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CONCEPTUAL DESIGN REPORT FOR PERMANENT FLOOD GATES AND PUMP STATIONS

Prepared for

U.S. Army Corps of Engineers
New Orleans District
New Orleans, Louisiana

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EXECUTIVE SUMMARY

1.0 EXECUTIVE SUMMARY

The purpose of this study is to provide both economic and non-economic criteria for evaluation of two options for permanent closure structures for the 17th Street, Orleans Avenue, and London Avenue Canals. The options were selected based on previous studies and were identified as requiring additional development of criteria. The two options are described as follows.

- *Option 1 – Construction of new permanent Gated Pump Stations at the mouths of the 17th Street, Orleans, and London Avenue Canals.* This alternative provides permanent gates and pump stations at the mouths of the outfall canals, with the permanent pumping stations serving as an integral part of the hurricane protection system. This alternative leaves in-place the floodwalls that flank the three outfall canals. The existing Sewerage and Water Board of New Orleans (S&WB) pump stations would be left in place to function in their current mode of operation, lifting water to lake level in the outfall canals, with gravity drainage to Lake Pontchartrain. The new permanent lakefront structures would be equipped with gates. The gates would remain open to allow flow-through drainage during ordinary conditions and close only during times of high storm surges. Normal lake elevations are generally higher than the ground elevations of the areas through which the canals pass (often by five feet or more), so, with the gates left open most of the time, the floodwalls would remain an integral part of the city's flood protection system.

- *Option 2 - Construction of new Replacement Pump Stations at the mouths of the 17th Street, Orleans, and London Avenue Canals.* The stations would be constructed as permanent closures of the canals requiring full time operation of these pump stations. The levee and floodwalls along the canals themselves would no longer be required as part of the Hurricane Protection system (HPS), eliminating nearly 13 miles of floodwalls. The existing S&WB pump stations on the outfall canals would be taken out of commission. Because the new stations would completely separate the canals from Lake Pontchartrain's influence with the canals at a new and much lower flowline, the banks of these canals would be reshaped to lower elevations, essentially reconstructing the canal system. The canals' hydraulic grade lines would be lowered substantially and the canals would be lined with concrete. The canal modifications will require substantial bridge modifications along the length of each canal.

Criteria under which this study was performed were comprehensive, regarding both subject matter and source. Evaluation is based on numerous factors that are interrelated and result in a variety of combinations and outcomes. These factors include hydraulic, geotechnical, civil/site, mechanical, electrical, structural, utilities, bridges, environmental, and constructability considerations. All these factors have considerations that increase or decrease the desirability of an option. The challenge of this study is to identify a plausible scenario that is within reason, be consistent with the application of this scenario, and determine the related cost. In that way, the two options can be analytically compared. Other factors also have significant influence on option selection which are difficult or cannot be quantified. Non-technical issues such as political influence, public acceptance, future parish development, funding sources, operation and maintenance responsibilities and others, are outside the scope of this study. Therefore, this study focuses on defining the technical issues to identify a cost basis that ultimately provides the basis for an informed decision between Option 1 and Option 2.

1.1 Hydraulic Analysis

Several hydraulic issues required resolution under this study. Via HEC-RAS computer modeling, canal hydraulics were analyzed for a variety of pertinent design, safety, and operating conditions.

1.1.1 Option 1

Safe (maximum) water surface elevations in each canal provided by the New Orleans District were a significant driver of the Option 1 analysis. The required maximum elevations as provided are listed as follows:

- 17th Street Canal: +5.0 ft. (NAVD 88 datum)
- Orleans Ave. Canal: +9.0 ft. (NAVD 88 datum)
- London Ave. Canal: +5.0 ft. (NAVD 88 datum)

Thus, for Option 1 under these criteria, the railroad bridge over the 17th Street Canal represents a hydraulic control that must be removed or modified to achieve the required safe water elevation upstream of that bridge. Also, raising the Gentilly Road Bridge over the London Avenue Canal will be necessary to achieve the defined safe canal water elevation upstream of that bridge at the design discharge condition. Note that safe water elevations under the required criteria were met in the existing canal for the Orleans Avenue Canal. Analysis also revealed that gate closure for Option 1 is dependent not only on Lake Pontchartrain elevation, but also on concurrent canal discharge.

1.1.2 Option 2

Key limiting criteria for the Option 2 hydraulic analysis established that the design discharge canal water elevation cannot exceed the pump station suction-side water surface elevation at the existing pump station for each canal. Specifically, these criteria are listed as follows:

- DPS 6 (17th St. Canal): -10.9 ft
- DPS 7 (Orleans Canal): -9.4 ft
- DPS 3 (London Canal): -9.9 ft
- DPS 4 (London Canal): -10.4 ft

The modeling approach for the Option 2 analysis required modifications to the canal invert profile and cross-section to provide a gravity-flow canal alignment. A concrete-lined, rectangular canal cross-section was selected with the following standard widths.

- 17th St. Canal: 150 ft.
- Orleans Canal: 75 ft.
- London Canal: 100 ft.

For each canal, iterative modeling runs resulted in the determination of the canal profile that yields the required suction-side water surface elevation. In this way, canal modification geometrics were established.

1.2 Pumps

Two principle types of pumps were considered for these permanent pump stations. Horizontal types similar to the Woods Screw Pumps are extensively used in the older pumping stations. Vertical types are also used in the Parish and are commonly used in newer pump stations. Considering size limitations of roughly 1000 cubic feet per second and the benefits of the different pump types, vertical pumps are recommended for the purposes of this study.

Pump sizing is highly dependent on both the flow and head conditions. Although the flow requirements are clearly defined, the head requirements were debated. The pumping units and their associated drivers are based on the Sewer & Water Board designated worst-case between the primary condition of maximum flow at normal lake water surface elevation with tide and the secondary condition of 60 percent of maximum flow at maximum lake elevation with tide. In addition, to accommodate the disparate flow conditions, a combination of pump sizes (150 to 1000 CFS) and drivers (electric driven motors with engine generator backup or direct engine driven pumps) were incorporated. Therefore, this study assumes approximately 60 percent motor driven pumps and 40 percent engine driven pumps.

1.3 Electrical

The total storm event electric-driven pump load will be supplied from local standby generators located at each of the three new stations. Stand-by diesel generators will utilize an N+1 design. That is, if a generator goes off-line or one is down for maintenance, the full pump station load will be supplied by standby power. The utility service, standby generators, and pump motors will all operate at 4160 volts. All electrical distribution circuits will be routed underground in concrete-encased ductbank.

Option 2 removes existing inland pump stations from service and therefore requires more power to achieve the same flow as Option 1. The Option 2 general arrangement of electrical equipment remains unchanged from Option 1, but larger electrical equipment is required to meet the increased load demands. Utility power, from Entergy, will only be supplied for the normal pump loads. Incoming utility service will not be sized to accommodate the storm event pump loads.

In order to minimize required operation and maintenance at three separate power plants, the concept of a central power plant with power distribution to the three pump station sites was also investigated. The central plant results in a capital cost increase and significant schedule extension, but also results in a significant decrease of ongoing operation and maintenance costs.

1.4 Geotechnical

Using existing data primarily from the IPET report, several geotechnical analyses were required to address geotechnical impacts of pump station layouts in both Options1 and 2. Canal side-slope stability was analyzed for all cases. Under Option 1, existing side slopes are stable. For the canal deepening required under Option 2, analysis indicates the following stable slopes at the three subject canals.

- 17th Street Canal: 5:1
- Orleans Avenue Canal: 4:1
- London Avenue Canal: 5:1

These slopes result in canal widths that significantly exceed the available existing canal right-of-way. Therefore, canal lining alternatives were evaluated and vertical sheet pile walls were selected for Option 2 canal deepening.

Seepage was analyzed for Option 2 under two conditions; relief valves or water-tight liner. Total drawdown with relief valves resulted in a drawdown at the canal of 6 feet and a drawdown 300 feet from the canal of 2 feet. Total drawdown with a water-tight liner resulted in a drawdown at the canal of 4 feet and a drawdown 70 feet from the canal of 1 foot. Finally, preliminary pump station building stability analysis was also performed, all to support the development of costs for construction of those buildings.

1.5 Civil/Site

A total of three pump station locations were developed for each of the three canals, each for both Options 1 and 2. Thus, a total of 18 civil/site plans were developed and analyzed, all in support of cost development.

1.5.1 17th Street Canal

Alternative A is attractive for its protection of the pump station from lake surge effects, thus requiring no breakwater and minimal erosion protection requirements elsewhere. However, it requires a relatively large residential right-of-way acquisition. Layout Alternative B is attractive for its cost savings in converting the temporary gate structure to a permanent feature, however, it requires a significant right of way acquisition and affects the historic Bucktown area. Layout Alternative C is attractive for its minimal right-of-way acquisition requirements, its relative ease of constructability, and its in-line, shore-front location. Given that shore-front location, its erosion control requirements are substantial and it does impact the historic Bucktown pedestrian bridge.

1.5.2 Orleans Avenue Canal

Alternative A layout is attractive for its protection of the pump station from lake surge effects, resulting in no need for a breakwater structure. The location requires right-of-way acquisition in a residential area, but acquisition appears to be exclusively undeveloped property.

Further, there is no significant construction sequencing is required to maintain canal flow. The potential relocation of the existing temporary power plant, depending on the final precise location of the pump station, is unfortunate but manageable. Layout Alternative B is attractive for its convenient fit within the existing canal width. The location requires right-of-way acquisition in a residential area, but acquisition appears to be exclusively undeveloped property. Due to the lakeshore discharge location, a major breakwater structure is required. In summary, this option seems to create negatives compared to Layout A, while adding no advantages over Layout A. Thus, it is not recommended for further consideration. Layout Alternative C is attractive for its relatively minimal right-of-way acquisition requirements. Constructability is a mixture of positives and negatives. Constructing the pump station building in one phase is positive; however, construction in the lake offers other complications that are costly to overcome. Further, a major breakwater structure and significant plant armoring is required.

1.5.3 London Avenue Canal

Alternative A is attractive for its protection of the pump station from lake surge effects, resulting in no need for a major breakwater structure. The location requires right-of-way acquisition in a residential area, but acquisition appears to be property developed only with parking areas and other relatively low value improvements. Further, it is attractive since no significant construction sequencing is required to maintain canal flow during the construction duration. Layout Alternative B is attractive for its convenient fit within the existing canal width. The location requires right-of-way acquisition in a residential area, but acquisition appears to be exclusively undeveloped property. Due to the lakeshore discharge location, a major breakwater structure is required. Some construction complexity is introduced, since the pump station must be constructed in two stages to maintain channel flow during construction. In summary, this option seems to create negatives compared to Layout A, while adding no advantages over Layout A. Thus, it is not recommended for further consideration. Layout Alternative C is attractive for its relatively minimal right-of-way acquisition requirements. Constructing the pump station building in one phase is positive; however, construction in the lake offers other complications that are costly to overcome. Further, a major breakwater structure and significant plant armoring its required.

1.6 Bridges

Under Option 1, bridges along each canal are virtually unaffected, except those directly affected by a particular pump station layout. Under Option 2, however, the lowering of the canal, poor soil conditions and constructability issues under bridges results in many of the bridge structures being replaced or rehabilitated. Many of these bridge structures can be replaced at considerable cost while maintaining traffic but not without significant impacts to property along the roadway. The most significant bridge structures affected by Option 2 are the 17th Street Canal I-10 Bridge and Southern Railroad Bridge as well as the Southern Railroad Bridge at London Avenue Canal. The I-10 over 17th Street Canal represents significant challenge for maintaining traffic, limiting impacts to adjacent property owners, constructability and roadway geometry. The Southern Railroad Bridge also involves maintaining rail traffic, adjacent impacts and railroad geometry throughout the corridor.

1.7 Utilities

A number of existing utilities conflict with the proposed improvements at each canal in each of the two options. Generally, utility conflicts at each canal are minimal in Option 1. Utility conflicts are more significant and costly to adjust in the Option 2 scenario.

1.8 Environmental

Environmental databases were reviewed for sites containing hazardous, toxic or radioactive wastes (HTRW). A site reconnaissance was also performed to search for visible indications of the presence of HTRW. No HTRW sites have been identified within the project boundaries.

A Certified Industrial Hygienist (CIH) Investigation was conducted in February 2006 (report dated March 2006) by GEC with assistance from Professional Technical Services, Inc. (ProTech). This study focused on sediment quality in all three canals that are the subject of the present study and determined that bottom sediments sampled in all three canals are contaminated with petroleum related constituents.

National Wetland Inventory (NWI) maps were reviewed and field checked during a site reconnaissance. Jurisdictional wetlands have been identified along both the 17th Street Canal and the London Avenue Canal.

At the Corps request, analysis of protected species impacts has been turned over to the Hurricane Protection Office (HPO) for investigation and determination of effect.

A coordination letter requesting comment from the Louisiana State Historic Preservation Officer (SHPO) has been prepared and sent. A response has not yet been received.

1.9 Constructability

Constructability has been addressed for all 18 pump station layout alternatives, including Option 2 canal degradation scenarios. The constructability analysis has been conceptual in nature, but sufficiently detailed as necessary to support the development of realistic rough-order-of-magnitude costs.

1.10 Costs

Costs were developed for all considered layout plans. However, for the ultimate purpose of comparison between Option 1 and Option 2, the following Layout Alternatives were selected.

- 17th Street Canal – Layout Alternative C
- Orleans Avenue Canal – Layout Alternative A
- London Avenue Canal – Layout Alternative A

The total estimated cost developed under this study to implement Option 1 for all three canals is $475,676,894, which compares favorably with the cost previously estimated by New Orleans District of $530,000,000.

The total estimated cost developed under this study to implement Option 2 for all three canals is $1,413,939,450, which compares less favorably with the cost previously estimated by others of $720,000,000.

CONCEPTUAL DESIGN STUDY

2.0 INTRODUCTION

The following background excerpt is from a USACE New Orleans District White Paper on Permanent Flood Gates and Pump Station Project dated 6 June 06.

2.1 Background

The existing major pump stations are located on what generally constituted the fringe of New Orleans when the city had not expanded to the shore of Lake Pontchartrain. Drainage pump stations (DPS) Nos. 3, 6, and 7 were constructed between 1897 and 1903, and DPS 4 was constructed in the 1940's. As the city expanded north to Lake Pontchartrain, a need developed to carry interior drainage water from the pump stations to the lake. In the 1980s the Corps recommended constructing gated closure structures at the lakefront for the 17th Street, Orleans, and London Avenue canals as part of the Lake Pontchartrain and Vicinity Hurricane Protection Project. The local sponsor disagreed with this recommendation and at their request Congress directed the Corps to construct parallel protection along the outfall canals.

Jefferson Parish has proposed an option to redirect flow away from 17th Street Canal and into temporary storage and divert flow to the Mississippi River. This option would reduce flow to DPS No. 6 by approximately 800 cubic feet per seconds (cfs). The option was reviewed in detail as part of the Project Information Report for Orleans East Bank – revision #02 (May 2002) and determined to be not a cost effective alternative. Jefferson Parish is aware of this analysis and determination.

Following Hurricane Katrina, an Interagency Performance Task Force (IPET) convened to evaluate the cause and identify corrective measures in the event of future major storm/hurricane events. IPET recommended that interim followed with permanent closure structures and increased pumping capacity be installed.

Serving as an interim measure, temporary closure gates and pump stations are currently being installed at the mouths of all three canals. These facilities are intended to be a stop-gap measure to protect against severe storm/hurricane flooding until permanent facilities/measures are installed. These permanent measures are the focus of this conceptual study.

2.2 Permanent Measure Description

The permanent closure structures will prevent storm surge from Lake Pontchartrain entering the three outfall canals. The recommended pumping capacities at the permanent structures are 12,500 cfs at 17th Street Canal, 3,390 cfs at Orleans Canal, and 8,980 cfs at London Avenue Canal. The conceptual plan is to review the following two options:

> *Option 1 – Construction of new permanent Gated Pump Stations at the mouths of the 17th Street, Orleans, and London Avenue Canals.* This alternative provides permanent gates and pump stations at the mouths of the outfall canals, with the permanent pumping stations serving as an integral part of the hurricane protection system. This alternative leaves in-place the floodwalls that flank the three outfall canals. The existing Sewerage and Water Board of New Orleans (S&WB) pump stations would be left in place to function in their current mode of operation, lifting water to lake level in the outfall canals, with gravity drainage to Lake Pontchartrain. The new permanent lakefront structures would be equipped with gates. The gates would remain open to allow flow-through drainage during ordinary conditions and close only during times of high storm surges. Normal lake elevations are generally higher than the ground elevations of the areas through which the canals pass (often by five feet or more), so, with the gates left open most of the time, the floodwalls would remain an integral part of the city's flood protection system.

> *Option 2 - Construction of new Replacement Pump Stations at the mouths of the 17th Street, Orleans, and London Avenue Canals.* The stations would be constructed as permanent closures of the canals requiring full time operation of these pump stations. The levee and floodwalls along the canals themselves would no longer be required as part of the Hurricane Protection system (HPS), eliminating nearly 13 miles of floodwalls. The existing S&WB pump stations on the outfall canals would be taken out of commission. Because the new stations would completely separate the canals from Lake Pontchartrain's influence with the canals at a new and much lower flowline, the banks of these canals would be reshaped to lower elevations, essentially reconstructing the canal system. The canals' hydraulic grade lines would be lowered substantially and the canals would be lined with concrete. The canal modifications will require substantial bridge modifications along the length of each canal.

Dry weather flows have been identified as an issue that is currently being dealt with via dedicated pumps at each pump station (DPS3, DPS6, and DPS7) that divert these low flows to a separate discharge to the Mississippi River. These flows have been determined to contain elevated levels of sewage that can be tolerated better in the river than the lake. This study does not address this issue and is considered to be an upstream issue of pump stations DPS3, DPS6, and DPS7 and will be addressed separately along with other upstream hydraulic and capacity issues.

2.3 Conceptual Study Scope

The G.E.C., Inc. (GEC) and Black & Veatch (B&V) team was contracted to perform a study to provide economic as well as other considerations that will allow the stakeholders to make an educated decision on which option to pursue. Both pump station options are to be studied and results provided. The first option is that these pump stations (Gates Pump Stations) will operate only during elevated lake levels and when the canal outfalls closure structures are in the closed position. The second option is that these pump stations (Replacement Pump Stations) would completely replace the existing S&WB pump stations, would operate constantly, and would become the drainage outfall pump stations for the city of New Orleans. The final report will provide a comparison of the options including life cycle management cost analyses.

The study was a collaborative effort between the GEC/B&V team and the New Orleans District of the United States Army Corps of Engineers. A series of workshops, weekly conference calls, and submittals culminated in this report. This study was performed using the following data and criteria:

- Use only existing data furnished by the New Orleans District
- No new field data will be generated
- Best engineering judgment must prevail in many instances to meet time constraints
- Site locations for the closure structures and pump stations are approximate and not fixed
- Conditions in the field are indicated on New Orleans District provided data and maps
- The New Orleans District provides all available subsurface data, including boring logs and locations and test data
- The New Orleans District provides all relevant modeling results to date

3.0 CRITERIA

New Orleans District and stakeholder provided design criteria:

1. Fuel system storage requirement = 4 days full flow

2. Static Lake Water Elevation = 12’

3. Lake Wave Run-up for vertical walls = 9’

4. Lake Wave Run-up for sloped walls = 4.5’

5. Canal Contamination Depth (not hazardous) = 3’

6. Interim Gate Capacity equals or exceeds permanent pump station capacities in scope.

7. Life cycle cost calculations based on 50 year life

8. Hammond Bridge is flood proof and can be added to the HPS system without modification.

9. No adverse subsidence will be imparted to the surrounding neighborhood.

10. General Groundwater Elevation in Orleans parish = -4’ to -6’

11. General Groundwater Elevation in Jefferson parish = -3’ to -1’

12. Use of property in the Bucktown area is a viable alternative but due to its historic significance, property acquisition could be as long as 10 years.

13. Safe Water Elevations are as follows (NAVD88 Datum). For Option 1, these safe water elevations are defined to equal maximum allowable canal water surface elevation at any point along the canal.

    - 17th Street Canal = 5.0 ft
    - Orleans Canal = 9.0 ft
    - London Avenue Canal = 5.0 ft

14. The steady state HEC-RAS hydraulic model provided by the USACE provides the basis for canal hydraulic evaluations. The model is based on NGVD 29 datum.

15. Design canal and pumping station capacities are as follows:

17th Street Canal	Capacity
Existing DPS 6 capacity	9,480 cfs
Potential DPS 6 capacity increase	2,000 cfs
Canal Street Pump Station	160 cfs
I-10 Pump Station	860 cfs
Required capacity of new pumping station	**12,500 cfs**

Orleans Avenue Canal	Capacity
Existing DPS 7 capacity	2,690 cfs
Potential DPS 7 capacity increase	700 cfs
Required capacity of new pumping station	**3,390 cfs**

London Avenue Canal	Capacity
Existing DPS 3 capacity	4,260 cfs
Existing DPS 4 capacity	3,720 cfs
Potential new pumping station capacity, to be located on opposite side of canal from DPS 4	1,000 cfs
Required capacity of new pumping station	**8,980 cfs**

16. Property acquisition for Right-of-Way will be difficult but should not disqualify a location being considered.

17. Option 1 – Gated Pump Stations. New Gated Pump Stations will operate only when the combination of Lake Pontchartrain elevation and canal discharge would cause the canal water surface elevation to exceed the defined safe water elevation.

18. Option 2 – Replacement Pump Stations. The Replacement Pump Stations would operate to pump all canal discharges, except for any dry weather discharges presently pumped to the Mississippi River from each pump station. Design canal flow line elevations (NAVD 88) at design canal discharge are as follows:

17th Street Canal	**WSEL**
DPS 6 suction side	-10.9 ft
Orleans Canal	WSEL
DPS 7 suction side	-9.4 ft
London Avenue Canal	WSEL
DPS 3 suction side	-9.9 ft
DPS 4 suction side	-10.4 ft

19. Criteria for sizing the new Option 1 gates, in terms of combination of canal discharge and lake elevation needed.

20. The following datum conversions are to be used:

   - NGVD29 – 0.5 ft = NAVD88 (2004.65)
   - Cairo Datum – 20.93 ft = NAVD88 (2004.65)

Study criteria developed using engineering calculations and models:

1. **Pump station capacity.** The flow capacity is clearly delineated in the scope, but the conditions at which this design flow must occur was not clear. Two schools of thought were debated by USACE and the stakeholders. An extreme condition of design flow occurring at the maximum head condition imposed by the lake was discussed. A more moderate position is the worst case between the primary condition of maximum flow at normal lake water surface elevation with tide and the secondary condition of 60 percent of maximum flow at maximum lake elevation with tide. The flowing table illustrates the calculations and results evaluated.

	1000 cfs Pump			**500 cfs Pump**			**250 cfs Pump**	
	Opt 1	**Opt 2**		**Opt 1**	**Opt 2**		**Opt 1**	**Opt 2**
Original Corps requirement of maximum flow at maximum head.								
Maximum still Lake w/o wave runup	12	12		12	12		12	12
Max design canal operating level	1	-11.7		1	-11.7		1	-11.7
Min design canal operating level	-1	-13.7		-1	-13.7		-1	-13.7
Design static head - max canal to max lake	11	23.7		11	23.7		11	23.7
Pump Station Losses	1.5	1.5		1.5	1.5		1.5	1.5
Screen Losses, needing cleaning	0.5	0.5		0.5	0.5		0.5	0.5
Total Dynamic Head	13	25.7		13	25.7		13	25.7
Pump Flow Rate, cfs	1000	1000		500	500		250	250
Pump Flow Rate, gpm	453000	453000		226500	226500		113250	113250
Pump water horsepower	1487	2940		744	1470		372	735
Pump efficiency	85%	85%		85%	85%		85%	85%
Gear reducer efficiency	98%	98%		98%	98%		98%	98%
Engine de-rating	15%	15%		15%	15%		15%	15%
Motor efficiency	94%	94%		94%	94%		94%	94%
Minimum engine rating for direct drive	**2064**	**4080**		**1032**	**2040**		**516**	**1020**
Maximum bhp to size motor	**1750**	**3459**		**875**	**1729**		**437**	**865**

Parish requirement of max flow at normal lake w/tide - Primary Condition								
Normal still Lake w/o surge or tide	1	1		1	1		1	1
Normal still Lake w/tides	4	4		4	4		4	4
Max design canal operating level	1	-11.7		1	-11.7		1	-11.7
Min design canal operating level	-0.5	-13.7		-0.5	-13.7		-0.5	-13.7
Design static head - max canal to max lake	3	15.7		3	15.7		3	15.7
Pump Station Losses	1.5	1.5		1.5	1.5		1.5	1.5
Screen Losses, needing cleaning	0.5	0.5		0.5	0.5		0.5	0.5
Total Dynamic Head	5	17.2		5	17.2		5	17.7
Primary Pump Flow Rate, cfs	1000	1000		500	500		250	250
Pump Flow Rate, gpm	453000	453000		226500	226500		113250	113250
Pump water horsepower	572	1968		286	984		143	506
Pump efficiency	85%	85%		85%	85%		85%	85%
Gear reducer efficiency	98%	98%		98%	98%		98%	98%
Engine de-rating	15%	15%		15%	15%		15%	15%
Motor efficiency	94%	94%		94%	94%		94%	94%
Minimum engine rating for direct drive	794	2730		397	1365		198	702
Maximum bhp to size motor	673	2315		336	1157		168	596

Parish requirement of 60 percent flow at lake w/tide - Secondary Condition								
Maximum still Lake w/o wave runup	12	12		12	12		12	12
Max design canal operating level	1	-11.7		1	-11.7		1	-11.7
Min design canal operating level	-0.5	-13.7		-0.5	-13.7		-0.5	-13.7
Design static head - max canal to max lake	11	23.7		11	23.7		11	23.7
Pump Station Losses	1.5	1.5		1.5	1.5		1.5	1.5
Screen Losses, needing cleaning	0.5	0.5		0.5	0.5		0.5	0.5
Total Dynamic Head	13	25.7		13	25.7		13	25.7
Primary Pump Flow Rate, cfs	1000	1000		500	500		250	250
Percent of Primary Flow, cfs	60%	60%		60%	60%		60%	60%

Secondary Pump Flow Rate, cfs	600	600		300	300		150	150
Pump Flow Rate, gpm	271800	271800		135900	135900		67950	67950
Pump water horsepower	892	1764		446	882		223	441
Pump efficiency	75%	75%		75%	75%		75%	75%
Gear reducer efficiency	98%	98%		98%	98%		98%	98%
Engine de-rating	15%	15%		15%	15%		15%	15%
Motor efficiency	94%	94%		94%	94%		94%	94%
Minimum engine rating for direct drive	1403	2774		702	1387		351	694
Maximum bhp to size motor	1190	2352		595	1176		297	588
For the Orleans Water Board direction								
Minimum engine rating for direct drive	**1403**	**2774**		**702**	**1387**		**351**	**702**
Maximum bhp to size motor	**1190**	**2352**		**595**	**1176**		**297**	**596**

Based on the impact the Corps requirement would have on the pump sizing and project cost, the stakeholders directed the study to be conducted based on Parish requirement. The sizing was then further refined to include additional losses and start-up sizing requirements to arrive at the appropriate motor and engine sizes to use for costing and building sizing. Following is a table of the final sizing:

Option 1

	1000 cfs	500 cfs	250 cfs
Nameplate rating, hp For priming <15 minutes	2750	1500	750
Max continuous duty load, hp	1800	900	450

Option 2

	1000 cfs	500 cfs	250 cfs
Nameplate rating, hp For priming <15 minutes	4550	2250	1250
Max continuous duty load, hp	3825	1925	950

2. **Site Selection.** Three sites were considered for each of the three canals and for each option in a given canal resulting in 18 different site alternatives. Although each of the alternatives have desirable and undesirable traits, for the purposes of this study, Alternative C for 17th Street Canal, Alternative A for Orleans Canal, and Alternative A for London Avenue Canal are used for study development and costing purposes.

3. **Central Power Plant vs Site-Dedicated Power Plant.** The pump stations that form the permanent solution require a significant amount of electrical power and

pose concerns because of the location and the intermittent use. An evaluation of the economics of using a central plant versus site-dedicated plants was started to determine if one of the two approaches provided a greater benefit. A complete evaluation of this type was beyond the scope of this study, but preliminary results are presented in Appendix E. For the purposes of this study, site dedicated plants are used

4. **Canal Hydraulics.** Based on safe water surface elevations, canal inverts, and other water data provided by USACE, hydraulic modeling was performed to determine channel cross-sections and inverts that satisfy the flow requirements. The following table summaries the results of the modeling that impacts canal geometry which is used as the study basis throughout the report.

New Orleans Pump Stations
Option 2 Canal Hydraulics
Degraded Canal Section - Rectangular Cross Section, Concrete Lined

Canal	Total Q	x-Section	At Suction Side-New PS		At Existing DPS	
	cfs	Width ft	Flowline EL ft, NAVD88	Invert EL ft, NAVD88	Flowline EL ft, NAVD88	Invert EL ft, NAVD88
17th	12500	150	-13.0	-25.3	-10.9	-25.3
Orleans	3390	75	-13.0	-19.5	-9.4	-19.5
London	8980	100	-13.0	-25.6	-9.9	-19.6

New Orleans Pump Stations
Option 1 Canal Hydraulics - Pumping Mode
Existing Canal Geometry

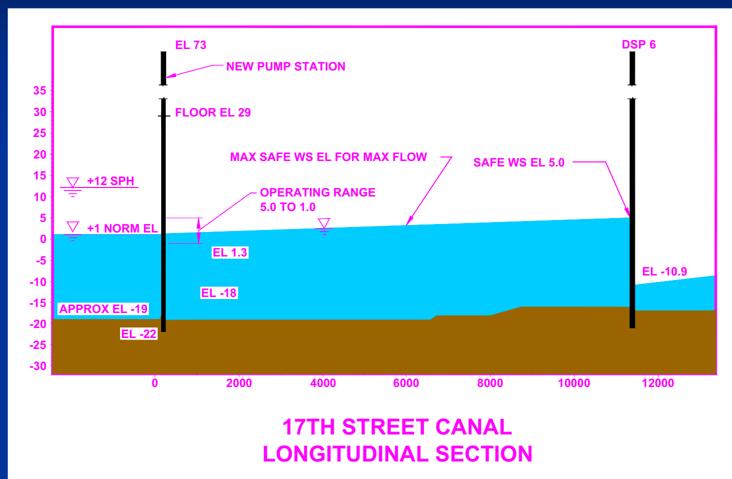
Canal	Total Q	Safe Canal	At Suction Side-New PS		At Existing DPS	
	cfs	WSEL ft, NAVD88	Flowline EL ft, NAVD88		Flowline EL ft, NAVD88	
17th	12500	5.0	1.3		5.0	
Orleans	3390	9.0	8.3		9.0	
London	8980	5.0	-0.5		7.3	Cannot achieve El. 5.0 with exist. Canal geometry

5. **Higher Water Level Cost (Lake Level Increased 5 feet).** To address the scope requirement of generating a supplemental initial cost for each option using an

increased lake level of 5 feet above the lake level used for the studies basis. The higher lake level primarily affects the pump structure, generator structure, and the pumping equipment costs. A brief discussion of the changes required and the associated costs can be found in Appendix H.

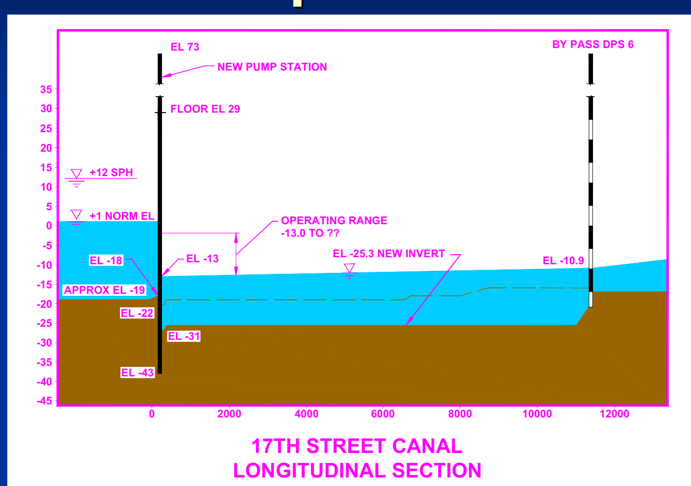
6. **Elevation Summary.** Water surface, canal invert, support/foundation, and pump station elevations are compiled and presented on figures to better describe the overall hydraulics that resulted from the studies calculations and findings.

17 Street Canal
Option 1

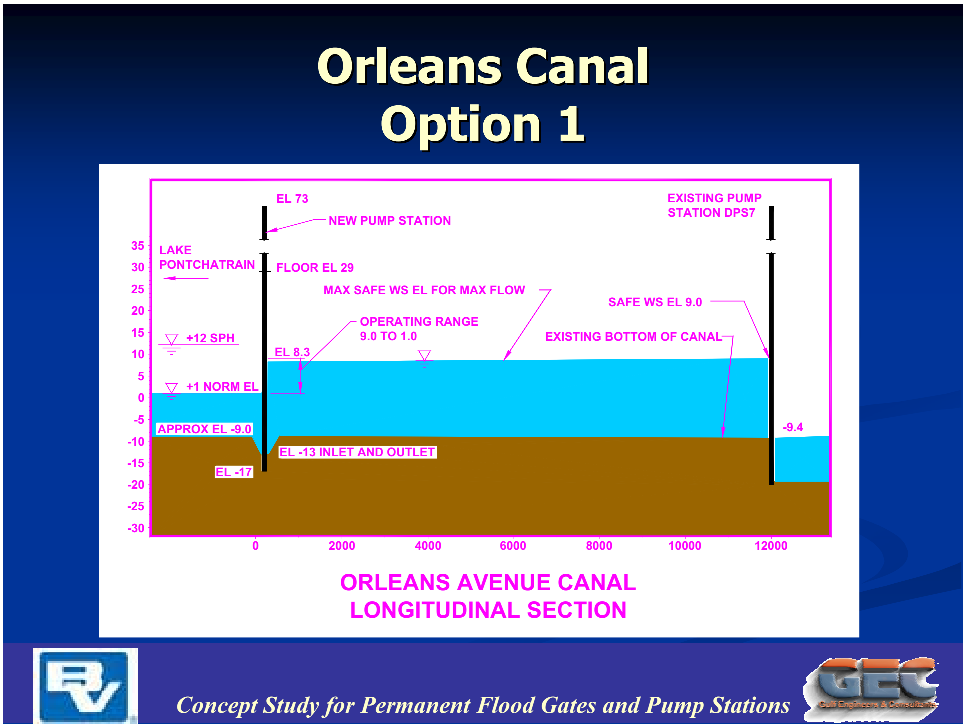


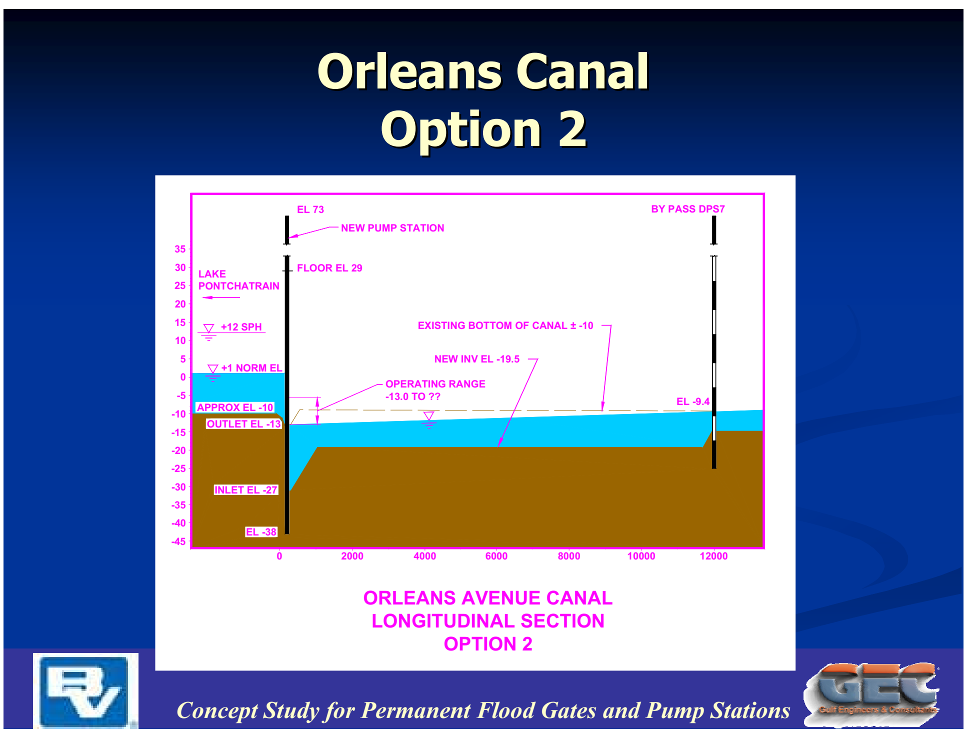
Concept Study for Permanent Flood Gates and Pump Stations

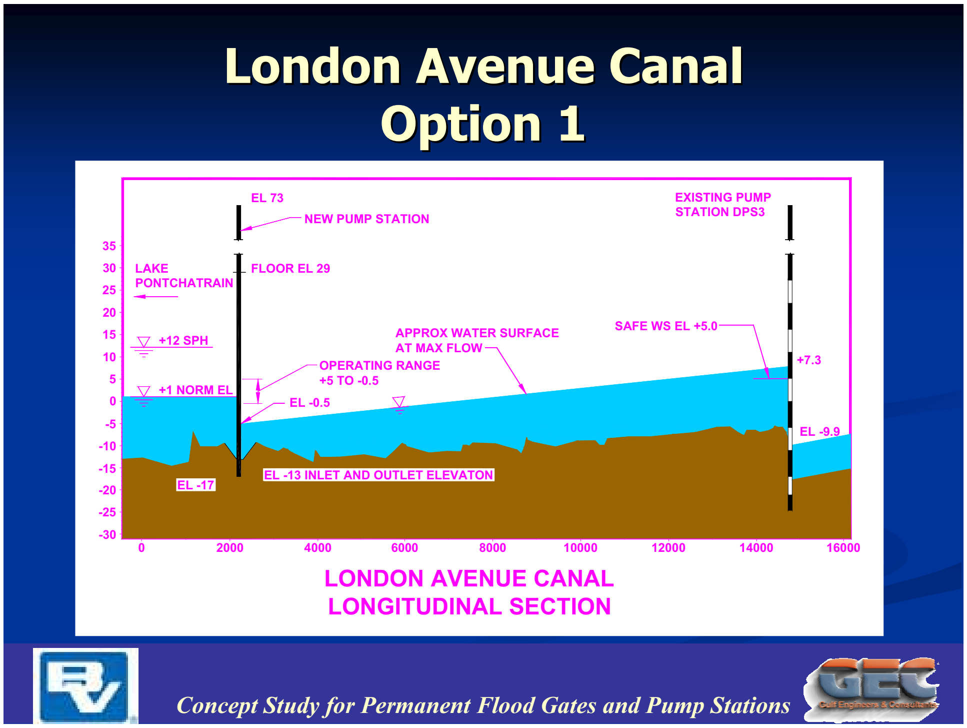
17 Street Canal
Option 2

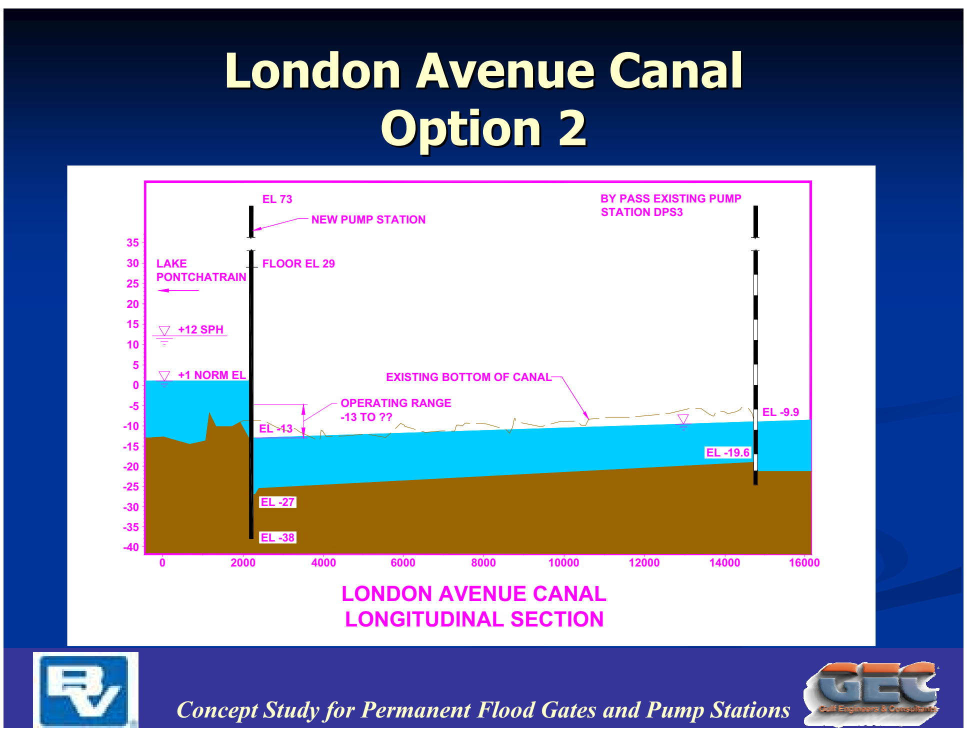


Concept Study for Permanent Flood Gates and Pump Stations

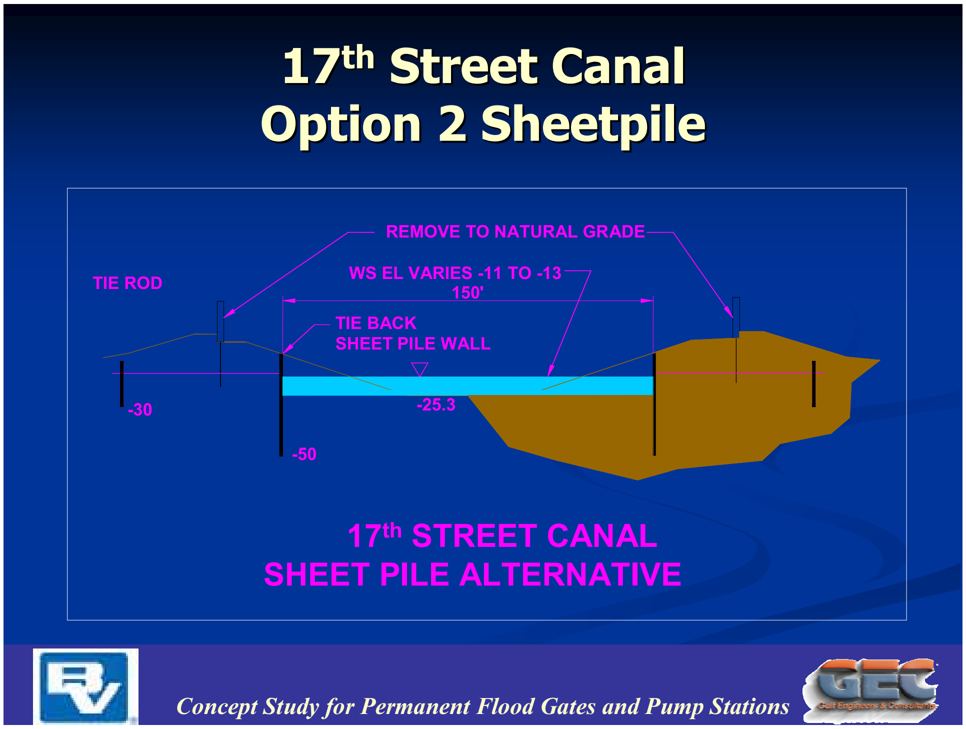

Orleans Canal
Option 1
EL 73
NEW PUMP STATION
EXISTING PUMP
STATION DPS7
35
30
25
20
15
10
5
0
-5
-10
-15
-20
-25
-30
LAKE
PONTCHATRAIN
FLOOR EL 29
MAX SAFE WS EL FOR MAX FLOW
SAFE WS EL 9.0
OPERATING RANGE
9.0 TO 1.0
+12 SPH
EL 8.3
EXISTING BOTTOM OF CANAL
+1 NORM EL
APPROX EL -9.0
-9.4
EL -13 INLET AND OUTLET
EL -17
0
2000
4000
6000
8000
10000
12000
ORLEANS AVENUE CANAL
LONGITUDINAL SECTION
Concept Study for Permanent Flood Gates and Pump Stations

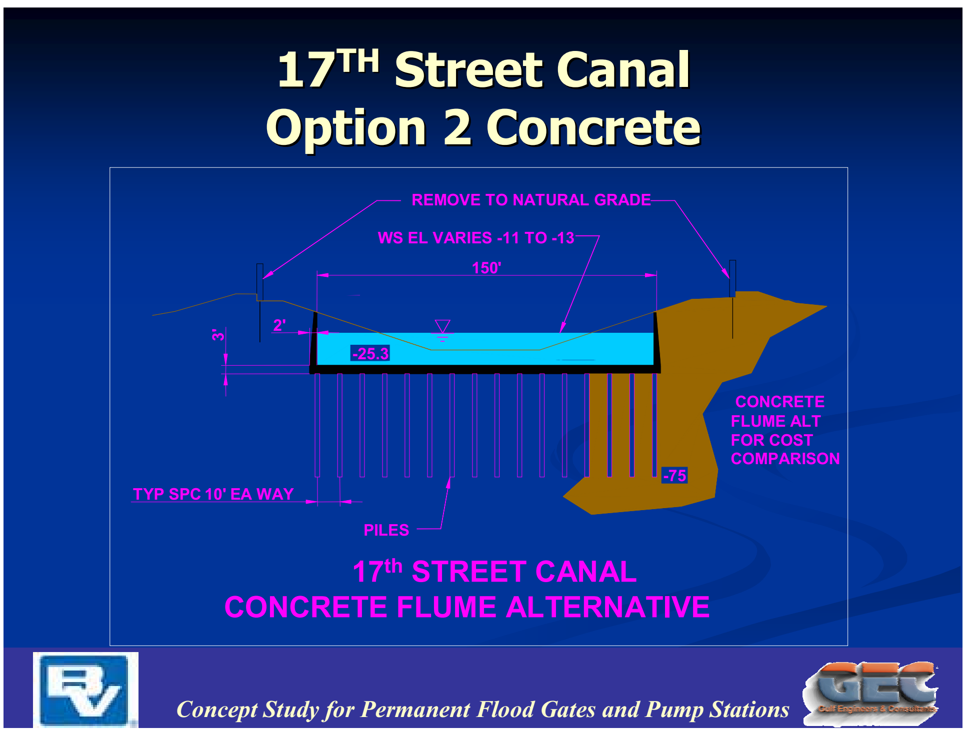

Orleans Canal
Option 2
EL 73
NEW PUMP STATION
BY PASS DPS7
35
30
25
20
15
10
5
0
-5
-10
-15
-20
-25
-30
-35
-40
-45
LAKE
PONTCHATRAIN
FLOOR EL 29
+12 SPH
EXISTING BOTTOM OF CANAL ± -10
NEW INV EL -19.5
+1 NORM EL
OPERATING RANGE
-13.0 TO ??
EL -9.4
APPROX EL -10
OUTLET EL -13
INLET EL -27
EL -38
0
2000
4000
6000
8000
10000
12000
ORLEANS AVENUE CANAL
LONGITUDINAL SECTION
OPTION 2
Concept Study for Permanent Flood Gates and Pump Stations

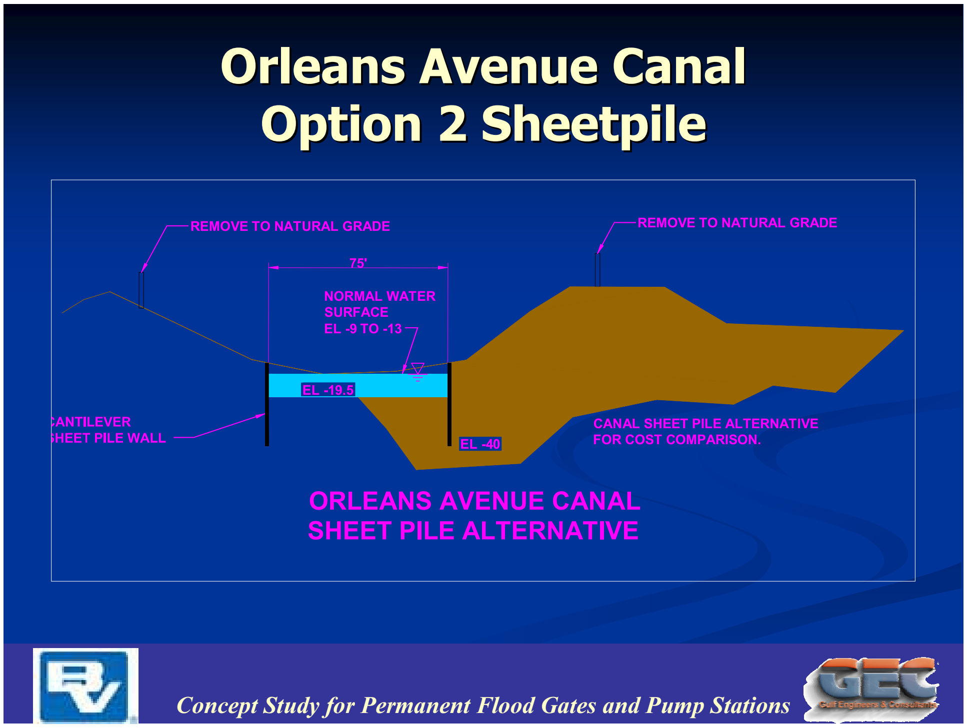

London Avenue Canal
Option 1
EL 73
NEW PUMP STATION
EXISTING PUMP
STATION DPS3
LAKE
PONTCHATRAIN
FLOOR EL 29
+12 SPH
+1 NORM EL
APPROX WATER SURFACE
AT MAX FLOW
SAFE WS EL +5.0
+7.3
OPERATING RANGE
+5 TO -0.5
EL -0.5
EL -9.9
EL -17
EL -13 INLET AND OUTLET ELEVATON
35
30
25
20
15
10
5
0
-5
-10
-15
-20
-25
-30
0
2000
4000
6000
8000
10000
12000
14000
16000
LONDON AVENUE CANAL
LONGITUDINAL SECTION
Concept Study for Permanent Flood Gates and Pump Stations

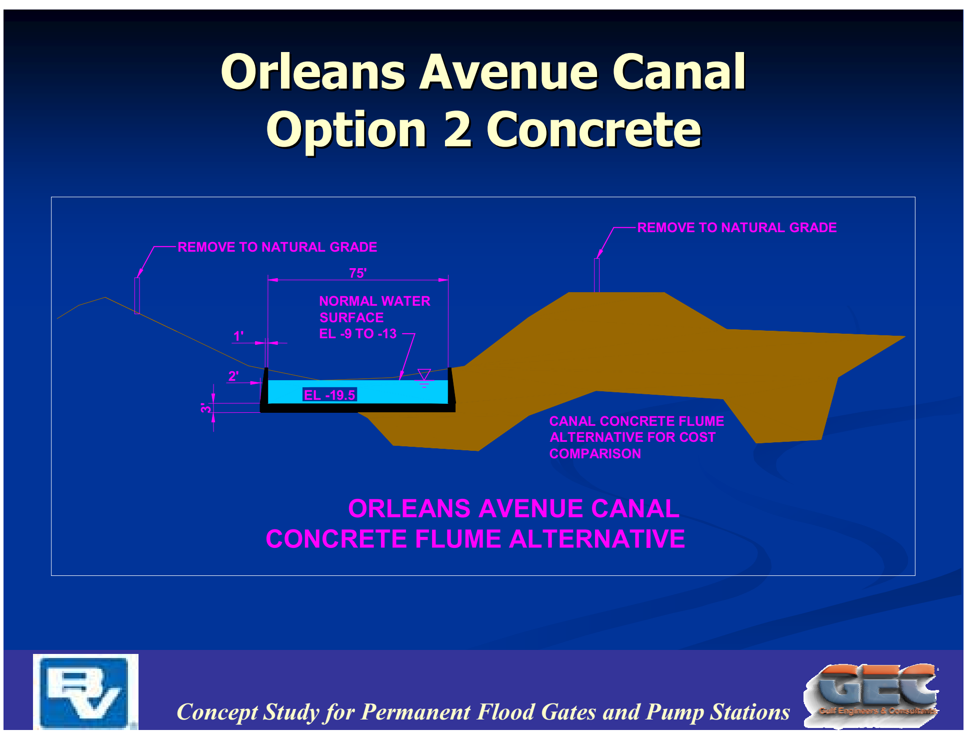

London Avenue Canal
Option 2
EL 73
NEW PUMP STATION
BY PASS EXISTING PUMP
STATION DPS3
LAKE
PONTCHATRAIN
FLOOR EL 29
+12 SPH
+1 NORM EL
EXISTING BOTTOM OF CANAL
OPERATING RANGE
-13 TO ??
EL -13
EL -9.9
EL -19.6
EL -27
EL -38
35
30
25
20
15
10
5
0
-5
-10
-15
-20
-25
-30
-35
-40
0
2000
4000
6000
8000
10000
12000
14000
16000
LONDON AVENUE CANAL
LONGITUDINAL SECTION
Concept Study for Permanent Flood Gates and Pump Stations

7. **Canal Liner.** Two types of canal liners were evaluated and the discussion is located in Geotechnical section. Following are graphical representations of these liner types. For the purposes of this study, the sheet pile type was used for costing.

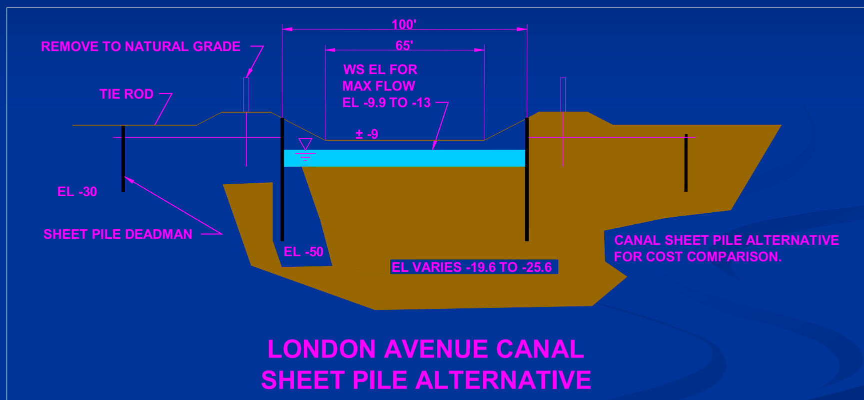





Orleans Avenue Canal
Option 2 Sheetpile
REMOVE TO NATURAL GRADE
REMOVE TO NATURAL GRADE
75'
NORMAL WATER
SURFACE
EL -9 TO -13
EL -19.5
ANTILEVER
HEET PILE WALL
EL -40
CANAL SHEET PILE ALTERNATIVE
FOR COST COMPARISON.
ORLEANS AVENUE CANAL
SHEET PILE ALTERNATIVE
Concept Study for Permanent Flood Gates and Pump Stations

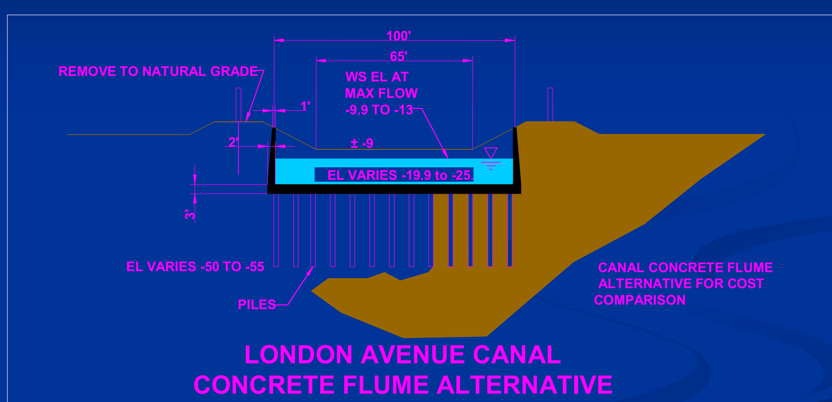

Orleans Avenue Canal
Option 2 Concrete
REMOVE TO NATURAL GRADE
REMOVE TO NATURAL GRADE
75'
NORMAL WATER
SURFACE
EL -9 TO -13
1'
2'
3'
EL -19.5
CANAL CONCRETE FLUME
ALTERNATIVE FOR COST
COMPARISON
ORLEANS AVENUE CANAL
CONCRETE FLUME ALTERNATIVE
Concept Study for Permanent Flood Gates and Pump Stations

London Avenue Canal
Option 2 Sheetpile



Concept Study for Permanent Flood Gates and Pump Stations

London Avenue Canal
Option 2 Concrete



Concept Study for Permanent Flood Gates and Pump Stations

4.0 EXISTING FACILITIES

The existing equipment that is within the scope of this study currently and is part of the flood protection system includes pump stations DPS 3, DPS4, DPS6, and DPS7 and canals for 17th Street, Orleans, and London Avenue. Basic information is included below as reference to better understand the function and capability of each of these flood protection elements. Other pump stations and canals also form part of the flood protection system but are outside the scope of this study.

4.1 Pump Station DPS3

DPS3 is located at the head of the London Avenue canal and currently lifts drainage water to allow gravity flow from the pump station discharge to Lake Pontchartrain. 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 and is currently on the National Register of Historic Places due to the Wood screw pumps, the early architectural style and the historical importance of the drainage system of New Orleans. The following table lists the major equipment that constitutes this pump station.

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
B	550	H/E	25	
C	1000	H/E	25	
D	1000	H/E	25	
E	1000	H/E	25	
CD1L/1R	80	C/E	25	
CD2L/1R	80	C/E	25	
Total	4260			

4.2 Pump Station DPS4

DPS4 is located at the midpoint of the London Avenue canal and currently lifts drainage water to allow gravity flow from the pump station discharge to Lake Pontchartrain. 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 and is not listed on the HRHP. The following table lists the major equipment that constitutes this pump station.

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
2	320	C/E	60	
C	1000	H/E	25	
D	1000	H/E	25	
E	1000	H/E	25	
CD1	80	V/E	25	
Total	3720			

4.3 Pump Station DPS6

DPS6 is located at the head of the 17th Street canal and currently lifts drainage water to allow gravity flow from the pump station discharge to Lake Pontchartrain. 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 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 and is currently on the National Register of Historic Places due to the Wood screw pumps, the early architectural style and the historical importance of the drainage system of New Orleans. Two additions to the pump station were added in years unknown and are not currently considered historic. The following table lists the major equipment that constitutes this pump station.

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
B	550	H/E	25	
C	1000	H/E	25	
D	1000	H/E	25	
E	1000	H/E	25	
F	1000	H/E	25	
G	1000	H/E	25	
H	1100	H/E	60	
I	1100	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	9480			

4.4 Pump Station DPS7

DPS7 is located at the head of the Orleans canal and currently lifts drainage water to allow gravity flow from the pump station discharge to Lake Pontchartrain. 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 and is currently on the National Register of Historic Places due to the Wood screw pumps, the early architectural style and the historical importance of the drainage system of New Orleans. The following table lists the major equipment that constitutes this pump station.

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	1000	H/E	25	
D	1000	H/E	60	
CD1	70	V/E	25	
CD2	70	V/E	25	
Total	2690			

4.5 17th Street Canal

This canal serves to convey drainage water from the western edge of Orleans Parish and the eastern edge of Jefferson Parish. It was constructed at the same time DPS6 was constructed and has undergone canal improvements since its installation. The canal is in excess of two miles in length, has earthen banks and bottom. It is currently lined with a combination of concrete and sheet pile flood walls. It has both railroad (near pump station DPS6) and automobile bridges (I-10, Veterans Memorial Boulevard, and Old Hammond Highway) that span its width. The channel geometry has various configurations along its length which can be found imbedded in the USACE HECRAS models.

4.6 Orleans Canal

This canal serves to convey drainage water from the central area of Orleans Parish. It was constructed at the same time DPS7 was constructed and has undergone canal improvements since its installation. The canal is in excess of two miles in length, has earthen banks and bottom. It is currently lined with a combination of concrete and sheet pile flood walls. It has automobile bridges (I-610, Harrison Avenue, Filmore Avenue, Robert E. Lee Boulevard, and Lakeshore Drive) that span its width. The channel geometry has various configurations along its length which can be found imbedded in the USACE HECRAS models.

4.7 London Avenue Canal

This canal serves to convey drainage water from the eastern edge of Orleans Parish. It was constructed at the same time DPS3 was constructed and has undergone canal improvements since its installation. DPS4 discharges drainage ditch water into the London Avenue Canal. The canal is in excess of 2.5 miles in length, has earthen banks and bottom. It is currently lined with a combination of concrete and sheet pile flood walls. It has both railroad (one near DPS3) and 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. The channel geometry has various configurations along its length which can be found imbedded in the USACE HECRAS models.

5.0 ANALYSIS

This conceptual study began with the project team assembling in New Orleans for the week of Monday, June 5 through Friday, June 9, 2006. Early in the week, the project team visited each subject canal, each ongoing temporary gate structure construction, and Pump Station DPS19 across the IHNC from the Ninth Ward. Later in the week, the team conducted brainstorming sessions, in concert with Corps personnel, to provide a "first cut" review of potential layout arrangements for pump station and canal modification schemes for the two required options. Discussions were intended to be free ranging, and devoid of pre-conceived limitations or constraints, in order to thoroughly consider possible layouts. For each of the three canals, this initial session resulted in three primary layout alternatives that were considered and preliminarily analyzed during that initial brainstorming week. A subsequent Interim Project Review (IPR) was conducted with USACE and other stakeholders to further refine the findings and report.

Note that this study is not a site optimization study. That is, this study's purpose is not to select the best layout alternative possible at each canal. Rather, the intent of this study is to simply select a layout that is, based upon this conceptual analysis, a layout option that represents a reasonably attractive engineering solution that may ultimately be selected by a comprehensive site selection study to be performed subsequently. Therefore, the following sections describe the layout alternatives that were initially considered.

5.1 Option 1 – 17th Street Canal

5.1.1 Alternative Approaches

Three location alternatives were considered for option 1 on the 17th Street canal. The merits of each were evaluated and discussed in detail in the Civil/Site section. For the purposes of this study, Alternative C was chosen as the location to base costing and other engineering considerations.

5.1.2 Engineering Considerations

5.1.2.1 Civil/Site

The 17th Street Pump Station for Option 1 is anticipated to be 450 feet long, when including flood gates, by 155 feet wide. The total length reduces to 400 long when, under Layout Alternative B, the temporary gate structure is made permanent. The total width includes a 45 foot inlet works including trash screens, a 70 foot pump station building housing pumps and motors, and a 40 foot outlet works. Finish grade for the Generator and Tank Farm Complex is always approximately +16.0 elevation.

- The **Alternative A Pump Station Layout** is as shown in Exhibit 5.1.1.A, Appendix C.

<u>General Location and Description</u> - Under this alternative, the pump station is located in the existing canal, as near the Hammond Avenue Bridge as possible without

creating the need for any modifications to that flood-proofed bridge. Thus, the pump station is approximately 700 feet upstream of Hammond Avenue. The pump station width requires residential right-of-way acquisition, which is selected to be acquired on the east side, rather than on the west, in order to preserve homes undamaged on the west bank versus those that are very significantly damaged on the east canal bank.

Under this alternative, the temporary gate structure downstream of Hammond Avenue may remain in place or it may be removed. That is, there may be value in retaining the structure in place, both to avoid demolition costs and to serve as a partial wave attenuator during lake surge events. Levees will be extended back to the pump station from the lake-front system, to maintain the integrity of the lake-front hurricane protection system facing Lake Pontchartrain. No breakwater protection is required.

Right-of-Way Acquisition - Permanent right-of-way acquisition will occur almost exclusively on the east bank of this proposed site, as stated previously. Minor temporary construction easement may be necessary along a relatively narrow strip of the canal west bank.

Demolitions and Earthwork - This layout requires the demolition and removal of heavily damaged residential structures on the east bank, existing levee, and miscellaneous site features in the area. Earthwork at this site is almost exclusively excavation, resulting in a significant volume of earth materials to be removed from the project site.

Channel Transitions - Channel transitions are required both immediately upstream and downstream of the pump station on both banks to ensure laminar flow between the trapezoidal canal cross-section and the rectangular pump station cross-section. The upstream transition assumes a maximum preferred divergence angle of 10 degrees, while the maximum preferred convergence angle for the outlet is approximately 25-30 degrees. Under this study, transition walls are anticipated to be constructed as reinforced concrete counterforted retaining walls that provide a smoothly warped flow surface over the length of the transition. Counterforted retaining walls, in this application, offer the maximum in very long-term durability, low maintenance, and good flow characteristics. Clearly, transition structures could be constructed in other ways that might offer significant cost savings over counterforted walls. Tied-back sheet pile walls could be used, but long term durability and corrosion resistance are issues. Therefore, for conservatism under this study, concrete retaining walls are anticipated, pending subsequent optimization studies on the subject.

Erosion Protection - Given the inland location of this pump station, a relatively small volume of erosion protection armoring will be required; specifically, a strip of riprap protection is anticipated in the widened canal floor, both immediately upstream and downstream of the pump station. No breakwater in Lake Pontchartrain is anticipated to be necessary to protect the pump station discharge under this alternative.

Generator Building and Tank Farm Complex - Supporting the pump station on the east bank adjacent to the right downstream channel transition is a Generator Building and Tank Farm Complex. Finish grade of the Complex is anticipated to be approximately +16.0, to insure its functionality during the design storm event. The Generator Building is anticipated to be approximately 80 by 184 feet, and each of seven 12,000 gallon fuel tank pads is anticipated to be approximately 12 by 28 feet in size. According to NFPA 30, 2003 Edition, diesel fuel is a Class II Combustible Liquid. As such, tanks require only a five foot clear distance from the public way or important buildings on the site. An electrical substation, approximately 20 by 20 feet, is also anticipated. The complex also includes an allowance for parking, general staging and storage space, all concrete paved, and including sidewalk and local site storm drainage features. Utilities will include potable water service, sanitary sewer and natural gas, all connected to the station from existing utilities available within several hundred feet of the proposed site. Although not a significant cost item, the provision of these utilities when the station is operating under total self-sufficiency is anticipated. The paved Complex area will also be enclosed by a chain link security fence, with minor landscaping improvements. All non-paved areas will be seeded to re-establish healthy turf for aesthetics and erosion control.

Summary – This layout is attractive for its protection of the pump station from lake surge effects, thus requiring no breakwater and minimal erosion protection requirements elsewhere. However, it requires a relatively large residential right-of-way acquisition, albeit in a highly damaged area. Further, some construction sequencing is required to maintain canal flow during the construction duration.

- The **Alternative B Pump Station Layout** is as shown in Exhibit 5.1.1.B, Appendix C.

General Location and Description - The primary intent of this alternative is to achieve savings by preserving and modifying the temporary gate structure to remain as a permanent functional gate structure, thus slightly reducing the size of the required pump station. Therefore, this pump station is located just west of the existing canal, angled slightly west to the existing canal centerline. Again, this layout requires right-of-way acquisition. The west canal bank is selected, in order to preserve the more densely developed residential property on the east bank, as well as to take advantage of the significantly shorter distance from pump station to lake discharge. The alternative also requires the removal and replacement of the Hammond Avenue Bridge.

Right-of-Way Acquisition - Permanent right-of-way acquisition will occur almost exclusively on the west bank of this proposed site. North of Hammond Avenue, the property contains some commercial development, but is largely lightly developed or undeveloped. South of Hammond Avenue, the required area is highly developed with commercial operations that remain relatively undamaged. Minor temporary construction easement should be anticipated.

Demolition and Earthwork - As noted, this layout requires the demolition and removal of commercial structures on the east bank, existing levee walls, and miscellaneous site features in the area. Earthwork at this site is almost exclusively excavation, resulting in a significant volume of earth materials to be removed from the project site.

Channel Transitions - Channel transitions are required in this alternative immediately upstream and downstream of the pump station on both banks to ensure laminar flow between the trapezoidal canal cross-section and the rectangular pump station cross-section. For this layout, the west bank upstream transition wall and the east bank downstream transition wall are anticipated to be constructed as reinforced concrete counterforted retaining walls that provide a smoothly warped flow surface over the length of the transition. Note that both the west bank downstream transition and the east bank upstream transition, for this layout, are vertical training walls, since there is no sloped bank to match, These vertical walls represent a significantly simpler required construction than the warped counterforted retaining walls required elsewhere.

Erosion Protection - Given the lakeshore location of this pump station, a significant volume of erosion protection armoring will be required; specifically, a strip of riprap protection is anticipated in the widened canal floor, both immediately upstream and downstream of the pump station. Also, a breakwater in Lake Pontchartrain is anticipated to be necessary to protect the pump station discharge. A detailed discussion of the conceptual breakwater design is included in Appendix C.

Generator Building and Tank Farm Complex - Supporting the pump station on the west bank adjacent to the left upstream channel transition is a Generator Building and Tank Farm Complex. The Generator Building is anticipated to be approximately 80 by 184 feet, and each of seven 12,000 gallon fuel tank pads is anticipated to be approximately 12 by 28 feet in size. According to NFPA 30, 2003 Edition, diesel fuel is a Class II Combustible Liquid. As such, tanks require only a five foot clear distance from the public way or important buildings on the site. An electrical substation, approximately 20 by 20 feet, is also anticipated. The complex also includes parking, general staging and storage space, all concrete paved, and including sidewalk and local site storm drainage features. Utilities will include potable water service, sanitary sewer and natural gas, all connected to the station from existing utilities available within several hundred feet of the proposed site. Although not a significant cost item, the provision of these utilities when the station is operating under total self-sufficiency is anticipated. The paved Complex area is also anticipated to be enclosed by a chain link security fence, with minor landscaping improvements. All non-paved areas will be seeded to re-establish healthy turf for aesthetics and erosion control.

Summary – This layout is attractive for its cost savings in converting the temporary gate structure to a permanent feature, which avoids demolition cost and reduces the pump station structure somewhat. However, it requires a significant right of way acquisition of active, fully-developed commercial property, including much of the historic Bucktown area. It also requires the demolition and replacement of the recently completed Hammond Avenue Bridge, and it may impact property on the west bank currently in active use by the U.S. Coast Guard. Thus, since negative aspects of the layout are significant, this alternative is not recommended for further consideration.

- The **Alternative C Pump Station Layout** is as shown in Exhibit 5.1.1.C Appendix C.

General Location and Description - Under this alternative, the pump station is located in the existing canal, downstream of the Hammond Avenue Bridge as far as required to avoid the need for any modifications to that flood-proofed bridge. Thus, the pump station is approximately 1000 feet downstream of Hammond Avenue. The presumed channel transition criteria limit the ability to shift the pump station solely to one side of the canal or the other. Thus, the pump station must impinge on both canal banks. Under this alternative, the temporary gate structure downstream of Hammond Avenue will be removed upon pump station completion.

Right-of-Way Acquisition - Permanent right-of-way acquisition will occur on both banks of the canal for this proposed site, effecting some residential property on the east bank and primarily undeveloped property on the west.

Demolition and Earthwork – This layout requires the demolition and removal of existing, residential structures on the east bank, the temporary gate structure, existing levees, and other miscellaneous site features in the area. Earthwork at this site is almost exclusively excavation, resulting in a significant volume of earth materials to be removed from the project site.

Channel Transitions - Channel transitions are required in this alternative immediately upstream and downstream of the pump station on both banks to ensure laminar flow between the trapezoidal canal cross-section and the rectangular pump station cross-section. For this layout, both upstream transitions are anticipated to be constructed as reinforced concrete counterforted retaining walls that provide a smoothly warped flow surface over the length of the transition. Downstream, only the east bank requires a counterforted retaining wall transition, with the west bank discharging via a short vertical training wall.

Erosion Protection - Given the lakeshore location of this pump station, a significant volume of erosion protection armoring will be required; specifically, a strip of riprap protection is anticipated in the widened canal floor, both immediately upstream and downstream of the pump station. Also, a breakwater in Lake Pontchartrain is anticipated to be necessary to protect the pump station discharge. A detailed discussion of the conceptual breakwater design is included in Appendix C.

Generator Building and Tank Farm Complex - Supporting the pump station on the east bank adjacent to the right downstream channel transition is a Generator Building and Tank Farm Complex. The Generator Building is anticipated to be approximately 80 by 184 feet, and each of seven 12,000 gallon fuel tank pads is anticipated to be approximately 12 by 28 feet in size. According to NFPA 30, 2003 Edition, diesel fuel is a Class II Combustible Liquid. As such, tanks require only a five foot clear distance from the public way or important buildings on the site. An electrical substation, approximately

20 by 20 feet, is also anticipated. The complex also includes parking, general staging and storage space, all concrete paved, and including sidewalk and local site storm drainage features. Utilities will include potable water service, sanitary sewer and natural gas, all connected to the station from existing utilities available within several hundred feet of the proposed site. Although not a significant cost item, the provision of these utilities when the station is operating under total self-sufficiency is anticipated. The paved Complex area is also anticipated to be enclosed by a chain link security fence, with minor landscaping improvements. All non-paved areas will be seeded to re-establish healthy turf for aesthetics and erosion control.

Summary – This layout is attractive for its minimal right-of-way acquisition requirements, its relative ease of constructability, and its in-line, shore-front location. Given that shore-front location, its erosion control requirements are substantial and it does impact the historic Bucktown pedestrian bridge.

5.1.2.2 Bridges and Utilities

Bridges

The 17th Street Canal is crossed by four bridges between the Pump Station No. 6 and the outfall into Lake Pontchartrain (three roadway bridges and one railroad bridge). These bridges and their locations are identified as follows.

Interstate 10 Bridge – Exhibit 5.1.2.2A, Appendix F

The I-10 bridges (eastbound and westbound) cross the 17th Street Outfall Canal approximately 0.5 miles downstream of Pump Station No. 6. Each bridge consists of three continuous concrete slab spans totaling 215’-10” supported by 24” P. P. C. piles with an approximate tip elevation of –93’. The end bents consist of HP 12x48 steel piles with an approximate tip elevation of –106’. There is a sheet pile wall outside of the end bents that extends to an elevation of –2’ on the west side and –5’ on the east side.

Veterans Boulevard Bridges – Exhibit 5.1.2.2B, Appendix F

The two bridges over the 17th Street Outfall Canal at Veterans Boulevard (eastbound and westbound) are located approximately 0.8 miles downstream of Pump Station No. 6. Each bridge consists of five P. P. C. girder spans totaling 228’-04” supported by 24” P. P. C. piles with an approximate tip elevation of –92.5’. The end bents consist of HP 14x73 steel piles with an approximate tip elevation of –95’. There is a sheet pile wall outside of the end bents that extends to an approximate tip elevation of –6’.

Hammond Highway Bridge – Exhibit 5.1.2.2C, Appendix F

The Hammond Highway Bridge over the 17th Street Outfall Canal is located 2.0 miles downstream of Pump Station No. 6. The bridge consists of five P. P. C. girder spans totaling 200’ supported by 24” P. P. C. piles with an approximate tip elevation of –78.5’. The end bents consist of HP 14x73 steel piles with an approximate tip elevation of –76’ with sheet pile walls outside the end bents extending to an approximate tip elevation of –41’.

Southern Railroad Bridge – No exhibits

No information has been made available for this bridge. The Southern R. R. Bridge over the 17th Street Outfall Canal is located less than 0.1 miles downstream of Pump Station No. 6.

None of the bridges are affected by Option 1 except the Hammond Avenue Bridge, which may be affected by the pump station site location. For pump stations layout alternates A and C, there are no affect to the Hammond Avenue Bridge for site location Alternate B the Hammond Avenue Bridge will be replaced and lengthened.

Utilities

The utilities studied in Option 1 are underground or pile supported water, sewer, drainage, electric (transmission and primary) telephone cables, fiber optic cables, and gas. In Option 1, the existing utilities impacted by construction in the vicinity of the 17th Street Canal are those utilities displaced as a result of the new pump station and gated structure in the vicinity of Lake Pontchartrain.

In alternatives A, B and C, for Option 1, the only utilities impacted are above ground secondary electric lines, and small diameter utility service lines that service existing residences and/or light commercial businesses within the required right of way (to be acquired) for each alternative. These utilities will need to be terminated at the edge of the required right of way and removed. The costs to adjust these existing utilities are, therefore, minimal and ancillary to the overall cost of the Option 1 construction costs.

5.1.2.3 Hydraulic

General

The 17th Street Canal segment considered in this study conveys pumped discharges from DPS 6, the Canal Street Pump Station, and the I-10 Pump Station to the canal outfall at Lake Pontchartrain. The safe water elevation within the 17th Street Canal, as provided by the USACE, is El. 5.0 NAVD 88. This elevation is considered to be the maximum allowable water surface elevation at any point along the canal. As a practical matter, the controlling location for this safe water level is DPS 6, since the down-gradient slope of the water surface profile within the canal during typical flow conditions will result in water surface elevations at all other points that are lower than the water surface elevation at DPS 6.

For purposes of this study, it is assumed the pumping capabilities of the existing pumping facilities would be modified, as necessary, to pump at the design discharge capacity and at a head corresponding to the defined safe canal water surface elevation. These modifications, if required, are not considered in this study. It is recognized the rated head of the existing pumping facilities may be less than the required head for the defined safe canal water elevation. If a lower canal water surface elevation at the discharge side of the existing pumping facilities is considered appropriate, the hydraulic analysis presented herein would require revision to account for this lower canal water elevation.

Hydraulic analysis of the canal was performed to determine the following:

- During pumping mode at the design canal discharge condition, determine the maximum canal water surface elevation at the suction side of the Gated Pump Station that will result in a canal water surface elevation at DPS 6 equal to the safe water elevation. This information is necessary to determine pumping head requirements.

- During gates-open operating mode, determine the canal water surface elevation at DPS 6 for various combinations of Lake Pontchartrain elevation and canal discharge. For the given safe water elevation, this will indicate when gate closure is required, with transition from gates-open to pumping mode. This information, in combination with annual canal discharge and Lake Pontchartrain elevation data, will be used to determine annual pumping requirements.

The USACE developed a HEC-RAS computer hydraulic model to estimate canal water surface profiles and other hydraulic information for various combinations of Lake Pontchartrain elevation and canal discharge. The model includes the existing canal cross-section geometry and invert slope between DPS 6 and Lake Pontchartrain. The model also includes the canal cross-section geometry at the several bridge crossings and accounts for inflows representing discharges from each canal pumping station. This hydraulic model was used as the basis for the hydraulic analyses performed for this study. Modeled canal inflows and starting water surface elevations were adjusted appropriately to represent the conditions being considered for this Option. The existing canal geometry was considered to remain unchanged. The hydraulic model was developed based on NGVD29 datum; therefore, subtraction of 0.5 feet from model elevations is necessary for conversion to NAVD88 datum. A simplified flow schematic of the HEC- RAS model is shown in Figure 5.1.2.3-1.

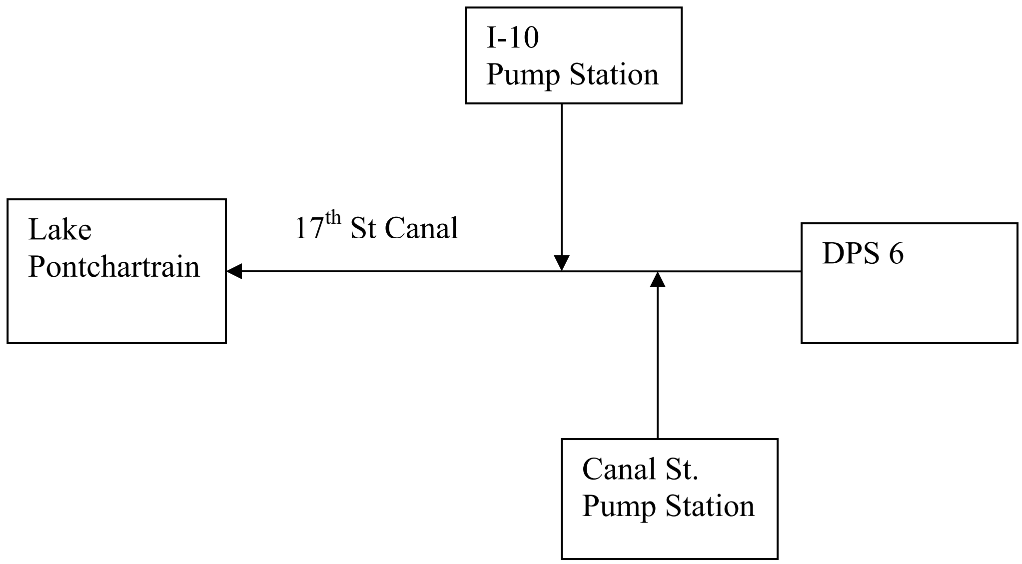


Figure 5.1.2.3-1. HEC-RAS Model Flow Schematic – 17th Street Canal

Hydraulic Analysis - Pumping Mode

The Gated Pumping Station was considered to have a pumping capacity corresponding to the combined capacity of each pumping station discharging into the canal. The existing and potential future capacities of the pumping stations used for this study, and the required pumping capacity of the Gated Pumping Station are as follows:

17th Street Canal Pumping 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
Gated Pumping Station required capacity	**12,500 cfs**

Using the existing canal cross-section geometry and invert profile, as provided in the USACE-developed HEC-RAS model, along with the design pumping station capacities indicated in the above table, an iterative approach was used to determine the maximum canal water surface elevation at the suction side of the Gated Pump Station that would result in a canal water surface elevation at DPS 6 equal to the defined safe water elevation. The HEC-RAS model was run for several starting suction side water surface elevations and the resulting canal water surface elevation at DPS 6 was determined for each case. Based on the results of this analysis for existing conditions, it was determined that the defined safe water surface elevation could not be achieved upstream from the railroad bridge. The results of this analysis are shown in Figure 5.1.2.3-2 and indicate that at a minimum starting suction side water elevation of

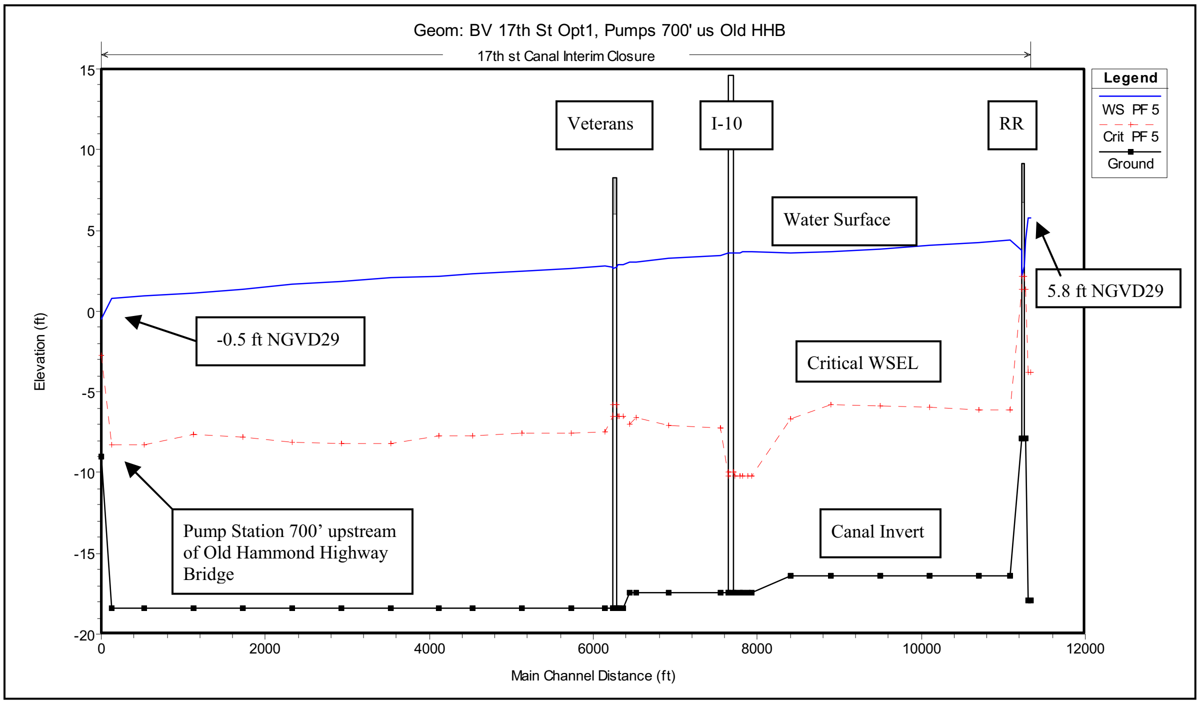


Figure 5.1.2.3-2. Water Surface Profile For Existing Conditions – 17th Street Canal (Option 1)

-10 ft., NAVD88, the water surface elevation in the canal upstream from the railroad bridge is El. 5.3 ft., NAVD88. This starting water surface elevation results in a flow condition approaching critical depth just upstream from the Gated Pump Station, and was selected to represent an extreme minimum starting water surface condition, rather than a practical flow condition.

