US Army Corps
Of Engineers

PHASE 2 CONCEPTUAL DESIGN SERVICES FOR PERMANENT PUMP STATIONS AND CANAL CLOSURES AT OUTFALLS

OPERATING SCENARIO ANALYSIS FINAL REPORT

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1.0 EXECUTIVE SUMMARY

The purpose of this Operating Scenario Analysis is to determine the potential range of operating conditions for both the existing drainage and proposed permanent pump stations on the 17th Street, Orleans Avenue, and London Avenue Outfall Canals. Hydrologic and hydraulic modeling was performed to determine the response of the canals to various combinations of Lake Pontchartrain elevations and canal discharges. The expected frequency of permanent pump and operational timing of pump start up were also investigated.

Of the three outfall canals, London Avenue Canal permanent flood gates and pump station are expected to be operated most frequently. London Avenue Canal has a proposed minimum safe water elevation of 8.0 ft NAVD88 (2004.65)[1] from approximately 4,000 ft downstream of drainage pump station (DPS) 3 north to Lake Pontchartrain. The remainder of London Avenue Canal, from 4,000 ft downstream of DPS 3 south to DPS 3, has a current safe water elevation of 9.0 ft. With flood gates open London Avenue Canal can pass the maximum design discharge of 8,980 cfs for Lake Pontchartrain elevations up to only 3.0 ft without exceeding these safe water elevations. The annual exceedance probability for this Lake Pontchartrain elevation is 95 percent, which corresponds to a 1.05-year return period.

The London Avenue Canal hydraulic model shows the water surface elevation for the maximum design discharge reaches safe water elevation 9.0 ft just downstream of DPS 3 before reaching safe water elevation 8.0 ft at a location 4,000 ft downstream of DPS 3. This is primarily due to increases in water surface elevations upstream of the bridges near DPS 3. The model shows the water surface elevations increase about 0.6 ft across Gentilly Boulevard Bridge and 0.4 ft across the I-610 Bridge. If bridge improvements are made or the safe water elevation downstream of DPS 3 increased, London Avenue Canal could pass the maximum design discharge for higher Lake Pontchartrain elevations.

Since Lake Pontchartrain is expected to reach 3.0 ft annually, consideration should be given to permanent operation of pump stations at the London Avenue Canal outfall. This would eliminate the need for flood gates and simplify permanent pump operations, although it would require an additional staffed pump station. The operating scenario analyses contained herein did not assume such a permanent closure.

Orleans Avenue Canal permanent flood gates and pump station are expected to be operated least frequently based on the current safe water elevation of 8.0 ft. With flood gates open, Orleans Avenue Canal can pass the maximum design discharge of 3,390 cfs for Lake Pontchartrain elevations up to 7.1 ft without exceeding safe water elevation. The annual exceedance probability for this Lake Pontchartrain elevation is 8 percent, which corresponds to a 12.5-year return period.

17th Street Canal permanent flood gates and pump station are expected to be operated less frequently than the London Avenue Canal flood gates and pump station based on a proposed minimum safe water elevation of 8.0 ft. With flood gates open, 17th Street Canal can pass the

[1] Throughout the report, the default vertical datum is NAVD88 epoch 2004.65.

maximum design discharge of 12,500 cfs for Lake Pontchartrain elevations up to 4.5 ft without exceeding safe water elevations. The annual exceedance probability for this Lake Pontchartrain elevation is 35 percent, which corresponds to approximately a 2.9-year return period.

1.1 Government Supplied Information

The following Government-supplied design criteria were used most frequently during the Operating Scenario Analysis. Additional information supplied by the Government is described in 3.0.

The current vertical reference system is NAVD88 (2004.65) and is the default datum used for the elevations reported in this report. The following datum conversions are to be used:

- NGVD29 – 0.5 ft = NAVD88 (2004.65)
- NAVD88 (1994, 1996) – 0.4 ft = NAVD88 (2004.65)
- Cairo Datum – 20.93 ft = NAVD88 (2004.65)

Safe Water Elevation in NAVD88 (2004.65):

- 17th Street Canal = 6.5 ft to 13.3[2] ft; 8.0[3] ft
- Orleans Avenue Canal = 8.0 ft
- London Avenue Canal = 5.0[4] ft; 8.0[5] ft; 9.0[6] ft

Design Discharge:

- 17th Street Canal = 12,500 cfs
- Orleans Avenue Canal = 3,390 cfs
- London Avenue Canal = 8,980 cfs

1.2 Operating Scenarios

The key decision parameters for permanent pump station operation are current and projected Lake Pontchartrain elevations and current and projected drainage pump station discharges.

As described in Section 4.3.2, it is recommended that the permanent flood gates on a particular canal be closed when Lake Pontchartrain reaches an elevation for which that canal cannot safely pass the maximum design discharge. Such operation would allow sufficient time for permanent flood gates to close and permanent pump station pumps to match any canal discharge being produced by the existing drainage pump stations (DPS). Based on expected Lake Pontchartrain elevations, the frequency of permanent flood gate closure is shown in Table 1.2-1.

[2] Current safe water elevations vary along length of 17th Street Canal
[3] Proposed minimum safe water elevation for 17th Street Canal
[4] Current safe water elevation from approximately 4,000 ft downstream of DPS 3 north to Lake Pontchartrain
[5] Proposed safe water elevation from approximately 4,000 ft downstream of DPS 3 north to Lake Pontchartrain
[6] Current safe water elevation from DPS 3 north to approximately 4,000 ft downstream of DPS 3

Table 1.2-1. Frequency of Permanent Pump Station Operation Based on Lake Pontchartrain Elevation

Canal	Lake Pontchartrain Gate Closure Elevation (ft NAVD88, 2004.65)	Permanent Pump Station Operation Frequency (Percent-Annual-Chance)
17th Street	4.5	35
Orleans Avenue	6.0	8
London Avenue	3.0	95

The bounding design conditions of (1) maximum canal discharge and associated Lake Pontchartrain elevation and (2) maximum Lake Pontchartrain elevation and associated canal discharge are described in Section 4.2 and are recommended to be the following.

- Maximum canal discharge with Lake Pontchartrain elevation 2.5 ft NAVD88 (2004.65)
- Maximum static Lake Pontchartrain elevation and 5-year canal discharge

Section 4.3.3 describes expected average annual operation of the permanent pump stations. Based on the above frequencies of permanent flood gate closures, the expected average annual permanent pump operation would be up to 7 hours, 4 hours, and 40 hours for 17th Street, Orleans Avenue and London Avenue Canals, respectively.

1.3 Pumped Discharge Frequency Analysis

The pumped flow-duration curves for the existing drainage pump stations are contained in Section 5.1 and are based on historic pump operating logs for DPS 3, DPS 4, DPS 6, and DPS 7. The flow-duration curves indicate that the existing drainage pump stations are operated between 1.5 and 3.7 percent of the time, or on average from 130 to 320 hours annually.

Historic annual peak flows obtained from the pumping logs were used to develop future conditions pumped discharge-frequency curves. Increases in the annual peak flows were estimated based on proposed drainage pump station capacity increases. The projected pumped discharge-frequency curves are contained in Section 5.2 and summarized in Table 1.3-1. Because the discharge in each outfall canal is limited by the drainage pump stations discharging to the canals, the estimated total pumped discharges shown in Table 1.3-1 are limited to the proposed future pumping capacities. Such a limitation is required only at London Avenue Canal for the least frequent (50- and 100-year) return periods.

Based on historic operation of the drainage pumping stations and the anticipated increases in drainage pump station capacities, there is a 10 percent annual chance (10-year return period) that 17th Street Canal would have pumped discharges exceeding 9,700 cfs. For the same return period, Orleans and London Avenue Canals would be expected to have pumped discharges exceeding 3,000 cfs and 8,000 cfs, respectively. As is described in Section 5.0, the magnitude of the discharges pumped to the canals does not necessarily correlate to a particular rainfall event.

Table 1.3-1. Estimated Pumped Discharge-Frequency for Proposed Future Conditions

Return Period of Pumped Discharge (Yr)	Percent Annual Chance Exceedance	17th Street Canal Pumped Discharge (cfs)	Orleans Avenue Canal Pumped Discharge (cfs)	London Avenue Canal Pumped Discharge (cfs)
1.01	99	4,550	890	2,150
2	50	7,640	2,390	5,710
5	20	8,950	2,830	7,160
10	10	9,710	3,020	8,010
20	5	10,390	3,150	8,780
50	2	11,230	3,270	8,980
100	1	11,840	3,330	8,980

The US Army Corps of Engineers Vicksburg District conducted hydrologic and hydraulic modeling using HEC-HMS and HEC-RAS models described in Sections 5.3 and 6.0. The modeling indicates that stormwater is not efficiently transported to the suction side of the existing drainage pump stations. Therefore, pumped discharge predicted for synthetic rainfall events is less than that shown above. In practice the Sewerage & Water Board of New Orleans operates the pumps to maximize use of the sump storage, which results in higher canal discharges than might be expected for a particular rainfall event.

1.4 Hydraulic Modeling

Water surface elevations for various combinations of Lake Pontchartrain elevations and canal discharges were modeled to determine what combinations would result in safe water elevation being exceeded. All three canals can pass the 99-percent-annual-chance canal discharge (approximately 1-year discharge) for Lake Pontchartrain elevations up to 7.6 ft NAVD88 (2004.65). The combination of variables yielding safe water elevations for each canal is fully described in Sections 4.1 and 6.1.

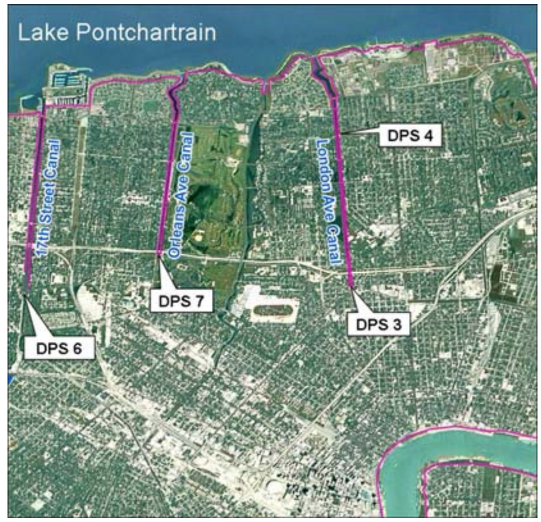
1.5 Additional Recommendations

Section 7.0 provides recommendations for the controls system and physical models. These are summarized here.

- The control system for the permanent pump station should be automated and built upon that installed for the Interim Closure Structure. Changes to the DPS manual control system could be investigated; however, the costs associated with automating the existing pump stations may be prohibitive.

- It is recommended that a physical hydraulic model study of each permanent pump station facility be performed during the preliminary design phase to assist in the development of cost effective designs that satisfy specified design and performance criteria.

2.0 SITE DESCRIPTION



Three outfall canals serve the central area of the City of New Orleans and a portion of Jefferson Parish. The three outfall canals are 17th Street, Orleans Avenue and London Avenue Canals. Each canal flows northward to Lake Pontchartrain, draining Orleans East Bank in Orleans Parish, and in the case of 17th Street Canal, some portion of the East Bank Drainage Basin of Jefferson Parish. Although the canals gravity drain to Lake Pontchartrain, the interior drainage areas they serve consist of a network of interior stormwater drainage canals and pump stations that convey water to major drainage pump stations at the upstream end of each canal. These pump stations lift drainage water to the outfall canals. The major drainage pump stations (DPS) are DPS 6 for 17th Street Canal, DPS 7 for Orleans Avenue Canal, and DPS 3 and DPS 4 for London Avenue Canal. The DPS and canals are owned and operated by the Sewerage and Water Board of New Orleans (S&WB). Canal Street Pump Station and the I-10 Pump Station both have smaller pumping capacities and discharge to 17th Street Canal. With the exception of the Canal Street Pump Station, which is owned and operated by the Jefferson Parish Department of Drainage, all pump stations that discharge to the three canals are owned and operated by S&WB. The following Sections provide an overview of each of the outfall canals and the pump stations that discharge to them.

2.1 17th Street Canal

This canal conveys drainage water from the western edge of Orleans Parish and the eastern edge of Jefferson Parish north to Lake Pontchartrain. Three pump stations discharge to 17th Street Canal: DPS 6, Canal Street Pump Station, and I-10 Pump Station. The 17th Street Canal was constructed during the late 1800s and early 1900s and has undergone canal improvements since its installation. The canal exceeds two miles in length and has earthen banks and bottom. It is currently lined with a combination of concrete flood walls and sheet pile flood walls. It has both railroad (near DPS 6) and automobile bridges (I-10, Veterans Memorial Boulevard, and Old Hammond Highway) that span its width.

2.1.1 Drainage Pump Station 6 (DPS 6)

DPS 6 is located at the head of the 17th Street Canal and lifts drainage water to discharge to the canal. The pumps are all electric motor driven, with some pumps receiving power from the Entergy lines and others from the dedicated 25 Hz S&WB power system. The station is manned full-time and has smaller pumps sized to operate for dry weather flows and larger pumps dedicated to the higher flows experienced during storm events. The dry weather flow pumps are piped to discharge to the Mississippi River. The equipment is housed in a brick building built in stages between 1897 and 1930, with a later addition in 1986-1989; this structure is also listed on the National Register of Historic Places. The following table lists the major equipment that constitutes this pump station.

Table 2.1-1. 17th Street Canal – DPS 6

Pump ID	Pump Capacity (CFS)	Pump/Driver Type	Power Supply (Hz)	Remarks
A	550	H/E	25	• 60Hz – power from Entergy with back-up dual feed, switched by Entergy • 25Hz – S&WB power with back-up feeders H = horizontal pump V = vertical pump C = centrifugal pump E = electric motor
B	550	H/E	25	
C	1,000	H/E	25	
D	1,000	H/E	25	
E	1,000	H/E	25	
F	1,000	H/E	25	
G	1,000	H/E	25	
H	1,100	H/E	60	
I	1,100	H/E	60	
V1	250	V/E	60	
V2	250	V/E	60	
V3	250	V/E	60	
V4	250	V/E	60	
CD1	90	C/E	25	
CD2	90	C/E	25	
Total	9,480			

2.1.2 Canal Street Pump Station

The Canal Street Pump Station is located in Jefferson Parish and pumps water from the Canal Street Canal to 17th Street Canal. The station consists of four 40 cfs vertical pumps with electric 60 Hz motors.

Table 2.1-2. 17th Street Canal – Canal Street Pump Station

Pump ID	Pump Capacity (CFS)	Pump/Driver Type	Power Supply (Hz)	Remarks
1	40	V/E	60	V = vertical pump E = electric motor
2	40	V/E	60	
3	40	V/E	60	
4	40	V/E	60	
Total	160			

2.1.3 I-10 Pump Station

The I-10 Pump Station was completed in 2004. It is an automated pump station that drains the I-10 railroad underpass and the general vicinity and discharges to 17th Street Canal.

Table 2.1-3. 17th Street Canal – I-10 Pump Station

Pump ID	Pump Capacity (CFS)	Pump/Driver Type	Power Supply (Hz)	Remarks
1	250	V/E	60	V = vertical pump C = centrifugal pump E = electric motor
2	250	V/E	60	
3	250	V/E	60	
CD1	100	C/E	60	
Total	850[7]			

2.2 Orleans Avenue Canal

This canal conveys drainage water from the central area of Orleans Parish to Lake Pontchartrain. Only DPS 7 discharges to the canal. It was constructed between 1897 and 1900 and has undergone canal improvements since its installation. The canal exceeds two miles in length and has earthen banks and bottom. It is currently lined with a combination of concrete flood walls and sheet pile flood walls. Five automobile bridges (I-610, Harrison Avenue, Filmore Avenue, Robert E. Lee Boulevard, and Lakeshore Drive) span its width.

2.2.1 Drainage Pump Station 7 (DPS 7)

DPS 7 is located at the head of the Orleans Avenue Canal and lifts drainage water to discharge to the canal. The pumps are all electric motor driven with some receiving power from the Entergy lines and others from the dedicated 25 Hz S&WB power system. The station is manned full-time and has smaller pumps sized to operate for dry weather flows and larger pumps dedicated to the higher flows experienced during storm events. The equipment is housed in a brick building built between 1897 and 1900; this structure is also listed on the National Register of Historic Places. The following table lists the major equipment that constitutes this pump station.

Table 2.2-1. Orleans Avenue Canal – DPS 7

Pump ID	Pump Capacity (CFS)	Pump/Driver Type	Power Supply (Hz)	Remarks
A	550	H/E	25	• 60Hz – power from Entergy without back-up • 25Hz – S&WB power with back-up feeders
C	1,000	H/E	25	
D	1,000	H/E	60	
CD1	70	V/E	25	H = horizontal pump V = vertical pump E = electric motor
CD2	70	V/E	25	
Total	2,690			

2.3 London Avenue Canal

Both DPS 3 and DPS 4 discharge to the London Avenue Canal, which conveys drainage water from the eastern edge of Orleans Parish to Lake Pontchartrain. The canal was constructed between 1901 and 1931 and has undergone canal improvements since its completion. It exceeds 2.5 miles in length and has earthen banks and bottom. It is currently lined with a combination of concrete flood walls and sheet pile flood walls. It has both railroad (one near DPS 3) and

[7] Pump capacities reported in IPET Volume VI Technical Appendix. Total capacity for analysis is 860 cfs.

automobile bridges (I-610, Gentilly Boulevard, Mirabeau Avenue, Filmore Avenue, Robert E. Lee Boulevard, Leon C. Simon Drive, and Lakeshore Drive) that span its width.

2.3.1 Drainage Pump Station 3 (DPS 3)

DPS 3 is located at the head of the London Avenue Canal and lifts drainage water to discharge to the canal. The pumps are all electric motor driven with some receiving power from the Entergy lines and others from the dedicated 25 Hz S&WB power system. The station is manned full-time and has smaller pumps sized to operate for dry weather flows and larger pumps dedicated to the higher flows experienced during storm events. The equipment is housed in a brick building built in three stages between 1901 and 1931; this structure is also listed on the National Register of Historic Places. The following table lists the major equipment that constitutes this pump station.

Table 2.3-1. London Avenue Canal – DPS 3

Pump ID	Pump Capacity (CFS)	Pump/Driver Type	Power Supply (Hz)	Remarks
A	550	H/E	25	• Pumps CD1 and CD2 each have 2 pumps. 1 motor (40 cfs each) • 25Hz – S&WB power with back-up feeders H = horizontal pump C = centrifugal pump E = electric motor
B	550	H/E	25	
C	1,000	H/E	25	
D	1,000	H/E	25	
E	1,000	H/E	25	
CD1L/1R	80	C/E	25	
CD2L/1R	80	C/E	25	
Total	4,260			

2.3.2 Drainage Pump Station 4 (DPS 4)

DPS 4 is located near the midpoint of the London Avenue Canal and lifts drainage water to discharge to the canal. The pumps are all electric motor driven with some receiving power from the Entergy lines and others from the dedicated 25 Hz S&WB power system. The station is manned full-time and has smaller pumps sized to operate for dry weather flows and larger pumps dedicated to the higher flows experienced during storm events. The equipment is housed in a brick building built in 1945 to 1946. The following table lists the major equipment that constitutes this pump station.

Table 2.3-2. London Avenue Canal – DPS 4

Pump ID	Pump Capacity (CFS)	Pump/Driver Type	Power Supply (Hz)	Remarks
1	320	C/E	60	• 60Hz – power from Entergy without back-up • 25Hz – S&WB power with back-up feeders H = horizontal pump V = vertical pump C = centrifugal pump E = electric motor
2	320	C/E	60	
C	1,000	H/E	25	
D	1,000	H/E	25	
E	1,000	H/E	25	
CD1	80	V/E	25	
Total	3,720			

3.0 GOVERNMENT SUPPLIED INFORMATION

3.1 Design Discharge

The maximum design discharges for the permanent pump stations are 12,500 cfs, 3,390 cfs, and 8,980 cfs for the 17th Street Canal, Orleans Avenue Canal, and London Avenue Canal, respectively. These are projected capacities that reflect S&WB and Jefferson Parish plans to increase capacity at DPS 6 and 7 and construct a new pump station along London Avenue Canal.

The DPS pumping capacities for each of the three canals are summarized with the proposed capacity increases below.

Table 3.1-1. Existing and Potential Drainage Pump Station Capacities

17th Street Canal Pump Station Capacities	
Existing DPS 6 capacity	9,480 cfs
Potential DPS 6 capacity increase	2,000 cfs
Canal Street Pump Station capacity	160 cfs
I-10 Pump Station capacity	860 cfs
Permanent Pumping Station required capacity	**12,500 cfs**
Orleans Avenue Canal Pump Station Capacities	
Existing DPS 7 capacity	2,690 cfs
Potential DPS 7 capacity increase	700 cfs
Permanent Pump Station required capacity	**3,390 cfs**
London Avenue Canal Pump Station Capacities	
Existing DPS 3 capacity	4,260 cfs
Existing DPS 4 capacity	3,720 cfs
Potential New Pump Station capacity	1,000 cfs
Permanent Pump Station required capacity	**8,980 cfs**

3.2 Safe Water Elevations

Safe water elevations are defined to equal maximum allowable canal water surface elevation at any point along the canal. The HPO directed that a proposed minimum safe water elevation of 8.0 ft NAVD88 (2004.65) be used to analyze the operating scenarios for all canals. The current and proposed safe water elevations are as follows:

- 17th Street Canal = 6.5 ft to 13.3[8] ft; 8.0[9] ft
- Orleans Avenue Canal = 8.0 ft
- London Avenue Canal = 5.0[10] ft; 8.0[11] ft; 9.0[12] ft

[8] See footnote 2 on page 1.
[9] See footnote 3 on page 2.
[10] See footnote 4 on page 1.
[11] See footnote 5 on page 2.
[12] See footnote 6 on page 2.

3.3 Lake Pontchartrain

3.3.1 Design Stage

The Lake Pontchartrain 1-percent-annual-chance-exceedance future conditions (2057) design elevation for all three canals is 16.0 ft NAVD88 (2004.65). The value to be added for structural superiority is 2.0 ft, for a total of 18.0 ft. Design stage information, including future conditions, was supplied by the US Army Corps of Engineers New Orleans District (MVN).

The design stage is based upon a storm surge analysis performed by the US Army Corps of Engineers Engineer Research and Development Center (ERDC). Approximately 150 hypothetical hurricanes were used in the Advanced Circulation Model (ADCIRC) analysis. MVN performed further analyses to account for sea level rise, subsidence, and the effect of waves. The 16.0 ft design elevation is calculated so that the overtopping rate from waves is less than 0.1 cfs/ft at the 90-percent assurance level and less than 0.03 cfs/ft at the 50-percents assurance level.

For existing conditions, Mean High Water (MHW) is 0.77 ft and mean range of tide is 0.51 ft. MVN used the USCG New Canal gage to determine these values, which are appropriate for all three canals.

For future conditions, MVN estimated sea level rise to be 1.3 ft per century, or 0.65 ft in 50 years (Intergovernmental Panel on Climate Change Third Assessment Report: Climate Change 2001). Therefore, MHW for future conditions (2057) is estimated to be 1.42 ft. The mean range of tide should not change.

For deterministic design processes, such as stability analysis, the design water surface elevation for future conditions (2057) is 11.2 ft. This water surface elevation does not include wave setup or runup. It is calculated from the future conditions still water level of 10.2 ft plus an additional increment to account for uncertainty.

3.3.2 Stage-Duration

Lake Pontchartrain stage gage 85625 is located at West End, Louisiana. The gage is attached to a pier adjacent to the Orleans Marina harbor master building at West End. Gage records are available from September 25, 1931 to November 30, 1946, and from March 9, 1949 to present.

MVN used 48 years of record (1958-2005) to produce a stage-duration curve for Lake Pontchartrain at West End. It appears that other record years were not used because of limited data availability prior to 1958. The stage-duration curve is used for the coincident frequency analysis described in Section 4.3.

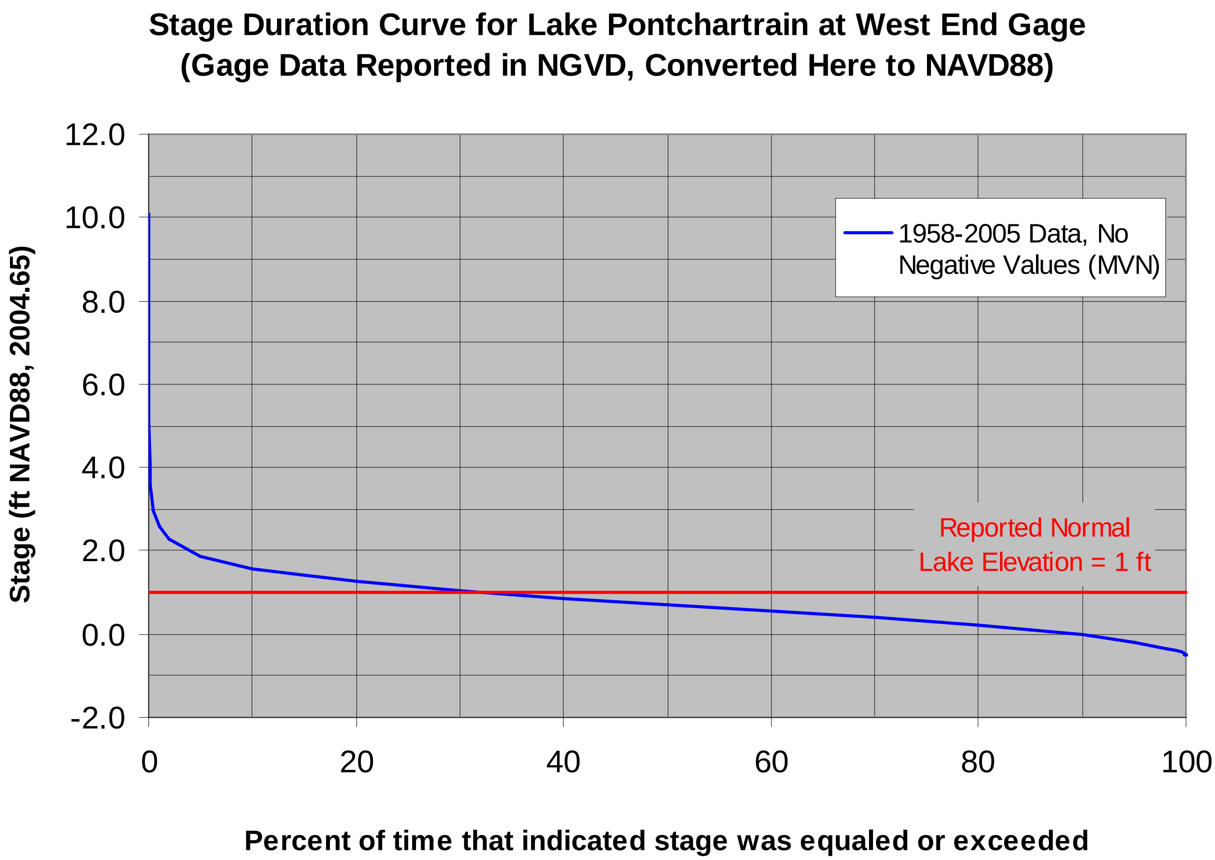


Figure 3.3-1. Lake Pontchartrain Stage-Duration Curve

The results of the stage-duration analysis indicate that the elevation of Lake Pontchartrain is less than 2.3 ft NAVD88 (2004.65) 98 percent of the time. Table 3.3-1 provides stage-duration computed results.

Normal lake elevation is generally accepted to be 1.0 ft NAVD88 (2004.65).

Table 3.3-1. Stage-Duration Curve for Lake Pontchartrain at West End, Louisiana

Percent Equaled or Exceeded	Lake Pontchartrain Elevation (ft NAVD88, 2004.65)
0.01	10.10
0.05	5.09
0.10	4.09
0.20	3.60
0.50	2.94
1.00	2.58
2.00	2.28
5.00	1.87
10.00	1.58
20.00	1.26
50.00	0.70
90.00	-0.01
95.00	-0.19
98.00	-0.34
99.00	-0.40
99.50	-0.44
99.99	-0.49
100.00	-0.50

3.3.3 Lake Stage Frequency

Lake Pontchartrain stage-frequency curves were provided by MVN based on both the ERDC storm surge modeling noted in Section 3.1 above and an analysis of the West End gage. Lake Pontchartrain elevations for selected return periods are shown in Table 3.3-2. For return periods below the 100-year, the elevations determined from the West End Gage analysis are used for the operating scenario analyses in this report, while the ADCIRC results are used for the 100- and 500-year return periods.

Curves generated from the ADCIRC analysis show expected Lake Pontchartrain elevations for the 20-year to 500-year return periods. Three curves, corresponding to locations near the outfalls of 17th Street, Orleans Avenue, and London Avenue Canals, are shown in Figure 3.3-2 through Figure 3.3-4.

The West End gage stage-frequency curve is shown in Figure 3.3-5. The elevations shown on the stage-frequency curves reflect still water levels and include surge, tide, and MHW.

