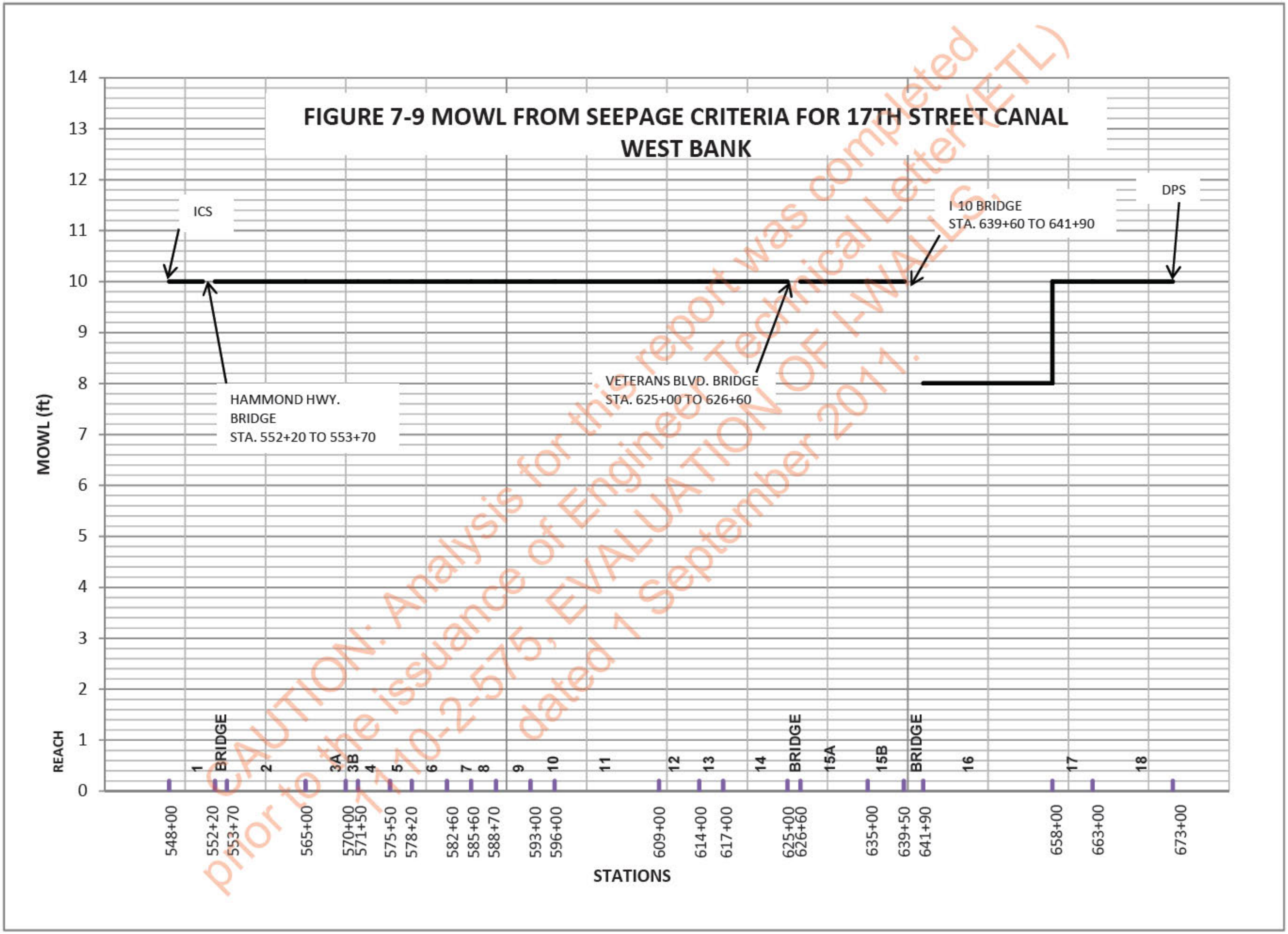

FIGURE 7-9 MOWL FROM SEEPAGE CRITERIA FOR 17TH STREET CANAL WEST BANK
MOWL (ft)
REACH
STATIONS
ICS
HAMMOND HWY. BRIDGE STA. 552+20 TO 553+70
VETERANS BLVD. BRIDGE STA. 625+00 TO 626+60
I 10 BRIDGE STA. 639+60 TO 641+90
DPS
1 BRIDGE 2 3A 3B 4 5 6 7 8 9 10 11 12 13 14 BRIDGE 15A 15B BRIDGE 16 17 18
548+00 552+20 553+70 565+00 570+00 571+50 575+50 578+20 582+60 585+60 588+70 593+00 596+00 609+00 614+00 617+00 625+00 626+60 635+00 639+50 641+90 658+00 663+00 673+00