The canal invert elevation beneath the railroad bridge is approximately 10 feet higher than the invert elevation on the upstream and downstream sides of the bridge. This is likely attributed to construction of the canal with the railroad bridge already in place. This raised invert elevation beneath the railroad bridge was found to act as a hydraulic control at the design discharge. Consideration was given to modifying the railroad bridge by lowering the canal invert to eliminate the hydraulic control. The hydraulic model was modified for this case and the model was re-run. Based on this condition, the defined safe water surface elevation could be achieved upstream from the railroad bridge. The maximum corresponding suction side water surface elevation at the Gated Pump Structure was determined to be El. 1.3 ft, NAVD 88. The water surface profile within the canal for this flow condition is provided in Figure 5.1.2.3-3.

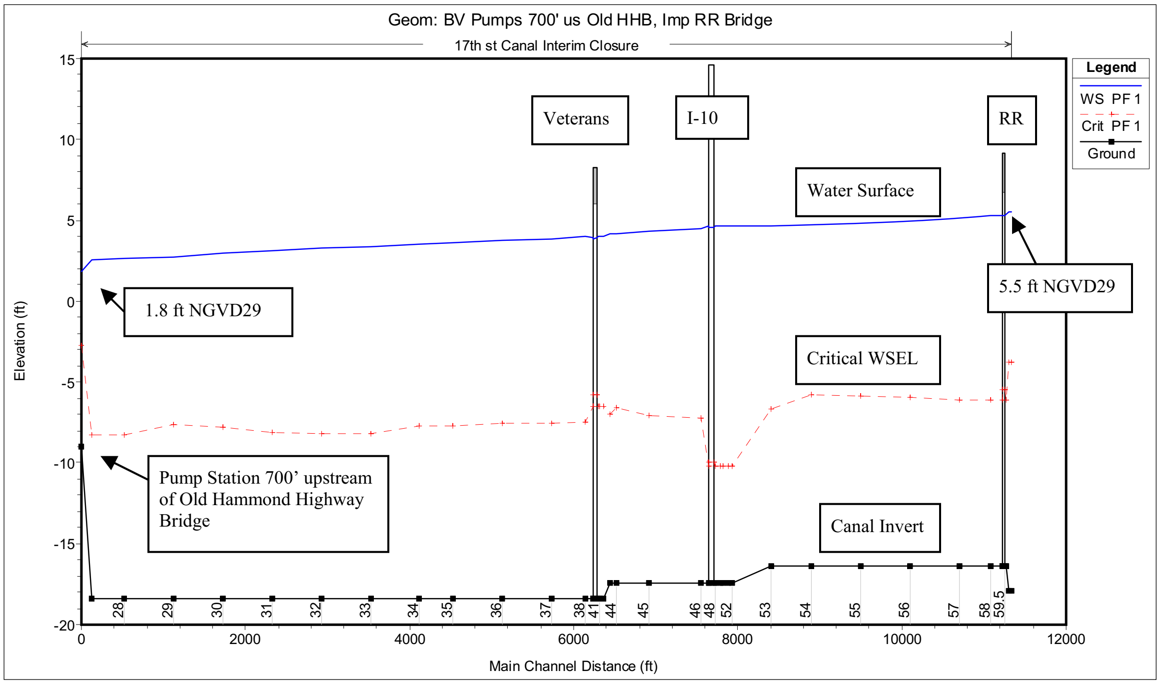


Figure 5.1.2.3-3. Water Surface Profile With Modified Railroad Bridge – 17th Street Canal (Option 1)

Hydraulic Analysis – Gates Open Mode

The Gated Pump Station would be designed to pass canal discharges through the gate openings for combinations of Lake Pontchartrain elevation and canal discharge that do not cause the safe water elevation in the canal to be exceeded. If conditions are expected to occur that would cause the safe water elevation to be exceeded, the gates would be closed and the Gated Pump Station would be operated in pumping mode.

Using the existing canal cross-section geometry and invert profile, as provided in the USACE-developed HEC-RAS model, and with the invert elevation beneath the railroad bridge modified, the canal water surface elevation at DPS 6 was determined for various combinations of Lake Pontchartrain elevation and canal discharge, as shown in Figure 5.1.2.3-4.

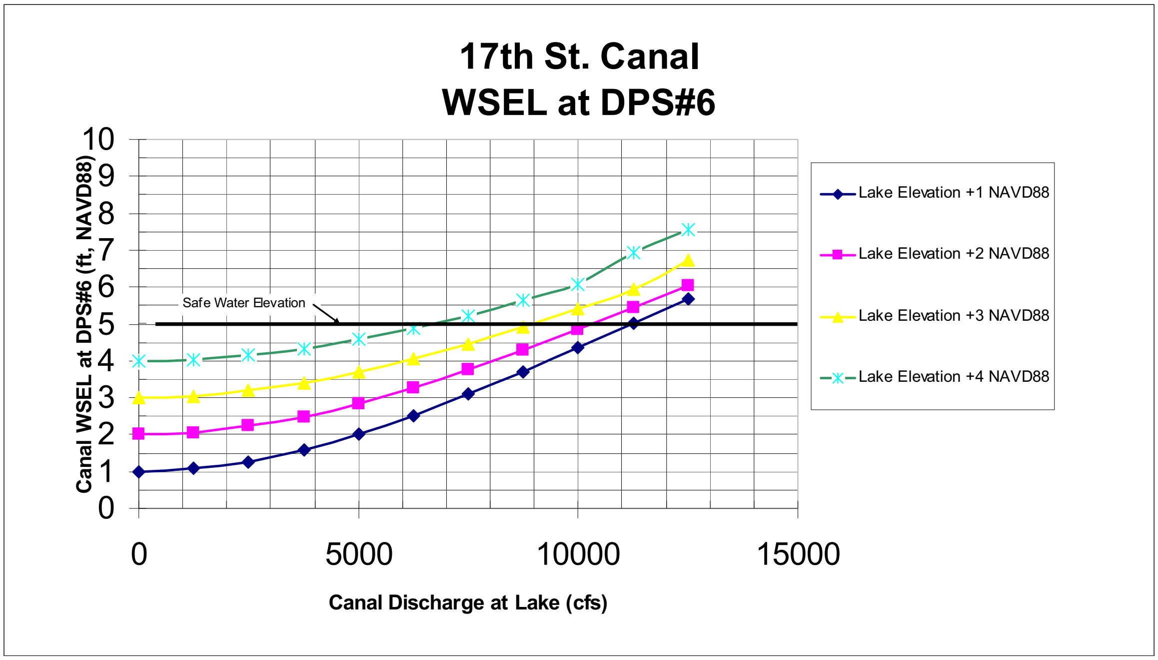


Figure 5.1.2.3-4. Water Surface Elevation at DPS 6 for Various Combinations of Lake Pontchartrain Elevation and Canal Discharge – 17th Street Canal

For all combinations of Lake Pontchartrain elevation and canal discharge that fall below the safe water elevation, operation in the gates-open mode would be possible. For combinations of lake elevation and canal discharge that exceed the indicated safe water elevation, closure of the gates and operation in pumping mode would be required.

5.1.2.4 Geotechnical

The typical stratification for the 17th Street Canal is taken from the IPET Report, Volume V. From the top down the stratification includes; Marsh Clay, a peat layer, Lacustrine Clay, Relic Beach Sand, and Bay Sound Clay. Pleistocene sand and clay strata are below the Bay Sound Clay. The peat is only present in a limited length of the canal. A typical representation of the canal geology is shown in the figure below. The section is taken from the IPET report. Corresponding IPET report figures are included in Appendix B for reference.

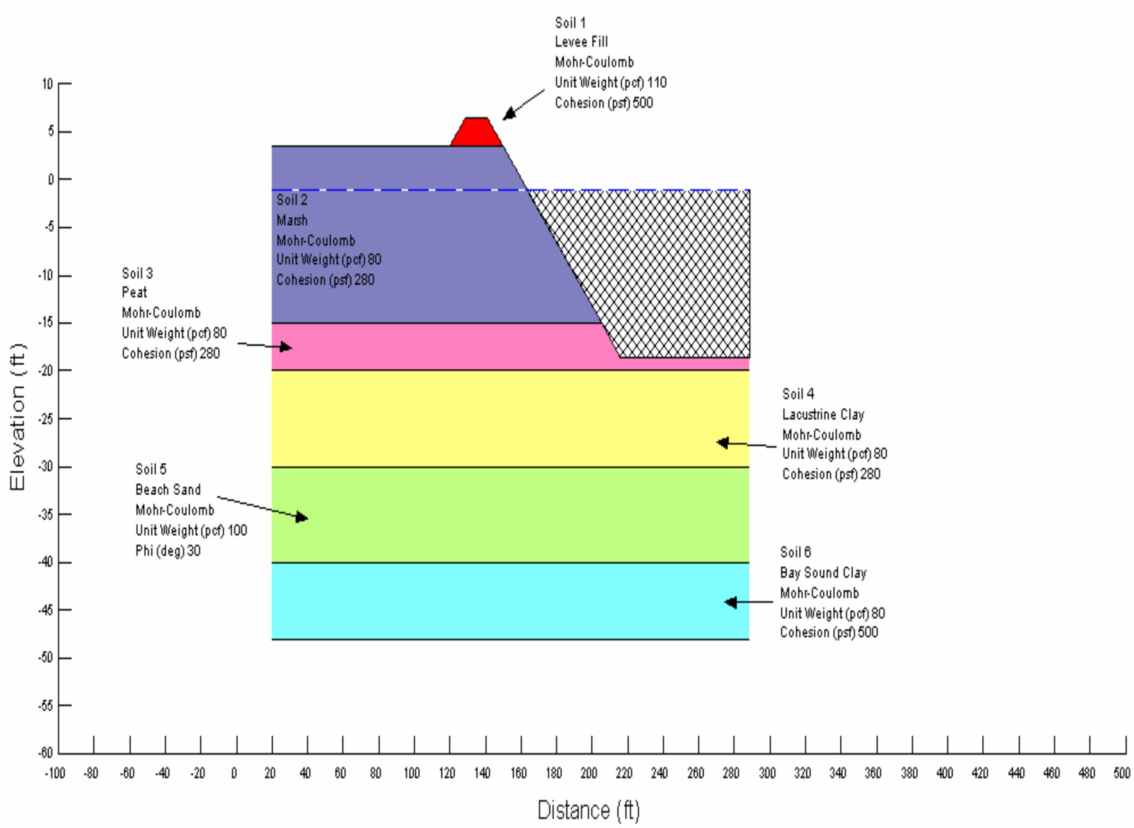


Figure 5.1.2.4-1. 17th Street Geology, Option 1

The strength and physical properties used to characterize these strata are also taken from the IPET Report, Volume V, appendices 1, 3, 4 and 6. The parameters used in these various appendices are consistent and are taken as conservative. The parameters used in this study are shown in the figure above. Tables from the IPET report are reproduced in Appendix B to this report. These tables report strength evaluations performed as part of the IPET Report. In all cases the average strength values for each stratum reported are higher than those shown above. They are provided to demonstrate why the values used in this analysis are considered to be conservative and suitable for this study.

Canal

The canal is stable in its current state and does not require modification for Option 1. Stability analysis has been performed for the current state as a calibration of the model to be used in this study. The results of these calibration analyses (figures B-1, 2, and 3, Appendix B) are essentially the same as similar analyses performed for the IPET. Calibration ensures the channel models developed for this study are reasonable and appropriate. This calibration procedure also demonstrates the validity of the strength parameters listed in Table 3-1 above. The slope stability models developed here are used as the basis for evaluation of Option 2 (discussed later).

Pump Station Foundation

Evaluation of foundation issues for the Option 1 pump station at the 17th Street Canal is under way at the time of this writing. Preliminary stability evaluation of the pump station has been completed and is included as calculation file 4.1 in Appendix B. This stability analysis is a two-dimensional analysis of a typical section cut perpendicular to the long axis of the pump station. This section includes all driving and resisting loads. Analysis results are reported on a per foot basis representing the nominal one foot thickness of a two dimensional analysis. Conservative assumptions are made throughout so the two-dimensional analysis should represent conservative evaluation for this study. The issues identified for evaluation are described in the following paragraphs.

Sliding: The excavation for the intake basin will remove most of the material from the upstream side of the pump station. The pumps being considered for this study will require 14 ft of water to operate properly. The critical sliding case will be when the gates are closed and the downstream side is subjected to a lake surge. Even though the lake surge is relatively short duration, the structure must be stable during the few hours it is present. The net pressure on the pump building is substantial. Several elements of the foundation design will contribute to or have an effect on sliding resistance.

The thickness of the base slab has a direct effect on sliding resistance. Slab thickness is dictated by uplift considerations (see below). The slab will be entirely below grade so it will develop shear resistance in the adjacent soil. At the anticipated depth of the base slab, this resistance will develop in the Lacustrine Clay. The clay has a weak shear strength with s_u = 280 psf. With only four feet of embedment passive resistance alone is not expected to be adequate. Deep soil mixing is being used at the temporary protection structure at this canal to substantially improve the foundation soils and may be considered for application at this pump station as well.

The weight of the structure will generate friction on the base of the slab. Because of the high uplift pressures, net weight will be relatively low. This means base friction will not provide much resistance for sliding. Base friction can be improved somewhat by soil modification but will never be the controlling factor unless weight is added to the structure explicitly for this purpose. The addition of weight to improve base shear is not efficient and will not be considered for this study.

Preliminary calculations demonstrate a deficiency in sliding resistance of approximately 23 kip/ft. This deficiency must be corrected to achieve a factor of safety of 1.5 for sliding. Design elements will have to be added to provide this additional resistance.

The greatest influence on sliding resistance will likely come from the foundation piling. One method of providing required lateral resistance can be including lateral resistance into the design of vertical piles. Another viable method of providing the resistance is the addition of battered piles. Battered piles are more efficient lateral resistance elements and will likely be used. In any case, soil modification can improve this resistance and will be considered as a possible supplemental measure.

Uplift: When the gates are closed and the structure is subjected to lake surge, substantial uplift will develop on the base of the structure. The uplift is assumed to instantaneously reflect the lake surge pressures. Hydraulic conductivity of the Lacustrine Clay is somewhat low but the Beach Sand below the clay must be assumed to be hydraulically connected to the lake. Uplift pressure is calculated by assuming a linear variation from the lake head to the upstream head in the canal. If needed, this can be modified by installing a cut-off wall below the foundation. Cut-off walls can be effective but will be difficult to coordinate with a pile foundation anticipated for this application. For this study it will be assumed that a cut-off wall will be not be used to limit uplift.

Conservative design methodology resists uplift forces with dead load. At this stage of the study the base slab is being sized to provide the necessary dead load to resist uplift with a factor of safety of 1.1. This requires a base slab of 4 ft thickness with bottom at elevation -17 ft (NAVD). It is recognized that the vertical piling supporting the structure will also provide uplift resistance. For this study the piling will be considered to provide supplemental resistance to raise the factor of safety above 1.5, but will not provide the principal resistance system. This is a conservative approach that ensures long term stability against uplift failures.

Underseepage: Underseepage is a potential failure mode with the combination of high heads and weak foundation soils. Since the base slab is only 4 ft thick, the length of the flow path is short and the threat of underseepage problems increases. Underseepage can result in the loss of the foundation material through piping beneath the foundation. Seepage calculations will be performed to check this failure mode. It is likely that a cut-off wall will be required.

***Foundation Support*:** Preliminary calculations of overturning for the pump station indicate the structure requires additional tension elements with capacity of 70 kip/ft to have the base be 100 percent in compression. The 70 kip/ft capacity is based on being able achieve a centroid of the tension elements at 2/3 the width of the base slab. The needed tension capacity will be provided by piling.

The principle vertical resistance system for the pump station will likely be piling. This is common practice for the area when foundation soils are too weak for the structural loading. The strength of the soils can be improved by soil modification techniques and this will be considered. However the big issue for this structure will be settlement. Ground modification would have to extend to greater depths if it is desired to reduce the need for piling. For simplicity the structure will be founded on piling for this study. The concept design of the piling will ensure it will resist static vertical load when the gates are open and the eccentric loading produced by unbalanced water pressure when the gates are closed.

5.1.2.5 Structural

The concept design has just recently reached the point where the structures can be sized. No work has been done to develop a conceptual structural system. The interior of the structures will be clear spans to accommodate bridge cranes for maintenance. Similarly, the height of the structure will be dictated by maintenance requirements. The overall framing material will likely be concrete to reduce maintenance considerations. Space will be provided to ensure the control system and the safe room stays above the highest anticipated water levels.

The cladding for the structure will likely be an item of public concern. The location of the pump structures at the lake front will make them highly visible. Development of an architectural scheme which will be palatable to the public is not within the scope of this concept level study. However it is recognized this will become a project requirement. Therefore, when costs are assigned to the structure, additional costs will be added for architectural cladding.

The structural framing system is relatively straight forward except at the foundation level. The difficulties of the foundation are discussed in the Geotechnical sections of this report.

5.1.2.6 Mechanical

The function of the pumping station is to lift water from the canals to the lake. The principle piece of machinery to do this is the pump. High capacity, low head pumps are essentially large propellers in a tube. For this application there are two principle types of pumps defined by the orientation of the propeller. The propeller can be installed horizontally or vertical. The horizontal types are similar to the Woods Screw Pumps which are extensively used in the older pumping stations.

The horizontal pumps are installed horizontally on an operating floor above the maximum canal level. As such, the propeller is above the water surface. To operate the pump, a vacuum is used to extract air out of the pump and pump discharge piping until the propeller is submerged. Once the propeller is submerged, the pump can be turned on and the pump will complete the filling of the discharge pipeline and establishing a siphon discharge. The major advantage of the horizontal pumps is that the pump bearings and propeller are located above the canal and the easily accessible for maintenance. The pump can actually be started before the propeller becomes fully submerged permitting a low startup torque which minimizes engine generator sizing. The major disadvantage is that the pumps need to be primed by a vacuum system. Due to the volume of air needed to be evacuated, it can take 10 to 15 minutes to get the pump started.

The vertical pump has the propeller mounted down below the minimum canal water surface level. Like the horizontal screw pump vertical pumps are also used extensively in the Parish. As a result the pump design, the pump is self-priming and can start pumping within seconds of a start command which is a significant advantage in controlling pumping units when pumping stations are located in series. Also with this design, the motors are located on top of the pump and out of any danger of being damaged by flooding. The major disadvantage is that the propeller is below the water surface and that any major maintenance requires fully disassembling the pump. Also a disadvantage is that the pump starts under load and has a high startup toque which can require over sizing engine generators.

Either type pump is applicable to this pumping station. For simplicity of this analysis, only vertical pumps which provide the maximum flood protection with the elevated motors are considered in the station. During detailed design the use of vertical, horizontal, or a combination of both should be considered.

For reverse flow protection, the discharge pipe from the pumps is elevated such that the invert of the pipe at the highest point is at or above the floodwall elevation so that reverse flow through the pump is not likely. The discharge pipe is then brought down below the minimum lake level forming a siphon. A siphon discharge permits recovering the energy so that when normally pumping the pumps only see the difference between the canal elevation and the lake. A vacuum breaker is provided at the highest point in the discharge pipe to permit breaking the siphon when the pump stops. For added protection, sluice gates can be added to the discharge pipe for protection against reverse flow. To minimize submergence and hydraulic losses through the station, a formed suction inlet was used in the analysis. A typical cross-section of the pumping station is attached to Appendix D – Mechanical.

The pumping units can be driven by either electric motor with electric generator backup or direct driven by engines. In the final design, there may be a combination of drivers in the pumping station. Direct driven engines are cheaper since they eliminate the engine generator and motor. Motor driven pumps are quieter and more efficient. A determining factor may well be the ability or willingness of the power company to build power lines and reserve generating capacity for pumping units which may only occasionally be operated. A detailed study should be done during design to determine the optimum combination of electric driven motors with engine generator backup or direct engine driven pumps. For the purpose of this analysis approximately 60 percent motor driven pumps and 40 percent engine driven pumps are assumed.

A polling of pumping manufacturers indicated that the maximum practical size of pumping units is roughly 1,000 cfs. This is limited by the physical size of the equipment and the ability to move the equipment along major roadways. 1,000 cfs also matches up with the largest pumping stations in the major feeder pumping stations so was chosen as the main pumps in the new pumping stations. The existing pumping stations also have a number of smaller pumps. Smaller pumps permit pumping lower flows without having frequent starts and stops and also provided the ability to match flows when pumping stations operate in series. Therefore combinations of 1,000, 500, and 250 cfs pumps were selected at as the primary capacities. Using only three sizes will permit the sharing of parts between the pumping stations.

For Option 1, the screens are optional as there is no un-screened inflow to the canal downstream of the major pumping stations however screens ahead of the pumping station inlets will smooth the inflow to the pump and have a hydraulic function. Based on the length of canal and the potential for additional debris to enter canal, a trash rack system is included in the costing.

There are a number of additional mechanical systems required for operation of the pumping station. All major pumps require a clean source of water for bearing lubrication. This can be from the water system. However, based on experience during Katrina in which the water system failed, a secondary source of water should be provided. There are two sources available, either canal water or well water. Because the water to the pumps needs to be of high quality, well water is being considered for the pumping station. Canal water can be used but requires a high level of treatment to remove abrasives. Because of the size of the equipment in the facility, the facility should include overhead crane, lay down space, truck loading access, and workshop areas. Whether motor driven or engine pumps are used, there will be significant opening for ventilations. All inlet air vents should be shrouded to inhibit the entry of wind blown water.

Engine driven pumps have additional mechanical considerations including engine and gear reducer cooling systems, fuel system, starting air systems, lubricating oil and waste oil systems. In addition, engines require both exhaust air and combustion air systems.

Pumping station hydraulics are critical to successful operation of the pumping station to achieve maximum hydraulic performance. A physical model test of the pumping station including canal entrance, screens, and pump inlet, and discharge siphon pipe must be conducted as a follow-on effort to ensure correct sizing and configuration.

For the purpose of this study, the following combination of vertical pumping unit capacities was chosen. In addition, a combination of direct drive diesel engines and diesel generators are assumed to provide an approximate 60 percent electric motor drive and 40 percent diesel engine drive ratio.

Pump Driver Type		
Pump Number	**Pump Capacity cfs**	**Driver Type**
1	1000	Direct Drive Engine
2	1000	Direct Drive Engine
3	1000	Direct Drive Engine
4	1000	Direct Drive Engine
5	1000	Motor on Generator
6	1000	Motor on Generator
7	1000	Motor on Generator
8	1000	Motor on Generator
9	1000	Motor on Generator
10	1000	Motor on Generator
11	1000	Motor on Power Grid or Generator
12	500	Motor on Power Grid or Generator
13	500	Motor on Power Grid or Generator
14	250	Motor on Power Grid or Generator
15	250	Motor on Power Grid or Generator

Fuel storage capacity for the pump station was selected at four days of full pumping capacity. Based on this duration and usage rate, the anticipated fuel storage required is slightly over 80,000 gallons. Assuming standard 12,000 gallon double wall fuel storage tanks, a minimum of seven tanks will be required.

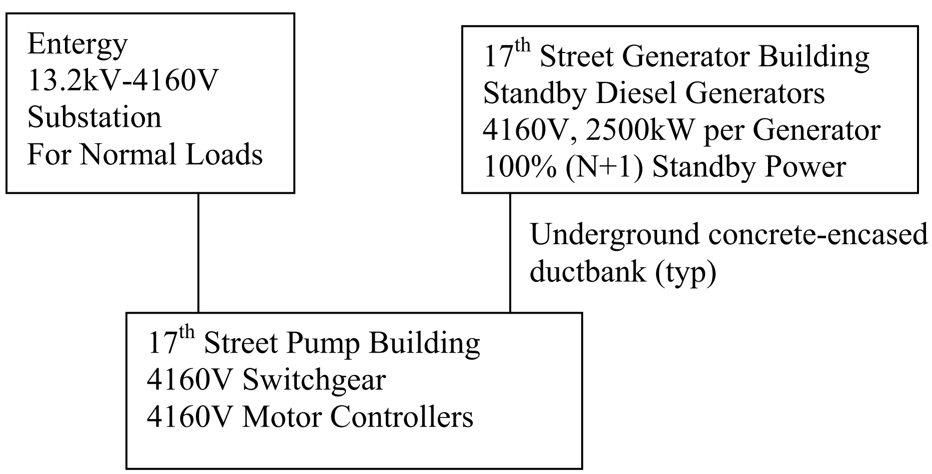
The estimated pump ratings are as follows:

Pump Rating			
Capacity, cfs	1,000	500	250
Bowl Head, ft	13	13	13
Pump Speed	162	227	321
Engine Rating, bhp	2100	---	---
Motor rating, hp	1750	900	500

In the Mechanical Appendix D are representative pump performance curves submitted by the manufacturers. The curves are presented as typical curves only as the required pump rating have evolved during the study and the pump ratings on the curves will differ slightly from the latest hydraulic requirements.

5.1.2.7 Electrical

Option 1 uses existing pump station DPS6 along with a new pump station at 17th Street. Utility power, from Entergy, will only be supplied for the normal pump loads (see the Pump Driver Schedule below for pump utilization). The incoming utility service will not be sized to accommodate the storm event pump loads.



One hundred percent of the storm event electric-driven pump loads will be supplied from local standby generators dedicated to the 17th Street Pump Station. The standby diesel generators will utilize an N+1 design such that if a generator goes off line or one is down for maintenance, the full pump station load will still be supplied by standby power. The utility service, standby generators, and pump motors will all operate at 4160 volts. All electrical distribution circuits will be routed underground in concrete-encased ductbank. Below is a table to show the pump driver schedule.

Option 1 - 17th Street Canal – Pump Driver Schedule						
Pump cfs	**Driver bhp**	**Motor hp Nameplate**	**Motor hp Load**	**Source**	**Utilization**	
					Grid	**Stdby**
1000	1241			Engine		1%
1000	1241			Engine		1%
1000	1241			Engine		1%
1000	1241			Engine		1%
1000	1052	2750	1800	Standby Generator		1%
1000	1052	2750	1800	Standby Generator		1%
1000	1052	2750	1800	Standby Generator		1%
1000	1052	2750	1800	Standby Generator		1%
1000	1052	2750	1800	Standby Generator		1%
1000	1052	2750	1800	Standby Generator		1%
1000	1052	2750	1800	Grid/Generator	1%	1%
500	526	1500	900	Grid/Generator	5%	1%
500	526	1500	900	Grid/Generator	10%	1%
250	263	750	450	Grid/Generator	50%	1%
250	263	750	450	Grid/Generator	100%	1%
12500			**15300**	**Totals**		

Based on the pump driver information above, the next table below indicates the electrical equipment sizes used for this option. See the appendices for all cost information.

Option 1 – 17th Street Canal Pump Station - Electrical	
Electrical Equipment	**Quantity/Capacity**
Utility Substation	1 – 5000kVA
Generators	9 – 2500kW
Generator Bldg Size	202’ x 80’
Pump Bldg Switchgear	2 – 2500A
Total Pump Station Load	16.1 MVA, 2231A
Pump Station Load on Utility	5.3 MVA, 731A

5.1.2.8 Environmental

<u>Environmental Site Assessment (ESA)</u>

An environmental database search report of the area within a 0.25-mile radius of the canal route from the existing pump station to the mouth of the canal (i.e., 0.5 mile total width of the search area across the canal) was obtained to identify sites, incidents, conditions, etc. within the search corridor that may contain or formerly contained hazardous, toxic or radioactive waste (HTRW). These reports were utilized during in field observations to verify sites within the potential construction zones. Figure 5.1.2.8-1 presents the mapped sites within the 0.25-mile radius around the 17th Street Canal that are tracked on various environmental databases maintained by either the U.S. Environmental Protection Agency (USEPA) or the Louisiana Department of Environmental Quality (LDEQ).

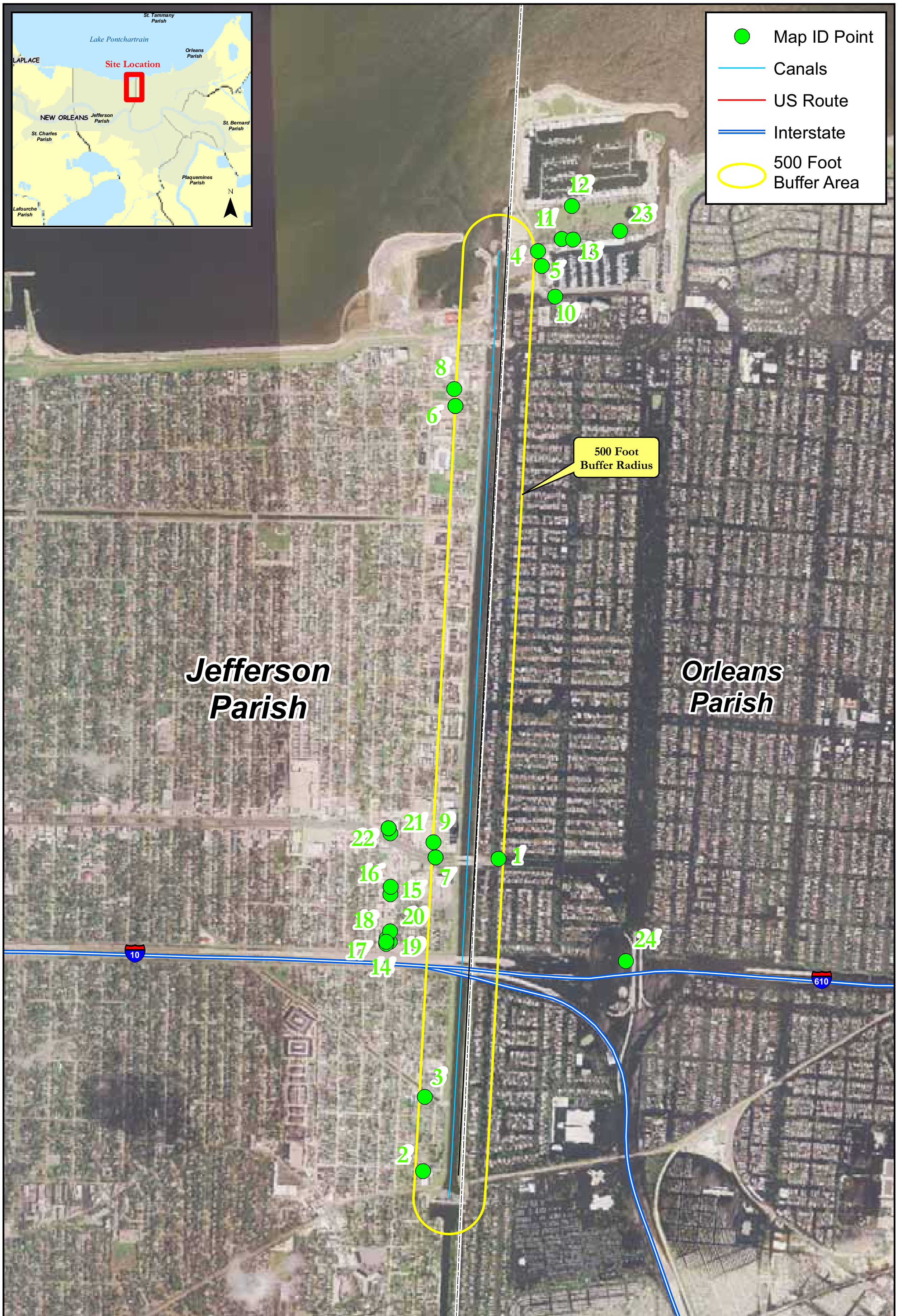
Although the database search was requested for the standard radius of 0.25 mile (the search radius specified under American Society for Testing and Materials (ASTM) Standard E1527-05), particular interest was given the 500-foot (ft.) radius around the canal as a buffer zone for construction activities to take place in connection with the proposed project.

Figure 5.1.2.8-1 provides the mapped 500-ft. radius around the 17th Street Canal. Located within the 500-ft. radius are three sites, map ID numbers 1, 2, and 3, and four sites that appear to be right on or near the 500-ft radius line, map ID numbers 6, 7, 8 and 9. These sites are identified below along with their measured distance from the centerline of the canal:

Map ID	Site Name	Address	Distance
1	Bell South	500 Veterans Boulevard	317 ft
2	EIU of LA, Inc.	383 Lake Avenue	422 ft
3	Labiche Plumbing	200 Canal Street	475 ft
6	Time Saver Stores	200 Live Oak Street	580 ft
7	Tenneco Oil Co.	205 Veterans Boulevard	580 ft
8	H.H. Philibert, M.D.	213 Live Oak Street	580 ft
9	Saluga Chiropractic	401 Veterans Boulevard	634 ft

Site numbers 6 and 7 are on the database for containing leaking underground storage tanks (the LUST database). The Time Saver Store at 200 Live Oak St. had incidents in 1989 and 1991, both of which appear to have been minor and quickly resolved. Information on the Tenneco Oil Co. site at 205 Veterans Blvd. is very limited with only two dates in 1989 given as dates of incidents. No other description was available. Given the age of the reports and distances from the canal centerline for both of these sites, it is determined that neither represent a significant concern for the construction of the proposed project. Similarly, all other sites identified within the 500-ft. radius were evaluated for potential to impose constraints on the proposed project. Based upon available information, no such sites have been identified for the 17th Street Canal.

The database search for the corridor around the 17th Street Canal also reported seven sites that were unmappable due to lacking database items. However, these sites were evaluated for their potential to pose a threat to the proposed project. In consideration of the database on which they are listed, type of facility, and whatever location information is given, none of these seven unmappable sites are considered a significant concern for the construction of the proposed project.



17th Street Canal Detail
Map ID Points
Jefferson/Orleans Parish, Louisiana

Note: Background image is post-Katrina USACE Aerial Photograph (September, 2005). Map ID Points provided by Banks Information Solutions, Inc. Other data not derived from registered survey and should be considered approximate.

0 500 1,000 2,000 Feet

1:15,000

GEC

Figure 5.1.2.8-1

Project # 60321CZ02 July 2006

A field site reconnaissance was conducted to determine whether any sites not identified through the database search process existed within the immediate project area that could potentially affect project design plans. The area surrounding the 17th Street Canal is almost entirely residential with a few small exceptions near the crossing of Veterans Blvd. and the area around the mouth of the canal at Lake Pontchartrain. No visible signs were noted that would indicate the presence of HTRW in quantities that would warrant additional investigation.

The addition of diesel powered pumps at the control structures near the mouth of the canal would constitute new sources of air emissions and noise for area residents. The air emissions would require permitting through the LDEQ and noise mitigation would likely be required to be incorporated into the design plans for the facility. These issues would be addressed following final selection of pump and power design.

Because there is very limited recreational use of the 17th Street Canal, for the most part recreational constraints would not be an issue. There is some recreational fishing at or near the mouth of the canal and this may be, at least temporarily, disrupted during the construction of the control structure, but the numbers of fishermen appear to be small and intermittent.

The entire area of New Orleans and Lake Pontchartrain is within the Louisiana Coastal Zone, therefore a Coastal Zone Consistency determination will have to be conducted and submitted to the Louisiana Department of Natural Resources for concurrence.

Sediments

A Certified Industrial Hygienist (CIH) Investigation into the quality of the sediments of the 17th Street Canal was conducted by GEC, with assistance from Professional Technical Services, Inc. (ProTech), under contract to the U.S. Army Corps of Engineers, New Orleans District, in February 2006. This investigation was conducted on all three canals that are the subject of this report near the mouths of the canals at the alternative locations under consideration for the interim control structures that are currently under construction. The following are excerpts of portions of that report that summarize the activities and findings on sediment quality near the mouths of the canals.

> Introduction: A field investigation team consisting of GEC and ProTech personnel under the guidance of USACE personnel and a CIH conducted the sampling on February 13 through February 16, 2006. GEC and ProTech staff collected, composited, and delivered sediment samples from the project area to the laboratory for analysis. GEC staff evaluated analytical results in order to determine whether the sampled material is contaminated with metals, TPH, volatile and semi-volatile organics (including PAHs), pesticides, and/or dioxins.
>
> Organization and responsibilities of the project team are contained in a Site Safety and Health Plan prepared by GEC for this investigation and in accordance with USACE regulations governing Hazardous, Toxic, and Radioactive Waste (HTRW) and CIH investigations

Methodology: GEC staff and vibracore operators from Professional Technical Services, Inc. (ProTech) mobilized to the project area on February 13, 2006. Sampling efforts began on February 13, 2006, at 1100 hours at Orleans Avenue Canal and continued daily through February 16, 2006, at 1700 hours. Sample locations were dictated in the Scope of Work, and selected in the field by GEC personnel in consultation with the project CIH and the USACE-NOD representative. Sediment samples were collected from the bottom of each canal with a backpack vibracore unit to a depth of approximately five feet below the surface sediments in three-inch aluminum barrels. Three samples were collected from each location: one near the edge of each bank and one from the center of the canal. The three samples from each location were consolidated into one composite sample for laboratory analysis. See Figure 5.1.2.8-2 for the locations of samples taken in the 17th Street Canal.

Laboratory results were evaluated in accordance with standards established by the Louisiana Department of Environmental Quality (LDEQ) Risk Evaluation/Corrective Action Program (RECAP), approved October 20, 2003, and the Resource Conservation and Recovery Act (RCRA).

General Findings (all canals): Several of the canal sediment samples exhibited properties that suppressed detection of volatile organic compounds. This interference required a 1:10 dilution of some samples in order to determine internal laboratory standard compliance. The dilution resulted in an elevated detection limit. Matrix spike and matrix spike duplicate analyses were performed on canal sediments in order to confirm the site-specific matrix interference.

Acetone, carbon disulfide, methyl ethyl ketone (2-butanone), methylene chloride, trichloroethene and/or tetrachloroethene were noted below RECAP screening standards in some sediment samples, as well as in the trip blanks and in the method blanks utilized by the laboratory for QA/QC purposes. The above contaminants are highly volatile and commonly present in the extraction area of the laboratory. The detection of these contaminants in similar concentrations in the blank samples as well as the sediment samples is attributed to laboratory contamination during the extraction process and not an indication of the presence of these contaminants in the canal sediments.

Concentrations of trichloroethene in excess of RECAP standards are also noted in some of the volatile organics analyses. Trichloroethene, as well as the above-mentioned contaminants, are components of TPH-Gasoline Range Organics (GRO); therefore elevated TCLP TPH-GRO results were also reported. The presence of these compounds in excess of RECAP is attributed to the previously discussed laboratory contamination, and not reflective of conditions in the canals.

Blank contamination of octachlorodibenzo-p-dioxin (OCDD), 1,2,3,4,6,7,8-heptachlorodibenzo-p-dioxin (1,2,3,4,6,7,8-HpCDD), and total heptachlorodibenzo-p-dioxin (HpCDD) was noted in the some dioxin samples.

17th Street Canal
CIH Investigation
Jefferson/Orleans Parish, Louisiana

Note: Post-Katirna Color Aerial Photography provided by 3001, Inc. through the USACE. Other data not derived from registered survey and should be considered approximate.

Figure 5.1.2.8-2

Project # 60321CZ02
July 2006

Polycyanodifluoroamino-ethyleneoxide (PCDE) interference was noted in some dioxin samples resulting in a concentration flagged with “E," or estimated maximum possible concentration. Other dioxin interferences are noted with an “I” flag, and are addressed in the Discussion section of the laboratory reports.

<u>17th Street Canal Specific Findings:</u> Sample 17th one exceeds RECAP standards for benzo(a)anthracene by 0.69 mg/kg (111 percent), benzo(a)pyrene by 1.05 mg/kg (318 percent), benzo(b)fluoranthene by 1.23 mg/kg (198 percent), and indeno(1,2,3,c,d)pyrene by 0.175 mg/kg (28 percent). Concentrations of TPH-DRO and TPH-ORO exceed standards by 100 mg/kg (154 percent) and 374 mg/kg (208 percent) respectively. Lead concentrations exceeded standards by 21 mg/kg (21 percent).

Sample 17th two exceeds standards for benzo(a)pyrene by 0.055 mg/kg (17 percent), TPH DRO by 143 (220 percent) and TPH-ORO by 377 mg/kg (209 percent). Sample 17th three exceeds standards for benzo(a)anthracene by 0.269 mg/kg (43 percent), benzo(a)pyrene by 0.425 mg/kg (129 percent), and benzo(b)fluoranthene by 0.36 mg/kg (58 percent). TPH-DRO exceeds standards by 64 mg/kg (98 percent) and TPH-ORO exceeds standards by 156 mg/kg (87 percent).

Volatile organic blank contamination was noted in the trip blank for the 17th Street Canal. Falsely elevated levels of trichloroethene and acetone were noted in the volatile organics analysis for sample 17the three samples.

None of the analyzed compounds that are regulated by RCRA are present in the 17th Street Canal TCLP samples in concentrations exceeding RCRA standards. TPH-DRO was detected in the TCLP leachate in the 17th 2 and 17th three samples. TPH-GRO was also detected in all three samples, possibly due to volatile organic blank contamination.

<u>Conclusions:</u> Based on site reconnaissance, laboratory analysis, and best engineering judgment, it is GEC’s professional opinion that the material sampled from Orleans Avenue, London Avenue, and 17th Street canals contains PAHs, lead, and total petroleum hydrocarbons in concentrations that are potentially hazardous to human health or the environment.

Dioxins, while not present in concentrations exceeding standards set by the State of Louisiana, are present in the sediments at levels that may preclude certain disposal options. GEC recommends further evaluation of the sediment material analysis prior to consideration of ocean dumping or use of the material as borrow or fill. GEC further recommends that prior to landfill disposal, the analysis of the sediment be evaluated in order to ensure its disposal in a landfill permitted to dispose of such material.

Personnel handling the sediment material should be outfitted in modified Level D personal protective equipment, including oil-resistant gloves and safety glasses. Special actions associated with state environmental regulations regarding the handling, storage, disposal or ownership of contaminated sediments (as described in LAC 33:V) may be required.

For additional information on the sediment sampling during the CIH investigation, please see Final Report, Orleans Avenue, London Avenue and 17th Street Outfall Canals, Certified Industrial Hygienist Investigation, Orleans Parish, Louisiana, March 2006, U.S. Army Corps of Engineers, New Orleans District, New Orleans, Louisiana.

In general, further study is recommended regarding sediment quality, sediment transport during construction of the project, and its potential effects on aquatic and marine species prior to construction of the proposed project.

Wetlands

National Wetland Inventory (NWI) maps were consulted for jurisdictional wetlands in the proposed project area. See Figure 5.1.2.8-3 for a presentation of the mapped jurisdictional wetlands in all three canals. The 17th Street Canal has mapped wetlands potentially within the potential construction zone, along the west bank of that canal from approximately Veterans Boulevard north to the mouth of the canal. Changes in tidal influence would not likely significantly affect the wetlands along the west bank of the 17th Street Canal as there does not presently appear to be any tidal influence on these wetland areas. Permits from the USACE would be required prior to disturbance of these areas.

Protected Species

Although the Scope of Work for this study requested the identification of endangered species in the Lake Pontchartrain vicinity and suggested mitigation of impacts, we have since received a request from the USACE Hurricane Protection Office (HPO) Environmental Team that all communication with the wildlife agencies happen through the Environmental team, not our company. Thus, we were not able to obtain a list of threatened and endangered species from the agencies, and therefore are not able to suggest possible mitigation. It is our understanding that The HPO Environmental Team is in the process of informally consulting with the agencies to obtain species information, and they will formulate a mitigation plan, if necessary.

Cultural Resources

A coordination letter requesting comment on the proposed project has been submitted to the Louisiana State Historic Preservation Officer (SHPO). However, at the time of preparation of this Draft report, no response has been received.

National Wetlands Inventory Data
Orleans, London, & 17th Street Canals
Jefferson/Orleans Parish, Louisiana

Note: Backgorund image is Post-Katrina USACE Aerial Photography (September, 2005). Wetlands data obtained from National Wetlands Inventory and downloaded from www.atlas.lsu.edu. Other data not derived from registered survey and should be considered approximate.

0 1,500 3,000 6,000 Feet

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Canals
US Route
Interstate
Parish Boundary

Figure 5.1.2.8-3

Project # 60321CZ02
July 2006

5.1.2.9 Constructability

The conceptual designs used throughout this project are constructible using conventional techniques. For Option 1 there are no channel modifications. All construction is focused on the pump station. The temporary gate and structure would be demolished after the new pump station becomes operational. Two major approaches can be taken to construct the new pump station in the 17th Street canal.

The first construction concept is based on construction of a sheet pile cofferdam enclosing the pump station. The cofferdam enables the contractor to construct the entire pump station in the dry. To accomplish this, a bypass channel must be constructed for alternate locations A and C. Location Alternate B is constructed entirely outside the existing canal and does not require a bypass channel.

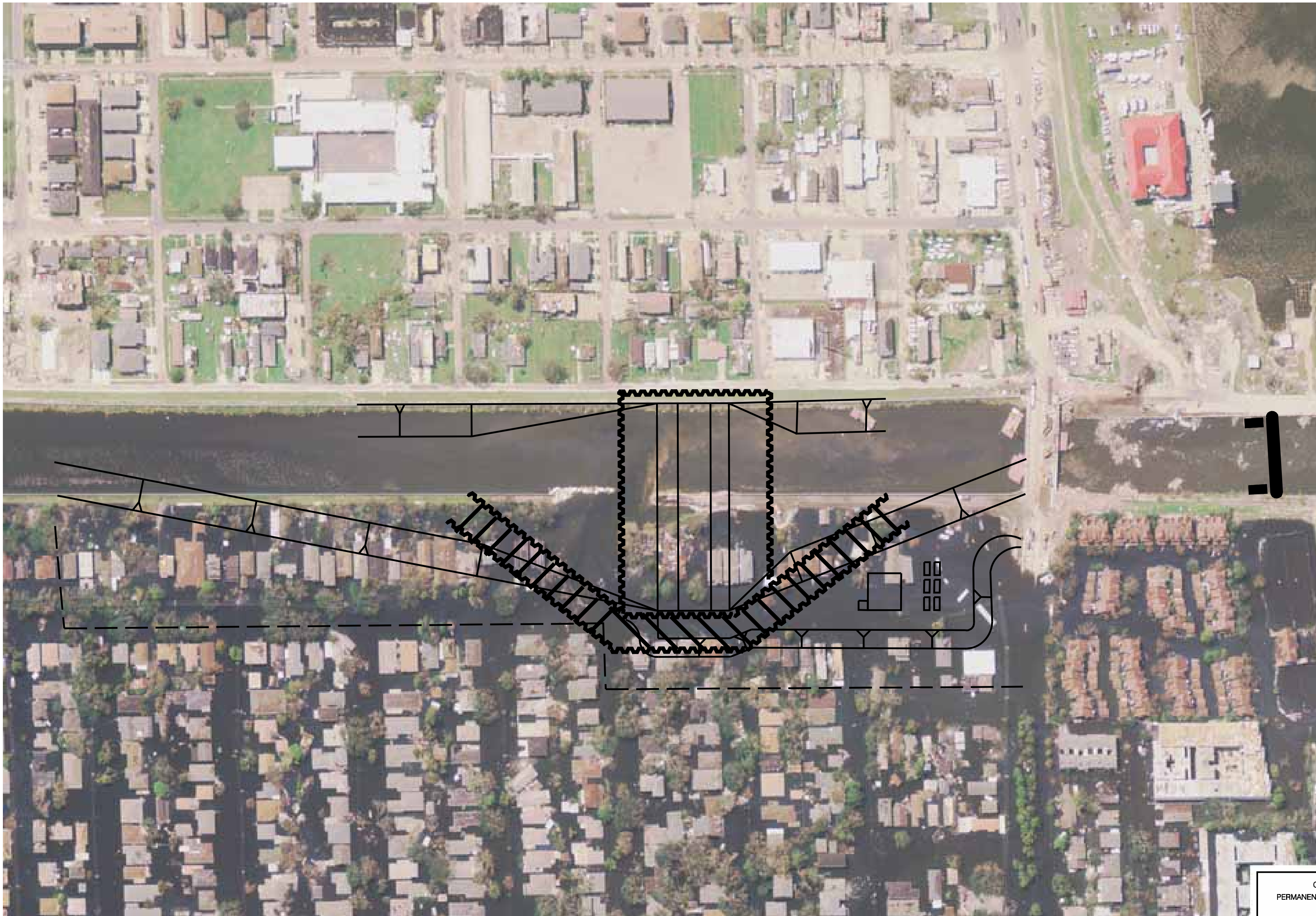
The bypass channel for Alternate A is formed with parallel sheet pile walls and is sized to pass the flows for which the temporary gate/pump structure is designed. The bypass channel for Alternate C is built by breaching the narrow strip of land between the canal and the coast guard bay. Only the north side of the breach would be formed with sheet pile. The south side would be laid back at a stable slope. Figures 5.1.2.9-1, 5.1.2.9-2 and 5.1.2.9-3 on the following pages present the cofferdam and bypass concepts.

Once the pump station is complete the cofferdam is removed and the bypass channel is dammed and filled. Construction of the concrete liner walls for the intake and discharge channel transitions can be constructed in the wet at the same time the pump station is being built.

As can be seen on the figure there are some problems with this concept for location Alternate A. At this site the best location for the bypass channel places it between the power station and the new pump station. The contractor would have to provide a temporary bridge to be able to access the pump station area. The by pass channel also interferes with significant portions of the channel transitions forcing that part of the construction to be delayed until the bypass channel can be filled. The cost of constructing the bypass channel is significant for this site.

The second concept for location alternates A and C is to construct the pump station in two parts. The substructure for the east half would be constructed first and would house the gates. A cofferdam would be set around just the east half, allowing the existing canal to pass flow with a small amount of constriction imposed by the cofferdam. The gates would be set in the open position. Upon completion of the gate substructure, the cofferdam would be moved to the west half of the structure. Flow would be redirected to pass through the new gates. The west half of the substructure could then be constructed followed by the complete super structure.

The second concept also has some disadvantages. The construction of the pump station in two halves is much more complicated than building it all as a single unit. The design would have to account for the connection of the two halves. Coordination and sequencing would



LEGEND

SHEET PILE:

BY PASS:

100' 50' 0 100'

APPROXIMATE SCALE: 1"= 100'-0"

CONCEPTUAL DESIGN STUDY
PERMANENT FLOOD GATES AND PUMP STATION
17th STREET – OPTION 1A
CONSTRUCTION CONCEPT
FIGURE 5.1.2.9-1

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CONCEPTUAL DESIGN STUDY
PERMANENT FLOOD GATES AND PUMP STATION
17th STREET – OPTION 1B
CONSTRUCTION CONCEPT
FIGURE 5.1.2.9-2

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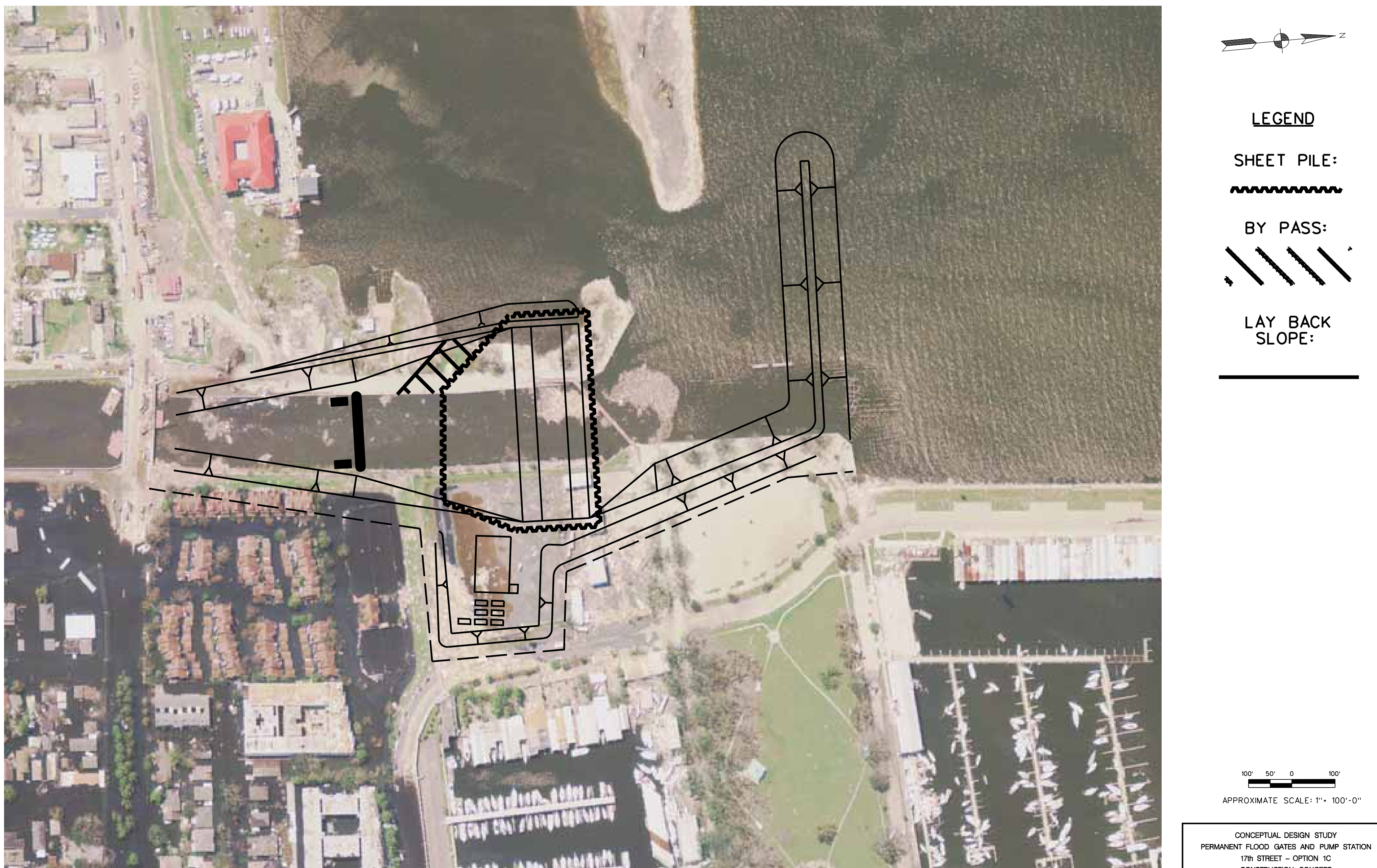

CONCEPTUAL DESIGN STUDY
PERMANENT FLOOD GATES AND PUMP STATION
17th STREET – OPTION 1C
CONSTRUCTION CONCEPT
FIGURE 5.1.2.9-3

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be significant issues. The cost of constructing the pump station in two halves would be higher than being able to construct it in as a unified whole

The bypass concept is illustrated here and used as a basis for cost estimating to be consistent with the other eight canal options. The additional cost of the bypass canal may be more expensive than the incremental cost for constructing the pump station in two halves. The design build contractor would be permitted chose either of these concepts or innovate a concept of his own if it would reduce project cost.

Location Alternate C also requires the construction of a breakwater. The breakwater could be constructed at any time but must be complete before putting the new pump station in operation. It would be best to complete the breakwater early in the construction because of the additional protection it provides.

5.2 Option 2 – 17th Street Canal

5.2.1 Alternative Approaches

Three location alternatives were considered for Option 2 on the 17th Street canal. The merits of each were evaluated and discussed in detail in the Civil/Site section. For the purposes of this study, Alternative C was chosen as the location to base costing and other engineering considerations.

5.2.2 Engineering Consideration

5.2.2.1 Civil/Site

The 17th Street Pump Station for Option 2 is anticipated to be 400 feet long by 165 feet wide. The total width includes a 45 foot inlet works including trash screens, an 80 foot pump station building housing pumps and motors, and a 40 foot outlet works. Finish grade for the Generator and Tank Farm Complex is always approximately +16.0 elevation.

- The **Alternative A Pump Station Layout** for Option 2 is as shown in Exhibit 5.2.1.A, Appendix C.

<u>General Location and Description</u> - The horizontal location of the pump station is identical to Option 1 for Alternative A. Changes from Option 1 to this Option 2 layout all result from the effects of the deeper canal and correspondingly deeper pump station inlet elevation, as described below.

<u>Right-of-Way Acquisition</u> - Permanent right-of-way acquisition remains almost exclusively on the east bank of this proposed site and increase slightly due to larger facilities. Minor temporary construction easement may be necessary along a relatively narrow strip of the canal west bank.

Demolitions - This layout requires the demolition and removal of heavily damaged residential structures on the east bank, existing levee, and miscellaneous site features in the area, in an amount slightly increased over Option 1.

Earthwork – Unlike Option 1, which requires localized earthwork only at the pump station facility, Option 2 requires significant canal excavation along the entirety of the 17th Street Canal. The basis of this anticipated canal excavation was developed and determined based on the results of the canal hydraulic analysis described in Section 5.1.2.3 and the geotechnical slope stability analysis described under Section 5.1.2.4. Note that the anticipated canal improvement cross-section consists of providing an added rectangular section in the existing canal invert, either as sheet pile walls or a concrete "U" channel section, as shown elsewhere. This approach maintains canal construction within the existing canal right-of-way. Laid back earth slopes (trapezoidal canal cross-section) have also been considered for required canal improvements. As described in Section 5.1.2.4, the acceptable stable slope for the 17th Street Canal is 5:1. Due to that extremely flat slope, and the correspondingly large volumes of right-of-way acquisition and channel excavation that would result, laid back slopes on the 17th Street Canal are not recommended for further consideration. Further analysis of the effects on right-of-way and excavation quantities brought about by laid back slopes is included in Appendix C.

Channel Transitions - Channel transitions for this Option 2 layout are similar to those described under Option 1, simply increasing in size due to the increased channel depth.

Erosion Protection – Option 2 erosion protection armoring is unchanged from Option 1 requirements.

Generator Building and Tank Farm Complex – The Generator Building and Tank farm Complex represent an area of substantial change from Option 1 to Option 2, due to the substantially increased head the pump station must overcome. Under this Option 2, the Generator Building is increase to be approximately 80 by 310 feet, and the number of 12,000 gallon fuel tanks required increases to 18. Further, the electrical substation is anticipated to grow in size to 20 by 40 feet. Like Option 1, the complex includes an allowance for parking, general staging and storage space, all concrete paved, and including sidewalk and local site storm drainage features. Utilities will include potable water service, sanitary sewer and natural gas, all connected to the station from existing utilities available within several hundred feet of the proposed site. The paved Complex area will also be enclosed by a chain link security fence, with minor landscaping improvements. All non-paved areas will be seeded to re-establish healthy turf for aesthetics and erosion control.

Summary – Like Option 1, this Option 2 layout is attractive for its protection of the pump station from lake surge effects, making no breakwater necessary and minimizing erosion protection requirements elsewhere. Also like Option 1, it requires a relatively large residential right-of-way acquisition, albeit in a highly damaged area and some construction sequencing is required to maintain canal flow during the construction duration.

- The **Alternative B Pump Station Layout** for Option 2 is as shown in Exhibit 5.2.1.B, Appendix C. As stated for the Option 1 Layout, this pump station location is not recommended for further consideration.

- The **Alternative C Pump Station Layout** for Option 2 is as shown in Exhibit 5.2.1.C Appendix C.

General Location and Description - The horizontal location of the pump station is identical to Option 1 for Alternative C. Changes from Option 1 to this Option 2 layout all result from the effects of the deeper canal and correspondingly deeper pump station inlet elevation, as described below.

Right-of-Way Acquisition - Permanent right-of-way acquisition increases somewhat under this Option 2, specifically requiring added property for the increased size of the Generator Building and Tank Farm Complex on the east bank.

Demolition – Demolition increases proportionately to the increased right-of-way acquisition for this Option 2 layout.

Earthwork – Unlike Option 1, which requires localized earthwork only at the pump station facility, Option 2 requires significant canal excavation along the entire length of the 17th Street Canal. The basis of this anticipated canal excavation is determined based on the results of the canal hydraulic analysis described in Section 5.2.2.3 and the geotechnical slope stability analysis described under Section 5.2.2.4. Note that the anticipated canal improvement cross-section consists of providing an added rectangular section in the existing canal invert, either as sheet pile walls or a concrete "U" channel section, as shown elsewhere. This approach maintains canal construction within the existing canal right-of-way. Similar to Layout Alternative A above, laid back slopes on the 17th Street Canal are not recommended for further consideration. due to the extremely flat side-slope required (5:1), and the correspondingly large volumes of right-of-way acquisition and channel excavation that would result, Further analysis of the effects on right-of-way and excavation quantities brought about by laid back slopes is included in Appendix C.

Channel Transitions - Like Option 1, both upstream transitions are anticipated to be constructed as reinforced concrete counterforted retaining walls that provide a smoothly warped flow surface over the length of the transition. Downstream, only the east bank requires a counterforted retaining wall transition, with the west bank discharging via a short vertical training wall. The upstream transitions simply increase in size due to the deeper canal invert under Option 2.

Erosion Protection - Option 2 erosion protection armoring is unchanged from Option 1 requirements.

Generator Building and Tank Farm Complex – The Generator Building and Tank Farm Complex represent an area of substantial change from Option 1 to Option 2, due to the substantially increased head the pump station must overcome. The Generator Building is anticipated to increase to 80 by 310 feet, and the number of 12,000 gallon fuel tanks required increases to 18. The required electrical substation is approximately 20 by 40 feet in size. As under Option 1, the complex includes parking, general staging and storage space, all concrete paved, and including sidewalk and local site storm drainage features. Utilities will include potable water service, sanitary sewer and natural gas, all connected to the station from existing utilities available within several hundred feet of the proposed site. The paved Complex area is also anticipated to be enclosed by a chain link security fence, with minor landscaping improvements. All non-paved areas will be seeded to re-establish healthy turf for aesthetics and erosion control.

Summary – Like Option 1, this layout is attractive for its minimal right-of-way acquisition requirements, and its in-line, shore-front location. Given the deepened canal, constructability is more complex compared to Option 1. Also like Option 1, the shore-front location requires substantial erosion control features, including a major breakwater structure. The layout does impact the historic Bucktown pedestrian bridge.

- **Existing Pump Station Demolition and Bypass.** Option 2 requires the abandonment of an historic inland pump station. In general, the historic elements of the pump station is to be preserved, while allowing the required canal flows to bypass the historic elements that remain, flowing into the newly degraded canals downstream of each existing pump station. For example, at DPS 6, the eastern portion of the pump station building is not historic; thus, it will be removed, while the western portion is largely historic and will be retained. Further, parking and access are also currently provided on the west bank, favoring removal of the eastern portion of the station. Some right-of-way acquisition is required to accomplish the work, with permanent acquisition anticipated on the east bank. Initially, interior wall improvements are anticipated to both reinforce and seal elements of the pump station building below the anticipated maximum adjacent water elevation. A temporary sheet pile training wall will be installed upstream of the pump station to route canal flow through the remaining (west bank) pump station during demolition and restoration activities (east bank). A permanent sheet pile wall will also be installed around and downstream of the pump station, located longitudinally in the canal. This wall effectively matches the canal cross-section employed to deepen the canal over its entire length. Once these site features are in place, demolition of the non-historic building will be performed from adjacent locations, all with water remaining in the canal. Upon the completion of building demolition, closure of the remaining existing building and removal of the upstream training wall may be completed.

5.2.2.2 Bridges and Utilities

Bridges

As discussed here, the existing bottom of the canal is approximately -18 (NAVD88) and the new bottom of canal for Option 2 being approximately -26 (NAVD88). It is this new channel section that will significantly affect all the bridges. Because of the lowering of the canal, weak soil conditions, and constructability under these bridges, significant impacts and cost are expected. Further investigation should be made and preliminary design developed for bridge modifications as replacement of these structures would be very costly. For the purposes of this report concrete box sections of equal hydraulic capacity are sunk between the support bents. This technique is constructable, accommodates stability of the slopes, while providing the needed hydraulic capacity. For purposes of this report, modification to the bridges under Option 2 utilize the box culvert technique.

The Southern Railroad Bridge located near Pump Station No. 6 for purposes of this report and because no information has been made available, will require replacement because of the insufficient capacity of the piles and the canal reconstruction close to Pump Station No. 6.

As previously stated Hammond Avenue Bridge is affected by the pump station site location as well as the Option 2 canal section. For pump station layout Alternate A Hammond Avenue Bridge is unaffected because of the pump station located on the upstream side of the bridge. Site Location B will require the bridge to be replaced and lengthened. Site Location C will require the Hammond Avenue Bridge to be modified as discussed previously.

For the I-10 bridges and the Veterans Boulevard Bridge the Option 2 canal section will require bridge modification as described above. It is also noted that should it later be discovered that these structures require replacement the impacts to the motoring public and local residents and businesses affected by additional rights-of-way could be significant along the roadway.

Utilities

The utilities studied in Option 2 are underground or pile supported water, sewer, drainage, electric (transmission and primary), telephone cables, fiber optic cables, and gas. In Option 2, the existing utilities impacted by construction in the vicinity of the 17th Street Canal are those utilities impacted by deepening the canal within the floodwalls from Pump Station No. 6 to the new pump station in the vicinity of Lake Pontchartrain.

Since the 17th Street Canal is approximately located on the boundary between Jefferson and Orleans Parishes, there are very few utilities that cross this canal. Jefferson and Orleans Parishes are each responsible for their own water, sewer, and drainage lines. Atmos Energy supplies gas within Jefferson Parish, and Entergy supplies gas service in Orleans Parish. Entergy supplies electricity in both parishes, but their (overhead only) crossing primary and secondary lines entirely span the floodwalls.

Entergy has 12 pile supported electric transmission poles located in the canal between the Southern Railroad and Veterans Boulevard. All 12 of these poles will need to be relocated outside of the channel. Ten of these poles are tangent poles and two are corner poles. The estimated relocation cost of these 12 poles is $1,080,000.

The only other known crossing utilities were a small (2") diameter gas line and a water line that previously were attached to the pedestrian bridge at the north end of Orpheum Street, and which no longer exists, since the Corps of Engineers acquired this property after Hurricane Katrina and removed the bridge and utilities in this vicinity.

5.2.2.3 Hydraulic

For this option, the Replacement Pump Station would replace the existing S&WB pump stations that discharge into the canal. Existing pump station facilities would be modified as necessary so that drainage would bypass the pump stations and be conveyed within the 17th Street Canal to the Replacement Pump Station. The required flowline elevation within the canal would be much lower than for existing conditions, and significant modifications to the canal would be required to accommodate the lowered flowline. These modifications would generally involve lowering the canal invert elevation with a modified cross-section to allow the design canal discharge to flow by gravity between the existing pump station locations and the Replacement Pump Station.

The maximum allowable upstream water surface elevation within the canal corresponds to the maximum allowable water surface elevation on the suction side of DPS 6. For purposes of this study, a maximum suction side elevation at DPS 6 of -10.9 ft. NAVD88 is used. This corresponds to the current "pumps on" operating condition at DPS 6 and is therefore conservative with respect to maximum allowable canal water elevation.

Hydraulic Analysis

The hydraulic analysis performed for this Option 2 was similar to the analysis for Option 1 – Pumping Mode. The USACE-developed HEC-RAS hydraulic model was used, with inflows to the canal representing bypass flows at DPS 6, the Canal Street Pump Station, and the I-10 Pump Station. Total design canal discharge at the Replacement Pump Station is 12,500 cfs, which includes the potential capacity increase of 2,000 cfs at DPS 6. The hydraulic model was developed based on NGVD29 datum; therefore, subtraction of 0.5 feet from model elevations is necessary for conversion to NAVD88 datum.

A starting water surface elevation at the Replacement Pump Station of -13.0 ft. NAVD88 was selected for use. The modified canal section was considered to be a concrete-lined, rectangular cross-section with a bottom width of 150 feet and vertical side walls. Inspection of the existing canal invert profile indicates the canal invert is configured in three horizontal steps with approximately constant elevations of -19.0 ft., -18.0 ft., and -17.0 ft., NAVD88. The modified canal invert profile was considered to be horizontal (constant elevation) between the Replacement Pump Station and DPS 6. This invert profile configuration approximates the profile configuration of the existing canal.

Using the selected starting water surface elevation and the rectangular canal cross-section, an iterative approach was used to determine the canal invert elevation that would result in a maximum canal water surface elevation at DPS 6 equal to the maximum allowable suction side elevation at this pump station. The HEC-RAS model was run for several canal invert elevations, and the resulting canal water surface elevation at DPS 6 was determined for each case. Based on the results of this analysis, a canal invert elevation of -25.3 ft., NAVD88 was determined to be required. The water surface profile within the canal for this flow condition is provided in Figure 5.2.2.3-1.

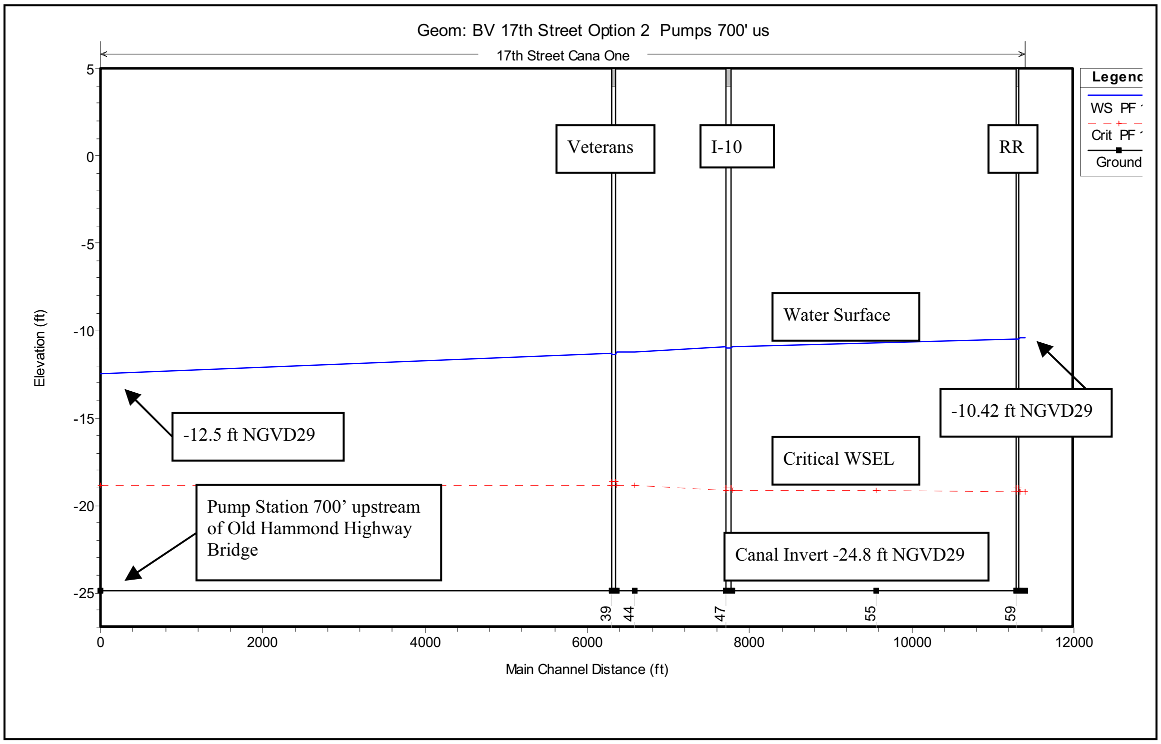


Figure 5.2.2.3-1. Water Surface Profile – 17th Street Canal (Option 2)

Numerous combinations of starting water surface elevation, canal cross-section geometry, and canal invert profile would result in the desired upstream canal water surface elevation. Consideration of such alternatives would be appropriate as part of an overall project evaluation comparing capital costs and annual operating costs, however this type of alternatives evaluation is not included in this study. The canal cross-section geometry that was selected represents a reasonable canal configuration for the given criteria.

5.2.2.4 Geotechnical

The typical stratification for the 17th Street Canal is taken from the IPET Report, Volume V. From the top down the stratification includes; Marsh Clay, a peat layer, Lacustrine Clay, Relic Beach Sand, and Bay Sound Clay. Pleistocene sand and clay strata are below the Bay Sound Clay. The peat layer is not present in all sections of the 17th Street Canal but is modeled to produce a conservative result. A typical representation of the canal geology modified for the deepened canal is shown in the figure below. The section is taken from the IPET report. Corresponding IPET report figures are included in Appendix B for reference.

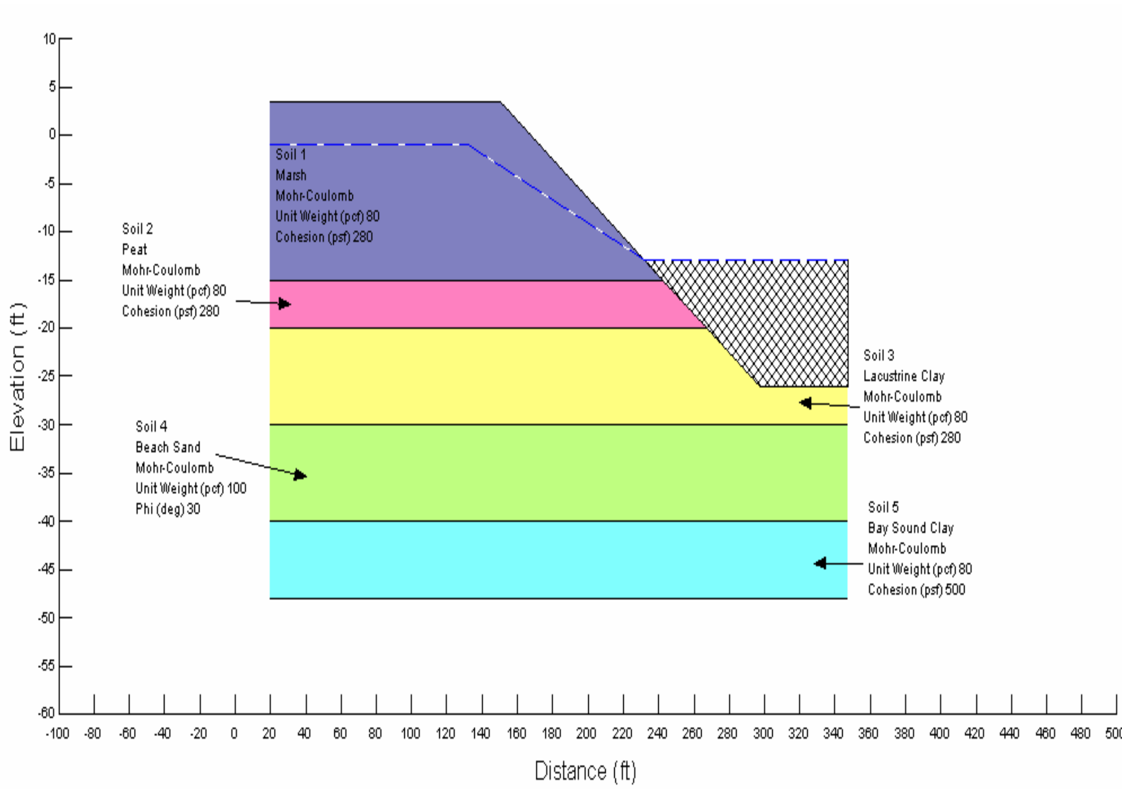


Figure 5.2.2.4-1. 17th Street Geology, Option 2

The strength and physical properties used to characterize these strata are also taken from the IPET Report, Volume V, Appendices 1, 3, 4 and 6. The parameters used in these various appendices are consistent and are taken as conservative. The parameters used in this study are shown in the figure above. Tables from the IPET report are reproduced in Appendix B to this report. These tables report strength evaluations performed as part of the IPET Report. In all cases the average strength values for each stratum reported are higher than those shown above. They are provided to demonstrate why the values used in this analysis are considered to be conservative and suitable for this study.

Canal

One of the significant changes required for Option 2 is the lowering of the flow line and the resulting deepening of the canal. The existing bottom of the canal is at approximate elevation -18 ft (NAVD). The new bottom needs to be at approximate elevation -26 ft (NAVD).

Stability analysis of the deeper canal shows the slopes of the canal do not meet safety criteria (figures B-13, 14, 15, Appendix B). The slopes of the canal were flattened to determine the safe slope which will produce a factor of safety of 1.5. Slopes of 5h:1v produce the required minimum factors of safety (Figure B-9, Appendix B). When 5:1 slopes are projected out, significant amounts of additional property are required. Therefore flatter slopes are not practical and the design is modified to include a concrete liner for the deepened canal. The concrete liner permits the slopes to be left at 3:1.

The lower flow line in the canal creates a potential recharging situation in which ground water from the adjoining properties would flow into the canal. This would result in dewatering of the nearby properties and very likely increase subsidence. Preliminary seepage flow analysis has been performed to estimate the shape and influence of the drawdown curve extending away from the deepened canal. When the canal liner is modeled with relief valves and the source of water is set 1000 ft from the canal, drawdown is unacceptable. The model shows a drawdown of 2 ft at a distance of 300 ft from the levee (Figure B-21, Appendix B). This extends well into the neighboring property and will produce unacceptable settlement. When the canal liner is modeled as watertight, the drawdown is 1 ft at 50 ft from the levee (Figure B-22, Appendix B). This magnitude of drawdown will not produce settlement of structures.

During discussion of the seepage analysis it was pointed out that the drainage system in the neighborhoods would act to artificially hold the water table up. When the recharge effect is added to the model, the drawdown is still present but is significantly reduced. The model predicts drawdown to be 1 ft at 100 ft from the levee. The recharging effect of the drain system is modeled based only on verbal description and is presented to show the possible effect. This needs to be verified before design decisions are made.

The design will have to be modified to control the drawdown if it the curve extends too far inland and would produce damaging settlements. Two possible modifications being considered are a cutoff wall and making the liner waterproof. The liner can be made waterproof to limit the drawdown but this will result in significant uplift pressures so tension piles or other measures would be required under the canal liner. Cutoff walls installed in or near the levees lengthen the flow path and can be designed to modify the drawdown curve to acceptable levels.

Pump Station Foundation

Evaluation of foundation issues for the Option 2 pump station at 17th Street Canal is under way at the time of this writing. Preliminary stability evaluation has been completed and is included as calculation file 1.1 in Appendix B. This stability analysis is a two-dimensional analysis of a typical section cut perpendicular to the long axis of the pump station. This section includes all driving and resisting loads. Analysis results are reported on a per foot basis representing the nominal one foot thickness of a two dimensional analysis. Conservative assumptions are made throughout so the two-dimensional analysis should represent conservative evaluation for this study. The issues identified for evaluation are described in the following paragraphs.

Sliding: The excavation for the intake basin will remove most of the material from the upstream side of the pump station. The pumps being considered for this study will require 14 ft of water to operate properly. The critical sliding case for the pump station will be when the downstream side is subjected to lake surge. Even though the lake surge is relatively short duration, the structure must be stable during the few hours it is present. Lake surge is assumed to be at +12 ft (NAVD) and the canal flow line is assumed to be at -13 ft (NAVD). The net pressure on the pump building is substantial. Several elements of the foundation design will contribute to or effect sliding resistance.

The thickness of the base slab is dictated by uplift considerations (see below). The slab will be entirely below grade so it will develop shear in the adjacent soil. At the anticipated depth of the base slab, this resistance will develop in the Lacustrine Clay. The clay has a weak shear strength with s_u = 280 psf. Therefore passive resistance alone is not expected to be adequate. Deep soil mixing is being used at the temporary protection structure to substantially improve the foundation soils and may be considered for this pump station as well.

The weight of the structure will generate friction on the base of the slab. Because of the high uplift pressures, net weight will be low. This means the base friction will not provide much resistance for sliding. Base friction can be improved somewhat by soil modification but will never be the controlling factor unless weight is added to the structure explicitly for this purpose. The addition of weight to improve base shear is not efficient and will not be considered for this study.

Preliminary calculations demonstrate a deficiency in sliding resistance of approximately 54 kip/ft to achieve a factor of safety of 1.5. Design elements will have to be added to provide this additional resistance so the desired factor of safety can be realized.

The greatest influence on sliding resistance will likely come from the foundation piling. Including lateral resistance into the design of vertical piles is one method of providing the additional needed lateral resistance. Another viable method of providing the resistance is the addition of battered piles. Battered piles are more efficient lateral resistance elements and will likely be used. In any case, soil modification can improve this resistance and will be considered as a possible supplemental measure.

Uplift: When the structure is subjected to lake surge, substantial uplift will develop on the base of the structure. The uplift is assumed to instantaneously reflect the lake surge pressures. Hydraulic conductivity of the Lacustrine Clay is somewhat low but the Beach Sand must be assumed to be hydraulically connected to the lake. Uplift pressure is calculated by assuming a linear variation from the lake head to the upstream head in the canal. If needed, this can be modified by installing a cut-off wall below the foundation. Cut-off walls can be effective but will be difficult to coordinate with a pile foundation anticipated for this application. For this study it will be assumed that no cut-off wall will be installed below the pump station for the purpose of reducing uplift pressures.

Conservative design resists uplift forces with dead load. At this stage of the study the base slab is being sized to provide the necessary dead load to resist uplift with a factor of safety of 1.1. This requires a base slab of 11 ft thickness with bottom at elevation -38 ft (NAVD). It is recognized that the vertical piling supporting the structure will also provide uplift resistance. For this study the piling will be considered to provide supplemental resistance to raise the factor of safety above 1.5, but will not provide the principal resistance system. This is a conservative approach that ensures long term stability against uplift failures.

Underseepage: Underseepage is a potential failure mode with the combination of high heads and weak foundation soils. Since the base slab is 11 ft thick, the length of the flow path may be long enough to eliminate this threat. Underseepage can result in the loss of the foundation material through piping beneath the foundation. Seepage calculations will be performed to check this failure mode. It is possible that a cut-off wall will be required.

Foundation Support: Preliminary calculations of overturning indicate the structure is not 100 percent in compression without the addition of tension elements. Tension elements with a capacity of 145 kip/ft will result in the required 100 percent base compression. The additional tension capacity is based on being able achieve a centroid of the tension elements at 2/3 the width of the base slab. Vertical tension piles will be assumed to develop the overturning stability.

The principle vertical resistance system for the pump station will likely be piling. This is common practice for the area when foundation soils are too weak for the structural loading. The strength of the soils can be improved by soil modification techniques which could be used to reduce or eliminate piling and this will be considered. However the big issue for this structure will be settlement. Ground modification would have to extend to greater depths if it is desired to eliminate piling. For simplicity the structure will be founded on piling for this study. The concept design of the piling will ensure it will resist static vertical load when the gates are open and the eccentric loading produced by unbalanced water pressure when the gates are closed.