Table 3.3-2. Lake Pontchartrain Stage-Frequency Data

Return Period (Yr)	Percent Annual Chance Exceedance	Lake Pontchartrain Elevation (ft NAVD88, 2004.65)
1.05	95	3.0
2	50	3.9
5	20	5.1
10	10	5.7
20	5	6.8
50	2	8.1
100	1	8.7
500	0.2	11.4

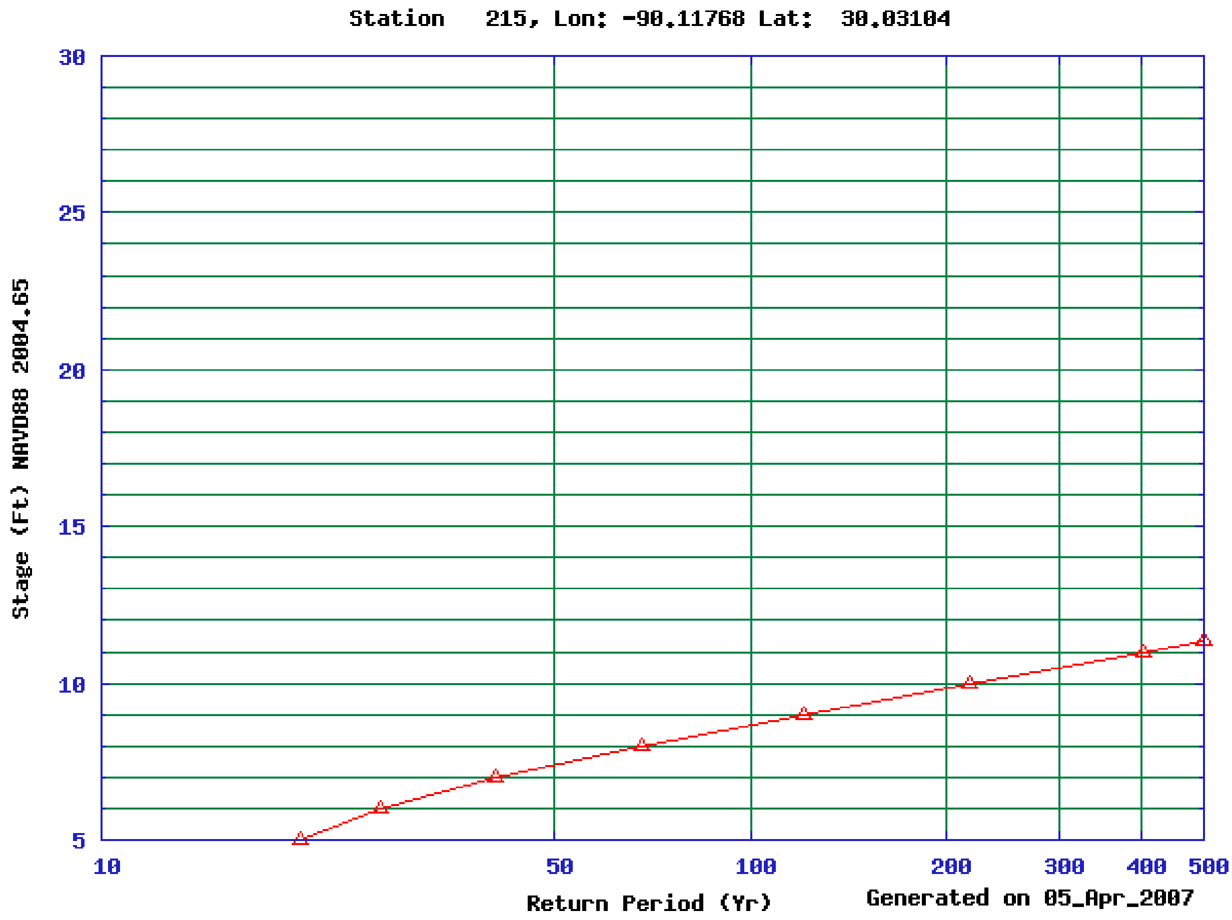


Figure 3.3-2. 17th Street Canal Outfall: Lake Pontchartrain Stage-Frequency Curve Generated from ADCIRC (Advanced Circulation Model)

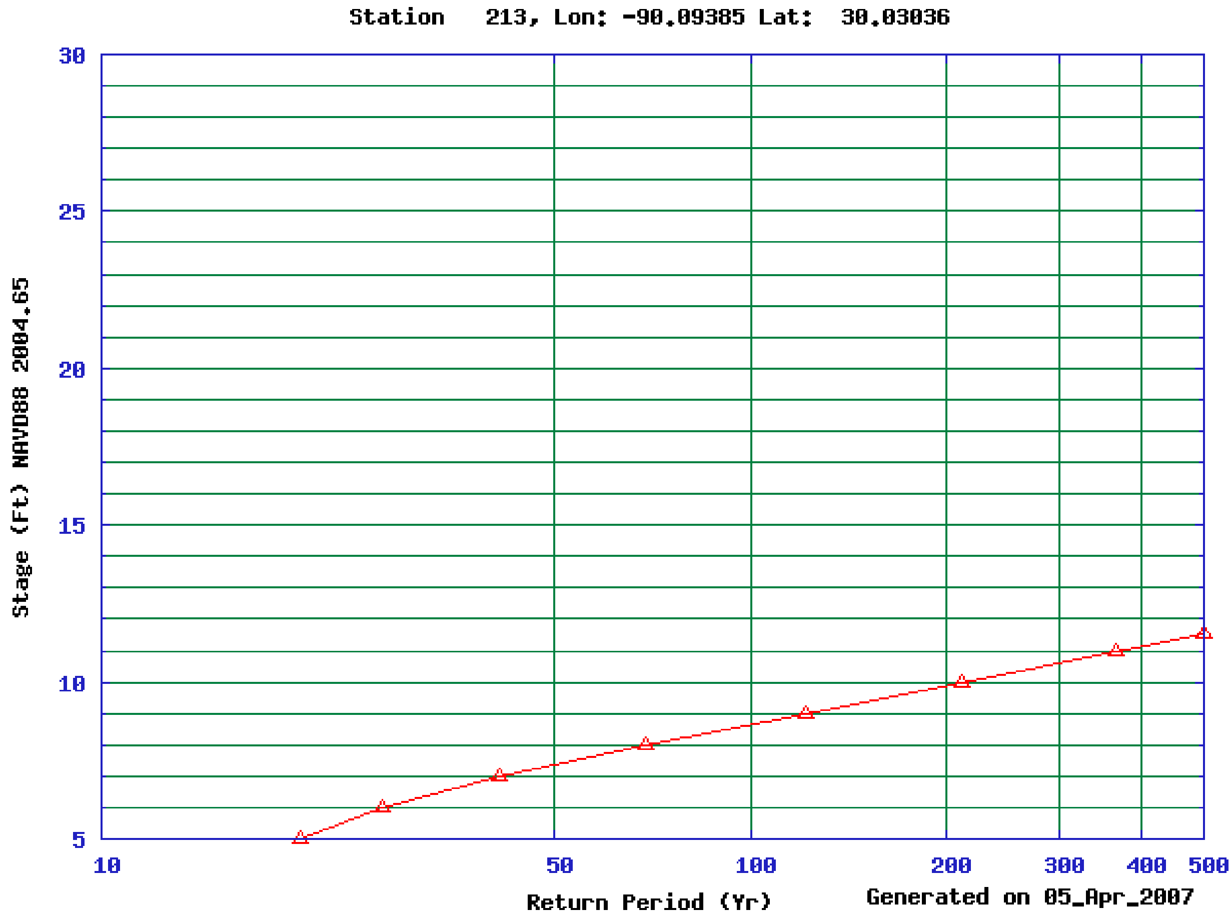


Figure 3.3-3. Orleans Avenue Canal Outfall: Lake Pontchartrain Stage-Frequency Curve Generated from ADCIRC (Advanced Circulation Model)

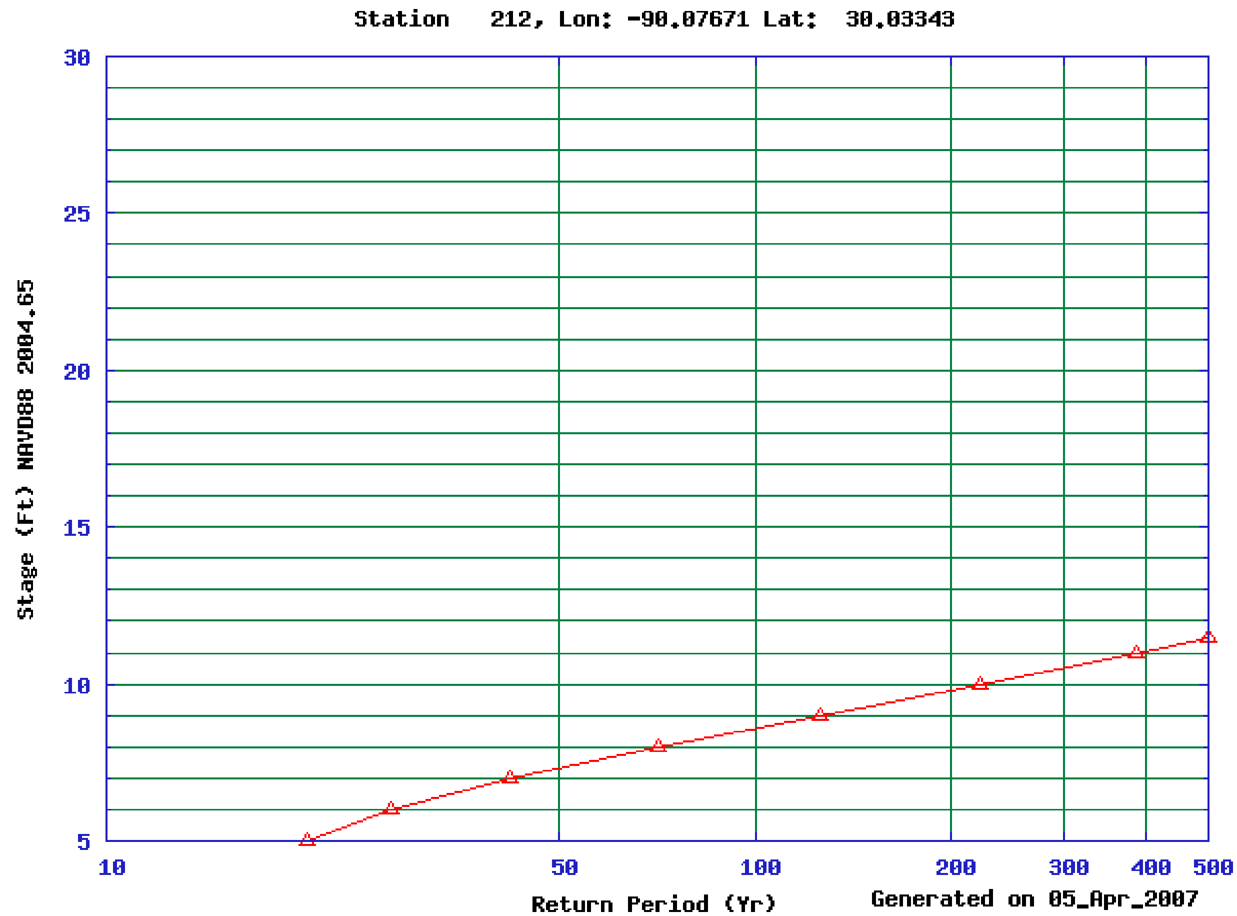


Figure 3.3-4. London Avenue Canal Outfall: Lake Pontchartrain Stage-Frequency Curve Generated from ADCIRC (Advanced Circulation Model)

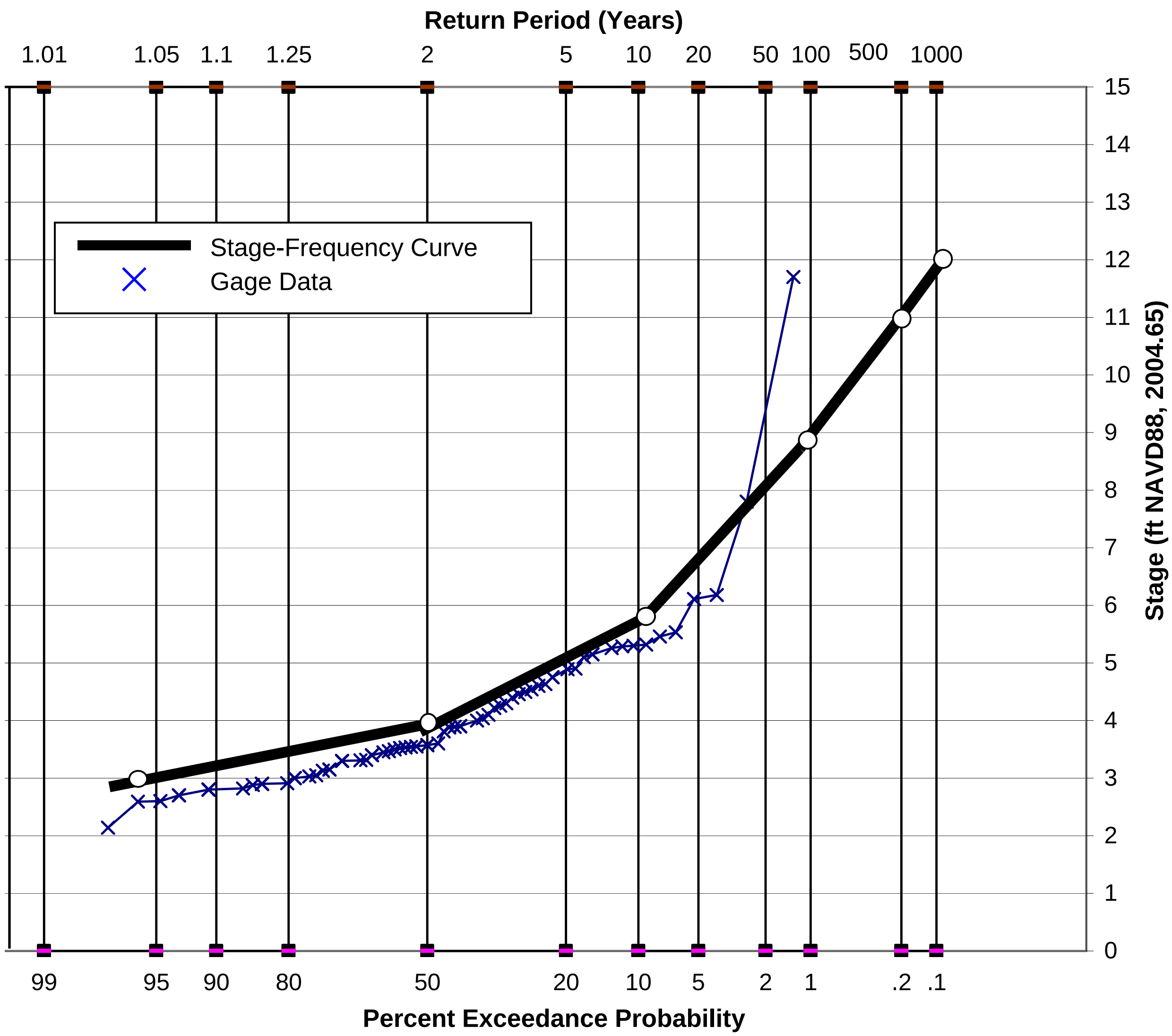


Figure 3.3-5. Lake Pontchartrain West End, Louisiana, Gage Stage-Frequency Curve

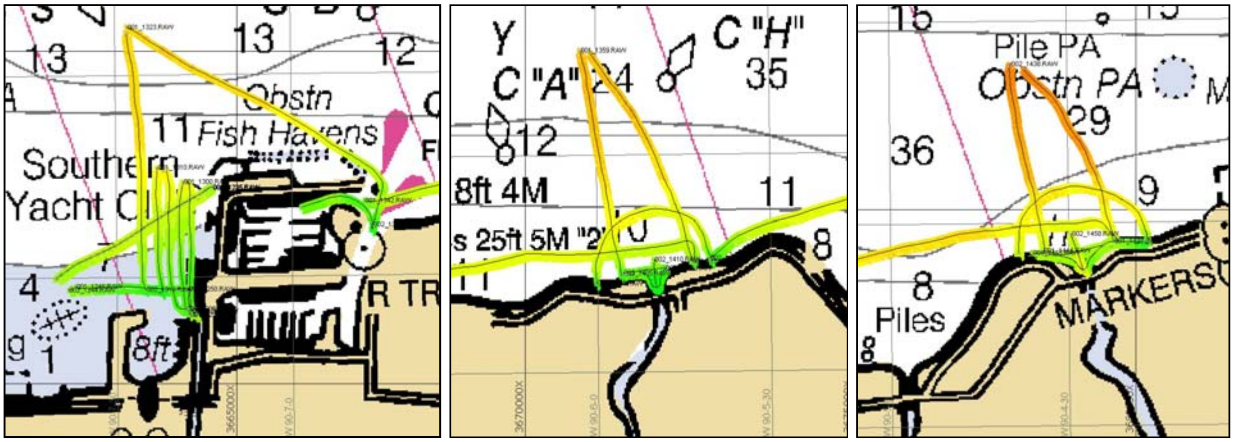
3.3.4 Wave and Wind Boundary Condition

To date detailed wave climatology data (design wave heights, periods, frequencies, and directions) have not been provided for Lake Pontchartrain. Discussions with ERDC indicate the storm surge analysis performed for 152 hypothetical hurricanes could provide the information, although the surge analysis was not designed to produce detailed wave climatology.

A 1:50-scale physical model of the northern portion of the 17th Street Canal was developed at ERDC to quantify the effect the Hammond Highway Bridge had on wave transmission and flow toward the floodwall breach. Modelers found wave heights near the lakeside of the bridge were 1 to 3 ft in height, reduced from the 6 to 9 ft wave heights in the open lake. Waves on the south side of the bridge were further reduced to heights below 1 ft. A full report of the physical modeling development and analysis is contained as a technical appendix to the Interagency Performance Evaluation Task Force (IPET) final report, Volume IV – The Storm.

3.3.5 Bathymetry

In January 2006 bathymetric surveys at the outfalls of 17th Street, London Avenue, and Orleans Avenue Canals were taken. Data were collected at approximately 5-ft intervals along the tracks shown in Figure 3.3-6. Data in xyz format are available in NAVD88 (2004.65).



(a) **(b)** **(c)**

Figure 3.3-6. January 2006 Bathymetric Survey Tracks for (a) 17th Street Canal, (b) Orleans Avenue Canal, (c) London Avenue Canal.

3.4 Canal Geometry

In March 2006 multi-beam surveys of the entire lengths of the 17th Street, Orleans Avenue, and London Avenue Canals were conducted. Surveys were performed using vertical datum NAVD88 (2004.65). Multi-beam surveys from each Interim Closure Structure (ICS) north to Lake Pontchartrain were also conducted. The data were provided to the Vicksburg District Corps of Engineers (MVK) for inclusion in the hydraulic models of the canals.

Bridge surveys were conducted on 17th Street Canal and Orleans Avenue Canal bridges in December 2006 and January 2007. The survey data were provided to MVK for inclusion in the hydraulic models of the canals.

Additional London Avenue Canal geometry data provided to MVK included February 2007 cross section surveys taken at three sites: (1) south of Robert E. Lee Boulevard, (2) south of Filmore Avenue and (2) south of Mirabeau Avenue. Cross section data from an August 2007 bank erosion survey along 17th Street Canal were also provided.

3.5 DPS Pump Operation

The US Army Corps of Engineers Hurricane Protection Office (HPO) developed DPS discharge-duration curves based on New Orleans Sewerage and Water Board (S&WB) pump operating logs at each of the existing drainage pump stations. For 17th Street and Orleans Avenue Canals, 23 years and 22 years of pump operating logs for DPS 6 and DPS 7, respectively, were analyzed from the period 1980 to 2006. For London Avenue Canal, 26 years of pump operating logs for DPS 3 and 13 years of logs for DPS 4 were analyzed. The pumped discharge-duration curves are presented in Section 5.1.

The following notes were provided by HPO to caution data users of the assumptions made during flow-duration curve development. Pump discharges were manually calculated based on handwritten records contained in the pump operating logs.

- Photos of the pump logs were taken at DPS 3, 4, 6 and 7 as well as the storage area at the S&WB Claiborne Office.
- Only logs showing drainage pumps running were collected. Constant-duty pumps were ignored because most pump to other locations, and S&WB personnel said these pumps were not used during rain events. Additionally, the amount of flow generated by the constant-duty pumps is minimal compared to pump station capacity.
- DPS 4 records had incomplete years where about half of the log sheets were missing. The logs that were missing were scattered throughout the year. Everything that was available was collected. The years this affected are 1980, 1981, 1982, 1987, and 1988. For 1990 the only dates missing were May 12, 13, 26 and 27.
- DPS 6 had issues with the discharge-side gage for several years. The logs indicate that the gage was periodically not working. In order to use the data for these times, a lake elevation of 23.0 ft (Cairo) was assumed for the discharge side of the station. This was done for all of 1986, 1987, 1988 and portions of 1984, 1985, 1989 and 1990.
- Throughout data entry were cases where half-hour values had to be estimated for the suction basin and the lake elevation because they had not been recorded on the log sheets. For the suction elevation, an average between the hour values was taken to get the half-hour value between them. For the lake elevation, the previous hour value was used.
- Illegible or conflicting entries in the logs were verified in the "remarks" column on the logs. If the pump was on for at least 20 minutes of the half-hour, it was considered on during that half-hour. Also, if the date on the log was not legible or incomplete, the date between the previous and next log was used.

In addition to discharge-duration curves, the HPO provided annual peak flows, or peak discharges, and discharge-frequency curves for each of the existing drainage pump stations. The peak flows for each of the existing drainage pump stations are shown in Table 3.5-1. Discharge-frequency curves were generated from the peak flows using HEC-SSP, Statistical Software Package, beta version1.0. The output curves and tabulated results from HEC-SSP are shown in Figure 3.5-1 through Figure 3.5-4 and Table 3.5-2 through Table 3.5-5.

Table 3.5-1. Existing Drainage Pump Station Annual Peak Flows (1979 to 2006)

	17th Street Canal	Orleans Avenue Canal	London Avenue Canal	
Year	DPS 6 Peak Flow (cfs)	DPS 7 Peak Flow (cfs)	DPS 3 Peak Flow (cfs)	DPS 4 Peak Flow (cfs)
1979	Insufficient Data	No Data	2,670	No Data
1980	Insufficient Data	2,030	4,090	3,340
1981	Insufficient Data	2,660	2,570	2,530
1982	Insufficient Data	2,700	3,480	3,420
1983	Insufficient Data	2,680	3,670	2,600
1984	3,940	1,350	2,690	1,140
1985	6,640	2,520	3,200	2,630
1986	5,620	2,680	3,140	1,680
1987	5,430	2,610	3,240	2,340
1988	6,950	2,620	3,240	3,320
1989	6,870	1,480	3,380	4,090
1990	6,800	1,380	3,730	3,690
1991	7,760	2,500	3,300	Insufficient Data
1992	7,820	Insufficient Data	3,520	Insufficient Data
1993	5,720	1,490	Insufficient Data	2,790
1994	6,900	Insufficient Data	Insufficient Data	3,330
1995	8,040	2,900	3,950	Insufficient Data
1996	6,540	1,670	3,490	Insufficient Data
1997	5,200	2,330	3,510	Insufficient Data
1998	8,510	2,710	3,710	Insufficient Data
1999	7,310	2,710	2,410	Insufficient Data
2000	5,000	2,550	2,160	Insufficient Data
2001	8,600	2,250	3,440	3,570
2002	7,310	2,580	3,290	2,820
2003	8,430	2,900	3,480	3,920
2004	9,960	Insufficient Data	3,170	2,730
2005	7,550	2,590	3,080	3,670
2006	3,940	Insufficient Data	2,420	2,030

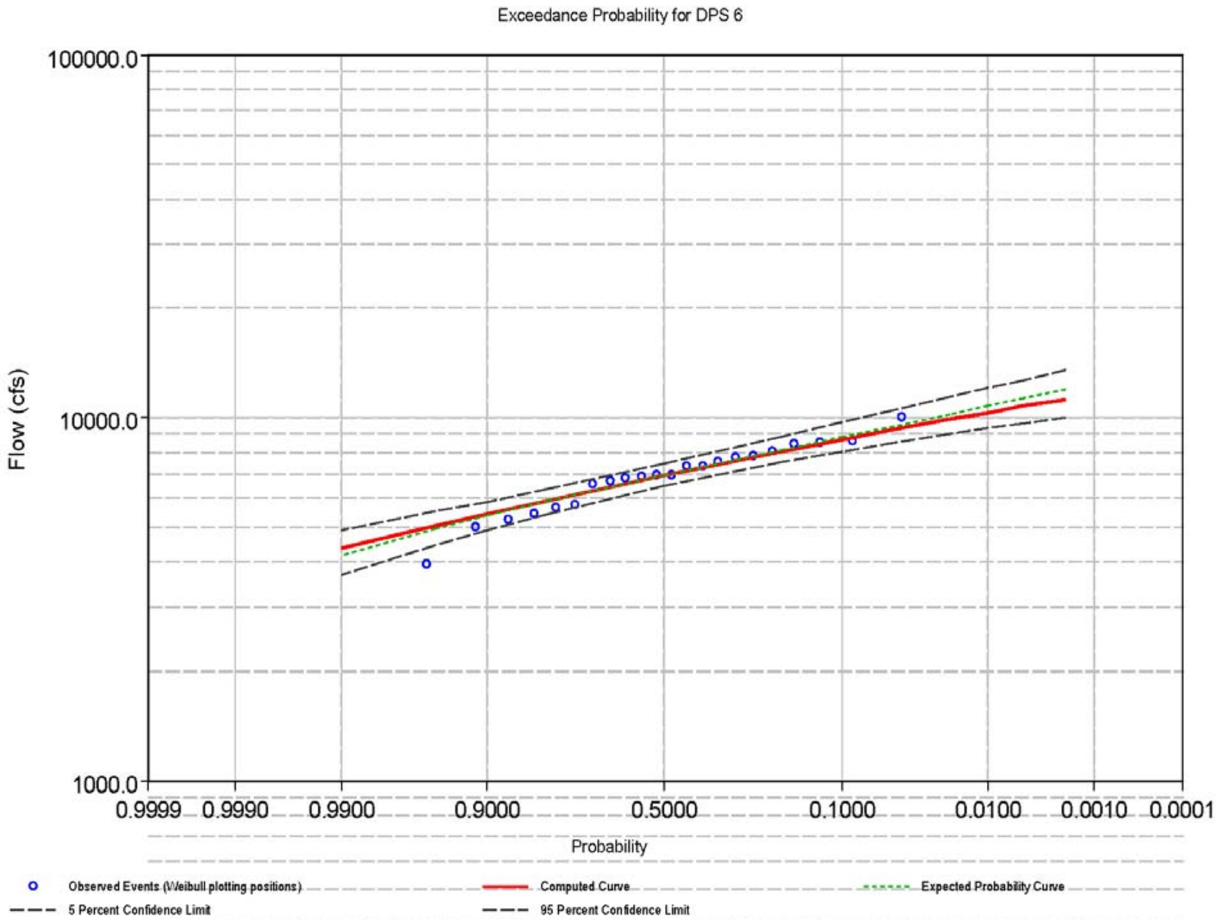


Figure 3.5-1. 17th Street Canal: DPS 6 Discharge Exceedance Probability Curve (HEC-SSP)

Table 3.5-2. 17th Street Canal: DPS 6 Tabulated Discharge Exceedance Probability

Percent Chance Exceedance	Computed Curve Flow (cfs)	Expected Probability Flow (cfs)	Confidence Limits	
			0.05	0.95
0.2	11,241	11,970	13,480	10,020
0.5	10,737	11,270	12,690	9,650
1	10,329	10,730	12,070	9,340
2	9,893	10,190	11,410	9,010
5	9,259	9,430	10,480	8,520
10	8,715	8,820	9,710	8,080
20	8,082	8,130	8,840	7,550
50	6,951	6,950	7,440	6,500
80	5,926	5,880	6,350	5,420
90	5,433	5,360	5,870	4,870
95	5,047	4,930	5,500	4,430
99	4,376	4,150	4,880	3,680

Station skew used for computed curve = -0.2.

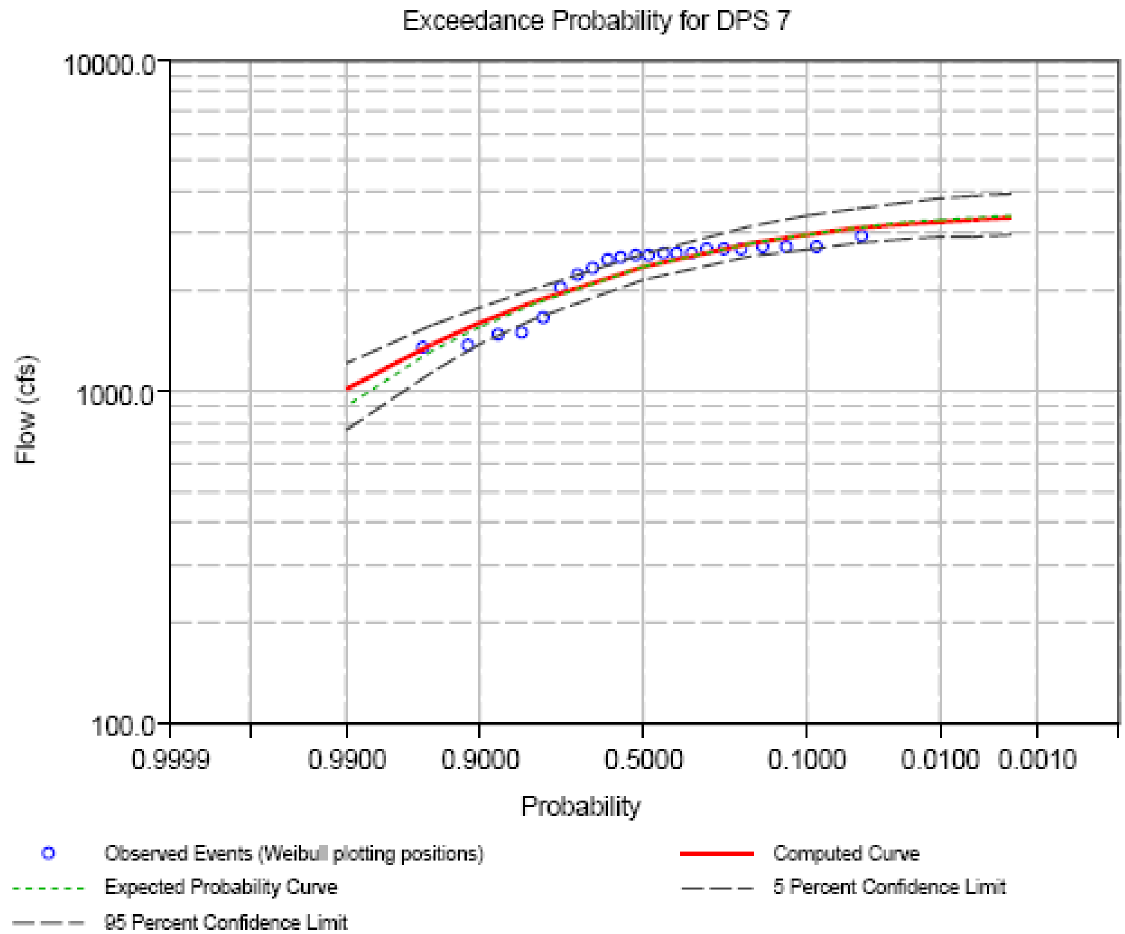


Figure 3.5-2. Orleans Avenue Canal: DPS 7 Discharge Exceedance Probability Curve (HEC-SSP)

Table 3.5-3. Orleans Avenue Canal: DPS 7 Tabulated Discharge Exceedance Probability

Percent Chance Exceedance	Computed Curve Flow (cfs)	Expected Probability Flow (cfs)	Confidence Limits	
			0.05	0.95
0.2	3,329	3,370	3,930	2,970
0.5	3,288	3,330	3,870	2,940
1	3,246	3,290	3,810	2,910
2	3,189	3,230	3,720	2,870
5	3,081	3,110	3,570	2,780
10	2,962	2,990	3,400	2,680
20	2,787	2,800	3,150	2,540
50	2,368	2,370	2,610	2,170
80	1,875	1,850	2,050	1,670
90	1,609	1,560	1,790	1,380
95	1,394	1,330	1,580	1,150
99	1,021	900	1,220	770

Station skew used for computed curve = -1.2.