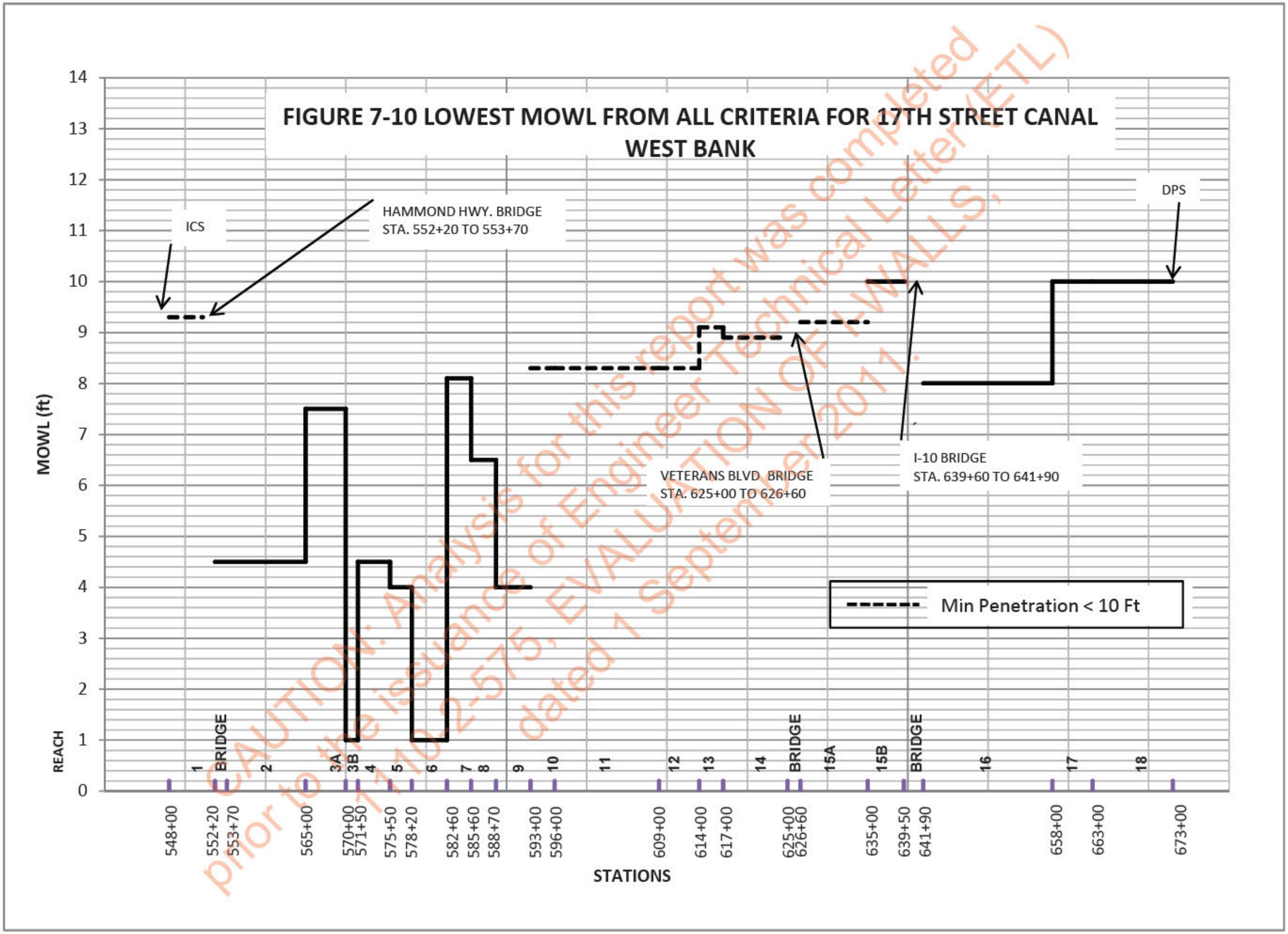

FIGURE 7-10 LOWEST MOWL FROM ALL CRITERIA FOR 17TH STREET CANAL WEST BANK
MOWL (ft)
ICS
HAMMOND HWY. BRIDGE
STA. 552+20 TO 553+70
DPS
VETERANS BLVD BRIDGE
STA. 625+00 TO 626+60
I-10 BRIDGE
STA. 639+60 TO 641+90
Min Penetration < 10 Ft
REACH
1 BRIDGE 2 3A 3B 4 5 6 7 8 9 10 11 12 13 14 BRIDGE 15A 15B BRIDGE 16 17 18
548+00 552+20 553+70 565+00 570+00 571+50 575+50 578+20 582+60 585+60 588+70 593+00 596+00 609+00 614+00 617+00 625+00 626+60 635+00 639+50 641+90 658+00 663+00 673+00
STATIONS
0 1 2 3 4 5 6 7 8 9 10 11 12 13 14

8.0 IMPACT TO CURRENT OPERATIONS

The analyses confirm that most problems along the 17th Street 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 6 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 experienced significantly higher hydraulic loading during Katrina than the current MOWL, with a canal water level of approximately El 8 to 9 NAVD88. The I-wall section that failed as a result of this loading was replaced. The remaining walls, including those in the reaches that are deficient based on the results of this study, 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. 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 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 6 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.

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