5.2.2.5 Structural

See the general discussion for the state of structural design in paragraph 5.1.2.5. The critical foundation design elements are discussed in paragraph 5.2.1.2 immediately above.

5.2.2.6 Mechanical

The function of the pumping station is to lift water from the canals to the lake. The principle piece of machinery to do this is the pump. High capacity, low head pumps are essentially large propellers in a tube. For this application there are two principle types of pumps defined by the orientation of the propeller. The propeller can be installed horizontally or vertical. The horizontal types are similar to the Woods Screw Pumps which are extensively used in the older pumping stations.

The horizontal pumps are installed horizontally on an operating floor above the maximum canal level. As such, the propeller is above the water surface. To operate the pump, a vacuum is used to extract air out of the pump and pump discharge piping until the propeller is submerged. Once the propeller is submerged, the pump can be turned on and the pump will complete the filling of the discharge pipeline and establishing a siphon discharge. The major advantage of the horizontal pumps is that the pump bearings and propeller are located above the canal and the easily accessible for maintenance. The pump can actually be started before the propeller becomes fully submerged permitting a low startup torque which minimizes engine generator sizing. The major disadvantage is that the pumps need to be primed by a vacuum system. Due to the volume of air needed to be evacuated, it can take 10 to 15 minutes to get the pump started.

The vertical pump has the propeller mounted down below the minimum canal water surface level. Like the horizontal screw pump vertical pumps are also used extensively in the Parish. As a result the pump design, the pump is self-priming and can start pumping within seconds of a start command which is a significant advantage in controlling pumping units when pumping stations are located in series. Also with this design, the motors are located on top of the pump and out of any danger of being damaged by flooding. The major disadvantage is that the propeller is below the water surface and that any major maintenance requires fully disassembling the pump. Also a disadvantage is that the pump starts under load and has a high startup toque which can require over sizing engine generators.

Either type pump is applicable to this pumping station. For simplicity of this analysis, only vertical pumps which provide the maximum flood protection with the elevated motors are considered in the station. During detailed design the use of vertical, horizontal, or a combination of both should be considered.

For reverse flow protection, the discharge pipe from the pumps are elevated such the invert of the pipe at the highest point is at or above the floodwall elevation so that reverse flow through the pump is not likely. The discharge pipe is then brought down below the minimum lake level forming a siphon. A siphon discharge permits recovering the energy so that when normally pumping the pumps only see the difference between the canal elevation and the lake. A vacuum breaker is provided at the highest point in the discharge pipe to permit breaking the siphon when the pump stops. For added protection, sluice gates can be added to the discharge pipe for protection against reverse flow. To minimize submergence and hydraulic losses through the station, a formed suction inlet was used in the analysis. A typical cross-section of the pumping station is attached to Appendix D – Mechanical.

The pumping units can be driven by either electric motor with electric generator backup or direct driven by engines. In the final design, there may be a combination of drivers in the pumping station. Direct driven engines are cheaper since they eliminate the engine generator and motor. Motor driven pumps are quieter and more efficient. A determining factor may well be the ability or willingness of the power company to build power lines and reserve generating capacity for pumping units which may only occasionally be operated. A detailed study should be done during design to determine the optimum combination of electric driven motors with engine generator backup or direct engine driven pumps. For the purpose of this analysis approximately 60 percent motor driven pumps and 40 percent engine driven pumps are assumed.

A polling of pumping manufacturers indicated that the maximum practical size of pumping units is roughly 1,000 cfs. This is limited by the physical size of the equipment and the ability to move the equipment along major roadways. 1,000 cfs also matches up with the largest pumping stations in the major feeder pumping stations so was chosen as the main pumps in the new pumping stations. The existing pumping stations also have a number of smaller pumps. Smaller pumps permit pumping lower flows without having frequent starts and stops and also provided the ability to match flows when pumping stations operate in series. Therefore, combinations of 1,000, 500, and 250 cfs pumps were selected at as the primary capacities. Using only three sizes will permit the sharing of parts between the pumping stations.

Under Option 2 the pumping station shall be provided with screens ahead of the pumping stations to protect the pumps for large solids as the screens at Pumping Station 3, 6, and 7 will be eliminated.

There are a number of additional mechanical systems required for operation of the pumping station. All major pumps require a clean source of water for bearing lubrication. This can be from the water system. However, based on experience during Katrina in which the water system failed, a secondary source of water should be provided. There are two sources available, either canal water or well water. Because the water to the pumps needs to be of high quality, well water is being considered for the pumping station. Canal water can be used but requires a high level of treatment to remove abrasives. Because of the size of the equipment in the facility, the facility should include overhead crane, lay down space, truck loading access, and workshop areas. Whether motor driven or engine pumps are used, there will be significant opening for ventilations. All inlet air vents should be shrouded to inhibit the entry of wind blown water.

Engine driven pumps have additional mechanical considerations including engine and gear reducer cooling systems, fuel system, starting air systems, lubricating oil and waste oil systems. In addition, engines require both exhaust air and combustion air systems.

Pumping station hydraulics are critical to successful operation of the pumping station to achieve maximum hydraulic performance. A physical model test of the pumping station including canal entrance, screens, and pump inlet, and discharge siphon pipe must be conducted as a follow-on effort to ensure correct sizing and configuration.

For the purpose of this study, the following combination of vertical pumping unit capacities was chosen. In addition, a combination of direct drive diesel engines and diesel generators are assumed to provide an approximate 60 percent electric motor drive and 40 percent diesel engine drive ratio.

Pump Driver Type		
Pump Number	**Pump Capacity cfs**	**Driver Type**
1	1000	Direct Drive Engine
2	1000	Direct Drive Engine
3	1000	Direct Drive Engine
4	1000	Direct Drive Engine
5	1000	Motor on Generator
6	1000	Motor on Generator

Pump Driver Type		
Pump Number	Pump Capacity cfs	Driver Type
7	1000	Motor on Generator
8	1000	Motor on Generator
9	1000	Motor on Generator
10	1000	Motor on Generator
11	1000	Motor on Power Grid or Generator
12	500	Motor on Power Grid or Generator
13	500	Motor on Power Grid or Generator
14	250	Motor on Power Grid or Generator
15	250	Motor on Power Grid or Generator

Fuel storage capacity for the pump station was selected at 4 days of full pumping capacity. Based on this duration and usage rate, the anticipated fuel storage required is slightly over 207,000 gallons. Assuming standard 12,000 gallon double wall fuel storage tanks, a minimum of 18 tanks will be required.

The estimated pump ratings are as follows:

Pump Rating			
Capacity, cfs	1,000	500	250
Bowl Head, ft	25.7	25.7	25.7
Pump Speed	162	227	321
Engine Rating, bhp	4100	---	---
Motor rating, hp	3500	2000	900

In the Mechanical Appendix D are representative pump performance curves submitted by the manufacturers. The curves are presented as typical curves only as the required pump rating have evolved during the study and the pump ratings on the curves will differ slightly from the latest hydraulic requirements.

5.2.2.7 Electrical

Option 2 removes existing pump station DPS6 from service and therefore requires more power to achieve the same flow as Option 1. The general arrangement of electrical equipment remains the same, but larger electrical equipment is required to meet the increased load demands. Utility power, from Entergy, will only be supplied for the normal pump loads (see Pump Driver Schedule below for pump utilization). The incoming utility service will not be sized to accommodate the storm event pump loads. One hundred percent of the storm event electric-driven pump loads will be supplied from standby generators dedicated to the 17th Street Pump Station. The standby generators will utilize an N+1 design such that if a generator goes off line or one is down for maintenance, the full pump station load will still be supplied by standby power. The utility service, standby generators, and pump motors will all operate at 4160 volts. All electrical distribution circuits will be routed underground in concrete-encased ductbank. The Pump Driver Schedule below illustrates the increased power requirements of Option 2.

Option 2 - 17th Street Canal – Pump Driver Schedule						
Pump	**Driver**	**Motor hp**	**Motor hp**		**Utilization**	
cfs	**bhp**	**Nameplate**	**Load**	**Source**	**Grid**	**Ind**
1000	3206			Engine		1%
1000	3206			Engine		1%
1000	3206			Engine		1%
1000	3206			Engine		1%
1000	2719	4550	3825	Standby Generator		1%
1000	2719	4550	3825	Standby Generator		1%
1000	2719	4550	3825	Standby Generator		1%
1000	2719	4550	3825	Standby Generator		1%
1000	2719	4550	3825	Standby Generator		1%
1000	2719	4550	3825	Standby Generator		1%
1000	2719	4550	3825	Grid/Generator	1%	1%
500	1359	2250	1925	Grid/Generator	5%	1%
500	1359	2250	1925	Grid/Generator	10%	1%
250	680	1250	950	Grid/Generator	50%	1%
250	680	1250	950	Grid/Generator	100%	1%
12500			**32525**	**Totals**		

Based on the pump driver information above, the next table below indicates the electrical equipment sizes used for this option. See the appendices for all cost information.

Option 2 – 17th Street Canal Pump Station - Electrical	
Electrical Equipment	**Quantity/Capacity**
Utility Substation	2 – 5000kVA
Generators	16 – 2500kW
Generator Bldg Size	328' x 80'
Pump Bldg Switchgear	3 – 3000A
Total Pump Station Load	34.2 MVA, 4743A
Pump Station Load on Utility	11.2 MVA, 1556A

5.2.2.8 Environmental

Environmental Site Assessment (ESA)

Because conditions/impacts and results for the ESA under Option 2 for the 17th Street Canal are identical to those discussed in Section 5.1.2.8, please see that section for this discussion.

Sediments

Because sediment quality and findings for Option 2 for the 17th Street Canal are identical to those discussed in Section 5.1.2.8, please see that section for the majority of this discussion. However, because this option contains the dredging of a large quantity of bottom sediments, costs will be significantly higher if disposal at a hazardous waste permitted land fill or on-site treatment is required. Transportation costs for disposal at a permitted land fill may render this option as cost prohibitive. On-site treatment (i.e., thermal treatment of organic contaminants) would most likely be the preferred method, if required. Once treated, sediments could be disposed in a construction debris land fill or perhaps beneficially used.

Wetlands

See discussion in Section 5.1.2.8.

Protected Species

See discussion in Section 5.1.2.8.

Cultural Resources

Please see discussion in Section 5.1.2.8.

5.2.2.9 Constructability

The construction concepts developed for Option 2 are very similar to the Option 1 concepts. The big differences between Option 1 and Option 2 as they affect constructability are the deeper excavation required for the pump station and the channel modifications.

The concept of enclosing the entire new pump station in a cofferdam for all location alternates is still valid. The pump station will be larger in plan and deeper. The deeper excavation for the pump station will require a more robust design for the cofferdam. The cofferdam concept remains the essentially same as described for Option 1 in 5.1.2.9 and illustrated in figures 5.1.2.9-1, 5.1.2.9-2, and 5.1.2.9-3 shown earlier.

When constructing the bypass channels for Option 2 at location alternates A and C, the bottoms of the bypass channels only need to match the existing channel bottom. Therefore the concept of the by pass channel is also the same as for Option 1.

The same problems and costs are associated with construction of by pass channels for location alternates A and C as described in Option 1.

The second concept of building the pump station in two parts to eliminate the need for a bypass channel becomes more complex. The Option 2 pump stations do not include a gate. The first half of the pump station can be constructed by leaving half of the existing channel open. Construction of the second half will result in complete closure of the existing canal.

Therefore the design of the first half would need to be modified to include gates adequate to pass the flows require for the temporary gate and pumps. This will make the structure larger and more expensive. Furthermore, the gates would only be used in the construction period and never required after that time. This adds a point of vulnerability to the pump station which could be avoided by constructing bypass channels.

Again, the question becomes one of cost. The costs developed for this report include the construction of bypass channels. The design build contractor would be permitted chose either of these concepts or innovate a third concept of his own if it would reduce project cost. See figures 5.1.2.9-1, 5.1.2.9-2, and 5.1.2.9-3 for plan representations of cofferdam and bypass requirements.

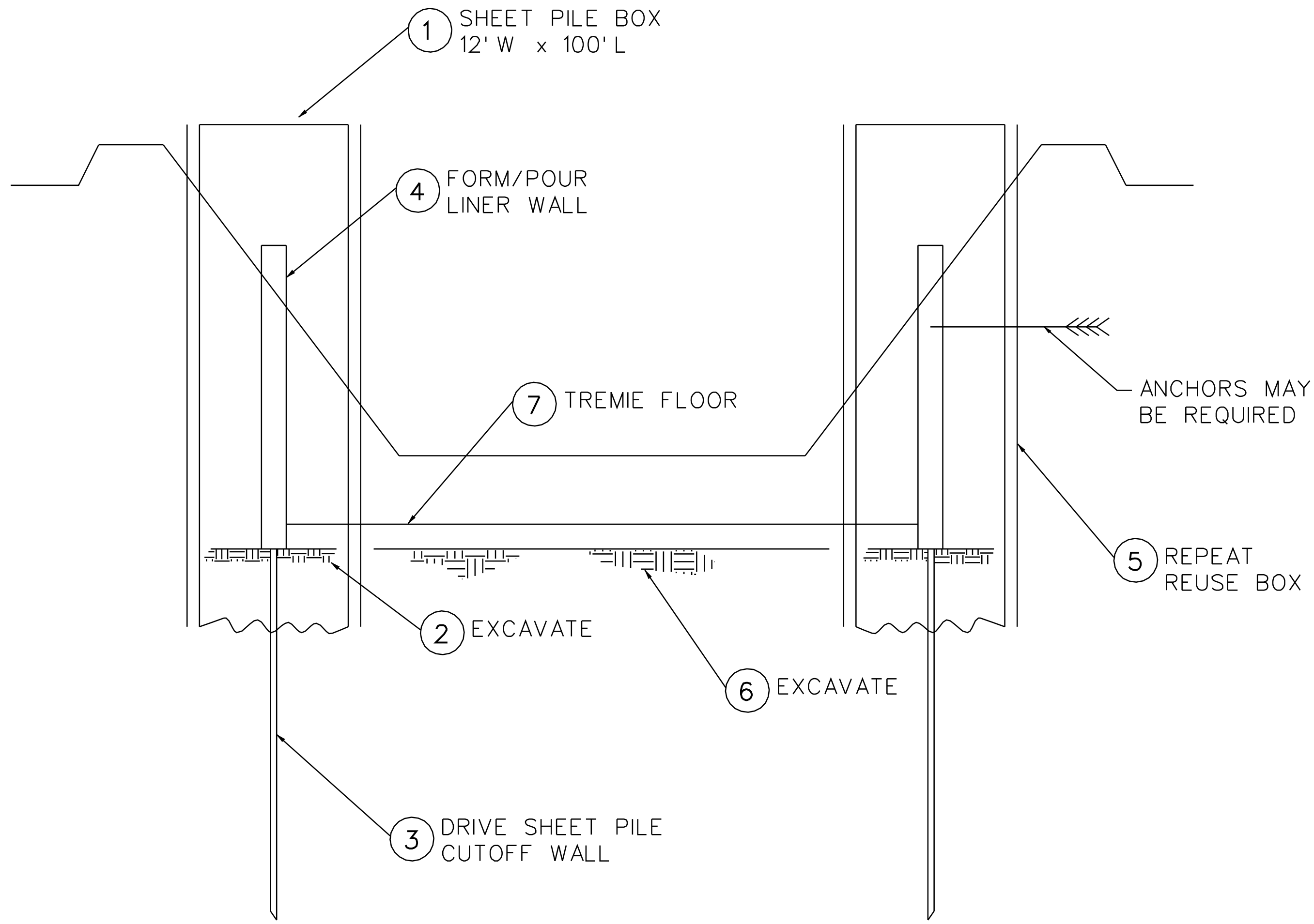
Option 2 also requires modification to the existing canal by deepening it. Construction of the concrete liner for the canal must be accomplished in such a way as to permit flows in the canal. The quantity of flow to be passed is determined by the design of the temporary structure. There is no need to pass more flow than that structure can pass. Two concepts have been developed for construction of the concrete liner.

The first concept is illustrated in Figure 5.2.2.9-1 on the following page. This concept minimizes constriction of the existing canal. Each wall of the concrete liner is constructed in the dry inside a cofferdam box. The box is then moved to the other side of the canal to construct the opposite wall. The box is moved to the next section and the floor of the liner is placed in the wet with tremie concrete. The cofferdam box is reused for each segment of the canal liner along the full length of canal to be modified. The length of the box can be adjusted to suit the design and schedule. Sheet pile cutoff walls are installed below each wall to improve stability and to provide a seepage barrier is required. (See the geotechnical discussion above.) Soil anchors or deadman anchors may be needed to provide stability to the walls.

A second concept uses a larger box to enclose half of the canal. This concept allows construction of the wall and floor both in the dry. It ensures a better connection of the two elements and may result in elimination of the cutoff wall. The wall is completed in two steps instead of three. The chief disadvantage is the available cross-section of the canal for flow is cut in half. This may not be adequate.

A third concept has been discussed during development of the project as having been used successfully in the past. This concept provides for damming both ends of the section being built and dewatering for construction in the dry. If an event requires it, the dam can be breached or allowed to overtop so the event flows can be passed. This technique has been used to line canals in New Orleans. However the deepening of this canal makes this technique impractical. Pressure heads would be on the order of 25 feet.

CANAL LINER CONSTRUCTION CONCEPT



CONCEPTUAL DESIGN STUDY
PERMANENT FLOOD GATES AND PUMP STATION
LONDON AVENUE – OPTION 1C
CONSTRUCTION CONCEPT
FIGURE 5.2.2.9-1

5.3 Option 1 – Orleans Canal

5.3.1 Alternative Approaches

Three location alternatives were considered for Option 1 on the Orleans canal. The merits of each were evaluated and discussed in detail in the Civil/Site section. For the purposes of this study, Alternative A was chosen as the location to base costing and other engineering considerations.

5.3.2 Engineering Considerations

5.3.2.1 Civil/Site

The Orleans Avenue Pump Station for Option 1 is anticipated to be 130 feet long for Layout Alternative A (the re-use of the temporary gate structure reduces the pump station length) and 155 feet long for Layout Alternatives B and C. The pump station width is 155 feet wide in every case. The total station width includes a 45 foot inlet works including trash screens, a 70 foot pump station building housing pumps and motors, and a 40 foot outlet works. Finish grade for the Generator and Tank Farm Complex is always approximately +16.0 elevation.

- The **Alternative A Pump Station Layout** is as shown in Exhibit 5.3.1.A, Appendix C.

<u>*General Location and Description*</u> - Under this alternative, the pump station is located on the east canal bank, immediately adjacent to the temporary gate structure, in order to obtain cost savings by converting the temporary gate structure to permanent status, which correspondingly reduces the pump station size. This layout alternative does require right-of-way acquisition of currently undeveloped property. This alternative also provides for convenient connection of existing levees to the new pump station structure. Finally, the inland pump station location shields the pump station from lake surge effects. That is, no breakwater structure is required.

<u>*Right-of-Way Acquisition*</u> - Permanent right-of-way acquisition will occur almost exclusively on the east bank of this proposed site, all currently undeveloped property. Temporary construction easement is assumed to be necessary along a relatively small area in the vicinity of the west end of the temporary gate structure, and to accommodate the relocated temporary power plant on the east bank.

<u>*Demolitions and Earthwork*</u> - This layout requires no significant demolition or removal of existing structures of any kind. Some existing levees, and miscellaneous site features in the area will be removed. Earthwork at this site is almost exclusively excavation, resulting in a volume of earth materials to be removed from the project site. The temporary power plant supporting the construction and operation of the temporary gate structure may conflict with the location of this permanent pump station. Perhaps, in future design stages, the pump station can be more precisely aligned to avoid this power

plant. However, for the purposes of this study, the power plant is assumed to be relocated to the east bank, just upstream of the upstream channel transition.

Channel Transitions - Channel transitions are required both immediately upstream and downstream of the pump station to ensure laminar flow between the trapezoidal canal cross-section and the rectangular pump station cross-section. However, due to site geometry, both upstream and downstream transitions are required only on the east bank. Also due to the placement of the pump station, maximum convergence/ divergence angles are not applicable. Counterforted retaining walls, in this application, offer the maximum in very long-term durability, low maintenance, and good flow characteristics. Clearly, transition structures could be constructed in other ways that might offer significant cost savings over counterforted walls. Tied-back sheet pile walls could be used, but long term durability and corrosion resistance are issues. Therefore, for conservatism under this study, concrete retaining walls are anticipated, pending subsequent optimization studies on the subject.

Erosion Protection - A relatively small volume of erosion protection armoring will be required in and around this pump station. Specifically, a strip of riprap protection is anticipated in the new canal floor, both immediately upstream and downstream of the pump station. Given the inland location of this pump station, no breakwater in Lake Pontchartrain is anticipated to be necessary to protect the pump discharge from lake surge effects.

Generator Building and Tank Farm Complex - Supporting the pump station on the east bank, immediately adjacent to the pump station, is a Generator Building and Tank Farm Complex. The Generator Building is anticipated to be approximately 76 by 80 feet, and each of two 12,000 gallon fuel tank pads is anticipated to be approximately 12 by 28 feet in size. According to NFPA 30, 2003 Edition, diesel fuel is a Class II Combustible Liquid. As such, tanks require a five foot clear distance from the public way or important buildings on the site. A 20 by 20 foot electrical substation will also be included. The complex also includes parking, general staging and storage space, all concrete paved, and including sidewalk and local site storm drainage features. Utilities will include potable water service, sanitary sewer and natural gas, all connected to the station from existing utilities available within several hundred feet of the proposed site. Although not a significant cost item, the provision of these utilities when the station is operating under total self-sufficiency is anticipated. The paved complex area is also anticipated to be enclosed by a chain link security fence, with minor landscaping improvements. All non-paved areas will be seeded to re-establish healthy turf for aesthetics and erosion control.

Summary – This layout is attractive for its protection of the pump station from lake surge effects, resulting in no need for a breakwater structure. The location requires right-of-way acquisition in a residential area, but acquisition appears to be exclusively undeveloped property. Further, it is attractive since no significant construction sequencing is required to maintain canal flow during the construction duration, given the conversion of the temporary gate structure to permanent service. The potential relocation of the existing temporary power plant, depending on the final precise location of the pump station, is unfortunate but manageable.

- The **Alternative B Pump Station Layout** is as shown in Exhibit 5.3.1.B, Appendix C.

General Location and Description - Under this alternative, the pump station is located in the existing canal, as near the Lake Shore Drive Bridge as possible without creating the need for modifications to that bridge. Thus, the pump station is only approximately 300 feet upstream of Lake Shore Drive. Due to the existing canal curve, inlet channel transition hydraulics are negatively, but not significantly, impacted. The near-shore location of the pump station discharge requires the inclusion of a major breakwater structure in Lake Pontchartrain. Right-of-way acquisition is required, but the acquisition appears to include only currently undeveloped property. The temporary gate structure upstream of this site will be removed after pump station construction is complete. This location provides for convenient connection of existing shore-front levees to the new pump station features.

Right-of-Way Acquisition - Permanent right-of-way acquisition of currently undeveloped property will occur almost exclusively on the west bank of this proposed site. Some minor temporary construction easement is assumed to be necessary along the east bank.

Demolition and Earthwork - This layout requires no significant demolition or removal of existing structures of any kind. Some existing levee and miscellaneous site features in the area will be removed. Earthwork at this site is almost exclusively excavation, resulting in a volume of earth materials to be removed from the project site. The temporary power plant supporting the construction and operation of the temporary gate structure may conflict with the location of this permanent pump station. Perhaps the proposed pump station can be aligned to avoid this temporary power plant. However, for the purposes of this study, the power plant is assumed to be relocated to the east bank, just upstream of the upstream channel transition.

Channel Transitions - Channel transitions are required both immediately upstream and downstream of the pump station, on both banks, to ensure laminar flow between the trapezoidal canal cross-section and the rectangular pump station cross-section. For upstream transitions, the maximum divergence angle is not applicable. For downstream convergence, the maximum preferred angle of 25 to 30 degrees does apply. Transitions as counterforted retaining walls offer the maximum in very long-term durability, low maintenance, and good flow characteristics. In subsequent design phases, transition structures could be constructed in other ways that might offer significant cost savings over counterforted walls. Tied-back sheet pile walls could be used, but long term durability and corrosion resistance are issues. Therefore, for conservatism under this study, concrete retaining walls are anticipated, pending subsequent optimization studies on the subject.

Erosion Protection - Given the lakeshore location of this pump station, a significant volume of erosion protection armoring will be required, primarily located in a major breakwater in Lake Pontchartrain. Also, a strip of riprap protection is anticipated in the new canal floor, both immediately upstream and downstream of the pump station.

Generator Building and Tank Farm Complex - Supporting the pump station on the west bank adjacent to the pump station is a Generator Building and Tank Farm Complex. The Generator Building is anticipated to be approximately 76 by 80 feet, and each of two 12,000 gallon fuel tank pads is anticipated to be approximately 12 by 28 feet in size. A 20 by 20 foot electrical substation is also included. The complex includes parking, general staging and storage space, all concrete paved, including sidewalk and local site storm drainage features. Utilities will include potable water service, sanitary sewer and natural gas, all connected to the station from existing utilities available within several hundred feet of the proposed site. The paved complex area is also anticipated to be enclosed by a chain link security fence, with minor landscaping improvements. All non-paved areas will be seeded to re-establish healthy turf for aesthetics and erosion control.

Summary - This layout is attractive for its convenient fit within the existing canal width. The location requires right-of-way acquisition in a residential area, but acquisition appears to be exclusively undeveloped property. Due to the lakeshore discharge location, a major breakwater structure is required. In summary, this option seems to create negatives compared to Layout A, while adding no advantages over Layout A. Thus, it is not recommended for further consideration.

- The **Alternative C Pump Station Layout** is as shown in Exhibit 5.3.1.C, Appendix C.

General Location and Description - This alternative provides a pump station location just downstream of the Lakeshore Drive Bridge, essentially constructed in its entirety in Lake Pontchartrain, positioned on the linear extension of the existing canal. The pump station is approximately 500 feet downstream of Lake Shore Drive. The in-lake location of the pump station requires the inclusion of a major breakwater structure in Lake Pontchartrain to protect the pump station discharge. The location also requires significant earthwork to create the site.

The temporary gate structure upstream of this site will be removed after pump station construction is complete. Note finally that this location requires some modifications to extend the existing shore-front levee line out into Lake Pontchartrain, including the removal and replacement of the Lake Shore Drive Bridge.

Right-of-Way Acquisition - Right-of-way acquisition is required primarily for shore-located support features, and may represent areas that are publicly-owned, rather than privately-owned, properties.

Demolition – The only significant removal (and replacement) this layout requires is the Lake Shore Drive Bridge. Some existing levee and minor miscellaneous site features in the area will be removed as well.

Earthwork - Earthwork at this site may approach a balance between cut and fill, given the in-lake location.

Channel Transitions - Channel transitions are required immediately upstream of the pump station, on both banks, to ensure laminar flow between the trapezoidal canal cross-section and the rectangular pump station cross-section. Due to the lake discharge location, relatively short vertical training walls serve as the downstream discharge transition.

Erosion Protection - Given the in-lake location of this pump station, a significant volume of erosion protection armoring will be required, primarily located around the banks of the pump station facility and the breakwater structure. Also, a strip of riprap protection is anticipated in the new canal floor, both immediately upstream and downstream of the pump station.

Generator Building and Tank Farm Complex - Supporting the pump station on the west shore of the mouth of the canal is a Generator Building and Tank Farm Complex. The Generator Building is anticipated to be approximately 76 by 80 feet, and each of two 12,000 gallon fuel tank pads is anticipated to be approximately 12 by 28 feet in size. A 20 by 20 foot electrical substation is also included. The complex includes parking, general staging and storage space, all concrete paved, including sidewalk and local site storm drainage features. Utilities will include potable water service, sanitary sewer and natural gas, all connected to the station from existing utilities available over 1,000 feet from the proposed site. The paved complex area is also anticipated to be enclosed by a chain link security fence, with minor landscaping improvements. All non-paved areas will be seeded to re-establish healthy turf for aesthetics and erosion control.

Summary - This layout is attractive for its relatively minimal right-of-way acquisition requirements. Constructability is a mixture of positives and negatives. Constructing the pump station building in one phase is positive; however, construction in the lake offers other complications that are costly to overcome. Further, a major breakwater structure and significant plant armoring is required.

5.3.2.2 Bridges and Utilities

Bridges

The Orleans Canal is crossed by five bridges between the Pump Station No. 7 and the outfall into Lake Pontchartrain. These bridges and their locations are identified as follows.

Interstate 610 Bridges – Exhibit 5.3.2.2A, Appendix F.

The I-610 Bridge over the Orleans Avenue Outfall Canal is located less than 0.1 miles downstream of Pump Station No. 7. The bridge consists of 2-170' composite plate girders over the canal supported by 36" columns with a top-of-foundation elevation of –1.20' on the west bank, 54" P. P. C. piles with an approximate tip elevation of –115' in the center of the canal, and 54" P. P. C. piles with an approximate tip elevation of –80' on the east bank.

Harrison Avenue Bridge – Exhibit 5.3.2.2B, Appendix F.

The Harrison Avenue Bridge over the Orleans Avenue Outfall Canal is located approximately 0.6 miles downstream of Pump Station No. 7. The bridge consists of four concrete slab spans totaling 154' – 08" supported by 24" P. P. C. piles with an approximate tip elevation of –78.5. The end bents consist of HP 14x73 steel piles with an approximate tip elevation of –76' with sheet pile walls outside the end bents extending to an elevation of approximately –11.5'.

Filmore Avenue Bridge -- Exhibit 5.3.2.2.C, Appendix F.

The Filmore Avenue Bridge over the Orleans Avenue Outfall Canal is located approximately 1.0 miles downstream of Pump Station No. 7. The bridge consists of four concrete slab spans totaling 178' – 08" supported by 24" P. P. C. piles with an approximate tip elevation of –88. The end bents consist of HP 14x73 steel piles with an approximate tip elevation of –90' with sheet pile walls outside the end bents extending to an elevation of approximately –9'.

Robert E. Lee Boulevard Bridge – Exhibit 5.3.2.2 D, Appendix F.

The Robert E. Lee Boulevard Bridge over the Orleans Avenue Outfall Canal is located approximately 1.5 miles downstream of Pump Station No. 7. The bridge consists of three concrete slab spans totaling 140' supported by 24" P. P. C. piles with an approximate tip elevation of –84. The end bents consist of HP 14x73 steel piles with an approximate tip elevation of –84' with sheet pile walls outside the end bents extending to an elevation of approximately –15.5'.

Lakeshore Drive Boulevard – Exhibit 5.3.2.2 E, Appendix F.

The Lakeshore Drive Bridge over the Orleans Avenue Outfall Canal is located approximately 2.1 miles downstream of Pump Station No. 7. The bridge consists of four spans totaling 212' supported by 20" P. P. C. piles with approximate tip elevations ranging from –69.36 to –70.25' (plan tip elevation –84'). The end bents consist of 18" P. P. C. piles with an approximate tip elevation of –72' (plan tip elevation –84'). There is a stepped seawall along the inner slopes of the canal extending approximately 3' below the surface of the normal water level. This seawall is supported by a double and single bent of 12" precast (non-prestressed) concrete piles with an approximate tip elevation of –37.17' The lower end of the seawall is supported by a 9" x 24" concrete sheet pile wall extending to an elevation of –37.58.

None of the bridges are affected by Option 1 except the Lakeshore Drive Bridge, which may be affected by the pump station site location. For pump station layout alternates A and B there are no affects to the Lakeshore Drive Bridge. For site location Alternate C the Lakeshore Drive Bridge would require replacement.

Utilities

The utilities studied in Option 1 are underground or pile supported water, sewer, drainage, electric (transmission and primary), telephone cables, fiber optic cables, and gas. In Option 1, the existing utilities impacted by construction in the vicinity of the Orleans Avenue Canal are those utilities displaced as a result of the new pump station and gated structure in the vicinity of Lake Pontchartrain.

In alternatives A and B, for Option 1, there are no impacted existing utilities. In Alternative C, the only utility impacted is a canal crossing 8" diameter water line that is attached to the Lakeshore Drive bridge deck. This water line will need to be replaced along with the Lakeshore Drive Bridge if Alternative C is selected. The cost to replace this existing 8" water line is $12,600 in addition to the Alternative C bridge replacement costs.

5.3.2.3 Hydraulic

General

The Orleans Avenue Canal segment considered in this study conveys pumped discharges from DPS 7 to the canal outfall at Lake Pontchartrain. The safe water elevation within the Orleans Avenue Canal, as provided by the USACE, is El. 9.0 NAVD 88. This elevation is considered to be the maximum allowable water surface elevation at any point along the canal. As a practical matter, the controlling location for this safe water level is DPS 7, since the down-gradient slope of the water surface profile within the canal during typical flow conditions will result in water surface elevations at all other points that are lower than the water surface elevation at DPS 7.

For purposes of this study, it is assumed the pumping capability of existing DPS 7 would be modified, as necessary, to pump at the design discharge capacity and at a head corresponding to the defined safe canal water surface elevation. These modifications, if required, are not considered in this study. It is recognized the rated head of the existing DPS 7 pumping facility may be less than the required head for the defined safe canal water elevation. If a lower canal water surface elevation at the discharge side of the existing DPS 7 is considered appropriate, the hydraulic analysis presented herein would require revision to account for this lower canal water elevation.

Hydraulic analysis of the canal was performed to determine the following:

- During pumping mode at the design canal discharge condition, determine the maximum canal water surface elevation at the suction side of the Gated Pump Station that will result in a canal water surface elevation at

DPS 7 equal to the safe water elevation. This information is necessary to determine pumping head requirements.

- During gates-open operating mode, determine the canal water surface elevation at DPS 7 for various combinations of Lake Pontchartrain elevation and canal discharge. For the given safe water elevation, this will indicate when gate closure is required, with transition from gates-open to pumping mode. This information, in combination with annual canal discharge and Lake Pontchartrain elevation data, will be used to determine annual pumping requirements.

The USACE developed a HEC-RAS computer hydraulic model to estimate canal water surface profiles and other hydraulic information for various combinations of Lake Pontchartrain elevation and canal discharge. The model includes the existing canal cross-section geometry and invert slope between DPS 7 and Lake Pontchartrain. The model also includes the canal cross-section geometry at the several bridge crossings. This hydraulic model was used as the basis for the hydraulic analyses performed for this study. Modeled canal inflow and starting water surface elevation was adjusted appropriately to represent the conditions being considered for this Option. The existing canal geometry was considered to remain unchanged. The hydraulic model was developed based on NGVD29 datum; therefore, subtraction of 0.5 feet from model elevations is necessary for conversion to NAVD88 datum. A simplified flow schematic of the HEC- RAS model is shown in Figure 5.3.2.3-1.

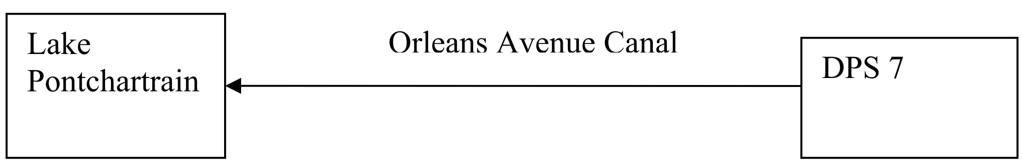


Figure 5.3.2.3-1. HEC-RAS Model Flow Schematic – Orleans Avenue Canal

Hydraulic Analysis - Pumping Mode

The Gated Pumping Station was considered to have a pumping capacity corresponding to the existing and potential future capacity of DPS 7 used for this study, as follows.

Orleans Avenue Canal Pumping Station Capacities	
Existing DPS 7 capacity	2,690 cfs
Potential DPS 7 capacity increase	700 cfs
Gated Pumping Station required capacity	**3,390 cfs**

Using the existing canal cross-section geometry and invert profile, as provided in the USACE-developed HEC-RAS model, along with the design pumping station capacities indicated in the above table, an iterative approach was used to determine the maximum canal water surface elevation at the suction side of the Gated Pump Station that would result in a canal water surface elevation at DPS 7 equal to the defined safe water elevation. The HEC-RAS model was run for several starting suction side water surface elevations and the resulting canal water surface elevation at DPS 7 was determined for each case. Based on the results of this analysis, the maximum suction side water surface elevation at the Gated Pump Structure was determined to be El. 8.3 ft, NAVD 88. The water surface profile within the canal for this flow condition is provided in Figure 5.3.2.3-2.

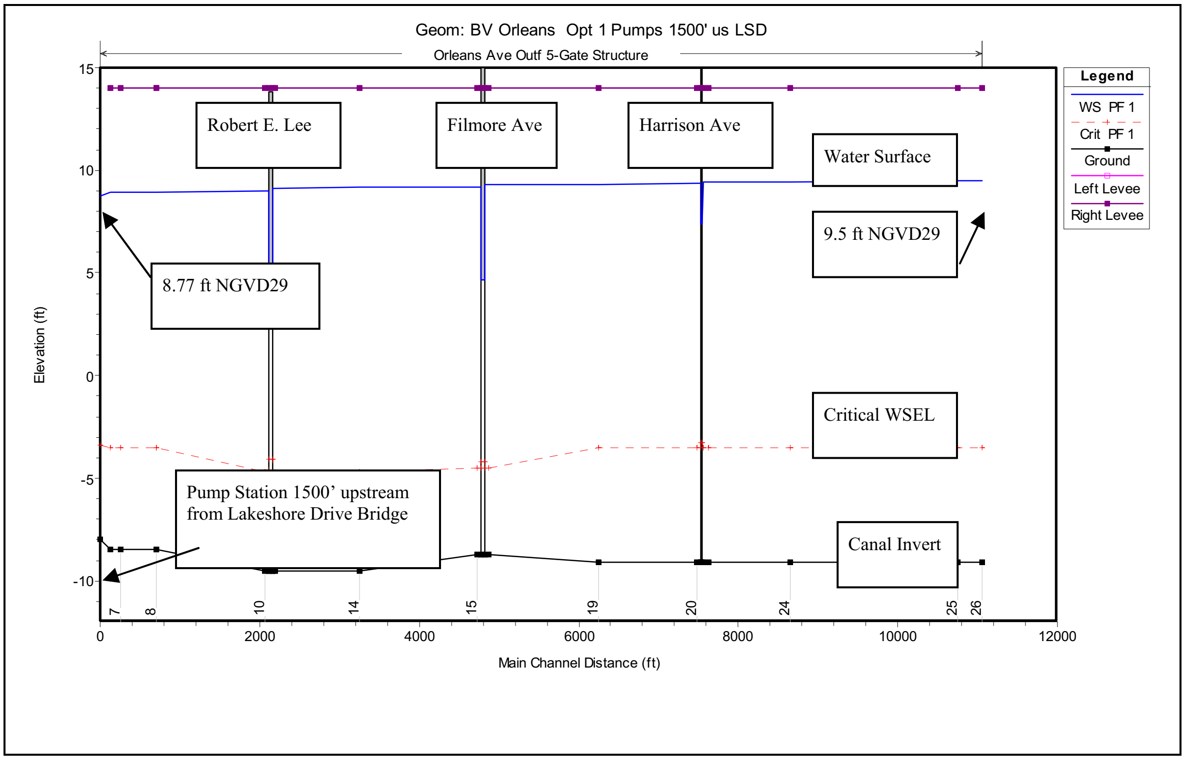


Figure 5.3.2.3-2. Water Surface Profile – Orleans Avenue Canal (Option 1)

<u>Hydraulic Analysis – Gates Open Mode</u>

The Gated Pump Station would be designed to pass canal discharges through the gate openings for combinations of Lake Pontchartrain elevation and canal discharge that do not cause the safe water elevation in the canal to be exceeded. If conditions are expected to occur that would cause the safe water elevation to be exceeded, the gates would be closed and the Gated Pump Station would be operated in pumping mode.

Using the existing canal cross-section geometry and invert profile, as provided in the USACE-developed HEC-RAS model, the canal water surface elevation at DPS 7 was determined for various combinations of Lake Pontchartrain elevation and canal discharge, as shown in Figure 5.3.2.3-3.

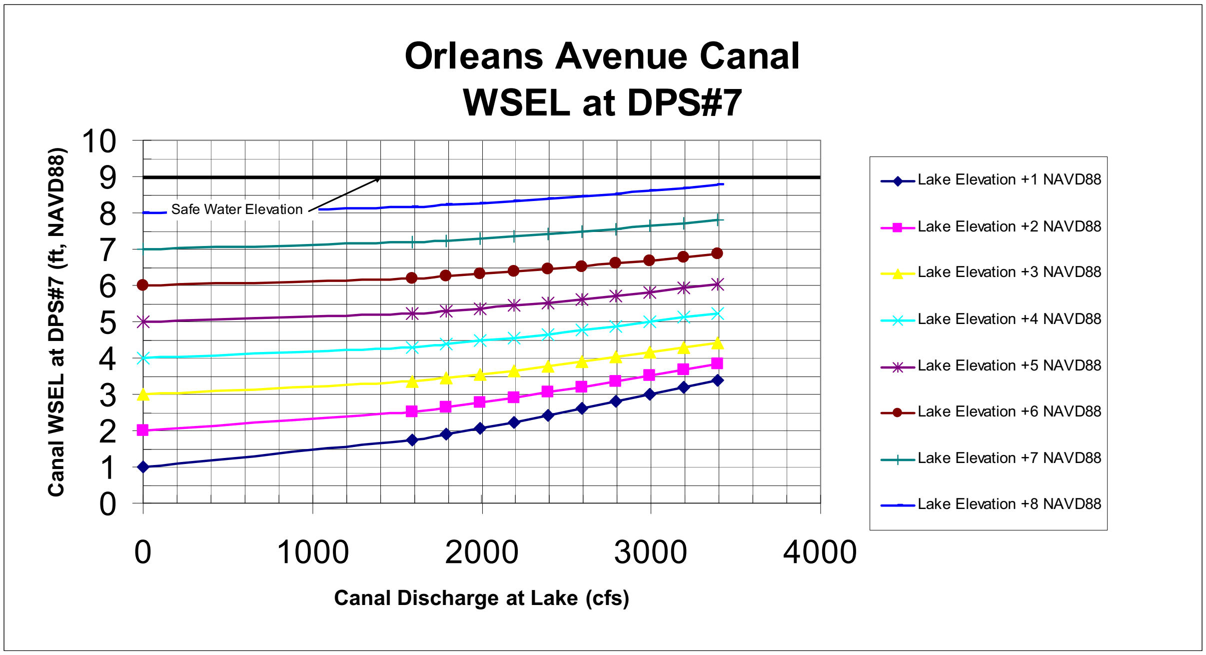


Figure 5.3.2.3-3. Water Surface Elevation at DPS 7 for Various Combinations of Lake Pontchartrain Elevation and Canal Discharge – Orleans Avenue Canal

For all combinations of Lake Pontchartrain elevation and canal discharge that fall below the safe water elevation, operation in the gates-open mode would be possible. For combinations of lake elevation and canal discharge that exceed the indicated safe water elevation, closure of the gates and operation in pumping mode would be required. Because of the relatively high safe water elevation that has been defined for the Orleans Avenue Canal, operation in gates-open mode would be possible except during significant surge conditions at Lake Pontchartrain.

5.3.2.4 Geotechnical

The typical stratification for the Orleans Canal is taken from the IPET Report, Volume V. From the top down the stratification includes; Marsh Clay, Relic Beach Sand, and Bay Sound Clay. Pleistocene sand and clay strata are below the Bay Sound Clay. A typical representation of the canal geology is shown in the figure below. The section is taken from the IPET report. Corresponding IPET Report figures that were used to develop this section are included in Appendix B for reference.

The strength and physical properties used to characterize these strata are also taken from the IPET Report, Volume V, Appendix 10. The parameters used in this study are shown in the figure above. Text from the IPET report showing where the strength data was obtained is reproduced in Appendix B to this report. The values used in this analysis are considered to be conservative and suitable for this study. See Figure 5.3.2.4-1.

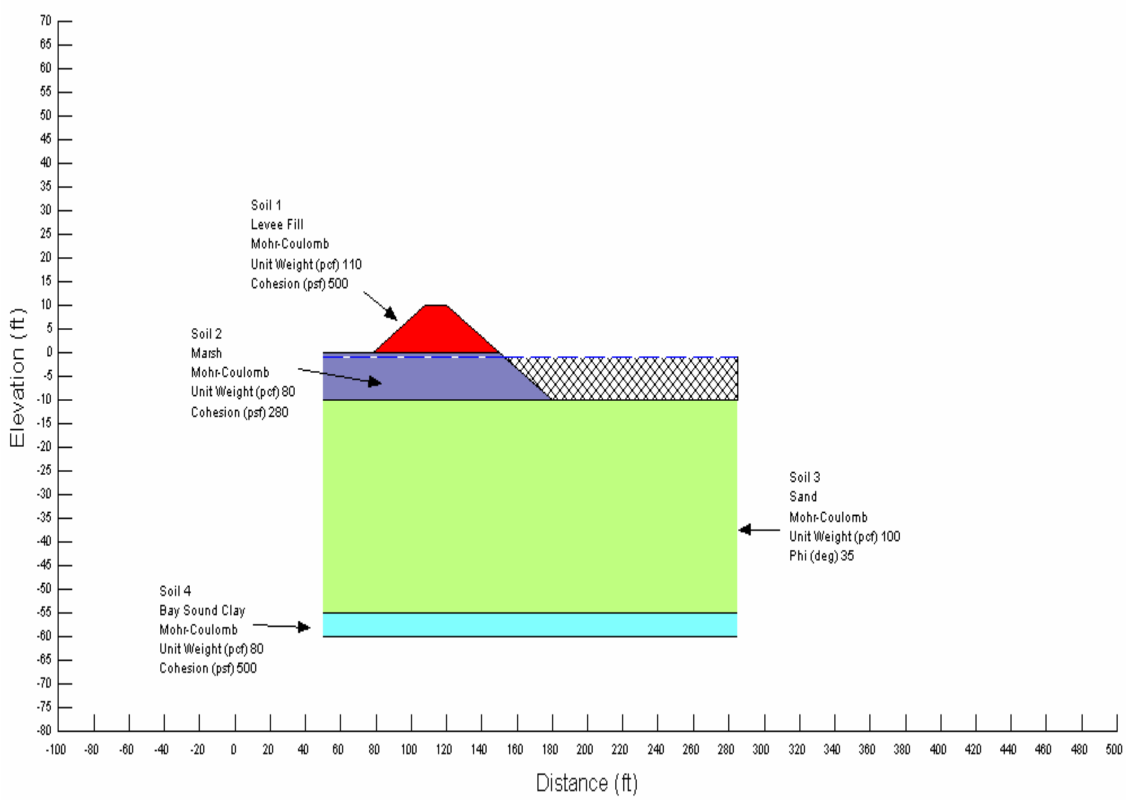


Figure 5.3.2.4-1 Orleans Canal Geology, Option 1

Canal

The canal is stable in its current state and does not require modification for Option 1. Stability analysis has been performed for the current state as a calibration of the model used in this study. The results of this calibration analysis (figures B-7 and 8, Appendix B) are essentially the same as similar analyses performed for the IPET. This ensures the channel models developed for this study are reasonable and appropriate. This calibration procedure also demonstrates the validity of the strength parameters listed in Table 3-1 above. The slope stability models developed here are used as the basis of models for evaluation of Option 2.

Pump Station Foundation

Evaluation of foundation issues for the Option 1 pump station at the Orleans Canal are under way at the time of this writing. Preliminary stability evaluation has been completed and is included as calculation file 6.1 in Appendix B. This stability analysis is a two-dimensional analysis of a typical section cut perpendicular to the long axis of the pump station. This section includes all driving and resisting loads. Analysis results are reported on a per foot basis representing the nominal one foot thickness of a two dimensional analysis. Conservative assumptions are made throughout so the two-dimensional analysis should represent conservative evaluation for this study. The issues identified for evaluation are described in the following paragraphs.

Sliding: The excavation for the intake basin will remove most of the material from the upstream side of the pump station. The pumps being considered for this study will require 14 ft of water to operate properly. The critical sliding case will be when the gates are closed and the downstream side is subjected to lake surge. Even though the lake surge is relatively short duration, the structure must be stable during the few hours it is present. The net pressure on the pump building is substantial. Several elements of the foundation design will contribute to or effect sliding resistance.

The thickness of the base slab is dictated by uplift considerations (see below). The slab will be entirely below grade so it will develop shear in the adjacent soil. At the anticipated depth of the base slab, this resistance will develop in the Relic Beach Sand. The sand has a moderate shear strength with phi = 35 degrees. However, with only four feet of embedment passive resistance alone is not expected to be adequate. Deep soil mixing is being used at the temporary protection structure at 17th Street to substantially improve the foundation soils and may be considered for this pump station as well.

The weight of the structure will generate friction on the base of the slab. Because of the high uplift pressures, net weight will be low. This means the base friction will not provide much resistance for sliding. Base friction can be improved somewhat by soil modification but will never be the controlling factor unless weight is added to the structure explicitly for this purpose. The addition of weight to improve base shear is not efficient and will not be considered for this study.

Preliminary calculations demonstrate a deficiency in sliding resistance of approximately 21 kip/ft. This deficiency must be corrected to achieve a factor of safety of 1.5 for sliding. Design elements will have to be added to provide this additional resistance.

The greatest influence on sliding resistance will likely come from the foundation piling. Including lateral resistance into the design of vertical piles is one method of providing the additional needed lateral resistance. Another viable method of providing the resistance is the addition of battered piles. Battered piles are more efficient lateral resistance elements and will likely be used. In any case, soil modification can improve this resistance and will be considered as a possible supplemental measure.

Uplift: When the gates are closed and the structure is subjected to lake surge, substantial uplift will develop on the base of the structure. The uplift is assumed to instantaneously reflect the lake surge pressures. Hydraulic conductivity of the Beach Sand is typical of sand and must be assumed to be hydraulically connected to the lake. Uplift pressure is calculated by assuming a linear variation from the lake head to the upstream head in the canal. If needed, this can be modified by installing a cut-off wall below the foundation. Cut-off walls can be effective but will be difficult to coordinate with a pile foundation anticipated for this application. For this study it will be assumed that no cut-off wall will be installed below the pump station for the purpose of limiting uplift pressures.

Conservative design resists uplift forces with dead load. At this stage of the study the base slab is being sized to provide the necessary dead load to resist uplift with a factor of safety of 1.1. This requires a base slab of 4 ft thickness with bottom at elevation -17 ft (NAVD). It is recognized that the vertical piling supporting the structure will also provide uplift resistance. For this study the piling will be considered to provide supplemental uplift resistance to raise the factor of safety above 1.5, but will not provide the principal resistance system. This is a conservative approach that ensures long term stability against uplift failures.

Underseepage: Underseepage is a potential failure mode with the combination of high heads and weak foundation soils. Since the base slab is only 4 ft thick, the length of the flow path is short and the threat of underseepage problems increases. Underseepage can result in the loss of the foundation material through piping beneath the foundation. Seepage calculations will be performed to check this failure mode. It is likely that a cut-off wall will be required.

Foundation Support: Preliminary calculations of overturning indicate the structure is not 100 percent in compression without the addition of tension elements. Tension elements with a capacity of 70 kip/ft will result in the required 100 percent base compression. The additional tension capacity is based on being able achieve a centroid of the tension elements at 2/3 the width of the base slab. Vertical tension piles will be assumed to develop the overturning stability.

The principle vertical resistance system for the pump station will likely be piling. This is common practice for the area when foundation soils are too weak for the structural loading. The strength of the soils can be improved by soil modification techniques and this will be considered. However the big issue for this structure will be settlement. Ground modification would have to extend to greater depths if it is desired to eliminate piling. For simplicity the structure will be founded on piling for this study. The concept design of the piling will ensure it will resist static vertical load when the gates are open and the eccentric loading produced by unbalanced water pressure when the gates are closed.

5.3.2.5 Structural

See the general discussion for the state of structural design in paragraph 5.1.2.5. The critical foundation design elements are discussed in paragraph 5.3.2.4 immediately above.

5.3.2.6 Mechanical

The function of the pumping station is to lift water from the canals to the lake. The principle piece of machinery to do this is the pump. High capacity, low head pumps are essentially large propellers in a tube. For this application there are two principle types of pumps defined by the orientation of the propeller. The propeller can be installed horizontally or vertical. The horizontal types are similar to the Woods Screw Pumps which are extensively used in the older pumping stations.

The horizontal pumps are installed horizontally on an operating floor above the maximum canal level. As such, the propeller is above the water surface. To operate the pump, a vacuum is used to extract air out of the pump and pump discharge piping until the propeller is submerged. Once the propeller is submerged, the pump can be turned on and the pump will complete the filling of the discharge pipeline and establishing a siphon discharge. The major advantage of the horizontal pumps is that the pump bearings and propeller are located above the canal and the easily accessible for maintenance. The pump can actually be started before the propeller becomes fully submerged permitting a low startup torque which minimizes engine generator sizing. The major disadvantage is that the pumps need to be primed by a vacuum system. Due to the volume of air needed to be evacuated, it can take 10 to 15 minutes to get the pump started.

The vertical pump has the propeller mounted down below the minimum canal water surface level. Like the horizontal screw pump vertical pumps are also used extensively in the Parish. As a result the pump design, the pump is self-priming and can start pumping within seconds of a start command which is a significant advantage in controlling pumping units when pumping stations are located in series. Also with this design, the motors are located on top of the pump and out of any danger of being damaged by flooding. The major disadvantage is that the propeller is below the water surface and that any major maintenance requires fully disassembling the pump. Also a disadvantage is that the pump starts under load and has a high startup toque which can require over sizing engine generators.

Either type pump is applicable to this pumping station. For simplicity of this analysis, only vertical pumps which provide the maximum flood protection with the elevated motors are considered in the station. During detailed design the use of vertical, horizontal, or a combination of both should be considered.

For reverse flow protection, the discharge pipe from the pumps are elevated such the invert of the pipe at the highest point is at or above the floodwall elevation so that reverse flow through the pump is not likely. The discharge pipe is then brought down below the minimum lake level forming a siphon. A siphon discharge permits recovering the energy so that when normally pumping the pumps only see the difference between the canal elevation and the lake. A vacuum breaker is provided at the highest point in the discharge pipe to permit breaking the siphon when the pump stops. For added protection, sluice gates can be added to the discharge pipe for protection against reverse flow. To minimize submergence and hydraulic losses through the station, a formed suction inlet was used in the analysis. A typical cross-section of the pumping station is attached to Appendix D – Mechanical.

The pumping units can be driven by either electric motor with electric generator backup or direct driven by engines. In the final design, there may be a combination of drivers in the pumping station. Direct driven engines are cheaper since they eliminate the engine generator and motor. Motor driven pumps are quieter and more efficient. A determining factor may well be the ability or willingness of the power company to build power lines and reserve generating capacity for pumping units which may only occasionally be operated. A detailed study should be done during design to determine the optimum combination of electric driven motors with engine generator backup or direct engine driven pumps. For the purpose of this analysis approximately 60 percent motor driven pumps and 40 percent engine driven pumps are assumed.

A polling of pumping manufacturers indicated that the maximum practical size of pumping units is roughly 1,000 cfs. This is limited by the physical size of the equipment and the ability to move the equipment along major roadways. 1,000 cfs also matches up with the largest pumping stations in the major feeder pumping stations so was chosen as the main pumps in the new pumping stations. The existing pumping stations also have a number of smaller pumps. Smaller pumps permit pumping lower flows without having frequent starts and stops and also provided the ability to match flows when pumping stations operate in series. Therefore combinations of 1,000, 500, and 250 cfs pumps were selected at as the primary capacities. Using only three sizes will permit the sharing of parts between the pumping stations.

For Option 1, the screens are optional as there is no un-screened inflow to the canal downstream of the major pumping stations however screens ahead of the pumping station inlets will smooth the inflow to the pump and have a hydraulic function. Based on the length of canal and the potential for additional debris to enter canal, a trash rack system is included in the costing.

There are a number of additional mechanical systems required for operation of the pumping station. All major pumps require a clean source of water for bearing lubrication. This can be from the water system. However, based on experience during Katrina in which the water system failed, a secondary source of water should be provided. There are two sources available, either canal water or well water. Because the water to the pumps needs to be of high quality, well water is being considered for the pumping station. Canal water can be used but requires a high level of treatment to remove abrasives. Because of the size of the equipment in the facility, the facility should include overhead crane, lay down space, truck loading access, and workshop areas. Whether motor driven or engine pumps are used, there will be significant opening for ventilations. All inlet air vents should be shrouded to inhibit the entry of wind blown water.

Engine driven pumps have additional mechanical considerations including engine and gear reducer cooling systems, fuel system, starting air systems, lubricating oil and waste oil systems. In addition, engines require both exhaust air and combustion air systems.

Pumping station hydraulics are critical to successful operation of the pumping station to achieve maximum hydraulic performance. A physical model test of the pumping station including canal entrance, screens, and pump inlet, and discharge siphon pipe must be conducted as a follow-on effort to ensure correct sizing and configuration.

For the purpose of this study, the following combination of vertical pumping unit capacities was chosen. In addition, a combination of direct drive diesel engines and diesel generators are assumed to provide an approximate 60 percent electric motor drive and 40 percent diesel engine drive ratio.

Pump Driver Type		
Pump Number	**Pump Capacity (cfs)**	**Driver Type**
1	1000	Direct Drive Engine
2	1000	Direct Drive Engine
3	500	Motor on Grid or Generator

Pump Driver Type		
Pump Number	**Pump Capacity (cfs)**	**Driver Type**
4	500	Motor on Grid or Generator
5	250	Motor on Grid or Generator
6	250	Motor on Grid or Generator

Fuel storage capacity for the pump station was selected at 4 days of full pumping capacity. Based on this duration and usage rate, the anticipated fuel storage required is slightly over 23,000 gallons. Assuming standard 12,000 gallon double wall fuel storage tanks, a minimum of 2 tanks will be required.

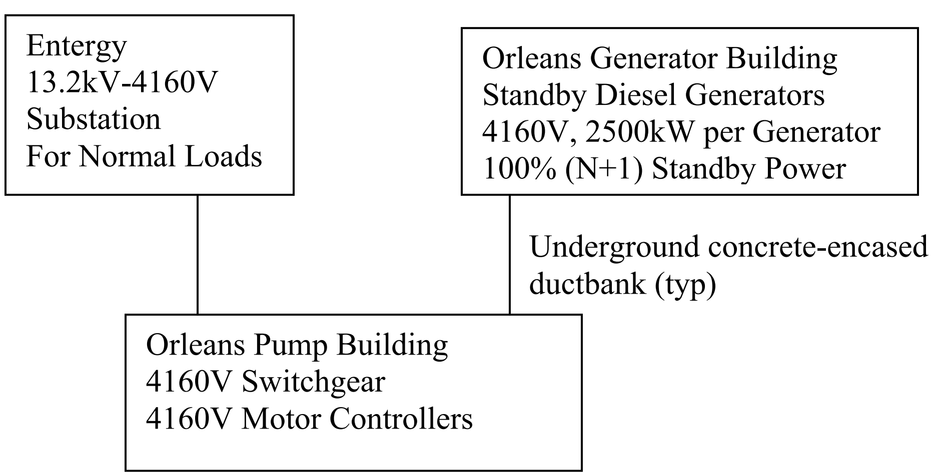
The estimated pump ratings are as follows:

Pump Rating			
Capacity, cfs	1,000	500	250
Bowl Head, ft	13	13	13
Pump Speed	162	227	321
Engine Rating, bhp	2100	---	---
Motor rating, hp	1750	900	500

In the Mechanical Appendix D are representative pump performance curves submitted by the manufacturers. The curves are presented as typical curves only as the required pump rating have evolved during the study and the pump ratings on the curves will differ slightly from the latest hydraulic requirements.

5.3.2.7 Electrical

Option 1 uses existing pump station DPS7 along with a new pump station at Orleans Avenue. Utility power, from Entergy, will only be supplied for the normal pump loads (see the Pump Driver Schedule below for pump utilization). The incoming utility service will not be sized to accommodate the storm event pump loads.



One hundred percent of the storm event electric-driven pump loads will be supplied from standby generators dedicated to the Orleans Avenue Pump Station. The standby generators will utilize an N+1 design such that if a generator goes off line or one is down for maintenance, the full pump station load will still be supplied by standby power. The utility service, standby generators, and pump motors will all operate at 4160 volts. All electrical distribution circuits will be routed underground in concrete-encased ductbank. Below is a table to show the pump driver schedule.

Option 1 - Orleans Canal – Pump Driver Schedule						
Pump cfs	**Driver bhp**	**Motor hp Nameplate**	**Motor hp Load**	**Source**	**Utilization**	
					Grid	**Ind**
1000	1241			Engine		1%
1000	1241			Engine		1%
500	526	1500	900	Grid/Generator	1%	1%
500	526	1500	900	Grid/Generator	5%	1%
250	263	750	450	Grid/Generator	50%	1%
150	158	500	300	Grid/Generator	100%	1%
3400			**2550**	**Totals**		

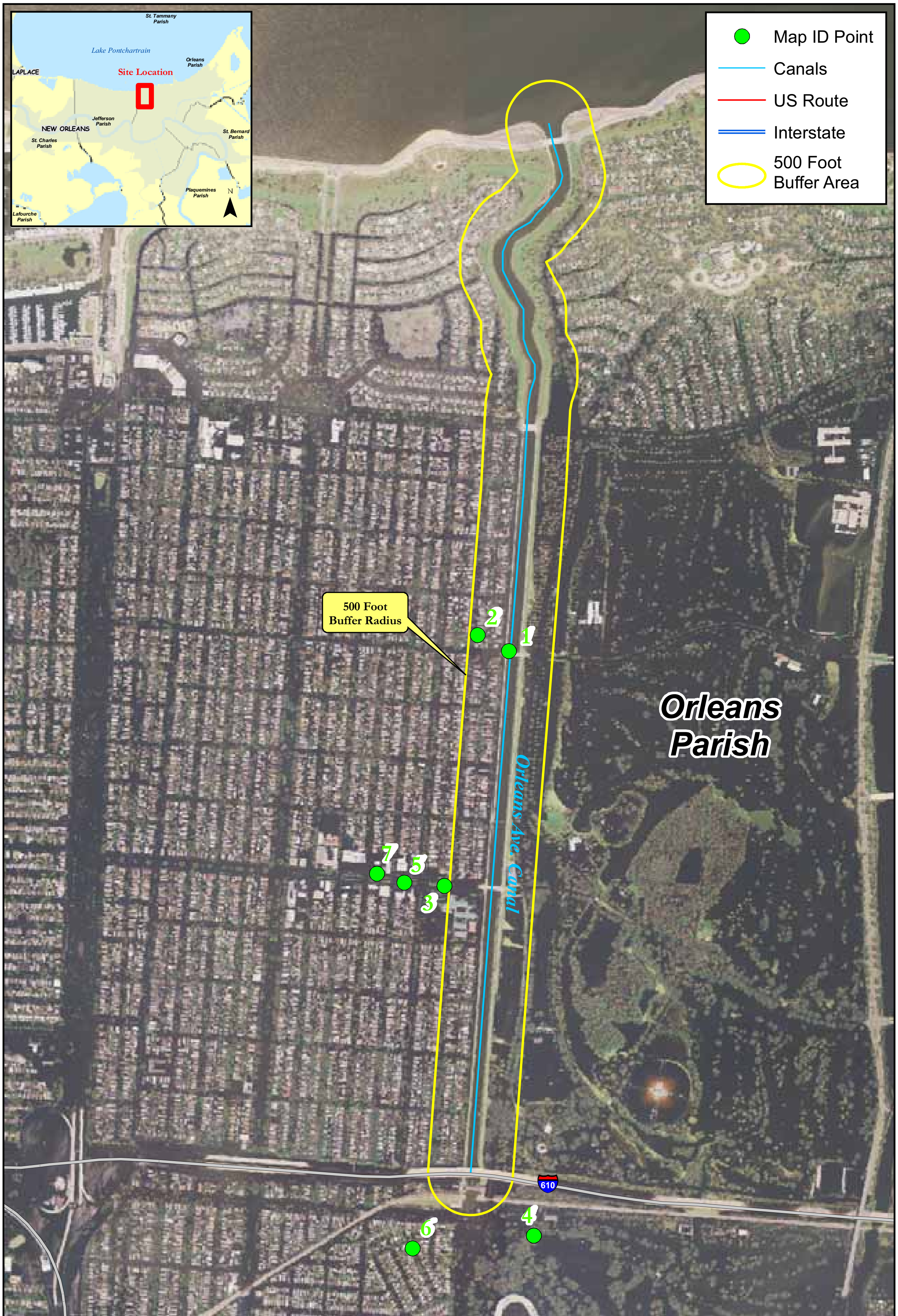
Based on the pump driver information above, the next table below indicates the electrical equipment sizes used for this option. See the appendices for all cost information.

Option 1 – Orleans Canal Pump Station - Electrical	
Electrical Equipment	**Quantity/Capacity**
Utility Substation	1 – 5000kVA
Generators	3 – 2500kW
Generator Bldg Size	94' x 80'
Pump Bldg Switchgear	2 – 600A
Total Pump Station Load	2.7 MVA, 372A
Pump Station Load on Utility	2.7 MVA, 372A

5.3.2.8 Environmental

Environmental Site Assessment (ESA)

An environmental database search report of the area within a 0.25-mile radius of the canal route from the existing pump station to the mouth of the canal (i.e., 0.5 mile total width of the search area across the canal) was obtained to identify sites, incidents, conditions, etc. within the search corridor that may contain or formerly contained hazardous, toxic or radioactive waste (HTRW). These reports were utilized during in field observations to verify sites within the potential construction zones. Figure 5.3.2.8-1 presents the mapped sites within the 0.25-mile



Orleans Avenue Canal Detail Map ID Points

Orleans Parish, Louisiana

Note: Background image is post-Katrina USACE Aerial Photograph (September, 2005). Map ID Points provided by Banks Information Solutions, Inc. Other data not derived from registered survey and should be considered approximate.

0 500 1,000 2,000 Feet

1:15,000

GEC

Figure 5.3.2.8-1

Project # 60321CZ02 July 2006

radius around the Orleans Avenue Canal that are tracked on various environmental databases maintained by either the U.S. Environmental Protection Agency (USEPA) or the Louisiana Department of Environmental Quality (LDEQ).

Although the database search was requested for the standard radius of 0.25 mile (the search radius specified under American Society for Testing and Materials (ASTM) Standard E1527-05), particular interest was given the 500-foot (ft.) radius around the canal as a buffer zone for construction activities to take place in connection with the proposed project. Figure 5.3.2.8-1 provides the mapped 500-ft. radius around the Orleans Avenue Canal. Located within the 500-ft. radius are two sites, map ID numbers 1 and 2. These sites are identified below along with their measured distance from the centerline of the canal:

Map ID	Site Name	Address	Distance
1	City Park Golf Course	1040 Filmore Avenue	52 ft
2	Unidentified	6725 General Haig Street	422 ft

Site numbers 1 and 2 are on the database for being registered hazardous waste generators under RCRA regulations (the RCRAGN database). Because no problems at these sites have been reported, they are not considered a potential problem for the proposed project at this time. Five unmappable sites were reported in the Orleans Avenue Canal database search. However, these sites were evaluated for their potential to pose a threat to the proposed project. In consideration of the database on which they are listed, type of facility, and whatever location information is given, none of these seven unmappable sites are considered a significant concern for the construction of the proposed project.

A field site reconnaissance was conducted to determine whether any sites not identified through the database search process existed within the immediate project area that could potentially affect project design plans. The area surrounding the Orleans Avenue Canal is almost entirely residential or City Park property. No visible signs were noted that would indicate the presence of HTRW in quantities that would warrant additional investigation.

The addition of diesel powered pumps at the control structure near the mouth of the canal would constitute new sources of air emissions and noise for area residents. The air emissions would require permitting through the LDEQ and noise mitigation would likely be required to be incorporated into the design plans for the facility. These issues would be addressed following final selection of pump and power design.

Because there is very limited recreational use of the Orleans Avenue Canal, for the most part recreational constraints would not be an issue. There is some recreational fishing at or near the mouth of the canal and this may be, at least temporarily disrupted during the construction of the control structure, but the numbers of fishermen appear to be small and intermittent.

The entire area of New Orleans and Lake Pontchartrain is within the Louisiana Coastal Zone, therefore a Coastal Zone Consistency determination will have to be conducted and submitted to the Louisiana Department of Natural Resources for concurrence.

Sediments

Because the discussion for sediment quality study covering all three canals has been discussed in some detail under Section 5.1.2.8, please see that section (sediment) for overall study information. See Figure 5.3.2.8-2 for the locations of samples taken in the Orleans Avenue Canal. The following is sediment quality information specific to the Orleans Avenue Canal:

Orleans Avenue Canal Specific Findings

Sample ORLEANS 1 exceeds RECAP screening standards for lead by 70 mg/kg (70 percent). TPH-Diesel Range Organics (DRO) and Oil Range Organics (ORO) exceed RECAP standards by 205 mg/kg (315 percent) and 210 mg/kg (117 percent) in sample ORLEANS 2 and by 155 mg/kg (238 percent) and 111 mg/kg (62 percent) in sample ORLEANS 3. Volatile organic blank contamination was noted in the trip blank for Orleans Avenue Canal. Falsely elevated concentrations of trichloroethene are noted in the volatile organics analysis for all three composite samples.

None of the analyzed compounds that are regulated by RCRA are present in the Orleans Avenue Canal TCLP samples in concentrations exceeding RCRA standards. TPH-DRO and TPH-ORO were detected in the TCLP leachate in the ORLEANS 1 sample, and TPH-DRO was detected in the ORLEANS 2 and ORLEANS 3 samples. TPH-GRO was also detected in all three samples, possibly due to volatile organic blank contamination.

For additional information, see Final Report, Orleans Avenue, London Avenue and 17th Street Outfall Canals, Certified Industrial Hygienist Investigation, Orleans Parish, Louisiana, March 2006, U. S. Army Corps of Engineers, New Orleans District, New Orleans, Louisiana.

Wetlands

National Wetland Inventory (NWI) maps were reviewed for jurisdictional wetlands in the proposed project area. See Figure 5.1.2.8-3 for a presentation of the mapped jurisdictional wetlands in the area of all three canals. The NWI maps show no mapped wetlands within the potential construction zone for the Orleans Avenue Canal. However, field reconnaissance observations revealed a large amount of bank area inside the levee walls on both banks of the canal near the mouth of the canal north of Robert E. Lee Blvd. that appear to be potential wetlands. Changes in tidal influence would not likely significantly affect the wetlands along the banks of the Orleans Avenue Canal as there does not presently appear to be any tidal influence on these wetland areas. Permits from the USACE would be required prior to disturbance of these areas.

Orleans Avenue Canal
CIH Investigation
Orleans Parish, Louisiana

Note: Post-Katirna Color Aerial Photography provided by 3001, Inc. through the USACE. Other data not derived from registered survey and should be considered approximate.

0 250 500 Feet

1:2,500

Sample Point
Transects
Canal Closure Structure

Figure 5.3.2.8-2

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Protected Species

See discussion in Section 5.1.2.8.

Cultural Resources

See discussion in Section 5.1.2.8.

5.3.2.9 Constructability

The conceptual designs used throughout this project are constructible using conventional techniques. For Orleans Canal, Option 1 there are no channel modifications. All construction is focused on the pump station. Location Alternate A reuses the gate which is part of the temporary structure. For location alternates B and C, the temporary gate and structure would be demolished after the new pump station becomes operational. Two major approaches can be taken to construct the new pump station in the Orleans Canal.

The first construction concept is based on construction of a sheet pile cofferdam enclosing the entire pump station. The cofferdam enables the contractor to construct the pump station in the dry. To accomplish this, a bypass channel must be constructed for alternate location B. Location alternates A and C are constructed entirely outside the existing canal and do not require a bypass channel. Figures 5.3.2.9-1, 5.3.2.9-2 and 5.3.2.9-3 on the following pages present the cofferdam and bypass concepts.

The bypass channel for Alternate B is formed with parallel sheet pile walls and is sized to pass the flows for which the temporary gate/pump structure is designed. A temporary bridge is required for Lake Shore Drive. This could be a precast structure set over the bypass channel. Once the pump station is complete the cofferdam is removed and the bypass channel is dammed and filled.

Construction of the concrete liner walls for the intake and discharge channel transitions can be constructed in the wet at the same time the pump station is being built for all three alternates.

As can be seen on the figure there are some problems with this concept for location Alternate B. At this site the best location for the bypass channel places it near residential property and creates a breach in the existing levee. The sheet pile forming the west side of the channel would have to be configured to provide flood protection to the adjacent property. The contractor would also have to provide a temporary bridge for Lake Shore Drive. The cost of constructing the bypass channel is significant for this location alternate.

The second concept for location Alternate B is to construct the pump station in two parts. The substructure for one half would be constructed first and would house the gates. A cofferdam would be set around just the first half, allowing the existing canal to pass flow with the constriction imposed by the cofferdam. The gates would be set in the open position. Upon completion of the gate substructure, the cofferdam would be moved to the other half of the

CONCEPTUAL DESIGN STUDY
PERMANENT FLOOD GATES AND PUMP STATION
ORLEANS AVENUE – OPTION 1A
CONSTRUCTION CONCEPT
FIGURE 5.3.2.9-1

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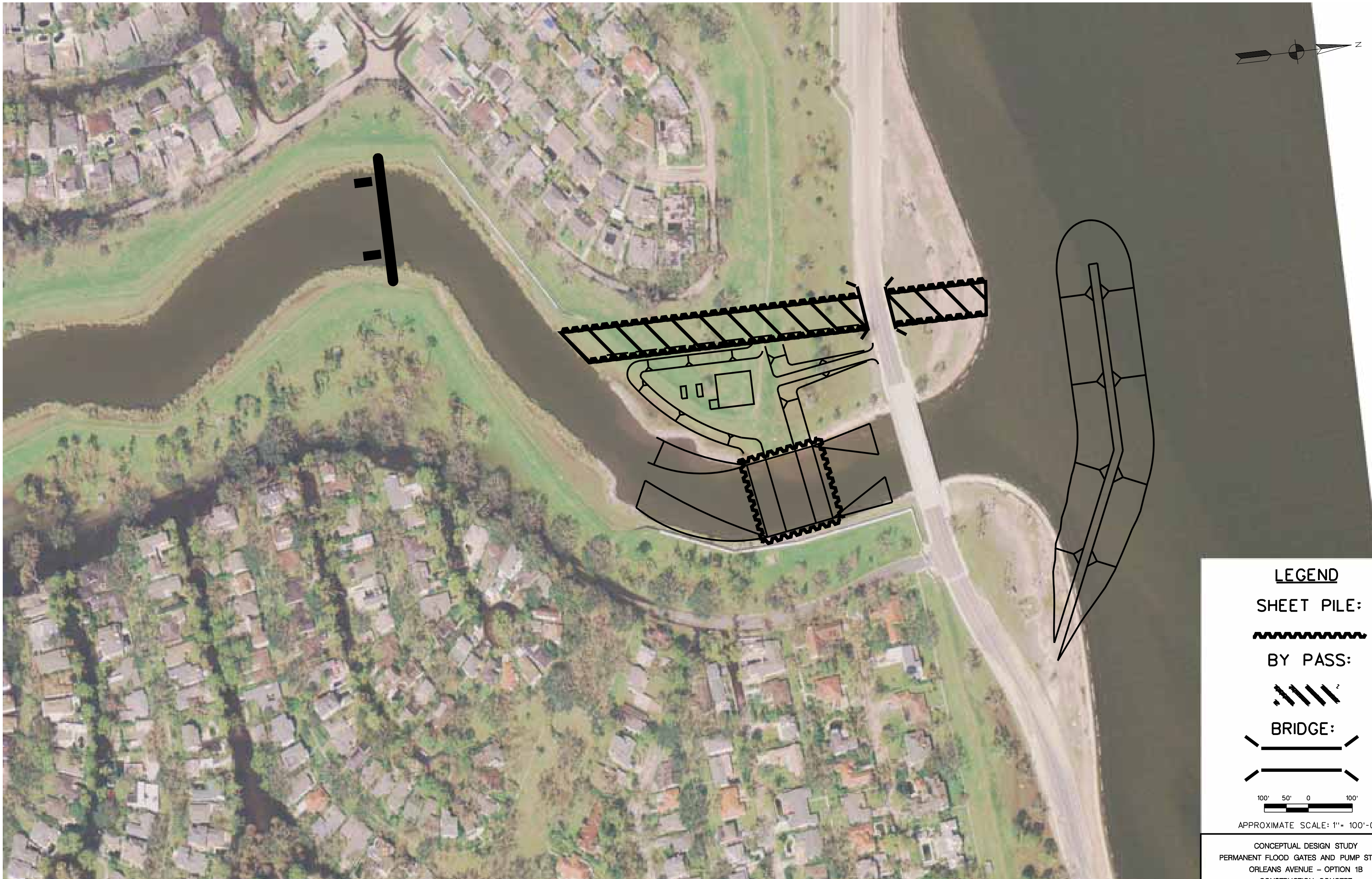

CONCEPTUAL DESIGN STUDY
PERMANENT FLOOD GATES AND PUMP STATION
ORLEANS AVENUE – OPTION 1B
CONSTRUCTION CONCEPT
FIGURE 5.3.2.9-2

CONCEPTUAL DESIGN STUDY
PERMANENT FLOOD GATES AND PUMP STATION
ORLEANS AVENUE – OPTION 1C
CONSTRUCTION CONCEPT
FIGURE 5.3.2.9-3

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structure. Flow would be redirected to pass through the new gates. The second half of the substructure could then be constructed followed by the complete super structure.

The second concept also has some disadvantages. The construction of the pump station in two halves is much more complicated than building it all as a single unit. The design would have to account for the connection of the two halves. Coordination and sequencing would be significant issues. The cost of constructing the pump station in two halves would be higher than being able to construct it in as a unified whole.

The bypass concept is illustrated here for Alternate B and used as a basis for cost estimating to be consistent with the other eight canal options. The additional cost of the bypass canal may be more expensive than the incremental cost for constructing the pump station in two halves. The design build contractor would be permitted chose either of these concepts or innovate a third concept of his own if it would reduce project cost.

Location alternates B and C also require the construction of a breakwater. The breakwater could be constructed at any time but must be complete before putting the new pump station in operation. It would be best to complete the breakwater early in the construction because of the additional protection it provides.

5.4 Option 2 – Orleans Canal

5.4.1 Alternative Approaches

Three location alternatives were considered for Option 2 on the Orleans canal. The merits of each were evaluated and discussed in detail in the Civil/Site section. For the purposes of this study, Alternative A was chosen as the location to base costing and other engineering considerations.

5.4.2 Engineering Considerations

5.4.2.1 Civil/Site

The Orleans Avenue Pump Station for Option 2 is anticipated to be 130 feet long for Layout Alternatives A, B and C. The pump station width is 155 feet wide in every case. The total station width includes a 45 foot inlet works including trash screens, a 70 foot pump station building housing pumps and motors, and a 40 foot outlet works. Finish grade for the Generator and Tank Farm Complex is always approximately +16.0 elevation.

- The **Alternative A Pump Station Layout** for Option 2 is as shown in Exhibit 5.4.1.A, Appendix C.

<u>*General Location and Description*</u> – As under Option 1 in this layout, the pump station is located on the east canal bank, immediately adjacent to the temporary gate structure. The Temporary gate structure is not necessary under Option 2. However, it may remain in place with gates permanently closed to serve as a levee closure, as

assumed under this study. It may also be removed and replaced with an earthen canal closure for improved aesthetics, all at additional cost.

This alternative also provides for convenient connection of existing levees to the new pump station structure. Finally, the inland pump station location shields the pump station from lake surge effects. That is, no breakwater structure is required.

Right-of-Way Acquisition - Permanent right-of-way acquisition and temporary construction easement acquisition are substantially unchanged from Option 1.

Demolitions – Demolitions under this Option 2 are unchanged from those described in Option 1, including the potential relocation of the temporary power plant.

Earthwork - Unlike Option 1, which requires localized earthwork only at the pump station facility, Option 2 requires significant canal excavation along the entirety of the Orleans Avenue Canal. The basis of this anticipated canal excavation was developed and determined based on the results of the canal hydraulic analysis described in Section 5.4.2.3 and the geotechnical slope stability analysis described under Section 5.4.2.4. Note that the anticipated canal improvement cross-section consists of providing an added rectangular section in the existing canal invert, either as sheet pile walls or a concrete "U" channel section, as shown elsewhere. This approach maintains canal construction within the existing canal right-of-way. Laid back earth slopes (trapezoidal canal cross-section) have also been considered for required canal improvements. As described in Section 5.4.2.4, the acceptable stable slope for the Orleans Avenue Canal is 4:1. Due to that extremely flat slope, and the correspondingly large volumes of right-of-way acquisition and channel excavation that would result, laid back slopes are not recommended for further consideration. Further analysis of the effects on right-of-way and excavation quantities brought about by laid back slopes is included in Appendix C.

Channel Transitions - Channel transitions for this Option 2 layout are similar to those described under Option 1, simply increasing in size due to the increased channel depth.

Erosion Protection – Option 2 erosion protection armoring is unchanged from Option 1 requirements.

Generator Building and Tank Farm Complex - The Generator Building and Tank Farm Complex represent an area of some change from Option 1 to Option 2, due to the substantially increased head the pump station must overcome. The Generator Building is anticipated to be increase to 112 by 80 feet, and each of five 12,000 gallon fuel tank pads is anticipated to be approximately 12 by 28 feet in size. According to NFPA 30, 2003 Edition, diesel fuel is a Class II Combustible Liquid. As such, tanks require a five foot clear distance from the public way or important buildings on the site. A 20 by 40 foot electrical substation will also be included. The complex also includes limited parking, general staging and storage space, all concrete paved, and including sidewalk and local site storm drainage features. This general space is somewhat limited, in order to maintain right-of-way acquisition to undeveloped properties. Utilities will include potable water service, sanitary sewer and natural gas, all connected to the station from existing utilities available within several hundred feet of the proposed site. Although not a significant cost item, the provision of these utilities when the station is operating under total self-sufficiency is anticipated. The paved complex area is also anticipated to be enclosed by a chain link security fence, with minor landscaping improvements. All non-paved areas will be seeded to re-establish healthy turf for aesthetics and erosion control.

Summary – Similar to Option 1, this layout is attractive for its protection of the pump station from lake surge effects, resulting in no need for a major breakwater structure. The location requires right-of-way acquisition in a residential area, but acquisition appears to be exclusively undeveloped property. Further, it is attractive since no significant construction sequencing is required to maintain canal flow during the construction duration, given the conversion of the temporary gate structure to permanent service. The potential relocation of the existing temporary power plant, depending on the final precise location of the pump station, is unfortunate but manageable.

- The **Alternative B Pump Station Layout** for Option 2 is as shown in Exhibit 5.4.1.B, Appendix C. As stated for the Option 1 Layout, this pump station location is not recommended for further consideration.

- The **Alternative C Pump Station Layout** for Option 2 is as shown in Exhibit 5.4.1.C, Appendix C.

General Location and Description - The horizontal location of the pump station is identical to Option 1 for Alternative C. Changes from Option 1 to this Option 2 layout all result from the effects of the deeper canal and correspondingly deeper pump station inlet elevation, as described below.

Right-of-Way Acquisition - Right-of-way acquisition under this Option 2 is largely unchanged from Option 1, due to the minimal overall need for shore-based support structures and the location of the pump station within Lake Pontchartrain.

Demolition – Demolitions under this Option 2 are unchanged from those described in Option 1.