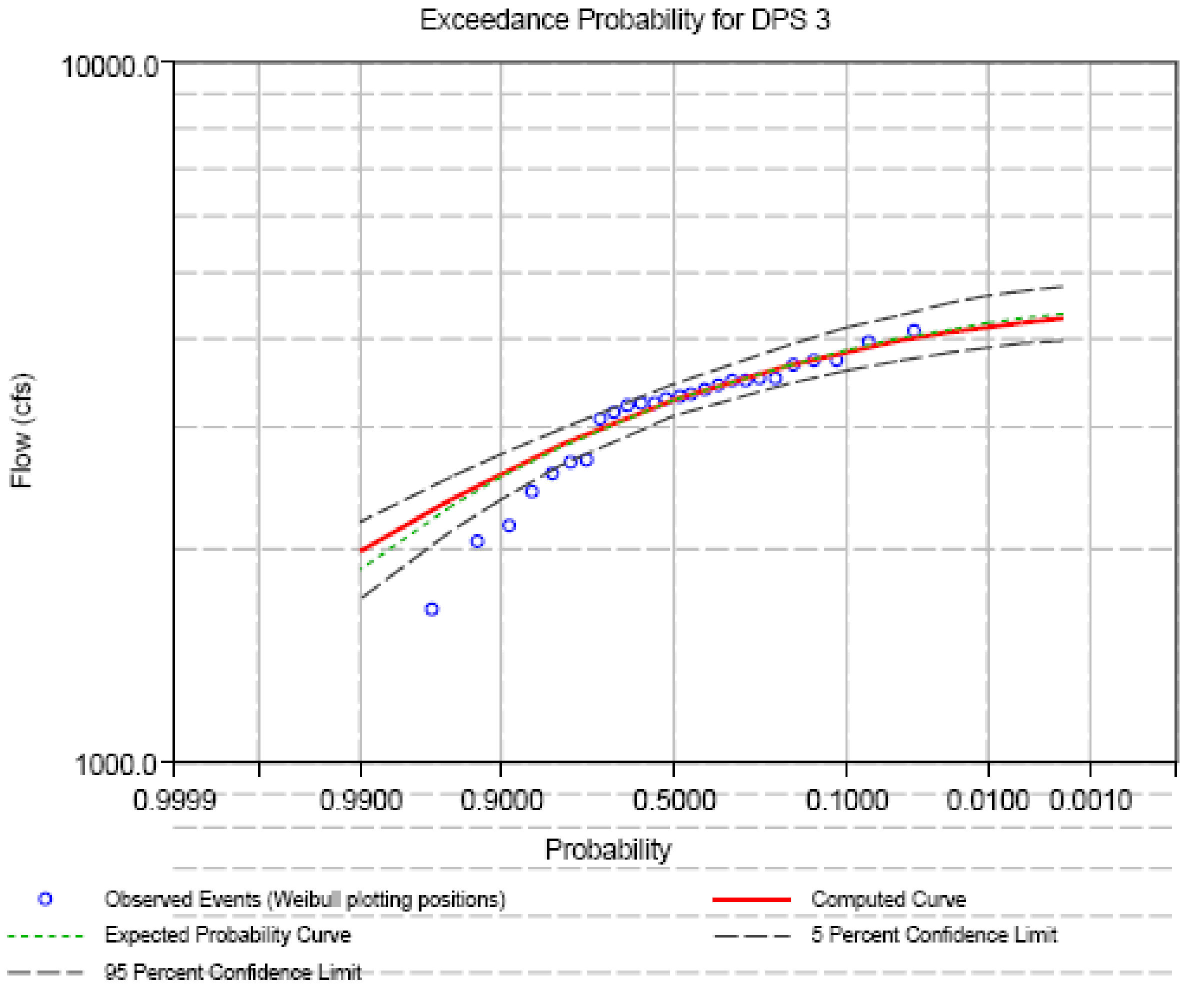


Figure 3.5-3. London Avenue Canal: DPS 3 Discharge Exceedance Probability Curve (HEC-SSP)

Table 3.5-4. London Avenue Canal: DPS 3 Tabulated Discharge Exceedance Probability

Percent Chance Exceedance	Computed Curve Flow (cfs)	Expected Probability Flow (cfs)	Confidence Limits	
			0.05	0.95
0.2	4,295	4,360	4,780	3,990
0.5	4,231	4,290	4,690	3,940
1	4,171	4,220	4,610	3,890
2	4,097	4,140	4,510	3,830
5	3,970	4,000	4,340	3,730
10	3,841	3,860	4,160	3,620
20	3,664	3,680	3,930	3,470
50	3,271	3,270	3,460	3,100
80	2,822	2,800	2,980	2,640
90	2,575	2,540	2,740	2,360
95	2,369	2,310	2,550	2,130
99	1,985	1,870	2,200	1,700

Station skew used for computed curve = -0.9.

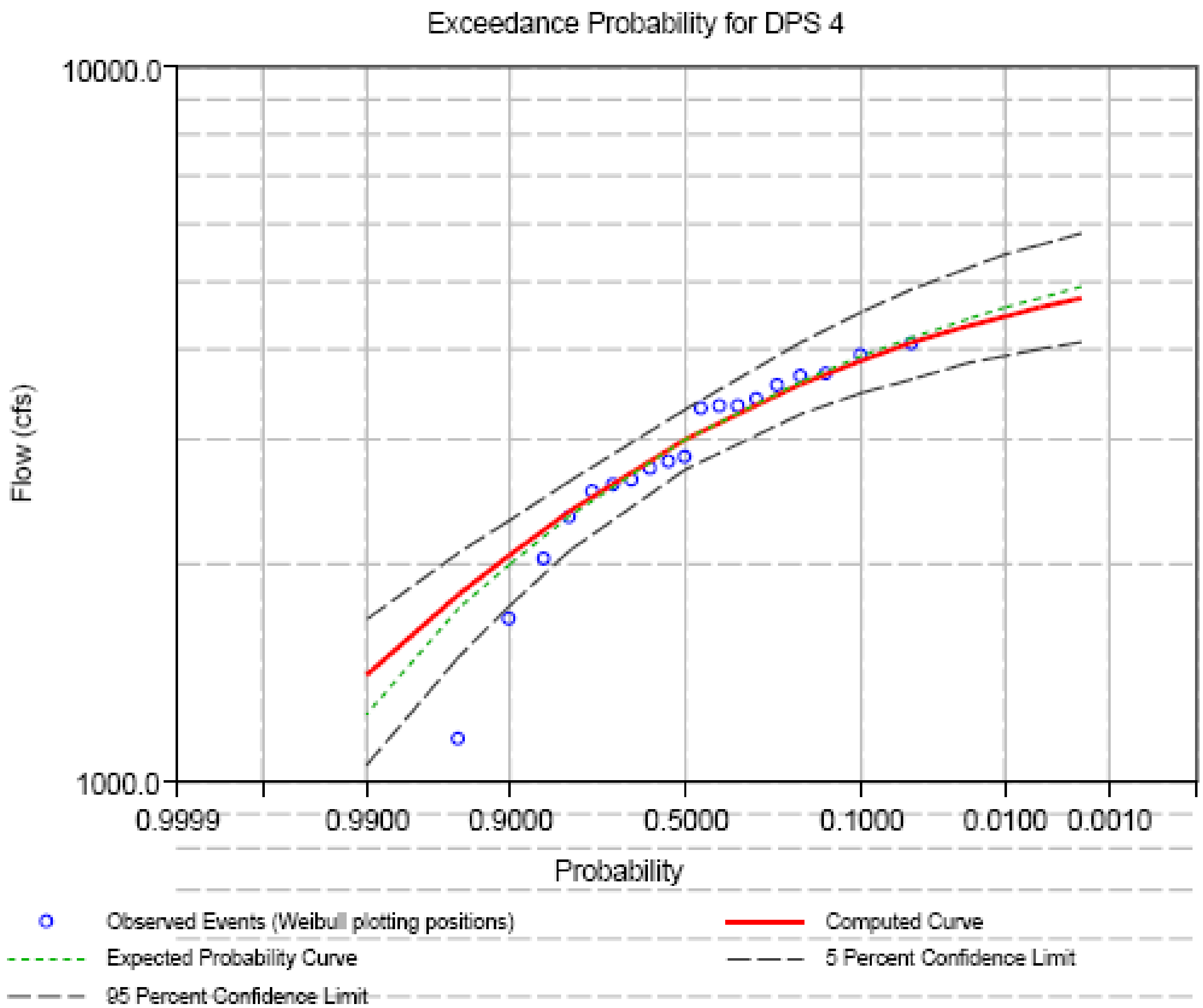


Figure 3.5-4. London Avenue Canal: DPS 4 Discharge Exceedance Probability Curve (HEC-SSP)

Table 3.5-5. London Avenue Canal: DPS 4 Tabulated Discharge Exceedance Probability

Percent Chance Exceedance	Computed Curve Flow (cfs)	Expected Probability Flow (cfs)	Confidence Limits	
			0.05	0.95
0.2	4,713	4,900	5,840	4,120
0.5	4,583	4,740	5,630	4,020
1	4,466	4,600	5,440	3,940
2	4,326	4,440	5,220	3,830
5	4,097	4,170	4,860	3,660
10	3,875	3,930	4,520	3,480
20	3,586	3,610	4,100	3,250
50	2,992	2,990	3,310	2,720
80	2,382	2,350	2,630	2,090
90	2,072	2,010	2,320	1,750
95	1,827	1,740	2,080	1,490
99	1,405	1,240	1,680	1,050

Station skew used for computed curve = -0.8.

4.0 OPERATING SCENARIOS

4.1 Combination of Variables Resulting in Safe Water Elevations

Canal water surface elevations are influenced by Lake Pontchartrain elevations and discharges to the canal from existing drainage pump stations. Based on the hydraulic modeling described in Section 6.0, combinations of Lake Pontchartrain elevations and maximum canal discharges that result in the canal water surface reaching safe water elevation are reported in Table 4.1-1 through Table 4.1-3. Additional combinations of lake elevations and canal discharges that yield canal water surface elevations below safe water elevations can be interpolated from Figure 6.1-1 through Figure 6.1-4.

As indicated in Section 3.2, the target minimum safe water elevation for all canals is 8.0 ft NAVD88 (2004.65).

Table 4.1-1. 17th Street Canal Combination of Variables Yielding Safe Water Elevation

Lake Pontchartrain Elevation (ft NAVD88, 2004.65)	Canal Discharge (cfs)
0.0	>12,500
1.0	>12,500
2.0	>12,500
3.0	>12,500
4.0	>12,500
5.0	11,820
6.0	10,130
7.0	7,220
8.0	0

Table 4.1-2. Orleans Avenue Canal Combination of Variables Yielding Safe Water Elevation

Lake Pontchartrain Elevation (ft NAVD88, 2004.65)	Canal Discharge (cfs)
0.0	>3,390
1.0	>3,390
2.0	>3,390
3.0	>3,390
4.0	>3,390
5.0	>3,390
6.0	>3,390
7.0	>3,390
8.0	0

Table 4.1-3. London Avenue Canal Combination of Variables Yielding Safe Water Elevation

Lake Pontchartrain Elevation (ft NAVD88, 2004.65)	Canal Discharge (cfs)
0.0	>8,980
1.0	>8,980
2.0	>8,980
3.0	8,980
4.0	8,480
5.0	7,820
6.0	6,960
7.0	5,830
8.0	0

4.2 Concurrent Combination of Flow Rates and Lake Stage

For each canal the following analysis addresses the bounding design conditions of (1) maximum canal discharge and associated Lake Pontchartrain elevation and (2) maximum Lake Pontchartrain elevation and associated canal discharge.

Based on nearly 50 years of Lake Pontchartrain West End Gage data and rainfall data at New Orleans Airport and Audubon gages, an analysis of rainfall events corresponding to Lake Pontchartrain stages was performed. Lake elevation and precipitation are not well-correlated. This lack of correlation is primarily attributable to precipitation being quite localized for a relatively small interior area, while Lake Pontchartrain stages are influenced by the Gulf of Mexico. Even so, general trends based on ranges of total daily rainfall versus lake elevations were observed. Because the New Orleans Audubon rain gage is closer to the area served by the three outfall canals, the results from its analysis were given greater weight.

4.2.1 Event Combination (1): Max Design Discharge with Lake Elevation 2.5 ft

For the highest recorded daily rainfall events, Lake Pontchartrain elevations ranged from 1 to 4 ft NAVD88 (2004.65), and were most often near 2 ft. West End historical gage data is reported once daily at 8:00 AM, so tidal fluctuation is averaged into the data. However, a conservative approach includes tide, and tidal fluctuation is approximately 0.5 ft. The first concurrent event combination recommended is a lake elevation of 2.5 ft NAVD88 (2004.65) combined with the maximum design discharge for each canal.

For this event combination, all three canals can pass the maximum design discharge, and the gates would remain open. Table 4.2-1 shows the canal water surface elevation for this event combination.

Table 4.2-1. Concurrent Event Combination (1): Maximum Design Discharge with Lake Pontchartrain Elevation 2.5 ft NAVD88 (2004.65) – Includes Tide

Canal Location	Total Canal Discharge (cfs)	Water Surface Elevation (ft NAVD88, 2004.65)
17th Street just downstream of DPS 6	12,500	7.1
Orleans Avenue just downstream of DPS 7	3,390	4.9
London Avenue just downstream of DPS 3	8,980	8.8
London Avenue 4,000 ft downstream of DPS 3	8,980	7.0

4.2.2 Event Combination (2): Max Design Lake Stage and 5-Year Canal Discharge

In general, for the highest recorded Lake Pontchartrain elevations, daily rainfall ranged from 0 to 3 inches, with an increasing percentage of rainfall events between 2 and 3 inches for higher lake stages. A minimal number of daily rainfall totals were between 7 and 8 inches for the highest lake stages. The 5-year, TP-40 24-hour rainfall for the area is about 7.4 inches. The second concurrent event combination recommended is Lake Pontchartrain maximum design stage with pumped canal discharges corresponding to a 5-year return period.

For this event combination the gates would be closed for all canals. Table 4.2-2 shows the canal discharge and the corresponding permanent pump station suction side elevation for the second concurrent event combination when operating at safe water elevation. The 5-year, or 20-percent-annual-chance exceedance, canal discharge is based on a frequency analysis of the expected future discharges. The future conditions canal discharge-frequency analysis is described in Section 5.2.

Table 4.2-2. Concurrent Event Combination (2): Maximum Static Lake Pontchartrain Elevation= 11.2[13] ft NAVD88 (2004.65) and 5-Year Canal Discharge

Canal	Discharge (cfs)	Canal Water Surface Elevation at Suction Side - Permanent Pump Station (ft NAVD88, 2004.65)
17th Street	8,950	6.5
Orleans Avenue	2,830	7.4
London Avenue	7,160	5.8

4.3 Permanent Pump Station Operation Frequency and Duration

The frequency of permanent pump station operation was evaluated by considering both Lake Pontchartrain elevation and canal discharge. A coincident frequency approach was used to generate an elevation-frequency relationship for each canal, which is affected by coincident drainage pump station discharge and Lake Pontchartrain elevations. The approach follows US Army Corps of Engineers Engineer Manual 1110-2-1413, Hydrologic Analysis of Interior Areas.

[13] Future condition (2057) design elevation without wave runup or setup (Section 3.3.1)

The canal discharge frequency depends upon drainage pump station operation. Therefore, canal discharge-frequency estimates based on historic DPS operation, adjusted to account for expected increased pumping capacity, were used for the coincident frequency analysis.

As shown below in Section 4.3.1, the coincident frequency analysis demonstrates that the combined canal discharges and high Lake Pontchartrain stages necessary to exceed proposed safe water elevations along the canals occur infrequently. More frequently Lake Pontchartrain is expected to reach elevations for which the canals cannot pass the maximum design discharges. Therefore, as described in Section 4.3.2, estimates of the frequency of permanent pump station operation are based on Lake Pontchartrain elevations. Expected duration of operation is described in Section 4.3.3.

4.3.1 Coincident Frequency Analysis

The coincident frequency approach utilizes a series of hypothetical single event hydrographs for the interior analysis and stage-duration for Lake Pontchartrain. The stage-duration curve (described in Section 3.3.2) is divided into segments, and the middle of each segment is considered the index lake elevation (B_i). The segment interval, $P(B_i)$, for the duration represents the probability of the interval. The sum of the probabilities must equal 1. The index lake elevations used for this analysis are shown in Table 4.3-1.

Table 4.3-1. Lake Pontchartrain Index Stages and Probabilities for Coincident Frequency Analysis

Stage Interval (ft NAVD88, 2004.65)	Index Stage (B_i) (ft NAVD88 2004.65)	Proportion of Time Stage Interval Exceeded, $P(B_i)$
-0.5 to 0.5	0	0.3690
0.5 to 1.5	1	0.5091
1.5 to 2.5	2	0.1094
2.5 to 3.5	3	0.0102
3.5 to 6.5	5	0.0021
6.5 to 9.5	8	0.0002
	Sum of probabilities:	1.0000

As described in Section 6.0, a range of canal discharges were analyzed for a number of the Lake Pontchartrain tailwater conditions. The probability of canal discharges is based on the discharge-frequency curves developed from the DPS pumping log data adjusted to account for increased drainage pump station capacity. The discharge-frequency curves used in the analysis are contained in Section 5.2.

A canal stage-frequency function, $P(A/B_i)$, is developed for each Lake Pontchartrain tailwater condition. This provides conditional probability curves, i.e., the probability of exceeding canal elevation "A" given Lake Pontchartrain elevation "B_i."

Finally, the total probability theorem is used to develop canal elevation probability functions.

$$P(A) = \sum_{i=1}^{n} \left(P(A/B_i) \times P(B_i) \right)$$

P(A) = probability of exceeding a given canal elevation;
$P(B_i)$ = probability Lake Pontchartrain is at the specific stage interval (i), where i assumes full range of values which have affect on canal elevation;
$P(A/B_i)$ = probability of exceeding a given canal elevation if the lake stage is at stage interval (i).

The results of the coincident frequency analysis are shown in Table 4.3-2 through Table 4.3-5. According to the analysis the canal water surface in each canal has a low probability of reaching or exceeding the proposed minimum safe water elevations. For example, Table 4.3-2 shows 17th Street Canal water surface elevation has a 1.5-percent chance of equaling or exceeding 8 ft NAVD88 (2004.65) when Lake Pontchartrain is at 5.0 ft. This probability takes into account the 17th Street Canal discharge-probability curve shown in Table 5.2-1, the modeled canal water surface elevation corresponding to a particular canal discharge as described in Section 6.0, and the percent of time Lake Pontchartrain stage is expected to be within the stage interval corresponding to 5.0 ft (Table 4.3-1).

Given the range of Lake Pontchartrain elevations investigated, the annual exceedance probability that 17th Street Canal water surface elevations will reach or exceed 8.0 ft is 0.02 percent. This is also the case for Orleans Avenue Canal. For London Avenue Canal, the probability that the canal water surface elevation will reach or exceed 9.0 ft is 0.09 percent. For London Avenue Canal 4,000 ft downstream of DPS 3, Table 4.3-5 shows the current safe water elevation of 5.0 ft has a 20 percent probability of being equaled or exceeded.

Table 4.3-2. Canal Water Surface Elevation - Frequency Relationship for 17th Street Canal

Maximum Canal Water Surface Elevation (A) (ft NAVD88, 2004.65)	Probability of Exceeding Canal Elevation "A" Given Lake Stage "B_i" (Conditional Probability $P_i(A/B_i)$)						Exceedance Probability of Canal Elevation (P(A))
	Lake Stage $B_1 = 0.0$ $P(B_1) =$ 0.3690	Lake Stage $B_2 = 1.0$ $P(B_2) =$ 0.5091	Lake Stage $B_3 = 2.0$ $P(B_3) =$ 0.1094	Lake Stage $B_4 = 3.0$ $P(B_4) =$ 0.0102	Lake Stage $B_5 = 5.0$ $P(B_5) =$ 0.0021	Lake Stage $B_6 = 8.0$ $P(B_6) =$ 0.0002	
0	1.0000	1.0000	1.0000	1.0000	1.0000	1.0000	1.0000
1	0.9999	1.0000	1.0000	1.0000	1.0000	1.0000	1.0000
2	0.8825	0.9999	1.0000	1.0000	1.0000	1.0000	0.9566
3	0.6433	0.7581	0.9553	1.0000	1.0000	1.0000	0.7403
4	0.2597	0.4132	0.6435	0.8917	1.0000	1.0000	0.3880
5	0.0716	0.1027	0.1880	0.4395	1.0000	1.0000	0.1061
6	0.0157	0.0178	0.0239	0.0754	0.6986	1.0000	0.0199
7	0.0000	0.0000	0.0000	0.0140	0.1263	1.0000	0.0006
8	0.0000	0.0000	0.0000	0.0000	0.0150	1.0000	0.0002

Table 4.3-3. Canal Water Surface Elevation – Frequency Relationship for Orleans Avenue Canal

Maximum Canal Water Surface Elevation (A) (ft NAVD88, 2004.65)	Probability of Exceeding Canal Elevation "A" Given Lake Stage "B_i" (Conditional Probability $P_i(A/B_i)$)						Exceedance Probability of Canal Elevation (P(A))
	Lake Stage $B_1 = 0.0$ $P(B_1) =$ 0.3690	Lake Stage $B_2 = 1.0$ $P(B_2) =$ 0.5091	Lake Stage $B_3 = 2.0$ $P(B_3) =$ 0.1094	Lake Stage $B_4 = 3.0$ $P(B_4) =$ 0.0102	Lake Stage $B_5 = 5.0$ $P(B_5) =$ 0.0021	Lake Stage $B_6 = 8.0$ $P(B_6) =$ 0.0002	
0	1.0000	1.0000	1.0000	1.0000	1.0000	1.0000	1.0000
1	0.9424	1.0000	1.0000	1.0000	1.0000	1.0000	0.9788
2	0.7273	0.8429	1.0000	1.0000	1.0000	1.0000	0.8194
3	0.4681	0.5713	0.7270	1.0000	1.0000	1.0000	0.5556
4	0.0372	0.1002	0.2917	0.5937	1.0000	1.0000	0.1050
5	0.0000	0.0000	0.0000	0.0273	1.0000	1.0000	0.0026
6	0.0000	0.0000	0.0000	0.0000	0.1756	1.0000	0.0006
7	0.0000	0.0000	0.0000	0.0000	0.0000	1.0000	0.0002
8	0.0000	0.0000	0.0000	0.0000	0.0000	1.0000	0.0002

Table 4.3-4. Canal Water Surface Elevation – Frequency Relationship for London Avenue Canal at DPS 3

Maximum Canal Water Surface Elevation (A) (ft NAVD88, 2004.65)	Probability of Exceeding Canal Elevation "A" Given Lake Stage "B_i" (Conditional Probability $P_i(A/B_i)$)						Exceedance Probability of Canal Elevation (P(A))
	Lake Stage $B_1 = 0.0$ $P(B_1) =$ 0.3690	Lake Stage $B_2 = 1.0$ $P(B_2) =$ 0.5091	Lake Stage $B_3 = 2.0$ $P(B_3) =$ 0.1094	Lake Stage $B_4 = 3.0$ $P(B_4) =$ 0.0102	Lake Stage $B_5 = 5.0$ $P(B_5) =$ 0.0021	Lake Stage $B_6 = 8.0$ $P(B_6) =$ 0.0002	
0	1.0000	1.0000	1.0000	1.0000	1.0000	1.0000	1.0000
1	0.9999	1.0000	1.0000	1.0000	1.0000	1.0000	1.0000
2	0.9870	0.9999	1.0000	1.0000	1.0000	1.0000	0.9952
3	0.8237	0.8907	0.9999	1.0000	1.0000	1.0000	0.8793
4	0.6515	0.7012	0.7863	0.9357	1.0000	1.0000	0.6953
5	0.4623	0.5068	0.5715	0.6841	1.0000	1.0000	0.5004
6	0.2570	0.2810	0.3383	0.4592	0.8397	1.0000	0.2815
7	0.1255	0.1343	0.1690	0.2565	0.5726	1.0000	0.1372
8	0.0598	0.0607	0.0748	0.1145	0.3073	1.0000	0.0632
9	0.0000	0.0000	0.0000	0.0442	0.1271	0.7734	0.0009

Table 4.3-5. Canal Water Surface Elevation – Frequency Relationship for London Avenue Canal Approximately 4,000 ft Downstream of DPS 3

Canal Water Surface Elevation (A) 4,000' ds DPS 3 (ft NAVD88, 2004.65)	Probability of Exceeding Canal Elevation "A" Given Lake Stage "B_i" (Conditional Probability $P_i(A/B_i)$)						Exceedance Probability of Canal Elevation 4,000' ds DPS 3 (P(A))
	Lake Stage $B_1 = 0.0$ $P(B_1) = 0.3690$	Lake Stage $B_2 = 1.0$ $P(B_2) = 0.5091$	Lake Stage $B_3 = 2.0$ $P(B_3) = 0.1094$	Lake Stage $B_4 = 3.0$ $P(B_4) = 0.0102$	Lake Stage $B_5 = 5.0$ $P(B_5) = 0.0021$	Lake Stage $B_6 = 8.0$ $P(B_6) = 0.0002$	
0	1.0000	1.0000	1.0000	1.0000	1.0000	1.0000	1.0000
1	0.9999	1.0000	1.0000	1.0000	1.0000	1.0000	1.0000
2	0.8338	0.9659	1.0000	1.0000	1.0000	1.0000	0.9213
3	0.6291	0.7162	0.8776	1.0000	1.0000	1.0000	0.7053
4	0.3944	0.4748	0.5896	0.7890	1.0000	1.0000	0.4621
5	0.1658	0.1990	0.2925	0.4838	1.0000	1.0000	0.2018
6	0.0678	0.0699	0.1033	0.2010	0.6934	1.0000	0.0756
7	0.0000	0.0000	0.0000	0.0634	0.3096	1.0000	0.0015
8	0.0000	0.0000	0.0000	0.0000	0.0834	1.0000	0.0004

4.3.2 Expected Frequency of Permanent Pump Station Operation

As described in Section 4.3.1, a combination of high Lake Pontchartrain elevations and high canal discharges is expected to occur infrequently. However, because hydraulic model results show that the canals cannot pass the maximum design discharge for certain Lake Pontchartrain elevations, it is recommended that the permanent pump station flood gates be closed and permanent pumps operated when Lake Pontchartrain reaches those elevations that cannot pass the maximum discharge. Table 4.3-6 shows the expected frequency of permanent flood gate closure for each canal. It is assumed that the permanent pump station pumps are operated to some degree each time the permanent flood gates are closed.

17th Street Canal maximum design discharge is 12,500 cfs. The canal cannot pass this discharge for Lake Pontchartrain elevations greater than 4.5 ft NAVD88 (2004.65) without exceeding the safe water elevation. The safe water elevation along the canal is assumed to be a minimum of 8.0 ft. Based on the stage-frequency curve contained in Section 3.3.3, there is a 35-percent annual chance that Lake Pontchartrain would reach an elevation of 4.5 ft. This corresponds to about a 2.9-year return period.

Table 4.3-6. Probability of Permanent Pump Station Operation Assuming Maximum Design Capacity at Drainage Pump Stations

Canal	Lake Pontchartrain Gate Closure Elevation (ft NAVD88, 2004.65)	Exceedance Probability (Percent-Annual-Chance)
17th Street	4.5	35
Orleans Avenue	6.0	8
London Avenue	3.0	95

Orleans Avenue Canal maximum design discharge is 3,390 cfs. The canal can pass this discharge for Lake Pontchartrain elevations up to 7.1 ft without exceeding the safe water elevation. The safe water elevation along the canal is assumed to be a minimum of 8.0 ft. It is recommended gates be closed at lake elevation 6.0 ft rather than 7.0 ft to facilitate gate closure at high lake elevations. Based on the stage-frequency curve contained in Section 3.3.3, there is an 8-percent annual chance that Lake Pontchartrain would reach an elevation of 6.0 ft. This corresponds to a 12.5-year return period.

London Avenue Canal maximum design discharge is 8,980 cfs. The canal cannot pass this discharge for Lake Pontchartrain elevations greater than 3.0 ft without exceeding the safe water elevation. The safe water elevation along the canal is assumed to be a minimum of 8.0 ft. Based on the stage-frequency curve contained in Section 3.3.3, there is a 95-percent annual chance that Lake Pontchartrain would reach an elevation of 3.0 ft. This corresponds to a 1.05-year return period.

4.3.3 Duration of Operation

The permanent pump stations would be operated less frequently than the existing drainage pump stations. This is because each canal can pass a certain discharge for a given Lake Pontchartrain elevation with the flood gates open without exceeding safe water elevation. A summary of the canal discharge and lake elevation combinations that each canal can safely pass is contained in Section 4.1, while the full range of combinations can be interpolated from Figure 6.1-1 through Figure 6.1-4.

The pumped flow-duration curves for the existing drainage pump stations are contained in Section 5.1. These indicate that the existing drainage pump stations are operated between 1.5 and 3.7 percent of the time, or on average from 130 to 320 hours annually.

Table 4.3-7 shows the expected average annual permanent pump station operation. It includes the Lake Pontchartrain gate closure elevations described in Section 4.3.2. The lake stage-duration curve shown in Figure 3.3-5 provides the percent of time Lake Pontchartrain elevations are expected to be equaled or exceeded, and these are included in the table below. For each canal, the average number of hours that the permanent flood gates could be expected to be closed in any given year are based on the Lake Pontchartrain stage-duration curve.

Table 4.3-7. Expected Average Annual Permanent Pump Station Operation

Canal	Existing DPS Avg Annual Pumping (Hrs)	Lake Pontchartrain Gate Closure Elevation (ft NAVD88, 2004.65)	Percent Time Elevation Equaled or Exceeded	Expected Avg Annual Gate Closure Time (Hrs)
17th Street	275	4.5	0.075	7
Orleans Avenue	150	6.0	0.043	4
London Avenue (DPS 3)	320	3.0	0.47	40
London Avenue (DPS 4)	130	3.0	0.47	40

It is assumed that the permanent pump station pumps will be operated for some fraction of the time the permanent flood gates are closed. Based on expected average annual gate closures, the

17th Street Canal permanent pump stations might be operated up to 7 hours annually. Orleans Avenue Canal and London Avenue Canal would be expected to be operated up to 4 hours and 40 hours annually, respectively. It is very possible the permanent flood gates would need to be closed based on Lake Pontchartrain elevations even when the existing drainage pump stations are not operating. This would result in fewer pump operating hours annually than described above.

4.4 Operational Timing

Most often the permanent pump station flood gates will be left open while existing drainage pump stations are operating. The following analysis describes potential operational timing for the case when Lake Pontchartrain elevations are expected to exceed those recommended in Table 4.3-6 while the existing drainage pump stations are operating. The timing recommended here focuses on concurrent combination (2) described in Section 4.2, that is, maximum static Lake Pontchartrain elevation and 5-year canal discharge. As noted, the gates would remain open for concurrent combination (1) since all canals can pass the maximum design discharge for Lake Pontchartrain elevation 2.5 ft NAVD88 (2004.64). The operational start-up times are based on hydraulic modeling results described in Section 6.3. Permanent pump station gates are assumed to close in 15 minutes, and pumps are assumed to start when gates reach water surface. A summary of recommended Lake Pontchartrain gate closure elevations and permanent pump station pump start-up times are included in Table 4.4-1.

Table 4.4-1. Operational Timing for Concurrent Event Combination (2): Maximum Static Lake Pontchartrain Elevation= 11.2[14] ft NAVD88 (2004.65) and 5-Year Canal Discharge

Canal	Canal Discharge (cfs)	Lake Elevation to Close Flood Gates (ft NAVD88, 2004.65)	Required Permanent Pump Start-Up Time (min)
17th Street	8,950	4.5	20
Orleans Avenue	2,800	6.0	30
London Avenue	7,160	3.0	20

As described in Section 5.2, the 17th Street Canal 5-year discharge is 8,950 cfs. It is recommended that the 17th Street Canal flood gates be closed when Lake Pontchartrain reaches elevation 4.5 ft NAVD88 (2004.65).

As shown in Table 4.4-1, if the flood gates are closed when Lake Pontchartrain elevation is at 4.5 ft, the permanent pump station pumps must have a pump start-up time of 20 minutes in order to not exceed safe water elevation. The 20-minute pump start-up time is defined as the time from when the pumps are turned on to the time they reach full capacity. This timing assumes the drainage pump stations are operating when Lake Pontchartrain reaches elevation 4.5 ft.

The Orleans Avenue Canal 5-year discharge is 2,800 cfs. It is recommended that the Orleans Avenue Canal flood gates be closed when Lake Pontchartrain reaches elevation 6.0 ft.

If the flood gates are closed when Lake Pontchartrain elevation is at 6.0 ft, the permanent pump station pumps need only have a pump start-up time of 30 minutes. This is a long pump start-up

[14] Future condition (2057) design elevation without wave runup or setup (Section 3.3.1)

time. As described in Section 6.3, reasonable pump start-up times can be as short as 5 to 10 minutes.

The London Avenue Canal 5-year discharge is 7,160 cfs. It is recommended that the London Avenue Canal flood gates be closed when Lake Pontchartrain reaches elevation 3.0 ft.

If the flood gates are closed when Lake Pontchartrain elevation is at 3.0 ft, the permanent pump station pumps must have a pump start-up time of 20 minutes in order to not exceed safe water elevation. This timing assumes the drainage pump stations are operating when Lake Pontchartrain reaches elevation 3.0 ft.

5.0 CANAL PUMPED DISCHARGE FREQUENCY ANALYSIS

5.1 Pumped Flow-Duration

The data to develop flow duration curves were collected and processed electronically by the Hurricane Protection Office as described in Section 3.5. The historic use of DPS 6, DPS 7, DPS 3, and DPS 4 pumps are described below.

5.1.1 17th Street Canal

Pump operating logs for DPS 6 were obtained for the period 1984 to 2006. From this 23-year period of record, 22 years of pump operating logs had sufficient data available for analysis. As described in Table 2.1-1, DPS 6 has two 1,100 cfs capacity pumps, five 1,000 cfs pumps, two 550 cfs pumps, and four 250 cfs pumps. It also has two 90 cfs constant duty pumps that pump dry weather flow to the Mississippi River. The pump operating data for the constant duty pumps were not collected and are not included in the analysis.

The flow duration curve for DPS 6 is shown in Figure 5.1-1. The historical record shows that DPS 6 pumps to 17th Street Canal about 3.1 percent of the time, or about 275 hours per year on average. During the time the pumps are operating, the pumped discharge is most often less than 2,500 cfs. That is, about 90 percent of the operating time, DPS 6 discharges no more than 2,500 cfs to 17th Street Canal.

According to Table 5.1-1 the discharge from DPS 6 has exceeded 6,000 cfs about 4 hours per year on average. In other words, about two percent of the time the pumps are in use the discharge exceeds 6,000 cfs. The discharge from DPS 6 has exceeded 8,000 cfs only seven hours during the 23-year period of record. In 2004 the pumped discharge from all DPS 6 pumps (excluding constant duty) exceeded 8,000 cfs for 3 hours total. The longest continuous duration of pumping exceeding 8,000 cfs occurred May 12-13, 2004 for 1.5 hours. DPS 6 discharged continuously for seven hours during that event. Again, the current capacity of DPS 6 is 9,480 cfs, with 180 cfs of that not included in this analysis.

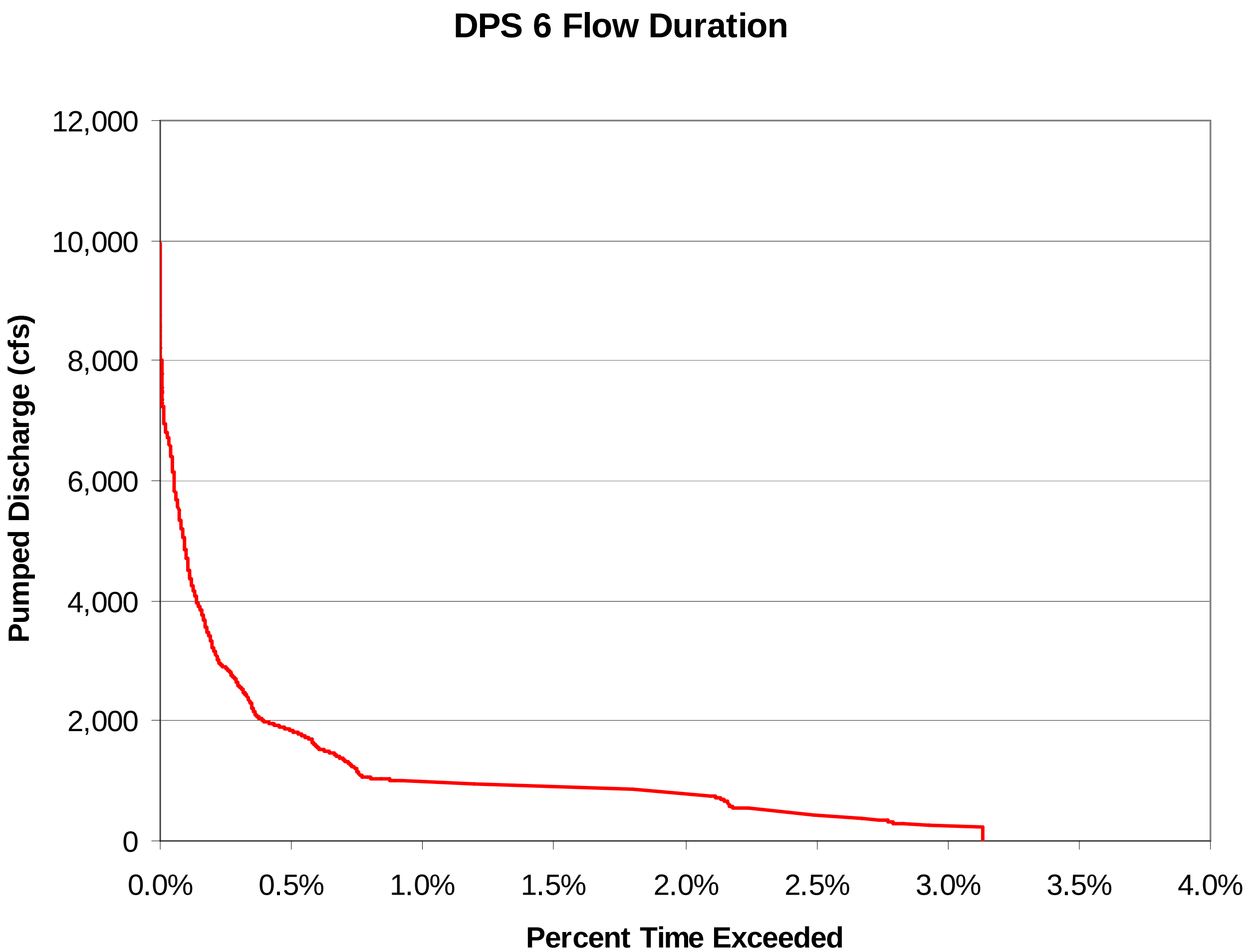


Figure 5.1-1. 17th Street Canal: DPS 6 Pumped Flow Duration Curve

Table 5.1-2 and Table 5.1-3 show the total hours of operating time for all but the constant duty pumps at DPS 6. For each year the table indicates the number of hours a pump operated and the percent of total time those hours represent. For example, in 2004 Pump I operated 244 hours over the course of the year, or 2.8 percent of the time. Over the period of record analyzed, the DPS 6 pumps operated most frequently in 1991 for a total of 693 hours and least frequently in 2000 for 120 hours. Pump H has a capacity of 1,100 cfs and is operated most frequently in comparison to the other pumps at DPS 6.

Table 5.1-1. 17th Street Canal - DPS 6 Annual Discharges

Year	Hours Pumping at Given Flows | Percent Time Pumping within Flow Range															
	0 – 500 cfs		501 – 1,000 cfs		1,001 – 2,000 cfs		2,001 – 4,000 cfs		4,001 – 6,000 cfs		6,001 – 8,000 cfs		8,001 – 10,000 cfs		10,000+ cfs	
2006	31	6%	253	48%	207	39%	25	5%	6	1%	4	1%	0	0%	0	0%
2005	0	0%	273	74%	64	17%	21	6%	7	2%	3	1%	3	1%	0	0%
2004	0	0%	213	73%	39	13%	19	6%	15	5%	5	2%	1	0%	0	0%
2003	1	0%	257	79%	45	14%	19	6%	4	1%	1	0%	0	0%	0	0%
2002	49	15%	177	53%	60	18%	35	10%	11	3%	3	1%	2	1%	0	0%
2001	9	8%	70	58%	23	19%	17	14%	2	1%	0	0%	0	0%	0	0%
2000	20	14%	85	58%	24	16%	11	7%	6	4%	2	1%	0	0%	0	0%
1999	3	1%	162	57%	43	15%	40	14%	21	7%	17	6%	1	0%	0	0%
1998	6	4%	92	70%	24	18%	10	7%	1	1%	0	0%	0	0%	0	0%
1997	13	8%	81	55%	28	19%	23	15%	4	2%	1	1%	0	0%	0	0%
1996	13	6%	140	62%	25	11%	21	9%	14	6%	14	6%	1	0%	0	0%
1995	75	33%	96	42%	37	16%	15	6%	6	2%	2	1%	0	0%	0	0%
1994	93	42%	70	32%	35	16%	20	9%	4	2%	0	0%	0	0%	0	0%
1993	66	24%	122	43%	51	18%	25	9%	10	3%	8	3%	0	0%	0	0%
1992	292	42%	241	35%	74	11%	57	8%	22	3%	9	1%	0	0%	0	0%
1991	111	51%	41	19%	33	15%	26	12%	4	2%	4	2%	0	0%	0	0%
1990	73	40%	40	22%	44	24%	14	7%	6	3%	6	3%	0	0%	0	0%
1989	171	50%	65	19%	49	14%	32	9%	10	3%	15	4%	0	0%	0	0%
1988	119	53%	54	24%	33	15%	14	6%	4	2%	0	0%	0	0%	0	0%
1987	121	59%	47	23%	21	10%	11	5%	4	2%	0	0%	0	0%	0	0%
1986	175	51%	54	16%	71	20%	32	9%	12	3%	3	1%	0	0%	0	0%
1985	144	69%	33	16%	21	10%	11	5%	0	0%	0	0%	0	0%	0	0%
1984	0	0%	273	74%	64	17%	21	6%	7	2%	3	1%	3	1%	0	0%
Total	1,582	26%	2,671	44%	1,033	17%	491	8%	167	3%	93	2%	7	0.1%	0	0%

Table 5.1-2. 17th Street Canal - DPS 6 Annual Pumping Operation (Pumps A through F)

Year	Hours Pump On | Percent Time Pumping													
	Total		Pump A (550 cfs)		Pump B (550 cfs)		Pump C (1,000 cfs)		Pump D (1,000 cfs)		Pump E (1,000 cfs)		Pump F (1,000 cfs)	
2006	524	6.0%	10	0.1%	8	0.1%	7	0.1%	13	0.1%	39	0.4%	19	0.2%
2005	Insufficient Data													
2004	370	4.2%	5	0.1%	5	0.1%	5	0.1%	11	0.1%	10	0.1%	34	0.4%
2003	290	3.3%	5	0.1%	7	0.1%	7	0.1%	20	0.2%	19	0.2%	38	0.4%
2002	327	3.7%	1	0.0%	6	0.1%	6	0.1%	7	0.1%	21	0.2%	22	0.2%
2001	335	3.8%	13	0.1%	30	0.3%	9	0.1%	51	0.6%	38	0.4%	68	0.8%
2000	120	1.4%	0	0.0%	0	0.0%	0	0.0%	0	0.0%	4	0.0%	22	0.2%
1999	147	1.7%	1	0.0%	1	0.0%	2	0.0%	8	0.1%	5	0.1%	16	0.2%
1998	285	3.2%	11	0.1%	8	0.1%	20	0.2%	22	0.3%	41	0.5%	72	0.8%
1997	132	1.5%	6	0.1%	6	0.1%	0	0.0%	0	0.0%	2	0.0%	10	0.1%
1996	149	1.7%	2	0.0%	8	0.1%	4	0.0%	6	0.1%	8	0.1%	19	0.2%
1995	227	2.6%	25	0.3%	22	0.3%	3	0.0%	23	0.3%	31	0.3%	50	0.6%
1994	229	2.6%	18	0.2%	26	0.3%	7	0.1%	10	0.1%	6	0.1%	9	0.1%
1993	221	2.5%	3	0.0%	62	0.7%	4	0.0%	13	0.1%	10	0.1%	14	0.2%
1992	280	3.2%	3	0.0%	49	0.6%	7	0.1%	19	0.2%	25	0.3%	32	0.4%
1991	693	7.9%	10	0.1%	91	1.0%	25	0.3%	26	0.3%	22	0.2%	72	0.8%
1990	217	2.5%	13	0.1%	128	1.5%	10	0.1%	15	0.2%	12	0.1%	33	0.4%
1989	183	2.1%	10	0.1%	117	1.3%	11	0.1%	12	0.1%	17	0.2%	21	0.2%
1988	341	3.9%	167	1.9%	258	2.9%	23	0.3%	42	0.5%	32	0.4%	67	0.8%
1987	223	2.5%	95	1.1%	178	2.0%	4	0.0%	12	0.1%	15	0.2%	25	0.3%
1986	203	2.3%	85	1.0%	155	1.8%	7	0.1%	6	0.1%	9	0.1%	15	0.2%
1985	345	3.9%	102	1.2%	249	2.8%	11	0.1%	27	0.3%	20	0.2%	76	0.9%
1984	208	2.4%	104	1.2%	102	1.2%	5	0.1%	6	0.1%	1	0.0%	22	0.2%
Total	**6,043**	**3.1%**	**685**	**0.4%**	**1510**	**0.8%**	**174**	**0.1%**	**343**	**0.2%**	**383**	**0.2%**	**750**	**0.4%**

Table 5.1-3. 17th Street Canal - DPS 6 Annual Pumping Operation (Pumps G through I and Pumps 1 through 4)

Year	Hours Pump On | Percent Time Pumping													
	Pump G (1,000 cfs)		Pump H (1,100 cfs)		Pump I (1,100 cfs)		Pump 1 (250 cfs)		Pump 2 (250 cfs)		Pump 3 (250 cfs)		Pump 4 (250 cfs)	
2006	9	0.1%	67	0.8%	393	4.5%	77	0.9%	19	0.2%	38	0.4%	8	0.1%
2005	Insufficient Data													
2004	19	0.2%	182	2.1%	244	2.8%	16	0.2%	15	0.2%	13	0.1%	12	0.1%
2003	18	0.2%	204	2.3%	129	1.5%	18	0.2%	17	0.2%	15	0.2%	13	0.1%
2002	9	0.1%	249	2.8%	100	1.1%	15	0.2%	15	0.2%	12	0.1%	8	0.1%
2001	15	0.2%	182	2.1%	48	0.5%	84	1.0%	36	0.4%	26	0.3%	21	0.2%
2000	7	0.1%	67	0.8%	47	0.5%	29	0.3%	16	0.2%	15	0.2%	16	0.2%
1999	10	0.1%	87	1.0%	67	0.8%	4	0.0%	27	0.3%	10	0.1%	8	0.1%
1998	41	0.5%	222	2.5%	151	1.7%	3	0.0%	43	0.5%	39	0.4%	39	0.4%
1997	5	0.1%	71	0.8%	69	0.8%	1	0.0%	14	0.2%	8	0.1%	13	0.1%
1996	11	0.1%	93	1.1%	72	0.8%	1	0.0%	17	0.2%	5	0.1%	4	0.0%
1995	33	0.4%	140	1.6%	125	1.4%	9	0.1%	23	0.3%	18	0.2%	4	0.0%
1994	8	0.1%	107	1.2%	74	0.8%	36	0.4%	51	0.6%	15	0.2%	1	0.0%
1993	9	0.1%	48	0.5%	39	0.4%	72	0.8%	85	1.0%	71	0.8%	1	0.0%
1992	28	0.3%	116	1.3%	74	0.8%	97	1.1%	83	0.9%	82	0.9%	1	0.0%
1991	48	0.5%	135	1.5%	51	0.6%	211	2.4%	428	4.9%	393	4.5%	1	0.0%
1990	19	0.2%	81	0.9%	13	0.1%	33	0.4%	7	0.1%	4	0.0%	1	0.0%
1989	13	0.1%	101	1.2%	10	0.1%	1	0.0%	1	0.0%	1	0.0%	1	0.0%
1988	76	0.9%	0	0.0%	0	0.0%	1	0.0%	1	0.0%	1	0.0%	1	0.0%
1987	25	0.3%	0	0.0%	0	0.0%	1	0.0%	1	0.0%	1	0.0%	1	0.0%
1986	27	0.3%	0	0.0%	0	0.0%	1	0.0%	1	0.0%	1	0.0%	1	0.0%
1985	75	0.9%	0	0.0%	0	0.0%	1	0.0%	1	0.0%	1	0.0%	1	0.0%
1984	23	0.3%	2	0.0%	0	0.0%	1	0.0%	1	0.0%	1	0.0%	1	0.0%
Total	**522**	**0.3%**	**2,150**	**1.1%**	**1704**	**0.9%**	**695**	**0.4%**	**886**	**0.5%**	**752**	**0.4%**	**138**	**0.1%**

5.1.2 Orleans Avenue Canal

Pump operating logs for DPS 7 were obtained for the period 1980 to 2006. From this 27-year period of record, 22 years of pump operating logs had sufficient data available for analysis. As described in Table 2.2-1, DPS 7 has two 1,000 cfs capacity pumps and one 550 cfs pump. It also has two 70 cfs constant duty pumps that pump dry weather flow to the Mississippi River. The pump operating data for the constant duty pumps were not collected and are not included in the analysis.

The flow duration curve for DPS 7 is shown in Figure 5.1-2. The historical record shows that DPS 7 pumps to Orleans Avenue Canal about 1.7 percent of the time, or about 150 hours per year on average.

Table 5.1-4 shows the magnitude of flows DPS 7 most frequently discharges to the Orleans Avenue Canal when the pumps are operating. During the time the pumps are operating, the pumped discharge is most often less than 1,000 cfs. That is, about 90 percent of the operating time, DPS 7 discharges no more than 1,000 cfs to Orleans Avenue Canal. The discharge from DPS 7 has exceeded 2,000 cfs about 4 hours per year on average. In other words, about three percent of the time the pumps are in use the discharge exceeds 2,000 cfs. Again, the current capacity of DPS 7 is 2,690 cfs, with 140 cfs of that not included in this analysis.

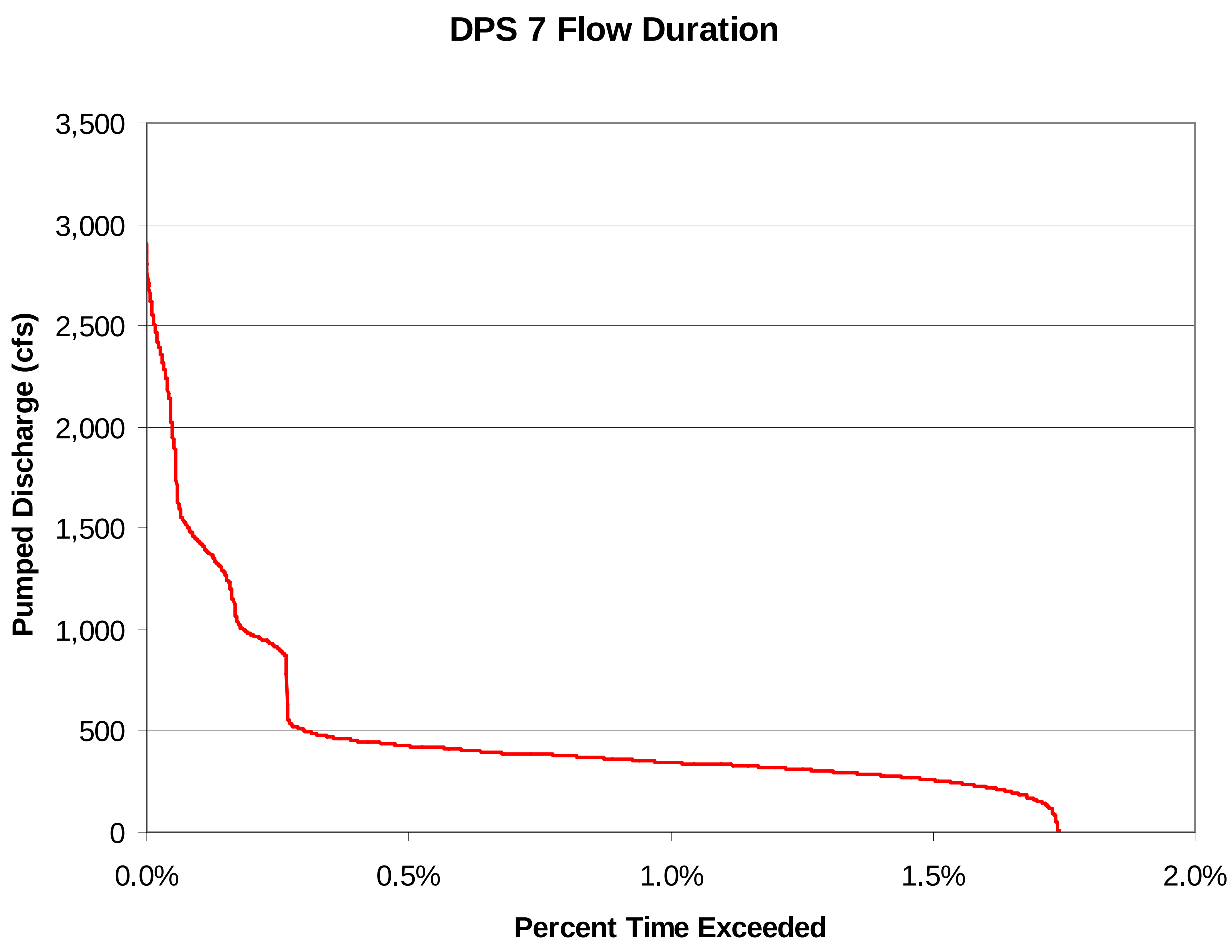


Figure 5.1-2. Orleans Avenue Canal: DPS 7 Pumped Flow Duration Curve

Table 5.1-5 shows the total hours of operating time for all but the constant duty pumps at DPS 7. For each year the table indicates the number of hours a pump operated and the percent of total time those hours represent. For example, in 1997 Pump A operated 66 hours over the course of the year, or 0.8 percent of the time. Over the period of record analyzed, the DPS 7 pumps operated most frequently in 1991 for a total of 318 hours and least frequently in 1986 for 67 hours. Pump A has a capacity of 550 cfs and is operated most frequently in comparison to the other pumps at DPS 7.

Table 5.1-4. Orleans Avenue Canal - DPS 7 Annual Discharges

Year	Hours Pumping at Given Flows | Percent Time Pumping within Flow Range							
	0 – 500 cfs		501 – 1,000 cfs		1,001 – 2,000 cfs		2,000+ cfs	
2006	Insufficient Data							
2005	148	82%	14	8%	15	8%	4	2%
2004	Insufficient Data							
2003	104	76%	6	4%	21	15%	6	4%
2002	89	76%	12	10%	14	12%	3	3%
2001	125	77%	20	12%	15	9%	3	2%
2000	Insufficient Data							
1999	67	83%	6	7%	5	6%	3	4%
1998	173	71%	17	7%	34	14%	21	8%
1997	60	86%	4	6%	5	6%	1	1%
1996	56	73%	13	17%	8	10%	0	0%
1995	115	74%	7	4%	14	9%	19	12%
1994	Insufficient Data							
1993	138	96%	2	1%	4	3%	0	0%
1992	Insufficient Data							
1991	279	88%	15	5%	22	7%	3	1%
1990	103	98%	2	1%	1	0%	0	0%
1989	99	92%	1	0%	8	7%	0	0%
1988	190	87%	4	2%	19	9%	5	2%
1987	117	88%	9	7%	6	5%	1	0%
1986	52	78%	10	15%	4	6%	1	1%
1985	149	88%	10	6%	9	5%	2	1%
1984	95	99%	0	0%	1	1%	0	0%
1983	190	89%	4	2%	14	6%	7	3%
1982	193	80%	21	9%	18	8%	9	4%
1981	119	70%	29	17%	16	9%	6	4%
1980	115	72%	29	18%	15	9%	1	1%
Total	**2,770**	**83%**	**230**	**7%**	**262**	**8%**	**93**	**3%**

Table 5.1-5. Orleans Avenue Canal - DPS 7 Annual Pumping Operation

Year	Hours Pump On | Percent Time Pumping							
	Total		Pump A (550 cfs)		Pump C (1,000 cfs)		Pump D (1,000 cfs)	
2006	Insufficient Data							
2005	180	2.0%	172	2.0%	7	0.1%	24	0.3%
2004	Insufficient Data							
2003	136	1.7%	128	1.6%	9	0.1%	32	0.4%
2002	117	1.3%	105	1.2%	8	0.1%	24	0.3%
2001	162	1.8%	144	1.6%	9	0.1%	32	0.4%
2000	Insufficient Data							
1999	81	0.9%	77	0.9%	5	0.1%	10	0.1%
1998	243	2.8%	228	2.6%	32	0.4%	62	0.7%
1997	70	0.8%	66	0.8%	4	0.0%	7	0.1%
1996	77	0.9%	68	0.8%	0	0.0%	16	0.2%
1995	154	1.8%	147	1.7%	24	0.3%	37	0.4%
1994	Insufficient Data							
1993	144	1.6%	144	1.6%	4	0.0%	0	0.0%
1992	Insufficient Data							
1991	318	3.6%	309	3.5%	26	0.3%	10	0.1%
1990	105	1.2%	105	1.2%	1	0.0%	0	0.0%
1989	107	1.2%	106	1.2%	6	0.1%	2	0.0%
1988	218	2.5%	216	2.5%	14	0.2%	17	0.2%
1987	132	1.5%	131	1.5%	2	0.0%	6	0.1%
1986	67	0.8%	66	0.7%	1	0.0%	4	0.0%
1985	170	1.9%	169	1.9%	5	0.1%	9	0.1%
1984	96	1.1%	96	1.1%	1	0.0%	1	0.0%
1983	213	2.4%	209	2.4%	9	0.1%	22	0.3%
1982	240	2.7%	213	2.4%	16	0.2%	39	0.4%
1981	169	1.9%	136	1.5%	7	0.1%	45	0.5%
1980	160	1.8%	130	1.5%	10	0.1%	33	0.4%
Total	**3353**	**1.7%**	**3157**	**1.6%**	**196**	**0.1%**	**428**	**0.2%**

5.1.3 London Avenue Canal

Pump operating logs for DPS 3 were obtained for the period 1979 to 2006. From this 28-year period of record, 26 years of pump operating logs had data available for analysis. As described in Table 2.3-1, DPS 3 has three 1,000 cfs capacity pumps and two 550 cfs pumps. It also has two 80 cfs constant duty pumps that pump dry weather flow to the Mississippi River. The pump operating data for the constant duty pumps were not collected and are not included in the analysis.

The flow duration curve for DPS 3 is shown in Figure 5.1-3. The historical record shows that DPS 3 pumps to London Avenue Canal about 3.7 percent of the time, or about 320 hours per year on average.

Table 5.1-6 shows the magnitude of flows DPS 3 most frequently discharges to the London Avenue Canal when the pumps are operating. During the time the pumps are operating, the pumped discharge is most often less than 1,000 cfs. That is, about 92 percent of the operating time, DPS 3 discharges no more than 1,000 cfs to London Avenue Canal. The discharge from DPS 3 has exceeded 3,000 cfs about 2.5 hours per year on average. In other words, about one percent of the time the pumps are in use the discharge exceeds 3,000 cfs. Again, the current capacity of DPS 3 is 4,260 cfs, with 160 cfs of that not included in this analysis.

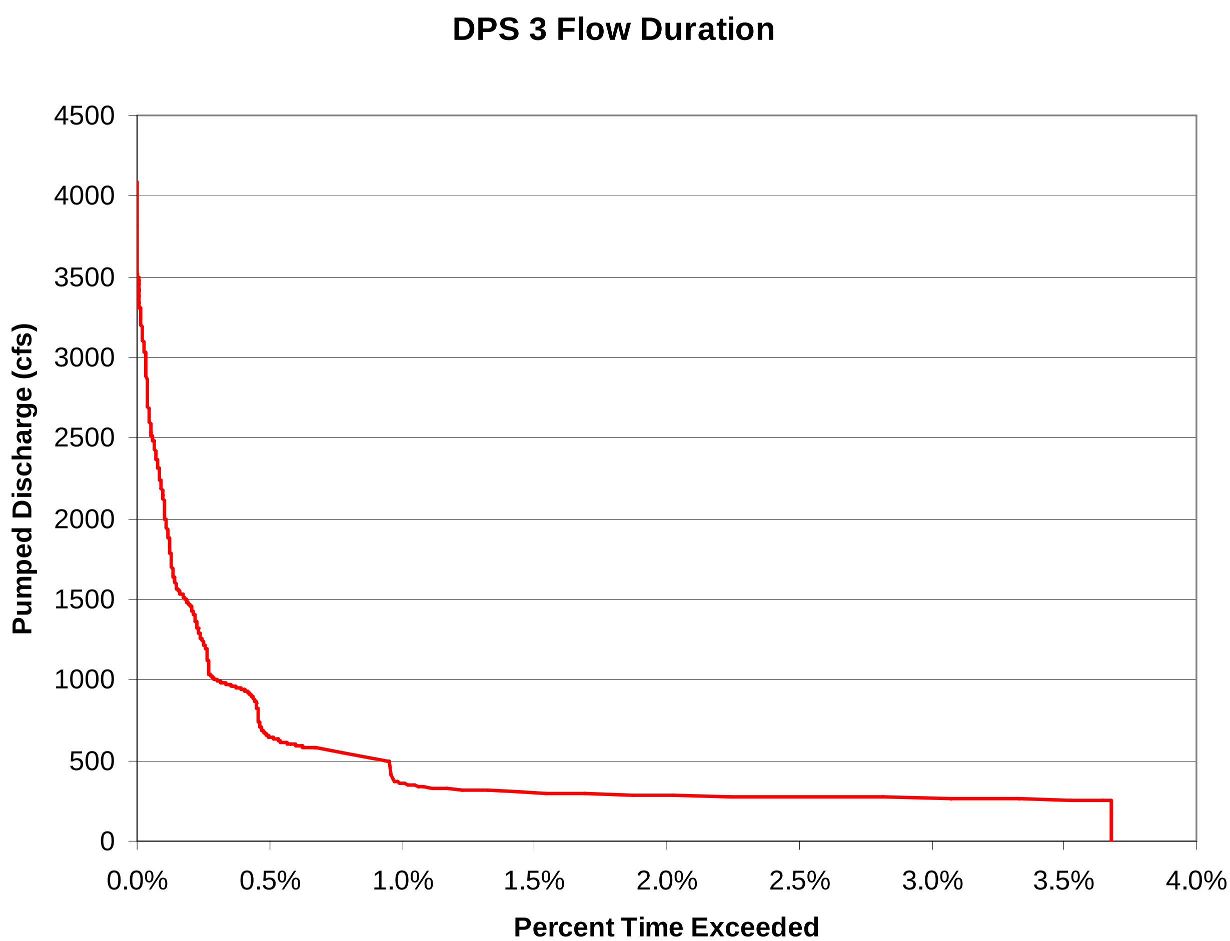


Figure 5.1-3. London Avenue Canal: DPS 3 Pumped Flow Duration Curve

Table 5.1-7 shows the total hours of operating time for all but the constant duty pumps at DPS 3. For each year the table indicates the number of hours a pump operated and the percent of total time those hours represent. For example, in 1997 Pump A operated 220 hours over the course of the year, or 2.5 percent of the time. Over the period of record analyzed, the DPS 3 pumps operated most frequently in 1985 for a total of 615 hours and least frequently in 2000 for 51 hours. Pump A has a capacity of 550 cfs and is operated most frequently in comparison to the other pumps at DPS 3.