Earthwork - Unlike Option 1, which requires localized earthwork only at the pump station facility, Option 2 requires significant canal excavation along the entire length of the Orleans Avenue Canal. The basis of this anticipated canal excavation is determined based on the results of the canal hydraulic analysis described in Section 5.4.2.3 and the geotechnical slope stability analysis described under Section 5.4.2.4. Note that the anticipated canal improvement cross-section consists of providing an added rectangular section in the existing canal invert, either as sheet pile walls or a concrete "U" channel section, as shown elsewhere. This approach maintains canal construction within the existing canal right-of-way. Similar to Layout Alternative A above, laid back slopes on the Orleans Avenue Canal are not recommended for further consideration, due to the extremely flat side-slope required (4:1), and the correspondingly large volumes of right-of-way acquisition and channel excavation that would result, Further analysis of the effects on right-of-way and excavation quantities brought about by laid back slopes is included in Appendix C.

Channel Transitions – As in Option 1, counterforted retaining wall channel transitions are required immediately upstream of the pump station on both banks. Under Option 2, their size is increased due to the deeper channel invert elevation entering the pump station. There is no change to the downstream training wall transitions.

Erosion Protection – Erosion protection requirements are largely unchanged from Option 1 under this Option 2.

Generator Building and Tank Farm Complex - The Generator Building and Tank Farm Complex represent an area of some change from Option 1 to Option 2, due to the substantially increased head the pump station must overcome. The Generator Building is anticipated to increase to 112 by 80 feet, and each of five 12,000 gallon fuel tank pads is anticipated to be approximately 12 by 28 feet in size. A 20 by 40 foot electrical substation is also included. As under Option 1, the complex includes parking, general staging and storage space, all concrete paved, including sidewalk and local site storm drainage features. Utilities will include potable water service, sanitary sewer and natural gas, all connected to the station from existing utilities available over 1,000 feet from the proposed site. The paved complex area is also anticipated to be enclosed by a chain link security fence, with minor landscaping improvements. All non-paved areas will be seeded to re-establish healthy turf for aesthetics and erosion control.

Summary – Like Option 1, this layout is attractive for its relatively minimal right-of-way acquisition requirements. Constructability is a mixture of positives and negatives. Constructing the pump station building in one phase is positive; however, construction in the lake offers other complications that are costly to overcome. Further, a major breakwater structure and significant plant armoring is required.

- **Existing Pump Station Demolition and Bypass.** Option 2 requires the abandonment of an historic inland pump station. In general, the historic elements of the pump station is to be preserved, while allowing the required canal flows to bypass the historic elements that remain, flowing into the newly degraded canals downstream of each existing pump station. Bypass of this pump station requires further development based on identification of the historic elements of the structure. In general, the demolition and bypass will be conducted in similar fashion as described for 17th Street canal.

5.4.2.2 Bridges and Utilities

Bridges

As discussed herein, the existing bottom of the canal is approximately -10 (NAVD88) and the new bottom of channel for Option 2 being approximately -20 (NAVD88). It is the new channel section that may significantly affect some of the bridges. Because of the lowering of the canal, weak soil conditions, and constructability under these bridges, significant impacts and cost are expected. Further investigation should be made and preliminary design developed for bridge modifications as replacement of these structures would be very expensive. For purposes of this report concrete box sections of equal hydraulic capacity are sunk between the support bents. This technique is constructable, accommodates the slope stability, while providing the required hydraulic capacity. For purposes of this report modification of the bridges under Option 2 utilizes the box culvert technique.

The Interstate 610 Bridge located just downstream of Pump Station No. 7 is virtually unaffected by Option 2 because of the span lengths, foundation design and roadway geometrics.

For the Harrison Avenue Bridge, Robert E. Lee Boulevard Bridge and the Filmore Avenue Bridge the Option 2 canal section will require bridge modification as described above. It is noted, that should it later be discovered these structures require replacement impacts to the motoring public and local residents and businesses affected by additional right-of-way along the roadway.

As previously stated the Lakeshore Drive Bridge is affected by the pump station site location as well as the Option 2 canal section. For pump station layout alternates A and B, the Lakeshore Bridge is unaffected since the pump station being located upstream from the bridge. Site location C will require the bridge to be replaced.

Utilities

The utilities studied in Option 2 are underground or pile supported water, sewer, drainage, electric (transmission and primary), telephone cables, fiber optic cables, and gas. Entergy supplies electricity in the vicinity of this canal, however, their overhead crossing primary and secondary lines entirely span the floodwalls. BellSouth owns the telephone and fiber optic cables in the vicinity of this project. In Option 2, the existing utilities affected by construction in the vicinity of the Orleans Avenue Canal are those utilities impacted by deepening the canal within the floodwalls from Pump Station No. 7 to the new pump station in the vicinity of Lake Pontchartrain.

Although the Orleans Avenue Canal is located completely within the boundaries of Orleans Parish, there are very few utilities that cross this canal. Other than the aforementioned 8” diameter water main on the Lakeshore Drive Bridge, there is a 30” diameter water main which runs down the length of Bragg Street, crosses the canal and continues in to City Park. This water line crosses the canal above grade on three pile supports within the canal.

Each of the existing pile supports should be replaced by driving two longer piles and placing a support cap underneath the pipeline between the two piles adjacent to the existing pile supports. The existing supports can then be cut off below the new channel grade or removed. In this manner, the water line need not be replaced. Estimated Bragg Street water adjustment cost equals $50,000.

The other known crossing utilities are located on the north side of the Robert E. Lee Boulevard Bridge. A 12" diameter water line is attached to the bridge deck estimated cost $10,400. Enclosed in a steel conduit and attached to the bridge deck are 144k and 36k fiber optic cables. Estimated replacement cost equals $150,000. Approximately 20' north of the bridge are buried crossings of the following utilities:

10' x 7' box drainage siphon
24" diameter gravity sewer
8" diameter high pressure gas - $56,000
Primary electric feed - $37,500

These utilities will need to be replaced after the canal is deepened.

The 10' x 7' siphon drainage box will need to be replaced. However, if the limits of the canal deepening and excavation are confined to within the floodwalls as proposed, it should not be necessary to replace any siphon pumps, which may exist immediately outside of the floodwalls. Estimated replacement cost is $210,000.

The flow line of the existing buried 24" diameter gravity sewer is unknown, but it is almost certain that it is not buried 10' or more. Therefore, a new duplex submersible sewer pump station outside of the floodwalls on the east side of the canal will be needed. A new smaller diameter force main from the new sewer pump station, replacing the 24" gravity sewer under the canal, and discharging into the existing manhole on the west side of the canal will be required. Estimated cost equals $112,000. It is estimated that the new sewer pump station and force main will cost approximately $300,000.

5.4.2.3 Hydraulic

For this option, the Replacement Pump Station would replace existing DPS 7 that discharges into the canal. The existing facility would be modified as necessary so that drainage would bypass the pump station and be conveyed within the Orleans Avenue Canal to the Replacement Pump Station. The required flowline elevation within the canal would be much lower than for existing conditions, and significant modifications to the canal would be required to accommodate the lowered flowline. These modifications would generally involve lowering the canal invert elevation with a modified cross-section to allow the design canal discharge to flow by gravity between the location of DPS 7 and the Replacement Pump Station.

The maximum allowable upstream water surface elevation within the canal corresponds to the maximum allowable water surface elevation on the suction side of DPS 7. For purposes of this study, a maximum suction side elevation at DPS 7 of -9.4 ft. NAVD88 is used. This corresponds to the current “pumps on” operating condition at DPS 7 and is therefore conservative with respect to maximum allowable canal water elevation.

Hydraulic Analysis

The hydraulic analysis performed for this Option 2 was similar to the analysis for Option 1 – Pumping Mode. The USACE-developed HEC-RAS hydraulic model was used, with inflow to the canal representing the bypass flow at DPS 7. Total design canal discharge at the Replacement Pump Station is 3,390 cfs, which includes the potential capacity increase of 700 cfs at DPS 7. The hydraulic model was developed based on NGVD29 datum; therefore, subtraction of 0.5 feet from model elevations is necessary for conversion to NAVD88 datum.

A starting water surface elevation at the Replacement Pump Station of -13.0 ft. NAVD88 was selected for use. The modified canal section was considered to be a concrete-lined, rectangular cross-section with a bottom width of 75 feet and vertical side walls. Inspection of the existing canal invert profile indicates the canal invert does not vary significantly, with invert elevation ranging from approximately -9.0 ft. to -10.0 ft., NAVD88. The modified canal invert profile was considered to be horizontal (constant elevation) between the Replacement Pump Station and DPS 6. This invert profile configuration approximates the profile configuration of the existing canal.

Using the selected starting water surface elevation and the rectangular canal cross-section, an iterative approach was used to determine the canal invert elevation that would result in a maximum canal water surface elevation at DPS 7 equal to the maximum allowable suction side elevation at this pump station. The HEC-RAS model was run for several canal invert elevations, and the resulting canal water surface elevation at DPS 7 was determined for each case. Based on the results of this analysis, a canal invert elevation of -19.5 ft., NAVD88 was determined to be required. The water surface profile within the canal for this flow condition is provided in Figure 5.4.2.3-1.

Numerous combinations of starting water surface elevation, canal cross-section geometry, and canal invert profile would result in the desired upstream canal water surface elevation. Consideration of such alternatives would be appropriate as part of an overall project evaluation comparing capital costs and annual operating costs, however this type of alternatives evaluation is not included in this study. The canal cross-section geometry that was selected represents a reasonable canal configuration for the given criteria.

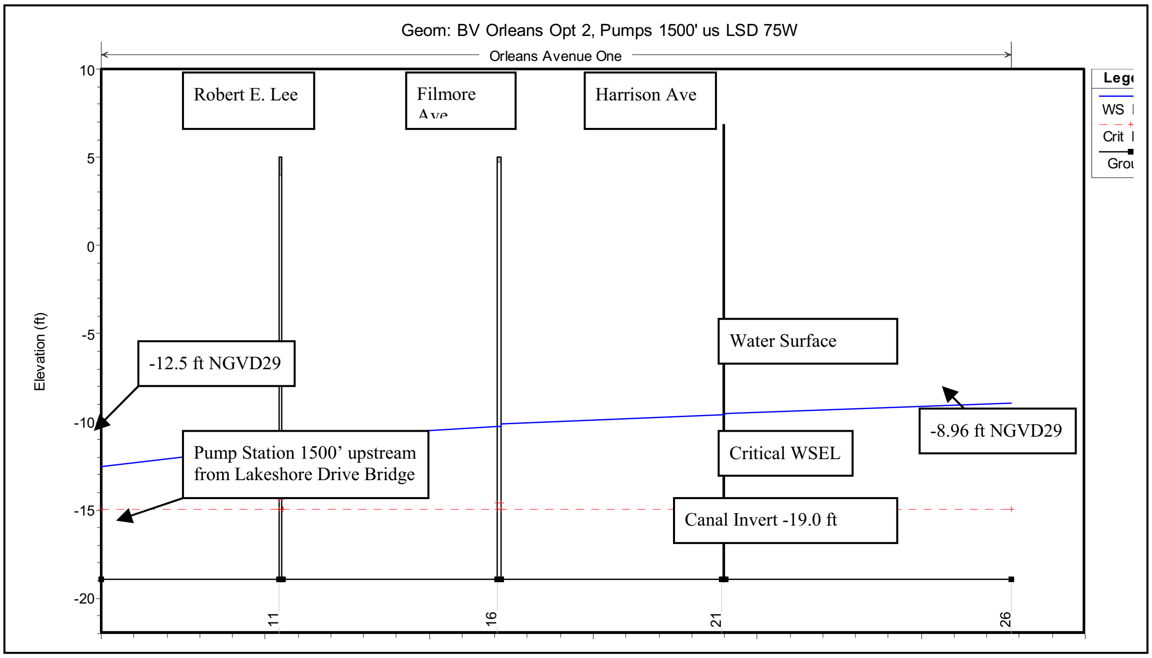


Figure 5.4.2.3-1. Water Surface Profile – Orleans Avenue Canal (Option 2)

5.4.2.4 Geotechnical

The typical stratification for the Orleans Canal is taken from the IPET Report, Volume V. From the top down the stratification includes; Marsh Clay, Relic Beach Sand, and Bay Sound Clay. Pleistocene sand and clay strata are below the Bay Sound Clay. A typical representation of the canal geology as modified for the deeper canal is shown in the figure below. The section is taken from the IPET report. Corresponding IPET report figures are included in Appendix B for reference.

The strength and physical properties used to characterize these strata are also taken from the IPET Report, Volume V, Appendix 10. Text from the IPET Report is included in Appendix B to show the source of the strength parameters for this section. The parameters used in this study are shown in Figure 5.4.2.4-1. The data and strength parameters are consistent with the IPET Report and are taken as conservative.

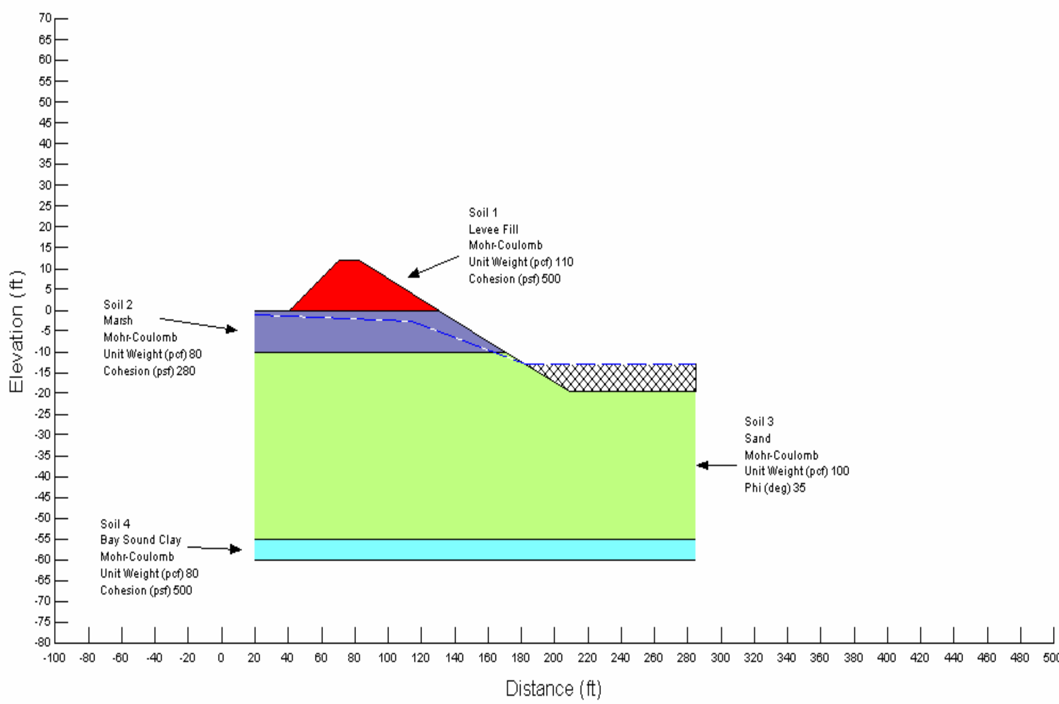


Figure 5.4.2.4-1 Orleans Canal Geology, Option 2

Canal

One of the significant changes required for Option 2 is the lowering of the flow line and the resulting deepening of the canal. The existing bottom of the canal is at approximate elevation -10 ft (NAVD). The new bottom needs to be at approximate elevation -26 ft (NAVD).

Stability analysis of the deeper canal shows the slopes of the canal do not meet safety criteria (figures B-19, 20, Appendix B). The slopes of the canal were flattened to determine the safe slope which will produce a factor of safety of 1.5. Slopes of 4h:1v produce the required minimum factors of safety (Figure B-11, Appendix B). When 4:1 slopes are projected out, significant amounts of additional property are required. Therefore flatter slopes are not practical and the design is modified to include a concrete liner for the deepened canal.

The lower flow line in the canal creates a potential recharging situation in which ground water from the adjoining properties would flow into the canal. The analysis of this threat is discussed in full in Section 5.2.2.4. Since the analysis for is identical, it is not repeated here.

Pump Station Foundation

Evaluation of foundation issues for the Option 2 pump station at Orleans Canal is under way at the time of this writing. Preliminary stability evaluation has been completed and is included as file 3.1 in Appendix B. This stability analysis is a two-dimensional analysis of a typical section cut perpendicular to the long axis of the pump station. This section includes all driving and resisting loads. Analysis results are reported on a per foot basis representing the nominal one foot thickness of a two dimensional analysis. Conservative assumptions are made throughout so the two-dimensional analysis should represent conservative evaluation for this study. The issues identified for evaluation are described in the following paragraphs.

Sliding: The excavation for the intake basin will remove most of the material from the upstream side of the pump station. The pumps being considered for this study will require 14 ft of water to operate properly. The critical sliding case will be when the downstream side is subjected to lake surge. Even though the lake surge is relatively short duration, the structure must be stable during the few hours it is present. The net pressure on the pump building is substantial. Several elements of the foundation design will contribute to or effect sliding resistance.

The thickness of the base slab is dictated by uplift considerations (see below). The slab will be entirely below grade so it will develop shear in the adjacent soil. At the anticipated depth of the base slab, this resistance will develop in the Relic Beach Sand. The sand has a moderate shear strength with phi = 35 degrees. Passive resistance alone is not expected to be adequate. Deep soil mixing is being used at the temporary protection structure at 17^{th} Street to substantially improve the foundation soils and may be considered for this pump station as well.

The weight of the structure will generate friction on the base of the slab. Because of the high uplift pressures, net weight will be low. This means the base friction will not provide much resistance for sliding. Base friction can be improved somewhat by soil modification but will never be the controlling factor unless weight is added to the structure explicitly for this purpose. The addition of weight to improve base shear is not efficient and will not be considered for this study.

Preliminary calculations demonstrate a deficiency in sliding resistance of approximately 50 kip/ft. This deficiency must be corrected to achieve a factor of safety of 1.5 for sliding. Design elements will have to be added to provide this additional resistance.

The greatest influence on sliding resistance will likely come from the foundation piling. Including lateral resistance into the design of vertical piles is one method of providing the additional needed lateral resistance. Another viable method of providing the resistance is the addition of battered piles. Battered piles are more efficient lateral resistance elements and will likely be used. In any case, soil modification can improve this resistance and will be considered as a possible supplemental measure.

Uplift: When the structure is subjected to lake surge, substantial uplift will develop on the base of the structure. The uplift is assumed to instantaneously reflect the lake surge pressures. Hydraulic conductivity of the Beach Sand is typical of sand and must be assumed to be hydraulically connected to the lake. Uplift pressure is calculated by assuming a linear variation from the lake head to the upstream head in the canal across the bottom of the pump station. If needed, this can be modified by installing a cut-off wall below the foundation. Cut-off walls can be effective but will be difficult to coordinate with a pile foundation anticipated for this application. For this study it will be assumed that no cut-off wall will be installed.

Conservative design resists uplift forces with dead load. At this stage of the study the base slab is being sized to provide the necessary dead load to resist uplift with a factor of safety of 1.1. This requires a base slab of 11 ft thickness with bottom at elevation -38 ft (NAVD). It is recognized that the vertical piling supporting the structure will also provide uplift resistance. For this study the piling will be considered to provide supplemental resistance to raise the factor of safety above 1.5, but will not provide the principal resistance system. This is a conservative approach that ensures long term stability against uplift failures.

Underseepage: Underseepage is a potential failure mode with the combination of high heads and weak foundation soils. Since the base slab is 11 ft thick, the length of the flow path may be long enough to eliminate this failure mode. Underseepage can result in the loss of the foundation material through piping beneath the foundation. Seepage calculations will be performed to check this failure mode. It is possible that a cut-off wall will be required.

Foundation Support: Preliminary calculations of overturning indicate the structure is not 100 percent in compression without the addition of tension elements. Tension elements with a capacity of 140 kip/ft will result in the required 100 percent base compression. The additional tension capacity is based on being able achieve a centroid of the tension elements at 2/3 the width of the base slab. Vertical tension piles will be assumed to develop the overturning stability.

The principle vertical resistance system for the pump station will likely be piling. This is common practice for the area when foundation soils are too weak for the structural loading. The strength of the soils can be improved by soil modification techniques and this will be considered. However the big issue for this structure will be settlement. Ground modification would have to extend to greater depths if it is desired to eliminate piling. For simplicity the structure will be founded on piling for this study. The concept design of the piling will ensure it will resist static vertical load when the gates are open and the eccentric loading produced by unbalanced water pressure when the gates are closed.

5.4.2.5 Structural

See the general discussion for the state of structural design in paragraph 5.1.2.5. The critical foundation design elements are discussed in paragraph 5.4.2.4 immediately above.

5.4.2.6 Mechanical

The function of the pumping station is to lift water from the canals to the lake. The principle piece of machinery to do this is the pump. High capacity, low head pumps are essentially large propellers in a tube. For this application there are two principle types of pumps defined by the orientation of the propeller. The propeller can be installed horizontally or vertical. The horizontal types are similar to the Woods Screw Pumps which are extensively used in the older pumping stations.

The horizontal pumps are installed horizontally on an operating floor above the maximum canal level. As such, the propeller is above the water surface. To operate the pump, a vacuum is used to extract air out of the pump and pump discharge piping until the propeller is submerged. Once the propeller is submerged, the pump can be turned on and the pump will complete the filling of the discharge pipeline and establishing a siphon discharge. The major advantage of the horizontal pumps is that the pump bearings and propeller are located above the canal and the easily accessible for maintenance. The pump can actually be started before the propeller becomes fully submerged permitting a low startup torque which minimizes engine generator sizing. The major disadvantage is that the pumps need to be primed by a vacuum system. Due to the volume of air needed to be evacuated, it can take 10 to 15 minutes to get the pump started.

The vertical pump has the propeller mounted down below the minimum canal water surface level. Like the horizontal screw pump vertical pumps are also used extensively in the Parish. As a result the pump design, the pump is self-priming and can start pumping within seconds of a start command which is a significant advantage in controlling pumping units when pumping stations are located in series. Also with this design, the motors are located on top of the pump and out of any danger of being damaged by flooding. The major disadvantage is that the propeller is below the water surface and that any major maintenance requires fully disassembling the pump. Also a disadvantage is that the pump starts under load and has a high startup toque which can require over sizing engine generators.

Either type pump is applicable to this pumping station. For simplicity of this analysis, only vertical pumps which provide the maximum flood protection with the elevated motors are considered in the station. During detailed design the use of vertical, horizontal, or a combination of both should be considered.

For reverse flow protection, the discharge pipe from the pumps are elevated such the invert of the pipe at the highest point is at or above the floodwall elevation so that reverse flow through the pump is not likely. The discharge pipe is then brought down below the minimum lake level forming a siphon. A siphon discharge permits recovering the energy so that when normally pumping the pumps only see the difference between the canal elevation and the lake. A vacuum breaker is provided at the highest point in the discharge pipe to permit breaking the siphon when the pump stops. For added protection, sluice gates can be added to the discharge pipe for protection against reverse flow. To minimize submergence and hydraulic losses through the station, a formed suction inlet was used in the analysis. A typical cross-section of the pumping station is attached to Appendix D – Mechanical.

The pumping units can be driven by either electric motor with electric generator backup or direct driven by engines. In the final design, there may be a combination of drivers in the pumping station. Direct driven engines are cheaper since they eliminate the engine generator and motor. Motor driven pumps are quieter and more efficient. A determining factor may well be the ability or willingness of the power company to build power lines and reserve generating capacity for pumping units which may only occasionally be operated. A detailed study should be done during design to determine the optimum combination of electric driven motors with engine generator backup or direct engine driven pumps. For the purpose of this analysis approximately 60 percent motor driven pumps and 40 percent engine driven pumps are assumed.

A polling of pumping manufacturers indicated that the maximum practical size of pumping units is roughly 1,000 cfs. This is limited by the physical size of the equipment and the ability to move the equipment along major roadways. 1,000 cfs also matches up with the largest pumping stations in the major feeder pumping stations so was chosen as the main pumps in the new pumping stations. The existing pumping stations also have a number of smaller pumps. Smaller pumps permit pumping lower flows without having frequent starts and stops and also provided the ability to match flows when pumping stations operate in series. Therefore combinations of 1,000, 500, and 250 cfs pumps were selected at as the primary capacities. Using only three sizes will permit the sharing of parts between the pumping stations.

Under Option 2 the pumping station should be provided with screens ahead of the pumping stations to protect the pumps for large solids as the screens at Pumping Station 3, 6, and 7 will be eliminated.

There are a number of additional mechanical systems required for operation of the pumping station. All major pumps require a clean source of water for bearing lubrication. This can be from the water system. However, based on experience during Katrina in which the water system failed, a secondary source of water should be provided. There are two sources available, either canal water or well water. Because the water to the pumps needs to be of high quality, well water is being considered for the pumping station. Canal water can be used but requires a high level of treatment to remove abrasives. Because of the size of the equipment in the facility, the facility should include overhead crane, lay down space, truck loading access, and workshop areas. Whether motor driven or engine pumps are used, there will be significant opening for ventilations. All inlet air vents should be shrouded to inhibit the entry of wind blown water.

Engine driven pumps have additional mechanical considerations including engine and gear reducer cooling systems, fuel system, starting air systems, lubricating oil and waste oil systems. In addition, engines require both exhaust air and combustion air systems.

Pumping station hydraulics are critical to successful operation of the pumping station to achieve maximum hydraulic performance. A physical model test of the pumping station including canal entrance, screens, and pump inlet, and discharge siphon pipe must be conducted as a follow-on effort to ensure correct sizing and configuration.

For the purpose of this study, the following combination of vertical pumping unit capacities was chosen. In addition, a combination of direct drive diesel engines and diesel generators are assumed to provide an approximate 60 percent electric motor drive and 40 percent diesel engine drive ratio.

Pump Driver Type		
Pump Number	Pump Capacity cfs	Driver Type
1	1000	Direct Drive Engine
2	1000	Direct Drive Engine
3	500	Motor on Grid or Generator
4	500	Motor on Grid or Generator
5	250	Motor on Grid or Generator
6	250	Motor on Grid or Generator

Fuel storage capacity for the pump station was selected at four days of full pumping capacity. Based on this duration and usage rate, the anticipated fuel storage required is slightly over 60,000 gallons. Assuming standard 12,000 gallon double wall fuel storage tanks, a minimum of five tanks will be required.

The estimated pump ratings are as follows:

Pump Rating			
Capacity, cfs	1,000	500	250
Bowl Head, ft	25.7	25.7	25.7
Pump Speed	162	227	321
Engine Rating, bhp	4100	---	---
Motor rating, hp	3500	2000	900

In the Mechanical Appendix D are representative pump performance curves submitted by the manufacturers. The curves are presented as typical curves only as the required pump rating have evolved during the study and the pump ratings on the curves will differ slightly from the latest hydraulic requirements.

5.4.2.7 Electrical

Option 2 removes existing pump station DPS7 from service and therefore requires more power to achieve the same flow as Option 1. The general arrangement of electrical equipment remains the same, but larger electrical equipment is required to meet the increased load demands. Utility power, from Entergy, will only be supplied for the normal pump loads (see Pump Driver Schedule below for pump utilization). The incoming utility service will not be sized to accommodate the storm event pump loads. One hundred percent of the storm event electric-driven pump loads will be supplied from standby generators dedicated to the Orleans Avenue Pump Station. The standby generators will utilize an N+1 design such that if a generator goes off line or one is down for maintenance, the full pump station load will still be supplied by standby power. The utility service, standby generators, and pump motors will all operate at 4160 volts. All electrical distribution circuits will be routed underground in concrete-encased ductbank. The Pump Driver Schedule below illustrates the increased power requirements of Option 2.

Option 2 - Orleans Canal – Pump Driver Schedule						
Pump cfs	Driver bhp	Motor hp Nameplate	Motor hp Load	Source*	Utilization Grid	Ind
1000	3206			Engine		1%
1000	3206			Engine		1%
500	1359	2250	1925	Grid/Generator	1%	1%
500	1359	2250	1925	Grid/Generator	5%	1%
250	680	1250	1250	Grid/Generator	50%	1%
150	408	1000	1000	Grid/Generator	100%	1%
3400			**5500**	**Totals**		

Based on the pump driver information above, the next table below indicates the electrical equipment sizes used for this option. See the appendices for all cost information.

Option 2 – Orleans Canal Pump Station - Electrical	
Electrical Equipment	**Quantity/Capacity**
Utility Substation	1 – 7500kVA
Generators	4 – 2500kW
Generator Bldg Size	112’ x 80’
Pump Bldg Switchgear	2 – 1200A
Total Pump Station Load	5.8 MVA, 802A
Pump Station Load on Utility	5.8 MVA, 802A

5.4.2.8 Environmental

Environmental Site Assessment (ESA)

Because conditions/impacts and results for the ESA under Option 2 for the Orleans Avenue Canal are identical to those discussed in Section 5.3.2.8, please see that section for this discussion.

Sediments

Because sediment quality and findings for Option 2 for the 17th Street Canal are nearly identical to those discussed in Section 5.3.2.8, please see that section the majority of this discussion. However, because this option contains the dredging of a much larger quantity of bottom sediments, costs will be significantly higher if disposal at a hazardous waste permitted land fill or on-site treatment is required. Transportation costs for disposal at a permitted land fill may render this option as cost prohibitive. On-site treatment (i.e., thermal treatment of organic contaminants) would most likely be the required preferred method. Once treated, sediments could be disposed in a construction debris land fill or perhaps beneficially used.

Wetlands

See discussion in Section 5.3.2.8.

Protected Species

See discussion in Section 5.1.2.8.

Cultural Resources

See discussion in Section 5.1.2.8.

5.4.2.9 Constructability

The construction concepts developed for Option 2 for the Orleans Canal are very similar to the Option 1 concepts. The big differences between Option 1 and Option 2 as they affect constructability are the deeper excavation required for the pump station and the channel modifications.

The concept of enclosing the entire new pump station in a cofferdam for all location alternates is still valid. The pump station will be larger in plan and deeper. The deeper excavation for the pump station will require a more robust design for the cofferdam. The cofferdam concept remains the essentially same as described for Option 1 in 5.3.2.9 and illustrated in figures 5.3.2.9-1, 5.3.2.9-2, and 5.3.2.9-3 shown earlier.

When constructing the bypass channel for Option 2 at location Alternate B, the bottom of the bypass channel only needs to match the existing channel bottom. Therefore the concept of the by pass channel is also the same as for Option 1. The same problems and costs are associated with construction of by pass channels for location Alternate B as described in Option 1.

The second concept of building the pump station in two parts to eliminate the need for a bypass channel for Alternate B becomes more complex. The Option 2 pump station does not include a gate. The first half of the pump station can be constructed by leaving half of the existing channel open. Construction of the second half will result in complete closure of the existing canal. Therefore the design of the first half would need to be modified to include gates adequate to pass the flows require for the temporary gate and pumps. This will make the structure larger and more expensive. Furthermore, the gates would only be used in the construction period and never required after that time. The gate adds a point of vulnerability during the working life of the pump station without adding a function

Again, the question becomes one of cost. The costs developed for this report include the construction of bypass channels. The design build contractor would be permitted chose either of these concepts or innovate a third concept of his own if it would reduce project cost. See figures 5.3.2.9-1, 5.3.2.9-2, and 5.3.2.9-3 for plan representations of cofferdam and bypass requirements.

Option 2 also requires modification to the existing canal by deepening it. Construction of the concrete liner for the canal must be accomplished in such a way as to permit flows in the canal. The quantity of flow to be passed is determined by the design of the temporary structure. There is no need to pass more flow than that structure can pass. Two concepts have been developed for construction of the concrete liner.

The first concept is illustrated in Figure 5.2.2.9-1 in the 17^{th} Street section of this report. That conceptual diagram is valid for the Orleans canal as well but is not repeated here. This concept minimizes constriction of the existing canal. Each wall of the concrete liner is constructed in the dry inside a cofferdam box. The box is then moved to the other side of the canal to construct the opposite wall. The box is moved to the next section and the floor of the liner is placed in the wet with tremie concrete. The cofferdam box is reused for each segment of the canal liner along the full length of canal to be modified. The length of the box can be adjusted to suit the design and schedule. Sheet pile cutoff walls are installed below each wall to improve stability and to provide a seepage barrier is required. (See the geotechnical discussion above.) Soil anchors or deadman anchors may be needed to provide stability to the walls.

A second concept uses a larger box to enclose half of the canal. This concept allows construction of the wall and floor both in the dry. It ensures a better connection of the two elements and may result in elimination of the cutoff wall. The wall is completed in two steps instead of three. The chief disadvantage is the available cross-section of the canal for flow is cut in half. This may not be adequate.

A third concept has been discussed during development of the project as having been used successfully in the past. This concept provides for damming both ends of the section being built and dewatering for construction in the dry. If an event requires it, the dam can be breached or allowed to overtop so the event flows can be passed. This technique has been used to line canals in New Orleans. However the deepening of this canal makes this technique impractical. Pressure heads would be on the order of 25 feet.

5.5 Option 1 – London Canal

5.5.1 Alternative Approaches

Three location alternatives were considered for Option 1 on the London Avenue canal. The merits of each were evaluated and discussed in detail in the Civil/Site section. For the purposes of this study, Alternative A was chosen as the location to base costing and other engineering considerations.

5.5.2 Engineering Considerations

5.5.2.1 Civil/Site

The London Avenue Pump Station for Option 1 is anticipated to be 350 feet long by 155 feet wide. The total station length includes space for flood gates, due to the anticipated removal of the temporary gate structure well upstream of this location. The total station width includes a 45 foot inlet works including trash screens, a 70 foot pump station building housing pumps and motors, and a 40 foot outlet works. Finish grade for the Generator and Tank Farm Complex is always approximately +16.0 elevation.

- The **Alternative A Pump Station Layout** is as shown in Exhibit 5.5.1.A, Appendix C.

General Location and Description - Under this alternative, the pump station is located on the east canal bank approximately 1000 feet upstream of Lake Shore Drive. The site utilizes an existing protrusion in the east canal bank, which allows most of the pump station to be constructed on the bank, rather than in the canal. Thus, maintaining canal flow during pump station construction is relatively convenient.

The location requires existing lake-front levees to be extended upstream to connect to the pump station site. This upstream location shields the pump station from lake surge effects. Note that, prior to the removal of the temporary gate structure upstream of this site, a closure levee will be constructed in the existing canal adjacent to the pump station as the final element of a continuous lake-front protection system.

Right-of-Way Acquisition - Permanent right-of-way acquisition will occur almost exclusively on the east bank of the proposed site. Temporary construction easement is assumed to be necessary only along a relatively small area in the vicinity of the west end of the closure levee for access.

Demolitions and Earthwork - This layout alternative does require right-of-way acquisition of currently developed property. However, that existing development appears to be limited to parking areas and other minor uninhabited features. Some levee removals will be required. Earthwork at this site is almost exclusively excavation, resulting in a volume of earth materials to be removed from the project site.

Channel Transitions - Channel transitions are required both immediately upstream and downstream of the pump station to ensure laminar flow between the trapezoidal canal cross-section and the rectangular pump station cross-section. However, due to site geometry, both upstream and downstream transitions are required only on the east bank. Also due to the placement of the pump station, a maximum convergence angle is not applicable. The layout as shown does satisfy the maximum preferred 25 degree divergence angle. Counterforted retaining walls, in this application, offer the maximum in very long-term durability, low maintenance, and good flow characteristics. Clearly, transition structures could be constructed in other ways that might offer significant cost savings over counterforted walls. Tied-back sheet pile walls could be used, but long term durability and corrosion resistance are issues. Therefore, for conservatism under this study, concrete retaining walls are anticipated, pending subsequent optimization studies on the subject.

Erosion Protection - Given the inland location of this pump station, a relatively small volume of erosion protection armoring will be required. Specifically, a strip of riprap protection is anticipated in the new canal floor, both immediately upstream and downstream of the pump station. Also required will be riprap protection on both faces of the closure levee constructed immediately adjacent to the west of the pump station building. No breakwater in Lake Pontchartrain is anticipated to be necessary to protect the pump station discharge under this Layout Alternative A.

Generator Building and Tank Farm Complex - Supporting the pump station on the east bank immediately adjacent to the pump station is a Generator Building and Tank Farm Complex. The Generator Building is anticipated to be approximately 80 by 130 feet, and each of five 12,000 gallon fuel tank pads is anticipated to be approximately 12 by 28 feet in size. According to NFPA 30, 2003 Edition, diesel fuel is a Class II Combustible Liquid. As such, tanks require a five foot clear distance from the public way or important buildings on the site. The complex also includes parking, general staging and storage space, all concrete paved, and including sidewalk and local site storm drainage features. Utilities will include potable water service, sanitary sewer and natural gas, all connected to the station from existing utilities available within several hundred feet of the proposed site. Although not a significant cost item, the provision of these utilities when the station is operating under total self-sufficiency is anticipated. The paved complex area is also anticipated to be enclosed by a chain link security fence, with minor landscaping improvements. All non-paved areas will be seeded to re-establish healthy turf for aesthetics and erosion control.

Summary – This layout is attractive for its protection of the pump station from lake surge effects, resulting in no need for a major breakwater structure. The location requires right-of-way acquisition in a residential area, but acquisition appears to be property developed only with parking areas and other relatively low value improvements. Further, it is attractive since no significant construction sequencing is required to maintain canal flow during the construction duration.

- The **Alternative B Pump Station Layout** is as shown in Exhibit 5.5.1.B, Appendix C.

General Location and Description - Under this alternative, the pump station is located in the existing canal, as near the Lake Shore Drive Bridge as possible without creating the need for modifications to that bridge. Thus, the pump station is only approximately 400 feet upstream of Lake Shore Drive. Inlet and outlet channel transitions to the pump station are slightly impacted due to the channel curve, but not significantly so. Also, the near-shore location of the pump station discharge makes the inclusion of a major breakwater structure in Lake Pontchartrain mandatory.

As more fully described in paragraph 5.1.2.9, the pump station would be constructed in two stages (east half, then west half) in order to maintain canal flow at all times during construction activities. The temporary gate structure upstream of this site will be removed after pump station construction is complete. This location is essentially in-line with the existing lake-front levee system. Therefore, it provides for convenient connection of those existing shore-front levees to the new pump station features.

Right-of-Way Acquisition - Right-of-way acquisition is likely required only on the west bank, exclusively in currently undeveloped property.

Demolitions and Earthwork - This layout alternative requires right-of-way acquisition of currently undeveloped property. Some levee removals will be required. Earthwork at this site is almost exclusively excavation, resulting in a volume of earth materials to be removed from the project site.

Channel Transitions - Channel transitions are required both immediately upstream and downstream of the pump station on both banks to ensure laminar flow between the trapezoidal canal cross-section and the rectangular pump station cross-section. Counterforted retaining walls offer the maximum in very long-term durability, low maintenance, and good flow characteristics. Clearly, transition structures could be constructed in other ways that might offer significant cost savings over counterforted walls. Tied-back sheet pile walls could be used, but long term durability and corrosion resistance are issues. Therefore, for conservatism under this study, concrete retaining walls are anticipated, pending subsequent optimization studies on the subject.

Erosion Protection – Given the lakeshore location of this pump station, a significant volume of erosion protection armoring will be required, primarily located in a major breakwater in Lake Pontchartrain. Also, a strip of riprap protection is anticipated in the new canal floor, both immediately upstream and downstream of the pump station.

Generator Building and Tank Farm Complex - Supporting the pump station on the east bank immediately adjacent to the pump station is a Generator Building and Tank Farm Complex. The Generator Building is anticipated to be approximately 80 by 130 feet, and each of five 12,000 gallon fuel tank pads is anticipated to be approximately 12 by 28 feet in size. A 20 by 20 foot electrical substation is also included. The complex also includes parking, general staging and storage space, all concrete paved, and including sidewalk and local site storm drainage features. Utilities will include potable water service, sanitary sewer and natural gas, all connected to the station from existing utilities available within several hundred feet of the proposed site. The paved complex area is also anticipated to be enclosed by a chain link security fence, with minor landscaping improvements. All non-paved areas will be seeded to re-establish healthy turf for aesthetics and erosion control.

Summary - This layout is attractive for its convenient fit within the existing canal width. The location requires right-of-way acquisition in a residential area, but acquisition appears to be exclusively undeveloped property. Due to the lakeshore discharge location, a major breakwater structure is required. Some construction complexity is introduced, since the pump station must be constructed in two stages to maintain channel flow during construction. In summary, this option seems to create negatives compared to Layout A, while adding no advantages over Layout A. Thus, it is not recommended for further consideration.

- **The Alternative C Pump Station layout** is as shown in Exhibit 5.5.1.C, Appendix C.

General Location and Description - This alternative provides a location just downstream of the Lakeshore Drive Bridge, essentially constructed in its entirety in Lake Pontchartrain, positioned on the linear extension of the existing canal. The pump station is approximately 800 feet downstream of Lake Shore Drive. The in-lake location of the pump station makes the inclusion of a major breakwater structure in Lake Pontchartrain mandatory. It also requires significant earthwork to create the site.

The temporary gate structure upstream of this site will be removed after pump station construction is complete. Finally, note that this location requires modifications to extend the existing shore-front levee line out into Lake Pontchartrain, including the removal and replacement of the Lake Shore Drive Bridge.

Right-of-Way Acquisition - Right-of-way acquisition is required primarily for shore-located support features, and may represent areas publicly-owned, rather than privately-owned, properties

Demolition and Earthwork – The only significant removal (and replacement) this layout requires is the Lake Shore Drive Bridge. Some existing levee and minor miscellaneous site features in the area will be removed as well. Due to the in-lake location, earthwork for this layout may approach a cut and fill balance. For this study, some disposal of earth materials is still anticipated.

Channel Ttransitions – Counterforted retaining walls serve as upstream channel transitions immediately upstream of the pump station to ensure laminar flow between the trapezoidal canal cross-section and the rectangular pump station inlet cross-section. Downstream transitions will be relatively short length, vertical training walls, given the in-lake discharge location.

Erosion Protection - Given the in-lake location of this pump station, a significant volume of erosion protection armoring will be required, primarily located around the banks of the pump station facility and the in-lake breakwater structure. Also, a strip of riprap protection is anticipated in the new canal floor, both immediately upstream and downstream of the pump station.

Generator Building and Tank Farm Complex - Supporting the pump station on the west shore is a Generator Building and Tank Farm Complex. The Generator Building is anticipated to be approximately 80 by 130 feet, and each of five 12,000 gallon fuel tank pads is anticipated to be approximately 12 by 28 feet in size. According to NFPA 30, 2003 Edition, diesel fuel is a Class II Combustible Liquid. As such, tanks require a five foot clear distance from the public way or important buildings on the site. A 20 by 20 foot electrical substation is also anticipated. The complex also includes parking, general staging and storage space, all concrete paved, and including sidewalk and local site storm drainage features. Utilities will include potable water service, sanitary sewer and natural gas, all connected to the station from existing utilities available from just over 1,000 feet from the proposed site. The paved complex area is also anticipated to be enclosed by a chain link security fence, with minor landscaping improvements. All non-paved areas will be seeded to re-establish healthy turf for aesthetics and erosion control.

Summary - This layout is attractive for its relatively minimal right-of-way acquisition requirements. Constructability is a mixture of positives and negatives. Constructing the pump station building in one phase is positive; however, construction in the lake offers other complications that are costly to overcome. Further, a major breakwater structure and significant plant armoring is required..

5.5.2.2 Bridges and Utilities

Bridges

The London Avenue Canal is crossed by 6 bridges between the Pump Station No. 3 and the outfall into Lake Pontchartrain. These bridges and their locations are identified as follows.

Interstate 610 Bridges – Exhibit 5.5.2.2.A, Appendix F.

There are two I-610 bridges that cross the London Avenue Outfall Canal approximately 0.2 miles downstream of Pump Station No. 3 (one east bound and one west bound). The bridges consist of 130' steel girder spans over the canal supported by 36" reinforced concrete columns with a top-of0footing elevation of 3' on the west bank and 2' on the east bank. These footings are supported by 14" cast in place concrete piles with an approximate tip elevation of –78' on the west bank and –80' on the east bank. These columns and footings are outside the floodwalls for the canal.

Gentilly Boulevard Bridge – Exhibit 5.5.2.2.B, Appendix F.

The Gentilly Boulevard Bridge over the London Avenue Outfall Canal is located approximately 0.2 miles downstream of Pump Station No. 3. The bridge consists of three P. P. C. girder spans totaling 136'-04$^1/_2$" supported by 30" steel pipe piles with an approximate tip elevation of –100. The end bents consist of HP 14x73 steel piles with an approximate tip elevation of –90' with sheet pile walls outside the end bents extending to an elevation of approximately –17.88'.

Mirabeau Avenue Bridge – Exhibit 5.5.2.2.C, Appendix F.

The Mirabeau Avenue Bridge over the London Avenue Outfall Canal is located approximately 1.2 miles downstream of Pump Station No. 3. The bridge consists of 2-30' concrete spans in the center with 2-20' concrete spans on each side of these forming a continuous concrete slab span totaling 140'. The bridge is supported by 18" P. P. C. piles with an approximate tip elevation of –73.5'. The end bents consist of HP 14x73 steel piles with an approximate tip elevation of -76' with sheet pile walls outside the end bents extending to an elevation of approximately –16'.

Filmore Avenue Bridge – Exhibit 5.5.2.2.D, Appendix F.

The Filmore Avenue Bridge over the London Avenue Outfall Canal is located approximately 1.4 miles downstream of Pump Station No. 3. The bridge consists of five equal spans forming a continuous concrete slap span totaling 150’. The bridge is supported by 18” P.P.C. piles with an approximate tip elevation of –75.5’. The end bents consist of HP 14x73 steel piles with an approximate tip elevation of –78’ with sheet pile walls outside the end bents extending to an elevation of approximately –14’ on the west bank and –16’ on the east bank.

Robert E. Lee Boulevard Bridge – Exhibit 5.5.2.2.E, Appendix F.

The Robert E. Lee Boulevard Bridge over the London Avenue Outfall Canal is located approximately 2.0 miles downstream of Pump Station No. 3. The bridge consists of five spans totaling 180’-08$^{1}/_{2}$” supported by 12’ concrete-filled steel piles with an approximate tip elevation of –48.16. The end bents consist of 12” treated timber piles with an approximate tip elevation of –10.23’.

Leon C. Simon Boulevard Bridge – Exhibit 5.5.2.2.F, Appendix F.

The Leon C. Simon Boulevard Bridge over the London Avenue Outfall Canal is located approximately 2.1 miles downstream of Pump Station No. 3. The bridge consists of four P.P.C. girder spans totaling 187’-2” supported by 30” steel pipe piles with an approximate tip elevation of –92. The end bents consist of HP 14x73 steel piles with an approximate tip elevation of –90’ with sheet pile walls outside the end bents extending to an elevation of approximately –22.88’.

Lakeshore Drive Bridge – Exhibit 5.5.2.2.G, Appendix F.

The Lakeshore Drive Bridge over the London Avenue Outfall Canal is located approximately 2.6 miles downstream of Pump Station No. 3. The bridge consists of four spans totaling 170’ supported by 18” P. P. C. piles with approximate tip elevation of –76’. The end bents consist of 16” P. P. C. piles with an approximate tip elevation of –76’. There is a stepped seawall along the inner slopes of the canal extending approximately 3’ below the surface of the normal water level. This seawall is supported by a double and single bent of 12” precast (non-prestressed) concrete piles with an approximate tip elevation of –37.17’ The lower end of the seawall is supported by 9” x 24” concrete sheet pile wall extending to an elevation of –37.17’.

Southern Railroad Bridge – No exhibits.

The Southern Railroad Bridge over London Avenue Outfall Canal is located just down stream of Pump Station No. 3. There is no information available on this bridge at this time.

None of the bridges are affected by Option 1 except the Gentilly Boulevard Bridge and the Lakeshore Drive Bridge. The Gentilly Boulevard Bridge will require replacement because of the insufficient hydraulic capacity under the conditions of Option 1. The

Lakeshore Drive Bridge may be affected by the pump station site locations. For pump station Layout A and B there are no affects to the Lakeshore Drive Bridge. For site location Alternate C the Lakeshore Drive Bridge would require replacement.

Utilities

The utilities studied in Option 1 are underground or pile supported water, sewer, drainage, electric (transmission and primary), telephone cables, fiber optic cables, and gas. In Option 1, the existing utilities impacted by construction in the vicinity of the London Avenue Canal are those utilities displaced as a result of the new pump station and gated structure in the vicinity of Lake Pontchartrain.

In alternatives A and B of Option 1, there are no impacted existing utilities. In Alternative C, the only utility impacted is a canal crossing 6" diameter water line that is attached to the Lakeshore Drive bridge deck, and a 6" diameter high pressure buried gas line crossing the canal about 30' south of the bridge. Estimated gas line replacement cost equals $38,400. The water line will need to be replaced along with the Lakeshore Drive Bridge in Alternative C. The cost to replace the existing 6" water line is $4,800 in addition to the cost of the Option 1, Alternative C bridge replacement costs.

5.5.2.3 Hydraulic

<u>General</u>

The London Avenue Canal segment considered in this study conveys pumped discharges from DPS 3 and DPS 4 to the canal outfall at Lake Pontchartrain. A future third pumping station, with capacity of 1,000 cfs, is considered in this study. This future pumping station would be located on the opposite side of the canal from DPS 4.

The safe water elevation within the London Avenue Canal, as provided by the USACE, is El. 5.0 NAVD 88. This elevation is considered to be the maximum allowable water surface elevation at any point along the canal. As a practical matter, the controlling location for this safe water level is DPS 3, since the down-gradient slope of the water surface profile within the canal during typical flow conditions will result in water surface elevations at all other points that are lower than the water surface elevation at DPS 3.

For purposes of this study, it is assumed the pumping capability of existing DPS 3 and DPS 4 would be modified, as necessary, to pump at the design discharge capacity and at a head corresponding to the defined safe canal water surface elevation. These modifications, if required, are not considered in this study. It is recognized the rated head of the existing DPS pumping facilities may be less than the required head for the defined safe canal water elevation. If a lower canal water surface elevation at the discharge side of the existing DPS facilities is considered appropriate, the hydraulic analysis presented herein would require revision to account for this lower canal water elevation.

Hydraulic analysis of the canal was performed to determine the following:

- During pumping mode at the design canal discharge condition, determine the maximum canal water surface elevation at the suction side of the Gated Pump Station that will result in a canal water surface elevation at DPS 3 equal to the safe water elevation. This information is necessary to determine pumping head requirements.

- During gates-open operating mode, determine the canal water surface elevation at DPS 3 for various combinations of Lake Pontchartrain elevation and canal discharge. For the given safe water elevation, this will indicate when gate closure is required, with transition from gates-open to pumping mode. This information, in combination with annual canal discharge and Lake Pontchartrain elevation data, will be used to determine annual pumping requirements.

The USACE developed a HEC-RAS computer hydraulic model to estimate canal water surface profiles and other hydraulic information for various combinations of Lake Pontchartrain elevation and canal discharge. The model includes the existing canal cross-section geometry and invert slope between DPS 3 and Lake Pontchartrain. The model also includes the canal cross-section geometry at the several bridge crossings and accounts for inflows representing discharges from each canal pumping station. This hydraulic model was used as the basis for the hydraulic analyses performed for this study. Modeled canal inflows and starting water surface elevations were adjusted appropriately to represent the conditions being considered for this Option. The existing canal geometry was considered to remain unchanged. The hydraulic model was developed based on NGVD29 datum; therefore, subtraction of 0.5 feet from model elevations is necessary for conversion to NAVD88 datum. A simplified flow schematic of the HEC- RAS model is shown in Figure 5.5.2.3-1.

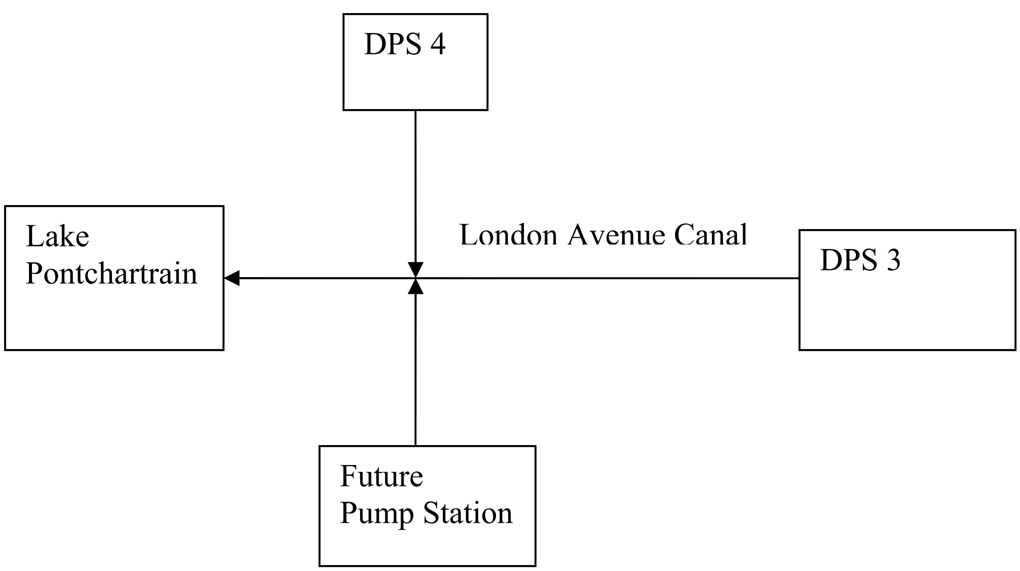


Figure 5.5.2.3-1. HEC-RAS Model Flow Schematic – London Avenue Canal

Hydraulic Analysis - Pumping Mode

The Gated Pumping Station was considered to have a pumping capacity corresponding to the combined capacity of each pumping station discharging into the canal. The capacities of the existing and potential future pumping stations used for this study, and the required pumping capacity of the Gated Pumping Station, are as follows:

London Avenue Canal Pumping Station Capacities	
Existing DPS 3 capacity	4,260 cfs
Existing DPS 4 capacity	3,720 cfs
Potential New Pump Station capacity	1,000 cfs
Gated Pumping Station required capacity	**8,980 cfs**

Using the existing canal cross-section geometry and invert profile, as provided in the USACE-developed HEC-RAS model, along with the design pumping station capacities indicated in the above table, an iterative approach was used to determine the maximum canal water surface elevation at the suction side of the Gated Pump Station that would result in a canal water surface elevation at DPS 3 equal to the defined safe water elevation. The HEC-RAS model was run for several starting suction side water surface elevations and the resulting canal water surface elevation at DPS 3 was determined for each case.

Based on the results of this analysis, it was determined that the defined safe water elevation could not be achieved at DPS 3. All starting water surface elevations selected at the Gated Pump Station resulted in a water surface elevation at DPS 3 that was higher than the safe water elevation. The results of this analysis are shown in Figure 5.5.2.3-2 and indicate that a minimum starting water surface elevation of -0.5 ft., NAVD 88 at the Gated Pumping Station results in a water surface elevation of 5.6 ft., NAVD 88 at DPS 3. This starting water surface elevation results in a flow condition approaching critical depth just upstream from the Gated Pump Station, and was selected to represent an extreme minimum starting water surface elevation, rather than a practical flow condition.

It is noted that the Gentilly Road Bridge is relatively low and it acts as a significant flow constriction at the design discharge. Consideration was given to raising the Gentilly bridge to eliminate the flow constriction. The hydraulic model was modified for this case and the model was re-run. Based on this condition, the defined safe water surface elevation could be achieved upstream from the bridge at DPS 3. The maximum corresponding suction side water surface elevation at the Gated Pump Station was determined to be El. 0.5 ft., NAVD 88. The water surface profile within the canal for this flow condition is provided in Figure 5.5.2.3-3.

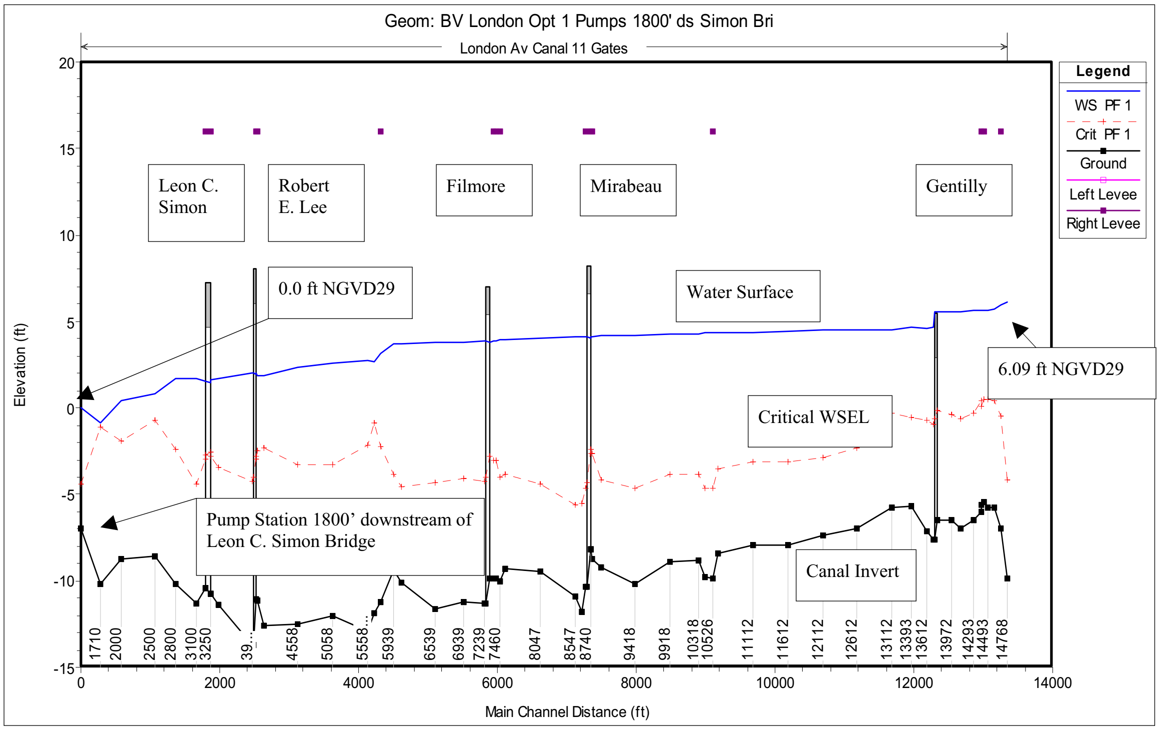


Figure 5.5.2.3-2. Water Surface Profile For Existing Conditions – London Avenue Canal (Option 1)

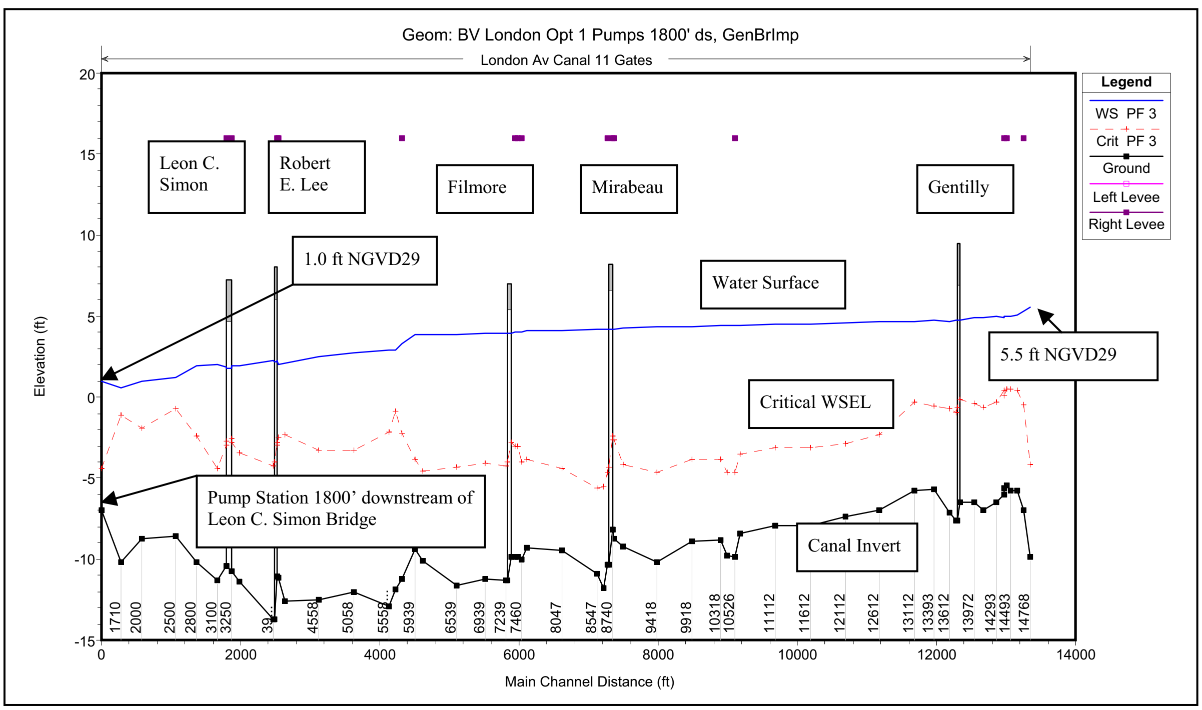


Figure 5.5.2.3-3. Water Surface Profile With Gentilly Bridge Raised – London Avenue Canal (Option 1)

Hydraulic Analysis – Gates Open Mode

The Gated Pump Station would be designed to pass canal discharges through the gate openings for combinations of Lake Pontchartrain elevation and canal discharge that do not cause the safe water elevation in the canal to be exceeded. If conditions are expected to occur that would cause the safe water elevation to be exceeded, the gates would be closed and the Gated Pump Station would be operated in pumping mode.

Using the existing canal cross-section geometry and invert profile, as provided in the USACE-developed HEC-RAS model, and with the assumption of Gentilly Bridge raised, the canal water surface elevation at DPS 3 was determined for various combinations of Lake Pontchartrain elevation and canal discharge, as shown in Figure 5.5.2.3-4.

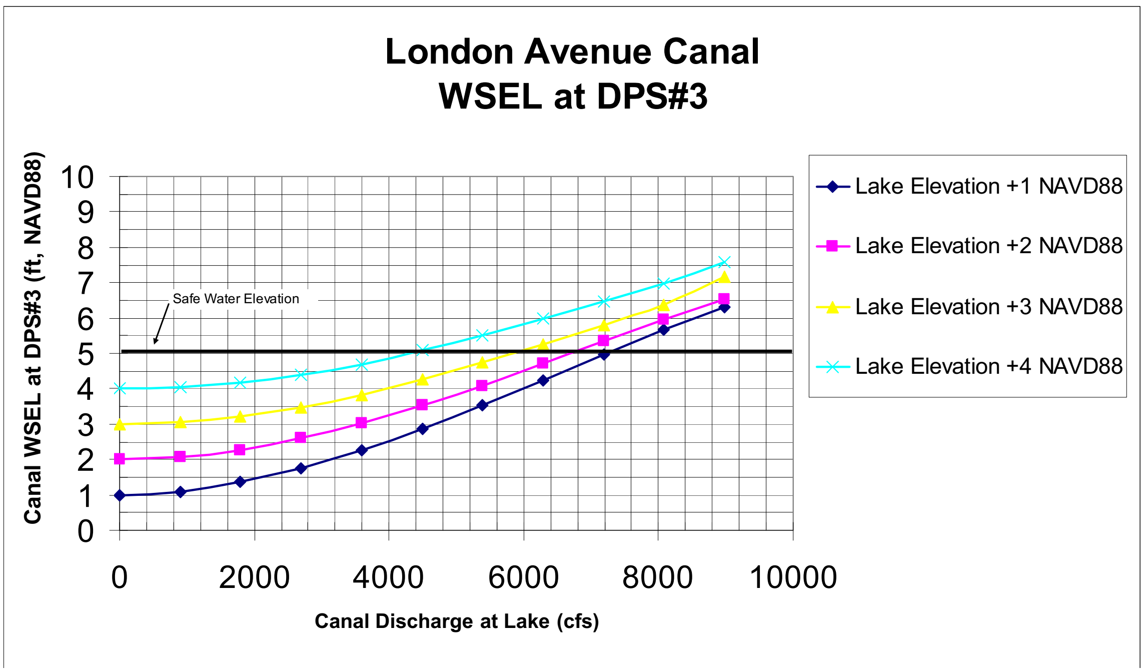


Figure 5.5.2.3-4. Water Surface Elevation at DPS 3 for Various Combinations of Lake Pontchartrain Elevation and Canal Discharge – London Avenue Canal

For all combinations of Lake Pontchartrain elevation and canal discharge that fall below the safe water elevation, operation in the gates-open mode would be possible. For combinations of lake elevation and canal discharge that exceed the indicated safe water elevation, closure of the gates and operation in pumping mode would be required.

5.5.2.4 Geotechnical

The typical stratification for the London Canal is taken from the IPET Report, Volume V. From the top down the stratification includes; Marsh Clay, Relic Beach Sand, and

Bay Sound Clay. Pleistocene sand and clay strata are below the Bay Sound Clay. A typical representation of the canal geology as modified for the deeper canal is shown in Figure 5.5.2.4-1. The section is taken from the IPET report. Corresponding IPET report figures are included in Appendix B for reference.

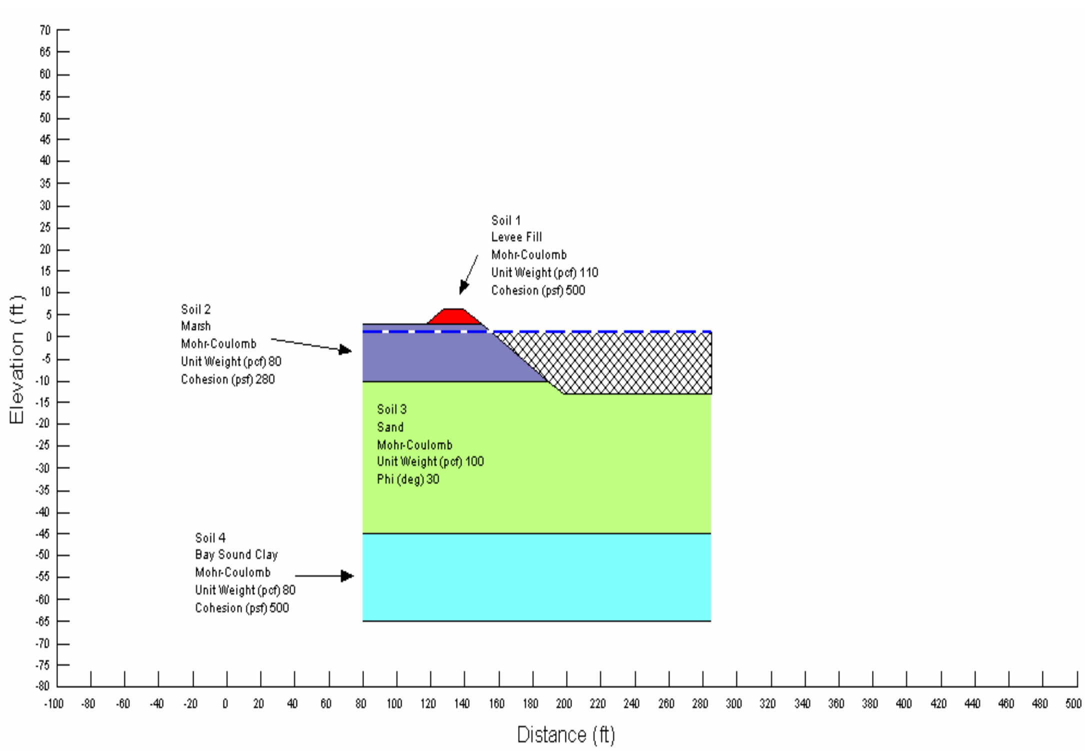


Figure 5.5.2.4-1 London Canal, Option 1

The strength and physical properties used to characterize these strata are also taken from the IPET Report, Volume V, Appendix 7. Diagrams from the IPET Report are included in Appendix B to show the source of the strength parameters for this section. The parameters used in this study are shown in the figure above. The data and strength parameters are consistent with the IPET Report and are taken as conservative.

<u>Canal</u>

The canal is stable in its current state and does not require modification for Option . Stability analysis has been performed for the current state as a calibration of the model used in this study. The results of these calibration analyses (figures 4, 5, and 6, Appendix B) are essentially the same as similar analyses performed for the IPET. This ensures the channel models developed for this study are reasonable and appropriate. This calibration procedure also demonstrates the validity of the strength parameters listed in Table 3-1 above. The slope stability models developed here will be the basis of models used later in the study for evaluation of Option 2.

Pump Station Foundation

Evaluation of foundation issues for the Option 1 pump station at the London Canal are under way at the time of this writing. Preliminary stability evaluation has been completed and is included as file 5.1 in Appendix B. This stability analysis is a two-dimensional analysis of a typical section cut perpendicular to the long axis of the pump station. This section includes all driving and resisting loads. Analysis results are reported on a per foot basis representing the nominal one foot thickness of a two dimensional analysis. Conservative assumptions are made throughout so the two-dimensional analysis should represent conservative evaluation for this study. The issues identified for evaluation are described in the following paragraphs.

Sliding: The excavation for the intake basin will remove most of the material from the upstream side of the pump station. The pumps being considered for this study will require 14 ft of water to operate properly. The critical sliding case will be when the gates are closed and the downstream side is subjected to lake surge. Even though the lake surge is relatively short duration, the structure must be stable during the few hours it is present. The net pressure on the pump building is substantial. Several elements of the foundation design will contribute to or effect sliding resistance.

The thickness of the base slab is dictated by uplift considerations (see below). The slab will be entirely below grade so it will develop shear in the adjacent soil. At the anticipated depth of the base slab, this resistance will develop in the Relic Beach Sand. The sand has a moderate shear strength with phi = 35 degrees. However, with only four feet of embedment passive resistance alone is not expected to be adequate. Deep soil mixing is being used at the temporary protection structure to substantially improve the foundation soils and may be considered for this pump station as well.

The weight of the structure will generate friction on the base of the slab. Because of the high uplift pressures, net weight will not be very high. This means the base friction will not provide much resistance for sliding. Base friction will be improved somewhat by soil modification but will never be the controlling factor unless weight is added to the structure explicitly for this purpose. The addition of weight to improve base shear is not efficient and will not be considered for this study.

Preliminary calculations demonstrate a deficiency in sliding resistance of approximately 26 kip/ft. This deficiency must be corrected to achieve a factor of safety of 1.5 for sliding. Design elements will have to be added to provide this additional resistance.

The greatest influence on sliding resistance will likely come from the foundation piling. Including lateral resistance into the design of vertical piles is one method of providing the additional needed lateral resistance. Another viable method of providing the resistance is the addition of battered piles. Battered piles are more efficient lateral resistance elements and will likely be used. In any case, soil modification can improve this resistance and will be considered as a possible supplemental measure.

Uplift: When the gates are closed and the structured is subjected to lake surge, substantial uplift will develop on the base of the structure. The uplift is assumed to instantaneously reflect the lake surge pressures. Hydraulic conductivity of the Beach Sand is typical of sand and must be assumed to be hydraulically connected to the lake. Uplift pressure is calculated by assuming a linear variation from the lake head to the upstream head in the canal. If needed, this can be modified by installing a cut-off wall below the foundation. Cut-off walls can be effective but will be difficult to coordinate with a pile foundation anticipated for this application. For this study it will be assumed that no cut-off wall will be installed.

Conservative design resists uplift forces with dead load. At this stage of the study the base slab is being sized to provide the necessary dead load to resist uplift with a factor of safety of 1.1. This requires a base slab of 4 ft thickness with bottom at elevation -18 ft (NAVD). It is recognized that the vertical piling supporting the structure will also provide uplift resistance. For this study the piling will be considered to provide supplemental resistance to raise the factor of safety above 1.5, but will not provide the principal resistance system. This is a conservative approach that ensures long term stability against uplift failures.

Underseepage: Underseepage is a potential failure mode with the combination of high heads and weak foundation soils. Since the base slab is only 4 ft thick, the length of the flow path is short and the threat of underseepage problems increases. Underseepage can result in the loss of the foundation material through piping beneath the foundation. Seepage calculations will be performed to check this failure mode. It is likely that a cut-off wall will be required.

Foundation Support: Preliminary calculations of overturning indicate the structure is not 100 percent in compression without the addition of tension elements. Addition of tension elements with a capacity of 77 kip/ft will result in the required 100 percent base compression. The additional tension capacity is based on being able achieve a centroid of the tension elements at 2/3 the width of the base slab. Vertical tension piles will be assumed to develop the overturning stability.

The principle vertical resistance system for the pump station will likely be piling. This is common practice for the area when foundation soils are too weak for the structural loading. The strength of the soils can be improved by soil modification techniques and this will be considered. However the big issue for this structure will be settlement. Ground modification would have to extend to greater depths if it is desired to eliminate piling. For simplicity the structure will be founded on piling for this study. The concept design of the piling will ensure it will resist static vertical load when the gates are open and the eccentric loading produced by unbalanced water pressure when the gates are closed.

5.5.2.5 Structural

See the general discussion for the state of structural design in paragraph 5.1.2.5. The critical foundation design elements are discussed in paragraph 5.5.2.4 immediately above.

5.5.2.6 Mechanical

The function of the pumping station is to lift water from the canals to the lake. The principle piece of machinery to do this is the pump. High capacity, low head pumps are essentially large propellers in a tube. For this application there are two principle types of pumps defined by the orientation of the propeller. The propeller can be installed horizontally or vertical. The horizontal types are similar to the Woods Screw Pumps which are extensively used in the older pumping stations.

The horizontal pumps are installed horizontally on an operating floor above the maximum canal level. As such, the propeller is above the water surface. To operate the pump, a vacuum is used to extract air out of the pump and pump discharge piping until the propeller is submerged. Once the propeller is submerged, the pump can be turned on and the pump will complete the filling of the discharge pipeline and establishing a siphon discharge. The major advantage of the horizontal pumps is that the pump bearings and propeller are located above the canal and the easily accessible for maintenance. The pump can actually be started before the propeller becomes fully submerged permitting a low startup torque which minimizes engine generator sizing. The major disadvantage is that the pumps need to be primed by a vacuum system. Due to the volume of air needed to be evacuated, it can take 10 to 15 minutes to get the pump started.

The vertical pump has the propeller mounted down below the minimum canal water surface level. Like the horizontal screw pump vertical pumps are also used extensively in the Parish. As a result the pump design, the pump is self-priming and can start pumping within seconds of a start command which is a significant advantage in controlling pumping units when pumping stations are located in series. Also with this design, the motors are located on top of the pump and out of any danger of being damaged by flooding. The major disadvantage is that the propeller is below the water surface and that any major maintenance requires fully disassembling the pump. Also a disadvantage is that the pump starts under load and has a high startup toque which can require over sizing engine generators.

Either type pump is applicable to this pumping station. For simplicity of this analysis, only vertical pumps which provide the maximum flood protection with the elevated motors are considered in the station. During detailed design the use of vertical, horizontal, or a combination of both should be considered.

For reverse flow protection, the discharge pipe from the pumps are elevated such the invert of the pipe at the highest point is at or above the floodwall elevation so that reverse flow through the pump is not likely. The discharge pipe is then brought down below the minimum lake level forming a siphon. A siphon discharge permits recovering the energy so that when normally pumping the pumps only see the difference between the canal elevation and the lake. A vacuum breaker is provided at the highest point in the discharge pipe to permit breaking the siphon when the pump stops. For added protection, sluice gates can be added to the discharge pipe for protection against reverse flow. To minimize submergence and hydraulic losses through the station, a formed suction inlet was used in the analysis. A typical cross-section of the pumping station is attached to Appendix D – Mechanical.

The pumping units can be driven by either electric motor with electric generator backup or direct driven by engines. In the final design, there may be a combination of drivers in the pumping station. Direct driven engines are cheaper since they eliminate the engine generator and motor. Motor driven pumps are quieter and more efficient. A determining factor may well be the ability or willingness of the power company to build power lines and reserve generating capacity for pumping units which may only occasionally be operated. A detailed study should be done during design to determine the optimum combination of electric driven motors with engine generator backup or direct engine driven pumps. For the purpose of this analysis approximately 60 percent motor driven pumps and 40 percent engine driven pumps are assumed.

A polling of pumping manufacturers indicated that the maximum practical size of pumping units is roughly 1,000 cfs. This is limited by the physical size of the equipment and the ability to move the equipment along major roadways. 1,000 cfs also matches up with the largest pumping stations in the major feeder pumping stations so was chosen as the main pumps in the new pumping stations. The existing pumping stations also have a number of smaller pumps. Smaller pumps permit pumping lower flows without having frequent starts and stops and also provided the ability to match flows when pumping stations operate in series. Therefore combinations of 1,000, 500, and 250 cfs pumps were selected at as the primary capacities. Using only three sizes will permit the sharing of parts between the pumping stations.

For Option 1, the screens are optional as there is no un-screened inflow to the canal downstream of the major pumping stations however screens ahead of the pumping station inlets will smooth the inflow to the pump and have a hydraulic function. Based on the length of canal and the potential for additional debris to enter canal, a trash rack system is included in the costing.

There are a number of additional mechanical systems required for operation of the pumping station. All major pumps require a clean source of water for bearing lubrication. This can be from the water system. However, based on experience during Katrina in which the water system failed, a secondary source of water should be provided. There are two sources available, either canal water or well water. Because the water to the pumps needs to be of high quality, well water is being considered for the pumping station. Canal water can be used but requires a high level of treatment to remove abrasives. Because of the size of the equipment in the facility, the facility should include overhead crane, lay down space, truck loading access, and workshop areas. Whether motor driven or engine pumps are used, there will be significant opening for ventilations. All inlet air vents should be shrouded to inhibit the entry of wind blown water.

Engine driven pumps have additional mechanical considerations including engine and gear reducer cooling systems, fuel system, starting air systems, lubricating oil and waste oil systems. In addition, engines require both exhaust air and combustion air systems.

Pumping station hydraulics are critical to successful operation of the pumping station to achieve maximum hydraulic performance. A physical model test of the pumping station including canal entrance, screens, and pump inlet, and discharge siphon pipe must be conducted as a follow-on effort to ensure correct sizing and configuration.

For the purpose of this study, the following combination of vertical pumping unit capacities was chosen. In addition, a combination of direct drive diesel engines and diesel generators are assumed to provide an approximate 60 percent electric motor drive and 40 percent diesel engine drive ratio.

Pump Driver Type		
Pump Number	**Pump Capacity cfs**	**Driver Type**
1	1000	Direct Drive Engine
2	1000	Direct Drive Engine
3	1000	Direct Drive Engine
4	1000	Direct Drive Engine
5	1000	Motor on Generator
6	1000	Motor on Generator
7	1000	Motor on Generator
8	1000	Motor on Grid or Generator
9	500	Motor on Grid or Generator
10	250	Motor on Grid or Generator
11	250	Motor on Grid or Generator

Fuel storage capacity for the pump station was selected at 4 days of full pumping capacity. Based on this duration and usage rate, the anticipated fuel storage required is slightly over 60,000 gallons. Assuming standard 12,000 gallon double wall fuel storage tanks, a minimum of 5 tanks will be required.

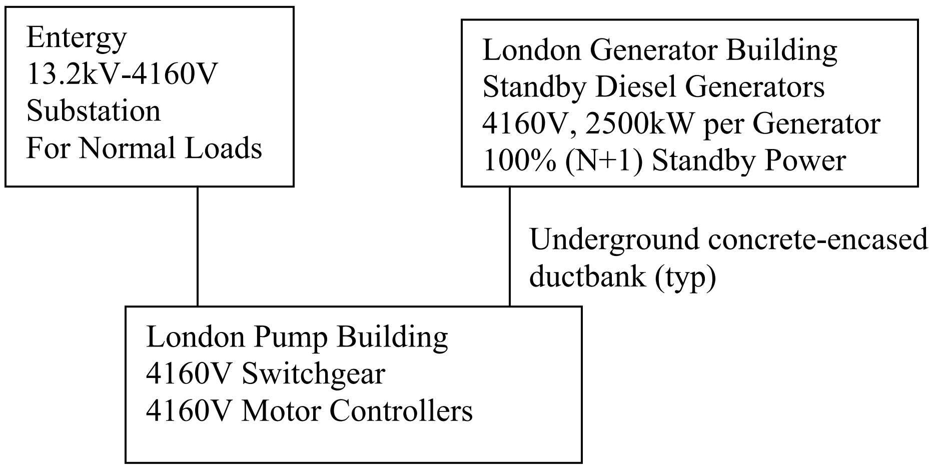
The estimated pump ratings are as follows:

Pump Rating			
Capacity, cfs	1,000	500	250
Bowl Head, ft	13	13	13
Pump Speed	162	227	321
Engine Rating, bhp	2100	---	---
Motor rating, hp	1750	900	500

In the Mechanical Appendix D are representative pump performance curves submitted by the manufacturers. The curves are presented as typical curves only as the required pump rating have evolved during the study and the pump ratings on the curves will differ slightly from the latest hydraulic requirements.

5.5.2.7 Electrical

Option 1 uses existing pump stations DPS 3and DPS4 along with a new pump station at London Avenue Canal. Utility power, from Entergy, will only be supplied for the normal pump loads (see the Pump Driver Schedule below for pump utilization). The incoming utility service will not be sized to accommodate the storm event pump loads.



One hundred percent of the storm event electric-driven pump loads will be supplied from standby generators dedicated to the London Avenue Pump Station.

The standby generators will utilize an N+1 design such that if a generator goes off line or one is down for maintenance, the full pump station load will still be supplied by standby power. The utility service, standby generators, and pump motors will all operate at 4160 volts. All electrical distribution circuits will be routed underground in concrete-encased ductbank. Below is a table to show the pump driver schedule.

Option 1 - London Canal – Pump Driver Schedule						
Pump cfs	**Driver bhp**	**Motor hp Nameplate**	**Motor hp Load**	**Source***	**Utilization Grid**	**Ind**
1000	1241			Engine		1%
1000	1241			Engine		1%
1000	1241			Engine		1%
1000	1241			Engine		1%
1000	1052	2750	1800	Standby Generator		1%
1000	1052	2750	1800	Standby Generator		1%
1000	1052	2750	1800	Standby Generator		1%
1000	1052	2750	1800	Grid/Generator	1%	1%
500	526	1500	900	Grid/Generator	10%	1%
250	263	750	450	Grid/Generator	50%	1%
250	263	750	450	Grid/Generator	100%	1%
9000			**9000**	**Totals**		

Based on the pump driver information above, the next table below indicates the electrical equipment sizes used for this option. See the appendices for all cost information.