Table 5.1-6. London Avenue Canal - DPS 3 Annual Discharges

Year	Hours Pumping at Given Flows | Percent Time Pumping within Flow Range											
	0 – 500 cfs		501 – 1,000 cfs		1,001 – 2,000 cfs		2,001 – 3,000 cfs		3,001 – 4,000 cfs		4,000+ cfs	
2006	49	40%	53	43%	19	16%	1	1%	0	0%	0	0%
2005	103	51%	69	34%	25	12%	6	3%	1	0%	0	0%
2004	178	64%	72	26%	22	8%	6	2%	2	1%	0	0%
2003	159	60%	68	26%	22	8%	11	4%	6	2%	0	0%
2002	152	59%	66	26%	17	7%	8	3%	13	5%	0	0%
2001	159	70%	35	15%	22	10%	9	4%	3	1%	0	0%
2000	41	80%	5	10%	5	9%	1	1%	0	0%	0	0%
1999	150	76%	42	21%	6	3%	1	1%	0	0%	0	0%
1998	240	65%	78	21%	22	6%	29	8%	2	1%	0	0%
1997	195	77%	45	18%	10	4%	4	2%	1	0%	0	0%
1996	166	69%	57	24%	12	5%	5	2%	1	0%	0	0%
1995	203	68%	55	18%	21	7%	13	4%	9	3%	0	0%
1994	No Data Available											
1993	No Data Available											
1992	397	77%	87	17%	19	4%	9	2%	2	0%	0	0%
1991	372	70%	128	24%	21	4%	5	1%	7	1%	0	0%
1990	160	69%	51	22%	13	6%	4	2%	3	1%	0	0%
1989	171	74%	41	18%	11	5%	7	3%	3	1%	0	0%
1988	211	71%	59	20%	20	7%	6	2%	3	1%	0	0%
1987	507	92%	34	6%	9	2%	2	0%	1	0%	0	0%
1986	377	89%	33	8%	11	2%	5	1%	1	0%	0	0%
1985	399	65%	165	27%	42	7%	8	1%	1	0%	0	0%
1984	363	92%	20	5%	12	3%	2	0%	0	0%	0	0%
1983	400	84%	37	8%	25	5%	15	3%	3	1%	0	0%
1982	263	81%	34	10%	18	6%	8	2%	3	1%	0	0%
1981	135	78%	22	12%	10	5%	8	4%	0	0%	0	0%
1980	317	79%	54	14%	19	5%	5	1%	5	1%	1	0%
1979	366	81%	63	14%	19	4%	2	0%	0	0%	0	0%
Total	**6,227**	**74%**	**1477**	**18%**	**435**	**5%**	**175**	**2%**	**68**	**1%**	**1**	**0%**

Table 5.1-7. London Avenue Canal - DPS 3 Annual Pumping Operation

Year	Hours Pump On | Percent Time Pumping											
	Total		Pump A (550 cfs)		Pump B (550 cfs)		Pump C (1,000 cfs)		Pump D (1,000 cfs)		Pump E (1,000 cfs)	
2006	122	1.4%	64	0.7%	31	0.4%	23	0.3%	8	0.1%	30	0.3%
2005	204	2.3%	164	1.9%	71	0.8%	8	0.1%	43	0.5%	18	0.2%
2004	279	3.2%	226	2.6%	95	1.1%	9	0.1%	36	0.4%	19	0.2%
2003	265	3.0%	225	2.6%	84	1.0%	16	0.2%	42	0.5%	29	0.3%
2002	255	2.9%	222	2.5%	128	1.5%	17	0.2%	28	0.3%	34	0.4%
2001	227	2.6%	131	1.5%	149	1.7%	11	0.1%	14	0.2%	28	0.3%
2000	51	0.6%	29	0.3%	23	0.3%	1	0.0%	4	0.0%	4	0.0%
1999	198	2.3%	162	1.8%	84	1.0%	3	0.0%	0	0.0%	5	0.1%
1998	370	4.2%	285	3.2%	202	2.3%	8	0.1%	38	0.4%	43	0.5%
1997	254	2.9%	220	2.5%	84	1.0%	6	0.1%	4	0.0%	11	0.1%
1996	240	2.7%	218	2.5%	79	0.9%	7	0.1%	5	0.1%	19	0.2%
1995	300	3.4%	266	3.0%	71	0.8%	19	0.2%	30	0.3%	38	0.4%
1994	No Data Available											
1993	No Data Available											
1992	514	5.9%	509	5.8%	113	1.3%	14	0.2%	5	0.1%	24	0.3%
1991	532	6.1%	514	5.9%	166	1.9%	16	0.2%	12	0.1%	25	0.3%
1990	231	2.6%	223	2.5%	73	0.8%	13	0.1%	7	0.1%	12	0.1%
1989	231	2.6%	220	2.5%	64	0.7%	6	0.1%	11	0.1%	18	0.2%
1988	298	3.4%	271	3.1%	58	0.7%	19	0.2%	10	0.1%	33	0.4%
1987	551	6.3%	543	6.2%	39	0.4%	7	0.1%	4	0.0%	8	0.1%
1986	426	4.9%	400	4.6%	43	0.5%	6	0.1%	11	0.1%	15	0.2%
1985	615	7.0%	419	4.8%	1	0.0%	11	0.1%	163	1.9%	60	0.7%
1984	396	4.5%	380	4.3%	20	0.2%	5	0.1%	1	0.0%	17	0.2%
1983	479	5.5%	449	5.1%	47	0.5%	24	0.3%	14	0.2%	42	0.5%
1982	326	3.7%	315	3.6%	58	0.7%	15	0.2%	12	0.1%	19	0.2%
1981	174	2.0%	167	1.9%	33	0.4%	8	0.1%	11	0.1%	8	0.1%
1980	399	4.6%	382	4.4%	81	0.9%	14	0.2%	16	0.2%	19	0.2%
1979	449	5.1%	421	4.8%	103	1.2%	3	0.0%	12	0.1%	9	0.1%
Total	**8,381**	**3.7%**	**7,420**	**3.3%**	**1,993**	**0.9%**	**281**	**0.1%**	**532**	**0.2%**	**579**	**0.3%**

Pump operating logs for DPS 4 were obtained for the period 1980 to 2006. However, only 13 years of pump operating logs during this 27-year period of record had sufficient data available for analysis. As described in Table 2.3-2, DPS 4 has three 1,000 cfs capacity pumps and two 320 cfs pumps. It also has one 80 cfs constant duty pump that pumps dry weather flow to the Mississippi River. The pump operating data for the constant duty pump were not collected and are not included in the analysis.

The flow duration curve for DPS 4 is shown in Figure 5.1-4. The historical record shows that DPS 4 pumps to London Avenue Canal about 1.5 percent of the time, or about 130 hours per year on average.

Table 5.1-8 shows the magnitude of flows DPS 4 most frequently discharges to the London Avenue Canal when the pumps are operating. During the time the pumps are operating, the pumped discharge is most often less than 1,500 cfs. That is, about 90.5 percent of the operating time, DPS 4 discharges no more than 1,500 cfs to London Avenue Canal. The discharge from

DPS 4 has exceeded 3,000 cfs less than 1 hour per year on average. In other words, about 0.7 percent of the time the pumps are in use the discharge exceeds 3,000 cfs. Again, the current capacity of DPS 4 is 3,720 cfs, with 80 cfs of that not included in this analysis.

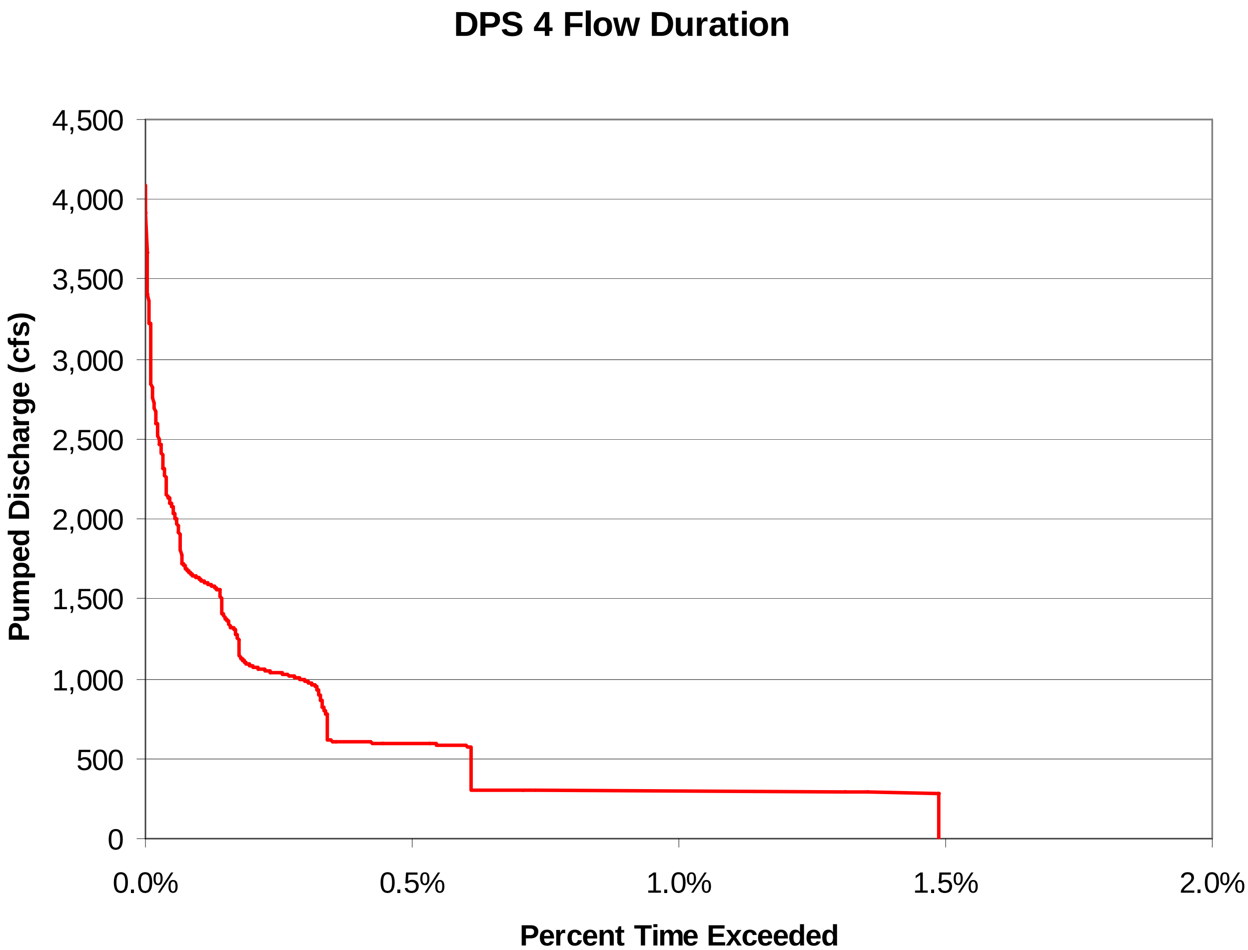


Figure 5.1-4. London Avenue Canal: DPS 4 Pumped Flow Duration Curve

Table 5.1-9 shows the total hours of operating time for all but the constant duty pump at DPS 4. For each year the table indicates the number of hours a pump operated and the percent of total time those hours represent. For example, in 2003 Pump C operated 28 hours over the course of the year, or 0.3 percent of the time. Over the period of record analyzed, the DPS 4 pumps operated most frequently in 1990 for a total of 371 hours and least frequently in 1986 for 11 hours. Pump 2 has a capacity of 320 cfs and is operated most frequently in comparison to the other pumps at DPS 4.

Table 5.1-8. London Avenue Canal - DPS 4 Annual Discharges

Year	Hours Pumping at Given Flows | Percent Time Pumping within Flow Range									
	0 – 500 cfs		501 – 1,000 cfs		1,001 – 2,000 cfs		2,001 – 3,000 cfs		3,000+ cfs	
2006	15	59%	4	16%	6	24%	1	2%	0	0%
2005	45	45%	29	29%	17	17%	8	8%	2	2%
2004	44	47%	25	27%	21	22%	4	4%	0	0%
2003	28	33%	30	34%	19	22%	8	9%	2	2%
2002	54	50%	27	25%	22	21%	5	4%	0	0%
2001	46	38%	24	20%	39	32%	8	6%	4	3%
2000	No Data Available									
1999	No Data Available									
1998	No Data Available									
1997	No Data Available									
1996	No Data Available									
1995	No Data Available									
1994	151	74%	21	10%	24	12%	8	4%	1	0%
1993	149	62%	63	26%	26	11%	5	2%	0	0%
1992	No Data Available									
1991	No Data Available									
1990	266	72%	80	22%	24	6%	2	0%	1	0%
1989	183	71%	47	18%	21	8%	3	1%	3	1%
1988	Insufficient Data									
1987	Insufficient Data									
1986	4	36%	1	9%	6	55%	0	0%	0	0%
1985	16	30%	10	19%	23	43%	4	8%	0	0%
1984	0	0%	0	0%	26	100%	0	0%	0	0%
1983	Insufficient Data									
1982	Insufficient Data									
1981	Insufficient Data									
1980	Insufficient Data									
Total	**1,000**	**59%**	**365**	**22%**	**265**	**16%**	**53**	**3%**	**12**	**0.7%**

Table 5.1-9. London Avenue Canal - DPS 4 Annual Pumping Operation

Year	Hours Pump On | Percent Time Pumping											
	Total		Pump C (1,000 cfs)		Pump D (1,000 cfs)		Pump E (1,000 cfs)		Pump 1 (320 cfs)		Pump 2 (320 cfs)	
2006	26	0.3%	6	0.1%	7	0.1%	0	0.0%	16	0.2%	1	0.0%
2005	100	1.1%	15	0.2%	11	0.1%	22	0.3%	46	0.5%	52	0.6%
2004	94	1.1%	6	0.1%	8	0.1%	19	0.2%	41	0.5%	64	0.7%
2003	86	1.0%	28	0.3%	12	0.1%	16	0.2%	36	0.4%	40	0.5%
2002	107	1.2%	21	0.2%	17	0.2%	14	0.2%	45	0.5%	47	0.5%
2001	121	1.4%	17	0.2%	35	0.4%	23	0.3%	75	0.9%	37	0.4%
2000	No Data Available											
1999	No Data Available											
1998	No Data Available											
1997	No Data Available											
1996	No Data Available											
1995	No Data Available											
1994	204	2.3%	23	0.3%	9	0.1%	9	0.1%	61	0.7%	170	1.9%
1993	242	2.8%	22	0.3%	10	0.1%	3	0.0%	138	1.6%	189	2.2%
1992	No Data Available											
1991	No Data Available											
1990	371	4.2%	5	0.1%	21	0.2%	3	0.0%	148	1.7%	326	3.7%
1989	257	2.9%	16	0.2%	12	0.1%	8	0.1%	86	1.0%	235	2.7%
1988	Insufficient Data											
1987	Insufficient Data											
1986	11	0.1%	1	0.0%	6	0.1%	0	0.0%	6	0.1%	2	0.0%
1985	52	0.6%	10	0.1%	5	0.1%	19	0.2%	19	0.2%	19	0.2%
1984	26	0.3%	0	0.0%	1	0.0%	25	0.3%	1	0.0%	1	0.0%
1983	Insufficient Data											
1982	Insufficient Data											
1981	Insufficient Data											
1980	Insufficient Data											
Total	**1,694**	**1.5%**	**167**	**0.1%**	**152**	**0.1%**	**159**	**0.1%**	**709**	**0.6%**	**1175**	**1.0%**

5.2 Pumped Flow Frequency

Future conditions flow-frequency curves were developed by estimating increases in the annual peak pumped discharges to each canal. Annual peak pumped discharges for the existing drainage pump stations were provided by HPO and are contained in Table 3.5-1. These were used to develop pumped discharge-frequency curves, or pumped discharge exceedance probability curves, shown in Figure 3.5-1 through Figure 3.5-4. The discharge-frequency curves in Section 3.5 only take into account the pumped flows from existing drainage pump stations DPS 3, DPS 4, DPS 6, and DPS 7. The curves do not account for the Canal Street Pump Station and I10 Pump Station which currently pump to 17th Street Canal. Nor do they account for planned future drainage pump station capacity increases at DPS 6 and DPS 7 or the planned future installation of an additional drainage pump station along London Avenue Canal. All of these discharges excluded from the existing drainage pump station discharges shown in Section 3.5 are accounted for in the analysis described herein.

5.2.1 17th Street Canal

The estimated future pumped flow frequency curve for 17th Street Canal was developed using HEC-SSP, Statistical Software Package. The estimated curve accounts for the discharges from Canal Street Pump Station and I-10 Pump Station, as well as the 2,000 cfs capacity increase at DPS 6.

A series of annual peak pumped flows for future discharge capacity to 17th Street Canal was developed for use with HEC-SSP. These future annual peak pumped discharges were based on the historic DPS 6 annual peak flows contained in Table 3.5-1. The historic annual peak flows were analyzed to determine the fraction of DPS 6 capacity discharged to the canal. Since the constant duty pumps were not included in the analysis, the current maximum capacity of DPS 6 was assumed to be 9,300 cfs. It was then assumed that the same fraction of Canal Street and I-10 Pump Station capacities (less constant duty) would be used during a particular event. To account for the 2,000 cfs capacity increase at DPS 6, any DPS 6 peak flows exceeding 9,000 cfs were increased by a fraction of the proposed capacity increase.

Table 5.2-1 shows the future conditions pumped flow frequency, or pumped discharge exceedance probability, distribution. The expected probability flow curve attempts to correct for a certain bias in the frequency curve computation due to the shortness of the record. The confidence limits are calculated from the computed curve flow values. Because HEC-SSP does not allow an upper threshold on its frequency curve computations, the 0.2-percent chance exceedance event exceeds the 12,500 cfs maximum design capacity for 17th Street Canal. This event would be limited to the maximum design discharge.

Based on the assumptions described above, there is about a 0.5 percent annual chance that the 17th Street Canal discharge will meet the maximum discharge of 12,500 cfs. This corresponds to a 200-year return period for pumping. Pumped discharges do not necessarily correlate to a particular rainfall event; rather the results described here are based on a statistical analysis of historic operation of DPS 6.

Table 5.2-1. 17th Street Canal Future Conditions Estimated Pumped Discharge Exceedance Probability

Percent Chance Exceedance	Computed Curve Pumping (cfs)	Expected Probability Pumping (cfs)	Computed Confidence Limits	
			0.05	0.95
0.2	12,400	12,500[15]	14,890	11,050
0.5	11,840	12,430	14,010	10,630
1	11,387	11,840	13,320	10,290
2	10,904	11,230	12,590	9,920
5	10,200	10,390	11,560	9,380
10	9,597	9,710	10,700	8,890
20	8,895	8,950	9,740	8,300
50	7,643	7,640	8,180	7,150
80	6,509	6,460	6,970	5,950
90	5,964	5,880	6,440	5,340
95	5,538	5,410	6,040	4,860
99	4,797	4,550	5,360	4,030

5.2.2 Orleans Avenue Canal

The estimated future pumped flow frequency curve for Orleans Avenue Canal was developed using HEC-SSP, Statistical Software Package. The estimated curve accounts for the 700 cfs capacity increase at DPS 7.

A series of annual peak pumped flows for the future discharge capacity to Orleans Avenue Canal was developed for use with HEC-SSP. These future annual peak pumped discharges were based on the historic DPS 7 annual peak flows contained in Table 3.5-1. The historic annual peak flows were analyzed to determine the fraction of DPS 7 capacity discharged to the canal. Since the constant duty pumps were not included in the analysis, the current maximum capacity of DPS 7 was assumed to be 2,550 cfs. To account for the 700 cfs capacity increase at DPS 7, any DPS 7 peak flows exceeding 2,500 cfs were increased by a fraction of the proposed capacity increase.

Table 5.2-2 shows the future conditions pumped flow frequency distribution. The expected probability flow curve attempts to correct for a certain bias in the frequency curve computation due to the shortness of the record. The confidence limits are calculated from the computed curve flow values. Because HEC-SSP does not allow an upper threshold on its frequency curve computations, the 0.2-percent chance exceedance event exceeds the 3,390 cfs maximum design capacity for Orleans Avenue Canal. This event would be limited to the maximum design discharge.

Based on the assumptions described above, there is about a 0.5 percent annual chance that the Orleans Avenue Canal discharge will meet the maximum discharge of 3,390 cfs. This corresponds to a 200-year return period for pumping. Pumped discharges do not necessarily correlate to a particular rainfall event; rather the results described here are based on a statistical analysis of historic operation of DPS 7.

[15] The proposed maximum discharge capacity of the pump stations discharging to 17th Street Canal is 12,500 cfs. Therefore, the expected flow will not exceed 12,500 cfs, unless pump station capacity is further increased.

Table 5.2-2. Orleans Avenue Canal Future Conditions Estimated Pumped Discharge Exceedance Probability

Percent Chance Exceedance	Computed Curve Pumping (cfs)	Expected Probability Pumping (cfs)	Computed Confidence Limits	
			0.05	0.95
0.2	3,376	3,420	4,000	3,010
0.5	3,334	3,380	3,940	2,980
1	3,290	3,330	3,870	2,940
2	3,231	3,270	3,780	2,900
5	3,120	3,150	3,620	2,810
10	2,997	3,020	3,440	2,710
20	2,816	2,830	3,190	2,560
50	2,385	2,390	2,630	2,180
80	1,880	1,860	2,060	1,670
90	1,608	1,560	1,790	1,380
95	1,390	1,320	1,580	1,140
99	1,011	890	1,210	750

5.2.3 London Avenue Canal

The estimated future pumped flow frequency curve for London Avenue Canal was developed using HEC-SSP, Statistical Software Package. The estimated curve accounts for an additional discharge pump station with 1,000 cfs capacity.

The first step in the London Avenue Canal analysis was to combine the DPS 3 and DPS 4 annual peak pumped discharges. The pumping logs provided by HPO were utilized to calculate the 30-minute combined DPS 3 and DPS 4 discharges for the period of record. The peak annual flows for London Avenue Canal were then obtained. Because of the short period of record for DPS 4 and insufficient data for some years at DPS 3, only 11 years of annual peak discharges were used in the analysis.

Future annual peak pumped discharges were based on the annual peak flows described above. These were analyzed to determine the fraction of DPS 3 and DPS 4 capacity discharged to the canal. Since the constant duty pumps were not included in the analysis, the current combined maximum capacity of DPS 3 and DPS 4 was assumed to be 7,740 cfs. To account for the proposed additional 1,000 cfs pump station, any combined DPS 3 and DPS 4 peak flows exceeding 7,700 cfs were increased by a fraction of the proposed capacity increase.

Table 5.2-3 shows the future conditions pumped flow frequency distribution. The expected probability flow curve attempts to correct for a certain bias in the frequency curve computation due to the shortness of the record. The confidence limits are calculated from the computed curve flow values. Because HEC-SSP does not allow an upper threshold on its frequency curve computations, the 2-percent chance exceedance event exceeds the 8,980 cfs maximum design capacity for London Avenue Canal. This event and any less frequent events would be limited to the maximum design discharge.

Based on the assumptions described above, there is about a 5 percent annual chance that the London Avenue Canal discharge will meet the maximum discharge of 8,980 cfs. This corresponds to a 20-year return period for pumping. Pumped discharges do not necessarily correlate to a particular rainfall event; rather the results described here are based on a statistical analysis of historic operation of DPS 3 and DPS 4.

Table 5.2-3. London Avenue Canal Future Conditions Estimated Pumped Discharge Exceedance Probability

Percent Chance Exceedance	Computed Curve Pumping (cfs)	Expected Probability Pumping (cfs)	Computed Confidence Limits	
			0.05	0.95
0.2	10,382[16]	8,980[16]	15,470	8,470
0.5	9,900[16]	8,980[16]	14,380	8,150
1	9,495[16]	8,980[16]	13,500	7,890
2	9,047[16]	8,980[16]	12,550	7,590
5	8,370	8,780	11,170	7,120
10	7,767	8,010	10,010	6,680
20	7,043	7,160	8,710	6,130
50	5,707	5,710	6,620	4,950
80	4,478	4,370	5,140	3,640
90	3,893	3,700	4,540	2,980
95	3,443	3,150	4,110	2,480
99	2,687	2,150	3,380	1,700

5.3 Hydrologic Modeling

The Vicksburg District Corps of Engineers (MVK) developed discharge hydrographs at the existing drainage pump stations (DPS) along the 17th Street, London Avenue, and Orleans Avenue Canals by coupling hydrologic and hydraulic models initially developed by the Interagency Performance Evaluation Task Force (IPET). The Hydrologic Engineering Center's Hydrologic Modeling System (HEC-HMS) and unsteady River Analysis System (HEC-RAS) were used to conduct the hydrologic modeling described herein. The models were modified from those used by the IPET to include additional storage areas and updated canal geometry.

The following seven different recurrence interval events were modeled: 1-, 2-, 5-, 10-, 25-, 50-, and 100-year, 24-hour storm events. The rainfall used for the synthetic storm events was determined from the US Department of Commerce Technical Paper 40 (1961). Precipitation values used in the modeling are shown in Table 5.3-1 and Table 5.3-2.

[16] The proposed maximum discharge capacity of the pump stations discharging to London Avenue Canal is 8,980 cfs. Therefore, the expected flow will not exceed 8,980 cfs, unless pump station capacity is further increased.

Table 5.3-1. 24-Hour Duration Rainfall Used for Hydrologic Modeling

Storm Return Period (Yr)	TP-40 24-Hour Duration Rainfall (Inches)
1	4.52
2	5.48
5	7.39
10	8.68
25	10.08
50	11.14
100	12.62

Table 5.3-2. Rainfall Intensity Used for Hydrologic Modeling

Duration	TP-40 Rainfall Intensity for Given Return Period (inches)						
	1-Yr	2-Yr	5-Yr	10-Yr	25-Yr	50-Yr	100-Yr
5 min	0.41	0.46	0.57	0.64	0.72	0.78	0.85
15 min	1.25	1.46	1.75	1.94	2.20	2.36	2.67
1 hr	2.03	2.31	2.84	3.22	3.59	3.88	4.26
2 hr	2.50	2.88	3.61	4.15	4.74	5.24	5.80
3 hr	2.78	3.24	4.09	4.75	5.52	6.06	6.62
6 hr	3.33	3.94	5.09	6.01	6.86	7.53	8.55
12 hr	3.84	4.78	6.31	7.31	8.51	9.39	10.50
24 hr	4.52	5.48	7.39	8.68	10.08	11.14	12.62

The flows modeled by HEC-HMS were routed to storage areas in HEC-RAS and then pumped through existing drainage pump stations to the 17th Street, Orleans Avenue, and London Avenue Canals. The total drainage area modeled was 42.8 square miles. The drainage areas of the pump stations included in the coupled HEC-HMS/HEC-RAS model are shown in Table 5.3-3. These values are approximate because some storage areas included in the HEC-RAS model supplied multiple pump stations.

Table 5.3-3. Modeled Drainage Areas of Outfall Canals (Estimated)

Drainage Pump Station	Discharges Water To:	Estimated Modeled Drainage Area (square miles)
6	17th Street Canal	16.3
I-10	17th Street Canal	0.9
	17th Street Canal Modeled Drainage Area	*17.2*
7	Orleans Avenue Canal	8.9
	Orleans Avenue Canal Modeled Drainage Area	*8.9*
3	London Avenue Canal	2.2
4	London Avenue Canal	3.5
	London Avenue Canal Modeled Drainage Area	*5.7*

The pump efficiency curves and pump on/off elevations used to model the existing pump stations were obtained from the New Orleans Sewerage and Water Board (S&WB). Two downstream boundary conditions were modeled at Lake Pontchartrain: 1.0 ft NAVD88 (2004.65) constant elevation to represent "normal" lake stage and a 100-year stage hydrograph developed by New Orleans District Corps of Engineers (MVN) using a lake surge model. For the second boundary condition, the time to peak was set to coincide with peak modeled pump station discharge.

The modeled peak drainage pump station discharges are shown in Table 5.3-4 and Table 5.3-5. Modeled discharge frequency curves for each outfall canal are shown in Figure 5.3-1 through Figure 5.3-3. The predicted discharges assume the existing pump stations are operated as prescribed by the Sewerage and Water Board of New Orleans (S&WB) and do not include proposed capacity increases for the canals.

Modeled flow hydrographs discharged from each DPS and at the outfall of each canal are included in the Appendix, Section A-1.

The maximum design discharge specified for the permanent pump stations is 12,500 cfs, 3,390 cfs, and 8,980 cfs for the 17th Street Canal, Orleans Avenue Canal, and London Avenue Canal, respectively. As can be seen from the hydrologic modeling results, the maximum design discharges exceed the predicted 1-percent-annual-chance exceedance discharge in each canal.

Table 5.3-4. Modeled Peak Pump Station Discharges for Lake Pontchartrain Elevation 1.0 ft NAVD88 (2004.65)

Return Period (Yr)	Percent Annual Chance Exceedance	17th Street[17] (DPS6+I10)	Orleans Avenue (DPS 7)	London Avenue (DPS3+4)
1	99.99	4,430	920	2,190
2	50	5,490	1,120	2,760
5	20	6,900	1,640	3,670
10	10	7,860	1,910	3,870
25	4	8,660	2,050	4,250
50	2	8,960	2,090	4,410
100	1	9,330	2,120	4,790

The modeled flows are lower than those developed from annual peak discharges and shown in Section 3.5. With the current model, it is not possible to generate a flow hydrograph that is not dampened by the HEC-RAS storage areas and restricted by the interior drainage system.

[17] The Canal Street pump station with capacity 160 cfs was not included in the modeled hydrograph for 17th Street Canal.