Option 1 – London Canal Pump Station - Electrical	
Electrical Equipment	**Quantity/Capacity**
Utility Substation	1 – 5000kVA
Generators	6 – 2500kW
Generator Bldg Size	148' x 80'
Pump Bldg Switchgear	2 – 2000A
Total Pump Station Load	9.5 MVA, 1313A
Pump Station Load on Utility	4.1 MVA, 563A

5.5.2.8 Environmental

Environmental Site Assessment (ESA)

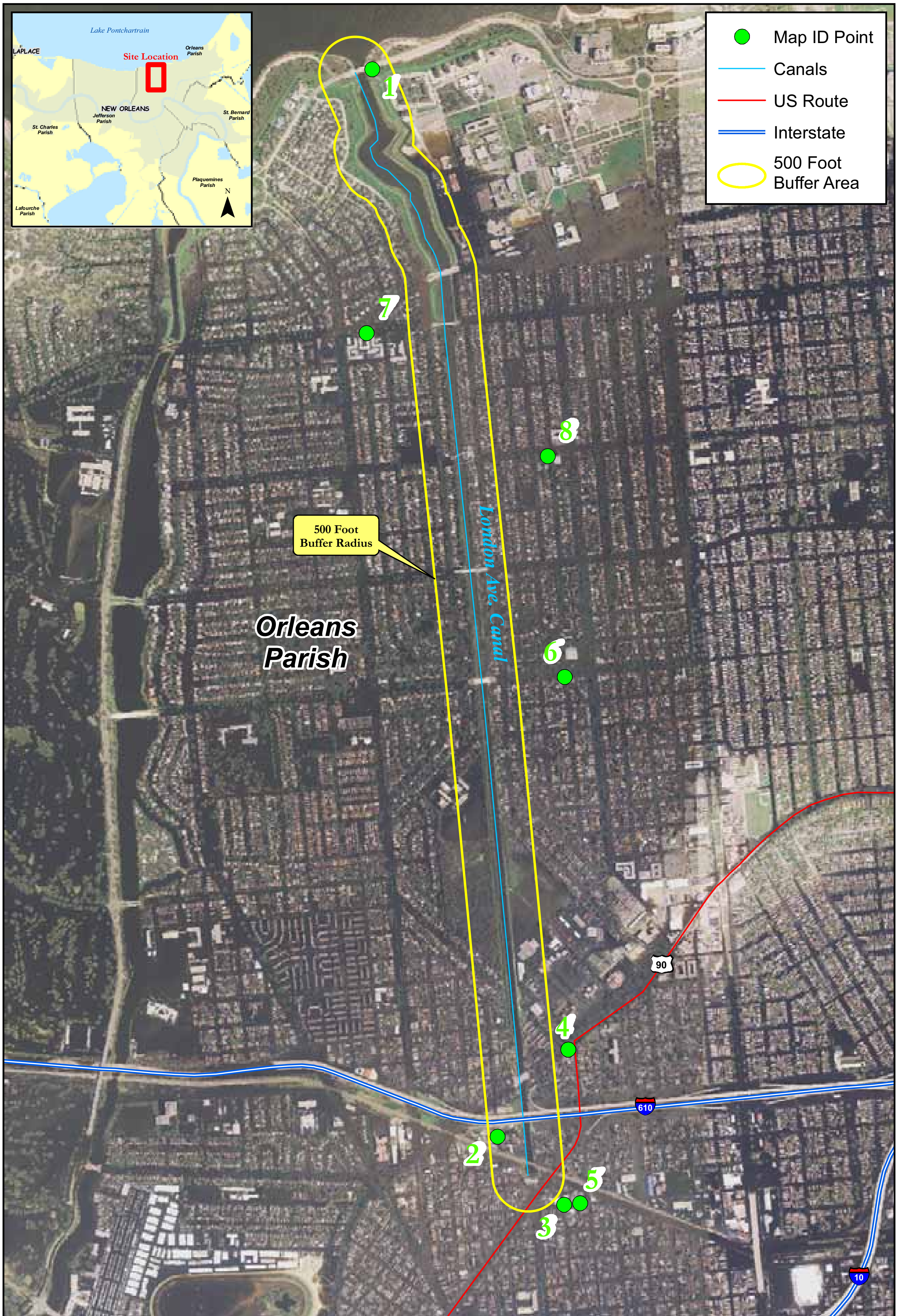
An environmental database search report of the area within a 0.25-mile radius of the canal route from the existing pump station to the mouth of the canal (i.e., 0.5 mile total width of the search area across the canal) was obtained to identify sites, incidents, conditions, etc. within the search corridor that may contain or formerly contained hazardous, toxic or radioactive waste (HTRW). These reports were utilized during in field observations to verify sites within the potential construction zones. Figure 5.5.2.8-1 presents the mapped sites within the 0.25-mile radius around the London Avenue Canal that are tracked on various environmental databases maintained by either the U.S. Environmental Protection Agency (USEPA) or the Louisiana Department of Environmental Quality (LDEQ).

Although the database search was requested for the standard radius of 0.25 mile (the search radius specified under American Society for Testing and Materials (ASTM) Standard E1527-05), particular interest was given the 500-foot (ft.) radius around the canal as a buffer zone for construction activities to take place in connection with the proposed project. Figure 5.5.2.8-1 provides the mapped 500-ft. radius around the London Avenue Canal. Located within the 500-ft. radius are two sites, map ID numbers 1 and 2. These sites are identified below along with their measured distance from the centerline of the canal:

Map ID	Site Name	Address	Distance
1	University of New Orleans	Elysian Fields at Lake Shore Drive	158 ft
2	Kingsmill Auto Service	1732 Benefit Street	370 ft

Site numbers 1 and 2 are on the database for containing underground storage tanks (the UST database). Because no problems at these sites have been reported, they are not considered a potential problem for the proposed project at this time. Seven unmappable sites were reported for the London Avenue Canal database search. However, these sites were evaluated for their potential to pose a threat to the proposed project. In consideration of the database on which they are listed, type of facility, and whatever location information is given, none of these seven unmappable sites are considered a significant concern for the construction of the proposed project.

A field site reconnaissance was conducted to determine whether any sites not identified through the database search process existed within the immediate project area that



London Avenue Canal Detail
Map ID Points
Orleans Parish, Louisiana

Note: Background image is post-Katrina USACE Aerial Photograph (September, 2005). Map ID Points provided by Banks Information Solutions, Inc. Other data not derived from registered survey and should be considered approximate.

0 500 1,000 2,000 Feet

1:15,000

GEC

Figure 5.5.2.8-1

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could potentially affect project design plans. The area surrounding the London Avenue Canal is almost entirely residential or University of New Orleans property. No visible signs were noted that would indicate the presence of HTRW in quantities that would warrant additional investigation.

The addition of diesel powered pumps at the control structure near the mouth of the canal would constitute new sources of air emissions and noise for area residents. The air emissions would require permitting through the LDEQ and noise mitigation would likely be required to be incorporated into the design plans for the facility. These issues would be addressed following final selection of pump and power design.

Because there is very limited recreational use of the Orleans Avenue Canal, for the most part recreational constraints would not be an issue. There is some recreational fishing at or near the mouth of the canal and this may be, at least temporarily disrupted during the construction of the control structure, but the numbers of fishermen appear to be small and intermittent.

The entire area of New Orleans and Lake Pontchartrain is within the Louisiana Coastal Zone, therefore a Coastal Zone Consistency determination will have to be conducted and submitted to the Louisiana Department of Natural Resources for concurrence.

Sediments

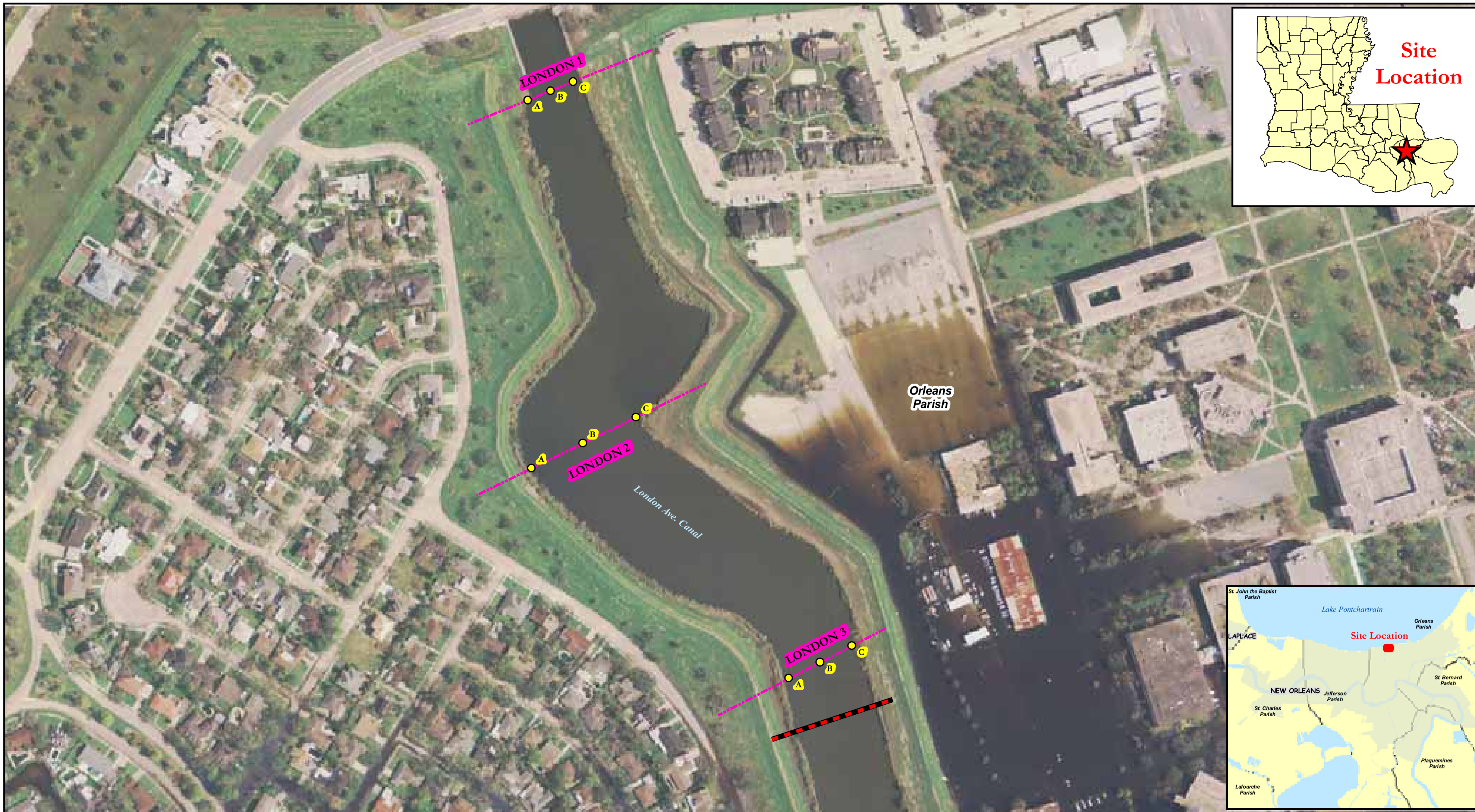
Because the discussion for sediment quality study covering all three canals has been discussed in some detail under Section 5.1.2.8, please see that section (sediments) for overall study information. See Figure 5.5.2.8-2 for the locations of samples taken in the London Avenue Canal. The following is sediment quality information specific to the London Avenue Canal:

London Avenue Canal Specific Findings

Sample LONDON 1 exhibits no compounds exceeding RECAP standards with the exception of the blank contamination noted below. Sample LONDON 2 exceeds standards for lead by 85 mg/kg (85 percent), TPH-DRO by 405 mg/kg (623 percent), and TPH-ORO by 940 mg/kg (522 percent). Sample LONDON 3 exceeds standards for TPH-DRO by 356 mg/kg (548 percent) and TPH-ORO by 850 mg/kg (472 percent).

Volatile organic blank contamination was also noted in the trip blank for the London Avenue Canal. Falsely elevated concentrations of trichloroethene are noted in the volatile organics analysis for all three composite samples. These falsely elevated volatile detections are also reflected in the TPH-GRO results.

None of the analyzed compounds that are regulated by RCRA are present in the London Avenue Canal TCLP samples in concentrations exceeding RCRA



London Avenue Canal
CIH Investigation
Orleans Parish, Louisiana

Note: Post-Katirna Color Aerial Photography provided by 3001, Inc. through the USACE. Other data not derived from registered survey and should be considered approximate.

0 250 500 Feet

1:3,000

- Sample Point
- Transects
- Canal Closure Structure

Figure 5.5.2.8-2

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July 2006

standards. TPH-DRO and TPH-ORO were detected in the TCLP leachate in the LONDON 2 sample, and TPH-DRO was detected in the LONDON 1 and LONDON three samples. TPH-GRO was also detected in all three samples, possibly due to volatile organic blank contamination.

For additional information, please see Final Report, Orleans Avenue, London Avenue and 17th Street Outfall Canals, Certified Industrial Hygienist Investigation, Orleans Parish, Louisiana, March 2006, U. S. Army Corps of Engineers, New Orleans District, New Orleans, Louisiana.

Wetlands

National Wetland Inventory (NWI) maps were consulted for jurisdictional wetlands in the proposed project area. See Figure 5.1.2.8-3 for a presentation of the mapped jurisdictional wetlands in all three canals. The NWI maps show no mapped jurisdictional wetlands within the potential construction zone for the London Avenue Canal. However, field reconnaissance observations revealed a small amount of bank area inside the levee walls on both banks of the canal north of Robert E. Lee Blvd. that may contain wetlands. Changes in tidal influence would not likely significantly affect the wetlands along the banks of the London Avenue Canal as there does not presently appear to be any tidal influence on these wetland areas. Permits from the USACE would be required prior to disturbance of these areas, if they are determined to be jurisdictional wetlands.

Protected Species

Please see discussion in Section 5.1.2.8.

Cultural Resources

See discussion in Section 5.1.2.8.

Wetlands

See discussion in Section 5.5.2.8.

Protected Species

See discussion in Section 5.1.2.8.

Cultural Resources

See discussion in Section 5.1.2.8.

5.5.2.9 Constructability

The conceptual designs used throughout this project are constructible using conventional techniques. For London Canal, Option 1 there are no channel modifications. All construction is focused on the pump station. For all location alternates, the temporary gate and structure would be demolished after the new pump station becomes operational. Two major approaches can be taken to construct the new pump station in the Orleans Canal.

The first construction concept is based on construction of a sheet pile cofferdam enclosing the entire pump station. The cofferdam enables the contractor to construct the pump station in the dry. To accomplish this, a bypass channel must be constructed for Alternate Location B. Location alternates A and C are constructed entirely outside the existing canal and do not require a bypass channel. Figures 5.5.2.9-1, 5.5.2.9-2 and 5.5.2.9-3 present the cofferdam and bypass concepts.

The bypass channel for Alternate B is formed with parallel sheet pile walls and is sized to pass the flows for which the temporary gate/pump structure is designed. A temporary bridge is required for Lake Shore Drive. This could be a precast structure set over the bypass channel. Once the pump station is complete the cofferdam is removed and the bypass channel is dammed and filled.

Construction of the concrete liner walls for the intake and discharge channel transitions can be constructed in the wet at the same time the pump station is being built for all three alternates.

As can be seen on the figure there are some problems with this concept for location Alternate B. At this site the best location for the bypass channel places it near residential property and creates a breach in the existing levee. The sheet pile forming the west side of the channel would have to be configured to provide flood protection to the adjacent property. The contractor would also have to provide a temporary bridge for Lake Shore Drive. The cost of constructing the bypass channel is significant for this location alternate.

The second concept for location Alternate B is to construct the pump station in two parts. The substructure for one half would be constructed first and would house the gates. A cofferdam would be set around just the first half, allowing the existing canal to pass flow with the constriction imposed by the cofferdam. The gates would be set in the open position. Upon completion of the gate substructure, the cofferdam would be moved to the other half of the structure. Flow would be redirected to pass through the new gates. The second half of the substructure could then be constructed followed by the complete super structure.

The second concept also has some disadvantages. The construction of the pump station in two halves is much more complicated than building it all as a single unit. The design would have to account for the connection of the two halves. Coordination and sequencing would be significant issues. The cost of constructing the pump station in two halves would be higher than being able to construct it in as a unified whole.

CONCEPTUAL DESIGN STUDY
PERMANENT FLOOD GATES AND PUMP STATION
LONDON AVENUE – OPTION 1A
CONSTRUCTION CONCEPT
FIGURE 5.5.2.9-1

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CONCEPTUAL DESIGN STUDY
PERMANENT FLOOD GATES AND PUMP STATION
LONDON AVENUE - OPTION 1B
CONSTRUCTION CONCEPT
FIGURE 5.5.2.9-2

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CONCEPTUAL DESIGN STUDY
PERMANENT FLOOD GATES AND PUMP STATION
LONDON AVENUE – OPTION 1C
CONSTRUCTION CONCEPT
FIGURE 5.5.2.9-3

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The bypass concept is illustrated here for Alternate B and used as a basis for cost estimating to be consistent with the other eight canal options. The additional cost of the bypass canal may be more expensive than the incremental cost for constructing the pump station in two halves. The design build contractor would be permitted chose either of these concepts or innovate a third concept of his own if it would reduce project cost.

Location alternates B and C also require the construction of a breakwater. The breakwater could be constructed at any time but must be complete before putting the new pump station in operation. It would be best to complete the breakwater early in the construction because of the additional protection it provides.

5.6 Option 2 – London Canal

5.6.1 Alternative Approaches

Three location alternatives were considered for Option 2 on the London Avenue canal. The merits of each were evaluated and discussed in detail in the Civil/Site section. For the purposes of this study, Alternative A was chosen as the location to base costing and other engineering considerations.

5.6.2 Engineering Considerations

5.6.2.1 Civil/Site

The London Avenue Pump Station for Option 2 is anticipated to be 300 feet long by 165 feet wide. The total station width includes a 45 foot inlet works including trash screens, an 80 foot pump station building housing pumps and motors, and a 40 foot outlet works. Finish grade for the Generator and Tank Farm Complex is always approximately +16.0 elevation.

- The **Alternative A Pump Station Layout** for Option 2 is as shown in Exhibit 5.6.1.A, Appendix C.

General Location and Description – The horizontal location of the pump station is identical to Option 1 for Alternative A. Changes from Option 1 to this Option 2 layout all result from the effects of the deeper canal and correspondingly deeper pump station inlet elevation, as described below.

Right- of-Way Acquisition - Right-of-way acquisition and temporary construction easements required for Option 2 are essentially unchanged for Option 2.

Demolitions – Demolitions under this Option 2 are unchanged from those described in Option 1.

Earthwork - Unlike Option 1, which requires localized earthwork only at the pump station facility, Option 2 requires significant canal excavation along the entire length of the London Avenue Canal. The basis of this anticipated canal excavation is determined based on the results of the canal hydraulic analysis described in Section 5.6.2.3 and the geotechnical slope stability analysis described under Section 5.6.2.4. Note that the anticipated canal improvement cross-section consists of providing an added rectangular section in the existing canal invert, either as sheet pile walls or a concrete "U" channel section, as shown elsewhere. This approach maintains canal construction within the existing canal right-of-way. Laid back slopes on this London Avenue Canal are not recommended for further consideration, due to the extremely flat side-slope required (5:1), and the correspondingly large volumes of right-of-way acquisition and channel excavation that would result. Further analysis of the effects on right-of-way and excavation quantities brought about by laid back slopes is included in Appendix C.

Channel Transitions – Channel transitions for this Option 2 layout are similar to those described under Option 1, simply increasing in size due to the increased channel depth.

Erosion Protection – Erosion protection requirements for Option 2 are essentially unchanged from those required under Option 1.

Generator Building and Tank Farm Complex - The Generator Building and Tank Farm Complex represent an area of some change from Option 1 to Option 2, due to the substantially increased head the pump station must overcome. The Generator Building is anticipated to be approximately 80 by 200 feet, and each of thirteen 12,000 gallon fuel tank pads is anticipated to be approximately 12 by 28 feet in size. A 20 by 40 foot electrical substation is also anticipated. The complex also includes parking, general staging and storage space, all concrete paved, and including sidewalk and local site storm drainage features. Utilities will include potable water service, sanitary sewer and natural gas, all connected to the station from existing utilities available within several hundred feet of the proposed site. The paved complex area is also anticipated to be enclosed by a chain link security fence, with minor landscaping improvements. All non-paved areas will be seeded to re-establish healthy turf for aesthetics and erosion control.

Summary – Similar to Option 1, this layout is attractive for its protection of the pump station from lake surge effects, resulting in no need for a major breakwater structure. The location requires right-of-way acquisition in a residential area, but acquisition appears to be property developed only with parking areas and other relatively low value improvements. Further, it is attractive since no significant construction sequencing is required to maintain canal flow during the construction duration.

- The **Alternative B Pump Station Layout** for Option 2 is as shown in Exhibit 5.6.1.B, Appendix C. As stated for the Option 1 Layout, this pump station location is not recommended for further consideration.

- **The Alternative C Pump Station layout** for Option 2 is as shown in Exhibit 5.6.1.C, Appendix C.

General Location and Description - The horizontal location of the pump station is identical to Option 1 for Alternative A. Changes from Option 1 to this Option 2 layout all result from the effects of the deeper canal and correspondingly deeper pump station inlet elevation, as described below.

Right- of-Way Acquisition - Right-of-way acquisition and temporary construction easements required for Option 2 are essentially unchanged for Option 2.

Demolitions – Demolitions under this Option 2 are unchanged from those described in Option 1.

Earthwork - Unlike Option 1, which requires localized earthwork only at the pump station facility, Option 2 requires significant canal excavation along the entire length of the London Avenue Canal. The basis of this anticipated canal excavation is determined based on the results of the canal hydraulic analysis described in Section 5.6.2.3 and the geotechnical slope stability analysis described under Section 5.6.2.4. Note that the anticipated canal improvement cross-section consists of providing an added rectangular section in the existing canal invert, either as sheet pile walls or a concrete "U" channel section, as shown elsewhere. This approach maintains canal construction within the existing canal right-of-way. Laid back slopes on this London Avenue Canal are not recommended for further consideration, due to the extremely flat side-slope required (5:1), and the correspondingly large volumes of right-of-way acquisition and channel excavation that would result. Further analysis of the effects on right-of-way and excavation quantities brought about by laid back slopes is included in Appendix C.

Channel Transitions – Channel transitions for this Option 2 layout are similar to those described under Option 1, simply increasing in size due to the increased channel depth.

Erosion Protection – Erosion protection requirements for Option 2 are essentially unchanged from those required under Option 1.

Generator Building and Tank Farm Complex - The Generator Building and Tank Farm Complex represent an area of some change from Option 1 to Option 2, due to the substantially increased head the pump station must overcome. The Generator Building is anticipated to be approximately 80 by 200 feet, and each of thirteen 12,000 gallon fuel tank pads is anticipated to be approximately 12 by 28 feet in size. A 20 by 40 foot electrical substation is also anticipated. The complex also includes parking, general staging and storage space, all concrete paved, and including sidewalk and local site storm drainage features. Utilities will include potable water service, sanitary sewer and natural gas, all connected to the station from existing utilities available from just over 1,000 feet from the proposed site. The paved complex area is also anticipated to be enclosed by a chain link security fence, with minor landscaping improvements. All non-paved areas will be seeded to re-establish healthy turf for aesthetics and erosion control.

Summary – As under Option 1, this Option 2 layout is attractive for its relatively minimal right-of-way acquisition requirements. Constructability is a mixture of positives and negatives. Constructing the pump station building in one phase is positive; however, construction in the lake offers other complications that are costly to overcome. Further, a major breakwater structure and significant plant armoring is required.

- **Existing Pump Station Demolition and Bypass.** Option 2 requires the abandonment of an historic inland pump station. In general, the historic elements of the pump station is to be preserved, while allowing the required canal flows to bypass the historic elements that remain, flowing into the newly degraded canals downstream of each existing pump station. Bypass of this pump station requires further development based on identification of the historic elements of the structure. In general, the demolition and bypass will be conducted in similar fashion as described for 17th Street canal.

5.6.2.2 Bridges and Utilities

Bridges

As discussed herein, the existing bottom the canal being approximately -12 (NAVD88) and the new bottom of the channel for Option 2 being approximately -26 (NAVD88) at the pump station. It is the new channel section that may significantly affect some of the bridges. Because of the lowering of the canal, weak soil conditions and constructability under these bridges, significant impact and cost are expected. Further investigation should be made and preliminary design developed for bridge modification as replacement of these structures would be very expensive. For the purposes of this report concrete box sections of equal hydraulic capacity are sunk between the support bents. This technique is constructable, accommodate the slope stability, while providing the required hydraulic capacity. For purposes of this report, modification of the bridges under Option 2 utilize the box culvert techniques.

The Interstate 10 Bridge located just downstream of Pump Station No. 3 is virtually unaffected by Option 2 because of the span lengths, foundation design and roadway geometrics.

For the Gentilly Boulevard Bridge, Mirabeau Avenue Bridge, Filmore Bridge, Robert E. Lee Boulevard Bridge, and Leon C. Simon Boulevard Bridge. The Option 2 canal section will require bridge modifications as described above. The Robert E. Lee Boulevard Bridge as well as the Southern Railroad Bridge would require replacement.

As previously stated, the Lakeshore Drive Bridge is affected by the pump station site location as well as the Option 2 canal section. For pump station layout alternates A and B, the Lakeshore Drive Bridge is unaffected since the pump station is located upstream from the bridge. Site location C will require the bridge to be replaced.

Utilities

The utilities studied in Option 2 are underground or pile supported water, sewer, drainage, electric (transmission and primary), telephone cables, fiber optic cables, and gas. Entergy supplies electricity in the vicinity of this canal, however, their overhead crossing primary and secondary lines entirely span the floodwalls. BellSouth owns the telephone and fiber optic cables in the vicinity of this project. In Option 2, the existing utilities impacted by construction in the vicinity of the London Avenue Canal are those utilities impacted by deepening the canal within the floodwalls from Pump Station No. 3 to the new pump station in the vicinity of Lake Pontchartrain.

The London Avenue Canal is located completely within the boundaries of Orleans Parish. There are several utilities that cross this canal that will need to be replaced or shored up. In addition to the aforementioned water main and gas line on/near the Lakeshore Drive Bridge, there are several significant crossing utilities. The water main and gas line on/near the Lakeshore Drive will need to be replaced at an estimated cost of $43,200. Between P.S. No. 3 and the Southern Railroad is a 48” diameter pressure drain pipe. This drainage pipe crosses the canal above grade on 14 pile supports within the pump station discharge basin. Each of the existing pile supports should be replaced by driving two longer piles and placing a support cap underneath the pipeline between the two new piles adjacent to the existing pile supports. The existing supports can then be cut off below the new channel grade or removed. In this manner, the drainage pressure pipe need not be replaced. Estimated cost equals $100,000.

South of the Mirabeau Avenue bridge is a buried crossing 8” diameter high pressure gas line. It will need to be replaced for channel deepening. Estimated cost equals $44,800. On the north side of this bridge a 12” water line is attached to the bridge deck and will need to be replaced with the bridge at a cost estimated to be $10,000.

Approximately 15’ south of Fillmore Avenue is a buried crossing 4” h.p. gas line. It will need to be replaced for channel deepening at an estimated cost of $40,000. 10’ south of the same bridge is a 50” diameter water main crossing at grade on four pile supports located in the canal. It is recommended these pile supports be replaced in the same manner previously discussed, leaving the existing water main in place. Estimated cost equals $60,000.

P.S. No. 4 is located at the foot of Prentiss Avenue on the east side of the canal. A telephone duct bank with 48k, 72k and 144k fiber optic cables enclosed crosses the canal in this vicinity on two pile supports located in the canal. 60’ north of this crossing, a large diameter drainage pressure pipe (~120” dia.) and a smaller drainage pressure pipe (48” dia.) cross the canal on one set of three pile supports located within the canal. All of these pile supports will need replacing. It will cost $150,000 for the telephone duct bank resupport and $100,000 for the two pressure pipes resupport.

One remaining conflicting utility is located on the north side of the Robert E. Lee Boulevard Bridge. A buried crossing primary electric feed is located about 10’ downstream of the bridge. This primary electric cable will need to be replaced to accommodate the canal deepening at an estimated cost of $54,000.

5.6.2.3 Hydraulic

For this option, the Replacement Pump Station would replace the existing S&WB pump stations that discharge into the canal. Existing pump station facilities would be modified as necessary so that drainage would bypass the pump stations and be conveyed within the London Avenue Canal to the Replacement Pump Station. The required flowline elevation within the canal would be much lower than for existing conditions, and significant modifications to the canal would be required to accommodate the lowered flowline. These modifications would generally involve lowering the canal invert elevation with a modified cross-section to allow the design canal discharge to flow by gravity between the existing pump station locations and the Replacement Pump Station.

The maximum allowable water surface elevation at certain points within the canal corresponds to the maximum allowable water surface elevation on the suction side of DPS 3 and DPS 4. For purposes of this study, a maximum suction side elevation of -9.9 ft. at DPS 3 and -10.4 ft. at DPS 4, both NAVD88, is used. This corresponds to the current "pumps on" operating condition and is therefore conservative with respect to maximum allowable canal water elevation. Because of the relative positions of DPS 3 and 4 along the canal, at design discharge the maximum suction side elevation at DPS 3 was found to be the controlling elevation.

Hydraulic Analysis

The hydraulic analysis performed for this Option 2 was similar to the analysis for Option 1 – Pumping Mode. The USACE-developed HEC-RAS hydraulic model was used, with inflows to the canal representing bypass flows at DPS 3 and DPS 4, as well at the potential new pump station to be located on the opposite side of the canal from DPS 4. Total design canal discharge at the Replacement Pump Station is 8,980 cfs, which includes the 1,000 cfs capacity of the potential new pump station. The hydraulic model was developed based on NGVD29 datum; therefore, subtraction of 0.5 feet from model elevations is necessary for conversion to NAVD88 datum.

A starting water surface elevation at the Replacement Pump Station of -13.0 ft. NAVD88 was selected for use. The modified canal section was considered to be a concrete-lined, rectangular cross-section with a bottom width of 100 feet and vertical side walls. The existing canal invert profile is variable in elevation, with an overall downward slope downstream from DPS 3. A modified canal invert profile with a similar overall downstream slope was considered, in order to approximate the profile configuration of the existing canal.

Using the selected starting water surface elevation and the rectangular canal cross-section, an iterative approach was used to determine the canal invert profile that would result in a maximum canal water surface elevation at DPS 3 equal to the maximum allowable suction side elevation at this pump station. The HEC-RAS model was run for several canal invert profiles, and the resulting canal water surface elevation at DPS 3 was determined for each case. Based on the results of this analysis, a canal invert elevation of -19.6 ft. at DPS 3 and uniformly sloping to El. -25.6 ft, NAVD88 at the Replacement Pump Station was determined to be required. The water surface profile within the canal for this flow condition is provided in Figure 5.6.2.3-1.

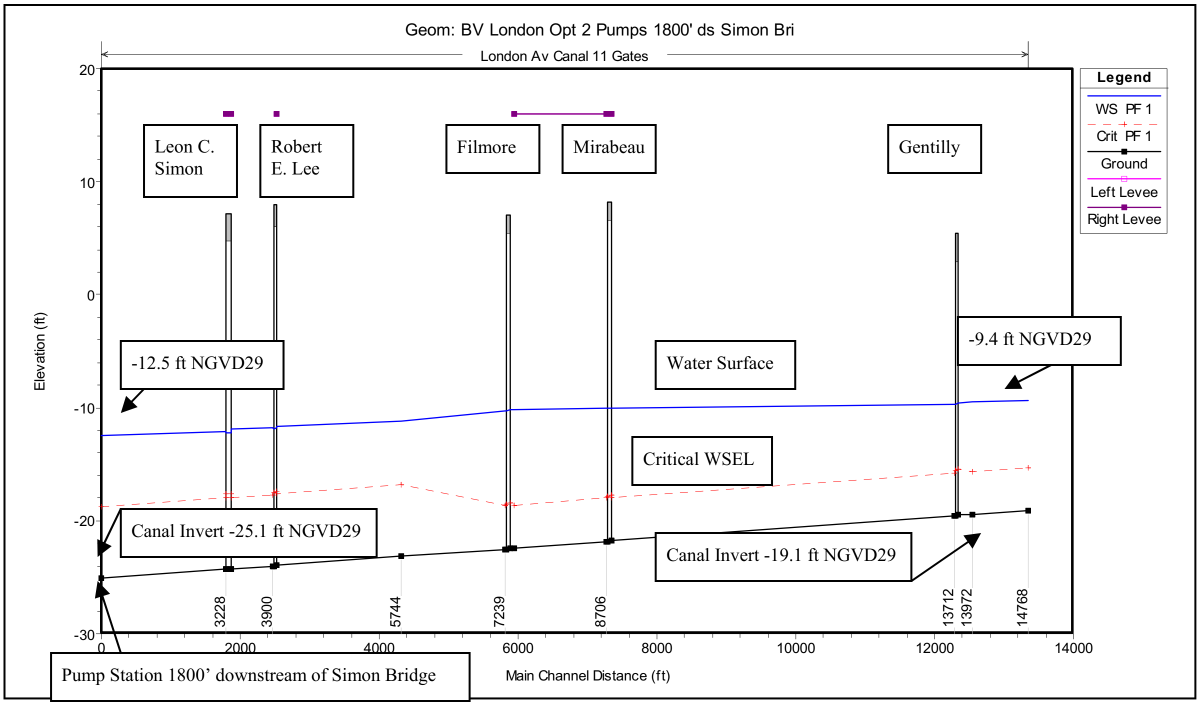


Figure 5.6.2.3-1. Water Surface Profile – London Avenue Canal (Option 2)

Numerous combinations of starting water surface elevation, canal cross-section geometry, and canal invert profile would result in the desired upstream canal water surface elevation. Consideration of such alternatives would be appropriate as part of an overall project evaluation comparing capital costs and annual operating costs, however this type of alternatives evaluation is not included in this study. The canal cross-section geometry that was selected represents a reasonable canal configuration for the given criteria.

5.6.2.4 Geotechnical

The typical stratification for the London Canal is taken from the IPET Report, Volume V. From the top down the stratification includes; Marsh Clay, Relic Beach Sand, and Bay Sound Clay. Pleistocene sand and clay strata are below the Bay Sound Clay. A typical representation of the canal geology as modified for the deeper canal is shown in Figure 5.6.2.4-1. The section is taken from the IPET report. Corresponding IPET report figures are included in Appendix B for reference.

The strength and physical properties used to characterize these strata are also taken from the IPET Report, Volume V, Appendix 7. Diagrams from the IPET Report are included in Appendix B to show the source of the strength parameters for this section. The parameters used in this study are shown in the figure above. The data and strength parameters are consistent with the IPET Report and are taken as conservative.

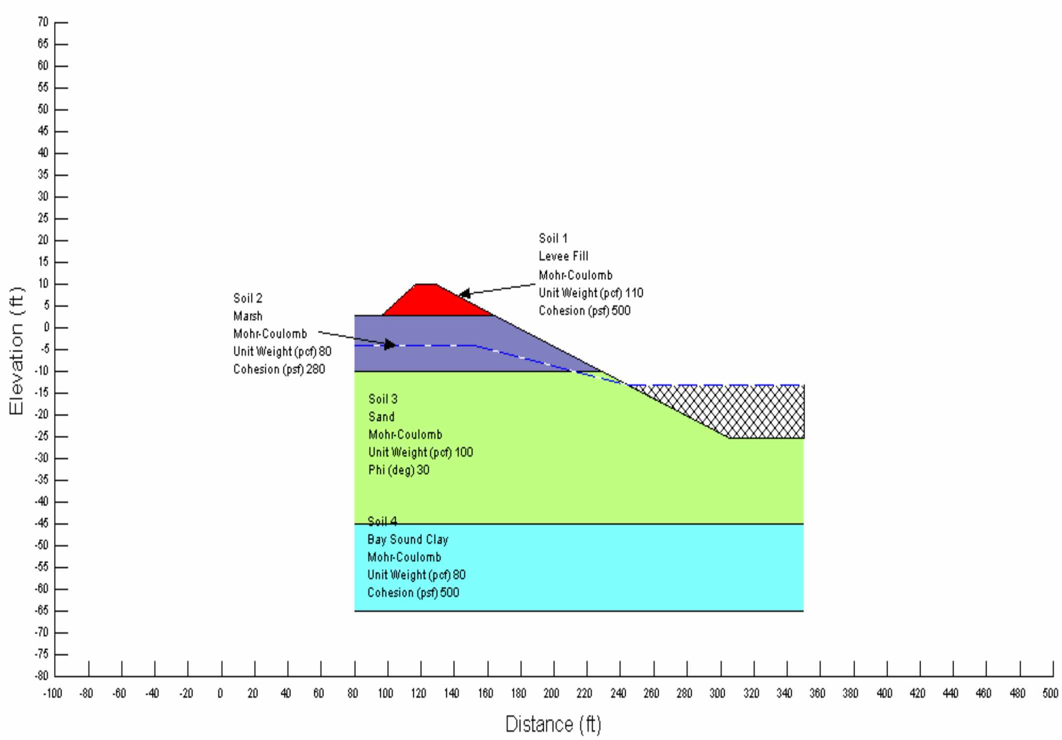


Figure 5.6.2.4-1 London Canal Geology – Option 2

<u>Canal</u>

One of the significant changes required for Option 2 is the lowering of the flow line and the resulting deepening of the canal. The existing bottom of the canal varies but is deepest at approximate elevation -13 ft (NAVD). The new bottom needs to be at approximate elevation -26 ft (NAVD).

Stability analysis of the deeper canal shows the slopes of the canal do not meet safety criteria (figures B-16, 17, 18, Appendix B). The slopes of the canal were flattened to determine the safe slope which will produce a factor of safety of 1.5. Slopes of 5h:1v produce the required minimum factors of safety (Figure B-10, Appendix B). When 5:1 slopes are projected out, significant amounts of additional property are required. Therefore flatter slopes are not practical and the design is modified to include a concrete liner for the deepened canal.

The lower flow line in the canal creates a potential recharging situation in which ground water from the adjoining properties would flow into the canal. The analysis of this threat is discussed in full in section 5.2.2.4. Since the analysis for is identical, it is not repeated here.

Pump Station Foundation

Evaluation of foundation issues for the Option 2 pump station at the London Canal are under way at the time of this writing. Preliminary stability evaluation has been completed and is included as file 2.1 in Appendix B. This stability analysis is a two-dimensional analysis of a typical section cut perpendicular to the long axis of the pump station. This section includes all driving and resisting loads. Analysis results are reported on a per foot basis representing the nominal one foot thickness of a two dimensional analysis. Conservative assumptions are made throughout so the two-dimensional analysis should represent conservative evaluation for this study. The issues identified for evaluation are described in the following paragraphs.

Sliding: The excavation for the intake basin will remove most of the material from the upstream side of the pump station. The pumps being considered for this study will require 14 ft of water to operate properly. The critical sliding case will be when the gates are closed and the downstream side is subjected to lake surge. Even though the lake surge is relatively short duration, the structure must be stable during the few hours it is present. The net pressure on the pump building is substantial. Several elements of the foundation design will contribute to or effect sliding resistance.

The thickness of the base slab is dictated by uplift considerations (see below). The slab will be entirely below grade so it will develop shear in the adjacent soil. At the anticipated depth of the base slab, this resistance will develop in the Relic Beach Sand. The sand has a moderate shear strength with phi = 35 degrees. Calculations demonstrate that passive resistance alone is not adequate to resist sliding. Deep soil mixing is being used at the temporary protection structure to substantially improve the foundation soils and may be considered for this pump station as well.

The weight of the structure will generate friction on the base of the slab. Because of the high uplift pressures, net weight will not be very high. This means the base friction will not provide much resistance for sliding. Base friction can be improved somewhat by soil modification but will never be the controlling factor unless weight is added to the structure explicitly for this purpose. The addition of weight to improve base shear is not efficient and will not be considered for this study.

Preliminary calculations demonstrate a deficiency in sliding resistance of approximately 54 kip/ft. This deficiency must be corrected to achieve a factor of safety of 1.5 for sliding. Design elements will have to be added to provide this additional resistance.

The greatest influence on sliding resistance will likely come from the foundation piling. Including lateral resistance into the design of vertical piles is one method of providing the additional needed lateral resistance. Another viable method of providing the resistance is the addition of battered piles. Battered piles are more efficient lateral resistance elements and will likely be used. In any case, soil modification can improve this resistance and will be considered as a possible supplemental measure.

Uplift: When the structure is subjected to lake surge, substantial uplift will develop on the base of the structure. The uplift is assumed to instantaneously reflect the lake surge pressures. Hydraulic conductivity of the Lacustrine Clay is somewhat low but the Beach Sand must be assumed to be hydraulically connected to the lake. Uplift pressure is calculated by assuming a linear variation from the lake head to the upstream head in the canal. If needed, this can be modified by installing a cut-off wall below the foundation. Cut-off walls can be effective but will be difficult to coordinate with a pile foundation anticipated for this application. For this study it will be assumed that no cut-off wall will be installed for the purpose of reducing uplift pressures.

Conservative design resists uplift forces with dead load. At this stage of the study the base slab is being sized to provide the necessary dead load to resist uplift with a factor of safety of 1.1. This requires a base slab of 11 ft thickness with bottom at elevation -38 ft (NAVD). It is recognized that the vertical piling supporting the structure will also provide uplift resistance. For this study the piling will be considered to provide supplemental resistance to raise the factor of safety above 1.5, but will not provide the principal resistance system. This is a conservative approach that ensures long term stability against uplift failures.

Underseepage: Underseepage is a potential failure mode with the combination of high heads and weak foundation soils. Since the base slab is 11 ft thick, the length of the flow path may be long enough to eliminate the threat of underseepage problems. Underseepage can result in the loss of the foundation material through piping beneath the foundation. Seepage calculations will be performed to check this failure mode. It is possible that a cut-off wall will be required.

Foundation Support: Preliminary calculations of overturning indicate the structure is not 100 percent in compression without the addition of tension elements. Tension elements with a capacity of 145 kip/ft will result in the required 100 percent base compression. The additional tension capacity is based on being able achieve a centroid of the tension elements at 2/3 the width of the base slab. Vertical tension piles will be assumed to develop the overturning stability.

The principle vertical resistance system for the pump station will likely be piling. This is common practice for the area when foundation soils are too weak for the structural loading. The strength of the soils can be improved by soil modification techniques and this will be considered. However the big issue for this structure will be settlement. Ground modification would have to extend to greater depths if it is desired to eliminate piling. For simplicity the structure will be founded on piling for this study. The concept design of the piling will ensure it will resist static vertical load when the gates are open and the eccentric loading produced by unbalanced water pressure when the gates are closed.

5.6.2.5 Structural

See the general discussion for the state of structural design in paragraph 5.1.2.5. The critical foundation design elements are discussed in paragraph 5.6.2.4 immediately above.

5.6.2.6 Mechanical

The function of the pumping station is to lift water from the canals to the lake. The principle piece of machinery to do this is the pump. High capacity, low head pumps are essentially large propellers in a tube. For this application there are two principle types of pumps defined by the orientation of the propeller. The propeller can be installed horizontally or vertical. The horizontal types are similar to the Woods Screw Pumps which are extensively used in the older pumping stations.

The horizontal pumps are installed horizontally on an operating floor above the maximum canal level. As such, the propeller is above the water surface. To operate the pump, a vacuum is used to extract air out of the pump and pump discharge piping until the propeller is submerged. Once the propeller is submerged, the pump can be turned on and the pump will complete the filling of the discharge pipeline and establishing a siphon discharge. The major advantage of the horizontal pumps is that the pump bearings and propeller are located above the canal and the easily accessible for maintenance. The pump can actually be started before the propeller becomes fully submerged permitting a low startup torque which minimizes engine generator sizing. The major disadvantage is that the pumps need to be primed by a vacuum system. Due to the volume of air needed to be evacuated, it can take 10 to 15 minutes to get the pump started.

The vertical pump has the propeller mounted down below the minimum canal water surface level. Like the horizontal screw pump vertical pumps are also used extensively in the Parish. As a result the pump design, the pump is self-priming and can start pumping within seconds of a start command which is a significant advantage in controlling pumping units when pumping stations are located in series. Also with this design, the motors are located on top of the pump and out of any danger of being damaged by flooding. The major disadvantage is that the propeller is below the water surface and that any major maintenance requires fully disassembling the pump. Also a disadvantage is that the pump starts under load and has a high startup toque which can require over sizing engine generators.

Either type pump is applicable to this pumping station. For simplicity of this analysis, only vertical pumps which provide the maximum flood protection with the elevated motors are considered in the station. During detailed design the use of vertical, horizontal, or a combination of both should be considered.

For reverse flow protection, the discharge pipe from the pumps are elevated such the invert of the pipe at the highest point is at or above the floodwall elevation so that reverse flow through the pump is not likely. The discharge pipe is then brought down below the minimum lake level forming a siphon. A siphon discharge permits recovering the energy so that when normally pumping the pumps only see the difference between the canal elevation and the lake. A vacuum breaker is provided at the highest point in the discharge pipe to permit breaking the siphon when the pump stops. For added protection, sluice gates can be added to the discharge pipe for protection against reverse flow. To minimize submergence and hydraulic losses through the station, a formed suction inlet was used in the analysis. A typical cross-section of the pumping station is attached to Appendix D – Mechanical.

The pumping units can be driven by either electric motor with electric generator backup or direct driven by engines. In the final design, there may be a combination of drivers in the pumping station. Direct driven engines are cheaper since they eliminate the engine generator and motor. Motor driven pumps are quieter and more efficient. A determining factor may well be the ability or willingness of the power company to build power lines and reserve generating capacity for pumping units which may only occasionally be operated. A detailed study should be done during design to determine the optimum combination of electric driven motors with engine generator backup or direct engine driven pumps. For the purpose of this analysis approximately 60 percent motor driven pumps and 40 percent engine driven pumps are assumed.

A polling of pumping manufacturers indicated that the maximum practical size of pumping units is roughly 1,000 cfs. This is limited by the physical size of the equipment and the ability to move the equipment along major roadways. 1,000 cfs also matches up with the largest pumping stations in the major feeder pumping stations so was chosen as the main pumps in the new pumping stations. The existing pumping stations also have a number of smaller pumps. Smaller pumps permit pumping lower flows without having frequent starts and stops and also provided the ability to match flows when pumping stations operate in series. Therefore combinations of 1,000, 500, and 250 cfs pumps were selected at as the primary capacities. Using only three sizes will permit the sharing of parts between the pumping stations.

Under Option 2 the pumping station should be provided with screens ahead of the pumping stations to protect the pumps for large solids as the screens at Pumping Station 3, 6, and 7 will be eliminated.

There are a number of additional mechanical systems required for operation of the pumping station. All major pumps require a clean source of water for bearing lubrication. This can be from the water system. However, based on experience during Katrina in which the water system failed, a secondary source of water should be provided. There are two sources available, either canal water or well water. Because the water to the pumps need to be of high quality, well water is being considered for the pumping station. Canal water can be used but requires a high level of treatment to remove abrasives. Because of the size of the equipment in the facility, the facility should include overhead crane, lay down space, truck loading access, and workshop areas. Whether motor driven or engine pumps are used, there will be significant opening for ventilations. All inlet air vents should be shrouded to inhibit the entry of wind blown water.

Engine driven pumps have additional mechanical considerations including engine and gear reducer cooling systems, fuel system, starting air systems, lubricating oil and waste oil systems. In addition, engines require both exhaust air and combustion air systems.

Pumping station hydraulics are critical to successful operation of the pumping station to achieve maximum hydraulic performance. A physical model test of the pumping station including canal entrance, screens, and pump inlet, and discharge siphon pipe must be conducted as a follow-on effort to ensure correct sizing and configuration.

For the purpose of this study, the following combination of vertical pumping unit capacities was chosen. In addition, a combination of direct drive diesel engines and diesel generators are assumed to provide an approximate 60 percent electric motor drive and 40 percent diesel engine drive ratio.

Pump Driver Type		
Pump Number	**Pump Capacity cfs**	**Driver Type**
1	1000	Direct Drive Engine
2	1000	Direct Drive Engine
3	1000	Direct Drive Engine
4	1000	Direct Drive Engine
5	1000	Motor on Generator
6	1000	Motor on Generator
7	1000	Motor on Generator
8	1000	Motor on Grid or Generator
9	500	Motor on Grid or Generator
10	250	Motor on Grid or Generator
11	250	Motor on Grid or Generator

Fuel storage capacity for the pump station was selected at 4 days of full pumping capacity. Based on this duration and usage rate, the anticipated fuel storage required is slightly over 150,000 gallons. Assuming standard 12,000 gallon double wall fuel storage tanks, a minimum of 13 tanks will be required

The estimated pump ratings are as follows:

Pump Rating			
Capacity, cfs	1,000	500	250
Bowl Head, ft	25.7	25.7	25.7
Pump Speed	162	227	321
Engine Rating, bhp	4100	---	---
Motor rating, hp	3500	2000	900

In the Mechanical Appendix D are representative pump performance curves submitted by the manufacturers. The curves are presented as typical curves only as the required pump rating have evolved during the study and the pump ratings on the curves will differ slightly from the latest hydraulic requirements.

5.6.2.7 Electrical

Option 2 removes existing pump stations DPS3 and DPS4 from service and therefore requires more power to achieve the same flow as Option 1. The general arrangement of electrical equipment remains the same, but larger electrical equipment is required to meet the increased load demands. Utility power, from Entergy, will only be supplied for the normal pump loads (see Pump Driver Schedule below for pump utilization). The incoming utility service will not be sized to accommodate the storm event pump loads. One hundred percent of the storm event electric-driven pump loads will be supplied from standby generators dedicated to the London Avenue Pump Station. The standby generators will utilize an N+1 design such that if a generator goes off line or one is down for maintenance, the full pump station load will still be supplied by standby power. The utility service, standby generators, and pump motors will all operate at 4160 volts. All electrical distribution circuits will be routed underground in concrete-encased ductbank. The Pump Driver Schedule below illustrates the increased power requirements of Option 2.

Option 2 - London Canal – Pump Driver Schedule						
Pump cfs	**Driver bhp**	**Motor hp Nameplate**	**Motor hp Load**	**Source***	**Utilization Grid**	**Ind**
1000	3206			Engine		1%
1000	3206			Engine		1%
1000	3206			Engine		1%
1000	3206			Engine		1%
1000	2719	4550	3825	Standby Generator		1%
1000	2719	4550	3825	Standby Generator		1%
1000	2719	4550	3825	Standby Generator		1%
1000	2719	4550	3825	Grid/Generator	1%	1%
500	1359	2250	1925	Grid/Generator	10%	1%
250	680	1250	950	Grid/Generator	50%	1%
250	680	1250	950	Grid/Generator	100%	1%
9000			**19125**	**Totals**		

Based on the pump driver information above, the next table below indicates the electrical equipment sizes used for this option. See the appendices for all cost information.

Option 2 – London Canal Pump Station - Electrical	
Electrical Equipment	**Quantity/Capacity**
Utility Substation	2 – 5000kVA
Generators	10 – 2500kW
Generator Bldg Size	220' x 80'
Pump Bldg Switchgear	2 – 3000A
Total Pump Station Load	20.1 MVA, 2789A
Pump Station Load on Utility	8.6 MVA, 1195A

5.6.2.8 Environmental

Environmental Site Assessment (ESA)

Because conditions/impacts and results for the ESA under Option 2 for the London Avenue Canal are identical to those discussed in Section 5.1.2.8, see that section for this discussion.

Sediments

Because sediment quality and findings for Option 2 for the London Avenue Canal are nearly identical to those discussed in Section 5.5.2.8, please see that section for most of this discussion. However, because this option contains the dredging of a large quantity of bottom sediments, costs will be significantly higher if disposal at a hazardous waste permitted land fill or on-site treatment is required. Transportation costs for disposal at a permitted land fill may render this option as cost prohibitive. On-site treatment (i.e., thermal treatment of organic contaminants) would most likely be the preferred method. Once treated, sediments could be disposed in a construction debris land fill or perhaps beneficially used.

Wetlands

See discussion in Section 5.5.2.8.

Protected Species

See discussion in Section 5.1.2.8.

Cultural Resources

See discussion in Section 5.1.2.8.

5.6.2.9 Constructability

The construction concepts developed for Option 2 for the London Canal are very similar to the Option 1 concepts. The big differences between Option 1 and Option 2 as they affect constructability are the deeper excavation required for the pump station and the channel modifications.

The concept of enclosing the entire new pump station in a cofferdam for all location alternates is still valid. The pump station will be larger in plan and deeper. The deeper excavation for the pump station will require a more robust design for the cofferdam. The cofferdam concept remains the essentially same as described for Option 1 in 5.5.2.9 and illustrated in figures 5.5.2.9-1, 5.5.2.9-2, and 5.5.2.9-3 above.

When constructing the bypass channel for Option 2 at location Alternate B, the bottom of the bypass channel only needs to match the existing channel bottom. Therefore the concept of the by pass channel is also the same as for Option 1. The same problems and costs are associated with construction of by pass channels for location Alternate B as described in Option 1.

The second concept of building the pump station in two parts to eliminate the need for a bypass channel for Alternate B becomes more complex. The Option 2 pump station does not include a gate. The first half of the pump station can be constructed by leaving half of the existing channel open. Construction of the second half will result in complete closure of the existing canal. Therefore the design of the first half would need to be modified to include gates adequate to pass the flows require for the temporary gate and pumps. This will make the structure larger and more expensive. Furthermore, the gates would only be used in the construction period and never required after that time. The gate adds a point of vulnerability during the working life of the pump station without adding a function

Again, the question becomes one of cost. The costs developed for this report include the construction of bypass channels. The design build contractor would be permitted chose either of these concepts or innovate a third concept of his own if it would reduce project cost. See figures 5.5.2.9-1, 2, and 3 for plan representations of cofferdam and bypass requirements.

Option 2 also requires modification to the existing canal by deepening it. Construction of the concrete liner for the canal must be accomplished in such a way as to permit flows in the canal. The quantity of flow to be passed is determined by the design of the temporary structure. There is no need to pass more flow than that structure can pass. Two concepts have been developed for construction of the concrete liner.

The first concept is illustrated in Figure 5.2.2.9-1 in the 17th Street section of this report. That conceptual diagram is valid for the London canal as well but is not repeated here. This concept minimizes constriction of the existing canal. Each wall of the concrete liner is constructed in the dry inside a cofferdam box. The box is then moved to the other side of the canal to construct the opposite wall. The box is moved to the next section and the floor of the liner is placed in the wet with tremie concrete. The cofferdam box is reused for each segment of the canal liner along the full length of canal to be modified. The length of the box can be adjusted to suit the design and schedule. Sheet pile cutoff walls are installed below each wall to improve stability and to provide a seepage barrier is required. (See the geotechnical discussion above). Soil anchors or deadman anchors may be needed to provide stability to the walls.

A second concept uses a larger box to enclose half of the canal. This concept allows construction of the wall and floor both in the dry. It ensures a better connection of the two elements and may result in elimination of the cutoff wall. The wall is completed in two steps instead of three. The chief disadvantage is the available cross-section of the canal for flow is cut in half. This may not be adequate.

A third concept has been discussed during development of the project as having been used successfully in the past. This concept provides for damming both ends of the section being built and dewatering for construction in the dry. If an event requires it, the dam can be breached or allowed to overtop so the event flows can be passed. This technique has been used to line canals in New Orleans. However the deepening of this canal makes this technique impractical. Pressure heads would be on the order of 25 feet.

6.0 ECONOMIC RESULTS SUMMARY

6.1 Economic Results Summary

Rough Order of Magnitude (ROM) costs have been generated for a variety of options and alternatives to provide adequate information to evaluate the economic considerations of this study. This cost estimate was performed with the intent of providing high level ROM costs that contain contingencies due to the inherent low level of detail typical at a concept study stage. The accuracy is adequate to allow comparison of the two options, but should not be used for decision support at the detail level. Cost detail for previous pump station work or the back-up for the estimate prepared for the Post Authorization Report was not available for comparison. The results of the additional studies identified in this report should be incorporated and factored into the cost estimate prior to using these costs for funding or ROM decision support. Additional detail and cost estimate back-up can be found in Appendix H.

6.1.1 Base Criteria Cost Estimate

In accordance with the scope, included are option costs for both options, broken down by major system components of pump station, canal, and breakwater. In addition to initial costs, a basic life cycle cost analysis (LCCA) was performed with the present worth and the annualized costs provided. LCCA used a six percent interest rate and a 50 years study period. The capital costs were generated based on pump station site locations of 17th Street Alternative C, Orleans Alternative A, and London Avenue Alternative A.

Plant Location	Initial Cost	Present Worth	Annualized Cost
Option 1			
17th Street Canal	$276,474,352	$424,200,000	$26,916,000
Pump Station	$197,804,992		
Canal	$0		
Breakwater	$78,669,360		
Orleans Canal	$65,830,534	$113,100,000	$7,177,000
Pump Station	$65,808,512		
Canal	$22,022		
Breakwater	$0		
London Canal	$133,372,008	$310,700,000	$19,712,000
Pump Station	$124,829,166		
Canal	$8,542,842		
Breakwater	$0		
Option 1 Cost:	$475,676,894	$848,000,000	$53,805,000

Option 2			
17th Street Canal	$687,904,602	$786,400,000	$49,894,000
Pump Station	$249,888,716		
Canal	$359,346,526		
Breakwater	$78,669,360		
Orleans Canal	$223,401,332	$254,900,000	$16,174,000
Pump Station	$75,237,316		
Canal	$148,164,016		
Breakwater	$0		
London Canal	$502,633,516	$573,600,000	$36,389,000
Pump Station	$157,228,610		
Canal	$345,404,906		
Breakwater	$0		
Option 2 Cost:	$1,413,939,450	$1,614,900,000	$102,457,000

6.1.2 Comparison of Base Criteria to Post Change Authorization Report Cost Estimates

The following table provides a side-by-side comparison of the costs generated as a part of this study with the costs presented in the Post change Authorization Report. The major system component breakdown of pump station, canal, and breakwater was intentionally used to allow comparison at this level. Note that the Option 1 costs compare favorably, but there is a substantial difference in the reported costs for Option 2. The majority of these differential costs are attributed to canal modifications.

Plant Location	**Base Criteria Initial Cost**	**Post Change Authorization**
Option 1		
17th Street Canal	$276,474,352	$206,000,000
Pump Station	$197,804,992	$206,000,000
Canal	$0	$0
Breakwater	$78,669,360	$0
Orleans Canal	$65,830,534	$115,200,000
Pump Station	$65,808,512	$115,200,000
Canal	$22,022	$0
Breakwater	$0	$0
London Canal	$133,372,008	$208,800,000
Pump Station	$124,829,166	$208,800,000
Canal	$8,542,842	$0
Breakwater	$0	$0
Option 1 Cost:	$475,676,894	$530,000,000

Option 2		
17th Street Canal	$687,904,602	$374,000,000
Pump Station	$249,888,716	$194,000,000
Canal	$359,346,526	$80,000,000
Breakwater	$78,669,360	$100,000,000
Orleans Canal	$223,401,332	$114,000,000
Pump Station	$75,237,316	$53,000,000
Canal	$148,164,016	$61,000,000
Breakwater	$0	$0
London Canal	$502,633,516	$232,000,000
Pump Station	$157,228,610	$139,000,000
Canal	$345,404,906	$93,000,000
Breakwater	$0	$0
Option 2 Cost:	$1,413,939,450	$720,000,000

6.1.3 Base Criteria Cost Estimate Plus Additional 5 Feet Lake Level

Scope paragraph 3.I.5 required an initial cost evaluation be performed to determine the additional cost if the lake criteria were increased 5' over the base criteria water level. The base criteria cost estimate was used as a starting point and differential cost for affected components were added to the base cost to develop the cost for the higher water level. As can be seen from the table, the capital cost increased marginally for the added requirement. The O&M cost will rise substantially due to the increased pumping head.

Plant Location	**Base Criteria Initial Cost**	**Base Plus 5 feet Initial Cost**
Option 1 Initial Costs		
17th Street Canal	$276,474,352	$310,225,492
Orleans Canal	$65,830,534	$71,288,724
London Canal	$133,372,008	$143,467,158
Option 1 Cost:	$475,676,894	$524,981,374
Option 2 Initial Costs		
17th Street Canal	$687,904,602	$718,844,712
Orleans Canal	$223,401,332	$227,344,992
London Canal	$502,633,516	$511,303,366
Option 2 Cost:	$1,413,939,450	$1,457,493,070

6.1.4 Site Alternatives Cost Estimate Comparison

Although each location and option utilized an alternative as the basis of cost, a table was developed to compare the relative cost of all alternatives. This was performed using the study basis cost estimate as the starting point and differential cost for components differing from the study basis were added or deducted to develop the cost. This comparison was performed on only initial costs.

Plant Location		Alternative A	Alternative B	Alternative C
Option 1 Initial Costs				
17th Street Canal		$218,355,874	$242,680,342	$276,474,352
Orleans Canal		$65,830,534	$148,572,096	$195,830,611
London Canal		$133,372,008	$197,397,741	$266,134,997
Option 1 Cost:		$417,558,416	$588,650,178	$738,439,960
Option 2 Initial Costs				
17th Street Canal		$612,969,949	$672,321,870	$687,904,602
Orleans Canal		$223,401,332	$308,349,414	$355,604,854
London Canal		$502,633,516	$566,960,817	$635,888,102
Option 2 Cost:		$1,339,004,797	$1,547,632,101	$1,679,397,557

7.0 FUTURE WORK REQUIRED

In considering future work, consideration should be given to the fact that the majority of the cost for this project hinges on only a few decision points. Keeping these decision points in mind will help to structure the future work without unnecessarily restricting future options. Many of these decisions will impact the future operation of the plant and so therefore, may need to be more fully developed to ensure acceptable long term operation of the pump stations. The key decision points are:

- What kind of pumps (size and configuration)
- What kind of drives (electric or diesel or mix)
- What kind of emergency back-up (central plant or distributed)
- What geometry of the intake and outfall basins
- What geometry of the pump station
- Does the location require a breakwater

The acquisition strategy for this project is to utilize the Design/Build method of contracting to execute the project. In order to properly plan and execute a Design/Build contract the following studies should be performed.

1. Site Selection Study – This study would evaluate in greater detail the advantages and disadvantages of the alternative locations, refine the cost estimate, engage the stakeholders to provide adequate information for making a fully informed decision on the preferred site to construct the pump stations on each of the three canals.

2. Environmental Impact Statement – This study is required to address the NEPA related issues and to provide approval of the project within the time frame required to commence construction.

3. Hydrology and Hydraulics Study – This study would include establishment of the design and operating criteria for the new pump stations. It would determine the flow rates and the lake stage requirements for design. As a part of this study, an evaluation of the expected conditions that would require operation of the pump station would be performed. Also included in this study would be the coastal engineering study to determine design wave heights, with and without breakwaters, and sediment transport modeling and analysis.

4. HTRW Investigations – Sediment sampling and testing of the canal in the reach that might be impacted by the construction of the pump stations. This information is required to determine the handling, disposal, and cost requirements for the required excavation.

5. Geotechnical Investigations – Some general geotechnical investigations to provide the Design/Build contractor with sufficient information to prepare a design for the purpose of the Design/Build proposal.

6. Topographic, Bathymetric, and Boundary Surveys – Topographic and Bathymetric information would be used in the Site Selection Study and the H&H analyses. Once the site selection is made, the boundary surveys and real estate acquisition plan (if required) should be performed.

7. Physical Model Tests – The tests include coastal modeling to determine design wave heights with and without breakwaters (to calibrate the numerical model). Other physical model test is the pump station intake and outfall basins. Once the site selection is made, a physical model of the pump station intake and outfall would be used to optimize the basins and define the canal impacts and the excavation requirements.

8. Central Power Plant Feasibility Study – This study would determine the feasibility of providing a central power generating station instead of distributed standby power for each pump station.

9. Project Description Report – This document will be used to continue to engage the stakeholders and define the criteria that will be required for the design and construction of the pump stations. This Report should also investigate alternative approaches to implementation of the design/build approach considering the contracting methodology (one or multiple contracts, pre-purchasing long lead items, etc.) performance criteria, the fixed criteria, and bid evaluation criteria considering cost, schedule, and O&M issues. The development of this document would be used as the mechanism through which stakeholder involvement would continue and through which they could impact the requirements to be place on the design/build contractor.

10. Preparation of the Design/Build Request for Proposal.

Appendix A

HYDRAULIC

Appendix A

Hydraulic Analyses

HEC-RAS Summary Printouts

For 17th Street, Orleans Avenue, and London Avenue Canals

Including Family of Curves, Option 1 Canal Profile, and Option 2 Canal

Profile Summary Tables

17^{th} Street Canal Option 1 Family of Curves Information at DPS#6

HEC-RAS River: 17th st Canal Reach: Interim Closure

Reach	River Sta	Profile	Plan	Q Total	Min Ch El	W.S. Elev	Crit W.S.	E.G. Elev	E.G. Slope	Vel Chnl	Flow Area	Top Width	Froude # Chl
				(cfs)	(ft)	(ft)	(ft)	(ft)	(ft/ft)	(ft/s)	(sq ft)	(ft)	
Interim Closure	62	PF 1	BV3 +1.5	1148.00	-17.90	1.58		1.58	0.000005	0.56	2047.41	231.00	0.03
Interim Closure	62	PF 1	BV3 +2.5	1148.00	-17.90	2.56		2.57	0.000004	0.50	2275.67	231.00	0.03
Interim Closure	62	PF 1	BV3 +3.5	1148.00	-17.90	3.55		3.56	0.000003	0.46	2506.60	235.77	0.02
Interim Closure	62	PF 1	BV3 +4.5	1148.00	-17.90	3.55		3.56	0.000003	0.46	2506.60	235.77	0.02
Interim Closure	62	PF 2	BV3 +1.5	2296.00	-17.90	1.77		1.79	0.000019	1.10	2093.34	231.00	0.06
Interim Closure	62	PF 2	BV3 +2.5	2296.00	-17.90	2.73		2.74	0.000014	0.99	2313.09	231.63	0.06
Interim Closure	62	PF 2	BV3 +3.5	2296.00	-17.90	3.69		3.70	0.000011	0.90	2538.04	236.43	0.05
Interim Closure	62	PF 2	BV3 +4.5	2296.00	-17.90	3.69		3.70	0.000011	0.90	2538.04	236.43	0.05
Interim Closure	62	PF 3	BV3 +1.5	3444.00	-17.90	2.10		2.14	0.000039	1.59	2168.62	231.00	0.09
Interim Closure	62	PF 3	BV3 +2.5	3444.00	-17.90	2.99		3.02	0.000029	1.45	2373.60	232.93	0.08
Interim Closure	62	PF 3	BV3 +3.5	3444.00	-17.90	3.90		3.93	0.000022	1.33	2589.00	237.51	0.07
Interim Closure	62	PF 3	BV3 +4.5	3444.00	-17.90	3.90		3.93	0.000022	1.33	2589.00	237.51	0.07
Interim Closure	62	PF 4	BV3 +1.5	4592.00	-17.90	2.52		2.59	0.000060	2.03	2265.82	231.00	0.11
Interim Closure	62	PF 4	BV3 +2.5	4592.00	-17.90	3.34		3.39	0.000047	1.87	2456.25	234.70	0.10
Interim Closure	62	PF 4	BV3 +3.5	4592.00	-17.90	4.19		4.24	0.000037	1.73	2658.02	238.96	0.09
Interim Closure	62	PF 4	BV3 +4.5	4592.00	-17.90	4.19		4.24	0.000037	1.73	2658.02	238.96	0.09
Interim Closure	62	PF 5	BV3 +1.5	5740.00	-17.90	3.02		3.11	0.000080	2.41	2382.34	233.12	0.13
Interim Closure	62	PF 5	BV3 +2.5	5740.00	-17.90	3.76		3.84	0.000065	2.25	2556.09	236.82	0.12
Interim Closure	62	PF 5	BV3 +3.5	5740.00	-17.90	4.55		4.62	0.000052	2.09	2743.97	240.75	0.11
Interim Closure	62	PF 5	BV3 +4.5	5740.00	-17.90	4.55		4.62	0.000052	2.09	2743.97	240.75	0.11
Interim Closure	62	PF 6	BV3 +1.5	6888.00	-17.90	3.59		3.71	0.000098	2.74	2515.83	235.96	0.15
Interim Closure	62	PF 6	BV3 +2.5	6888.00	-17.90	4.25		4.35	0.000082	2.58	2671.92	239.25	0.14
Interim Closure	62	PF 6	BV3 +3.5	6888.00	-17.90	4.97		5.06	0.000068	2.42	2844.33	242.19	0.12
Interim Closure	62	PF 6	BV3 +4.5	6888.00	-17.90	4.97		5.06	0.000068	2.42	2844.33	242.19	0.12
Interim Closure	62	PF 7	BV3 +1.5	8036.00	-17.90	4.21		4.35	0.000113	3.02	2661.66	239.03	0.16
Interim Closure	62	PF 7	BV3 +2.5	8036.00	-17.90	4.78		4.91	0.000097	2.87	2799.92	241.59	0.15
Interim Closure	62	PF 7	BV3 +3.5	8036.00	-17.90	5.43		5.54	0.000082	2.72	2956.23	243.69	0.14
Interim Closure	62	PF 7	BV3 +4.5	8036.00	-17.90	5.43		5.54	0.000082	2.72	2956.23	243.69	0.14
Interim Closure	62	PF 8	BV3 +1.5	9184.00	-17.90	4.85		5.02	0.000124	3.26	2816.47	241.81	0.17
Interim Closure	62	PF 8	BV3 +2.5	9184.00	-17.90	5.35		5.50	0.000109	3.13	2937.25	243.43	0.16
Interim Closure	62	PF 8	BV3 +3.5	9184.00	-17.90	5.92		6.06	0.000094	2.98	3077.52	245.30	0.15
Interim Closure	62	PF 8	BV3 +4.5	9184.00	-17.90	5.92		6.06	0.000094	2.98	3077.52	245.30	0.15
Interim Closure	62	PF 9	BV3 +1.5	10332.00	-17.90	5.51		5.70	0.000132	3.47	2976.68	243.96	0.18
Interim Closure	62	PF 9	BV3 +2.5	10332.00	-17.90	5.94		6.11	0.000119	3.35	3081.60	245.35	0.17
Interim Closure	62	PF 9	BV3 +3.5	10332.00	-17.90	6.45		6.61	0.000105	3.22	3206.66	247.00	0.16
Interim Closure	62	PF 9	BV3 +4.5	10332.00	-17.90	6.45		6.61	0.000105	3.22	3206.66	247.00	0.16
Interim Closure	62	PF 10	BV3 +1.5	11480.00	-17.90	6.18		6.39	0.000138	3.65	3141.05	246.14	0.18
Interim Closure	62	PF 10	BV3 +2.5	11480.00	-17.90	6.55		6.74	0.000127	3.55	3230.96	247.32	0.17
Interim Closure	62	PF 10	BV3 +3.5	11480.00	-17.90	7.22		7.40	0.000109	3.38	3398.44	249.51	0.16
Interim Closure	62	PF 10	BV3 +4.5	11480.00	-17.90	7.22		7.40	0.000109	3.38	3398.44	249.51	0.16

17th Street Canal Option 1 Pump Station Profile Improved Section at RR Bridge

HEC-RAS Plan: Pumps 700'us River: 17th st Canal Reach: Interim Closure Profile: PF 1

Reach	River Sta	Profile	Q Total	Min Ch El	W.S. Elev	Crit W.S.	E.G. Elev	E.G. Slope	Vel Chnl	Flow Area	Top Width	Froude # Chl
			(cfs)	(ft)	(ft)	(ft)	(ft)	(ft/ft)	(ft/s)	(sq ft)	(ft)	
Interim Closure	62	PF 1	11480.00	-17.90	5.50	-3.81	5.73	0.000164	3.86	2973.10	243.91	0.19
Interim Closure	61	PF 1	11480.00	-17.90	5.49	-3.81	5.72	0.000164	3.86	2971.45	243.89	0.20
Interim Closure	60	PF 1	11480.00	-16.40	5.33	-6.09	5.70	0.000088	4.88	2350.84	187.93	0.24
Interim Closure	59.5		Bridge									
Interim Closure	59	PF 1	11480.00	-16.40	5.28	-6.09	5.65	0.000089	4.90	2340.70	187.30	0.24
Interim Closure	58	PF 1	11480.00	-16.40	5.25	-6.12	5.63	0.000211	4.96	2315.35	171.90	0.24
Interim Closure	57	PF 1	11480.00	-16.40	5.16	-6.12	5.55	0.000215	4.99	2300.27	171.37	0.24
Interim Closure	56	PF 1	11480.00	-16.40	5.00	-5.94	5.41	0.000233	5.15	2229.93	168.40	0.25
Interim Closure	55	PF 1	11640.00	-16.40	4.83	-5.84	5.27	0.000247	5.28	2206.01	167.78	0.26
Interim Closure	54	PF 1	11640.00	-16.40	4.69	-5.81	5.12	0.000251	5.26	2214.69	172.05	0.26
Interim Closure	53	PF 1	11640.00	-16.40	4.61	-6.69	4.99	0.000205	4.90	2375.29	176.08	0.24
Interim Closure	52	PF 1	11640.00	-17.40	4.64	-10.17	4.92	0.000044	4.21	2766.94	153.20	0.17
Interim Closure	51	PF 1	11640.00	-17.40	4.64	-10.18	4.92	0.000044	4.21	2766.61	153.19	0.17
Interim Closure	50	PF 1	11640.00	-17.40	4.64	-10.17	4.91	0.000044	4.21	2766.22	153.18	0.17
Interim Closure	49	PF 1	11640.00	-17.40	4.64	-10.18	4.91	0.000044	4.21	2765.98	153.17	0.17
Interim Closure	48	PF 1	11640.00	-17.40	4.63	-10.17	4.91	0.000044	4.21	2765.48	153.15	0.17
Interim Closure	47.5		Bridge									
Interim Closure	47	PF 1	11640.00	-17.40	4.61	-10.18	4.88	0.000044	4.22	2761.39	153.03	0.17
Interim Closure	46	PF 1	11640.00	-17.40	4.48	-7.22	4.85	0.000198	4.85	2399.56	175.30	0.23
Interim Closure	45	PF 1	11640.00	-17.40	4.34	-7.05	4.72	0.000211	4.96	2345.90	173.85	0.24
Interim Closure	44	PF 1	12500.00	-17.40	4.16	-6.61	4.62	0.000255	5.43	2299.94	171.36	0.26
Interim Closure	43	PF 1	12500.00	-17.40	4.16	-6.96	4.59	0.000232	5.24	2387.10	175.39	0.25
Interim Closure	42	PF 1	12500.00	-18.40	4.04	-6.51	4.56	0.000166	5.82	2148.56	202.49	0.31
Interim Closure	41	PF 1	12500.00	-18.40	4.02	-6.51	4.55	0.000146	5.86	2132.56	177.51	0.30
Interim Closure	40	PF 1	12500.00	-18.40	4.02	-6.51	4.55	0.000146	5.86	2132.17	177.49	0.30
Interim Closure	39.5		Bridge									
Interim Closure	39	PF 1	12500.00	-18.40	3.92	-6.51	4.46	0.000149	5.91	2115.18	176.87	0.30
Interim Closure	38	PF 1	12500.00	-18.40	3.97	-7.44	4.39	0.000228	5.22	2395.25	174.19	0.25
Interim Closure	37	PF 1	12500.00	-18.40	3.87	-7.56	4.29	0.000225	5.19	2407.74	174.71	0.25
Interim Closure	36	PF 1	12500.00	-18.40	3.72	-7.53	4.15	0.000233	5.26	2375.28	173.74	0.25
Interim Closure	35	PF 1	12500.00	-18.40	3.59	-7.70	4.01	0.000228	5.22	2396.38	174.94	0.25
Interim Closure	34	PF 1	12500.00	-18.40	3.49	-7.69	3.92	0.000233	5.25	2379.13	174.35	0.25
Interim Closure	33	PF 1	12500.00	-18.40	3.40	-8.23	3.79	0.000206	4.99	2503.06	180.66	0.24
Interim Closure	32	PF 1	12500.00	-18.40	3.26	-8.22	3.66	0.000214	5.08	2458.90	178.08	0.24
Interim Closure	31	PF 1	12500.00	-18.40	3.11	-8.14	3.53	0.000224	5.16	2420.82	177.07	0.25
Interim Closure	30	PF 1	12500.00	-18.40	2.92	-7.82	3.38	0.000255	5.43	2301.72	171.92	0.26
Interim Closure	29	PF 1	12500.00	-18.40	2.73	-7.61	3.22	0.000278	5.62	2226.13	168.75	0.27
Interim Closure	28	PF 1	12500.00	-18.40	2.63	-8.30	3.06	0.000234	5.24	2387.52	177.06	0.25
Interim Closure	27.8	PF 1	12500.00	-18.40	2.53	-8.30	2.96	0.000239	5.28	2369.62	176.45	0.25
Interim Closure	27.75	PF 1	12500.00	-9.00	1.80	-2.78	2.83	0.002436	8.15	1533.60	142.00	0.44

17th Street Canal Option 2 Pump Station Profile

HEC-RAS Plan: Opt 2 +12.5 River: 17th Street Cana Reach: One Profile: PF 1

Reach	River Sta	Profile	Q Total	Min Ch El	W.S. Elev	Crit W.S.	E.G. Elev	E.G. Slope	Vel Chnl	Flow Area	Top Width	Froude # Chl
			(cfs)	(ft)	(ft)	(ft)	(ft)	(ft/ft)	(ft/s)	(sq ft)	(ft)	
One	62	PF 1	11480.00	-24.85	-10.42	-19.19	-9.98	0.000103	5.30	2165.01	150.00	0.25
One	60	PF 1	11480.00	-24.85	-10.42	-19.19	-9.99	0.000103	5.31	2163.76	150.00	0.25
One	59.5		Bridge									
One	59	PF 1	11480.00	-24.85	-10.46	-19.19	-10.03	0.000104	5.32	2157.79	150.00	0.25
One	55	PF 1	11640.00	-24.85	-10.68	-19.15	-10.21	0.000112	5.48	2125.53	150.00	0.26
One	48	PF 1	11640.00	-24.85	-10.90	-19.15	-10.42	0.000118	5.56	2092.37	150.00	0.26
One	47.5		Bridge									
One	47	PF 1	11640.00	-24.85	-10.94	-19.15	-10.46	0.000119	5.58	2085.78	150.00	0.26
One	44	PF 1	12500.00	-24.85	-11.20	-18.87	-10.62	0.000145	6.10	2048.09	150.00	0.29
One	40	PF 1	12500.00	-24.85	-11.23	-18.87	-10.65	0.000147	6.12	2042.60	150.00	0.29
One	39.5		Bridge									
One	39	PF 1	12500.00	-24.85	-11.29	-18.87	-10.70	0.000149	6.14	2034.21	150.00	0.29
One	30	PF 1	12500.00	-24.85	-12.50	-18.87	-11.79	0.000199	6.75	1852.50	150.00	0.34

Orleans Avenue Canal Option 1 Family of Curves Information at DPS#7

HEC-RAS River: Orleans Ave Outf Reach: 5-Gate Structure

Reach	River Sta	Profile	Plan	Q Total	Min Ch El	W.S. Elev	Crit W.S.	E.G. Elev	E.G. Slope	Vel Chnl	Flow Area	Top Width	Froude # Chl
				(cfs)	(ft)	(ft)	(ft)	(ft)	(ft/ft)	(ft/s)	(sq ft)	(ft)	
5-Gate Structure	26	PF 1	BV3 +1.5	1590.00	-9.10	2.23	-5.30	2.27	0.000054	1.69	943.37	134.48	0.11
5-Gate Structure	26	PF 1	BV3 +2.5	1590.00	-9.10	3.01	-5.30	3.05	0.000038	1.53	1050.40	138.17	0.09
5-Gate Structure	26	PF 1	BV3 +3.5	1590.00	-9.10	3.87	-5.30	3.90	0.000027	1.38	1170.24	142.19	0.08
5-Gate Structure	26	PF 1	BV3 +4.5	1590.00	-9.10	4.79	-5.30	4.82	0.000020	1.25	1304.19	146.55	0.07
5-Gate Structure	26	PF 1	BV3 +5.5	1590.00	-9.10	5.74	-5.30	5.76	0.000014	1.13	1445.07	151.01	0.06
5-Gate Structure	26	PF 1	BV3 +6.5	1590.00	-9.10	6.70	-5.30	6.72	0.000011	1.04	1592.15	155.52	0.05
5-Gate Structure	26	PF 1	BV3 +7.5	1590.00	-9.10	7.68	-5.30	7.70	0.000008	0.95	1747.12	160.14	0.05
5-Gate Structure	26	PF 1	BV3 +8.5	1590.00	-9.10	8.67	-5.30	8.69	0.000006	0.88	1907.94	163.99	0.04
5-Gate Structure	26	PF 2	BV3 +1.5	1790.00	-9.10	2.39	-5.06	2.45	0.000064	1.86	965.58	135.25	0.12
5-Gate Structure	26	PF 2	BV3 +2.5	1790.00	-9.10	3.14	-5.06	3.18	0.000046	1.69	1067.41	138.75	0.10
5-Gate Structure	26	PF 2	BV3 +3.5	1790.00	-9.10	3.96	-5.06	4.00	0.000034	1.54	1183.28	142.62	0.09
5-Gate Structure	26	PF 2	BV3 +4.5	1790.00	-9.10	4.88	-5.06	4.91	0.000024	1.39	1317.35	146.98	0.08
5-Gate Structure	26	PF 2	BV3 +5.5	1790.00	-9.10	5.80	-5.06	5.83	0.000018	1.27	1454.58	151.30	0.07
5-Gate Structure	26	PF 2	BV3 +6.5	1790.00	-9.10	6.75	-5.06	6.77	0.000013	1.16	1600.38	155.77	0.06
5-Gate Structure	26	PF 2	BV3 +7.5	1790.00	-9.10	7.73	-5.06	7.75	0.000010	1.07	1754.89	160.37	0.05
5-Gate Structure	26	PF 2	BV3 +8.5	1790.00	-9.10	8.72	-5.06	8.74	0.000008	0.99	1915.57	164.13	0.05
5-Gate Structure	26	PF 3	BV3 +1.5	1990.00	-9.10	2.56	-4.84	2.63	0.000073	2.02	989.07	136.07	0.13
5-Gate Structure	26	PF 3	BV3 +2.5	1990.00	-9.10	3.27	-4.84	3.32	0.000054	1.85	1085.73	139.37	0.11
5-Gate Structure	26	PF 3	BV3 +3.5	1990.00	-9.10	4.06	-4.84	4.10	0.000040	1.69	1197.50	143.09	0.10
5-Gate Structure	26	PF 3	BV3 +4.5	1990.00	-9.10	4.97	-4.84	5.01	0.000029	1.53	1329.97	147.38	0.08
5-Gate Structure	26	PF 3	BV3 +5.5	1990.00	-9.10	5.87	-4.84	5.90	0.000022	1.40	1465.11	151.63	0.07
5-Gate Structure	26	PF 3	BV3 +6.5	1990.00	-9.10	6.81	-4.84	6.84	0.000016	1.29	1609.52	156.04	0.07
5-Gate Structure	26	PF 3	BV3 +7.5	1990.00	-9.10	7.79	-4.84	7.81	0.000012	1.18	1763.56	160.62	0.06
5-Gate Structure	26	PF 3	BV3 +8.5	1990.00	-9.10	8.77	-4.84	8.79	0.000010	1.09	1924.08	164.29	0.05
5-Gate Structure	26	PF 4	BV3 +1.5	2190.00	-9.10	2.74	-4.63	2.82	0.000082	2.18	1013.63	136.91	0.14
5-Gate Structure	26	PF 4	BV3 +2.5	2190.00	-9.10	3.41	-4.63	3.47	0.000062	2.00	1105.20	140.02	0.12
5-Gate Structure	26	PF 4	BV3 +3.5	2190.00	-9.10	4.16	-4.63	4.22	0.000047	1.84	1212.80	143.59	0.11
5-Gate Structure	26	PF 4	BV3 +4.5	2190.00	-9.10	5.06	-4.63	5.11	0.000034	1.67	1343.74	147.82	0.09
5-Gate Structure	26	PF 4	BV3 +5.5	2190.00	-9.10	5.95	-4.63	5.99	0.000026	1.53	1476.61	151.99	0.08
5-Gate Structure	26	PF 4	BV3 +6.5	2190.00	-9.10	6.88	-4.63	6.91	0.000019	1.41	1619.55	156.35	0.07
5-Gate Structure	26	PF 4	BV3 +7.5	2190.00	-9.10	7.85	-4.63	7.87	0.000015	1.30	1773.12	160.90	0.06
5-Gate Structure	26	PF 4	BV3 +8.5	2190.00	-9.10	8.83	-4.63	8.85	0.000012	1.20	1933.48	164.46	0.06
5-Gate Structure	26	PF 5	BV3 +1.5	2390.00	-9.10	2.93	-4.43	3.01	0.000090	2.32	1038.99	137.78	0.14
5-Gate Structure	26	PF 5	BV3 +2.5	2390.00	-9.10	3.55	-4.43	3.62	0.000070	2.15	1125.68	140.71	0.13
5-Gate Structure	26	PF 5	BV3 +3.5	2390.00	-9.10	4.28	-4.43	4.34	0.000053	1.98	1229.10	144.12	0.11
5-Gate Structure	26	PF 5	BV3 +4.5	2390.00	-9.10	5.16	-4.43	5.21	0.000039	1.80	1357.82	148.27	0.10
5-Gate Structure	26	PF 5	BV3 +5.5	2390.00	-9.10	6.03	-4.43	6.07	0.000030	1.66	1489.12	152.37	0.09
5-Gate Structure	26	PF 5	BV3 +6.5	2390.00	-9.10	6.95	-4.43	6.98	0.000023	1.53	1630.50	156.68	0.08
5-Gate Structure	26	PF 5	BV3 +7.5	2390.00	-9.10	7.91	-4.43	7.94	0.000017	1.41	1783.56	161.20	0.07
5-Gate Structure	26	PF 5	BV3 +8.5	2390.00	-9.10	8.89	-4.43	8.92	0.000013	1.30	1943.75	164.65	0.06
5-Gate Structure	26	PF 6	BV3 +1.5	2590.00	-9.10	3.12	-4.23	3.21	0.000098	2.45	1065.08	138.67	0.15
5-Gate Structure	26	PF 6	BV3 +2.5	2590.00	-9.10	3.70	-4.23	3.78	0.000077	2.29	1147.05	141.42	0.14
5-Gate Structure	26	PF 6	BV3 +3.5	2590.00	-9.10	4.40	-4.23	4.47	0.000060	2.12	1246.32	144.69	0.12
5-Gate Structure	26	PF 6	BV3 +4.5	2590.00	-9.10	5.26	-4.23	5.32	0.000044	1.93	1373.50	148.76	0.11
5-Gate Structure	26	PF 6	BV3 +5.5	2590.00	-9.10	6.12	-4.23	6.17	0.000034	1.78	1502.54	152.79	0.09
5-Gate Structure	26	PF 6	BV3 +6.5	2590.00	-9.10	7.02	-4.23	7.06	0.000026	1.64	1642.33	157.03	0.08
5-Gate Structure	26	PF 6	BV3 +7.5	2590.00	-9.10	7.98	-4.23	8.01	0.000020	1.52	1794.84	161.53	0.07
5-Gate Structure	26	PF 6	BV3 +8.5	2590.00	-9.10	8.96	-4.23	8.99	0.000016	1.40	1954.87	164.85	0.07
5-Gate Structure	26	PF 7	BV3 +1.5	2790.00	-9.10	3.31	-4.05	3.41	0.000105	2.58	1091.61	139.57	0.16
5-Gate Structure	26	PF 7	BV3 +2.5	2790.00	-9.10	3.86	-4.05	3.95	0.000085	2.42	1169.19	142.16	0.14
5-Gate Structure	26	PF 7	BV3 +3.5	2790.00	-9.10	4.52	-4.05	4.60	0.000066	2.25	1264.38	145.27	0.13
5-Gate Structure	26	PF 7	BV3 +4.5	2790.00	-9.10	5.38	-4.05	5.44	0.000050	2.06	1390.12	149.29	0.11
5-Gate Structure	26	PF 7	BV3 +5.5	2790.00	-9.10	6.21	-4.05	6.27	0.000038	1.90	1516.88	153.23	0.10
5-Gate Structure	26	PF 7	BV3 +6.5	2790.00	-9.10	7.10	-4.05	7.15	0.000030	1.76	1655.00	157.41	0.09
5-Gate Structure	26	PF 7	BV3 +7.5	2790.00	-9.10	8.06	-4.05	8.10	0.000023	1.62	1807.04	161.89	0.08
5-Gate Structure	26	PF 7	BV3 +8.5	2790.00	-9.10	9.03	-4.05	9.06	0.000018	1.50	1966.02	165.05	0.07
5-Gate Structure	26	PF 8	BV3 +1.5	2990.00	-9.10	3.50	-3.87	3.61	0.000112	2.70	1118.58	140.47	0.16
5-Gate Structure	26	PF 8	BV3 +2.5	2990.00	-9.10	4.02	-3.87	4.12	0.000091	2.55	1192.11	142.91	0.15
5-Gate Structure	26	PF 8	BV3 +3.5	2990.00	-9.10	4.65	-3.87	4.74	0.000073	2.38	1283.20	145.88	0.13
5-Gate Structure	26	PF 8	BV3 +4.5	2990.00	-9.10	5.49	-3.87	5.57	0.000055	2.18	1407.63	149.84	0.12
5-Gate Structure	26	PF 8	BV3 +5.5	2990.00	-9.10	6.31	-3.87	6.38	0.000043	2.02	1532.09	153.69	0.11
5-Gate Structure	26	PF 8	BV3 +6.5	2990.00	-9.10	7.19	-3.87	7.24	0.000033	1.87	1668.51	157.81	0.09
5-Gate Structure	26	PF 8	BV3 +7.5	2990.00	-9.10	8.14	-3.87	8.18	0.000026	1.73	1820.08	162.27	0.08
5-Gate Structure	26	PF 8	BV3 +8.5	2990.00	-9.10	9.10	-3.87	9.14	0.000020	1.60	1978.80	165.27	0.07
5-Gate Structure	26	PF 9	BV3 +1.5	3190.00	-9.10	3.70	-3.69	3.82	0.000118	2.82	1146.04	141.39	0.17
5-Gate Structure	26	PF 9	BV3 +2.5	3190.00	-9.10	4.18	-3.69	4.29	0.000098	2.67	1215.49	143.68	0.15
5-Gate Structure	26	PF 9	BV3 +3.5	3190.00	-9.10	4.79	-3.69	4.88	0.000079	2.50	1302.80	146.51	0.14
5-Gate Structure	26	PF 9	BV3 +4.5	3190.00	-9.10	5.62	-3.69	5.70	0.000060	2.30	1425.98	150.41	0.12
5-Gate Structure	26	PF 9	BV3 +5.5	3190.00	-9.10	6.42	-3.69	6.49	0.000047	2.13	1548.14	154.18	0.11
5-Gate Structure	26	PF 9	BV3 +6.5	3190.00	-9.10	7.28	-3.69	7.34	0.000037	1.98	1682.84	158.24	0.10
5-Gate Structure	26	PF 9	BV3 +7.5	3190.00	-9.10	8.22	-3.69	8.27	0.000029	1.83	1833.97	162.63	0.09
5-Gate Structure	26	PF 9	BV3 +8.5	3190.00	-9.10	9.19	-3.69	9.23	0.000022	1.70	1992.38	165.30	0.08
5-Gate Structure	26	PF 10	BV3 +1.5	3390.00	-9.10	3.89	-3.52	4.03	0.000123	2.93	1173.82	142.31	0.17
5-Gate Structure	26	PF 10	BV3 +2.5	3390.00	-9.10	4.35	-3.52	4.47	0.000104	2.79	1239.38	144.46	0.16
5-Gate Structure	26	PF 10	BV3 +3.5	3390.00	-9.10	4.92	-3.52	5.03	0.000085	2.62	1322.96	147.16	0.15
5-Gate Structure	26	PF 10	BV3 +4.5	3390.00	-9.10	5.74	-3.52	5.83	0.000065	2.42	1445.16	151.01	0.13
5-Gate Structure	26	PF 10	BV3 +5.5	3390.00	-9.10	6.53	-3.52	6.60	0.000051	2.25	1565.03	154.70	0.12
5-Gate Structure	26	PF 10	BV3 +6.5	3390.00	-9.10	7.38	-3.52	7.44	0.000040	2.09	1697.99	158.69	0.10
5-Gate Structure	26	PF 10	BV3 +7.5	3390.00	-9.10	8.31	-3.52	8.37	0.000031	1.93	1848.69	162.90	0.09
5-Gate Structure	26	PF 10	BV3 +8.5	3390.00	-9.10	9.27	-3.52	9.32	0.000025	1.79	2006.81	165.32	0.08

Orleans Avenue Canal Option 1 Pump Station Profile

HEC-RAS Plan: Opt 1 Pumps River: Orleans Ave Outf Reach: 5-Gate Structure Profile: PF 1

Reach	River Sta	Profile	Q Total	Min Ch El	W.S. Elev	Crit W.S.	E.G. Elev	E.G. Slope	Vel Chnl	Flow Area	Top Width	Froude # Chl
			(cfs)	(ft)	(ft)	(ft)	(ft)	(ft/ft)	(ft/s)	(sq ft)	(ft)	
5-Gate Structure	26	PF 1	3390.00	-9.10	9.50	-3.52	9.55	0.000023	1.76	2043.92	165.39	0.08
5-Gate Structure	25	PF 1	3390.00	-9.10	9.49	-3.52	9.54	0.000023	1.76	2042.75	165.39	0.08
5-Gate Structure	24	PF 1	3390.00	-9.10	9.44	-3.53	9.49	0.000024	1.77	2021.96	164.84	0.08
5-Gate Structure	23	PF 1	3390.00	-9.10	9.42	-3.53	9.46	0.000024	1.77	2017.89	164.83	0.08
5-Gate Structure	22	PF 1	3390.00	-9.10	9.41	-3.52	9.46	0.000024	1.77	2017.61	164.83	0.08
5-Gate Structure	21.5		Bridge									
5-Gate Structure	21	PF 1	3390.00	-9.10	9.35	-3.52	9.40	0.000024	1.78	2006.65	164.82	0.08
5-Gate Structure	20	PF 1	3390.00	-9.10	9.35	-3.53	9.40	0.000024	1.78	2006.45	164.82	0.08
5-Gate Structure	19	PF 1	3390.00	-9.10	9.32	-3.53	9.37	0.000025	1.79	2001.46	164.57	0.08
5-Gate Structure	18	PF 1	3390.00	-8.70	9.30	-4.49	9.33	0.000017	1.58	2270.59	163.84	0.07
5-Gate Structure	17	PF 1	3390.00	-8.70	9.30	-4.48	9.33	0.000017	1.58	2270.45	163.84	0.07
5-Gate Structure	16.5		Bridge									
5-Gate Structure	16	PF 1	3390.00	-8.70	9.18	-4.48	9.22	0.000018	1.60	2252.15	163.82	0.07
5-Gate Structure	15	PF 1	3390.00	-8.70	9.18	-4.49	9.22	0.000018	1.60	2252.01	163.82	0.07
5-Gate Structure	14	PF 1	3390.00	-9.50	9.15	-4.70	9.19	0.000020	1.66	2089.61	150.00	0.08
5-Gate Structure	13	PF 1	3390.00	-9.50	9.13	-4.70	9.17	0.000021	1.67	2086.37	150.00	0.08
5-Gate Structure	12	PF 1	3390.00	-9.50	9.13	-4.70	9.17	0.000021	1.67	2086.24	150.00	0.08
5-Gate Structure	11.5		Bridge									
5-Gate Structure	11	PF 1	3390.00	-9.50	8.97	-4.70	9.01	0.000021	1.69	2061.85	150.00	0.08
5-Gate Structure	10	PF 1	3390.00	-9.50	8.96	-4.70	9.01	0.000022	1.66	2061.78	150.00	0.08
5-Gate Structure	9	PF 1	3390.00	-8.50	8.95	-3.49	8.98	0.000016	1.21	2793.39	258.55	0.06
5-Gate Structure	8	PF 1	3390.00	-8.50	8.95	-3.49	8.98	0.000015	1.31	2766.04	245.87	0.06
5-Gate Structure	7	PF 1	3390.00	-8.50	8.94	-3.49	8.97	0.000016	1.23	2764.78	245.83	0.06
5-Gate Structure	6.8	PF 1	3390.00	-8.50	8.94	-3.49	8.97	0.000016	1.23	2764.29	245.82	0.06
5-Gate Structure	6.75	PF 1	3390.00	-8.00	8.77	-3.37	8.95	0.000125	3.37	1006.20	60.00	0.14

Orleans Avenue Canal Option 2 Pump Station Profile

HEC-RAS Plan: Opt 2 Pumps River: Orleans Avenue Reach: One Profile: PF 1

Reach	River Sta	Profile	Q Total	Min Ch El	W.S. Elev	Crit W.S.	E.G. Elev	E.G. Slope	Vel Chnl	Flow Area	Top Width	Froude # Chl
			(cfs)	(ft)	(ft)	(ft)	(ft)	(ft/ft)	(ft/s)	(sq ft)	(ft)	
One	26	PF 1	3390.00	-18.90	-8.96	-14.93	-8.64	0.000135	4.55	745.19	75.00	0.25
One	22	PF 1	3390.00	-18.90	-9.52	-14.93	-9.16	0.000161	4.82	703.36	75.00	0.28
One	21.5		Bridge									
One	21	PF 1	3390.00	-18.90	-9.59	-14.93	-9.22	0.000165	4.85	698.62	75.00	0.28
One	17	PF 1	3390.00	-18.90	-10.12	-14.93	-9.71	0.000197	5.15	658.39	75.00	0.31
One	16.5		Bridge									
One	16	PF 1	3390.00	-18.90	-10.27	-14.93	-9.85	0.000208	5.24	647.04	75.00	0.31
One	12	PF 1	3390.00	-18.90	-10.98	-14.93	-10.47	0.000271	5.71	594.06	75.00	0.36
One	11.5		Bridge									
One	11	PF 1	3390.00	-18.90	-11.39	-14.93	-10.83	0.000320	6.02	563.23	75.00	0.39
One	6.5	PF 1	3390.00	-18.90	-12.50	-14.93	-11.73	0.000528	7.06	480.00	75.00	0.49

London Avenue Canal Option 1 Family of Curves Information at DPS#3

HEC-RAS River: London Av Canal Reach: 11 Gates

Reach	River Sta	Profile	Plan	Q Total	Min Ch El	W.S. Elev	Crit W.S.	E.G. Elev	E.G. Slope	Vel Chnl	Flow Area	Top Width	Froude # Chl
				(cfs)	(ft)	(ft)	(ft)	(ft)	(ft/ft)	(ft/s)	(sq ft)	(ft)	
11 Gates	14768	PF 1	BV1 +4.5	426.00	-9.90	4.54		4.55	0.000000	0.24	1767.08	159.80	0.01
11 Gates	14768	PF 1	BV1 +3.5	426.00	-9.90	3.55		3.56	0.000001	0.26	1609.05	159.61	0.01
11 Gates	14768	PF 1	BV1 +2.5	426.00	-9.90	2.57		2.57	0.000001	0.29	1451.47	159.42	0.02
11 Gates	14768	PF 1	BV1 +1.5	426.00	-9.90	1.59		1.60	0.000001	0.33	1296.45	159.23	0.02
11 Gates	14768	PF 2	BV1 +4.5	852.00	-9.90	4.68		4.68	0.000001	0.48	1788.15	159.83	0.03
11 Gates	14768	PF 2	BV1 +3.5	852.00	-9.90	3.72		3.72	0.000002	0.52	1634.90	159.64	0.03
11 Gates	14768	PF 2	BV1 +2.5	852.00	-9.90	2.76		2.77	0.000003	0.57	1482.41	159.46	0.03
11 Gates	14768	PF 2	BV1 +1.5	852.00	-9.90	1.86		1.87	0.000004	0.64	1338.74	159.28	0.04
11 Gates	14768	PF 3	BV1 +4.5	1278.00	-9.90	4.89		4.90	0.000003	0.70	1822.44	159.87	0.04
11 Gates	14768	PF 3	BV1 +3.5	1278.00	-9.90	3.98		3.99	0.000004	0.76	1676.64	159.69	0.04
11 Gates	14768	PF 3	BV1 +2.5	1278.00	-9.90	3.10		3.11	0.000005	0.83	1535.88	159.52	0.05
11 Gates	14768	PF 3	BV1 +1.5	1278.00	-9.90	2.26		2.27	0.000007	0.91	1402.19	159.36	0.05
11 Gates	14768	PF 4	BV1 +4.5	1704.00	-9.90	5.18		5.19	0.000005	0.91	1868.87	159.93	0.05
11 Gates	14768	PF 4	BV1 +3.5	1704.00	-9.90	4.33		4.35	0.000007	0.98	1732.94	159.76	0.05
11 Gates	14768	PF 4	BV1 +2.5	1704.00	-9.90	3.52		3.54	0.000008	1.06	1603.53	159.60	0.06
11 Gates	14768	PF 4	BV1 +1.5	1704.00	-9.90	2.75		2.77	0.000011	1.15	1479.90	159.45	0.07
11 Gates	14768	PF 5	BV1 +4.5	2130.00	-9.90	5.60		5.62	0.000007	1.10	1935.66	160.01	0.06
11 Gates	14768	PF 5	BV1 +3.5	2130.00	-9.90	4.76		4.78	0.000009	1.18	1800.99	159.84	0.06
11 Gates	14768	PF 5	BV1 +2.5	2130.00	-9.90	4.02		4.05	0.000011	1.26	1683.92	159.70	0.07
11 Gates	14768	PF 5	BV1 +1.5	2130.00	-9.90	3.38		3.40	0.000014	1.35	1580.55	159.58	0.08
11 Gates	14768	PF 6	BV1 +4.5	2556.00	-9.90	6.01		6.04	0.000009	1.28	2001.92	160.09	0.06
11 Gates	14768	PF 6	BV1 +3.5	2556.00	-9.90	5.24		5.27	0.000012	1.36	1878.69	159.94	0.07
11 Gates	14768	PF 6	BV1 +2.5	2556.00	-9.90	4.59		4.62	0.000014	1.44	1774.63	159.81	0.08
11 Gates	14768	PF 6	BV1 +1.5	2556.00	-9.90	4.03		4.07	0.000016	1.52	1685.67	159.70	0.08
11 Gates	14768	PF 7	BV1 +4.5	2982.00	-9.90	6.50		6.53	0.000011	1.43	2080.12	160.18	0.07
11 Gates	14768	PF 7	BV1 +3.5	2982.00	-9.90	5.77		5.81	0.000014	1.52	1963.07	160.04	0.08
11 Gates	14768	PF 7	BV1 +2.5	2982.00	-9.90	5.21		5.25	0.000016	1.59	1874.11	159.93	0.08
11 Gates	14768	PF 7	BV1 +1.5	2982.00	-9.90	4.73		4.78	0.000018	1.66	1797.14	159.84	0.09
11 Gates	14768	PF 8	BV1 +4.5	3408.00	-9.90	6.98		7.02	0.000013	1.58	2157.67	160.28	0.08
11 Gates	14768	PF 8	BV1 +3.5	3408.00	-9.90	6.29		6.33	0.000016	1.67	2045.85	160.14	0.08
11 Gates	14768	PF 8	BV1 +2.5	3408.00	-9.90	5.86		5.90	0.000017	1.72	1976.79	160.06	0.09
11 Gates	14768	PF 8	BV1 +1.5	3408.00	-9.90	5.47		5.51	0.000019	1.78	1914.32	159.98	0.09
11 Gates	14768	PF 9	BV1 +4.5	3834.00	-9.90	7.48		7.53	0.000015	1.71	2237.36	160.37	0.08
11 Gates	14768	PF 9	BV1 +3.5	3834.00	-9.90	6.87		6.92	0.000017	1.79	2139.81	160.26	0.09
11 Gates	14768	PF 9	BV1 +2.5	3834.00	-9.90	6.45		6.50	0.000019	1.85	2072.01	160.17	0.09
11 Gates	14768	PF 9	BV1 +1.5	3834.00	-9.90	6.16		6.21	0.000020	1.89	2025.35	160.12	0.09
11 Gates	14768	PF 10	BV1 +4.5	4260.00	-9.90	8.07		8.13	0.000016	1.83	2332.35	160.44	0.08
11 Gates	14768	PF 10	BV1 +3.5	4260.00	-9.90	7.68		7.73	0.000018	1.88	2268.62	160.41	0.09
11 Gates	14768	PF 10	BV1 +2.5	4260.00	-9.90	7.02		7.08	0.000021	1.97	2163.05	160.28	0.09
11 Gates	14768	PF 10	BV1 +1.5	4260.00	-9.90	6.80		6.86	0.000022	2.00	2127.82	160.24	0.10

London Avenue Canal Option1 Pump Station Profile Gentilly Bridge Improved

HEC-RAS Plan: GenBrImp River: London Av Canal Reach: 11 Gates Profile: PF 3

Reach	River Sta	Profile	Q Total	Min Ch El	W.S. Elev	Crit W.S.	E.G. Elev	E.G. Slope	Vel Chnl	Flow Area	Top Width	Froude # Chl
			(cfs)	(ft)	(ft)	(ft)	(ft)	(ft/ft)	(ft/s)	(sq ft)	(ft)	
11 Gates	14768	PF 3	4260.00	-9.90	5.54	-4.17	5.61	0.000030	2.21	1925.59	160.00	0.11
11 Gates	14685	PF 3	4260.00	-6.94	5.33	-0.46	5.59	0.000165	4.08	1044.04	125.99	0.25
11 Gates	14593	PF 3	4260.00	-5.80	5.06	0.41	5.55	0.000318	5.58	762.88	96.61	0.35
11 Gates	14493	PF 3	4260.00	-5.80	4.98	0.48	5.51	0.000354	5.84	729.59	94.69	0.37
11 Gates	14431	PF 3	4260.00	-5.43	4.96	0.53	5.49	0.000339	5.82	731.50	92.23	0.36
11 Gates	14406	PF 3	4260.00	-5.63	4.98	0.13	5.46	0.000294	5.57	765.07	92.23	0.34
11 Gates	14393	PF 3	4260.00	-6.00	4.94	0.39	5.46	0.000332	5.76	739.61	94.22	0.36
11 Gates	14293	PF 3	4260.00	-6.50	4.95	-0.27	5.41	0.000286	5.43	784.45	96.51	0.34
11 Gates	14093	PF 3	4260.00	-7.00	4.92	-0.63	5.34	0.000259	5.25	811.86	97.51	0.32
11 Gates	13972	PF 3	4260.00	-6.49	4.88	-0.42	5.31	0.000252	5.31	802.34	92.82	0.32
11 Gates	13900		Bridge									
11 Gates	13712	PF 3	4260.00	-7.60	4.75	-0.91	5.18	0.000261	5.24	812.33	96.52	0.32
11 Gates	13612	PF 3	4260.00	-7.10	4.69	-0.70	5.15	0.000286	5.44	783.28	95.91	0.34
11 Gates	13393	PF 3	4260.00	-5.70	4.78	-0.51	5.04	0.000170	4.10	1039.28	135.41	0.26
11 Gates	13112	PF 3	4260.00	-5.80	4.64	-0.31	4.98	0.000228	4.68	909.37	115.63	0.29
11 Gates	12612	PF 3	4260.00	-7.00	4.64	-2.35	4.86	0.000121	3.82	1115.59	119.25	0.22
11 Gates	12112	PF 3	4260.00	-7.40	4.59	-2.90	4.80	0.000108	3.64	1170.25	125.63	0.21
11 Gates	11612	PF 3	4260.00	-7.90	4.54	-3.09	4.75	0.000107	3.64	1170.80	124.63	0.21
11 Gates	11112	PF 3	4260.00	-7.90	4.48	-3.09	4.69	0.000109	3.66	1163.73	124.63	0.21
11 Gates	10612	PF 3	4260.00	-8.40	4.43	-3.52	4.64	0.000108	3.65	1167.80	123.13	0.21
11 Gates	10526	PF 3	4260.00	-9.87	4.45	-4.61	4.62	0.000075	3.29	1296.59	122.66	0.18
11 Gates	10418	PF 3	4260.00	-9.80	4.43	-4.63	4.61	0.000086	3.41	1250.83	123.05	0.19
11 Gates	10318	PF 3	4260.00	-8.80	4.39	-3.84	4.60	0.000105	3.61	1179.14	121.69	0.20
11 Gates	9918	PF 3	4260.00	-8.90	4.36	-3.86	4.55	0.000099	3.57	1192.88	121.13	0.20
11 Gates	9418	PF 3	4260.00	-10.20	4.32	-4.64	4.50	0.000084	3.41	1248.86	120.83	0.19
11 Gates	8918	PF 3	4260.00	-9.20	4.27	-4.14	4.46	0.000091	3.50	1215.87	119.79	0.19
11 Gates	8786	PF 3	4260.00	-8.72	4.19	-2.64	4.44	0.000134	3.97	1074.36	118.90	0.23
11 Gates	8776	PF 3	4260.00	-8.16	4.20	-2.63	4.43	0.000126	3.86	1104.10	121.55	0.23
11 Gates	8740		Bridge									
11 Gates	8706	PF 3	4260.00	-10.36	4.21	-4.62	4.38	0.000080	3.35	1269.96	121.90	0.18
11 Gates	8647	PF 3	4260.00	-11.80	4.22	-5.54	4.37	0.000068	3.19	1333.71	124.14	0.17
11 Gates	8547	PF 3	4260.00	-10.90	4.20	-5.64	4.37	0.000071	3.25	1311.71	120.70	0.17
11 Gates	8047	PF 3	4260.00	-9.50	4.12	-4.38	4.32	0.000099	3.57	1193.66	123.71	0.20
11 Gates	7547	PF 3	4260.00	-9.30	4.07	-3.85	4.27	0.000111	3.61	1181.66	131.97	0.21
11 Gates	7460	PF 3	4260.00	-10.05	4.05	-4.04	4.26	0.000115	3.64	1171.08	132.54	0.22
11 Gates	7407	PF 3	4260.00	-9.85	4.00	-3.02	4.25	0.000156	3.98	1069.74	133.20	0.25
11 Gates	7369	PF 3	4260.00	-9.85	4.00	-3.02	4.24	0.000156	3.99	1068.88	133.20	0.25
11 Gates	7300		Bridge									
11 Gates	7239	PF 3	4260.00	-11.30	3.97	-4.20	4.19	0.000128	3.76	1131.56	132.76	0.23
11 Gates	6939	PF 3	4260.00	-11.20	3.95	-4.11	4.15	0.000113	3.64	1170.31	132.00	0.22
11 Gates	6539	PF 3	4260.00	-11.60	3.90	-4.36	4.11	0.000108	3.63	1174.01	127.28	0.21
11 Gates	6039	PF 3	4260.00	-10.10	3.88	-4.56	4.05	0.000089	3.31	1286.28	140.63	0.19
11 Gates	5939	PF 3	4260.00	-9.40	3.85	-3.81	4.04	0.000100	3.47	1227.48	135.52	0.20
11 Gates	5744	PF 3	8980.00	-11.20	3.34	-2.24	3.96	0.000303	6.31	1423.40	149.50	0.36
11 Gates	5658	PF 3	8980.00	-11.90	2.87	-0.89	3.88	0.000625	8.05	1114.89	138.58	0.50
11 Gates	5558	PF 3	8980.00	-12.90	2.92	-2.16	3.79	0.000432	7.47	1202.92	125.47	0.42
11 Gates	5058	PF 3	8980.00	-12.00	2.78	-3.25	3.56	0.000368	7.11	1263.32	124.02	0.39
11 Gates	4558	PF 3	8980.00	-12.50	2.51	-3.25	3.36	0.000407	7.40	1212.82	121.46	0.41
11 Gates	4058	PF 3	8980.00	-12.60	2.07	-2.31	3.11	0.000516	8.20	1095.03	113.34	0.46
11 Gates	3973	PF 3	8980.00	-11.15	2.05	-2.48	3.06	0.000543	8.03	1118.39	125.90	0.47
11 Gates	3957	PF 3	8980.00	-11.05	2.27	-2.95	2.95	0.000380	6.65	1350.91	156.64	0.40
11 Gates	3920		Bridge									
11 Gates	3900	PF 3	8980.00	-13.70	2.24	-4.23	2.88	0.000301	6.44	1394.88	139.44	0.36
11 Gates	3400	PF 3	8980.00	-11.40	1.96	-3.43	2.70	0.000386	6.91	1300.41	142.14	0.40
11 Gates	3300	PF 3	8980.00	-10.75	1.90	-2.77	2.66	0.000487	6.98	1286.92	166.21	0.44
11 Gates	3250		Bridge									
11 Gates	3228	PF 3	8980.00	-10.45	1.85	-2.96	2.53	0.000418	6.65	1350.66	167.93	0.41
11 Gates	3100	PF 3	8980.00	-11.30	1.98	-4.39	2.41	0.000272	5.28	1701.49	218.86	0.33
11 Gates	2800	PF 3	8980.00	-10.20	1.93	-2.42	2.32	0.000268	4.95	1812.42	256.14	0.33
11 Gates	2500	PF 3	8980.00	-8.60	1.25	-0.68	2.13	0.000970	7.50	1196.96	219.73	0.57
11 Gates	2000	PF 3	8980.00	-8.70	0.95	-1.90	1.68	0.000694	6.85	1310.52	233.08	0.51
11 Gates	1710	PF 3	8980.00	-10.20	0.61	-1.12	1.41	0.001221	7.17	1252.17	270.83	0.59
11 Gates	1428	PF 3	8980.00	-7.00	1.00	-4.37	1.14	0.000100	3.03	2960.00	370.00	0.19

London Avenue Canal Option 2 Pump Station Profile

HEC-RAS Plan: Opt 2 Pumps River: London Av Canal Reach: 11 Gates Profile: PF 1

Reach	River Sta	Profile	Q Total	Min Ch El	W.S. Elev	Crit W.S.	E.G. Elev	E.G. Slope	Vel Chnl	Flow Area	Top Width	Froude # Chl
			(cfs)	(ft)	(ft)	(ft)	(ft)	(ft/ft)	(ft/s)	(sq ft)	(ft)	
11 Gates	14768	PF 1	4260.00	-19.10	-9.40	-15.28	-9.10	0.000120	4.39	969.95	100.00	0.25
11 Gates	13972	PF 1	4260.00	-19.46	-9.48	-15.64	-9.20	0.000110	4.27	997.95	100.00	0.24
11 Gates	13900		Bridge									
11 Gates	13712	PF 1	4260.00	-19.57	-9.65	-15.75	-9.36	0.000112	4.29	992.13	100.00	0.24
11 Gates	8776	PF 1	4260.00	-21.80	-10.01	-17.96	-9.80	0.000066	3.61	1179.36	100.00	0.19
11 Gates	8740		Bridge									
11 Gates	8706	PF 1	4260.00	-21.83	-10.07	-18.00	-9.86	0.000066	3.62	1176.19	100.00	0.19
11 Gates	7369	PF 1	4260.00	-22.43	-10.14	-18.59	-9.95	0.000058	3.47	1229.12	100.00	0.17
11 Gates	7300		Bridge									
11 Gates	7239	PF 1	4260.00	-22.49	-10.23	-18.67	-10.04	0.000058	3.47	1226.29	100.00	0.17
11 Gates	5744	PF 1	8980.00	-23.16	-11.20	-16.86	-10.33	0.000280	7.51	1195.71	100.00	0.38
11 Gates	3957	PF 1	8980.00	-23.96	-11.65	-17.66	-10.82	0.000256	7.29	1231.45	100.00	0.37
11 Gates	3920		Bridge									
11 Gates	3900	PF 1	8980.00	-23.99	-11.78	-17.69	-10.94	0.000262	7.35	1221.43	100.00	0.37
11 Gates	3300	PF 1	8980.00	-24.26	-11.92	-17.96	-11.10	0.000254	7.28	1234.17	100.00	0.36
11 Gates	3250		Bridge									
11 Gates	3228	PF 1	8980.00	-24.29	-12.09	-17.99	-11.25	0.000263	7.36	1220.37	100.00	0.37
11 Gates	1428	PF 1	8980.00	-25.10	-12.50	-18.80	-11.71	0.000238	7.13	1260.00	100.00	0.35

Appendix B

GEOTECHNICAL

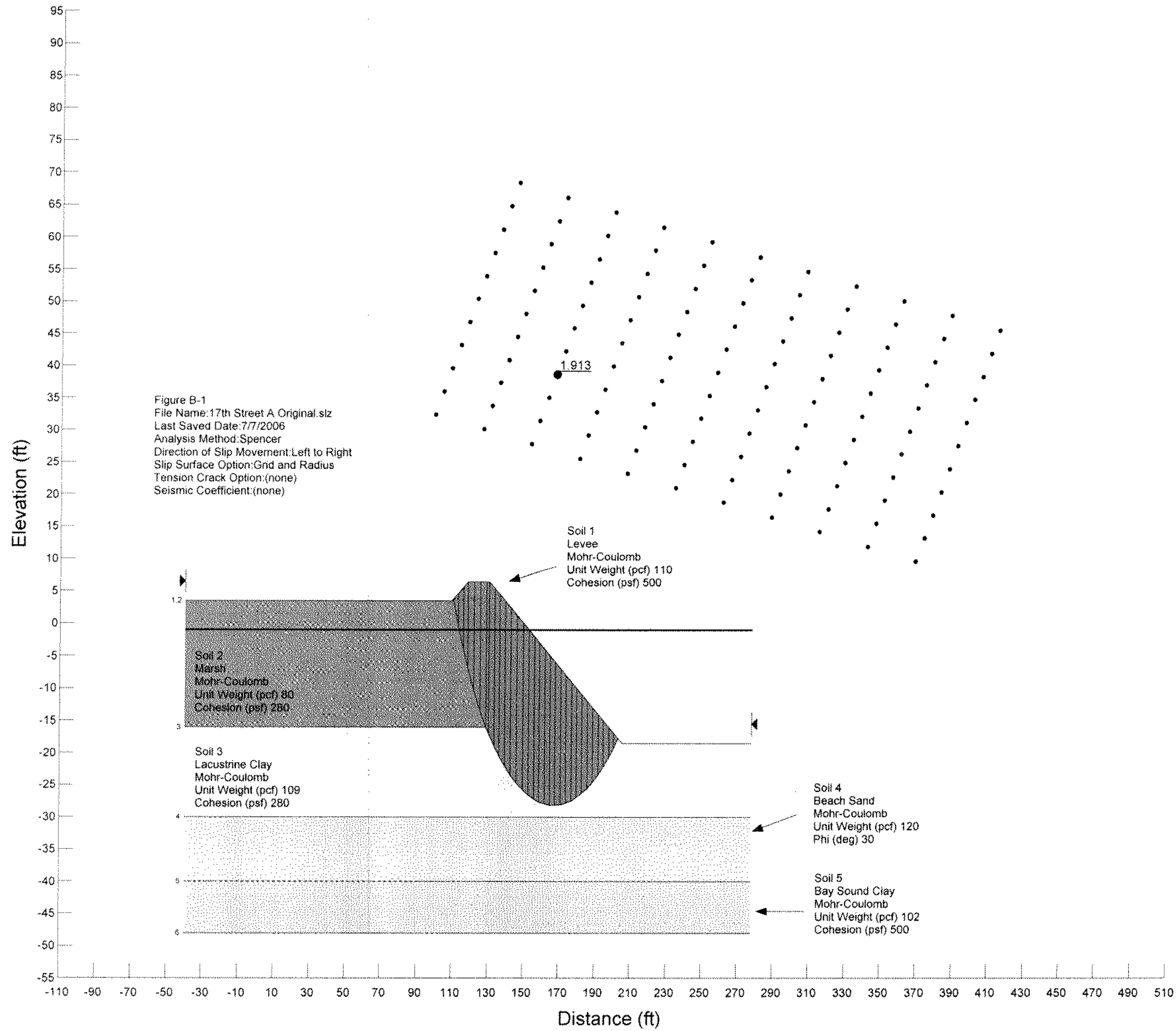


Figure B-1
File Name:17th Street A Original.slz
Last Saved Date:7/7/2006
Analysis Method:Spencer
Direction of Slip Movement:Left to Right
Slip Surface Option:Grid and Radius
Tension Crack Option:(none)
Seismic Coefficient:(none)
1.913
Soil 1
Levee
Mohr-Coulomb
Unit Weight (pcf) 110
Cohesion (psf) 500
Soil 2
Marsh
Mohr-Coulomb
Unit Weight (pcf) 80
Cohesion (psf) 280
Soil 3
Lacustrine Clay
Mohr-Coulomb
Unit Weight (pcf) 109
Cohesion (psf) 280
Soil 4
Beach Sand
Mohr-Coulomb
Unit Weight (pcf) 120
Phi (deg) 30
Soil 5
Bay Sound Clay
Mohr-Coulomb
Unit Weight (pcf) 102
Cohesion (psf) 500
Elevation (ft)
Distance (ft)

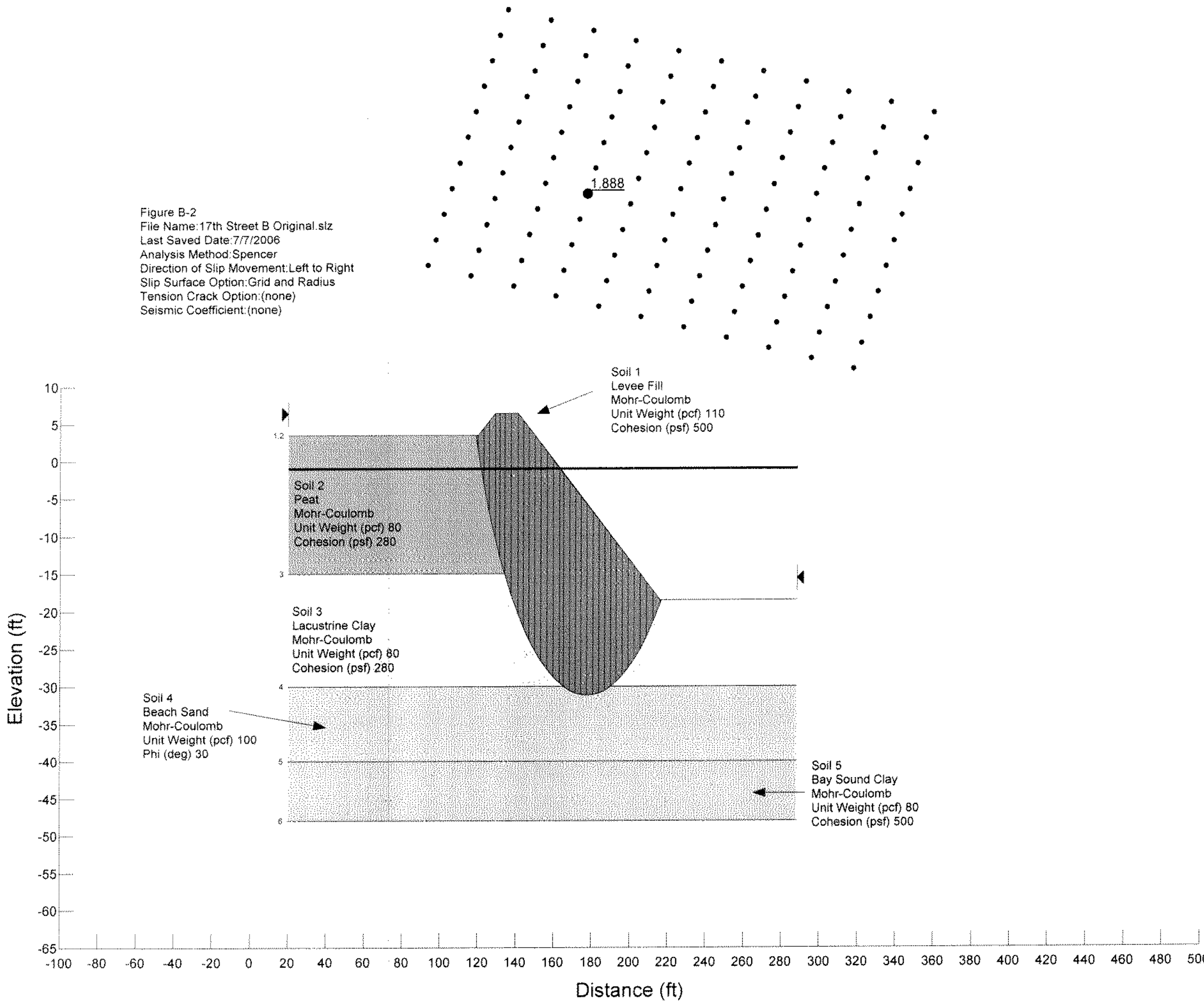


Figure B-2
File Name:17th Street B Original.slz
Last Saved Date:7/7/2006
Analysis Method:Spencer
Direction of Slip Movement:Left to Right
Slip Surface Option:Grid and Radius
Tension Crack Option:(none)
Seismic Coefficient:(none)
1.888
Soil 1
Levee Fill
Mohr-Coulomb
Unit Weight (pcf) 110
Cohesion (psf) 500
Soil 2
Peat
Mohr-Coulomb
Unit Weight (pcf) 80
Cohesion (psf) 280
Soil 3
Lacustrine Clay
Mohr-Coulomb
Unit Weight (pcf) 80
Cohesion (psf) 280
Soil 4
Beach Sand
Mohr-Coulomb
Unit Weight (pcf) 100
Phi (deg) 30
Soil 5
Bay Sound Clay
Mohr-Coulomb
Unit Weight (pcf) 80
Cohesion (psf) 500
Elevation (ft)
Distance (ft)

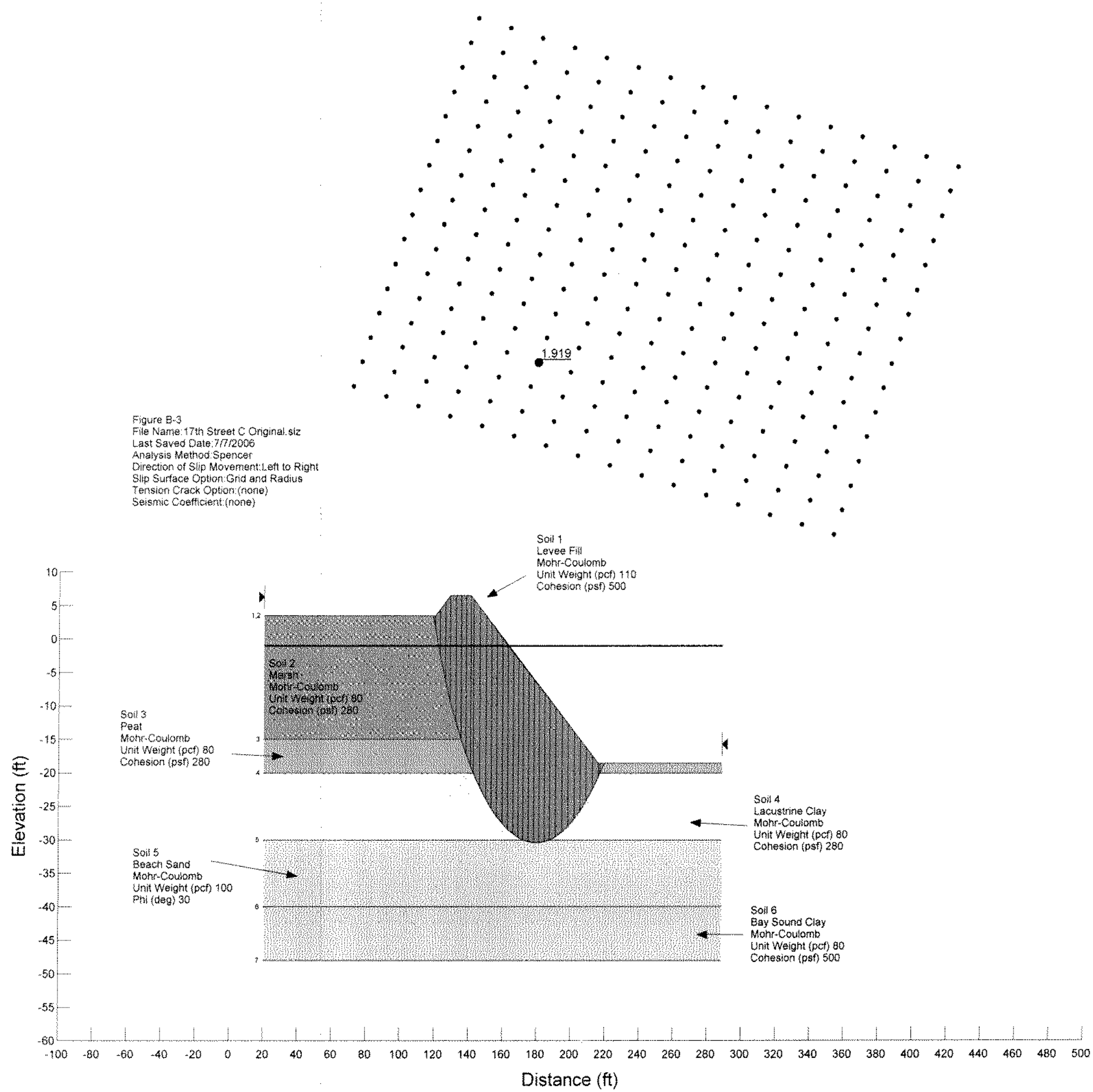


Figure B-3
File Name:17th Street C Original.slz
Last Saved Date:7/7/2006
Analysis Method:Spencer
Direction of Slip Movement:Left to Right
Slip Surface Option:Grid and Radius
Tension Crack Option:(none)
Seismic Coefficient:(none)
1.919
Soil 1
Levee Fill
Mohr-Coulomb
Unit Weight (pcf) 110
Cohesion (psf) 500
Soil 2
Marsh
Mohr-Coulomb
Unit Weight (pcf) 80
Cohesion (psf) 280
Soil 3
Peat
Mohr-Coulomb
Unit Weight (pcf) 80
Cohesion (psf) 280
Soil 4
Lacustrine Clay
Mohr-Coulomb
Unit Weight (pcf) 80
Cohesion (psf) 280
Soil 5
Beach Sand
Mohr-Coulomb
Unit Weight (pcf) 100
Phi (deg) 30
Soil 6
Bay Sound Clay
Mohr-Coulomb
Unit Weight (pcf) 80
Cohesion (psf) 500
Elevation (ft)
10 5 0 -5 -10 -15 -20 -25 -30 -35 -40 -45 -50 -55 -60
Distance (ft)
-100 -80 -60 -40 -20 0 20 40 60 80 100 120 140 160 180 200 220 240 260 280 300 320 340 360 380 400 420 440 460 480 500

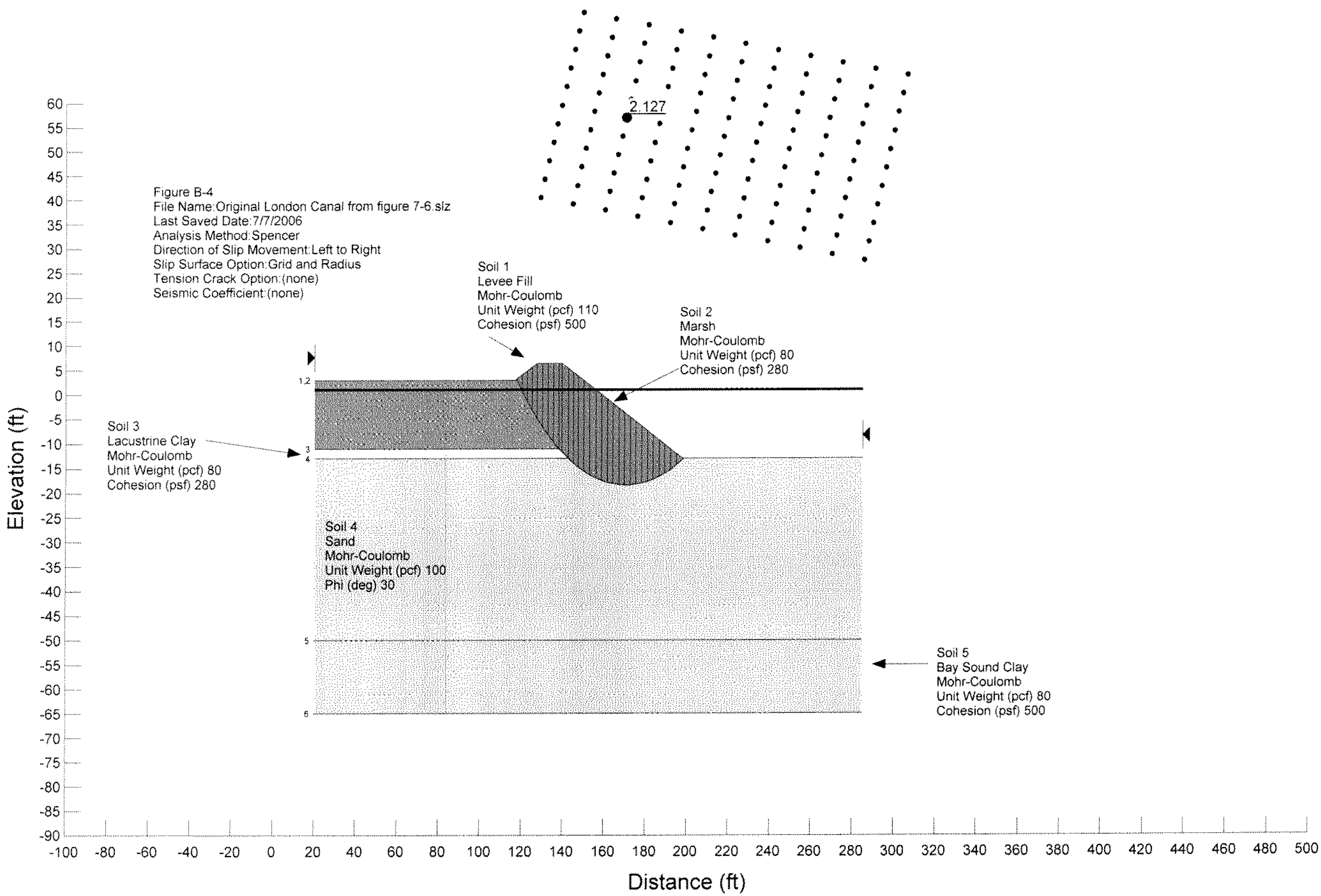


Figure B-4
File Name:Original London Canal from figure 7-6.slz
Last Saved Date:7/7/2006
Analysis Method:Spencer
Direction of Slip Movement:Left to Right
Slip Surface Option:Grid and Radius
Tension Crack Option:(none)
Seismic Coefficient:(none)
2.127
Soil 1
Levee Fill
Mohr-Coulomb
Unit Weight (pcf) 110
Cohesion (psf) 500
Soil 2
Marsh
Mohr-Coulomb
Unit Weight (pcf) 80
Cohesion (psf) 280
Soil 3
Lacustrine Clay
Mohr-Coulomb
Unit Weight (pcf) 80
Cohesion (psf) 280
Soil 4
Sand
Mohr-Coulomb
Unit Weight (pcf) 100
Phi (deg) 30
Soil 5
Bay Sound Clay
Mohr-Coulomb
Unit Weight (pcf) 80
Cohesion (psf) 500
Elevation (ft)
Distance (ft)

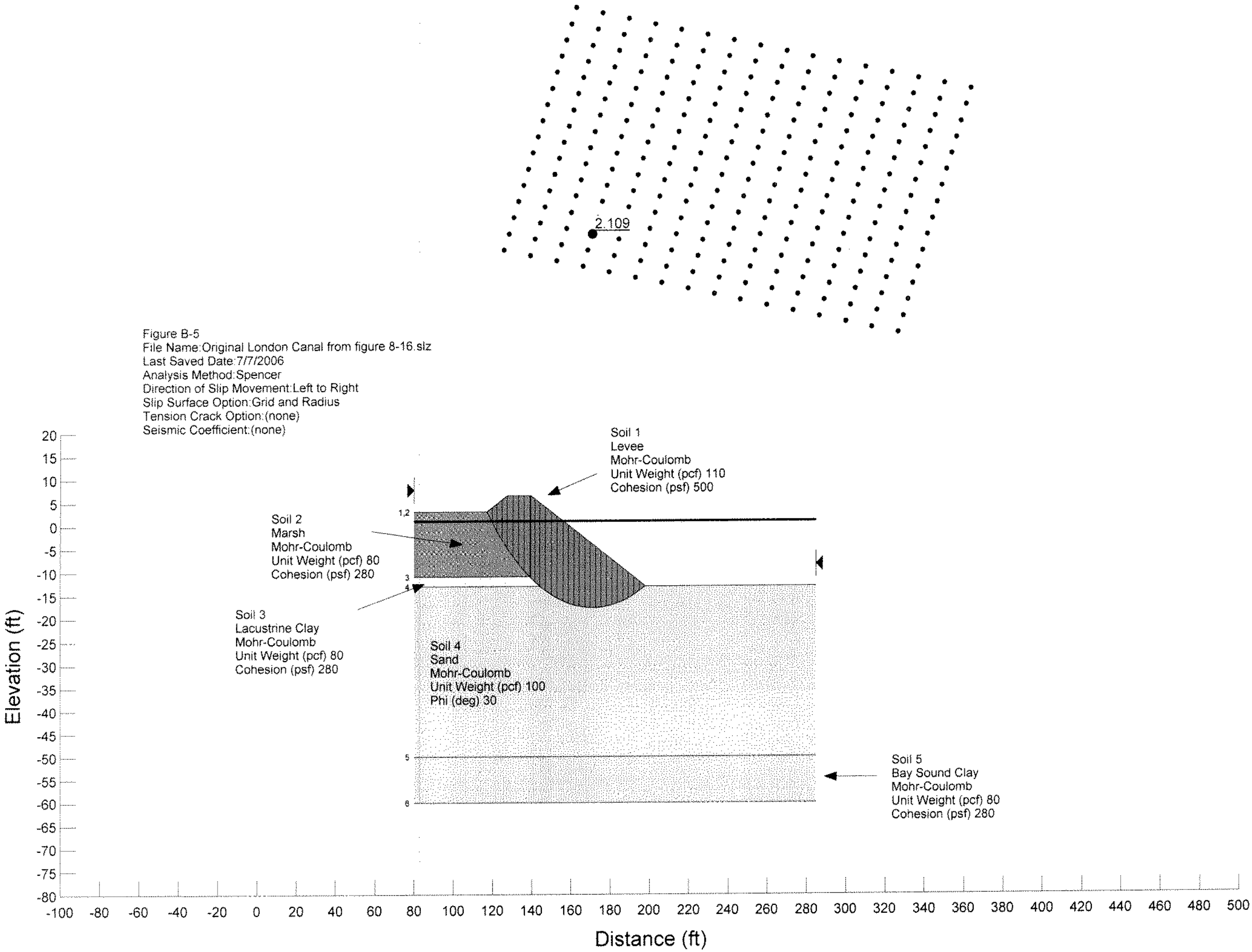


Figure B-5
File Name:Original London Canal from figure 8-16.slz
Last Saved Date:7/7/2006
Analysis Method:Spencer
Direction of Slip Movement:Left to Right
Slip Surface Option:Grid and Radius
Tension Crack Option:(none)
Seismic Coefficient:(none)
2.109
Soil 1
Levee
Mohr-Coulomb
Unit Weight (pcf) 110
Cohesion (psf) 500
Soil 2
Marsh
Mohr-Coulomb
Unit Weight (pcf) 80
Cohesion (psf) 280
Soil 3
Lacustrine Clay
Mohr-Coulomb
Unit Weight (pcf) 80
Cohesion (psf) 280
Soil 4
Sand
Mohr-Coulomb
Unit Weight (pcf) 100
Phi (deg) 30
Soil 5
Bay Sound Clay
Mohr-Coulomb
Unit Weight (pcf) 80
Cohesion (psf) 280
Elevation (ft)
Distance (ft)

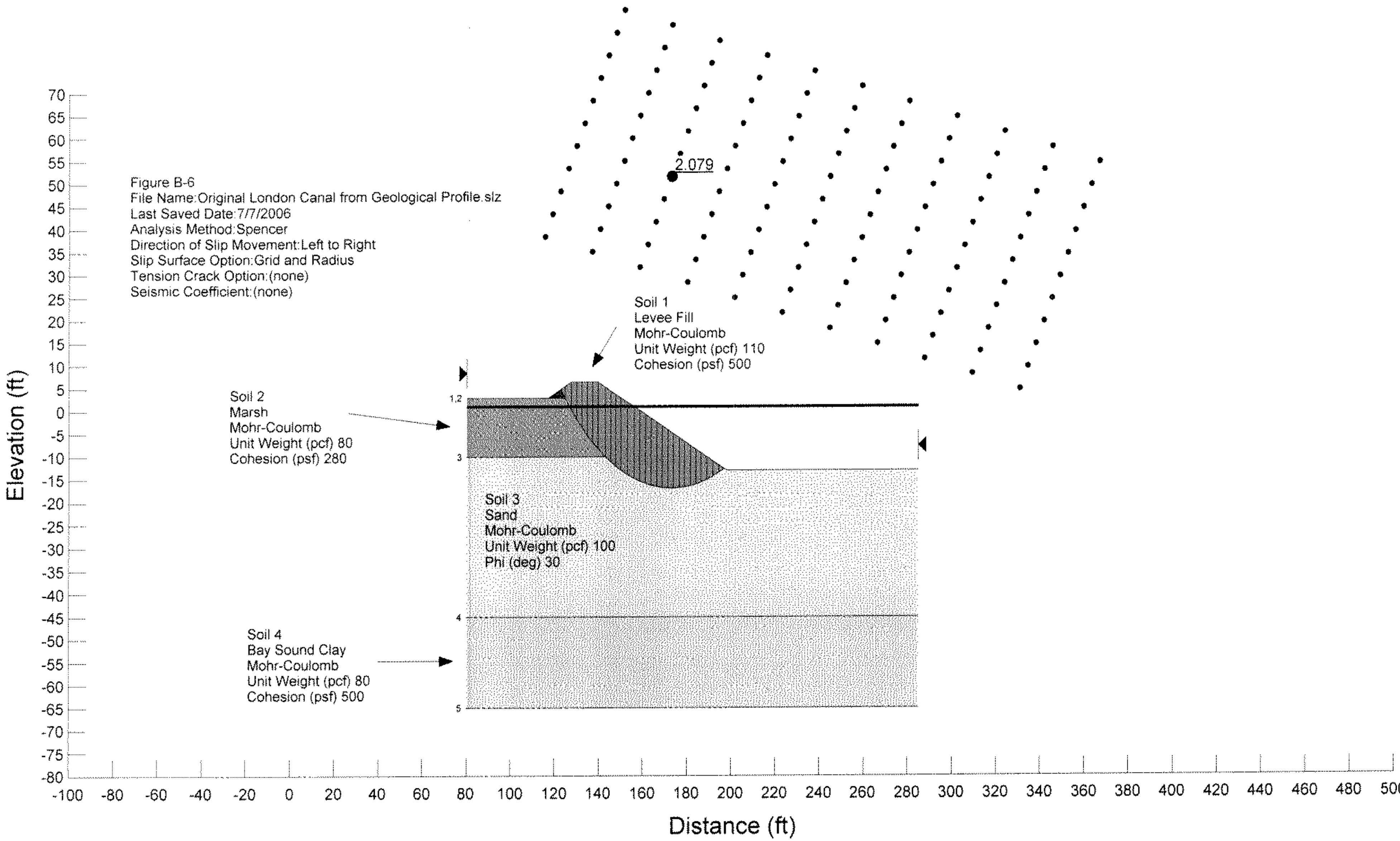


Figure B-6
File Name:Original London Canal from Geological Profile.slz
Last Saved Date:7/7/2006
Analysis Method:Spencer
Direction of Slip Movement:Left to Right
Slip Surface Option:Grid and Radius
Tension Crack Option:(none)
Seismic Coefficient:(none)
2.079
Soil 1
Levee Fill
Mohr-Coulomb
Unit Weight (pcf) 110
Cohesion (psf) 500
Soil 2
Marsh
Mohr-Coulomb
Unit Weight (pcf) 80
Cohesion (psf) 280
Soil 3
Sand
Mohr-Coulomb
Unit Weight (pcf) 100
Phi (deg) 30
Soil 4
Bay Sound Clay
Mohr-Coulomb
Unit Weight (pcf) 80
Cohesion (psf) 500
Elevation (ft)
Distance (ft)

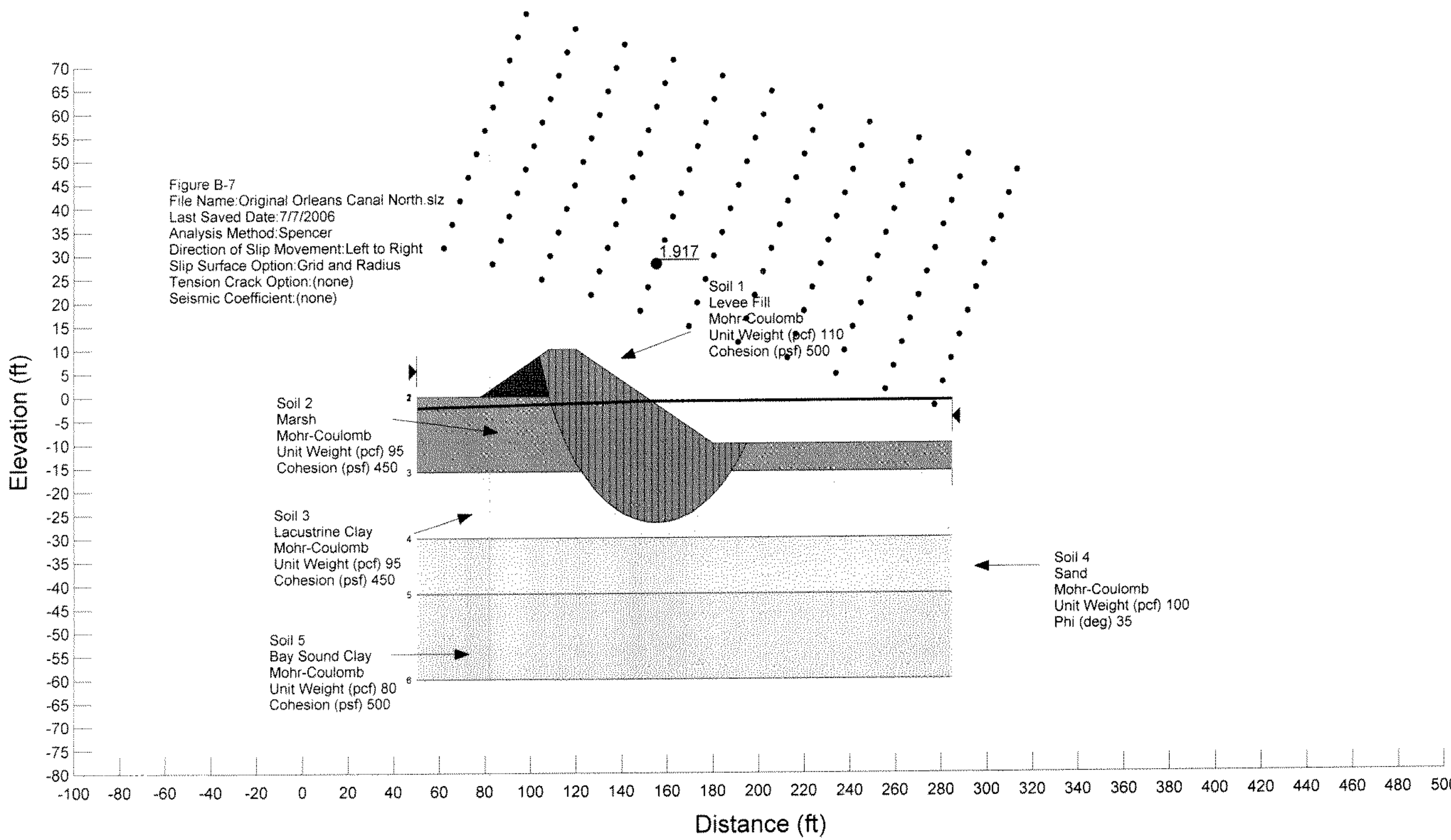


Figure B-7
File Name:Original Orleans Canal North.slz
Last Saved Date:7/7/2006
Analysis Method:Spencer
Direction of Slip Movement:Left to Right
Slip Surface Option:Grid and Radius
Tension Crack Option:(none)
Seismic Coefficient:(none)
1.917
Soil 1
Levee Fill
Mohr-Coulomb
Unit Weight (pcf) 110
Cohesion (psf) 500
Soil 2
Marsh
Mohr-Coulomb
Unit Weight (pcf) 95
Cohesion (psf) 450
Soil 3
Lacustrine Clay
Mohr-Coulomb
Unit Weight (pcf) 95
Cohesion (psf) 450
Soil 4
Sand
Mohr-Coulomb
Unit Weight (pcf) 100
Phi (deg) 35
Soil 5
Bay Sound Clay
Mohr-Coulomb
Unit Weight (pcf) 80
Cohesion (psf) 500
Elevation (ft)
Distance (ft)

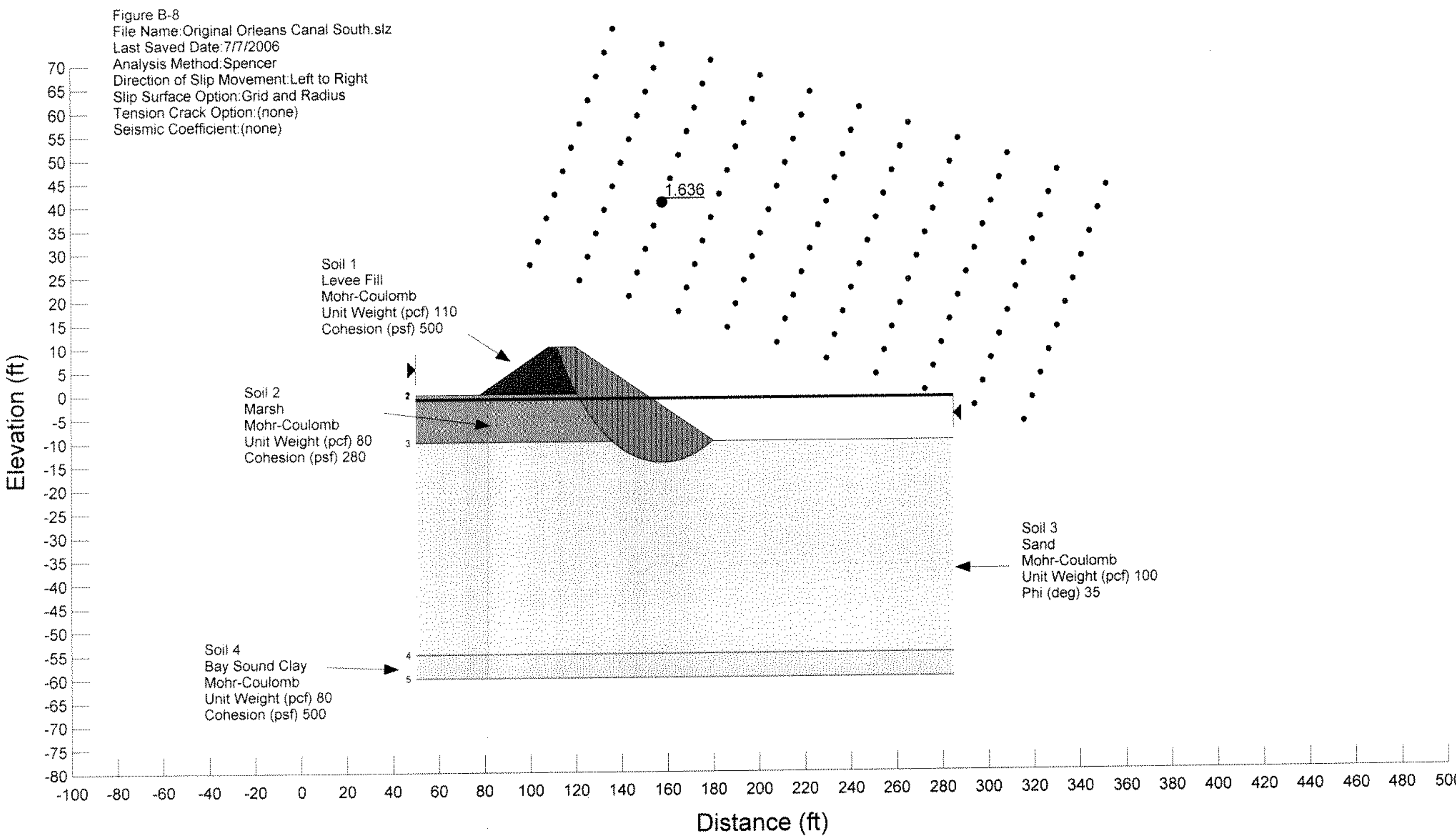


Figure B-8
File Name:Original Orleans Canal South.slz
Last Saved Date:7/7/2006
Analysis Method:Spencer
Direction of Slip Movement:Left to Right
Slip Surface Option:Grid and Radius
Tension Crack Option:(none)
Seismic Coefficient:(none)
1.636
Soil 1
Levee Fill
Mohr-Coulomb
Unit Weight (pcf) 110
Cohesion (psf) 500
Soil 2
Marsh
Mohr-Coulomb
Unit Weight (pcf) 80
Cohesion (psf) 280
Soil 3
Sand
Mohr-Coulomb
Unit Weight (pcf) 100
Phi (deg) 35
Soil 4
Bay Sound Clay
Mohr-Coulomb
Unit Weight (pcf) 80
Cohesion (psf) 500
Elevation (ft)
Distance (ft)

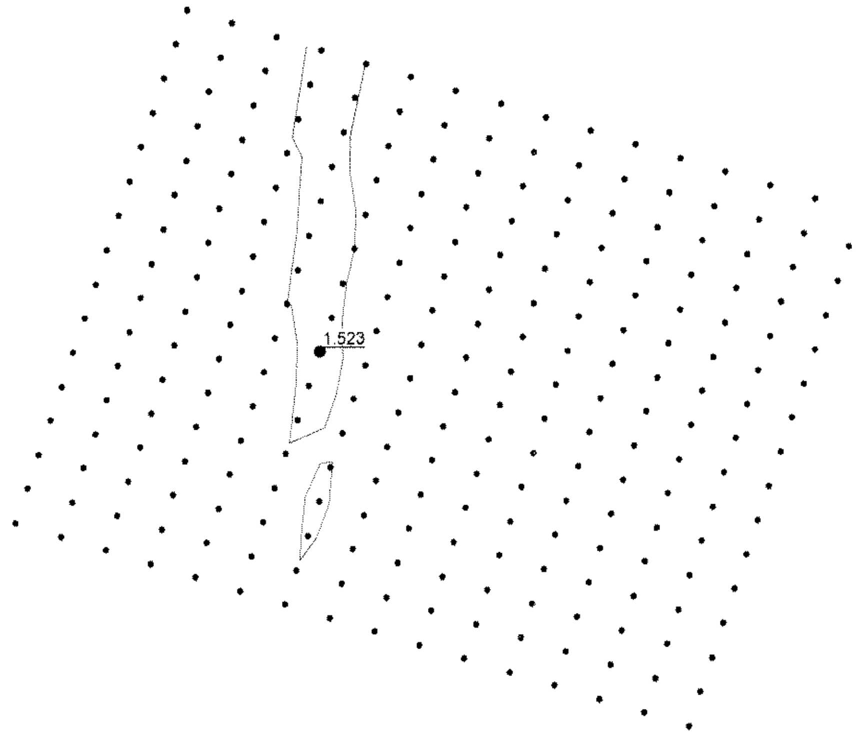
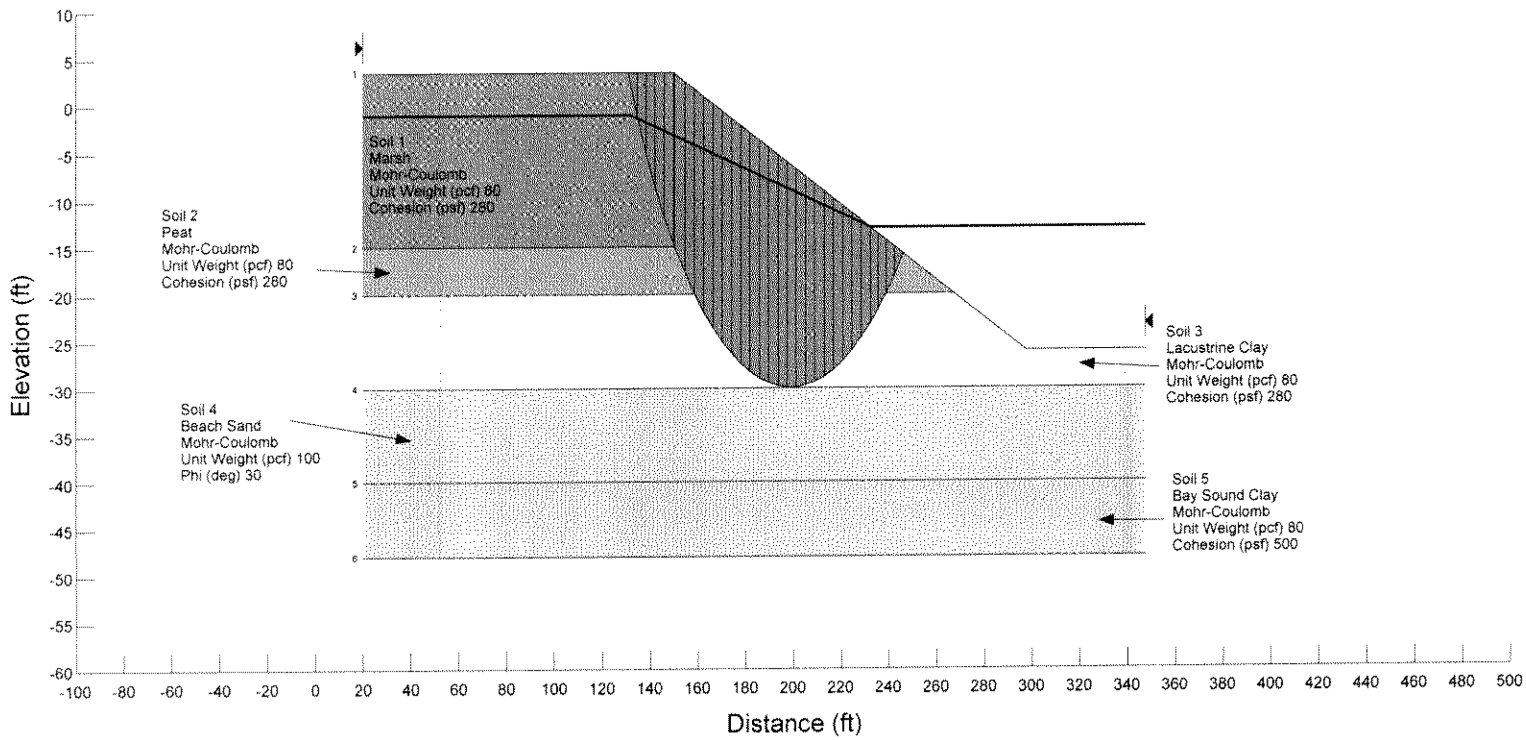


Figure B-9
File Name:17th Street C 5 -1.slz
Last Saved Date:7/7/2006
Analysis Method:Spencer
Direction of Slip Movement:Left to Right
Slip Surface Option:Grid and Radius
Tension Crack Option:(none)
Seismic Coefficient:(none)



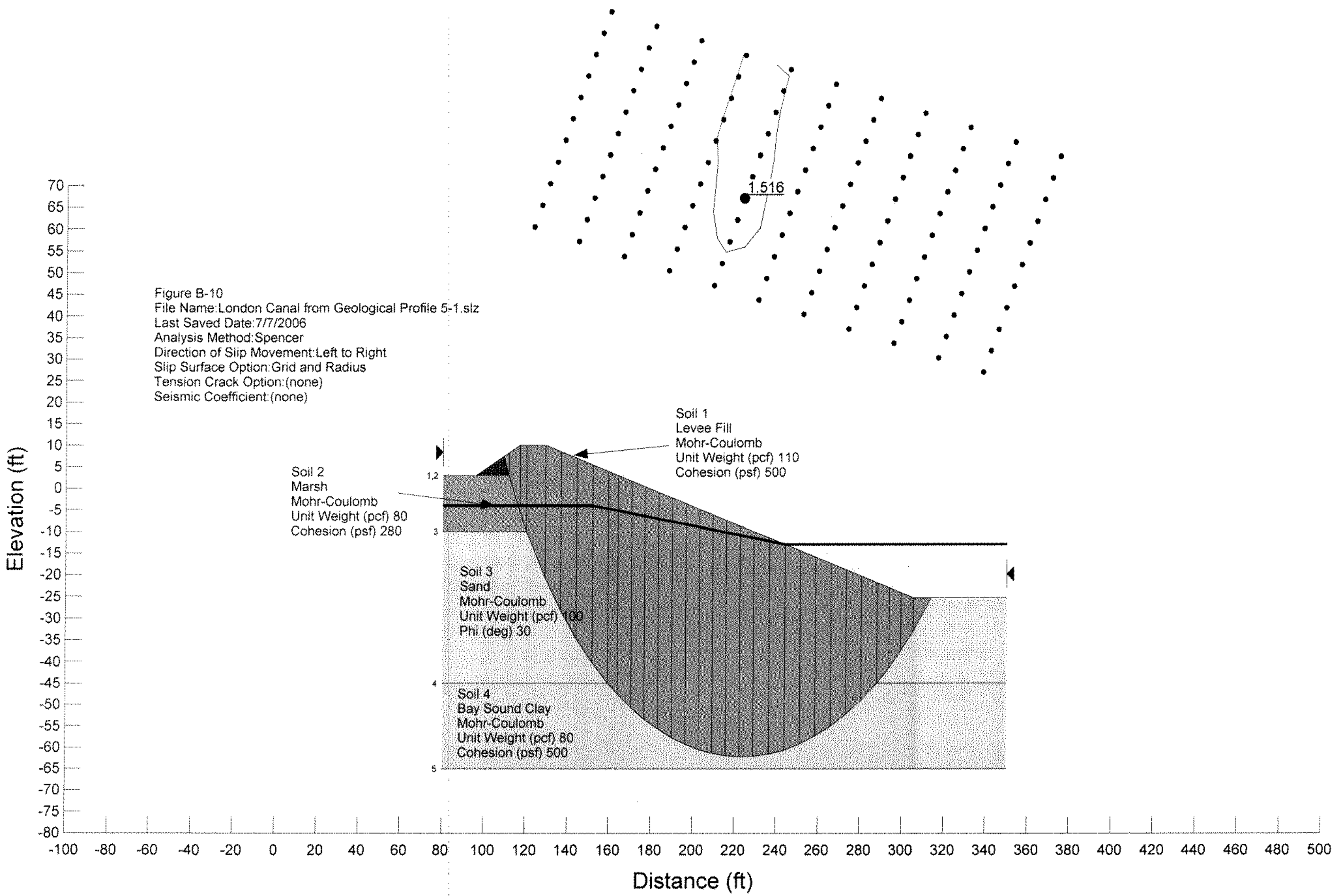
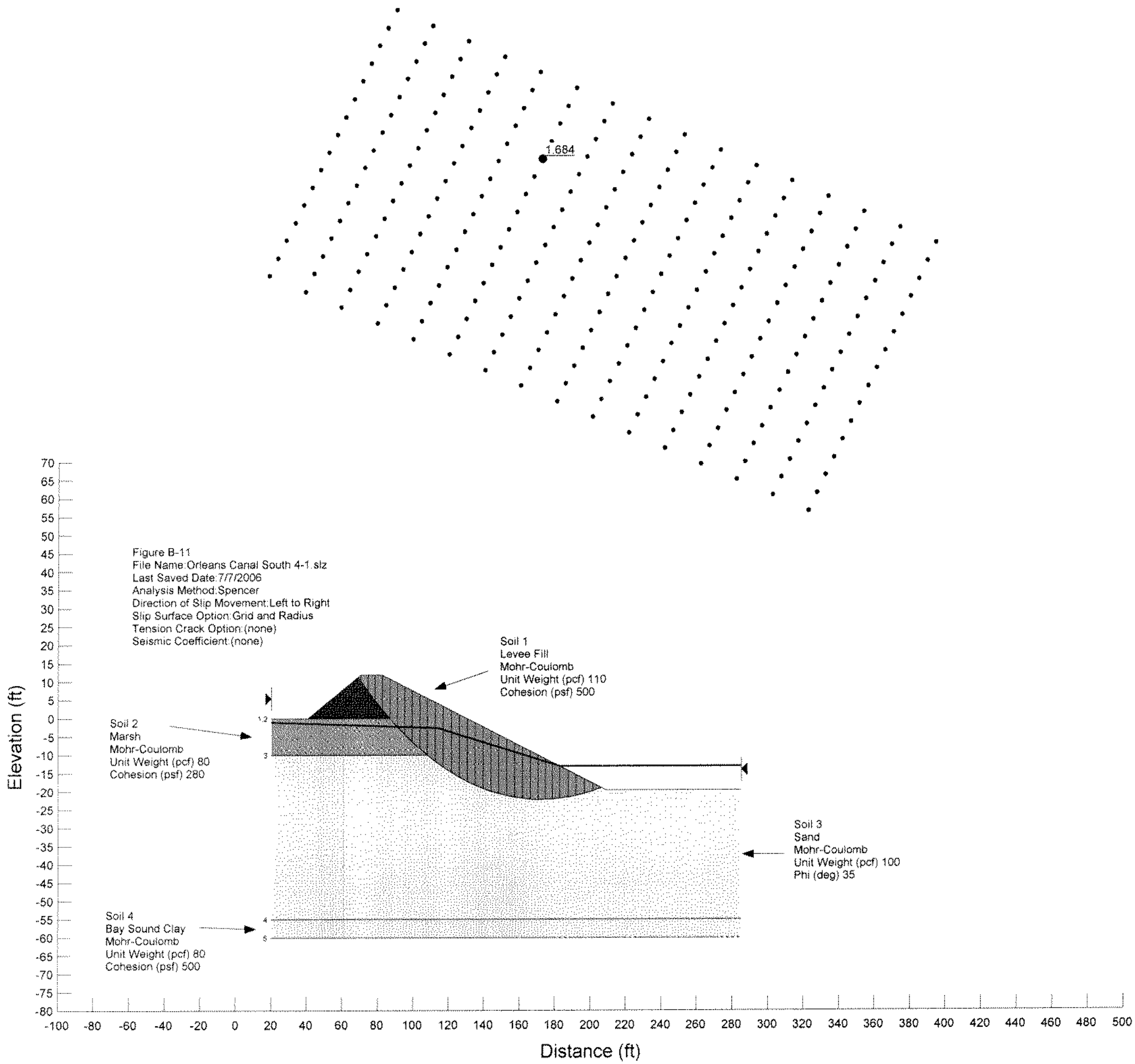


Figure B-10
File Name:London Canal from Geological Profile 5-1.slz
Last Saved Date:7/7/2006
Analysis Method:Spencer
Direction of Slip Movement:Left to Right
Slip Surface Option:Grid and Radius
Tension Crack Option:(none)
Seismic Coefficient:(none)
1.516
Soil 1
Levee Fill
Mohr-Coulomb
Unit Weight (pcf) 110
Cohesion (psf) 500
Soil 2
Marsh
Mohr-Coulomb
Unit Weight (pcf) 80
Cohesion (psf) 280
Soil 3
Sand
Mohr-Coulomb
Unit Weight (pcf) 100
Phi (deg) 30
Soil 4
Bay Sound Clay
Mohr-Coulomb
Unit Weight (pcf) 80
Cohesion (psf) 500
Elevation (ft)
Distance (ft)


1.684
Figure B-11
File Name:Orleans Canal South 4-1.slz
Last Saved Date:7/7/2006
Analysis Method:Spencer
Direction of Slip Movement:Left to Right
Slip Surface Option:Grid and Radius
Tension Crack Option:(none)
Seismic Coefficient:(none)
Soil 1
Levee Fill
Mohr-Coulomb
Unit Weight (pcf) 110
Cohesion (psf) 500
Soil 2
Marsh
Mohr-Coulomb
Unit Weight (pcf) 80
Cohesion (psf) 280
Soil 3
Sand
Mohr-Coulomb
Unit Weight (pcf) 100
Phi (deg) 35
Soil 4
Bay Sound Clay
Mohr-Coulomb
Unit Weight (pcf) 80
Cohesion (psf) 500
Elevation (ft)
Distance (ft)