Table 5.3-5. Modeled Peak Pump Station Discharges for 100-Year Hydrograph at Lake Pontchartrain

Return Period (Yr)	Percent Annual Chance Exceedance	17th Street[17] (DPS6+I10)	Orleans Avenue (DPS 7)	London Avenue (DPS3+4)
1	99.99	4,420	860	2,040
2	50	5,320	900	2,670
5	20	6,220	1,020	3,350
10	10	6,700	1,090	3,650
25	4	7,170	1,350	4,030
50	2	7,470	1,490	4,220
100	1	7,820	1,600	4,400

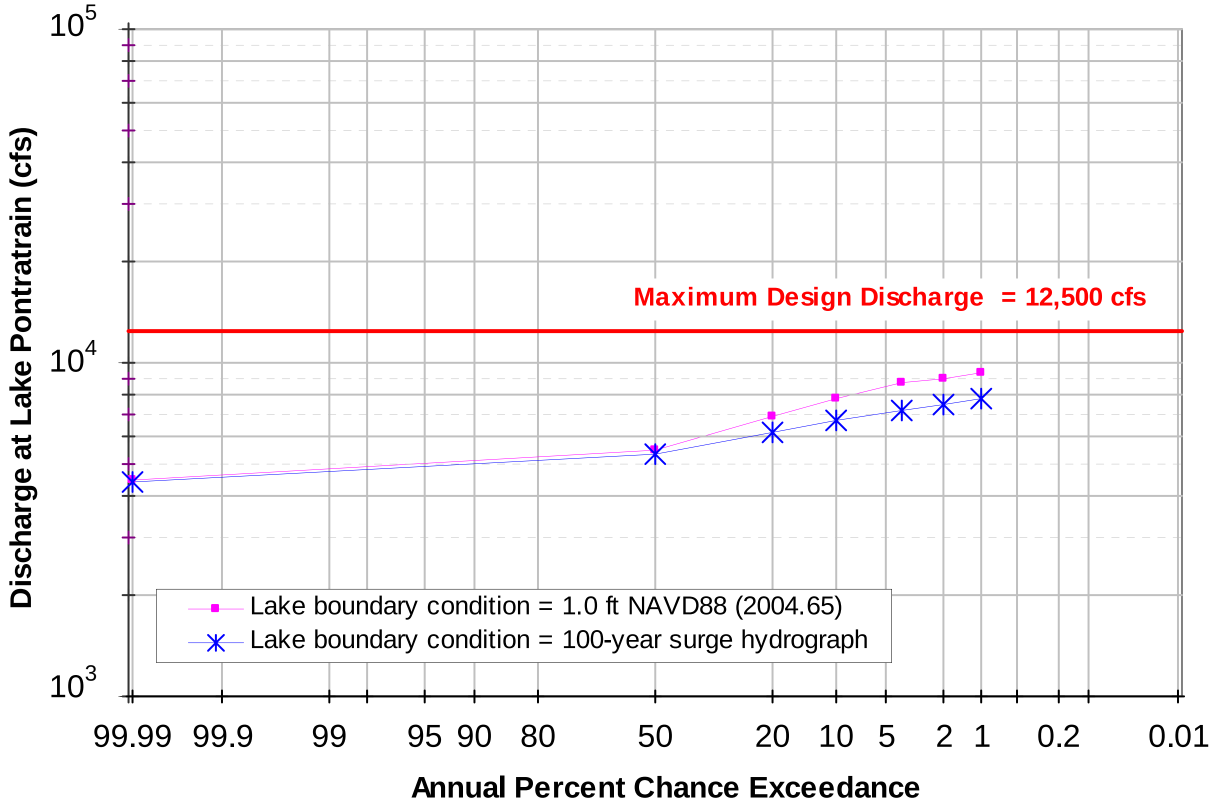


Figure 5.3-1. **17th Street Canal (DPS 6 + I-10) Modeled Discharge Frequency Curve**

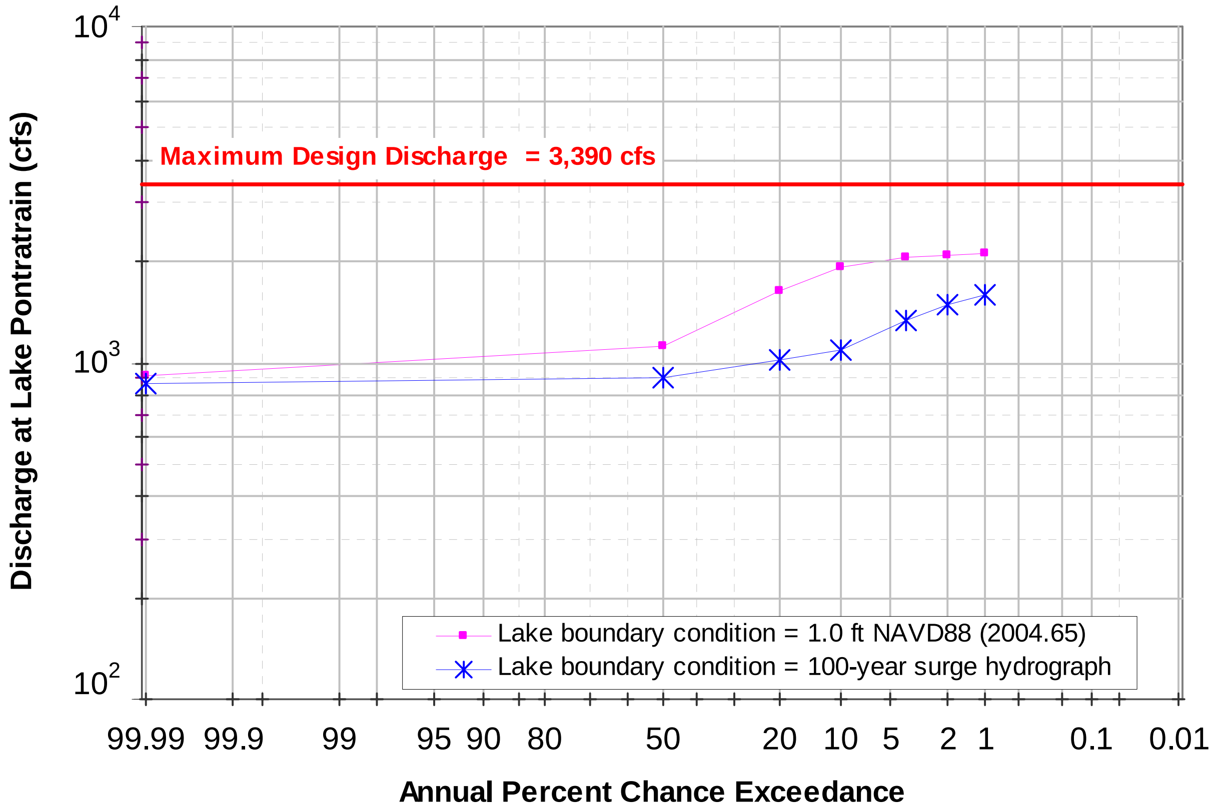


Figure 5.3-2. Orleans Avenue Canal (DPS 7) Modeled Discharge Frequency Curve

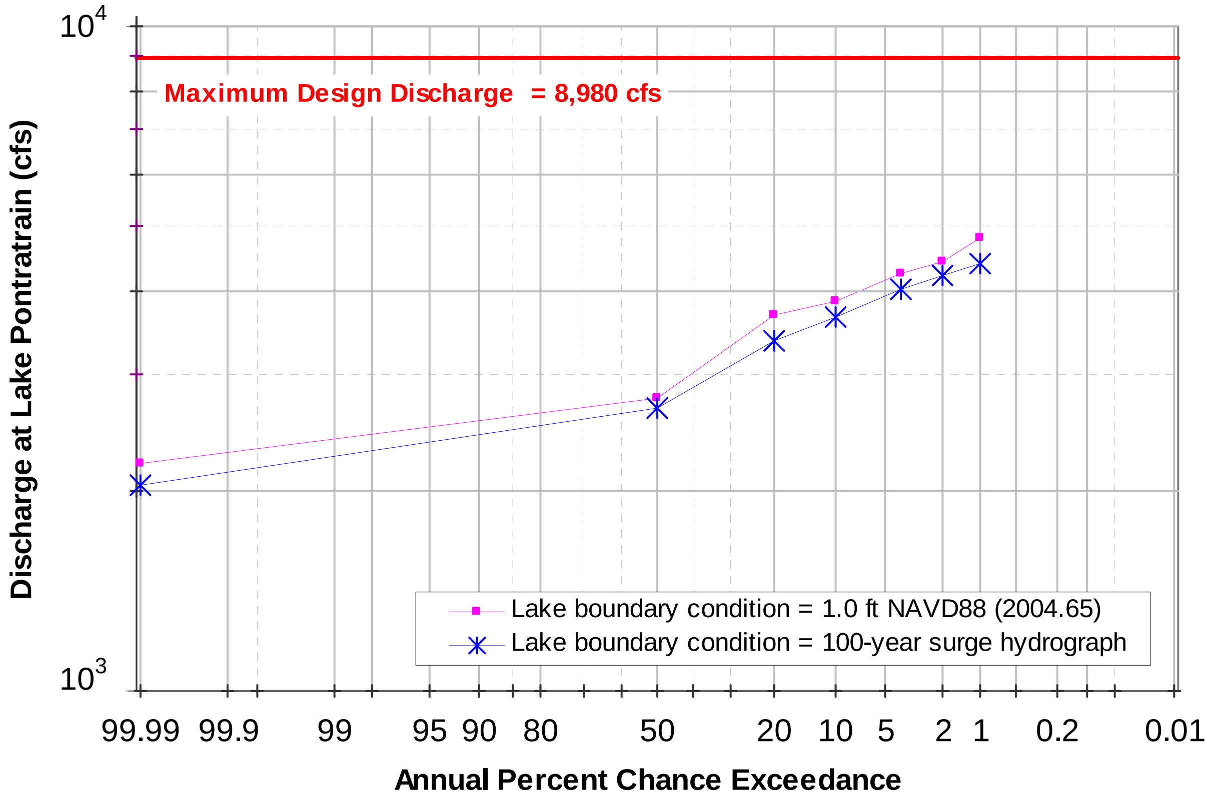


Figure 5.3-3. London Avenue Canal (DPS 3 + 4) Modeled Discharge Frequency Curve

6.0 HYDRAULIC MODELING

Hydraulic modeling was performed by MVK to determine canal water surface elevations resulting from a range of existing drainage pump station discharges and Lake Pontchartrain boundary conditions. The HEC-RAS model initially developed by the Interagency Performance Evaluation Task Force (IPET) was modified, including updating the canal geometry to incorporate the data described in Section 3.4. The data include 2006 multi-beam surveys collected along all three canals, 2007 bridge surveys on 17th Street and Orleans Avenue Canals, a 2007 bank erosion survey along 17th Street Canal, and London Avenue Canal 2007 test site surveys. The model was set up to be run in both steady-state and unsteady-state modes.

Inflows to the canal were determined based on the hydrologic modeling described above and the prescribed permanent pump station capacities. For each canal the permanent pump station location was assumed to be at the interim closure station location. The drainage pump stations pump efficiency curves and pump on/off elevations were obtained from the S&WB.

The hydraulic model for 17th Street Canal includes k-loss factors determined during physical modeling at the US Army Corps of Engineers Engineer Research and Development Center (ERDC). The Manning's n-values included in the London Avenue Canal model were calibrated to a December 30, 2006, rainfall and DPS 3 pumping event.

6.1 Rating Curves at Existing Drainage Pump Stations

For a range of canal discharges and Lake Pontchartrain water surface elevations, the maximum water surface elevation in each canal was determined and compared to the safe water elevation. Gates remained open and no permanent pumps were operated during the analysis. The canal discharges ranged from that corresponding to the 1-year rainfall event (approximately) to the maximum design discharge for each canal. The lake elevations ranged from 0.0 ft to 8.0 ft NAVD88 (2004.65), in 1-ft intervals.

Rating curves for each canal are shown in Figure 6.1-1 through Figure 6.1-4. Rating curves usually refer to the elevation-discharge relationship at a single location. For this analysis, the elevation is at the DPS and the discharge is the total canal discharge at Lake Pontchartrain.

6.1.1 17th Street Canal

Currently, 17th Street Canal safe water elevations vary from 6.5 to 13.3 ft NAVD88 (2004.65) along the canal. The analyses were performed for a proposed minimum safe water elevation of 8.0 ft.

Hydraulic model results for the 17th Street Canal indicate the canal can pass the 99-percent annual chance pumped discharge for Lake Pontchartrain elevations up to about 7.6 ft NAVD88 (2004.65) without exceeding safe water elevation anywhere along the canal. This corresponds to a 1.01-year return period. The canal can pass the maximum design discharge of 12,500 cfs for lake elevations up to 4.5 ft without exceeding safe water elevations.

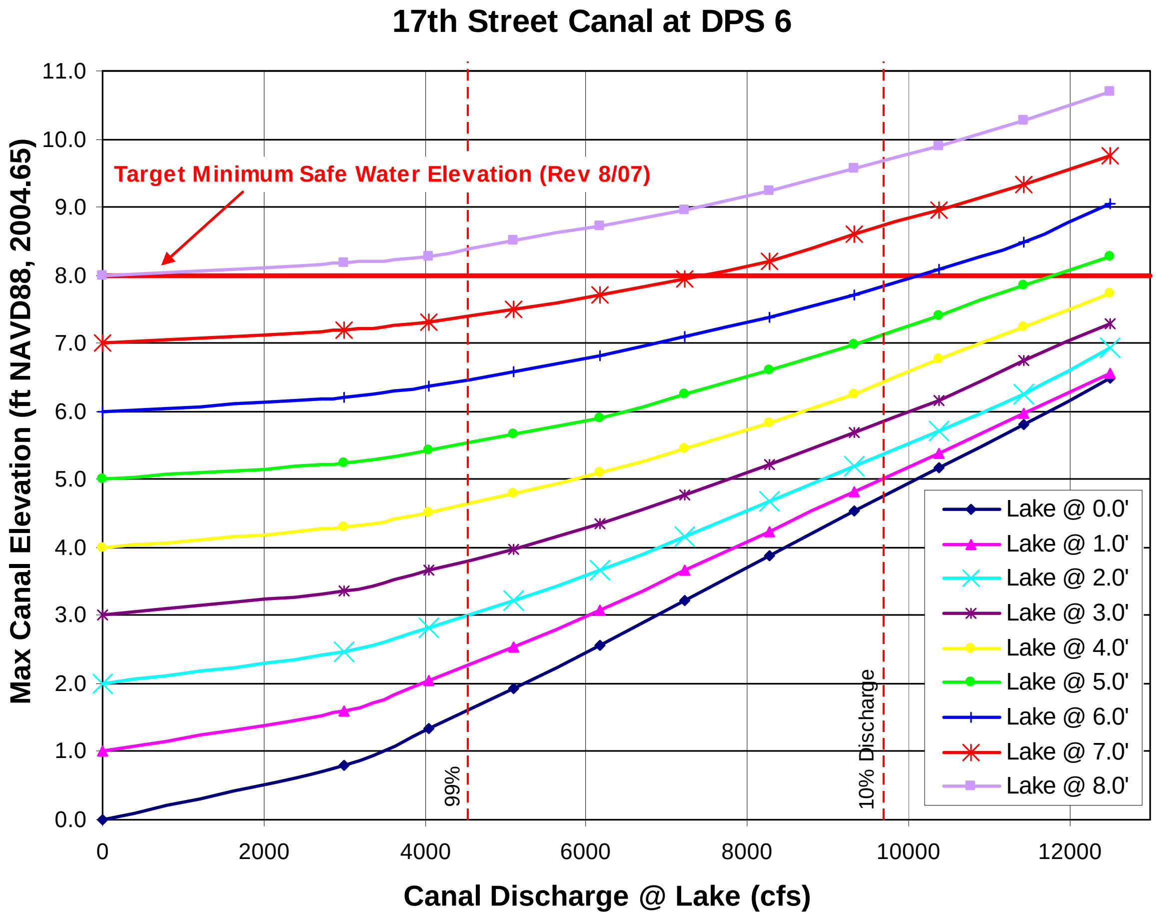


Figure 6.1-1. Water Surface Elevation at DPS 6 for Various Combinations of Lake Pontchartrain Elevation and Canal Discharge – 17th Street Canal

6.1.2 Orleans Avenue Canal

The results for Orleans Avenue Canal indicate the canal can pass the 99-percent annual chance pumped discharge (1.01-year return period) for Lake Pontchartrain elevations up to 7.9 ft NAVD88 (2004.65). The canal can pass the maximum design discharge of 3,390 cfs for lake elevations up to 7.1 ft without exceeding safe water elevations.

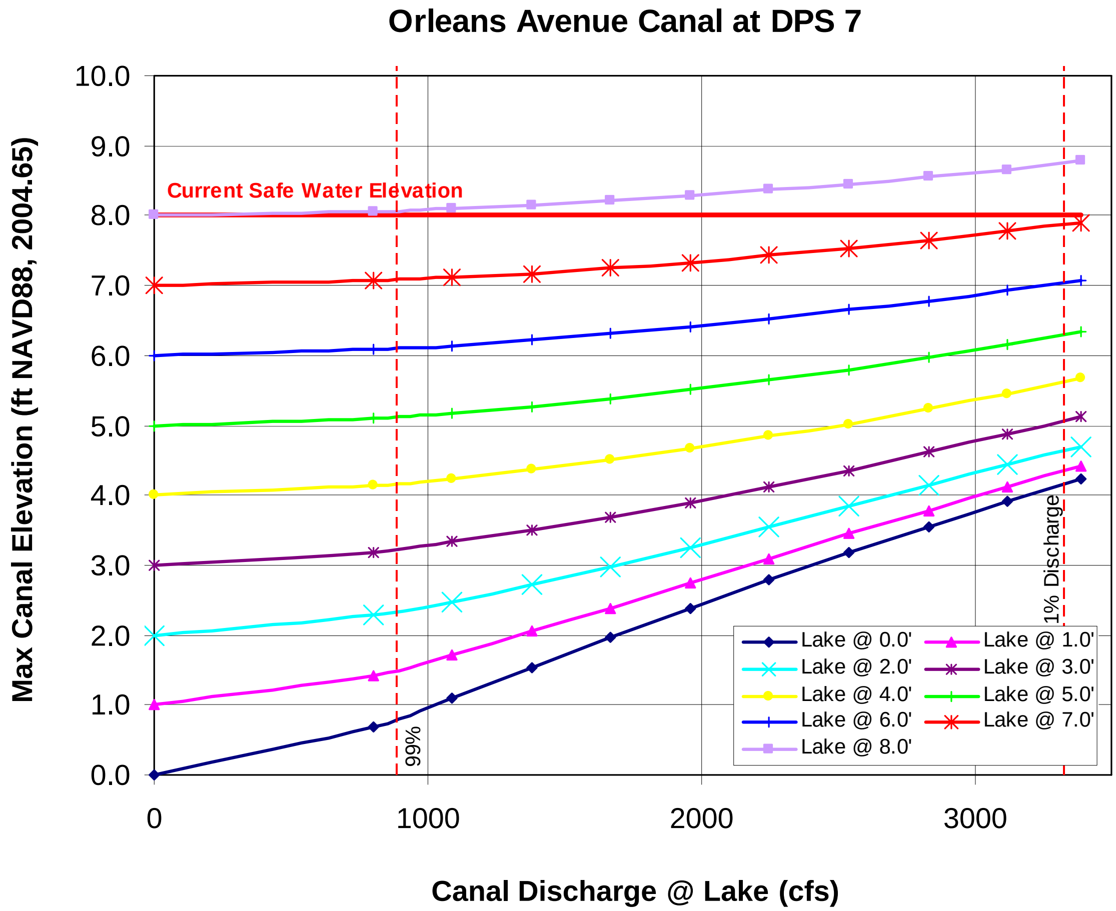


Figure 6.1-2. Water Surface Elevation at DPS 7 for Various Combinations of Lake Pontchartrain Elevation and Canal Discharge – Orleans Avenue Canal

6.1.3 London Avenue Canal

The London Avenue Canal analyses were performed for safe water elevations at two locations along the canal. The first location is at the upstream end of the canal, just downstream of DPS 3. From there to about 4,000 feet downstream of DPS 3 the canal has a safe water elevation of 9.0 ft NAVD88 (2004.65). The remainder of the canal currently has a safe water elevation of 5.0 ft and a proposed minimum safe water elevation of 8.0 ft. The second location is about 4,000 feet downstream of DPS 3. Results are reported for these two locations as “London Avenue Canal at DPS 3” and “London Avenue Canal 4000 ft Downstream of DPS 3.”

The results indicate for target safe water elevation 8.0 ft the canal can pass the 99-percent annual chance pumped discharge (1.01-year return period) for Lake Pontchartrain elevations up to about 7.7 ft without exceeding safe water elevations anywhere along the canal. The canal can pass the 10-percent annual chance discharge (10-year return period) for lake elevations up to 4.8 ft and the maximum design discharge of 8,980 cfs for lake elevations up to 3.0 ft without exceeding the safe water elevation anywhere along the canal.

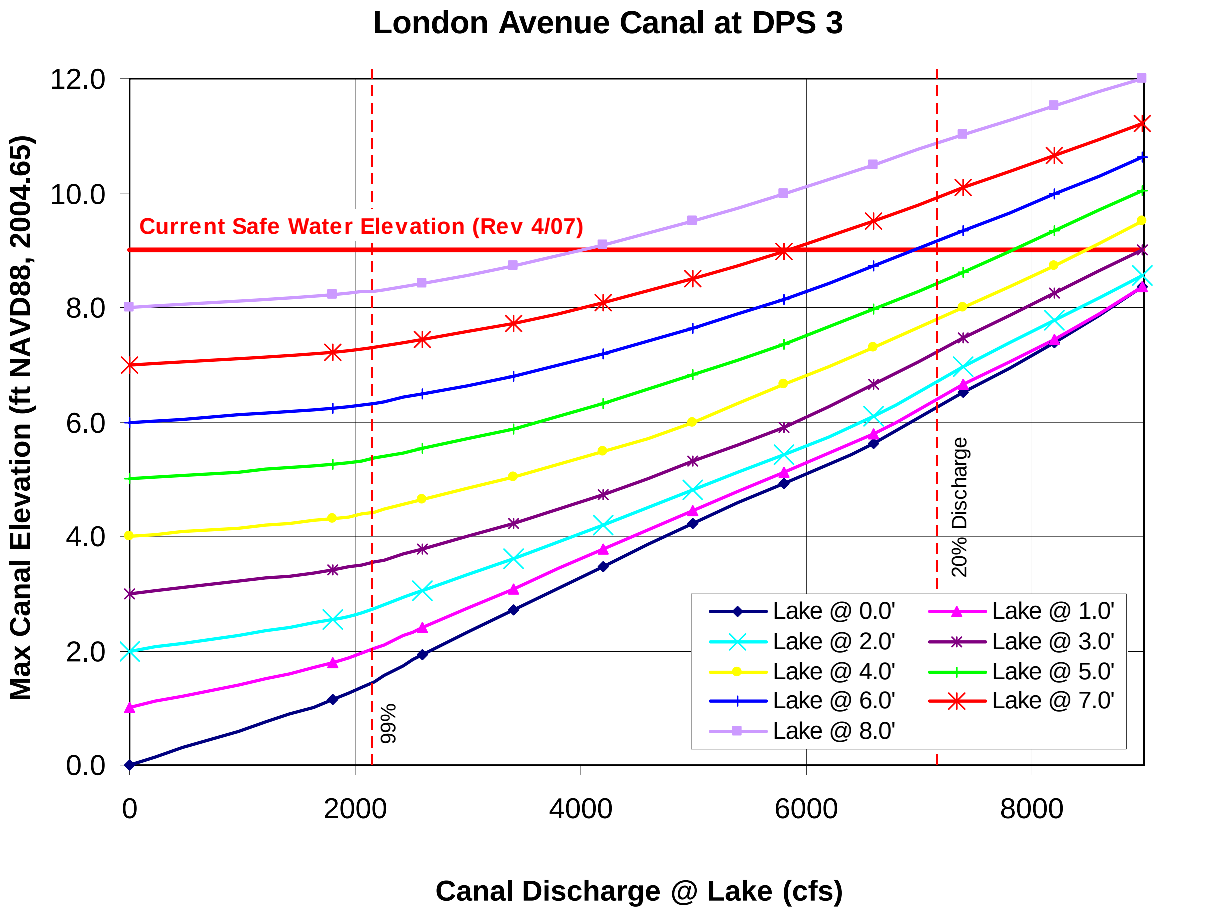


Figure 6.1-3. Water Surface Elevation at DPS 3 for Various Combinations of Lake Pontchartrain Elevation and Canal Discharge – London Avenue Canal

The limiting safe water elevation location for London Avenue Canal depends on the canal discharge. For canal discharges below 7,000 cfs, the modeled water surface elevation 4,000 ft downstream of DPS 3 exceeds 8.0 ft while the water surface elevation at DPS 3 remains below 9.0 ft. For canal discharges above 7,000 cfs, the modeled water surface elevation exceeds the 9.0 ft at DPS 3 while the water surface elevation 4,000 ft downstream of DPS 3 remains below 8.0 ft.

The hydraulic model shows the water surface elevations for the maximum design discharge increase about 0.6 ft across Gentilly Boulevard Bridge and 0.4 ft across the I-610 Bridge. If the bridges were improved or the safe water elevation increased just downstream of DPS 3, the limiting safe water elevation might be shifted to 4,000 ft downstream of DPS 3. Then London Avenue Canal could pass the maximum design discharge for Lake Pontchartrain elevations up to 4.3 ft NAVD88 (2004.65).

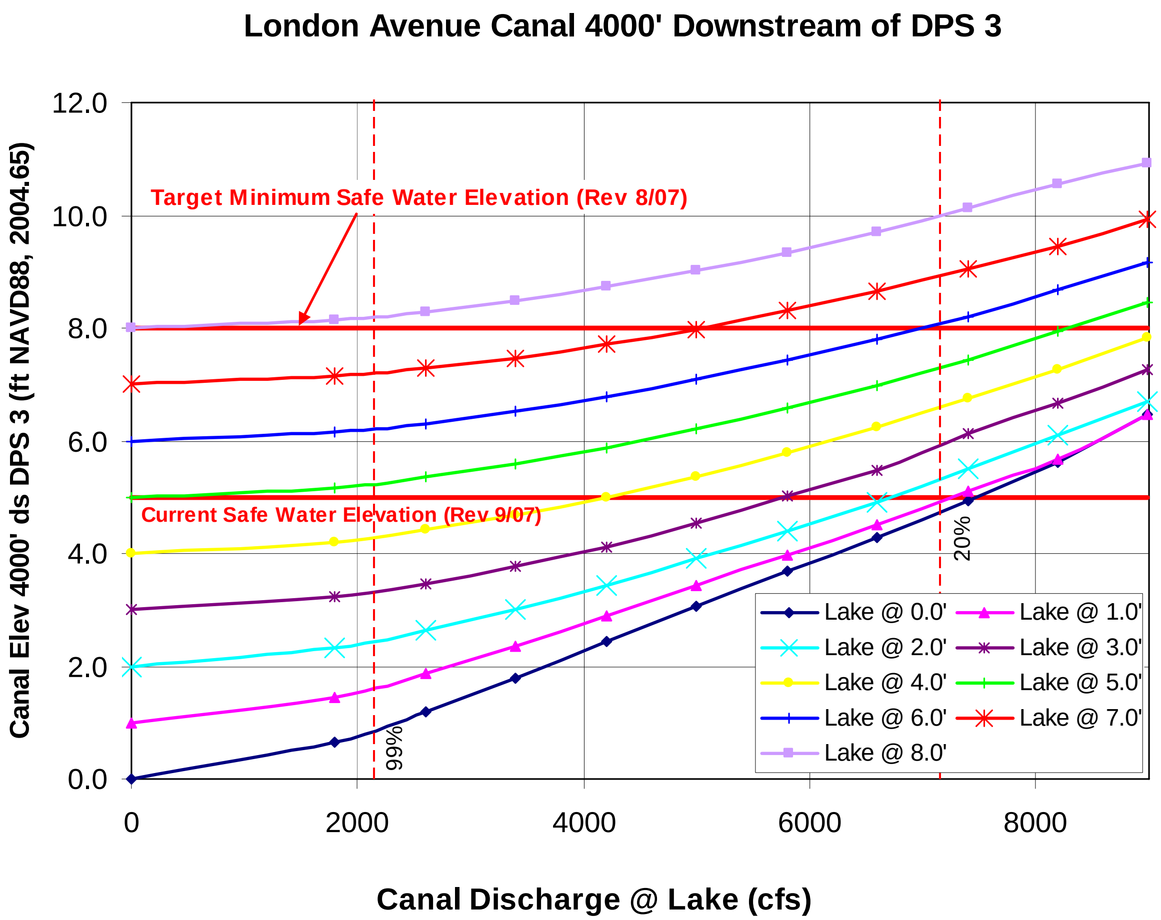


Figure 6.1-4. Water Surface Elevation Approximately 4,000 ft Downstream of DPS 3 for Various Combinations of Lake Pontchartrain Elevation and Canal Discharge – London Avenue Canal

6.2 Water Surface Profiles

The hydraulic model was used to determine the maximum suction side elevation at the permanent pump stations for a range of canal discharges such that safe water elevation is not exceeded anywhere along the canal. Canal discharges evaluated were approximately 25-, 50-, 75-, and 100-percent of maximum design discharge. Results of the analyses are shown in Table 6.2-1. Water surface profiles for each canal are shown in Figure 6.2-1 through Figure 6.2-3.

Table 6.2-1. Permanent Pump Station Maximum Suction Side Elevations for a Range of Canal Discharges

17th Street Canal		
Canal Discharge (cfs)	Permanent Pump Station Suction Side Elevation (ft NAVD88, 2004.65)	Max Canal Water Surface Elevation (ft NAVD88, 2004.65)
3,000	7.8	8.0
6,170	7.3	8.0
9,330	6.2	8.0
12,500	4.2	8.0

Orleans Avenue Canal		
Canal Discharge (cfs)	Permanent Pump Station Suction Side Elevation (ft NAVD88, 2004.65)	Max Canal Water Surface Elevation (ft NAVD88, 2004.65)
800	7.9	8.0
1,670	7.8	8.0
2,540	7.5	8.0
3,390	7.2	8.0

London Avenue Canal		
Canal Discharge (cfs)	Permanent Pump Station Suction Side Elevation (ft NAVD88, 2004.65)	Max Canal Water Surface Elevation[18] (ft NAVD88, 2004.65)
1,800	7.9	8.1
4,200	7.4	8.4
6,600	6.5	8.9
8,980	4.1	9.0

[18] For London Avenue Canal discharges less than 7,000 cfs, the canal water surface elevation is limited by safe water elevation 8.0 ft NAVD88 (2004.65), approximately 4,000 feet downstream of DPS 3.