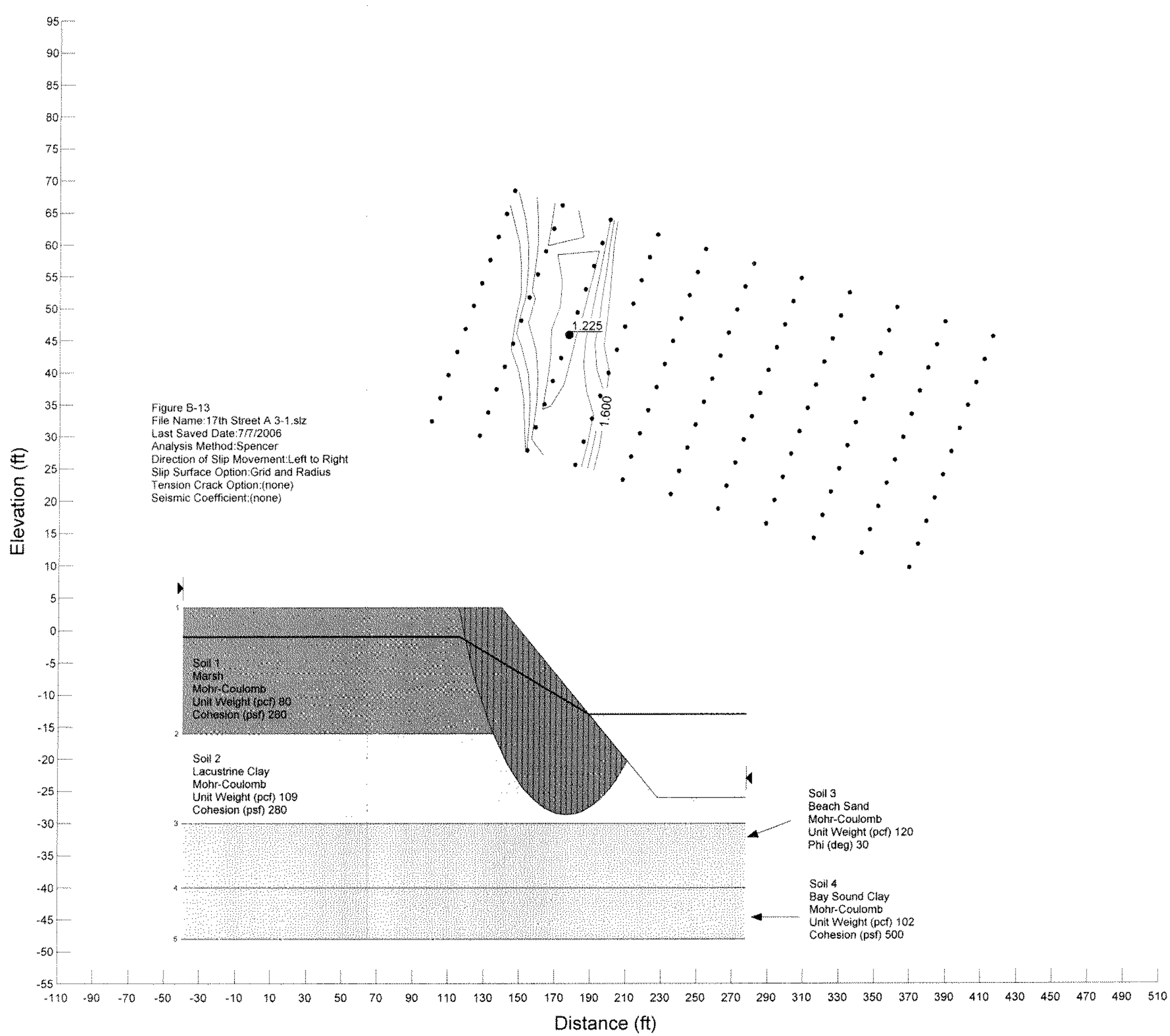


Figure B-13
File Name:17th Street A 3-1.slz
Last Saved Date:7/7/2006
Analysis Method:Spencer
Direction of Slip Movement:Left to Right
Slip Surface Option:Grid and Radius
Tension Crack Option:(none)
Seismic Coefficient:(none)
1.225
1.600
Soil 1
Marsh
Mohr-Coulomb
Unit Weight (pcf) 80
Cohesion (psf) 280
Soil 2
Lacustrine Clay
Mohr-Coulomb
Unit Weight (pcf) 109
Cohesion (psf) 280
Soil 3
Beach Sand
Mohr-Coulomb
Unit Weight (pcf) 120
Phi (deg) 30
Soil 4
Bay Sound Clay
Mohr-Coulomb
Unit Weight (pcf) 102
Cohesion (psf) 500
Elevation (ft)
Distance (ft)

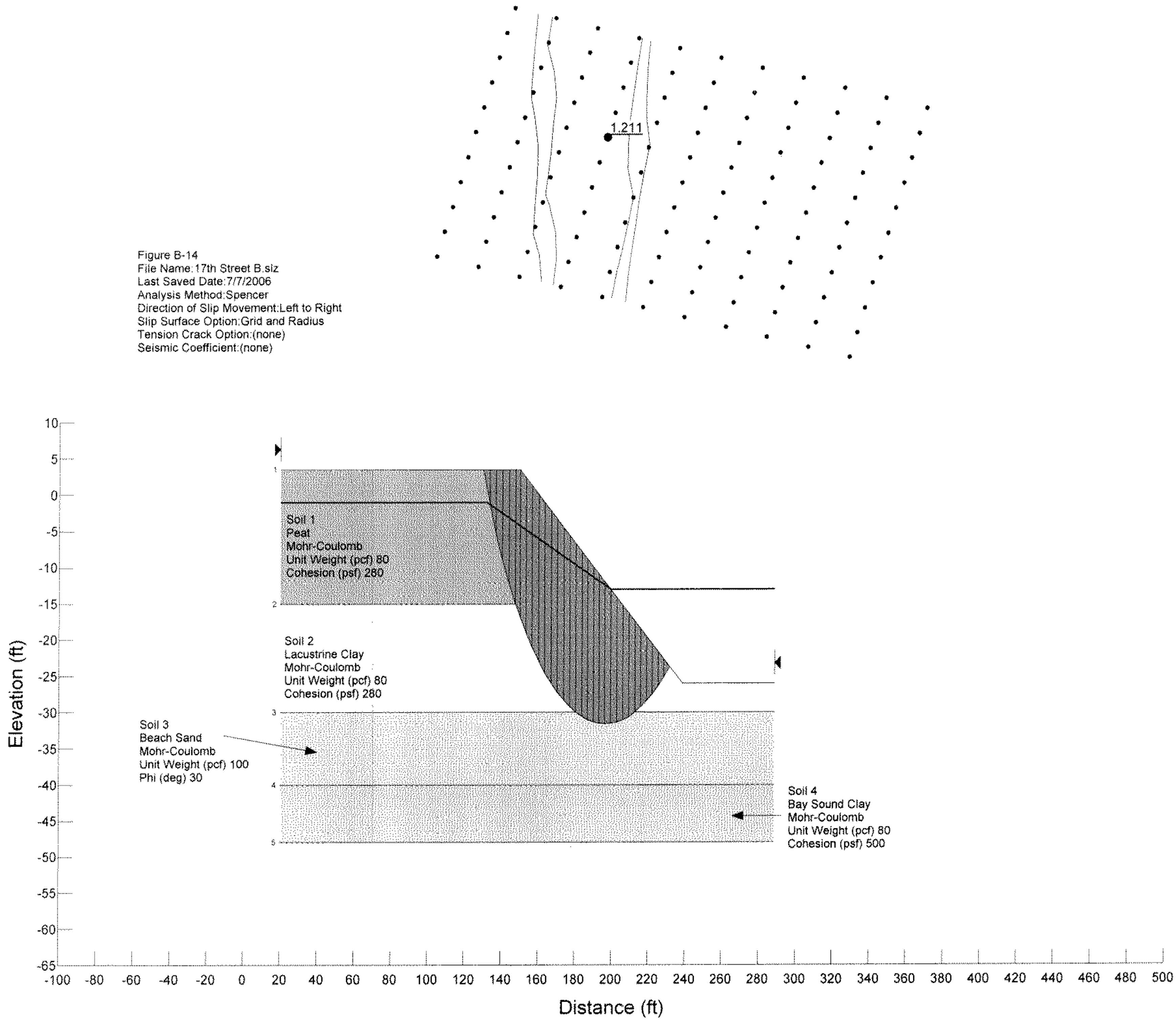
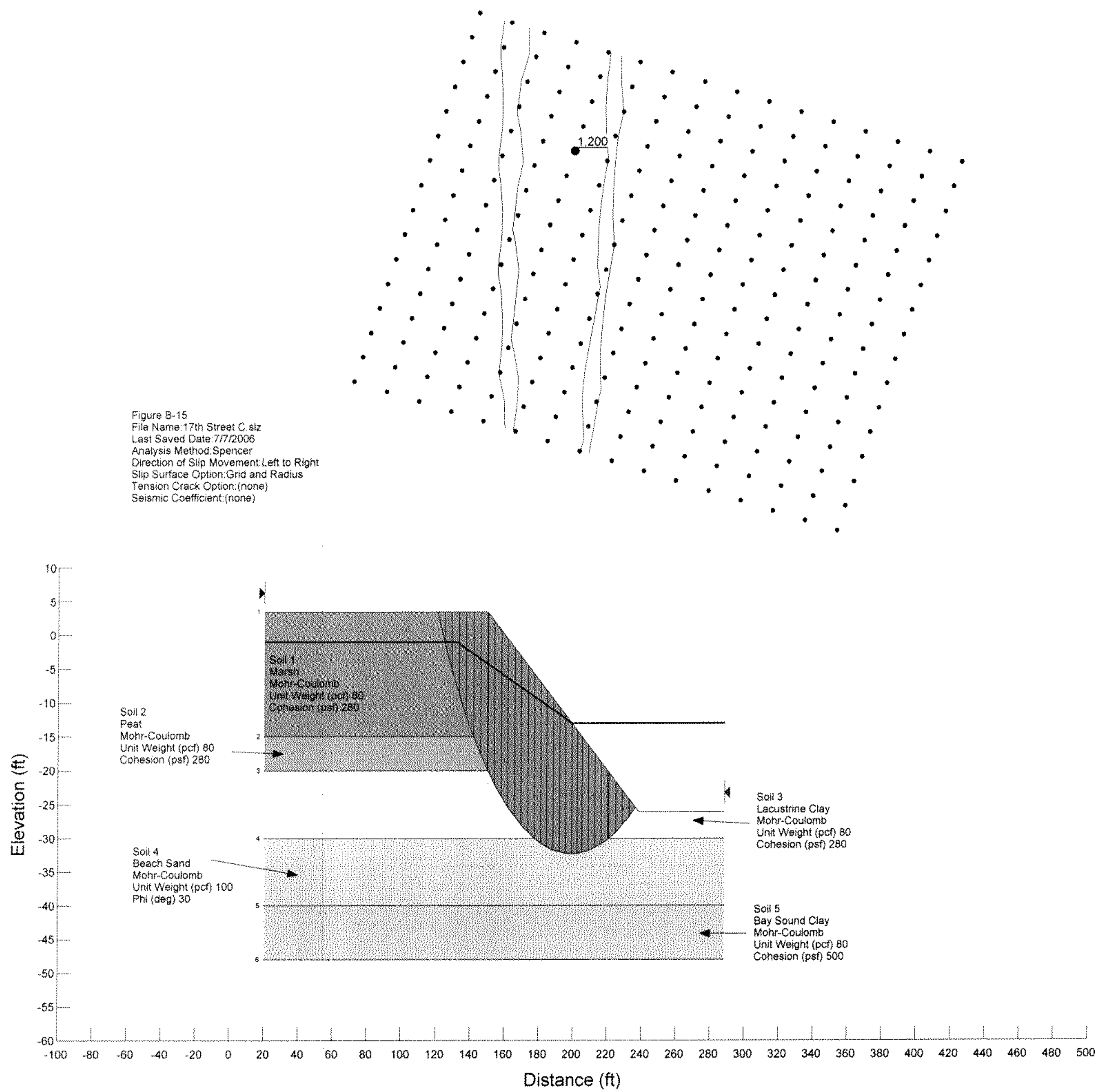


Figure B-14
File Name:17th Street B.siz
Last Saved Date:7/7/2006
Analysis Method:Spencer
Direction of Slip Movement:Left to Right
Slip Surface Option:Grid and Radius
Tension Crack Option:(none)
Seismic Coefficient:(none)
1.211
Soil 1
Peat
Mohr-Coulomb
Unit Weight (pcf) 80
Cohesion (psf) 280
Soil 2
Lacustrine Clay
Mohr-Coulomb
Unit Weight (pcf) 80
Cohesion (psf) 280
Soil 3
Beach Sand
Mohr-Coulomb
Unit Weight (pcf) 100
Phi (deg) 30
Soil 4
Bay Sound Clay
Mohr-Coulomb
Unit Weight (pcf) 80
Cohesion (psf) 500
Elevation (ft)
Distance (ft)



1.200
Figure B-15
File Name:17th Street C.slz
Last Saved Date:7/7/2006
Analysis Method:Spencer
Direction of Slip Movement:Left to Right
Slip Surface Option:Grid and Radius
Tension Crack Option:(none)
Seismic Coefficient:(none)
Soil 1
Marsh
Mohr-Coulomb
Unit Weight (pcf) 80
Cohesion (psf) 280
Soil 2
Peat
Mohr-Coulomb
Unit Weight (pcf) 80
Cohesion (psf) 280
Soil 3
Lacustrine Clay
Mohr-Coulomb
Unit Weight (pcf) 80
Cohesion (psf) 280
Soil 4
Beach Sand
Mohr-Coulomb
Unit Weight (pcf) 100
Phi (deg) 30
Soil 5
Bay Sound Clay
Mohr-Coulomb
Unit Weight (pcf) 80
Cohesion (psf) 500
Elevation (ft)
Distance (ft)

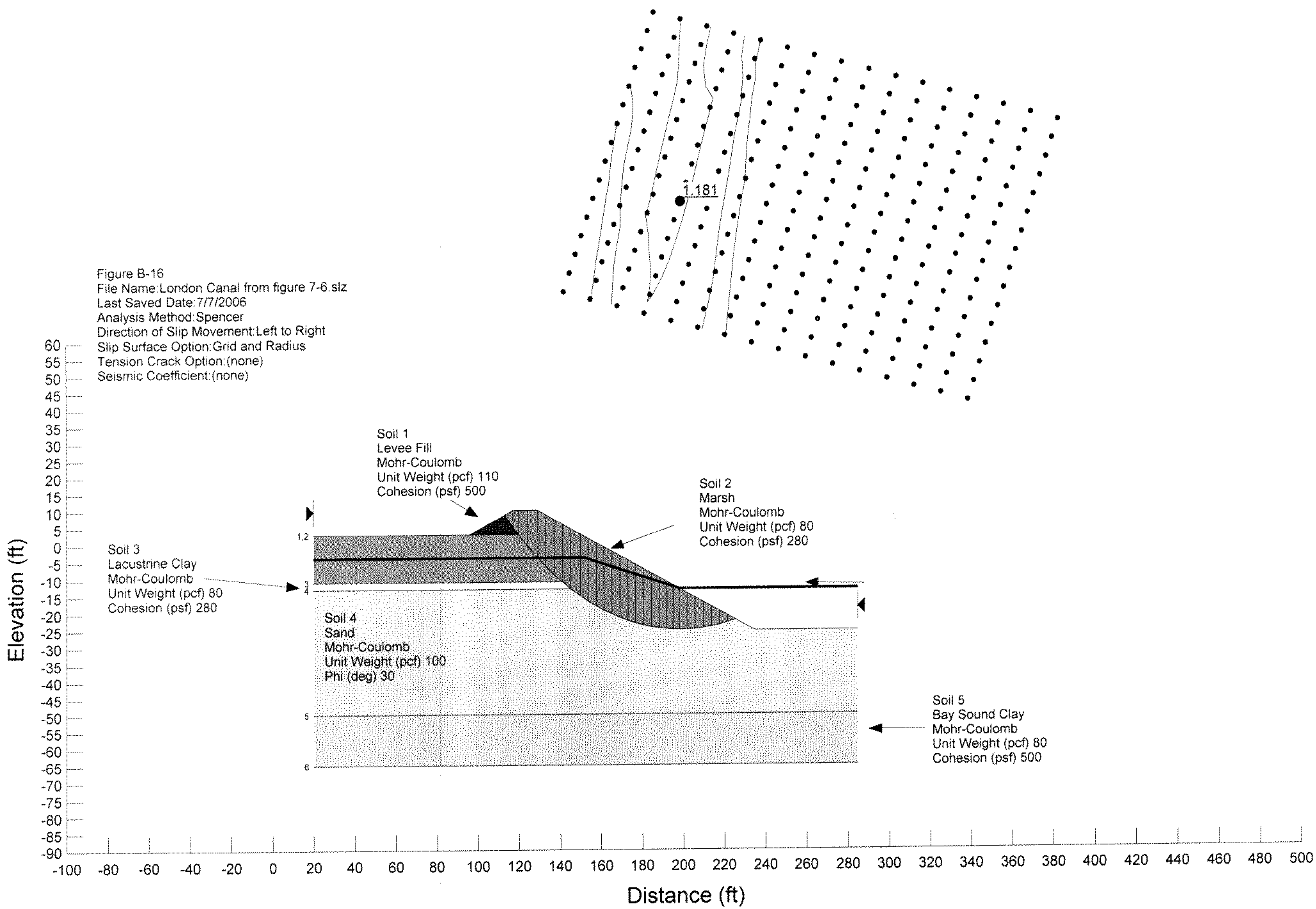
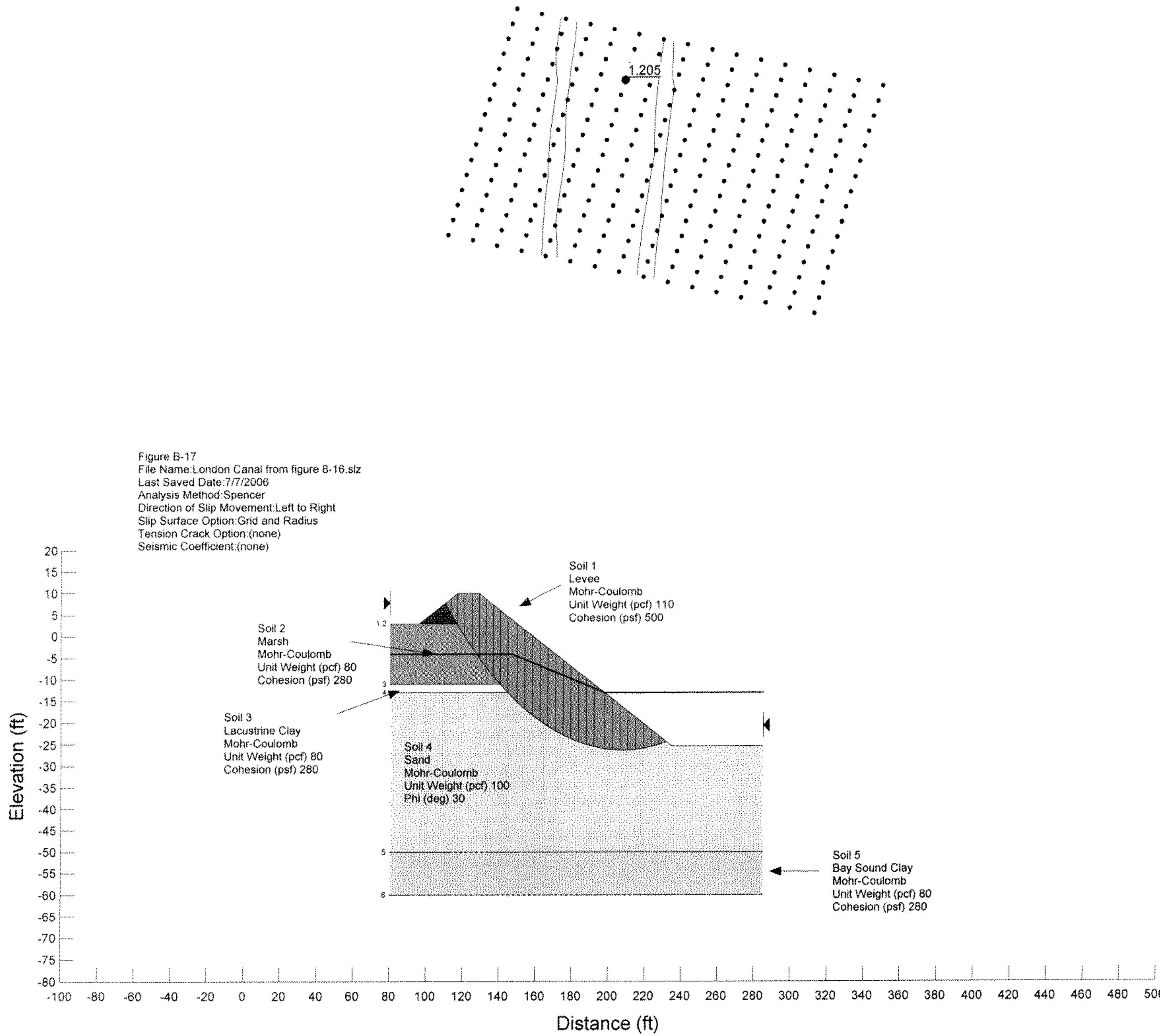


Figure B-16
File Name:London Canal from figure 7-6.slz
Last Saved Date:7/7/2006
Analysis Method:Spencer
Direction of Slip Movement:Left to Right
Slip Surface Option:Grid and Radius
Tension Crack Option:(none)
Seismic Coefficient:(none)
1.181
Soil 1
Levee Fill
Mohr-Coulomb
Unit Weight (pcf) 110
Cohesion (psf) 500
Soil 2
Marsh
Mohr-Coulomb
Unit Weight (pcf) 80
Cohesion (psf) 280
Soil 3
Lacustrine Clay
Mohr-Coulomb
Unit Weight (pcf) 80
Cohesion (psf) 280
Soil 4
Sand
Mohr-Coulomb
Unit Weight (pcf) 100
Phi (deg) 30
Soil 5
Bay Sound Clay
Mohr-Coulomb
Unit Weight (pcf) 80
Cohesion (psf) 500
Elevation (ft)
Distance (ft)



1.205
Figure B-17
File Name:London Canal from figure 8-16.slz
Last Saved Date:7/7/2006
Analysis Method:Spencer
Direction of Slip Movement:Left to Right
Slip Surface Option:Grid and Radius
Tension Crack Option:(none)
Seismic Coefficient:(none)
Soil 1
Levee
Mohr-Coulomb
Unit Weight (pcf) 110
Cohesion (psf) 500
Soil 2
Marsh
Mohr-Coulomb
Unit Weight (pcf) 80
Cohesion (psf) 280
Soil 3
Lacustrine Clay
Mohr-Coulomb
Unit Weight (pcf) 80
Cohesion (psf) 280
Soil 4
Sand
Mohr-Coulomb
Unit Weight (pcf) 100
Phi (deg) 30
Soil 5
Bay Sound Clay
Mohr-Coulomb
Unit Weight (pcf) 80
Cohesion (psf) 280
Elevation (ft)
Distance (ft)

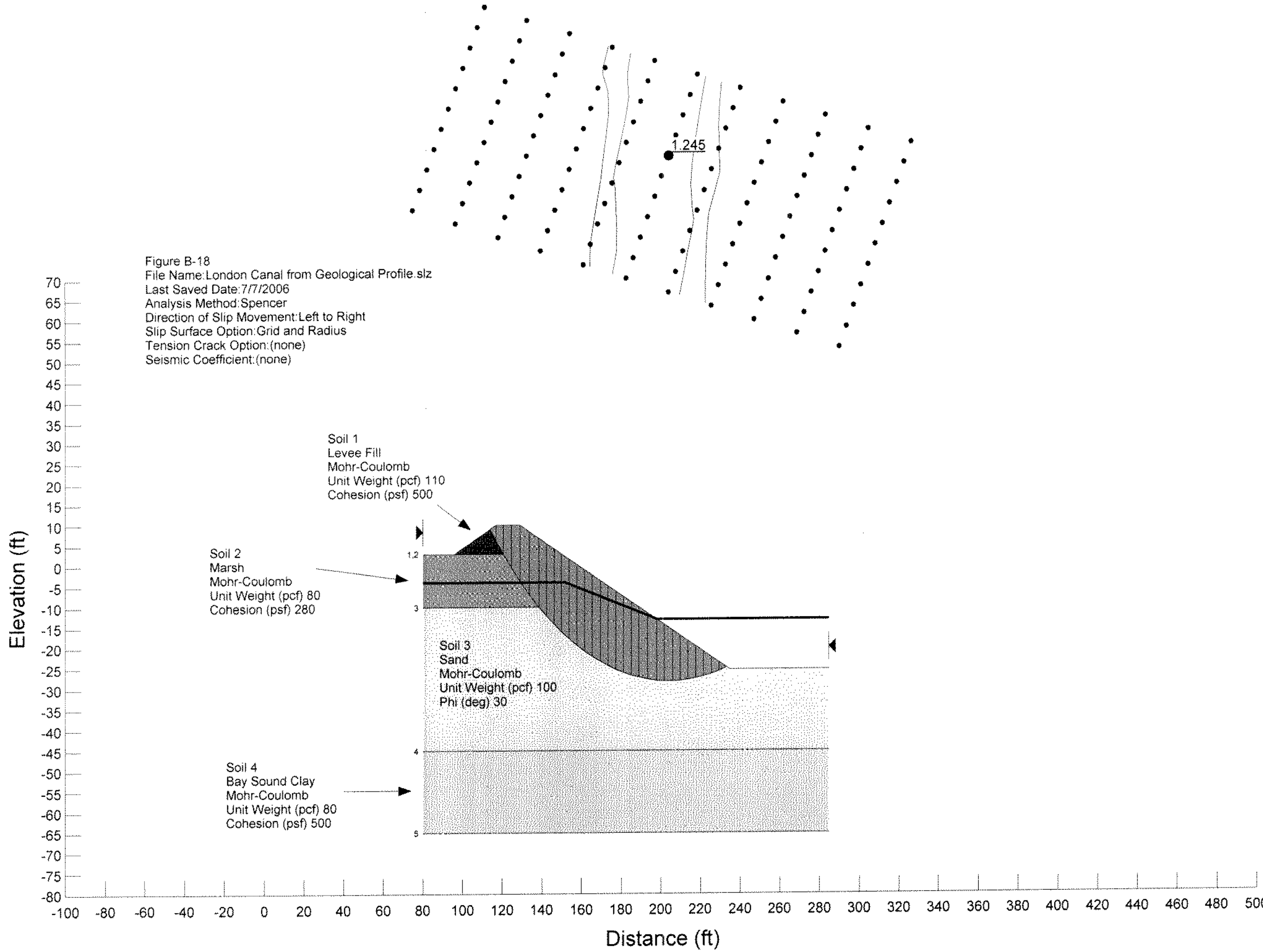


Figure B-18
File Name:London Canal from Geological Profile.slz
Last Saved Date:7/7/2006
Analysis Method:Spencer
Direction of Slip Movement:Left to Right
Slip Surface Option:Grid and Radius
Tension Crack Option:(none)
Seismic Coefficient:(none)
1.245
Soil 1
Levee Fill
Mohr-Coulomb
Unit Weight (pcf) 110
Cohesion (psf) 500
Soil 2
Marsh
Mohr-Coulomb
Unit Weight (pcf) 80
Cohesion (psf) 280
Soil 3
Sand
Mohr-Coulomb
Unit Weight (pcf) 100
Phi (deg) 30
Soil 4
Bay Sound Clay
Mohr-Coulomb
Unit Weight (pcf) 80
Cohesion (psf) 500
Elevation (ft)
Distance (ft)

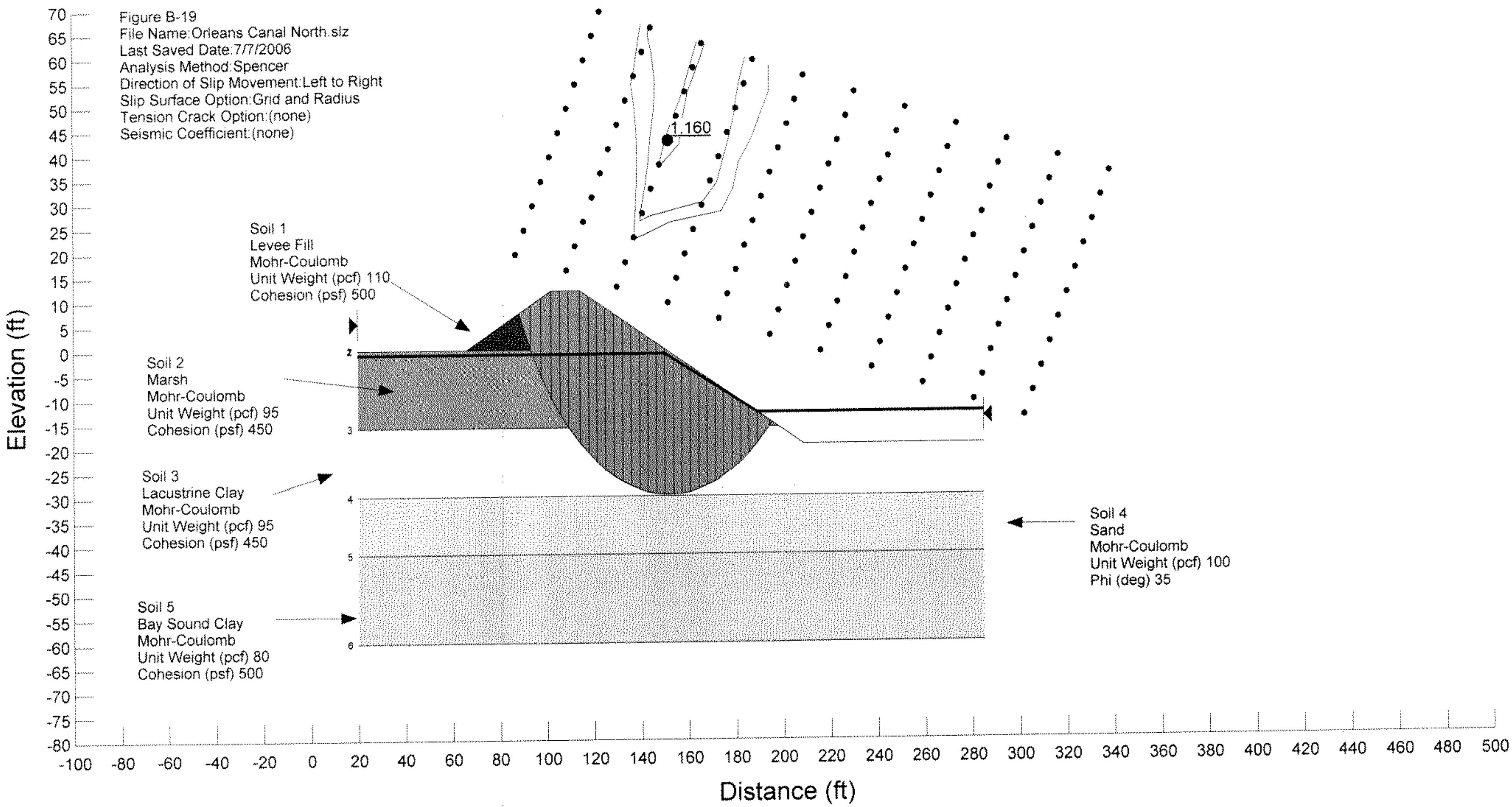


Figure B-19
File Name:Orleans Canal North.slz
Last Saved Date:7/7/2006
Analysis Method:Spencer
Direction of Slip Movement:Left to Right
Slip Surface Option:Grid and Radius
Tension Crack Option:(none)
Seismic Coefficient:(none)
1.160
Soil 1
Levee Fill
Mohr-Coulomb
Unit Weight (pcf) 110
Cohesion (psf) 500
Soil 2
Marsh
Mohr-Coulomb
Unit Weight (pcf) 95
Cohesion (psf) 450
Soil 3
Lacustrine Clay
Mohr-Coulomb
Unit Weight (pcf) 95
Cohesion (psf) 450
Soil 4
Sand
Mohr-Coulomb
Unit Weight (pcf) 100
Phi (deg) 35
Soil 5
Bay Sound Clay
Mohr-Coulomb
Unit Weight (pcf) 80
Cohesion (psf) 500
Elevation (ft)
Distance (ft)

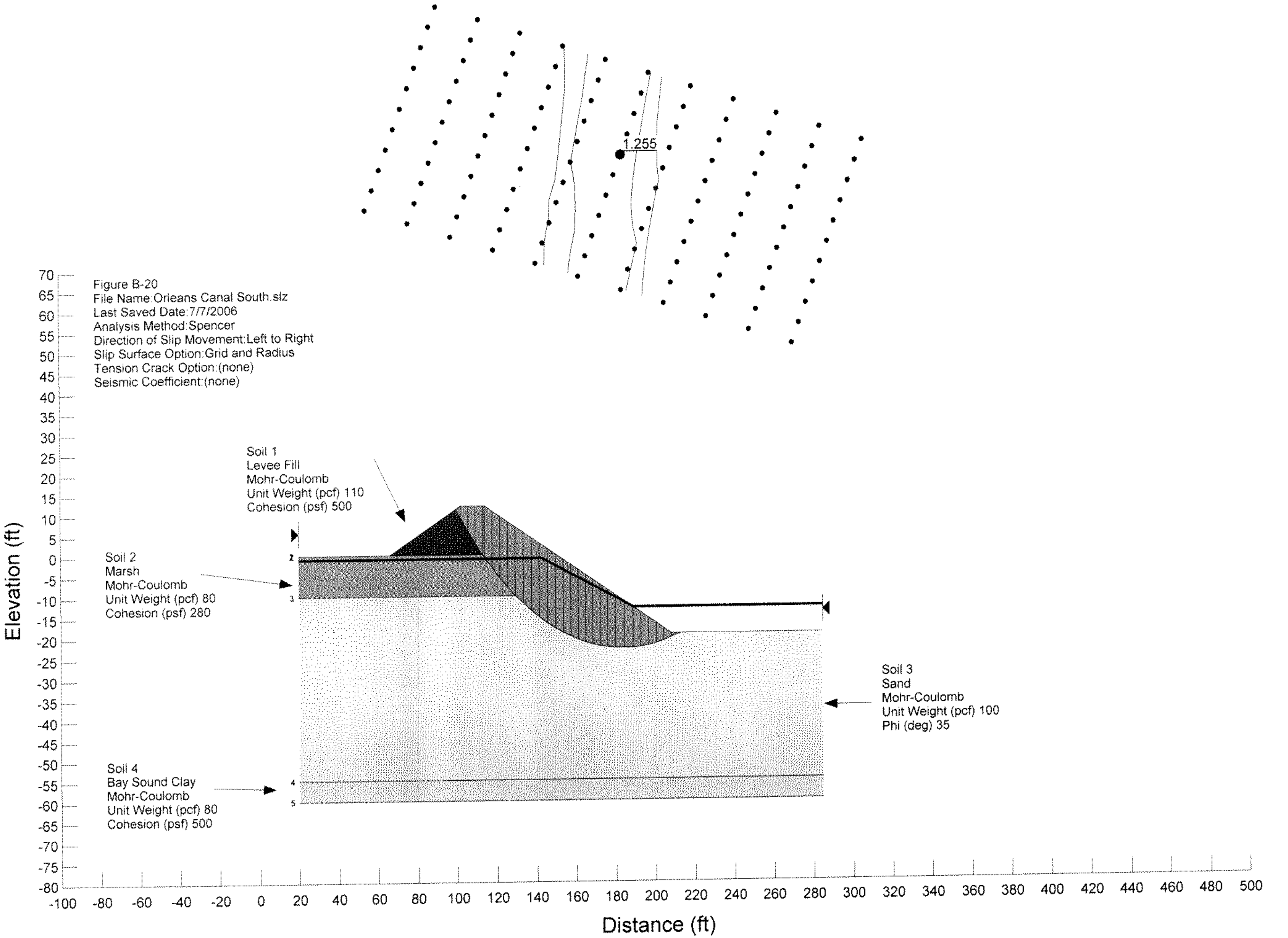
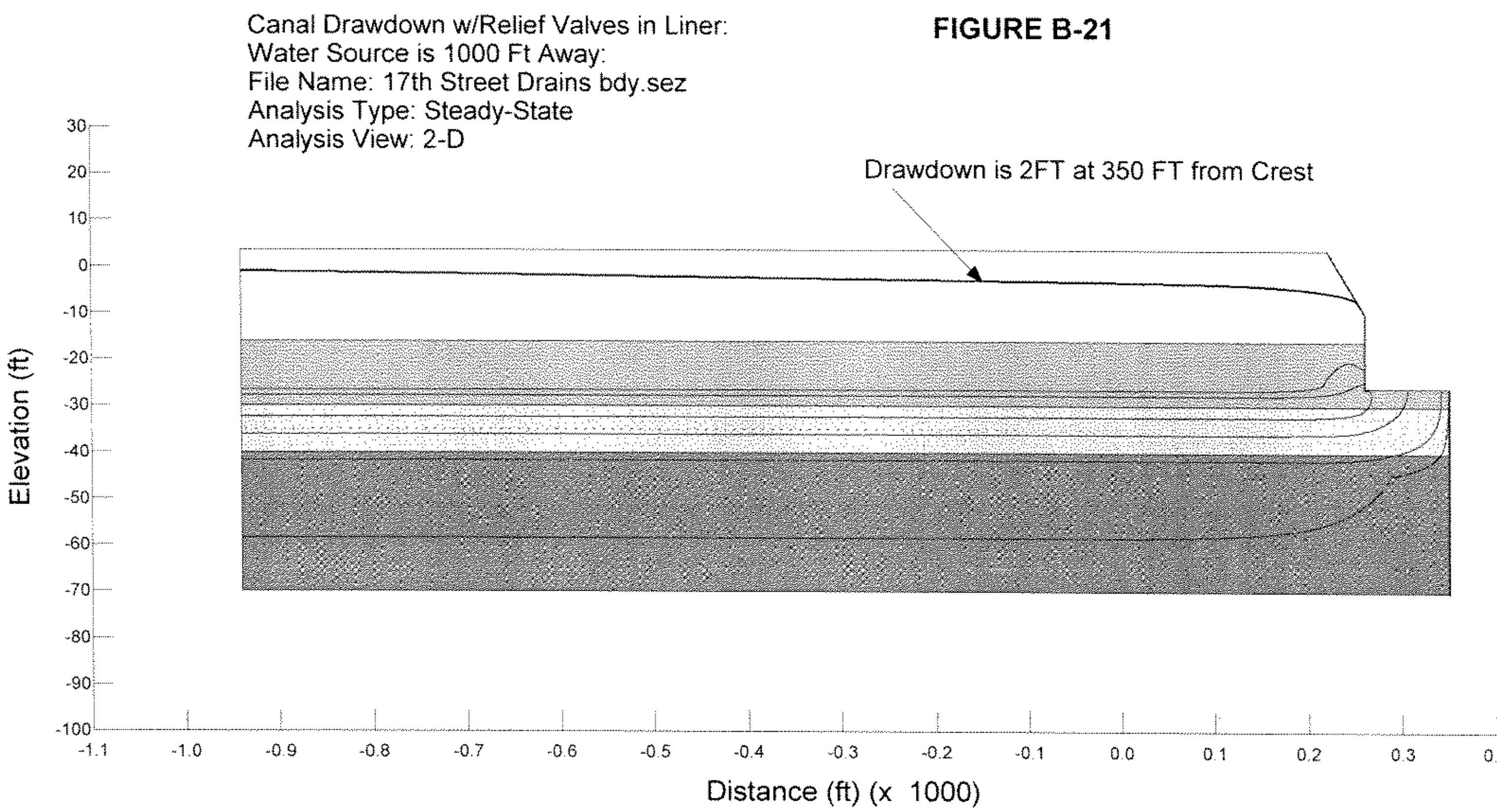


Figure B-20
File Name:Orleans Canal South.slz
Last Saved Date:7/7/2006
Analysis Method:Spencer
Direction of Slip Movement:Left to Right
Slip Surface Option:Grid and Radius
Tension Crack Option:(none)
Seismic Coefficient:(none)
1.255
Soil 1
Levee Fill
Mohr-Coulomb
Unit Weight (pcf) 110
Cohesion (psf) 500
Soil 2
Marsh
Mohr-Coulomb
Unit Weight (pcf) 80
Cohesion (psf) 280
Soil 3
Sand
Mohr-Coulomb
Unit Weight (pcf) 100
Phi (deg) 35
Soil 4
Bay Sound Clay
Mohr-Coulomb
Unit Weight (pcf) 80
Cohesion (psf) 500
Elevation (ft)
Distance (ft)

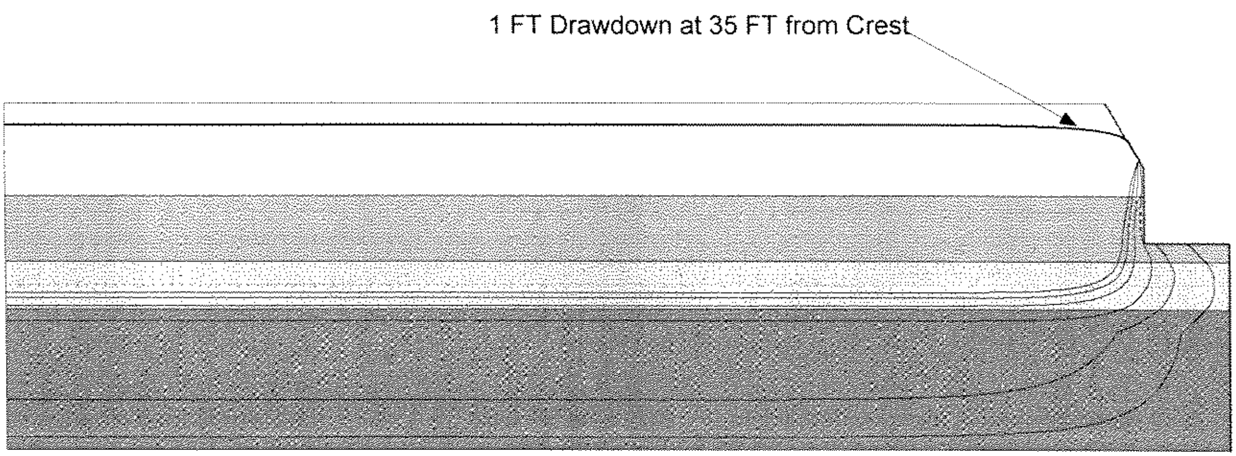
FIGURE B-21



Canal Drawdown w/Relief Valves in Liner:
Water Source is 1000 Ft Away:
File Name: 17th Street Drains bdy.sez
Analysis Type: Steady-State
Analysis View: 2-D
Drawdown is 2FT at 350 FT from Crest
Elevation (ft)
30
20
10
0
-10
-20
-30
-40
-50
-60
-70
-80
-90
-100
-1.1
-1.0
-0.9
-0.8
-0.7
-0.6
-0.5
-0.4
-0.3
-0.2
-0.1
0.0
0.1
0.2
0.3
Distance (ft) (x 1000)

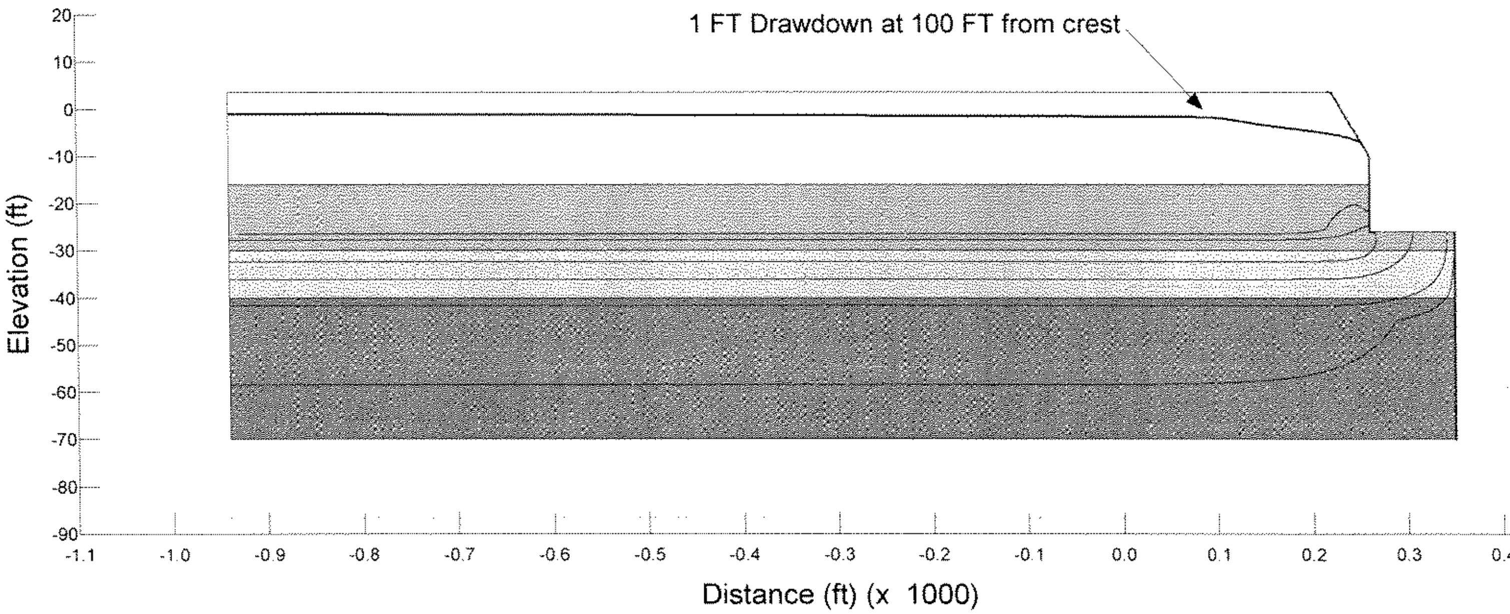
Canal Drawdown w/ Watertight Liner:
Water source 1000 ft away:
File Name: 17th Street watertight.sez
Analysis Type: Steady-State
Analysis View: 2-D

FIGURE B-22



Canal Drawdown w/ relief valves :
Includes internal drainage as water source:
File Name: 17th Street Drains Drains.sez
Analysis Type: Steady-State
Analysis View: 2-D

FIGURE B-23



17th St. Canal Pump Station (Option 2):

Structure Geometry:

Effective Width of the Structure (from Mechanical Calcs): $B_{eff} := 80ft$

Variables:

Elevation of Channel Invert: $EL_{invert} := -27ft$

Bottom of Pump Structure: $EL_{c\_bot} := -38ft$

Elevation of water in the channel: $EL_{wps} := -13ft$

Water Level in Lake Ponchatrain: $EL_{we} := 12ft$

Unit Weight of Water: $\gamma_w := 62.4pcf$

Unit Weight of Concrete: $\gamma_{concrete} := 145pcf$

Vertical Geometry:

Check the stability of the structure at the potential failure plane along the bottom of the structure.

Failure Plane: $\phi_{soil} := 30deg$

$EL_f := EL_{c\_bot}$ $\phi_{fail} := \phi_{soil}$ Plane := "Bottom of Cell"

Case 1

Determine the thickness of the base slab to resist uplift using only the dead weight of the slab. Solve by iterating the bottom of the slab elevation until FS=1.1

w_{str1} - is the lower bound value of the weight of the structure used for uplift: $w_{str1} := 250\text{psf}$

weight of the water in the canal: $w_{canal} := \left(EL_{wps} - EL_{invert}\right)\cdot\gamma_w$

weight of the structure below grade: $w_{slab} := \left(EL_{invert} - EL_{c\_bot}\right)\cdot\gamma_{concrete}$

uplift pressure from the lake: $Lake := \gamma_w\cdot\left(\left(EL_{we} - EL_{c\_bot}\right)\right)$

uplift pressure from the canal: $Canal := \gamma_w\cdot\left(EL_{wps} - EL_{c\_bot}\right)$

Average Uplift: $Uplift_{avg} := \dfrac{Lake + Canal}{2}$

Gravity Load: $w_{gravity} := w_{str1} + w_{canal} + w_{slab}$

$$FS := \frac{w_{gravity}}{Uplift_{avg}}$$

$$FS = 1.16$$

Lateral Loads:

Force of water:

$$F_{w1} := \frac{1}{2}\gamma_w\cdot\left(EL_{we} - EL_f\right)^2 \qquad F_{w1} = 78\,\frac{kip}{ft}$$

Force of the water from Lake Ponchatrain

$$F_{w2} := \frac{1}{2}\gamma_w\cdot\left(EL_{wps} - EL_f\right)^2 \qquad F_{w2} = 19.5\,\frac{kip}{ft}$$

Force of the water from the canal

Moment Arm: $l_{w1} := \left[\frac{1}{3}\cdot\left(EL_{we} - EL_f\right)\right]$

$$l_{w2} := \left[\frac{1}{3}\cdot\left(EL_{wps} - EL_f\right)\right]$$

Depth of slab of the pump structure:

$$H := EL_{invert} - EL_f$$

Active Pressure:

$$K_a := \frac{(1 - \sin(\phi_{soil}))}{(1 + \sin(\phi_{soil}))} \qquad P_a := 0.5 \cdot \gamma_{soil\_b} \cdot K_a \cdot H^2 \qquad P_a = 1.61 \frac{kip}{ft}$$

Moment Arm: $l_a := \left[\frac{1}{3} \cdot (EL_{invert} - EL_{c\_bot})\right]$

Passive Pressure:

In EM 1110-2-2502, the passive pressure phi angle is reduced and it is appropriate to perform the same operation here.

$$FS := 1.5$$

$$\phi_{soil\_p} := \operatorname{atan}\left(\frac{\tan(\phi_{soil})}{FS}\right) \qquad \phi_{soil\_p} = 21.05\,deg$$

$$K_p := \frac{(1 + \sin(\phi_{soil\_p}))}{(1 - \sin(\phi_{soil\_p}))} \qquad P_p := 0.5 \cdot \gamma_{soil\_b} \cdot K_p \cdot H^2 \qquad P_p = 10.27 \frac{kip}{ft}$$

Moment arm: $l_p := \left[\frac{1}{3} \cdot (EL_{invert} - EL_{c\_bot})\right]$

Vertical Loads:

Load from the structure(upper bound value for designing foundations) : $w_{str2} := 500psf$

Weight of the Structure: $W_{str} := B_{eff} \cdot w_{str2}$

$$W_{str} = 40 \frac{kip}{ft}$$

Moment Arm: $l_{str} := \frac{B_{eff}}{2}$

Total Uplift Pressure: $Area_1 := B_{eff} \cdot (\gamma_w) \cdot (EL_{wps} - EL_{c\_bot})$

$$l_{area1} := \frac{B_{eff}}{2}$$

$$Area_2 := 0.5 B_{eff} \cdot \gamma_w \cdot (EL_{we} - EL_{c\_bot})$$

$$l_{area2} := \frac{2}{3} B_{eff}$$

Weight of the Slab: $W_{slab} := B_{eff} \cdot \gamma_{concrete} \cdot (EL_{invert} - EL_{c\_bot})$

$$W_{slab} = 127.6 \frac{kip}{ft}$$

Moment Arm: $l_{slab} := \frac{B_{eff}}{2}$

Calculate Total Moments and Forces:

Moments are calculated about the point of rotation (the toe of the structure).

Resisting Moment: $M_{resist} := P_p \cdot l_p + F_{w2} \cdot l_{w2} + W_{str} \cdot l_{str} + W_{slab} \cdot l_{slab}$

Overturning Moment $M_{over} := F_{w1} \cdot l_{w1} + P_a \cdot l_a + Area_1 \cdot l_{area1} + Area_2 \cdot l_{area2}$

Net Moment: $M_{net} := (M_{resist}) - M_{over}$

$M_{resist} = 6904.1\,kip$ $\quad M_{over} = 1.3 \times 10^4\,kip$ $\quad M_{net} = -6050\,kip$

Since Mnet is negative, uplift resistance is necessary either through pile resistance or some other method.

Re- Calculate Total Moments and Forces with Required Tension:

Tension Required: $T_{rqd} := 145 \frac{kip}{ft}$

Moment Arm of Tension: $l_{rqd} := \frac{2}{3} \cdot B_{eff}$

Resisting Moment: $M_{resist} := P_p \cdot l_p + F_{w2} \cdot l_{w2} + W_{str} \cdot l_{str} + W_{slab} \cdot l_{slab} + T_{rqd} \cdot l_{rqd}$

Overturning Moment $M_{over} := F_{w1} \cdot l_{w1} + P_a \cdot l_a + Area_1 \cdot l_{area1} + Area_2 \cdot l_{area2}$

Net Moment: $M_{net} := (M_{resist}) - M_{over}$

$M_{resist} = 14637.5\,kip$ $\quad M_{over} = 1.3 \times 10^4\,kip$ $\quad M_{net} = 1684\,kip$

Total Gross Weight/Vertical Forces: $F_{vert} := W_{str} - Area_1 - Area_2 + W_{slab} + T_{rqd}$

$$F_{vert} = 63 \frac{kip}{ft}$$

Resultant Location:

$$x_{r1} := \frac{M_{net}}{F_{vert}} \qquad x_{r1} = 26.72\,\text{ft}$$

$$\%_{comp1} := \min\left(3 \cdot \frac{x_{r1}}{B_{eff}}, 1\right) \qquad \%_{comp1} = 100\,\%$$

Per criteria resultant must be within middle third which is equivalent to 100% of base in compression, overturning is OK

$$\%_{comp\_norm} := 100\%$$

$$Check_{overturn\_norm} := \text{if}\left(\%_{comp1} \ge \%_{comp\_norm}, \text{"OK"}, \text{"No Good"}\right)$$

$$Check_{overturn\_norm} = \text{"OK"} \qquad Plane = \text{"Bottom of Cell"}$$

Forces:

Total Driving Force: $F_{drive} := F_{w1} + P_a$ $\qquad F_{drive} = 79.6\,\frac{kip}{ft}$

Total Horizontal Force: $F_{resist} := P_p + F_{w2}$ $\qquad F_{resist} = 29.8\,\frac{kip}{ft}$

External Stability Check - Load Case 1

Sliding Factor of Safety Criteria: $FOS_{slide\_norm} := 1.5$

Shear Strength required: $S_{required} := 54\,\frac{kip}{ft}$

$$V_{all} := F_{vert} \cdot \tan(\phi_{fail}) + S_{required}$$

$$FOS_{slide} := \frac{(V_{all} + F_{resist})}{F_{drive}} \qquad FOS_{slide} = 1.51$$

$$Check_{slide\_norm} := \text{if}\left(FOS_{slide} \ge FOS_{slide\_norm}, \text{"OK"}, \text{"No Good"}\right)$$

$$Check_{slide\_norm} = \text{"OK"} \qquad Plane = \text{"Bottom of Cell"}$$

Bearing Capacity Check:

Eccentricity $e_1 := \frac{B_{eff}}{2} - x_{r1}$ $e_1 = 13.28\,ft$

Maximum Foundation Pressure:

$$q_{bearing} := if\left[\%_{comp1} < 1.0, \left[4 \cdot \frac{F_{vert}}{3 \cdot \left[\left(B_{eff}\right) - 2 \cdot e_1\right]}\right], \frac{F_{vert}}{B_{eff}} \cdot \left(1 + 6 \cdot \frac{e_1}{B_{eff}}\right)\right] \qquad q_{bearing} = 1.57\,\frac{kip}{ft^2}$$

$$q_{bearing} := if\left(\%_{comp1} \le 0.0, 0 \cdot ksf, q_{bearing}\right)$$

$$q_{min} := if\left[\%_{comp1} < 1.0, 0 \cdot ksf, \frac{F_{vert}}{B_{eff}} \cdot \left(1 - 6 \cdot \frac{e_1}{B_{eff}}\right)\right] \qquad q_{min} = 3.34 \times 10^{-3}\,ksf$$

London Canal Pump Station (Option 2):

Structure Geometry:

Effective Width of the Structure (from Mechanical Calcs): $B_{eff} := 80ft$

Variables:

Elevation of Channel Invert: $EL_{invert} := -27ft$

Bottom of Pump Structure: $EL_{c\_bot} := -38ft$

Elevation of water in the channel: $EL_{wps} := -13ft$

Water Level in Lake Ponchatrain: $EL_{we} := 12ft$

Unit Weight of Water: $\gamma_w := 62.4pcf$

Unit Weight of Concrete: $\gamma_{concrete} := 145pcf$

Vertical Geometry:

Check the stability of the structure at the potential failure plane along the bottom of the structure.

Failure Plane: $\phi_{soil} := 30deg$

$EL_f := EL_{c\_bot}$ $\phi_{fail} := \phi_{soil}$ Plane := "Bottom of Cell"

Case 1

Determine the thickness of the base slab to resist uplift using only the dead weight of the slab. Solve by iterating the bottom of the slab elevation until FS=1.1

w_{str1} - is the lower bound value of the weight of the structure used for uplift: $w_{str1} := 250psf$

weight of the water in the canal: $w_{canal} := (EL_{wps} - EL_{invert}) \cdot \gamma_w$

weight of the structure below grade: $w_{slab} := (EL_{invert} - EL_{c\_bot}) \cdot \gamma_{concrete}$

uplift pressure from the lake: $Lake := \gamma_w \cdot ((EL_{we} - EL_{c\_bot}))$

uplift pressure from the canal: $Canal := \gamma_w \cdot (EL_{wps} - EL_{c\_bot})$

Average Uplift: $Uplift_{avg} := \dfrac{Lake + Canal}{2}$

Gravity Load: $w_{gravity} := w_{str1} + w_{canal} + w_{slab}$

$$FS := \frac{w_{gravity}}{Uplift_{avg}}$$

$FS = 1.16$

Lateral Loads:

Force of water:

$F_{w1} := \frac{1}{2}\gamma_w \cdot (EL_{we} - EL_f)^2$ $\quad F_{w1} = 78 \frac{kip}{ft}$ Force of the water from Lake Ponchatrain

$F_{w2} := \frac{1}{2}\gamma_w \cdot (EL_{wps} - EL_f)^2$ $\quad F_{w2} = 19.5 \frac{kip}{ft}$ Force of the water from the canal

Moment Arm: $l_{w1} := \left[\frac{1}{3} \cdot (EL_{we} - EL_f)\right]$

$l_{w2} := \left[\frac{1}{3} \cdot (EL_{wps} - EL_f)\right]$

Depth of slab of the pump structure:

$H := EL_{invert} - EL_f$

Active Pressure:

$$K_a := \frac{(1 - \sin(\phi_{soil}))}{(1 + \sin(\phi_{soil}))} \qquad P_a := 0.5 \cdot \gamma_{soil\_b} \cdot K_a \cdot H^2 \qquad P_a = 1.61 \frac{kip}{ft}$$

Moment Arm: $l_a := \left[\frac{1}{3} \cdot (EL_{invert} - EL_{c\_bot})\right]$

Passive Pressure:

In EM 1110-2-2502, the passive pressure phi angle is reduced and it is appropriate to perform the same operation here.

$FS := 1.5$

$$\phi_{soil\_p} := \operatorname{atan}\left(\frac{\tan(\phi_{soil})}{FS}\right) \qquad \phi_{soil\_p} = 21.05\,deg$$

$$K_p := \frac{(1 + \sin(\phi_{soil\_p}))}{(1 - \sin(\phi_{soil\_p}))} \qquad P_p := 0.5 \cdot \gamma_{soil\_b} \cdot K_p \cdot H^2 \qquad P_p = 10.27 \frac{kip}{ft}$$

Moment arm: $l_p := \left[\frac{1}{3} \cdot (EL_{invert} - EL_{c\_bot})\right]$

Vertical Loads:

Load from the structure(upper bound value for designing foundations) : $w_{str2} := 500psf$

Weight of the Structure: $W_{str} := B_{eff} \cdot w_{str2}$

$$W_{str} = 40 \frac{kip}{ft}$$

Moment Arm: $l_{str} := \frac{B_{eff}}{2}$

Total Uplift Pressure: $Area_1 := B_{eff} \cdot (\gamma_w) \cdot (EL_{wps} - EL_{c\_bot})$

$$l_{area1} := \frac{B_{eff}}{2}$$

$$Area_2 := 0.5 B_{eff} \cdot \gamma_w \cdot (EL_{we} - EL_{c\_bot})$$

$$l_{area2} := \frac{2}{3} B_{eff}$$

Weight of the Slab: $W_{slab} := B_{eff} \cdot \gamma_{concrete} \cdot (EL_{invert} - EL_{c\_bot})$

$$W_{slab} = 127.6 \frac{kip}{ft}$$

Moment Arm: $l_{slab} := \frac{B_{eff}}{2}$

Calculate Total Moments and Forces:

Moments are calculated about the point of rotation (the toe of the structure).

Resisting Moment: $M_{resist} := P_p \cdot l_p + F_{w2} \cdot l_{w2} + W_{str} \cdot l_{str} + W_{slab} \cdot l_{slab}$

Overturning Moment $M_{over} := F_{w1} \cdot l_{w1} + P_a \cdot l_a + Area_1 \cdot l_{area1} + Area_2 \cdot l_{area2}$

Net Moment: $M_{net} := (M_{resist}) - M_{over}$

$M_{resist} = 6904.1\, kip$ $M_{over} = 1.3 \times 10^4\, kip$ $M_{net} = -6050\, kip$

Since Mnet is negative, uplift resistance is necessary either through pile resistance or some other method.

Re- Calculate Total Moments and Forces with Required Tension:

Tension Required: $T_{rqd} := 145 \frac{kip}{ft}$

Moment Arm of Tension: $l_{rqd} := \frac{2}{3} \cdot B_{eff}$

Resisting Moment: $M_{resist} := P_p \cdot l_p + F_{w2} \cdot l_{w2} + W_{str} \cdot l_{str} + W_{slab} \cdot l_{slab} + T_{rqd} \cdot l_{rqd}$

Overturning Moment $M_{over} := F_{w1} \cdot l_{w1} + P_a \cdot l_a + Area_1 \cdot l_{area1} + Area_2 \cdot l_{area2}$

Net Moment: $M_{net} := (M_{resist}) - M_{over}$

$M_{resist} = 14637.5\, kip$ $M_{over} = 1.3 \times 10^4\, kip$ $M_{net} = 1684\, kip$

Total Gross Weight/Vertical Forces: $F_{vert} := W_{str} - Area_1 - Area_2 + W_{slab} + T_{rqd}$

$$F_{vert} = 63 \frac{kip}{ft}$$

Resultant Location:

$$x_{r1} := \frac{M_{net}}{F_{vert}} \qquad x_{r1} = 26.72\,\text{ft}$$

$$\%_{comp1} := \min\left(3 \cdot \frac{x_{r1}}{B_{eff}}, 1\right) \qquad \%_{comp1} = 100\,\%$$

Per criteria resultant must be within middle third which is equivalent to 100% of base in compression, overturning is OK

$$\%_{comp\_norm} := 100\%$$

$$Check_{overturn\_norm} := \text{if}\left(\%_{comp1} \geq \%_{comp\_norm}, \text{"OK"}, \text{"No Good"}\right)$$

$$Check_{overturn\_norm} = \text{"OK"} \qquad Plane = \text{"Bottom of Cell"}$$

Forces:

Total Driving Force: $F_{drive} := F_{w1} + P_a$ $\quad F_{drive} = 79.6\,\frac{kip}{ft}$

Total Horizontal Force: $F_{resist} := P_p + F_{w2}$ $\quad F_{resist} = 29.8\,\frac{kip}{ft}$

External Stability Check - Load Case 1

Sliding Factor of Safety Criteria: $FOS_{slide\_norm} := 1.5$

Shear Strength required: $S_{required} := 54\,\frac{kip}{ft}$

$$V_{all} := F_{vert} \cdot \tan(\phi_{fail}) + S_{required}$$

$$FOS_{slide} := \frac{(V_{all} + F_{resist})}{F_{drive}} \qquad FOS_{slide} = 1.51$$

$$Check_{slide\_norm} := \text{if}\left(FOS_{slide} \geq FOS_{slide\_norm}, \text{"OK"}, \text{"No Good"}\right)$$

$$Check_{slide\_norm} = \text{"OK"} \qquad Plane = \text{"Bottom of Cell"}$$

Bearing Capacity Check:

Eccentricity

$$e_1 := \frac{B_{eff}}{2} - x_{r1} \qquad e_1 = 13.28\,\text{ft}$$

Maximum Foundation Pressure:

$$q_{bearing} := \text{if}\left[\%_{comp1} < 1.0, \left[4\cdot\frac{F_{vert}}{3\cdot\left[\left(B_{eff}\right) - 2\cdot e_1\right]}\right], \frac{F_{vert}}{B_{eff}}\cdot\left(1 + 6\cdot\frac{e_1}{B_{eff}}\right)\right] \qquad q_{bearing} = 1.57\,\frac{\text{kip}}{\text{ft}^2}$$

$$q_{bearing} := \text{if}\left(\%_{comp1} \le 0.0, 0\cdot\text{ksf}, q_{bearing}\right)$$

$$q_{min} := \text{if}\left[\%_{comp1} < 1.0, 0\cdot\text{ksf}, \frac{F_{vert}}{B_{eff}}\cdot\left(1 - 6\cdot\frac{e_1}{B_{eff}}\right)\right] \qquad q_{min} = 3.34\times10^{-3}\,\text{ksf}$$

Orleans Canal Pump Station (Option 2):

Structure Geometry:

Effective Width of the Structure (from Mechanical Calcs): $B_{eff} := 70ft$

Variables:

Elevation of Channel Invert: $EL_{invert} := -27ft$

Bottom of Pump Structure: $EL_{c\_bot} := -38ft$

Elevation of water in the channel: $EL_{wps} := -13ft$

Water Level in Lake Ponchatrain: $EL_{we} := 12ft$

Unit Weight of Water: $\gamma_w := 62.4pcf$

Unit Weight of Concrete: $\gamma_{concrete} := 145pcf$

Vertical Geometry:

Check the stability of the structure at the potential failure plane along the bottom of the structure.

Failure Plane: $\phi_{soil} := 30deg$

$EL_f := EL_{c\_bot}$ $\phi_{fail} := \phi_{soil}$ Plane := "Bottom of Cell"

Case 1

Determine the thickness of the base slab to resist uplift using only the dead weight of the slab. Solve by iterating the bottom of the slab elevation until FS=1.1

w_{str1} - is the lower bound value of the weight of the structure used for uplift: $w_{str1} := 250psf$

weight of the water in the canal: $w_{canal} := (EL_{wps} - EL_{invert}) \cdot \gamma_w$

weight of the structure below grade: $w_{slab} := (EL_{invert} - EL_{c\_bot}) \cdot \gamma_{concrete}$

uplift pressure from the lake: $Lake := \gamma_w \cdot ((EL_{we} - EL_{c\_bot}))$

uplift pressure from the canal: $Canal := \gamma_w \cdot (EL_{wps} - EL_{c\_bot})$

Average Uplift: $Uplift_{avg} := \frac{Lake + Canal}{2}$

Gravity Load: $w_{gravity} := w_{str1} + w_{canal} + w_{slab}$

$$FS := \frac{w_{gravity}}{Uplift_{avg}}$$

$$FS = 1.16$$

Lateral Loads:

Force of water:

$$F_{w1} := \frac{1}{2} \gamma_w \cdot (EL_{we} - EL_f)^2 \qquad F_{w1} = 78 \frac{kip}{ft}$$

Force of the water from Lake Ponchatrain

$$F_{w2} := \frac{1}{2} \gamma_w \cdot (EL_{wps} - EL_f)^2 \qquad F_{w2} = 19.5 \frac{kip}{ft}$$

Force of the water from the canal

Moment Arm:

$$l_{w1} := \left[\frac{1}{3} \cdot (EL_{we} - EL_f)\right]$$

$$l_{w2} := \left[\frac{1}{3} \cdot (EL_{wps} - EL_f)\right]$$

Depth of slab of the pump structure:

$$H := EL_{invert} - EL_f$$

Active Pressure:

$$K_a := \frac{(1 - \sin(\phi_{soil}))}{(1 + \sin(\phi_{soil}))} \qquad P_a := 0.5 \cdot \gamma_{soil\_b} \cdot K_a \cdot H^2 \qquad P_a = 1.61 \frac{kip}{ft}$$

Moment Arm: $l_a := \left[\frac{1}{3} \cdot (EL_{invert} - EL_{c\_bot})\right]$

Passive Pressure:

In EM 1110-2-2502, the passive pressure phi angle is reduced and it is appropriate to perform the same operation here.

$FS := 1.5$

$$\phi_{soil\_p} := \operatorname{atan}\left(\frac{\tan(\phi_{soil})}{FS}\right) \qquad \phi_{soil\_p} = 21.05\,deg$$

$$K_p := \frac{(1 + \sin(\phi_{soil\_p}))}{(1 - \sin(\phi_{soil\_p}))} \qquad P_p := 0.5 \cdot \gamma_{soil\_b} \cdot K_p \cdot H^2 \qquad P_p = 10.27 \frac{kip}{ft}$$

Moment arm: $l_p := \left[\frac{1}{3} \cdot (EL_{invert} - EL_{c\_bot})\right]$

<u>Vertical Loads:</u>

Load from the structure(upper bound value for designing foundations) : $w_{str2} := 500psf$

Weight of the Structure: $W_{str} := B_{eff} \cdot w_{str2}$

$$W_{str} = 35 \frac{kip}{ft}$$

Moment Arm: $l_{str} := \frac{B_{eff}}{2}$

Total Uplift Pressure: $Area_1 := B_{eff} \cdot (\gamma_w) \cdot (EL_{wps} - EL_{c\_bot})$

$$l_{area1} := \frac{B_{eff}}{2}$$

$$Area_2 := 0.5 B_{eff} \cdot \gamma_w \cdot (EL_{we} - EL_{c\_bot})$$

$$l_{area2} := \frac{2}{3} B_{eff}$$

Weight of the Slab: $W_{slab} := B_{eff} \cdot \gamma_{concrete} \cdot (EL_{invert} - EL_{c\_bot})$

$$W_{slab} = 111.65 \frac{kip}{ft}$$

Moment Arm: $l_{slab} := \frac{B_{eff}}{2}$

Calculate Total Moments and Forces:

Moments are calculated about the point of rotation (the toe of the structure).

Resisting Moment: $M_{resist} := P_p \cdot l_p + F_{w2} \cdot l_{w2} + W_{str} \cdot l_{str} + W_{slab} \cdot l_{slab}$

Overturning Moment $M_{over} := F_{w1} \cdot l_{w1} + P_a \cdot l_a + Area_1 \cdot l_{area1} + Area_2 \cdot l_{area2}$

Net Moment: $M_{net} := (M_{resist}) - M_{over}$

$M_{resist} = 5332.9\,kip$ $\quad M_{over} = 1 \times 10^4\,kip$ $\quad M_{net} = -4891\,kip$

Since Mnet is negative, uplift resistance is necessary either through pile resistance or some other method.

Re- Calculate Total Moments and Forces with Required Tension:

Tension Required: $T_{rqd} := 140 \frac{kip}{ft}$

Moment Arm of Tension: $l_{rqd} := \frac{2}{3} \cdot B_{eff}$

Resisting Moment: $M_{resist} := P_p \cdot l_p + F_{w2} \cdot l_{w2} + W_{str} \cdot l_{str} + W_{slab} \cdot l_{slab} + T_{rqd} \cdot l_{rqd}$

Overturning Moment $M_{over} := F_{w1} \cdot l_{w1} + P_a \cdot l_a + Area_1 \cdot l_{area1} + Area_2 \cdot l_{area2}$

Net Moment: $M_{net} := (M_{resist}) - M_{over}$

$M_{resist} = 11866.2\,kip$ $\quad M_{over} = 1 \times 10^4\,kip$ $\quad M_{net} = 1642\,kip$

Total Gross Weight/Vertical Forces: $F_{vert} := W_{str} - Area_1 - Area_2 + W_{slab} + T_{rqd}$

$$F_{vert} = 68.3 \frac{kip}{ft}$$

Resultant Location:

$$x_{r1} := \frac{M_{net}}{F_{vert}}$$ $x_{r1} = 24.06\,ft$

$$\%_{comp1} := \min\left(3\cdot\frac{x_{r1}}{B_{eff}}, 1\right)$$ $\%_{comp1} = 100\%$

Per criteria resultant must be within middle third which is equivalent to 100% of base in compression, overturning is OK

$$\%_{comp\_norm} := 100\%$$

$$Check_{overturn\_norm} := if\left(\%_{comp1} \geq \%_{comp\_norm}, \text{"OK"}, \text{"No Good"}\right)$$

$Check_{overturn\_norm} = \text{"OK"}$ Plane = "Bottom of Cell"

Forces:

Total Driving Force: $F_{drive} := F_{w1} + P_a$ $F_{drive} = 79.6\,\frac{kip}{ft}$

Total Horizontal Force: $F_{resist} := P_p + F_{w2}$ $F_{resist} = 29.8\,\frac{kip}{ft}$

External Stability Check - Load Case 1

Sliding Factor of Safety Criteria: $FOS_{slide\_norm} := 1.5$

Shear Strength required: $S_{required} := 50\,\frac{kip}{ft}$

$$V_{all} := F_{vert}\cdot\tan(\phi_{fail}) + S_{required}$$

$$FOS_{slide} := \frac{(V_{all} + F_{resist})}{F_{drive}}$$ $FOS_{slide} = 1.5$

$$Check_{slide\_norm} := if\left(FOS_{slide} \geq FOS_{slide\_norm}, \text{"OK"}, \text{"No Good"}\right)$$

$Check_{slide\_norm} = \text{"No Good"}$ Plane = "Bottom of Cell"

Bearing Capacity Check:

Eccentricity $e_1 := \frac{B_{eff}}{2} - x_{r1}$ $\qquad e_1 = 10.94\,ft$

Maximum Foundation Pressure:

$$q_{bearing} := if\left[\%_{comp1} < 1.0, \left[4 \cdot \frac{F_{vert}}{3 \cdot \left[\left(B_{eff}\right) - 2 \cdot e_1\right]}\right], \frac{F_{vert}}{B_{eff}} \cdot \left(1 + 6 \cdot \frac{e_1}{B_{eff}}\right)\right] \qquad q_{bearing} = 1.89\,\frac{kip}{ft^2}$$

$$q_{bearing} := if\left(\%_{comp1} \le 0.0, 0 \cdot ksf, q_{bearing}\right)$$

$$q_{min} := if\left[\%_{comp1} < 1.0, 0 \cdot ksf, \frac{F_{vert}}{B_{eff}} \cdot \left(1 - 6 \cdot \frac{e_1}{B_{eff}}\right)\right] \qquad q_{min} = 0.06\,ksf$$

17th St. Canal Pump Station (Option 1):

Structure Geometry:

Effective Width of the Structure (from Mechanical Calcs): $B_{eff} := 70ft$

Variables:

Elevation of Channel Invert: $EL_{invert} := -13ft$

Bottom of Pump Structure: $EL_{c\_bot} := -17ft$

Elevation of water in the channel: $EL_{wps} := -1.3ft$

Water Level in Lake Ponchatrain: $EL_{we} := 12ft$

Unit Weight of Water: $\gamma_w := 62.4pcf$

Unit Weight of Concrete: $\gamma_{concrete} := 145pcf$

Vertical Geometry:

Check the stability of the structure at the potential failure plane along the bottom of the structure.

Failure Plane: $\phi_{soil} := 30deg$

$EL_f := EL_{c\_bot}$ $\phi_{fail} := \phi_{soil}$ Plane := "Bottom of Cell"

Case 1

Determine the thickness of the base slab to resist uplift using only the dead weight of the slab. Solve by iterating the bottom of the slab elevation until FS=1.1

w_{str1} - is the lower bound value of the weight of the structure used for uplift: $w_{str1} := 250\text{psf}$

weight of the water in the canal: $w_{canal} := (EL_{wps} - EL_{invert}) \cdot \gamma_w$

weight of the structure below grade: $w_{slab} := (EL_{invert} - EL_{c\_bot}) \cdot \gamma_{concrete}$

uplift pressure from the lake: $Lake := \gamma_w \cdot ((EL_{we} - EL_{c\_bot}))$

uplift pressure from the canal: $Canal := \gamma_w \cdot (EL_{wps} - EL_{c\_bot})$

Average Uplift: $Uplift_{avg} := \dfrac{Lake + Canal}{2}$

Gravity Load: $w_{gravity} := w_{str1} + w_{canal} + w_{slab}$

$$FS := \frac{w_{gravity}}{Uplift_{avg}}$$

$$FS = 1.12$$

Lateral Loads:

Force of water:

$$F_{w1} := \frac{1}{2}\gamma_w \cdot (EL_{we} - EL_f)^2 \qquad F_{w1} = 26.24\frac{\text{kip}}{\text{ft}}$$

Force of the water from Lake Ponchatrain

$$F_{w2} := \frac{1}{2}\gamma_w \cdot (EL_{wps} - EL_f)^2 \qquad F_{w2} = 7.69\frac{\text{kip}}{\text{ft}}$$

Force of the water from the canal

Moment Arm:

$$l_{w1} := \left[\frac{1}{3} \cdot (EL_{we} - EL_f)\right]$$

$$l_{w2} := \left[\frac{1}{3} \cdot (EL_{wps} - EL_f)\right]$$

Depth of slab of the pump structure:

$$H := EL_{invert} - EL_f$$

Active Pressure:

$$K_a := \frac{(1 - \sin(\phi_{soil}))}{(1 + \sin(\phi_{soil}))} \qquad P_a := 0.5 \cdot \gamma_{soil\_b} \cdot K_a \cdot H^2 \qquad P_a = 0.21 \frac{kip}{ft}$$

Moment Arm: $l_a := \left[\frac{1}{3} \cdot (EL_{invert} - EL_{c\_bot})\right]$

Passive Pressure:

In EM 1110-2-2502, the passive pressure phi angle is reduced and it is appropriate to perform the same operation here.

$$FS := 1.5$$

$$\phi_{soil\_p} := \operatorname{atan}\left(\frac{\tan(\phi_{soil})}{FS}\right) \qquad \phi_{soil\_p} = 21.05\,deg$$

$$K_p := \frac{(1 + \sin(\phi_{soil\_p}))}{(1 - \sin(\phi_{soil\_p}))} \qquad P_p := 0.5 \cdot \gamma_{soil\_b} \cdot K_p \cdot H^2 \qquad P_p = 1.36 \frac{kip}{ft}$$

Moment arm: $l_p := \left[\frac{1}{3} \cdot (EL_{invert} - EL_{c\_bot})\right]$

Vertical Loads:

Load from the structure(upper bound value for designing foundations) : $w_{str2} := 500psf$

Weight of the Structure: $W_{str} := B_{eff} \cdot w_{str2}$

$$W_{str} = 35 \frac{kip}{ft}$$

Moment Arm: $l_{str} := \frac{B_{eff}}{2}$

Total Uplift Pressure: $Area_1 := B_{eff} \cdot (\gamma_w) \cdot (EL_{wps} - EL_{c\_bot})$

$$l_{area1} := \frac{B_{eff}}{2}$$

$$Area_2 := 0.5 B_{eff} \cdot \gamma_w \cdot (EL_{wc} - EL_{c\_bot})$$

$$l_{area2} := \frac{2}{3} B_{eff}$$

Weight of the Slab: $W_{slab} := B_{eff} \cdot \gamma_{concrete} \cdot (EL_{invert} - EL_{c\_bot})$

$W_{slab} = 40.6 \frac{kip}{ft}$

Moment Arm: $l_{slab} := \frac{B_{eff}}{2}$

Calculate Total Moments and Forces:

Moments are calculated about the point of rotation (the toe of the structure).

Resisting Moment: $M_{resist} := P_p \cdot l_p + F_{w2} \cdot l_{w2} + W_{str} \cdot l_{str} + W_{slab} \cdot l_{slab}$

Overturning Moment $M_{over} := F_{w1} \cdot l_{w1} + P_a \cdot l_a + Area_1 \cdot l_{area1} + Area_2 \cdot l_{area2}$

Net Moment: $M_{net} := (M_{resist}) - M_{over}$

$M_{resist} = 2688.1\,kip$ $M_{over} = 5609.8\,kip$ $M_{net} = -2922\,kip$

Since Mnet is negative, uplift resistance is necessary either through pile resistance or some other method.

Re- Calculate Total Moments and Forces with Required Tension:

Tension Required: $T_{rqd} := 70 \frac{kip}{ft}$

Moment Arm of Tension: $l_{rqd} := \frac{2}{3} \cdot B_{eff}$

Resisting Moment: $M_{resist} := P_p \cdot l_p + F_{w2} \cdot l_{w2} + W_{str} \cdot l_{str} + W_{slab} \cdot l_{slab} + T_{rqd} \cdot l_{rqd}$

Overturning Moment $M_{over} := F_{w1} \cdot l_{w1} + P_a \cdot l_a + Area_1 \cdot l_{area1} + Area_2 \cdot l_{area2}$

Net Moment: $M_{net} := (M_{resist}) - M_{over}$

$M_{resist} = 5954.7\,kip$ $M_{over} = 5609.8\,kip$ $M_{net} = 345\,kip$

Total Gross Weight/Vertical Forces: $F_{vert} := W_{str} - Area_1 - Area_2 + W_{slab} + T_{rqd}$

$F_{vert} = 13.7 \frac{kip}{ft}$

Resultant Location:

$$x_{r1} := \frac{M_{net}}{F_{vert}} \qquad x_{r1} = 25.2\,\text{ft}$$

$$\%_{comp1} := \min\left(3\cdot\frac{x_{r1}}{B_{eff}}, 1\right) \qquad \%_{comp1} = 100\,\%$$

Per criteria resultant must be within middle third which is equivalent to 100% of base in compression, overturning is OK

$$\%_{comp\_norm} := 100\%$$

$$Check_{overturn\_norm} := \text{if}\left(\%_{comp1} \geq \%_{comp\_norm}, \text{"OK"}, \text{"No Good"}\right)$$

$$Check_{overturn\_norm} = \text{"OK"} \qquad Plane = \text{"Bottom of Cell"}$$

Forces:

Total Driving Force: $F_{drive} := F_{w1} + P_a$ $\qquad F_{drive} = 26.5\,\frac{kip}{ft}$

Total Horizontal Force: $F_{resist} := P_p + F_{w2}$ $\qquad F_{resist} = 9\,\frac{kip}{ft}$

External Stability Check - Load Case 1

Sliding Factor of Safety Criteria: $FOS_{slide\_norm} := 1.5$

Shear Strength required: $S_{required} := 23\,\frac{kip}{ft}$

$$V_{all} := F_{vert}\cdot\tan(\phi_{fail}) + S_{required}$$

$$FOS_{slide} := \frac{(V_{all} + F_{resist})}{F_{drive}} \qquad FOS_{slide} = 1.51$$

$$Check_{slide\_norm} := \text{if}\left(FOS_{slide} \geq FOS_{slide\_norm}, \text{"OK"}, \text{"No Good"}\right)$$

$$Check_{slide\_norm} = \text{"OK"} \qquad Plane = \text{"Bottom of Cell"}$$

Bearing Capacity Check:

Eccentricity $e_1 := \frac{B_{eff}}{2} - x_{r1}$ $e_1 = 9.8\,ft$

Maximum Foundation Pressure:

$$q_{bearing} := if\left[\%_{comp1} < 1.0, \left[4 \cdot \frac{F_{vert}}{3 \cdot \left[\left(B_{eff}\right) - 2 \cdot e_1\right]}\right], \frac{F_{vert}}{B_{eff}} \cdot \left(1 + 6 \cdot \frac{e_1}{B_{eff}}\right)\right] \qquad q_{bearing} = 0.36\,\frac{kip}{ft^2}$$

$$q_{bearing} := if\left(\%_{comp1} \le 0.0, 0 \cdot ksf, q_{bearing}\right)$$

$$q_{min} := if\left[\%_{comp1} < 1.0, 0 \cdot ksf, \frac{F_{vert}}{B_{eff}} \cdot \left(1 - 6 \cdot \frac{e_1}{B_{eff}}\right)\right] \qquad q_{min} = 0.03\,ksf$$

London Canal Pump Station (Option 1):

Structure Geometry:

Effective Width of the Structure (from Mechanical Calcs): $B_{eff} := 70ft$

Variables:

Elevation of Channel Invert: $EL_{invert} := -14ft$

Bottom of Pump Structure: $EL_{c\_bot} := -18ft$

Elevation of water in the channel: $EL_{wps} := 0ft$

Water Level in Lake Ponchatrain: $EL_{wc} := 12ft$

Unit Weight of Water: $\gamma_w := 62.4pcf$

Unit Weight of Concrete: $\gamma_{concrete} := 145pcf$

Vertical Geometry:

Check the stability of the structure at the potential failure plane along the bottom of the structure.