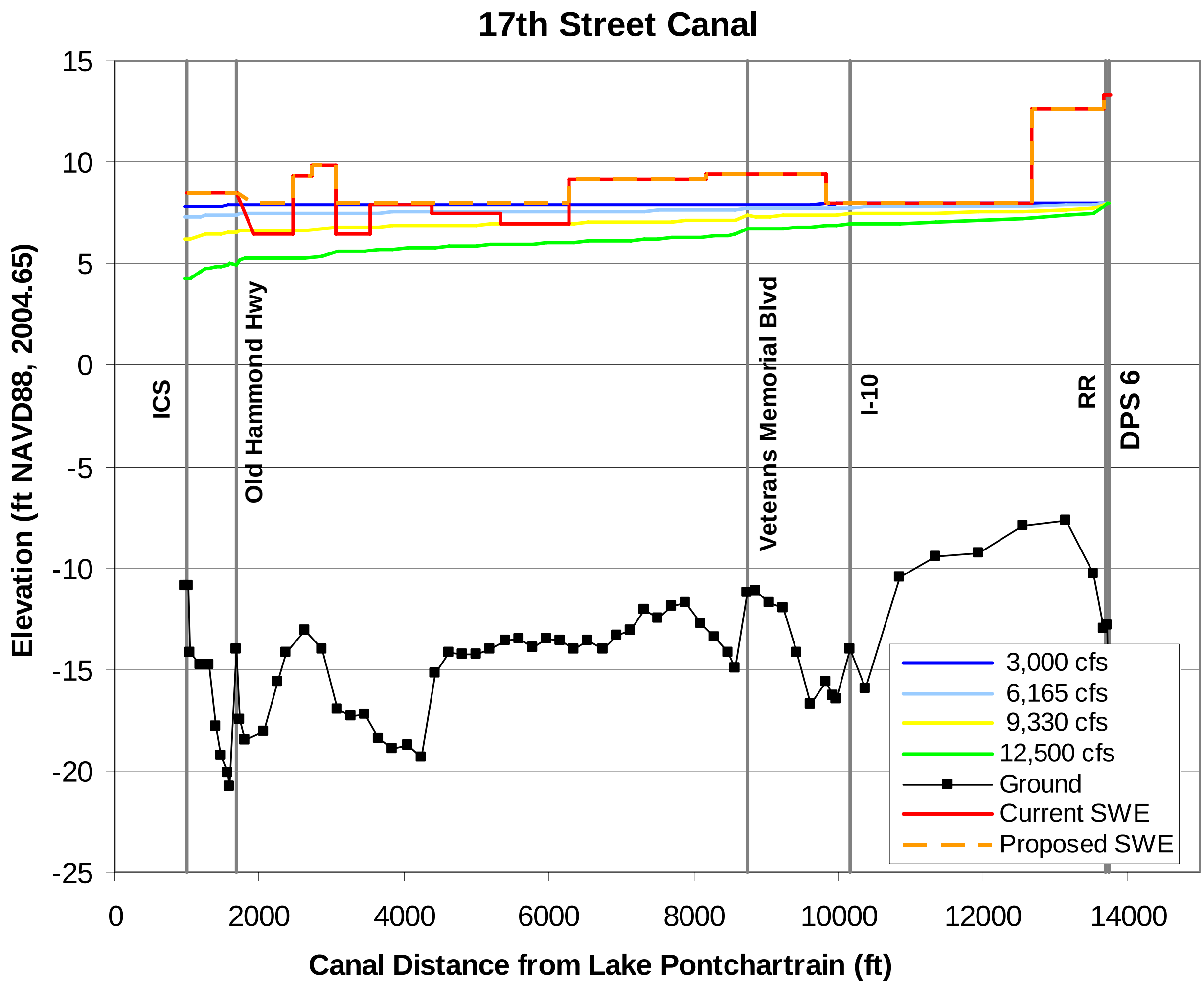


Figure 6.2-1. **17th Street Canal Water Surface Profiles for 25-, 50-, 75-, and 100-Percent of Design Discharge and Safe Water Elevation Not Exceeded**

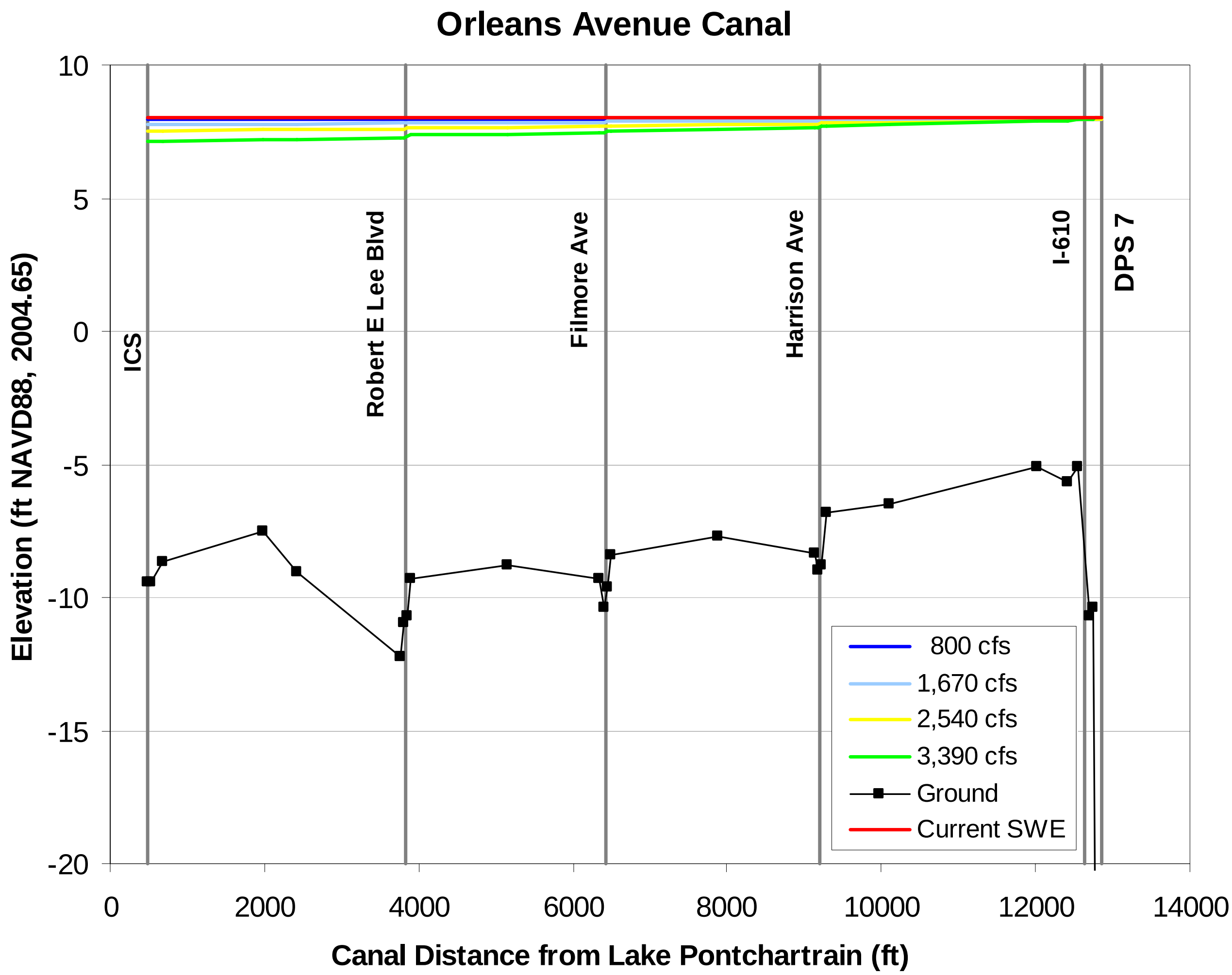


Figure 6.2-2. **Orleans Avenue Canal Water Surface Profiles for 25-, 50-, 75-, and 100-Percent of Design Discharge and Safe Water Not Exceeded**

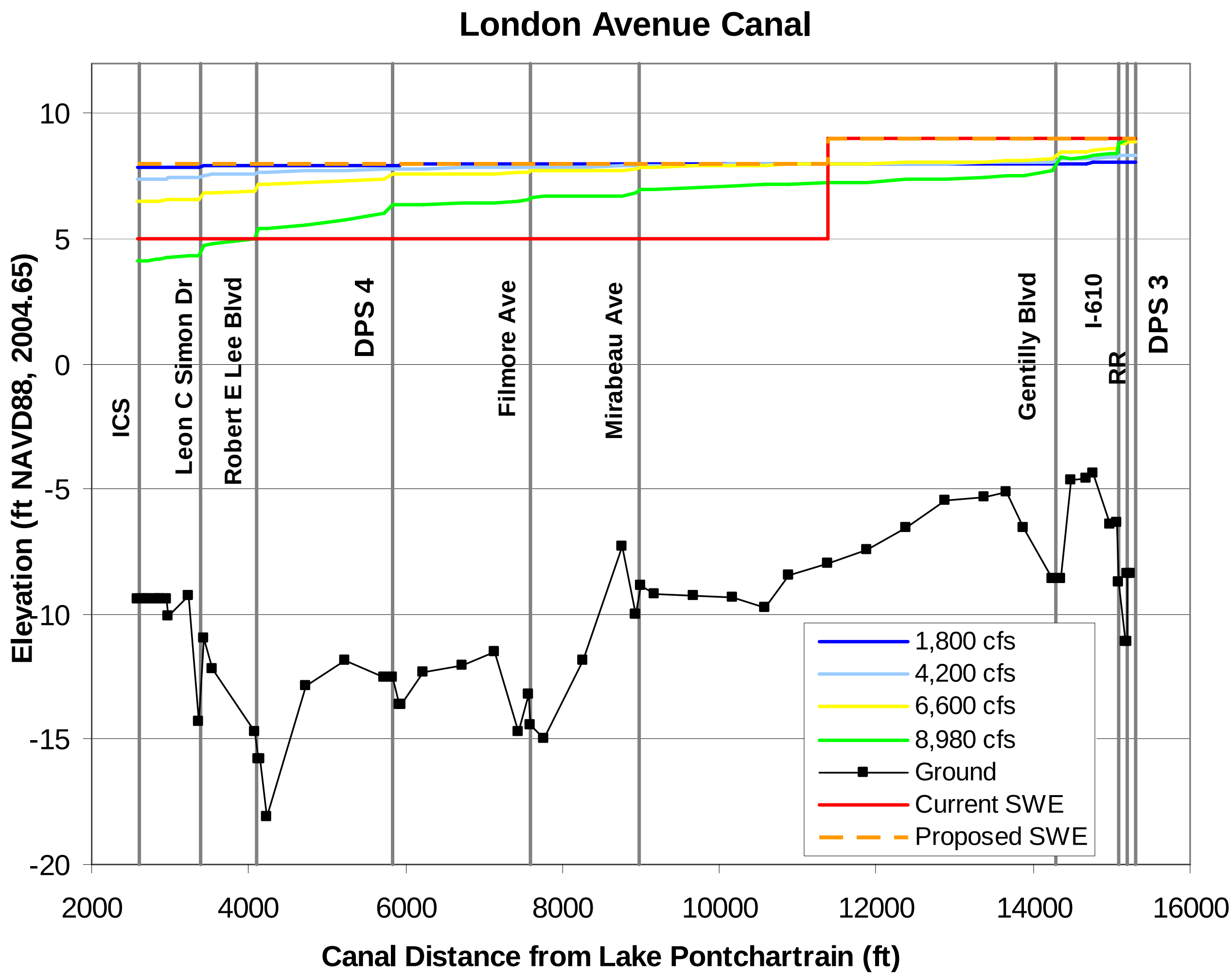


Figure 6.2-3. London Avenue Canal Water Surface Profiles for 25-, 50-, 75-, and 100-Percent of Design Discharge and Safe Water Elevation Not Exceeded

6.3 Operational Start-Up Analysis

The unsteady-state HEC-RAS hydraulic model was used to determine time available to pump operators to establish operation of permanent pump station pumps during gate closure. That is, if the existing drainage pump station pumps are discharging to the canals while the permanent pump station gates are closing, the permanent pump station pumps must reach that same discharge before safe water elevations are exceeded. For a range of Lake Pontchartrain elevations and canal discharges, the analysis determined how much time is available to permanent pump station operators.

The permanent pump station gate closure and pump start-up should occur such that canal safe water elevations are not exceeded. Two modeling scenarios were analyzed. (1) The first modeled the extreme case of gate closure with no permanent pumps operating. (2) The second assumed simultaneous gate closure and permanent pump station pump start-up. Both scenarios assume the existing drainage pump stations are discharging to the canal during permanent flood gate closure. For this assumption the first scenario is not considered a realistic operating scenario; rather the results were used to minimize the number of modeling simulations required for the second scenario.

The permanent pump station was modeled at the current Interim Closure Structure (ICS) location.

6.3.1 Gate Modeling Assumptions

Gates for the permanent pump stations have not been designed. For the purposes of modeling, the gates were assumed to close fully in 15 minutes at a rate of 1 ft per minute. Each canal was modeled with multiple gates to span the width of each canal.

Modeled gate geometry:

- 17th Street Canal: Width = 11 to 12 ft; Height = 15 ft; Invert = -10.0 ft NAVD88 (2004.65)
- Orleans Avenue Canal: Width = 5 to 12 ft; Height = 15 ft; Invert = -9.4 ft NAVD88 (2004.65)
- London Avenue Canal: Width = 11 to 12 ft; Height = 15 ft; Invert = -8.0 ft NAVD88 (2004.65)

6.3.2 Pump Modeling Assumptions

Pumps for the permanent pump stations have not been designed. The following pumps were included in the Conceptual Design Report for Permanent Flood Gates and Pump Stations (2006) and were used for the modeling described herein. The pump efficiency curves used in the modeling are the same as those contained in the Conceptual Design Report.

Table 6.3-1. Number of Pumps Used to Model Permanent Pump Station Operation

Canal	Permanent Pump Station Capacity (cfs)	Number of 1,000 cfs Pumps	Number of 500 cfs Pumps	Number of 250 cfs Pumps
17th Street	12,500	11	2	2
Orleans Avenue	3,500[19]	2	2	2
London Avenue	9,000	8	1	2

Pump start-up was assumed to begin when the gates enter the water, and a head differential across the gated structure is created. For modeling purposes, the type of pump is not important. However, the time required to bring the pump to full capacity, or pump start-up time, is important. In general, motor driven pumps would be able to come on line in only seconds. The pump discharges are self-priming, so on initial startup a pump would be pumping to the top of the siphon at only about 50 percent capacity. Over the next five to ten minutes the capacity would increase to 100 percent as the air is evacuated from the discharge tube. Engine driven pumps with jacket water heaters would be able to come up quickly also except the initial start

[19] The proposed pumping capacity for DPS 7 is 3,390 cfs. Conceptually, a permanent pump station with capacity of 3,400 cfs was designed. Here a 3,500 cfs capacity is modeled with two 250 cfs pumps, rather than one 250 cfs and one 150 cfs pump.

would take about one minute to get the engine to temperature before placing the maximum load. Results for both a 10-minute and 20-minute pump start-up time are reported.

6.3.3 Operational Start-Up Results

The results of gate closure without permanent pump station pumping are shown in Table 6.3-2.

For a 17th Street Canal existing drainage pump station discharge of 3,000 cfs and Lake Pontchartrain elevation of 4.0 ft NAVD88 (2004.65), it will take approximately one (1) hour to reach safe water elevation if the gates are closed and the permanent pump station pumps are not operated. For a DPS 7 discharge of 800 cfs and lake elevation of 5.0 ft, it will take about 2.5 hours to reach safe water elevation on Orleans Avenue Canal. For a London Avenue Canal existing drainage pump station discharge of 2,600 cfs and lake elevation 4.0 ft, it will take approximately one (1) hour to reach safe water elevation.

When the existing drainage pump stations are discharging to the canals, it is unlikely the gates would be closed without operating the permanent pump station pumps. However, the results of the modeling show that for lower existing drainage pump station discharges there should be sufficient time to commence operation of the permanent pump station pumps.

Table 6.3-2. Time to Reach Safe Water Elevation During Gate Closure Without Permanent Pump Station Pumping

Lake Pontchartrain Gate Closure Elevation (ft NAVD88, 2004.65)	17th Street Canal DPS 6 + I10 + Canal Street Discharge (cfs)				Orleans Avenue Canal DPS 7 Discharge (cfs)		London Avenue Canal DPS 3 + DPS 4 Discharge (cfs)		
	3,000	8,000	10,500	12,500	800	3,390	2,600	5000	8,890
2.0				22 min					19 min
3.0	77 min	31 min	24 min				68 min	32 min	16 min
4.0	58 min						57 min		
5.0					147 min	36 min			

The second modeling scenario was conducted for a limited range of canal discharges and Lake Pontchartrain elevations. The second modeling scenario allows permanent pump station pump start-up during gate closure. The results of the analysis are shown in Table 6.3-3 and Table 6.3-4.

Provided the permanent pump station pumps are started during gate closure and can reach full, required capacity in 10 minutes or less, the 17th Street Canal gates can be closed for Lake Pontchartrain elevations up to 5 ft NAVD88 (2004.65) while the existing drainage pump stations are discharging up to 10,500 cfs. If the permanent pump station pumps require 20 minutes to reach full capacity, then the 17th Street Canal gates can be closed at Lake Pontchartrain elevations up to 5 ft while the existing drainage pump stations are discharging up to 8,000 cfs.

Orleans Avenue Canal has the most operational start-up flexibility in terms of timing. For either a 10- or 20-minute permanent pump start-up time, the Orleans Avenue Canal gates can be closed at Lake Pontchartrain elevations up to 6 ft while DPS 7 is discharging up to 3,390 cfs.

London Avenue Canal gates cannot be closed at Lake Pontchartrain elevation 5.0 ft without exceeding safe water elevations when the existing drainage pump stations are discharging 8,890 cfs. However, for existing drainage pump station discharges of 5,000 cfs, the London Avenue Canal gates can be closed at Lake Pontchartrain elevations up to 5 ft assuming either a 10- or 20-minute permanent pump start-up time,.

Table 6.3-3. Maximum Canal Water Surface Elevation During Gate Closure With Permanent Pump Station Pumping - Assuming 10 Minute Pump Start-Up Time[20]

Lake Pontchartrain Gate Closure Elevation (ft NAVD88, 2004.65)	17th Street Canal DPS 6 + I10 + Canal Street Discharge (cfs)			Orleans Avenue Canal DPS 7 Discharge (cfs)	London Avenue Canal DPS 3 + DPS 4 Discharge (cfs)	
	8,000	10,500	12,500	3,390	5,000	8,890
3	4.1 ft	6.0 ft	6.8 ft			8.6 ft
4	5.6 ft	6.5 ft	7.4 ft		5.9 ft	> 9.0 ft
5	6.5 ft	7.3 ft	> 8.0 ft	6.3 ft	6.6 ft	> 9.0 ft
6	7.2 ft	> 8.0 ft	> 8.0 ft	7.2 ft	7.5 ft	> 9.0 ft

Table 6.3-4. Maximum Canal Water Surface Elevation During Gate Closure With Permanent Pump Station Pumping - Assuming 20 Minute Pump Start-Up Time

Lake Pontchartrain Gate Closure Elevation (ft NAVD88, 2004.65)	17th Street Canal DPS 6 + I10 + Canal Street Discharge (cfs)			Orleans Avenue Canal DPS 7 Discharge (cfs)	London Avenue Canal DPS 3 + DPS 4 Discharge (cfs)	
	8,000	10,500	12,500	3,500	5,000	9,000
3	5.9 ft	7.3 ft	> 8.0 ft			> 9.0 ft
4	6.7 ft	> 8.0 ft	> 8.0 ft		6.9 ft	> 9.0 ft
5	7.6 ft	> 8.0 ft	> 8.0 ft	6.8 ft	7.4 ft	> 9.0 ft
6	>8.0 ft	> 8.0 ft	> 8.0 ft	7.6 ft	8.4 ft	> 9.0 ft

6.4 Pump Failure Analysis

The unsteady-state HEC-RAS hydraulic model was used to determine reaction time available to pump operators in the case of failure of a pump at the permanent pump station. That is, if the

[20] Pump start-up time is the time from turning pumps on to reach 100 percent required pump capacity..

pump stations are operating in series and a pump at the permanent pump station (downstream) fails, the reaction time is the time available before safe water elevation is reached.

The following pump failure scenario was modeled for the maximum design discharge in each canal. Prior to pump failure a steady-state water surface profile was achieved with the upstream water surface elevation being 1 ft below safe water elevation. For each canal the loss of a 1,000 cfs capacity pump at the permanent pump station was evaluated. For purposes of the analysis, the loss of a 1,000 cfs pump was equivalent to a discharge reduction of 1,000 cfs. Table 6.4-1 below shows the results of the analysis.

Table 6.4-1. Reaction Time for Pump Failure Scenario Starting 1.0 ft Below Safe Water Elevation

Canal	DPS Discharge (cfs)	Permanent Pump Station Capacity Loss (cfs)	Starting WSEL at Target Safe Water Elevation Location (ft NAVD88, 2004.65)	Safe Water Elevation (ft NAVD88, 2004.65)	Reaction Time[21] (min)
17th Street	12,500	1,000	7.0	8.0	58
Orleans Avenue	3,390	1,000	7.0	8.0	48
London Avenue	8,980	1,000	7.0	8.0	49

The results indicated that for a single 1,000 cfs pump failure, there is over 45 minutes available to pump operators to react.

For the same starting water surface elevations, the reaction times will decrease approximately linearly with additional pump losses at the permanent pump station. For example, if the capacity loss is 2,000 cfs at the 17th Street Canal permanent pump station, the reaction time will be approximately 29 minutes. At London Avenue 4000 ft Downstream of DPS 3 a 2,000 cfs capacity loss results in a reaction time of about 25 minutes.

Various pump failure response scenarios were also evaluated. After pump failure at the permanent pump station, if an equal capacity pump is shut down at the existing drainage pump station, the water surface will nearly instantaneously stop increasing. If an additional pump is shut down such that the discharge at the existing pump station is less than the permanent pump station, then the water surface elevation in the canal will decrease.

Response scenarios at DPS 3 and DPS 4 were compared for London Avenue Canal. Following the pump failure described in Table 6.4-1 above, three different response scenarios were analyzed. (1) Respond to pump failure by reducing pumping at DPS 3 only. (2) Respond to pump failure by reducing pumping at DPS 4 only. (3) Respond to pump failure by reducing pumping equally at DPS 3 and DPS 4. A 1,000 cfs increment was selected for pump reduction at the existing drainage pump stations.

[21] Reaction time is defined here as the time from pump failure to safe water elevation being reached.

Model results show very little difference between responding to pump failure at DPS 3 versus at DPS 4. The water surface elevation in the canal can be brought back down to 1 ft below safe water elevation in approximately 20 minutes using DPS 3 alone. The recovery time when reducing pumping at DPS 4 is about 26 minutes. When reducing pumping at both pump stations, the recovery time is about 22 minutes.

6.5 Stage-Storage Curves

Stage-storage curves for each of the canals are contained in the Appendix.

7.0 ADDITIONAL RECOMMENDATIONS

7.1 Controls System

7.1.1 Existing Drainage Pump Station Operations

An investigation of the existing operations at DPS 3, DPS 4, DPS 6 and DPS 7 has concluded that all four stations have essentially no automatic control features. Station operation is strictly local control by station operators. Activation and de-activation of pumps, operation of valves, operation of gates, etc. is determined by Standard Operating Procedures (SOP) that are based on several years of experience. Because the operating staff is experienced, SOP documentation is sketchy and may not reflect current operations. Instrumentation monitoring of water levels and other station parameters is in disrepair and has not functioned for several years. However, float-type water staff gauges exist at the suction pools and discharge pools of each station. These are occasionally monitored by operator visual inspection. Various station instrument and meter readings are operator logged on a regular basis. Operating shifts overlap wherein shift-ending operating personnel brief the shift-starting personnel of station operation/status. Factors for drainage pump operation are communicated via land line telephone system or mobile radios. Several operators also have cellular phones. Operating instructions may come from the Sewerage Water Board supervisor or from other pump station operators. Current weather conditions are monitored by pump station staff. News media weather forecasts are monitored as well.

Modifying the existing DPS controls for automatic control could be accomplished. Design costs and construction costs would be very high. A major feature of a fully automated plant involves extensive monitoring to ensure automatic control schemes are reacting successfully. This monitoring adds significantly to the costs.

The present pumping scheme is to operate or not operate a specific number of pumps based on suction pool water levels, while considering both current and forecasted weather conditions. The variance in flow at each pump station is based on the pumping capacity of the various sized pumps and thus, water flow is an incremental-type process without any flow modulating capability. Since the existing pump stations do not have the capability to modulate flow, automating the existing stations is not cost effective. Local control of the pumps by the station operators is adequate for the existing stations today and for the future with the anticipated implementation of new permanent pump stations. A disadvantage of operating only in this local control mode is the risk to human life resulting from requiring personnel stationed at the pumping stations during major storm events.

7.1.2 Existing Interim Closure Structure (ICS) Controls

Since Hurricane Katrina, three interim closure structures with pumping capability have been installed at the shore of Lake Pontchartrain near the outfall of the 17th Street, London Avenue, and Orleans Avenue Canals. Each pumping station has been integrated with a Supervisory Control and Data Acquisition (SCADA) system and Closed Circuit Television (CCTV) system. The purpose of mentioning these systems is to consider building upon this existing elaborate control and monitoring platform for the incorporation of the permanent pump stations. Review of the system architecture shows that the system should be highly reliable, especially with the multiple network communication pathways. These pathways include land-laid fiber optic cables and Microwave radio systems. The logic is computed using a robust Programmable Logic Controller (PLC) system. The SCADA system provides human-machine interface (HMI) at personal computers (PCs) to control and monitor controlled equipment/systems. Ethernet TCP/IP (Transmission Control Protocol and Internet Protocol) allow network communication to several PCs in various locations. CCTV cameras installed at various locations provide operators with visual assessment capability of specific conditions. Another feature of the ICS control system is numerous solar-powered data gathering units strategically positioned on the canals and at existing drainage pump stations to monitor water levels, atmospheric winds, rainfall, ambient temperature, relative humidity and barometric pressure. The main control system equipment is powered from Uninterruptible Power Supply (UPS) systems, which has battery backup to continuously power the connected equipment. Also, a diesel-engine driven generator has been installed to provide power to the SCADA system during long duration commercial power outages.

Analysis of the ICS SCADA and CCTV systems should be done with the possibility of integrating the permanent pump station into them. Analysis should include capacity of the existing systems to add in the permanent pump station Input/Output (I/O) points and the environmental durability requirements of the system to operate in the extreme conditions being required for the permanent pump station. The process control is not time sensitive in the sense that reaction to changing water levels does not need to occur within seconds; rather minutes are available to respond to changing water levels. Therefore, speed involved with computer processing and network communication should not be an issue.

7.1.3 Recommended Permanent Pump Station Control System

Today's control technology permits complicated control schemes to be implemented inexpensively from a life cycle cost analysis standpoint. The ability to control more equipment with fewer operating staff than ever before and the predictability of automatic control are the main benefits of today's sophisticated control systems. Incorporating a sophisticated control system in a new facility is cost effective. Therefore, the permanent pump stations should have a high quality control and monitoring system. Control modes need to include automatic control as well as local control in the event automatic control fails. Local control bypasses the automatic control and is best incorporated near the particular piece of equipment – thus the term local control.

Operating the permanent pump stations in series with the existing drainage pump stations will require flow modulation at the permanent pump stations. The expected size of pumps at the permanent pump station will be too large to have valving or speed controls for modulation purposes. Therefore, modulating flow will be achieved by having a variety of pump sizes in each station and turning them on/off to more closely match the inflow to the outflow. The flow modulation is required to maintain minimum canal water levels required for the existing drainage pumps to operate with out breaking prime and to not exceed safe water elevations along the canal.

The existing ICS control and monitoring systems, which were installed in 2006, can be used as a platform for developing the permanent pump station controls. The ICS systems are up to date and add-on components should be readily available. The control system architecture is modern and multiple network communications pathways exist for reliability. Backup power sources are in-place. There is the benefit that operating staff of the ICS will be familiar with the expanded control system.

Modernizing the existing drainage pump station control systems is not justifiable from a cost standpoint as no process optimization will be gained.

The permanent pump stations should be constructed with control and monitoring systems to render the stations completely automatic with local control as a backup. Operating and monitoring from remote sites should be integrated in the new control and monitoring systems.

7.2 Physical Models

The presence of acceptable flow patterns within the permanent pump station approach and forebay areas, at the closure gates, and in particular at the pump suction inlets and pump throats is necessary for proper performance of these facilities. A properly organized and conducted physical hydraulic model study is a reliable method to identify unacceptable flow patterns, and may also be used during the early stages of design to develop permanent pump station designs that will achieve acceptable flow patterns and performance.

The governing Standard for design of pump intakes is ANSI/HI 9.8 – Pump Intake Design. This Standard also includes provisions for model testing. The planned features of the permanent pump stations meet one or more of the criteria listed in the Standard which require that a physical model study be conducted. These features include the following:

- The pumps have flows greater than 40,000 gpm (89.1 cfs) per pump or the total station flow is greater than 100,000 gpm (222.8 cfs).
- Proper pump operation is critical and pump repair, remediation of a poor design, and the impacts of inadequate performance or pump failure all together would cost more than ten times the cost of a model study.

It is recommended that a physical hydraulic model study be performed for each permanent pump station facility, including the approach, forebay and discharge areas. The model study should be performed in accordance with the requirements and recommendations of ANSI/HI 9.8, including the following:

- Model similitude and scale selection
- Model scope
- Instrumentation and measuring techniques
- Acceptance criteria
- Test plan
- Report preparation

- Other requirements as determined necessary for each pump station

It is recommended that the physical hydraulic model study of each permanent pump station facility be performed during the preliminary design phase to assist in the development of cost effective designs that satisfy specified design and performance criteria.

8.0 REFERENCES

Intergovernmental Panel on Climate Change, IPCC Third Assessment Report: Climate Change 2001

US Army Corps of Engineers, Conceptual Design Report for Permanent Flood Gates and Pump Stations, Prepared for New Orleans District, Jul 2006.

US Army Corps of Engineers, Engineering and Design: General Principles of Pump Station Design and Layout, Engineer Manual 1110-2-3102, Feb 1995.

US Army Corps of Engineers, Engineering and Design: Hydrologic Analysis of Interior Areas, Engineer Manual 1110-2-1413, Jan 1987.

US Army Corps of Engineers, Engineering and Design: Mechanical and Electrical Design of Pump Stations, Engineer Manual 1110-2-3105, Nov 1999 (change 2).

US Army Corps of Engineers, Performance Evaluation of the New Orleans and Southeast Louisiana Hurricane Protection System, Final Report of the Interagency Performance Evaluation Task Force (IPET), Volume IV – The Storm, Mar 2007.

US Army Corps of Engineers, Performance Evaluation of the New Orleans and Southeast Louisiana Hurricane Protection System, Final Report of the Interagency Performance Evaluation Task Force (IPET), Volume VI – The Performance – Interior Drainage and Pumping, Mar 2007.

US Department of Commerce, Weather Bureau, Rainfall Frequency Atlas of the United States for Durations from 30 minutes to 24 Hours and Return Periods from 1 to 100 Years, May 1961.

APPENDIX – ADDITIONAL MODELING RESULTS

A-1. Hydrographs

The Vicksburg District Corps of Engineers (MVK) conducted hydrologic modeling to develop discharge hydrographs at the existing drainage pump stations (DPS) along the 17th Street, London Avenue, and Orleans Avenue Canals. Seven different recurrence interval events were modeled using HEC-HMS and unsteady HEC-RAS. The events were as follows: 1-, 2-, 5-, 10-, 25-, 50-, and 100-year, 24-hour storm events. The flows modeled by HEC-HMS were routed to storage areas in HEC-RAS and then pumped through existing drainage pump stations to the 17th Street, Orleans Avenue, and London Avenue Canals. The efficiency curves and pump on/off elevations used to model the existing pump stations were obtained from the New Orleans Sewerage and Water Board (S&WB). Two downstream boundary conditions were modeled at Lake Pontchartrain: 1.0 ft NAVD88 (2004.65) constant elevation to represent "normal" lake stage and a 100-year stage hydrograph developed by New Orleans District Corps of Engineers (MVN) using a lake surge model. For the second boundary condition, the time to peak was set to coincide with peak modeled pump station discharge.

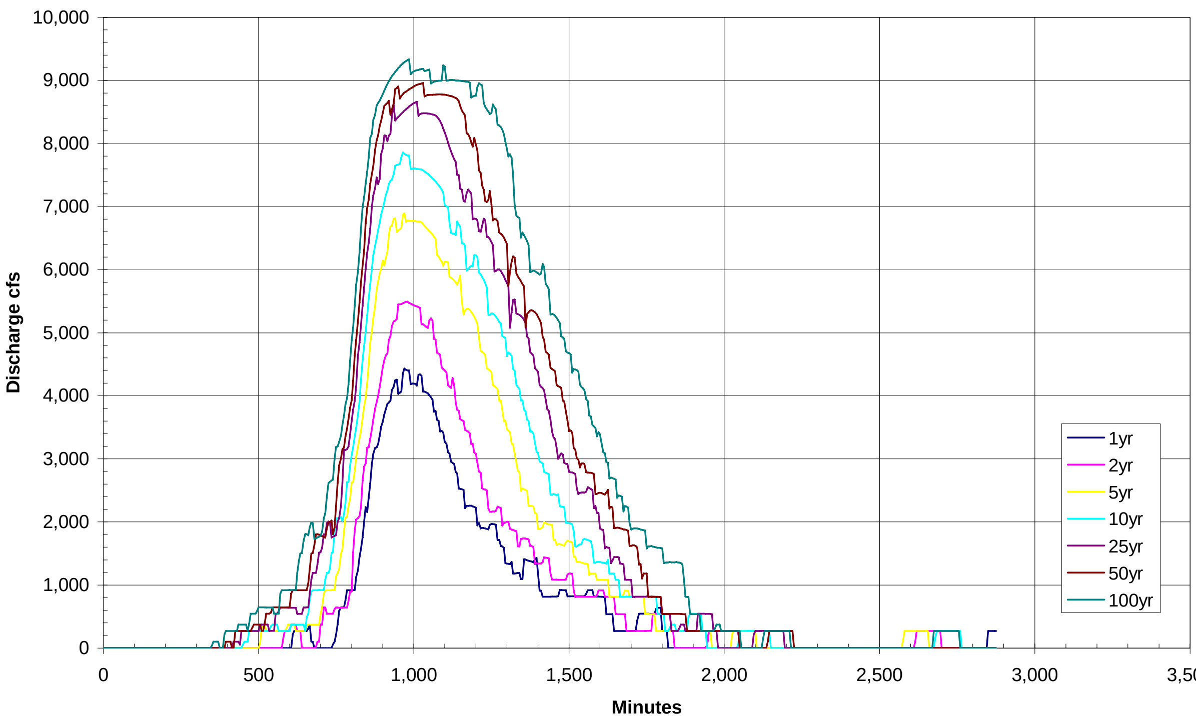


Figure A-1- 1. 17th Street Canal Modeled Hydrograph Combination of DPS 6 and I-10 for Lake Pontchartrain Elevation 1.0 ft NAVD88 (2004.65) (MVK 2007 Results)

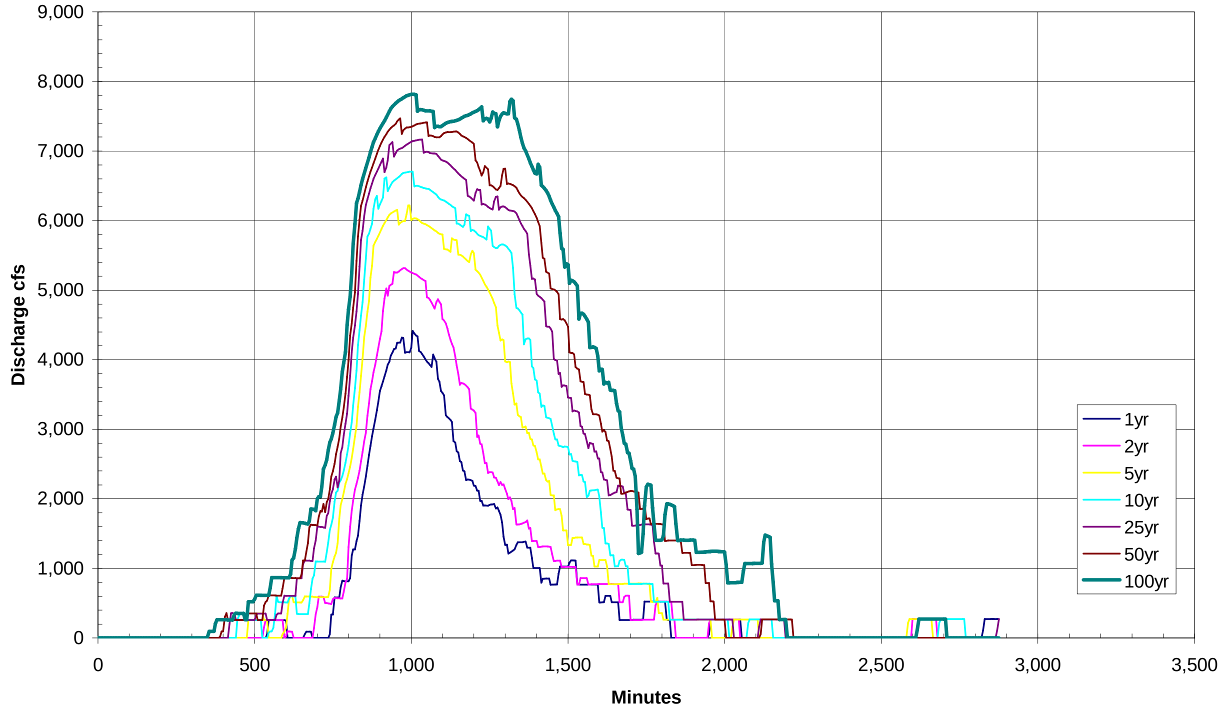


Figure A-1- 2. 17th Street Canal Modeled Hydrograph Combination of DPS 6 and I-10 for Lake Pontchartrain 100-Year Storm Surge Hydrograph (MVK 2007 Results)

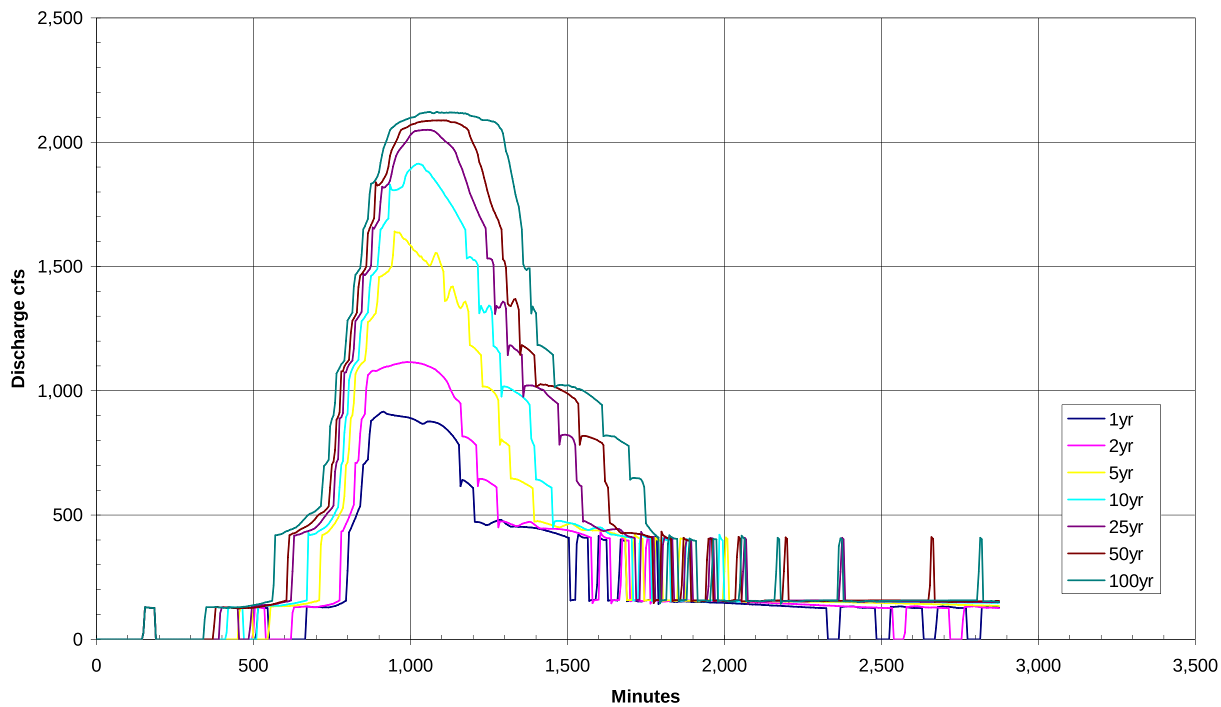


Figure A-1- 3. Orleans Avenue Canal Modeled Hydrograph for Lake Pontchartrain Elevation 1.0 ft NAVD88 (2004.65) (MVK 2007 Results)

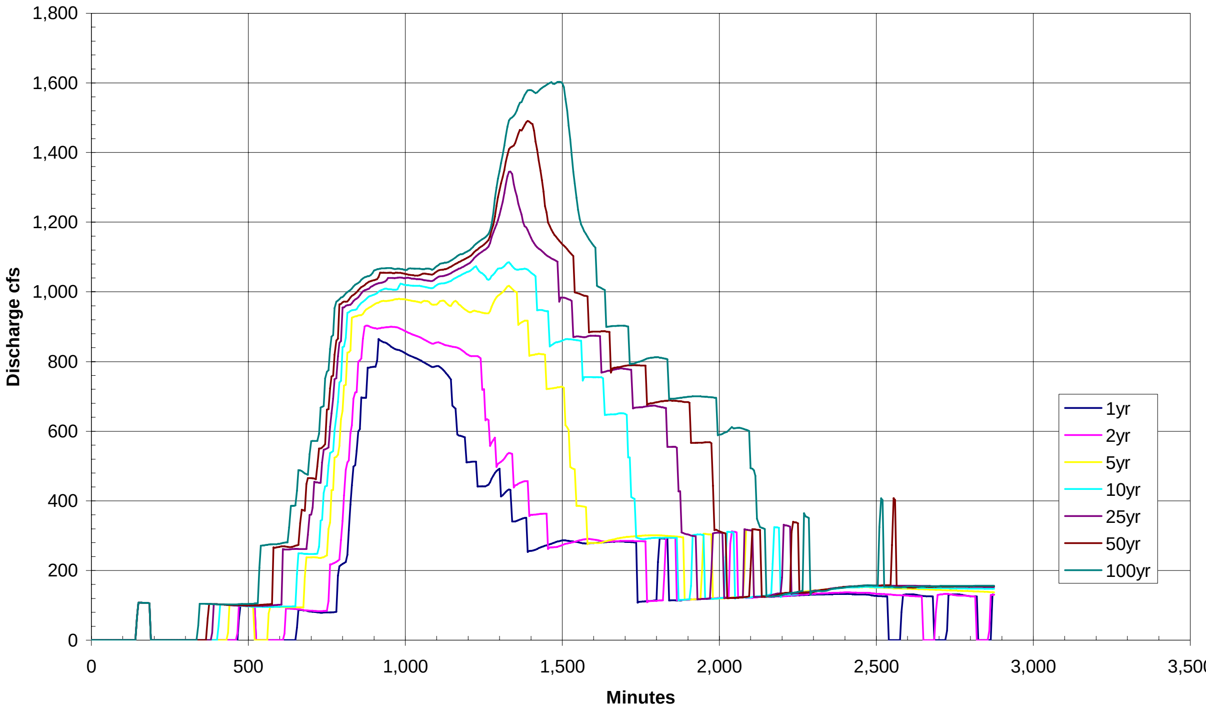


Figure A-1- 4. Orleans Avenue Canal Modeled Hydrograph for Lake Pontchartrain 100-Year Storm Surge Hydrograph (MVK 2007 Results)

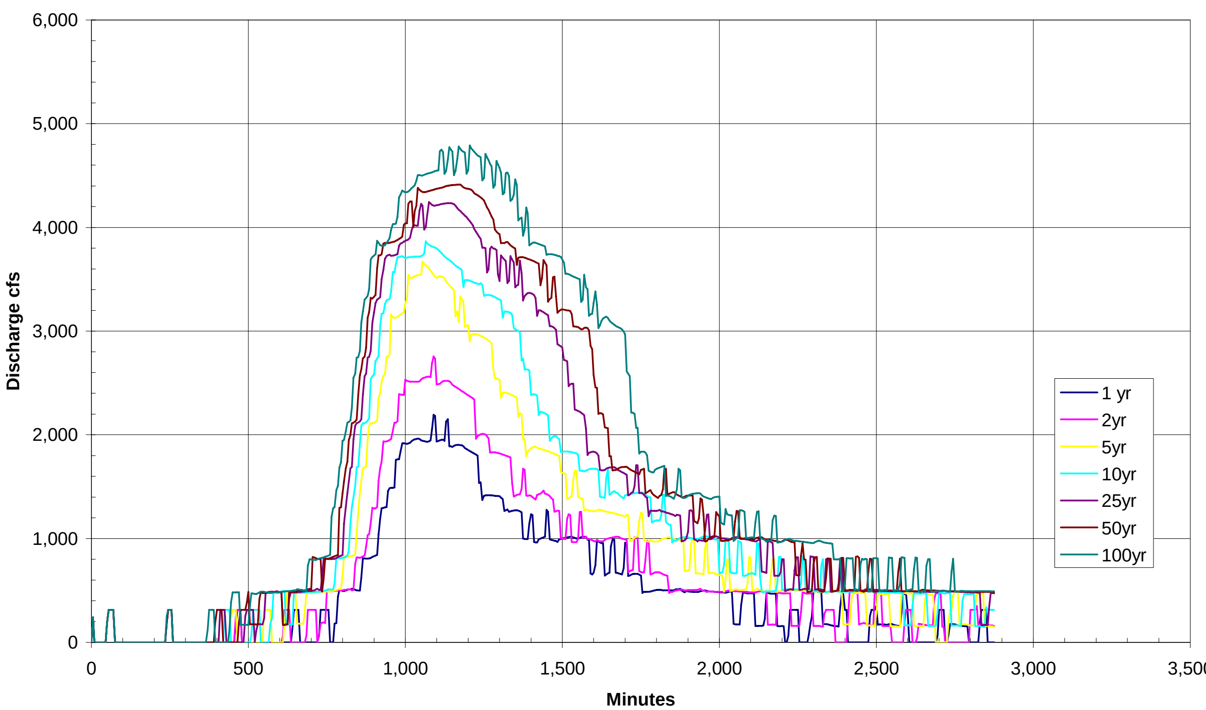


Figure A-1- 5. London Avenue Canal Modeled Hydrograph Combination of DPS 3 and 4 for Lake Pontchartrain Elevation 1.0 ft NAVD88 (2004.65) (MVK 2007 Results)

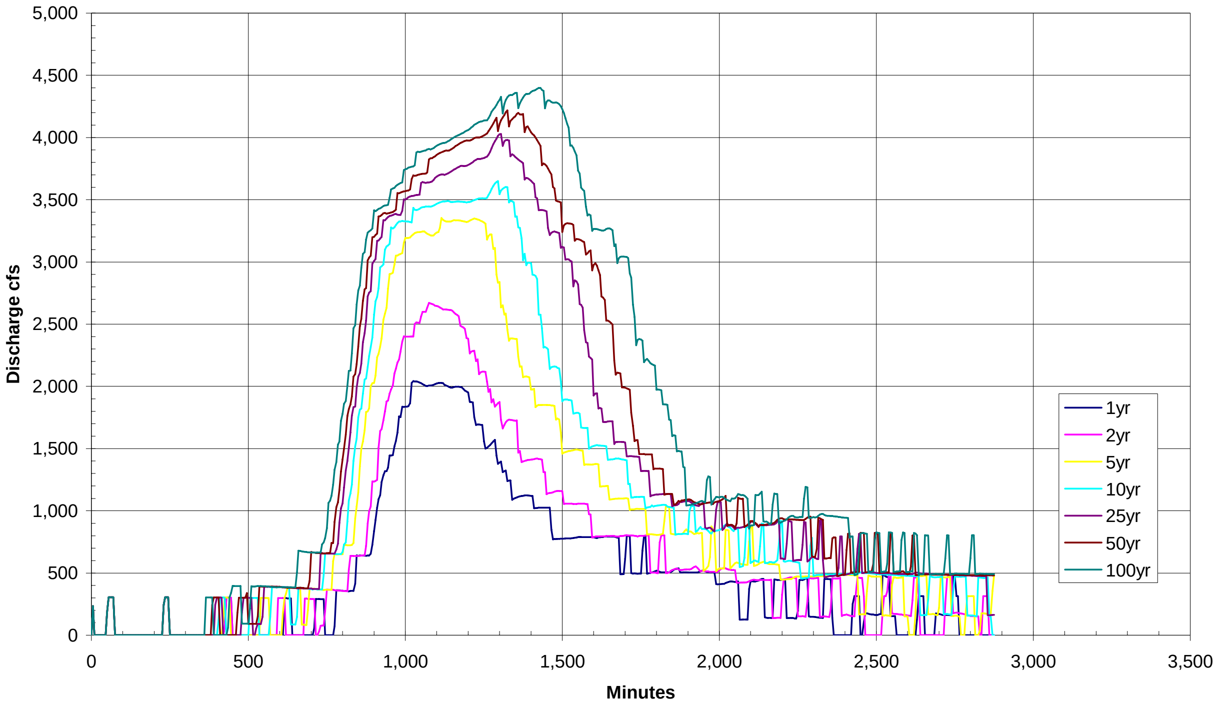


Figure A-1- 6. London Avenue Canal Modeled Hydrograph Combination of DPS 3 and 4 for Lake Pontchartrain 100-Year Storm Surge Hydrograph (MVK 2007 Results)

A-2. Canal Stage-Storage Curves

The stage-storage curves for each canal are shown graphically in Figure A-2-1 through Figure A-2-3. The data for the curves are tabulated in Table A-2-1 through Table A-2-3.

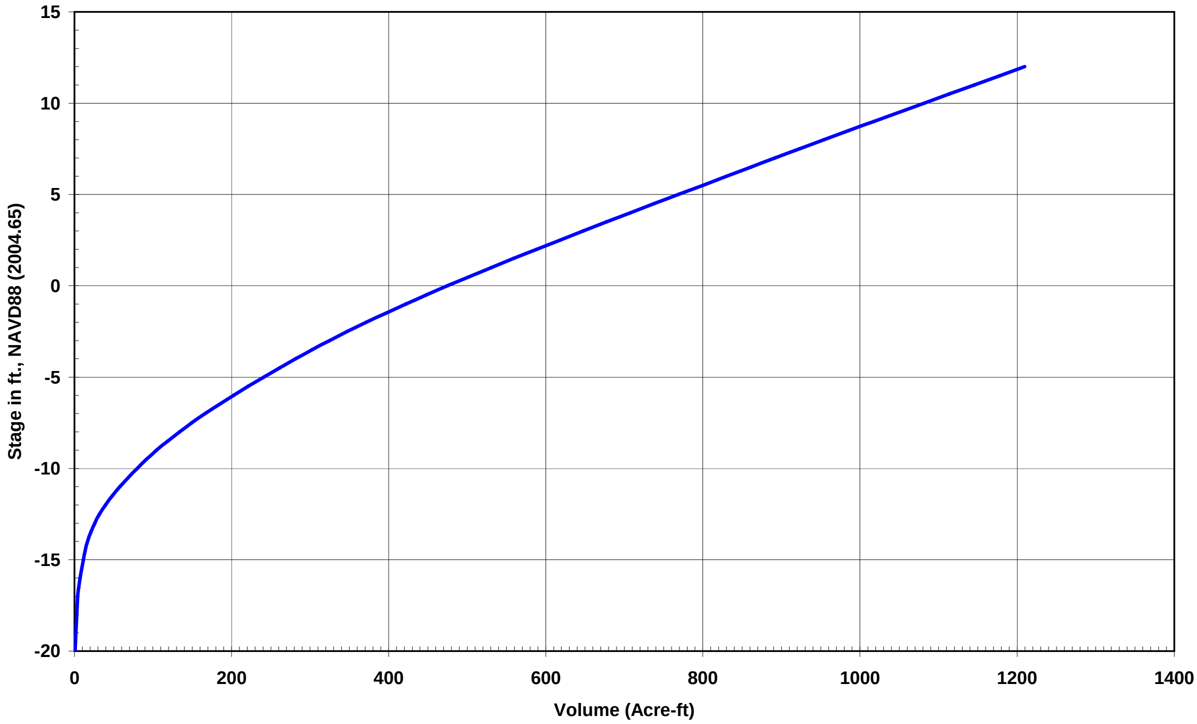


Figure A-2- 1. 17th Street Canal Stage-Storage Curve

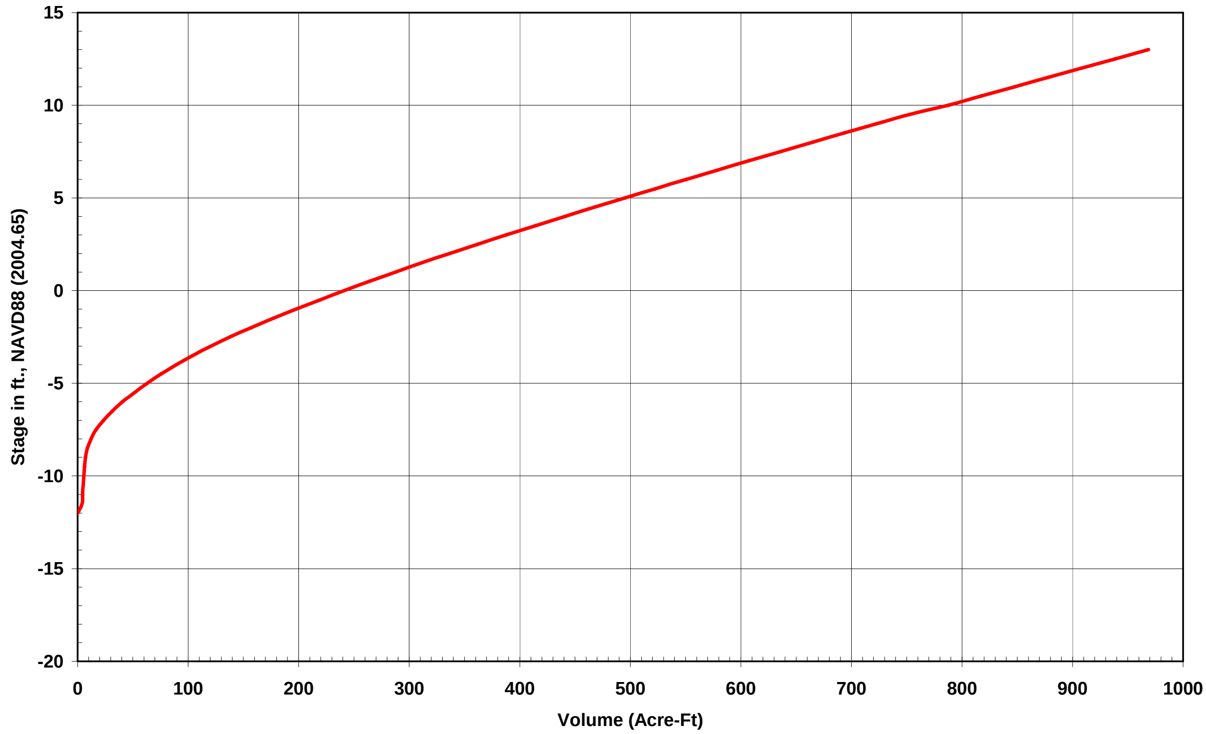


Figure A-2- 2. Orleans Avenue Canal Stage-Storage Curve

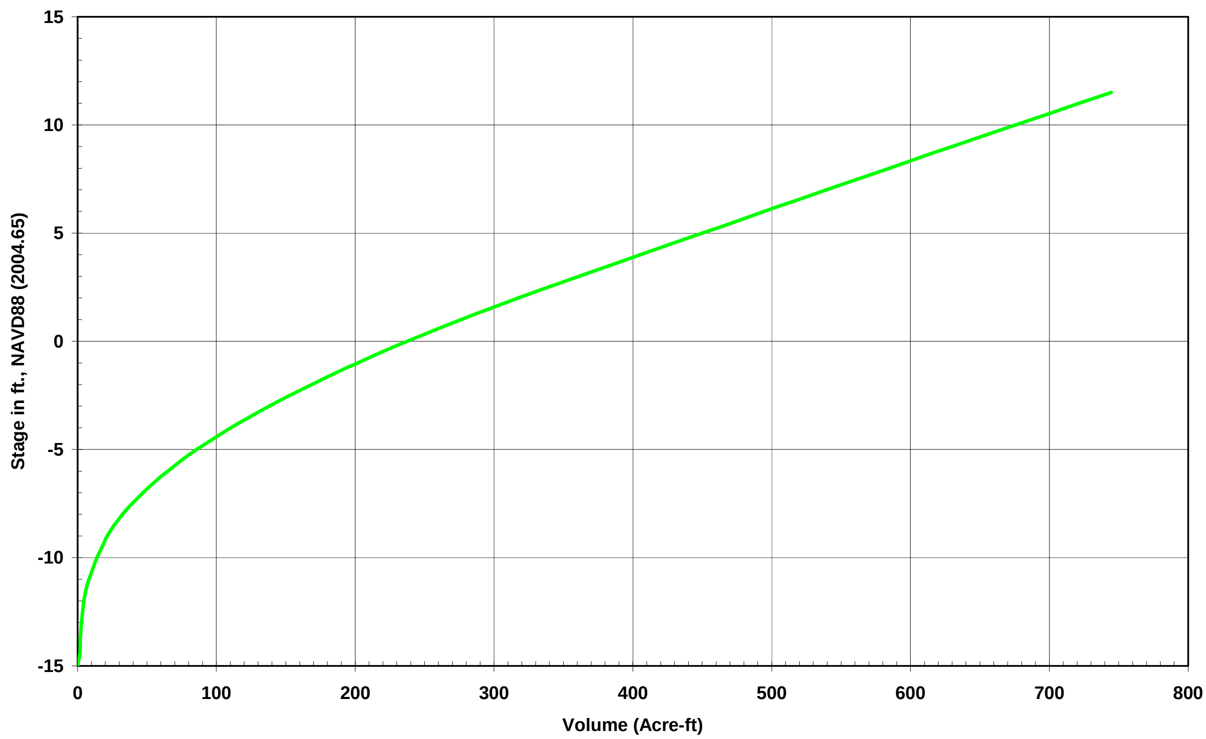


Figure A-2- 3. London Avenue Canal Stage-Storage Curve

Table A-2- 1. 17th Street Canal Stage-Storage Data

Canal Stage, ft NAVD88 (2004.65)	Volume (acre-ft)	Canal Stage, ft NAVD88 (2004.65)	Volume (acre-ft)
-20.8	0.0	-2.5	347.3
-17.0	4.2	-2.0	371.3
-16.5	5.5	-1.5	396.2
-16.0	7.2	-1.0	421.8
-15.5	9.1	-0.5	448.0
-15.0	11.3	0.0	475.0
-14.5	13.5	0.5	502.6
-14.0	16.6	1.0	530.8
-13.5	20.8	1.5	559.3
-13.0	26.0	2.0	588.4
-12.5	32.1	2.5	617.8
-12.0	39.5	3.0	647.6
-11.5	48.2	3.5	677.4
-11.0	57.9	4.0	707.5
-10.5	68.6	4.5	737.9
-10.0	79.8	5.0	768.6
-9.5	91.9	5.5	799.5
-9.0	104.7	6.0	830.3
-8.5	118.8	6.5	861.0
-8.0	133.8	7.0	891.8
-7.5	149.4	7.5	922.8
-7.0	166.4	8.0	954.1
-6.5	184.1	8.5	985.6
-6.0	202.5	9.0	1017.6
-5.5	221.4	9.5	1049.5
-5.0	240.9	10.0	1081.5
-4.5	260.9	10.5	1113.5
-4.0	281.4	11.0	1145.5
-3.5	302.7	11.5	1177.5
-3.0	324.6	12.0	1209.6

Table A-2- 2. Orleans Avenue Canal Stage-Storage Data

Canal Stage, ft NAVD88 (2004.65)	Volume (acre-ft)	Canal Stage, ft NAVD88 (2004.65)	Volume (acre-ft)
-12.0	0.0	1.0	287.5
-11.5	4.1	1.5	311.5
-11.0	4.6	2.0	336.3
-10.5	5.0	2.5	361.8
-10.0	5.7	3.0	387.7
-9.5	6.4	3.5	414.1
-9.0	7.2	4.0	440.7
-8.5	9.0	4.5	467.7
-8.0	12.3	5.0	495.0
-7.5	16.8	5.5	522.6
-7.0	23.6	6.0	550.6
-6.5	31.4	6.5	578.8
-6.0	40.7	7.0	607.2
-5.5	51.5	7.5	635.9
-5.0	63.0	8.0	664.7
-4.5	75.6	8.5	693.6
-4.0	89.3	9.0	722.7
-3.5	104.4	9.5	751.9
-3.0	120.6	10.0	787.7
-2.5	137.9	10.5	817.5
-2.0	156.9	11.0	847.6
-1.5	176.9	11.5	877.8
-1.0	197.7	12.0	908.1
-0.5	219.2	12.5	938.3
0.0	241.4	13.0	968.7
0.5	264.1		

Table A-2- 3. London Avenue Canal Stage-Storage Data

Canal Stage, ft NAVD88 (2004.65)	Volume (acre-ft)	Canal Stage, ft NAVD88 (2004.65)	Volume (acre-ft)
-15.0	0.0	-1.5	184.9
-14.5	1.6	-1.0	201.7
-14.0	1.9	-0.5	219.3
-13.5	2.3	0.0	237.5
-13.0	2.9	0.5	256.5
-12.5	3.6	1.0	276.1
-12.0	4.5	1.5	296.5
-11.5	6.0	2.0	317.6
-11.0	8.2	2.5	339.1
-10.5	11.0	3.0	361.0
-10.0	14.1	3.5	383.2
-9.5	17.6	4.0	405.4
-9.0	21.4	4.5	427.7
-8.5	26.3	5.0	450.0
-8.0	32.4	5.5	472.4
-7.5	39.3	6.0	494.7
-7.0	47.1	6.5	517.1
-6.5	55.5	7.0	539.6
-6.0	65.0	7.5	562.1
-5.5	75.1	8.0	584.7
-5.0	85.8	8.5	607.3
-4.5	97.8	9.0	630.0
-4.0	110.4	9.5	652.8
-3.5	123.9	10.0	675.7
-3.0	138.2	10.5	698.7
-2.5	153.1	11.0	721.7
-2.0	168.6	11.5	744.7