Failure Plane: $\phi_{soil} := 30deg$

$EL_f := EL_{c\_bot}$ $\phi_{fail} := \phi_{soil}$ Plane := "Bottom of Cell"

Case 1

Determine the thickness of the base slab to resist uplift using only the dead weight of the slab. Solve by iterating the bottom of the slab elevation until FS=1.1

w_{str1} - is the lower bound value of the weight of the structure used for uplift: $w_{str1} := 250psf$

weight of the water in the canal: $w_{canal} := (EL_{wps} - EL_{invert}) \cdot \gamma_w$

weight of the structure below grade: $w_{slab} := (EL_{invert} - EL_{c\_bot}) \cdot \gamma_{concrete}$

uplift pressure from the lake: $Lake := \gamma_w \cdot ((EL_{we} - EL_{c\_bot}))$

uplift pressure from the canal: $Canal := \gamma_w \cdot (EL_{wps} - EL_{c\_bot})$

Average Uplift: $Uplift_{avg} := \dfrac{Lake + Canal}{2}$

Gravity Load: $w_{gravity} := w_{str1} + w_{canal} + w_{slab}$

$$FS := \frac{w_{gravity}}{Uplift_{avg}}$$

$$FS = 1.14$$

Lateral Loads:

Force of water:

$F_{w1} := \dfrac{1}{2}\gamma_w \cdot (EL_{we} - EL_f)^2$ $\quad F_{w1} = 28.08 \dfrac{kip}{ft}$ Force of the water from Lake Ponchatrain

$F_{w2} := \dfrac{1}{2}\gamma_w \cdot (EL_{wps} - EL_f)^2$ $\quad F_{w2} = 10.11 \dfrac{kip}{ft}$ Force of the water from the canal

Moment Arm: $l_{w1} := \left[\dfrac{1}{3} \cdot (EL_{we} - EL_f)\right]$

$$l_{w2} := \left[\frac{1}{3} \cdot (EL_{wps} - EL_f)\right]$$

Depth of slab of the pump structure:

$$H := EL_{invert} - EL_f$$

Active Pressure:

$$K_a := \frac{(1 - \sin(\phi_{soil}))}{(1 + \sin(\phi_{soil}))} \qquad P_a := 0.5 \cdot \gamma_{soil\_b} \cdot K_a \cdot H^2 \qquad P_a = 0.21 \frac{kip}{ft}$$

Moment Arm: $l_a := \left[\frac{1}{3} \cdot (EL_{invert} - EL_{c\_bot})\right]$

Passive Pressure:

In EM 1110-2-2502, the passive pressure phi angle is reduced and it is appropriate to perform the same operation here.

$FS := 1.5$

$$\phi_{soil\_p} := \text{atan}\left(\frac{\tan(\phi_{soil})}{FS}\right) \qquad \phi_{soil\_p} = 21.05\,deg$$

$$K_p := \frac{(1 + \sin(\phi_{soil\_p}))}{(1 - \sin(\phi_{soil\_p}))} \qquad P_p := 0.5 \cdot \gamma_{soil\_b} \cdot K_p \cdot H^2 \qquad P_p = 1.36 \frac{kip}{ft}$$

Moment arm: $l_p := \left[\frac{1}{3} \cdot (EL_{invert} - EL_{c\_bot})\right]$

Vertical Loads:

Load from the structure(upper bound value for designing foundations) : $w_{str2} := 500psf$

Weight of the Structure: $W_{str} := B_{eff} \cdot w_{str2}$

$$W_{str} = 35 \frac{kip}{ft}$$

Moment Arm: $l_{str} := \frac{B_{eff}}{2}$

Total Uplift Pressure: $Area_1 := B_{eff} \cdot (\gamma_w) \cdot (EL_{wps} - EL_{c\_bot})$

$$l_{area1} := \frac{B_{eff}}{2}$$

$$Area_2 := 0.5B_{eff} \cdot \gamma_w \cdot (EL_{we} - EL_{c\_bot})$$

$$l_{area2} := \frac{2}{3} B_{eff}$$

Weight of the Slab: $W_{slab} := B_{eff} \cdot \gamma_{concrete} \cdot (EL_{invert} - EL_{c\_bot})$

$$W_{slab} = 40.6 \frac{kip}{ft}$$

Moment Arm: $l_{slab} := \frac{B_{eff}}{2}$

Calculate Total Moments and Forces:

Moments are calculated about the point of rotation (the toe of the structure).

Resisting Moment: $M_{resist} := P_p \cdot l_p + F_{w2} \cdot l_{w2} + W_{str} \cdot l_{str} + W_{slab} \cdot l_{slab}$

Overturning Moment $M_{over} := F_{w1} \cdot l_{w1} + P_a \cdot l_a + Area_1 \cdot l_{area1} + Area_2 \cdot l_{area2}$

Net Moment: $M_{net} := (M_{resist}) - M_{over}$

$M_{resist} = 2708.5\,kip$ $\quad$ $M_{over} = 6090.5\,kip$ $\quad$ $M_{net} = -3382\,kip$

Since Mnet is negative, uplift resistance is necessary either through pile resistance or some other method.

Re- Calculate Total Moments and Forces with Required Tension:

Tension Required: $T_{rqd} := 77 \frac{kip}{ft}$

Moment Arm of Tension: $l_{rqd} := \frac{2}{3} \cdot B_{eff}$

Resisting Moment: $M_{resist} := P_p \cdot l_p + F_{w2} \cdot l_{w2} + W_{str} \cdot l_{str} + W_{slab} \cdot l_{slab} + T_{rqd} \cdot l_{rqd}$

Overturning Moment $M_{over} := F_{w1} \cdot l_{w1} + P_a \cdot l_a + Area_1 \cdot l_{area1} + Area_2 \cdot l_{area2}$

Net Moment: $M_{net} := (M_{resist}) - M_{over}$

$M_{resist} = 6301.8\,kip$ $\quad$ $M_{over} = 6090.5\,kip$ $\quad$ $M_{net} = 211\,kip$

Total Gross Weight/Vertical Forces: $F_{vert} := W_{str} - Area_1 - Area_2 + W_{slab} + T_{rqd}$

$$F_{vert} = 8.5 \frac{kip}{ft}$$

Resultant Location:

$$x_{r1} := \frac{M_{net}}{F_{vert}} \qquad x_{r1} = 24.98\,\text{ft}$$

$$\%_{comp1} := \min\left(3\cdot\frac{x_{r1}}{B_{eff}}, 1\right) \qquad \%_{comp1} = 100\,\%$$

Per criteria resultant must be within middle third which is equivalent to 100% of base in compression, overturning is OK

$$\%_{comp\_norm} := 100\%$$

$$Check_{overturn\_norm} := if\left(\%_{comp1} \geq \%_{comp\_norm}, \text{"OK"}, \text{"No Good"}\right)$$

$$Check_{overturn\_norm} = \text{"OK"} \qquad Plane = \text{"Bottom of Cell"}$$

Forces:

Total Driving Force: $F_{drive} := F_{w1} + P_a$ $\quad F_{drive} = 28.3\,\frac{kip}{ft}$

Total Horizontal Force: $F_{resist} := P_p + F_{w2}$ $\quad F_{resist} = 11.5\,\frac{kip}{ft}$

External Stability Check - Load Case 1

Sliding Factor of Safety Criteria: $FOS_{slide\_norm} := 1.5$

Shear Strength required: $S_{required} := 26\,\frac{kip}{ft}$

$$V_{all} := F_{vert}\cdot\tan(\phi_{fail}) + S_{required}$$

$$FOS_{slide} := \frac{(V_{all} + F_{resist})}{F_{drive}} \qquad FOS_{slide} = 1.5$$

$$Check_{slide\_norm} := if\left(FOS_{slide} \geq FOS_{slide\_norm}, \text{"OK"}, \text{"No Good"}\right)$$

$$Check_{slide\_norm} = \text{"No Good"} \qquad Plane = \text{"Bottom of Cell"}$$

Bearing Capacity Check:

Eccentricity $e_1 := \frac{B_{eff}}{2} - x_{r1}$ $\quad e_1 = 10.02\,\text{ft}$

Maximum Foundation Pressure:

$$q_{bearing} := \text{if}\left[\%_{comp1} < 1.0, \left[4 \cdot \frac{F_{vert}}{3 \cdot \left[\left(B_{eff}\right) - 2 \cdot e_1\right]}\right], \frac{F_{vert}}{B_{eff}} \cdot \left(1 + 6 \cdot \frac{e_1}{B_{eff}}\right)\right] \qquad q_{bearing} = 0.22\,\frac{\text{kip}}{\text{ft}^2}$$

$$q_{bearing} := \text{if}\left(\%_{comp1} \le 0.0, 0 \cdot \text{ksf}, q_{bearing}\right)$$

$$q_{min} := \text{if}\left[\%_{comp1} < 1.0, 0 \cdot \text{ksf}, \frac{F_{vert}}{B_{eff}} \cdot \left(1 - 6 \cdot \frac{e_1}{B_{eff}}\right)\right] \qquad q_{min} = 0.02\,\text{ksf}$$

Orleans Canal Pump Station (Option 1):

Structure Geometry:

Effective Width of the Structure (from Mechanical Calcs): $B_{eff} := 65ft$

Variables:

Elevation of Channel Invert: $EL_{invert} := -13ft$

Bottom of Pump Structure: $EL_{c\_bot} := -17ft$

Elevation of water in the channel: $EL_{wps} := -1.0ft$

Water Level in Lake Ponchatrain: $EL_{we} := 12ft$

Unit Weight of Water: $\gamma_w := 62.4pcf$

Unit Weight of Concrete: $\gamma_{concrete} := 145pcf$

Vertical Geometry:

Check the stability of the structure at the potential failure plane along the bottom of the structure.

Failure Plane: $\phi_{soil} := 30deg$

$EL_f := EL_{c\_bot}$ $\phi_{fail} := \phi_{soil}$ Plane := "Bottom of Cell"

Case 1

Determine the thickness of the base slab to resist uplift using only the dead weight of the slab. Solve by iterating the bottom of the slab elevation until FS=1.1

w_{str1} - is the lower bound value of the weight of the structure used for uplift: $w_{str1} := 250psf$

weight of the water in the canal: $w_{canal} := (EL_{wps} - EL_{invert})\cdot\gamma_w$

weight of the structure below grade: $w_{slab} := (EL_{invert} - EL_{c\_bot})\cdot\gamma_{concrete}$

uplift pressure from the lake: $Lake := \gamma_w\cdot((EL_{we} - EL_{c\_bot}))$

uplift pressure from the canal: $Canal := \gamma_w\cdot(EL_{wps} - EL_{c\_bot})$

Average Uplift: $Uplift_{avg} := \dfrac{Lake + Canal}{2}$

Gravity Load: $w_{gravity} := w_{str1} + w_{canal} + w_{slab}$

$$FS := \frac{w_{gravity}}{Uplift_{avg}}$$

$FS = 1.12$

Lateral Loads:

Force of water:

$F_{w1} := \frac{1}{2}\gamma_w\cdot(EL_{we} - EL_f)^2$ $\quad F_{w1} = 26.24\frac{kip}{ft}$ Force of the water from Lake Ponchatrain

$F_{w2} := \frac{1}{2}\gamma_w\cdot(EL_{wps} - EL_f)^2$ $\quad F_{w2} = 7.99\frac{kip}{ft}$ Force of the water from the canal

Moment Arm: $l_{w1} := \left[\frac{1}{3}\cdot(EL_{we} - EL_f)\right]$

$l_{w2} := \left[\frac{1}{3}\cdot(EL_{wps} - EL_f)\right]$

Depth of slab of the pump structure:

$H := EL_{invert} - EL_f$

Active Pressure:

$$K_a := \frac{(1 - \sin(\phi_{soil}))}{(1 + \sin(\phi_{soil}))} \qquad P_a := 0.5 \cdot \gamma_{soil\_b} \cdot K_a \cdot H^2 \qquad P_a = 0.21 \frac{kip}{ft}$$

Moment Arm: $l_a := \left[\frac{1}{3} \cdot (EL_{invert} - EL_{c\_bot})\right]$

Passive Pressure:

In EM 1110-2-2502, the passive pressure phi angle is reduced and it is appropriate to perform the same operation here.

$$FS := 1.5$$

$$\phi_{soil\_p} := \operatorname{atan}\left(\frac{\tan(\phi_{soil})}{FS}\right) \qquad \phi_{soil\_p} = 21.05\,deg$$

$$K_p := \frac{(1 + \sin(\phi_{soil\_p}))}{(1 - \sin(\phi_{soil\_p}))} \qquad P_p := 0.5 \cdot \gamma_{soil\_b} \cdot K_p \cdot H^2 \qquad P_p = 1.36 \frac{kip}{ft}$$

Moment arm: $l_p := \left[\frac{1}{3} \cdot (EL_{invert} - EL_{c\_bot})\right]$

Vertical Loads:

Load from the structure(upper bound value for designing foundations) : $w_{str2} := 500psf$

Weight of the Structure: $W_{str} := B_{eff} \cdot w_{str2}$

$$W_{str} = 32.5 \frac{kip}{ft}$$

Moment Arm: $l_{str} := \frac{B_{eff}}{2}$

Total Uplift Pressure: $Area_1 := B_{eff} \cdot (\gamma_w) \cdot (EL_{wps} - EL_{c\_bot})$

$$l_{area1} := \frac{B_{eff}}{2}$$

$$Area_2 := 0.5 B_{eff} \cdot \gamma_w \cdot (EL_{wc} - EL_{c\_bot})$$

$$l_{area2} := \frac{2}{3} B_{eff}$$

Weight of the Slab: $W_{slab} := B_{eff} \cdot \gamma_{concrete} \cdot \left(EL_{invert} - EL_{c\_bot}\right)$

$W_{slab} = 37.7 \frac{kip}{ft}$

Moment Arm: $l_{slab} := \frac{B_{eff}}{2}$

Calculate Total Moments and Forces:

Moments are calculated about the point of rotation (the toe of the structure).

Resisting Moment: $M_{resist} := P_p \cdot l_p + F_{w2} \cdot l_{w2} + W_{str} \cdot l_{str} + W_{slab} \cdot l_{slab}$

Overturning Moment $M_{over} := F_{w1} \cdot l_{w1} + P_a \cdot l_a + Area_1 \cdot l_{area1} + Area_2 \cdot l_{area2}$

Net Moment: $M_{net} := \left(M_{resist}\right) - M_{over}$

$M_{resist} = 2325.9\,kip$ $\quad M_{over} = 4911.6\,kip$ $\quad M_{net} = -2586\,kip$

Since Mnet is negative, uplift resistance is necessary either through pile resistance or some other method.

Re- Calculate Total Moments and Forces with Required Tension:

Tension Required: $T_{rqd} := 70 \frac{kip}{ft}$

Moment Arm of Tension: $l_{rqd} := \frac{2}{3} \cdot B_{eff}$

Resisting Moment: $M_{resist} := P_p \cdot l_p + F_{w2} \cdot l_{w2} + W_{str} \cdot l_{str} + W_{slab} \cdot l_{slab} + T_{rqd} \cdot l_{rqd}$

Overturning Moment $M_{over} := F_{w1} \cdot l_{w1} + P_a \cdot l_a + Area_1 \cdot l_{area1} + Area_2 \cdot l_{area2}$

Net Moment: $M_{net} := \left(M_{resist}\right) - M_{over}$

$M_{resist} = 5359.2\,kip$ $\quad M_{over} = 4911.6\,kip$ $\quad M_{net} = 448\,kip$

Total Gross Weight/Vertical Forces: $F_{vert} := W_{str} - Area_1 - Area_2 + W_{slab} + T_{rqd}$

$F_{vert} = 16.5 \frac{kip}{ft}$

Resultant Location:

$x_{r1} := \frac{M_{net}}{F_{vert}}$ $\qquad x_{r1} = 27.14\,ft$

$\%_{comp1} := \min\left(3 \cdot \frac{x_{r1}}{B_{eff}}, 1\right)$ $\qquad \%_{comp1} = 100\,\%$

Per criteria resultant must be within middle third which is equivalent to 100% of base in compression, overturning is OK

$\%_{comp\_norm} := 100\%$

$Check_{overturn\_norm} := if(\%_{comp1} \geq \%_{comp\_norm}, "OK", "No\ Good")$

$Check_{overturn\_norm} = "OK"$ $\qquad Plane = "Bottom\ of\ Cell"$

Forces:

Total Driving Force: $F_{drive} := F_{w1} + P_a$ $\qquad F_{drive} = 26.5\,\frac{kip}{ft}$

Total Horizontal Force: $F_{resist} := P_p + F_{w2}$ $\qquad F_{resist} = 9.3\,\frac{kip}{ft}$

External Stability Check - Load Case 1

Sliding Factor of Safety Criteria: $FOS_{slide\_norm} := 1.5$

Shear Strength required: $S_{required} := 21\,\frac{kip}{ft}$

$V_{all} := F_{vert} \cdot \tan(\phi_{fail}) + S_{required}$

$FOS_{slide} := \frac{(V_{all} + F_{resist})}{F_{drive}}$ $\qquad FOS_{slide} = 1.51$

$Check_{slide\_norm} := if(FOS_{slide} \geq FOS_{slide\_norm}, "OK", "No\ Good")$

$Check_{slide\_norm} = "OK"$ $\qquad Plane = "Bottom\ of\ Cell"$

Bearing Capacity Check:

Eccentricity $e_1 := \frac{B_{eff}}{2} - x_{r1}$ $\qquad e_1 = 5.36\,\text{ft}$

Maximum Foundation Pressure:

$$q_{bearing} := \text{if}\left[\%_{comp1} < 1.0, \left[4\cdot\frac{F_{vert}}{3\cdot\left[\left(B_{eff}\right) - 2\cdot e_1\right]}\right], \frac{F_{vert}}{B_{eff}}\cdot\left(1 + 6\cdot\frac{e_1}{B_{eff}}\right)\right] \qquad q_{bearing} = 0.38\,\frac{\text{kip}}{\text{ft}^2}$$

$$q_{bearing} := \text{if}\left(\%_{comp1} \le 0.0, 0\cdot\text{ksf}, q_{bearing}\right)$$

$$q_{min} := \text{if}\left[\%_{comp1} < 1.0, 0\cdot\text{ksf}, \frac{F_{vert}}{B_{eff}}\cdot\left(1 - 6\cdot\frac{e_1}{B_{eff}}\right)\right] \qquad q_{min} = 0.13\,\text{ksf}$$

The following figures and text excerpts were taken from the IPET Report. These documents were used as a basis for the geological profiles and strength parameters found in the report.

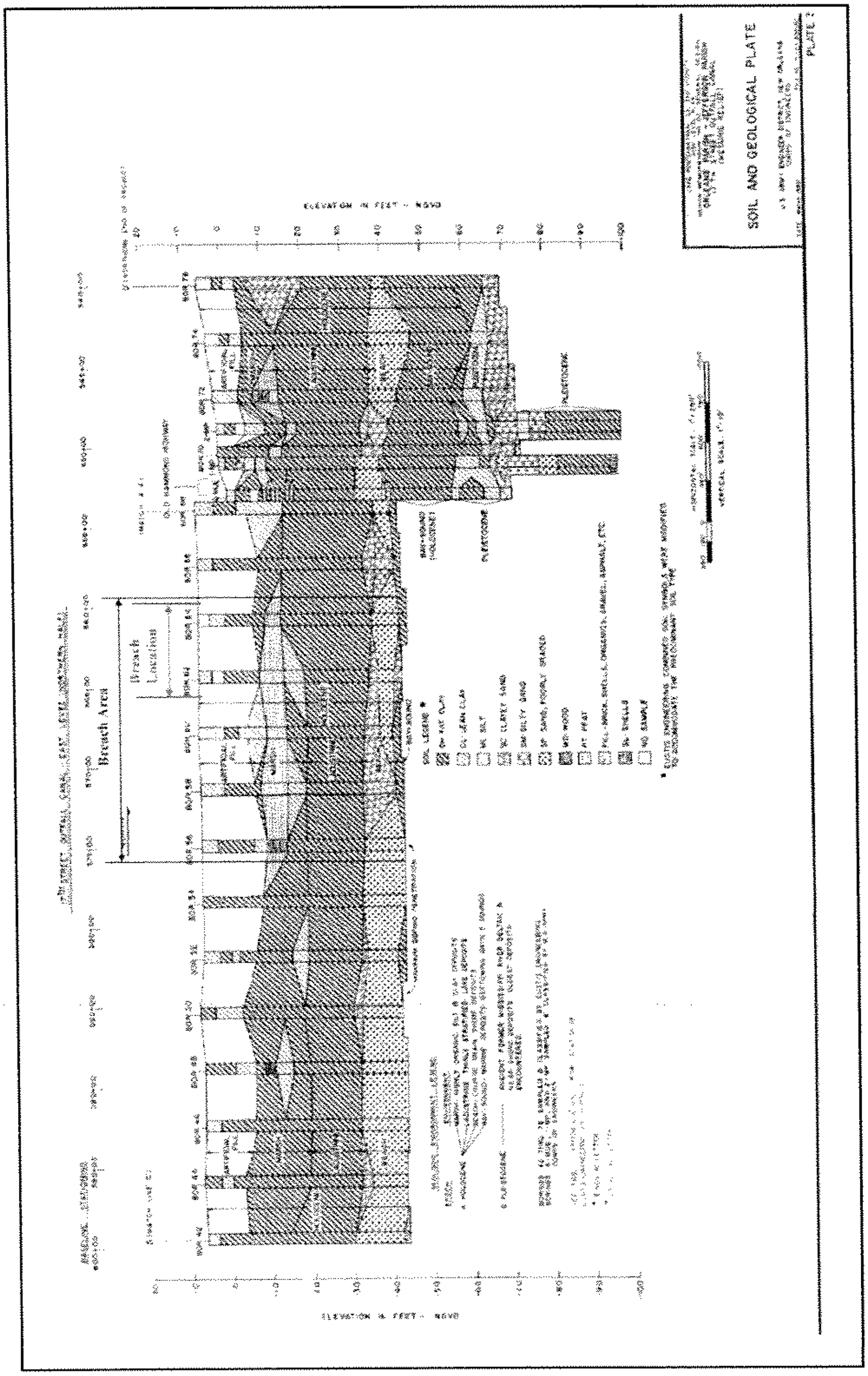


Figure 1-7. Geological Profile showing Breach Area (East Levee)

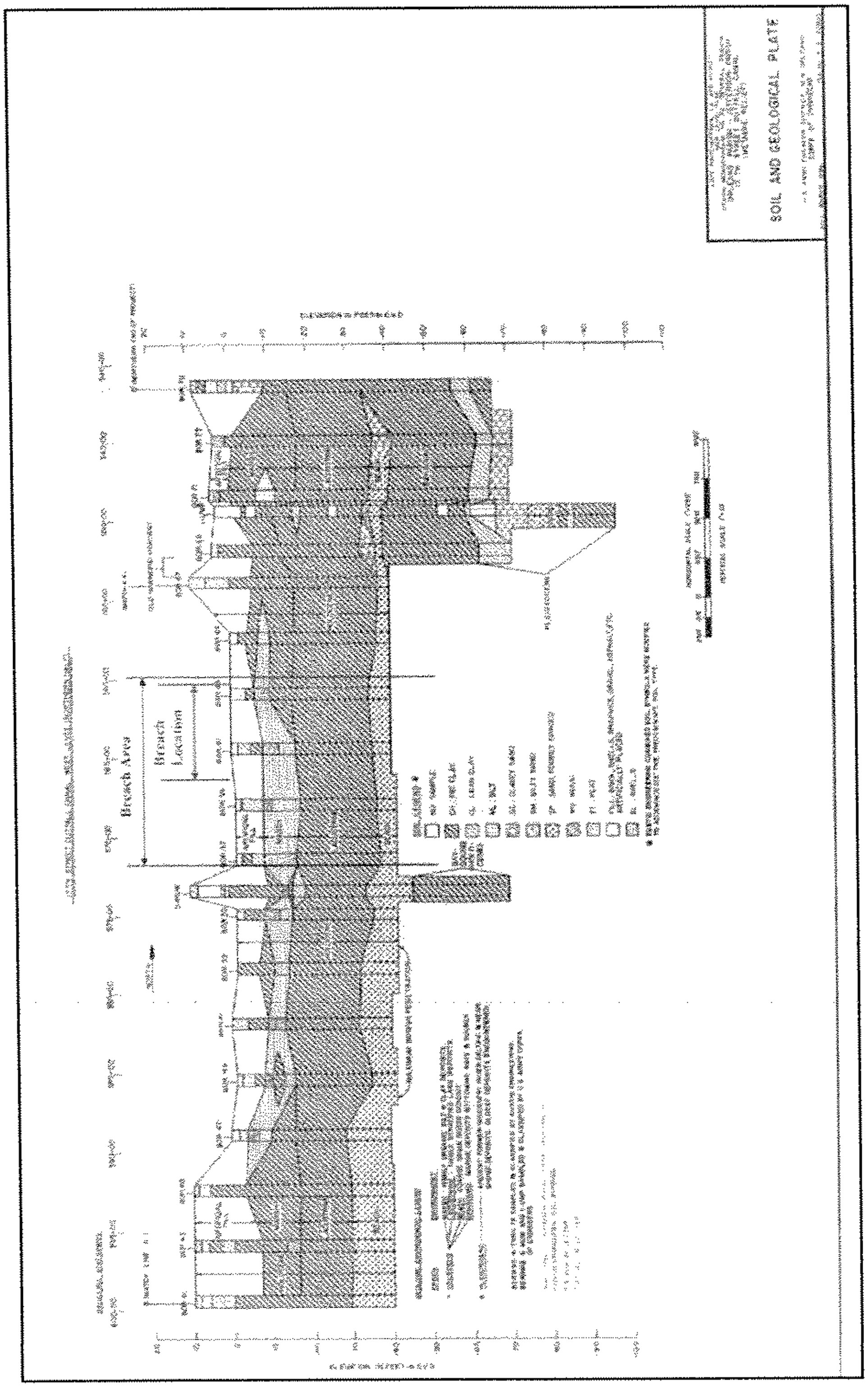


Figure 1-8. Geological Profile showing Breach Area (West Levee)

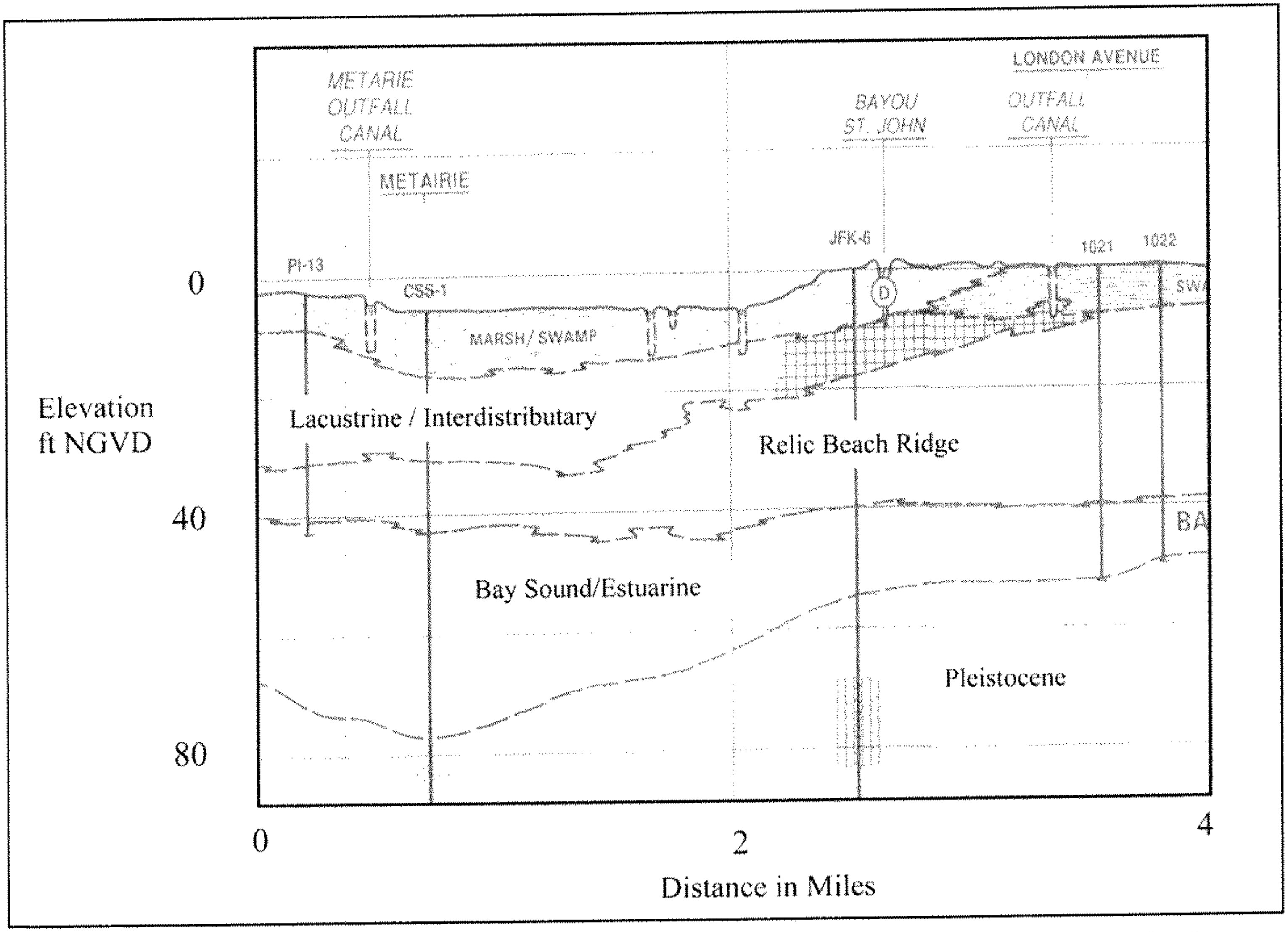


Figure 7-6. Portion of Cross-section C-C" from the Spanish Fort Quadrangle, which Extends Through the 17th and London Canal Breaches and Identifies the Sratigraphic Environments in the Subsurface (from Dunbar and Others, 1995).

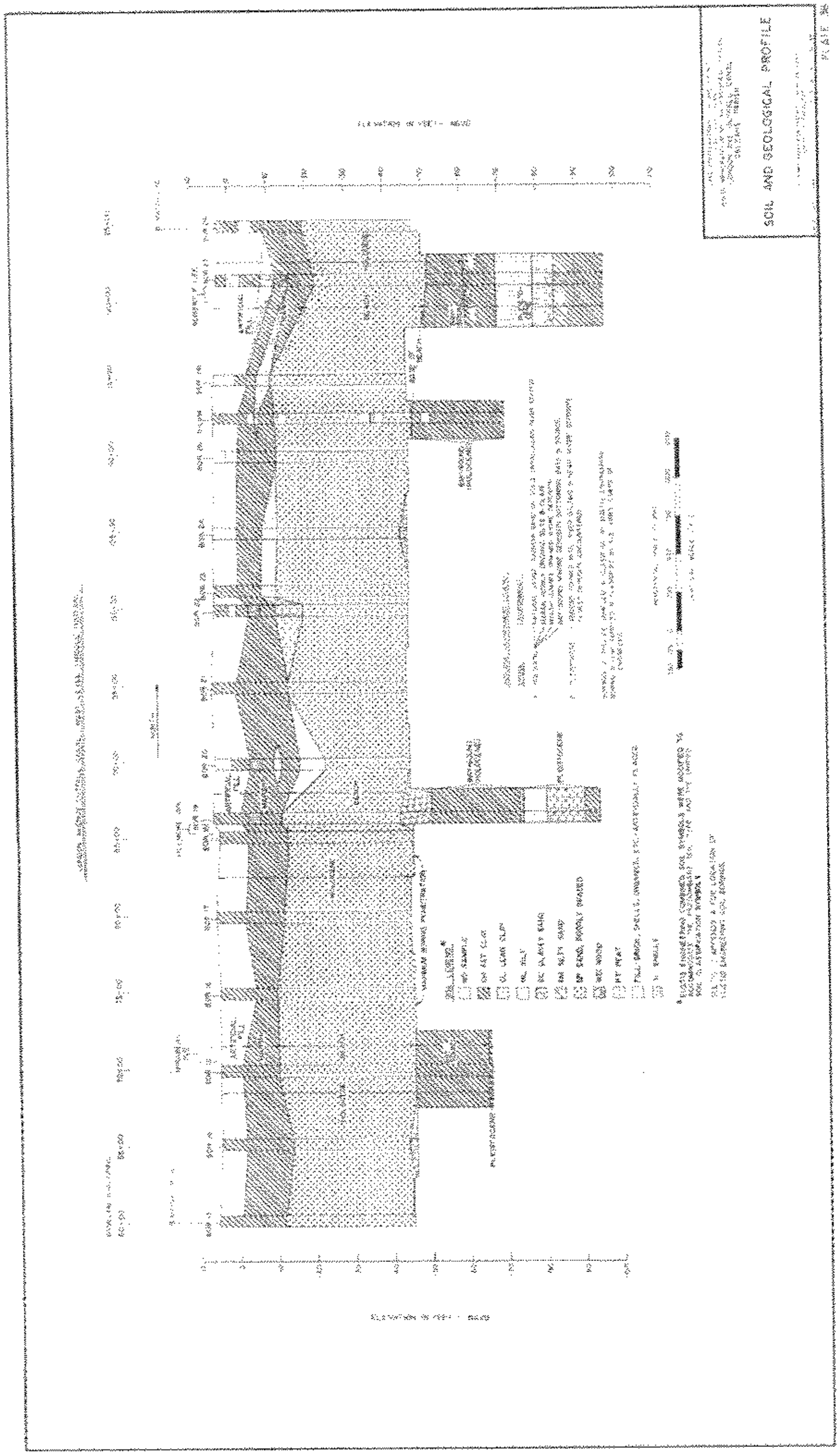


Figure 7-7. Geological Profile, along West Levee at London Canal. North London breach is South of Robert E. Lee Bridge as Shown (USACE, 1989, p. 134)

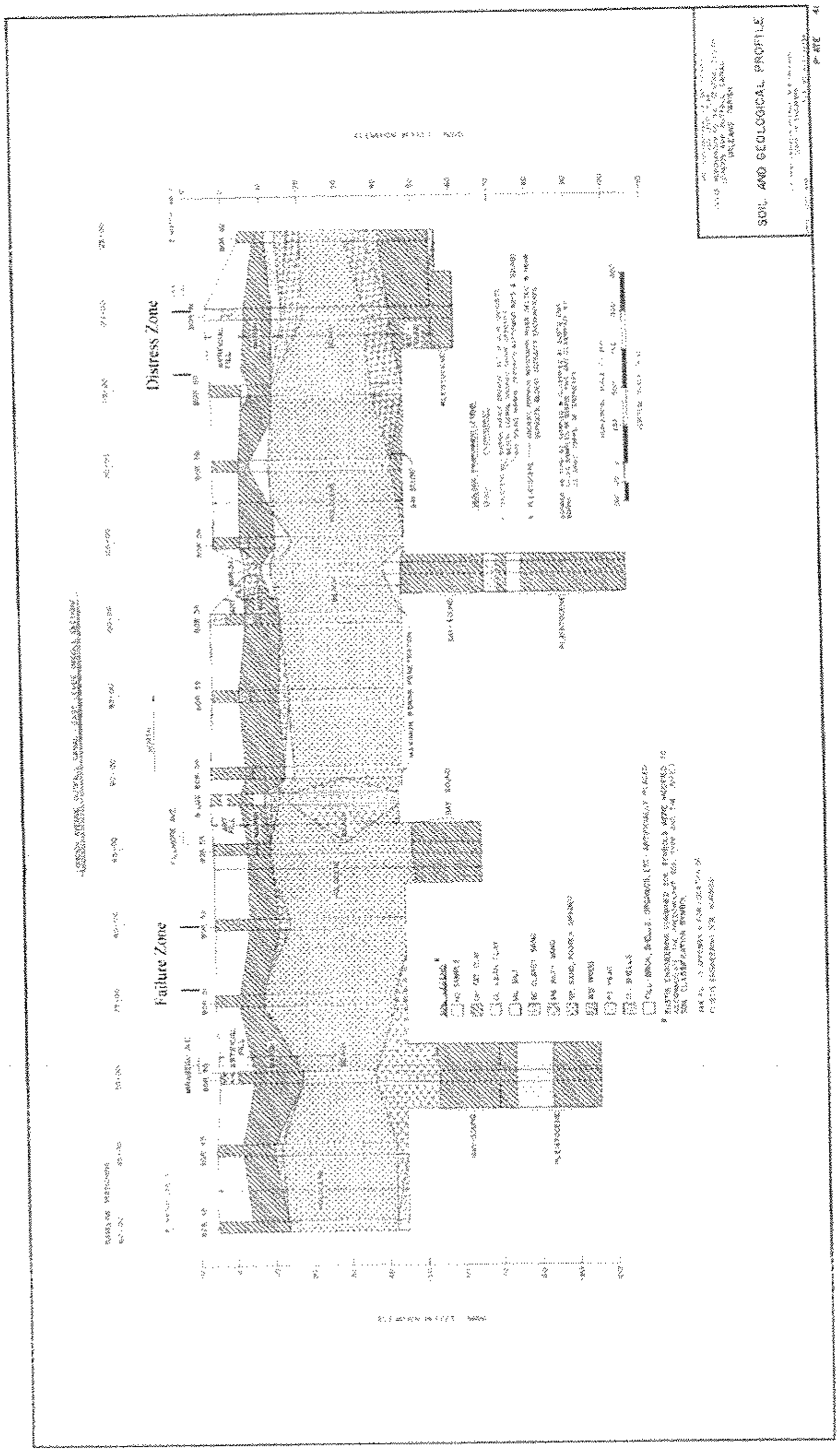


Figure 7-8. Geological Profile Along East Levee at London Canal. North London Area of Levee Distress is South of Robert E. Lee Bridge as Shown, and South London Breach Area is North of Mirabeau Bridge (USACE, 1989, p. 137).

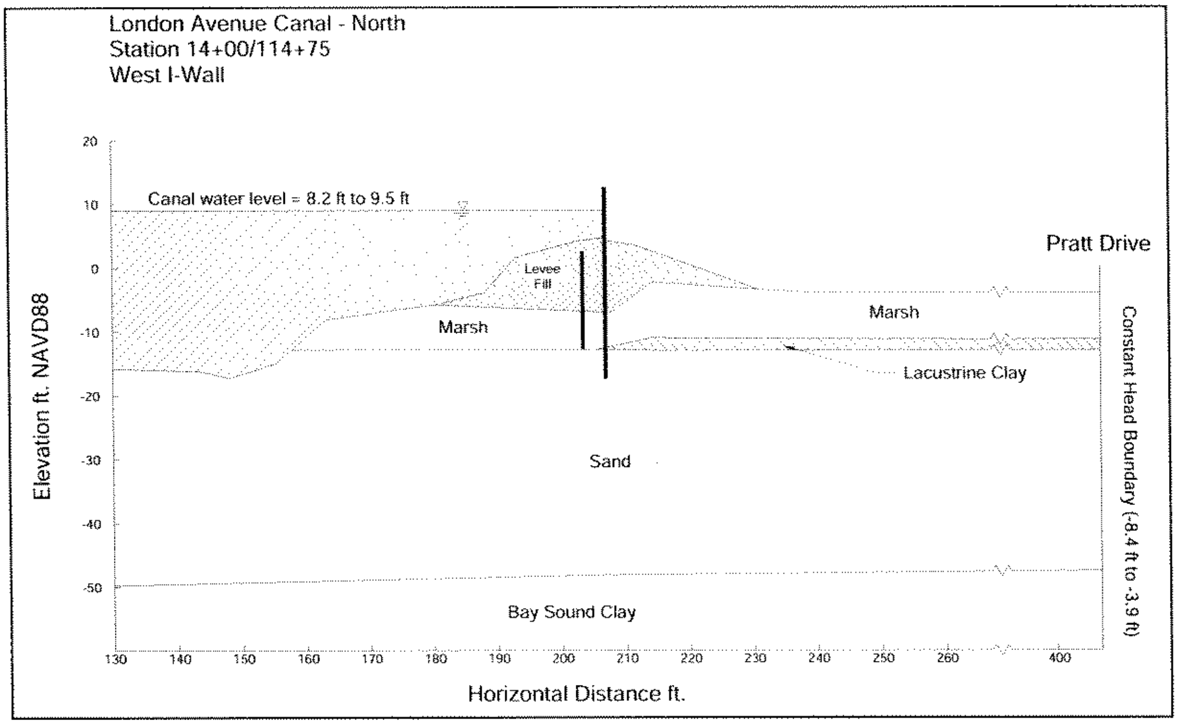


Figure 8-16. Schematic Cross Section at London North Breach, with Seepage Boundary Conditions

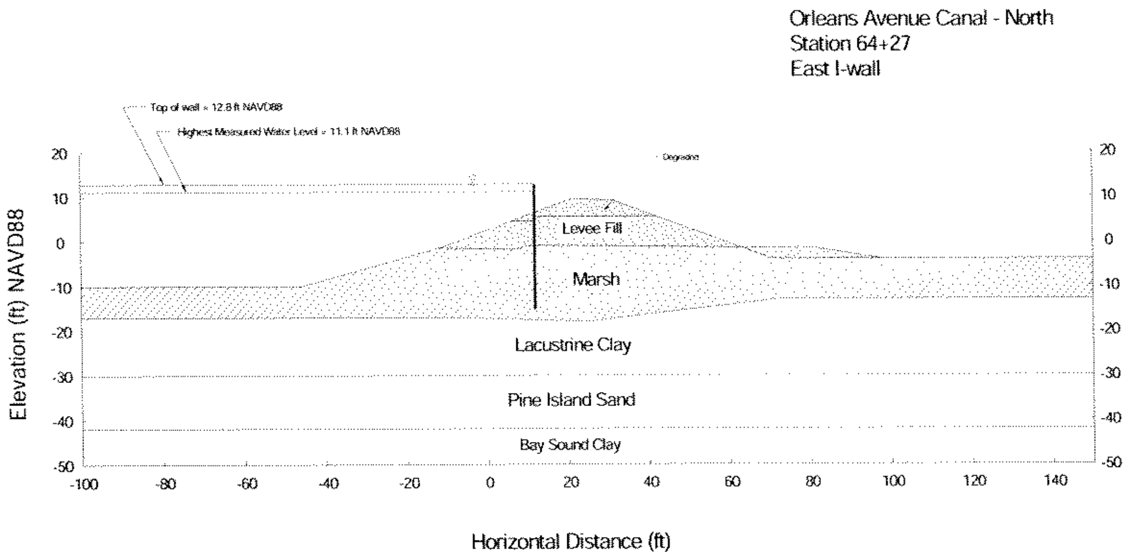


Figure 10-18. Schematic Cross Section of Orleans North

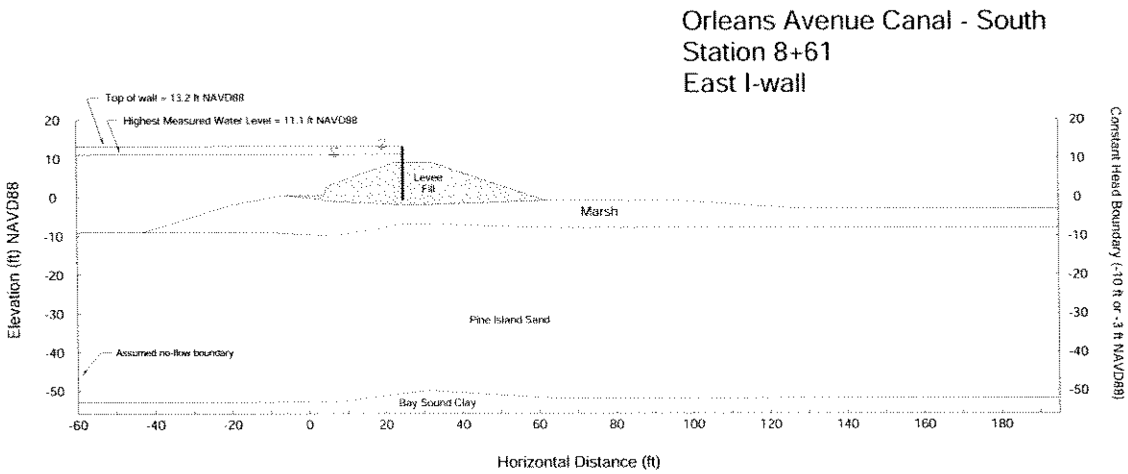


Figure 10-3. Schematic Cross Section at Orleans South, with Seepage Boundary Conditions

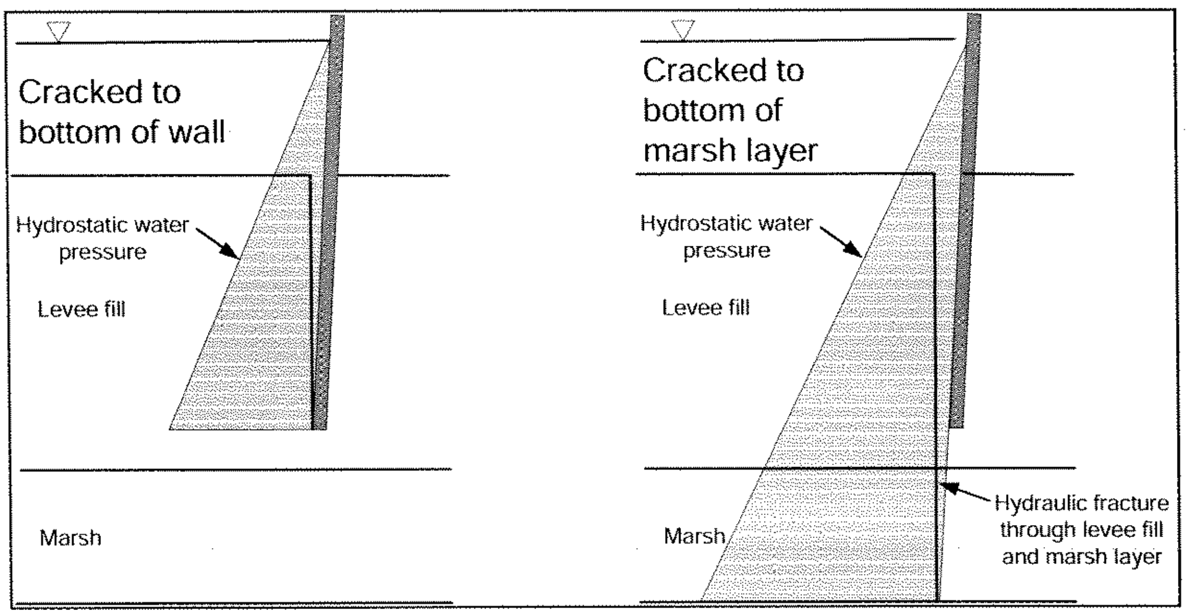


Figure 10-4. Schematic of Cracks Used in Orleans South Seepage and Stability Analyses

Strength differences of this magnitude are significant. They indicate that the reason the failure occurred where it did is very likely that the clay strengths in the breach area were lower than in adjacent areas to the north and south.

At the time of completion of this report the results of some vane shear tests and cone penetration tests are not yet available. Those results will be reflected in a revision of this report if that is found necessary.

Table 3-1
Comparison of Strengths of Levee Fill, Marsh Layer and Sand Used in Design for Stations 552+70 to 635+00 with the IPET Strengths

Material	Strengths used for design	IPET strength model
Levee fill	s_u = 500 psf, ϕ = 0	s_u = 900 psf, ϕ = 0
Marsh layer	s_u = 280 psf, ϕ = 0	s_u = 400 psf, ϕ = 0 beneath levee crest s_u = 300 psf, ϕ = 0 beneath levee toe
Sand	ϕ' = 30 degrees	ϕ' = 35 degrees

Table 3-2
Undrained Strengths of Clay for Specimens from the Breach

Depth	Test type	s_u	Average
Boring 62			
Depth	**Test type**	**s_u**	**Average**
24 ft	UC	305 psf	280 psf
34 ft	UC	260 psf	
42 ft	UU-1	178 psf (very loose clayey sand – ignore)	
Boring 64			
Depth	Test type	s_u	Average
22 ft	UC	103 psf	240 psf
33.5 ft	UC	383 psf	
41.5 ft	UC	168 psf (likely disturbed – ignore)	

Table 3-3
Undrained Strengths of Clay for Specimens from Borings North of the Breach

Depth	Test type	s_u	Average
Boring 66			
Depth	**Test type**	**s_u**	**Average**
28.5 ft	UC	235 psf	317 psf
38.5 ft	UC	398 psf	
Boring 68			
Depth	Test type	s_u	Average
33 ft	UC	340 psf	353 psf
33 ft	UU	360 psf	
39 ft	UU	360 psf	
42.5 ft	UU-1	250 psf (likely sand, not clay – ignore	
42.5 ft	UU	240 psf (likely sand, not clay - ignore	

Table 3-4
Undrained Strengths of Clay for Specimens from Borings South of the Breach

Boring 60			
Depth	**Test type**	**s_u**	**Average**
24 ft	UC	200 psf	326 psf
29 ft	UC	365 psf	
29 ft	UU	380 psf	
34 ft	UC	385 psf	
39 ft	UC	323 psf	
39 UU	UU	300 psf	
44 ft	UU-1	243 psf (loose clayey sand – ignore)	
Boring 58			
Depth	**Test type**	**s_u**	**Average**
24 ft	UC	183 psf	324 psf
29 ft	UC	313 psf	
39 ft	UC	475 psf	
Boring 56			
Depth	**Test type**	**s_u**	
29 ft	UC	295 psf	305 psf
39 ft	UC	315 psf	

Table 3-5
Comparison of Undrained Strengths from Breach Area Borings with Strengths from Borings North and South of the Breach

Area	Range of s_u	Average s_u
Breach (Borings 62 and 64)	240 psf to 280 psf	260 psf
North of breach (Borings 66 and 68)	317 psf to 353 psf	335 psf
South of breach (Borings 56, 58 and 60)	305 psf to 326 psf	318 psf

Slope Instability

Slope stability analyses were performed for Cases 1 through 8 shown in Table 10-1. The critical circles and factors of safety for these cases are shown in Figures 10-10 through 10-17. The cross section shown in theses figures is the same as that used in the seepage analyses. The analyses were performed using the computer program SLIDE[1], with Spencer's method.

Standard Penetration Tests performed in the Orleans south area showed that the sand had Standard Penetration Test blow counts (N_{SPT}) averaging about 40, which corresponds to a value of ϕ' in the range of 36 degrees to 40 degrees. Cone penetration tests in the area showed tip resistances that correspond to similar values of ϕ'. A value of $\phi' = 38$ degrees was used in the stability analyses.

The marsh layer was treated as undrained, with $s_u = 700$ psf and $\phi_u = 0$ beneath the levee crest, and $s_u = 450$ psf and $\phi_u = 0$ at the toe of the levee and beyond. These strength values, based on the available test results, are higher than the marsh strengths at the 17th Street Canal and the London Avenue Canal. The higher strengths at Orleans are likely due to the fact that the levee crest had been higher before the I-wall was constructed, and had compressed the marsh layer to a denser and stronger condition. The levee crest was degraded when the I-wall was built, and the marsh layer was overconsolidated as a result. A unit weight of 95 pcf was used for the marsh, based on available test results.

The levee fill was also treated as undrained, with $s_u = 1500$ psf, and $\phi_u = 0$. The slip circles do not intersect the levee fill, however, and the levee strength therefore has no influence on the calculated values of factor of safety. A value of unit weight = 109 pcf was used for the levee fill, based on available test results.

The analyses were performed using pore pressures in the sand from the finite element seepage analyses without a rupture or void through the marsh layer. The non-ruptured seepage analyses were used to determine pore pressures because a rupture or void would be of very limited size, not appropriate for inclusion in a two-dimensional cross section. At the bases of the slices where the pore pressures exceeded the overburden pressures near the top of the sand on the inland side, zero shear strength was assigned for the sand.

As discussed earlier, it was assumed in all analyses that deflection of the wall toward the land side would result in formation of a crack through the levee fill and the marsh in back of the wall, down to the bottom of the wall, or by hydraulic fracturing, down to the top of the sand. It was assumed that the crack would not extend below the top of the sand, because the sand is cohesionless, and would be expected to slump and fill any incipient crack. A crack to the bottom of the wall was assumed for Cases 1, 2, 5 and 6 (Figures 10-10, 10-11, 10-14 and 10-15), and a crack to the top of the sand was assumed for Cases 3, 4, 7, and 8 (Figures 10-12, 10-13, 10-16 and 10-17).

The critical circles extend to the bottom of the crack in all cases – to the bottom of the wall when the crack extends to that depth, and to the marsh/sand interface when the crack extends to

Appendix C

CIVIL/SITE

Approximate Equipment/Building Sizes
New Orleans District Conceptual Study

	Capacity	Width	Length	Notes
Option 1				
17th Street Canal				
Pump Building w/ Gates (cfs)	12500	155	450	width incl 40' outlet, 45' screen
Generator Building (MW)	22.5	80	202	houses gen related equipment
Tank Farm	7	48	56	12k gallon double wall
Orleans Canal				
Pump Building w/ Gates (cfs)	3400	150	170	width incl 40' outlet, 45' screen
Generator Building (MW)	7.5	80	94	houses gen related equipment
Tank Farm	2	24	28	12k gallon double wall
London Canal				
Pump Building w/ Gates (cfs)	9000	155	350	width incl 40' outlet, 45' screen
Generator Building (MW)	15	80	148	houses gen related equipment
Tank Farm	5	28	48	12k gallon double wall
Option 2				
17th Street Canal				
Pump Building w/o Gates (cfs)	12500	165	400	width incl 40' outlet, 45' screen
Generator Building (MW)	40	80	328	houses gen related equipment
Tank Farm	18	60	84	12k gallon double wall
Orleans Canal				
Pump Building w/o Gates (cfs)	3400	155	130	width incl 40' outlet, 45' screen
Generator Building (MW)	10	80	112	houses gen related equipment
Tank Farm	5	28	48	12k gallon double wall
London Canal				
Pump Building wo/ Gates (cfs)	9000	165	300	width incl 40' outlet, 45' screen
Generator Building (MW)	25	80	220	houses gen related equipment
Tank Farm	13	48	84	12k gallon double wall

Approximate Pump Building Lengths
New Orleans District Conceptual Study

Option 1	Quantity		Length (ft)		Extension (feet)	Study Qty
17th Street Canal						
1000 cfs engines	4	@	36	=	144	
1000 cfs motors	7	@	26	=	182	
500 cfs motors	2	@	16	=	32	
250 cfs motors	2	@	10	=	20	
Gate and Layout	1	@	60	=	60	
Total					438	450
Orleans Canal						
1000 cfs engines	2	@	36	=	72	
1000 cfs motors	0	@	26	=	0	
500 cfs motors	2	@	16	=	32	
250 cfs motors	2	@	10	=	20	
Gate and Layout	1	@	40	=	40	
Total					164	170
London Canal						
1000 cfs engines	4	@	36	=	144	
1000 cfs motors	4	@	26	=	104	
500 cfs motors	1	@	16	=	16	
250 cfs motors	2	@	10	=	20	
Gate and Layout	1	@	60	=	60	
Total					344	350
Option 2						
17th Street Canal						
1000 cfs engines	4	@	36	=	144	
1000 cfs motors	7	@	26	=	182	
500 cfs motors	2	@	16	=	32	
250 cfs motors	2	@	10	=	20	
Gate and Layout	1	@	0	=	0	
Total					378	400
Orleans Canal						
1000 cfs engines	2	@	36	=	72	
1000 cfs motors	0	@	26	=	0	
500 cfs motors	2	@	16	=	32	
250 cfs motors	2	@	10	=	20	
Gate and Layout	1	@	0	=	0	
Total					124	130
London Canal						
1000 cfs engines	4	@	36	=	144	
1000 cfs motors	4	@	26	=	104	
500 cfs motors	1	@	16	=	16	
250 cfs motors	2	@	10	=	20	
Gate and Layout	1	@	0	=	0	
Total					284	300

Approximate Tank Farm Sizes
New Orleans District Conceptual Study

	Hp		Usage*		Gals/Hr	Gals/Day	Gals/Event**	Add****	Tanks***
Option 1									
17th Street Canal									
	13906	/	20	=	695.3	16687	66749	80099	7
Orleans Canal									
	3955	/	20	=	197.75	4746	18984	22781	2
London Canal									
	10224	/	20	=	511.2	12269	49075	58890	5
Option 2									
17th Street Canal									
	35935	/	20	=	1796.8	43122	172488	206986	18
Orleans Canal									
	10218	/	20	=	510.9	12262	49046	58856	5
London Canal									
	26419	/	20	=	1321	31703	126811	152173	13

* Every 20 hp uses 1 gallon/hr
** An event requires 4 days fuel storage
*** 12,000 gal double wall tanks, Each tank is 8'x22', so allocate 12'x28' for each tank pad
**** Add 10% for motor effiecieny and 10% for bldg loads

Approximate Driver Sizes
New Orleans District Conceptual Study

Option 1

17th Street Canal		
Pump	Driver	Source*
cfs	hp	
1000	1241	Engine
1000	1241	Engine
1000	1241	Engine
1000	1241	Engine
1000	1052	Genset
1000	1052	Genset
1000	1052	Genset
1000	1052	Genset
1000	1052	Genset
1000	1052	Genset
1000	1052	Grd/Gen
500	526	Grd/Gen
500	526	Grd/Gen
250	263	Grd/Gen
250	263	Grd/Gen
12500	13906	

22.5 MW Gen

Orleans Canal		
Pump	Driver	Source*
cfs	hp	
1000	1241	Engine
1000	1241	Engine
500	526	Grd/Gen
500	526	Grd/Gen
250	263	Grd/Gen
150	158	Grd/Gen
3400	3955	

7.5 MW Gen

London Canal		
Pump	Driver	Source*
cfs	hp	
1000	1241	Engine
1000	1241	Engine
1000	1241	Engine
1000	1241	Engine
1000	1052	Genset
1000	1052	Genset
1000	1052	Genset
1000	1052	Grd/Gen
500	526	Grd/Gen
250	263	Grd/Gen
250	263	Grd/Gen
9000	10224	

15 MW Gen

Option 2

17th Street Canal		
Pump	Driver	Source*
cfs	hp	
1000	3206	Engine
1000	3206	Engine
1000	3206	Engine
1000	3206	Engine
1000	2719	Genset
1000	2719	Genset
1000	2719	Genset
1000	2719	Genset
1000	2719	Genset
1000	2719	Genset
1000	2719	Grd/Gen
500	1359	Grd/Gen
500	1359	Grd/Gen
250	680	Grd/Gen
250	680	Grd/Gen
12500	35935	

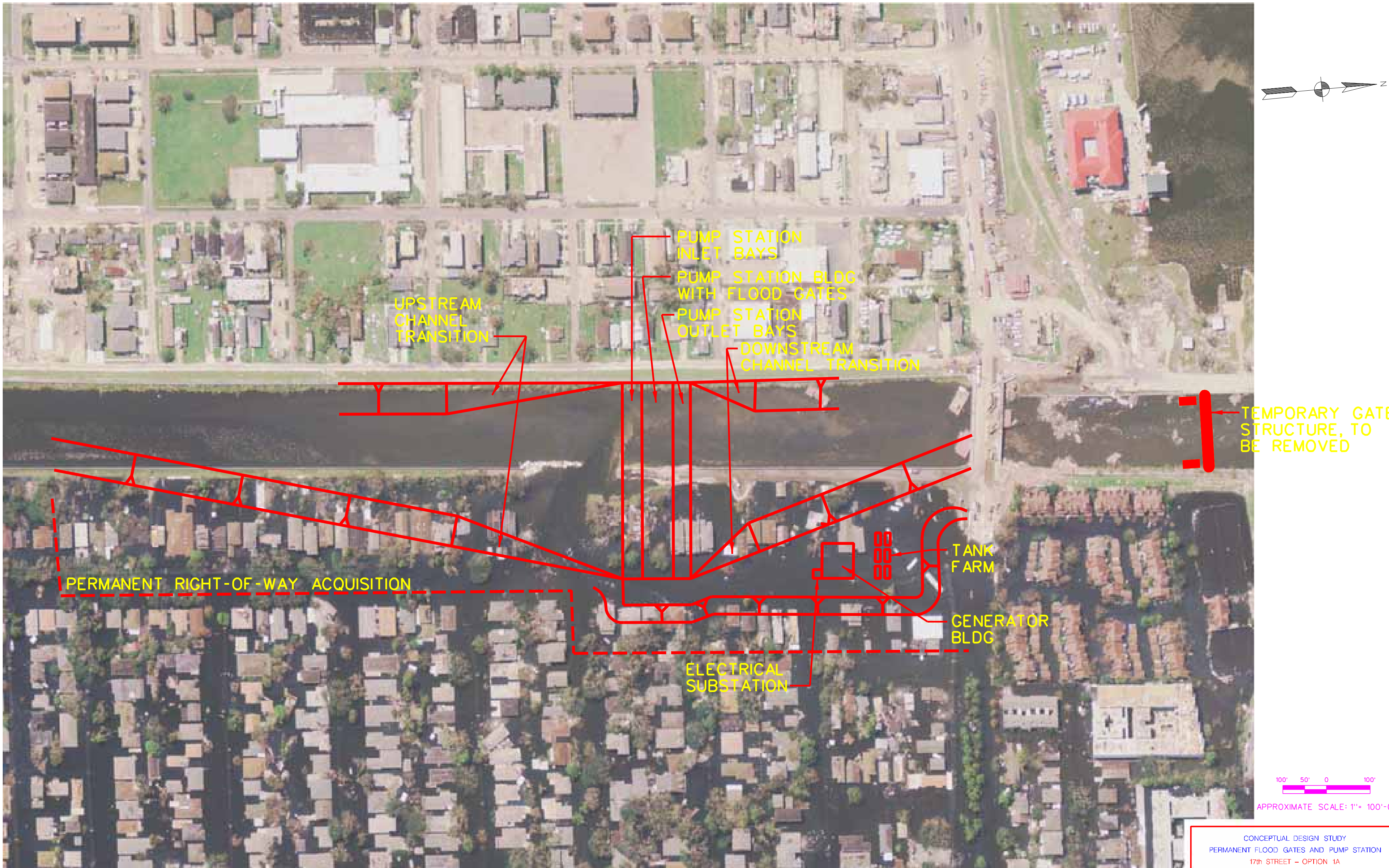
40 MW Gen

Orleans Canal		
Pump	Driver	Source*
cfs	hp	
1000	3206	Engine
1000	3206	Engine
500	1359	Grd/Gen
500	1359	Grd/Gen
250	680	Grd/Gen
150	408	Grd/Gen
3400	10218	

10 MW Gen

London Canal		
Pump	Driver	Source*
cfs	hp	
1000	3206	Engine
1000	3206	Engine
1000	3206	Engine
1000	3206	Engine
1000	2719	Genset
1000	2719	Genset
1000	2719	Genset
1000	2719	Grd/Gen
500	1359	Grd/Gen
250	680	Grd/Gen
250	680	Grd/Gen
9000	26419	

25 MW Gen



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17th STREET – OPTION 1A
EXHIBIT 5.1.1.A

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PERMANENT FLOOD GATES AND PUMP STATION
17th STREET – OPTION 1B
EXHIBIT 5.1.1.B

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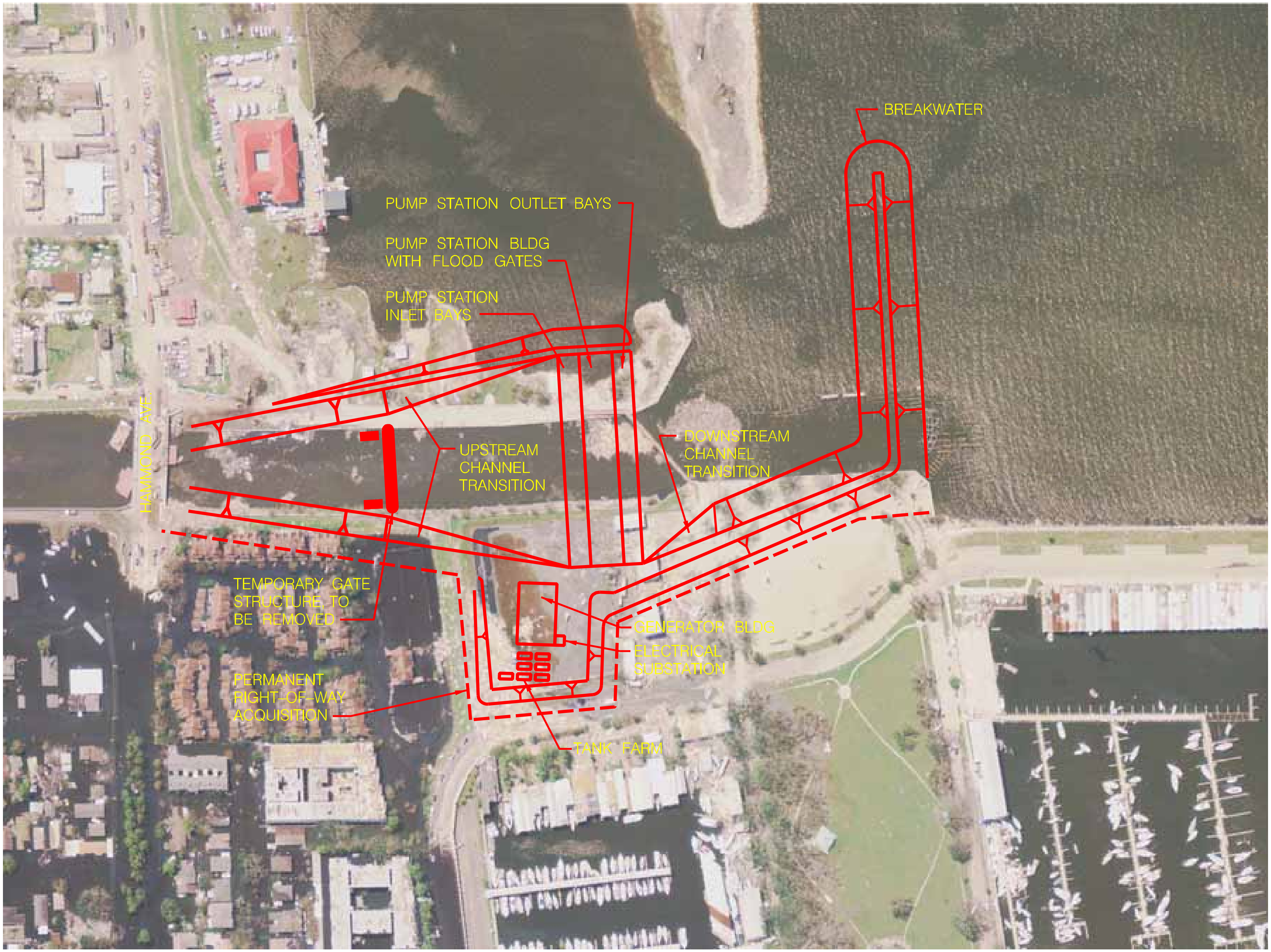

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17th STREET – OPTION 1C
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CONCEPTUAL DESIGN STUDY
PERMANENT FLOOD GATES AND PUMP STATION
17th STREET – OPTION 2A
EXHIBIT 5.2.1.A

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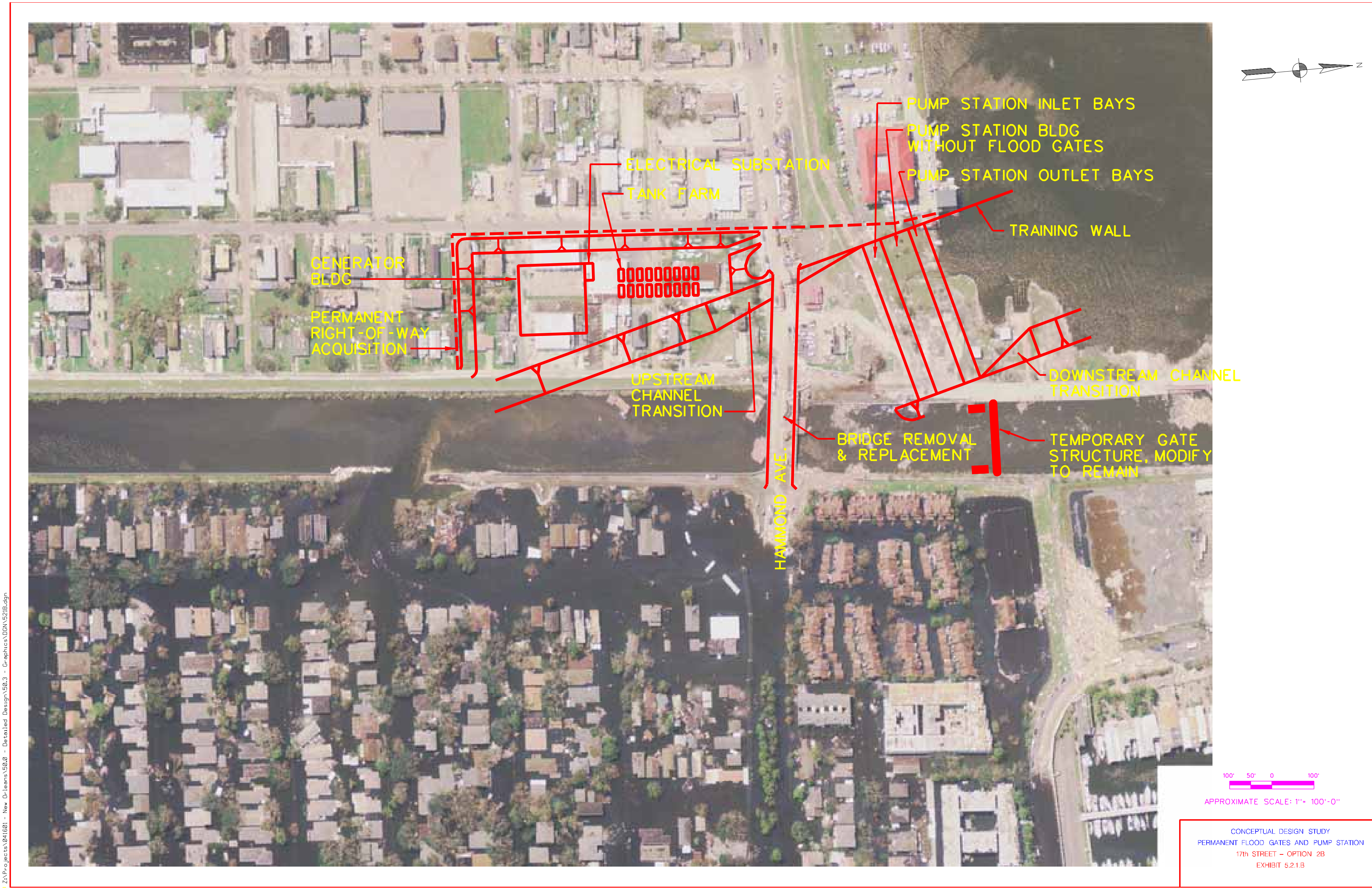

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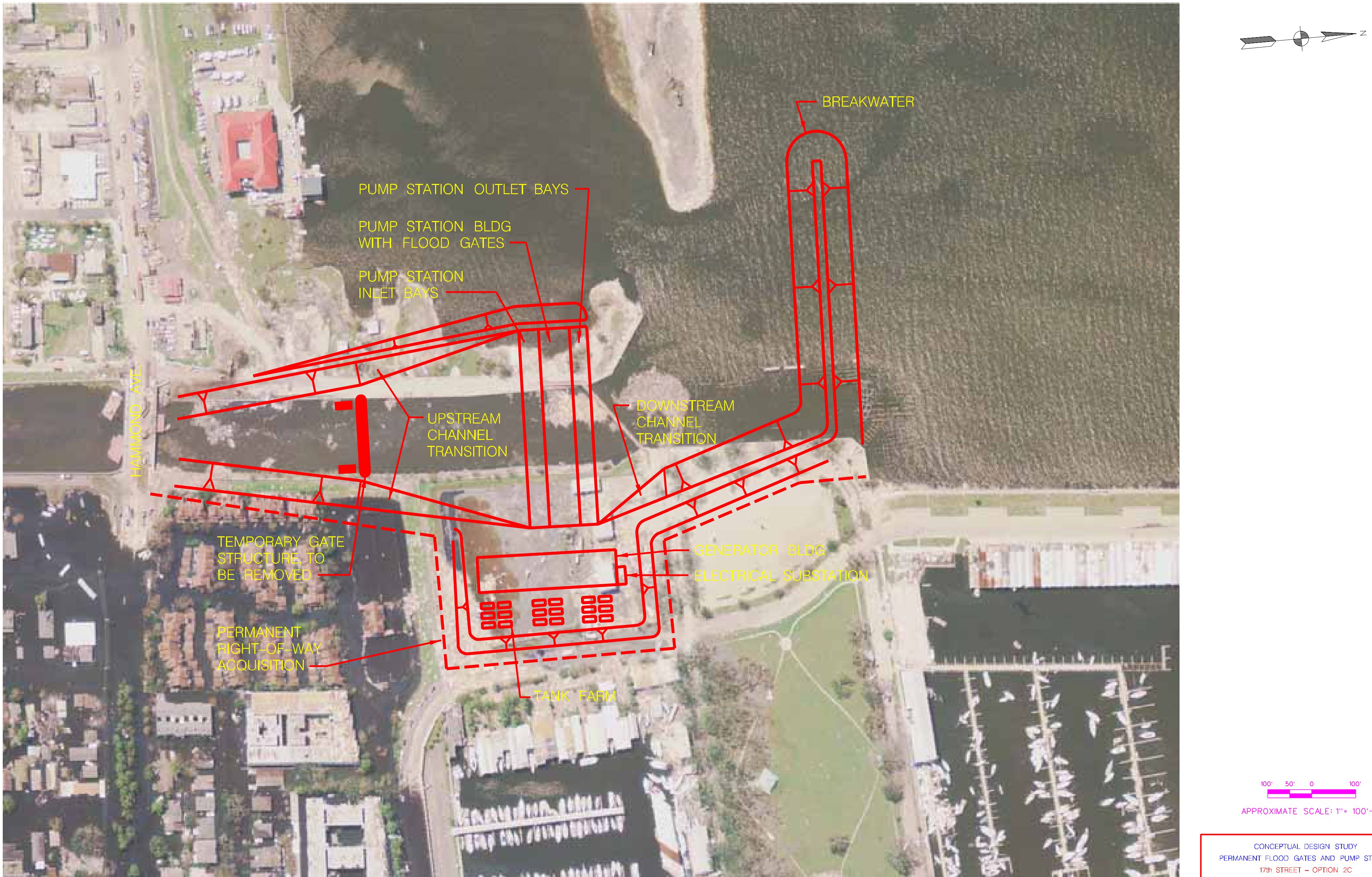

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17th STREET – OPTION 2C
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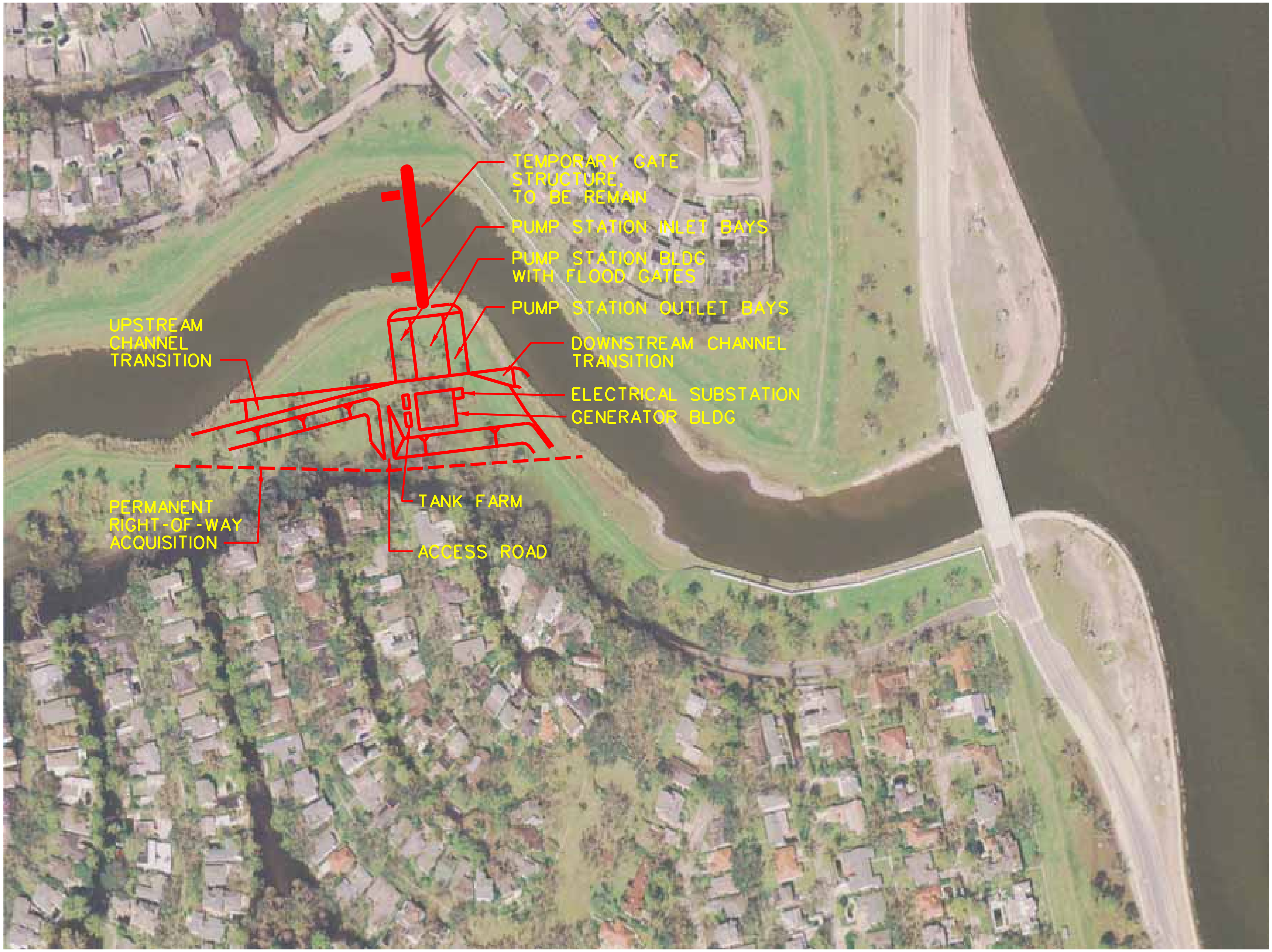

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PERMANENT FLOOD GATES AND PUMP STATION
ORLEANS AVENUE – OPTION 1A
EXHIBIT 5.3.1.A

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100' 50' 0 100'

APPROXIMATE SCALE: 1"= 100'-0"

CONCEPTUAL DESIGN STUDY
PERMANENT FLOOD GATES AND PUMP STATION
ORLEANS AVENUE – OPTION 1B
EXHIBIT 5.3.1.B

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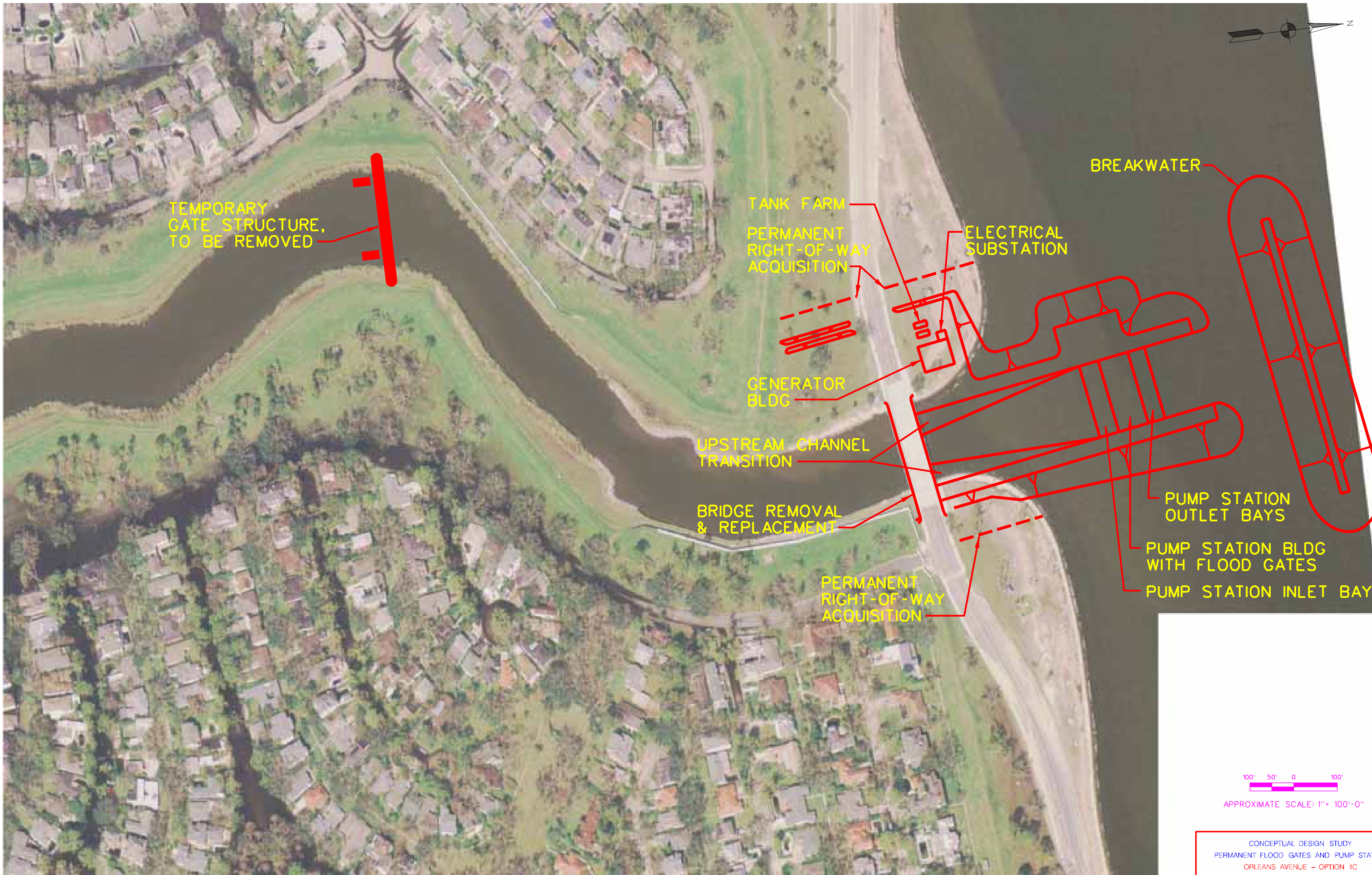

CONCEPTUAL DESIGN STUDY
PERMANENT FLOOD GATES AND PUMP STATION
ORLEANS AVENUE – OPTION 1C
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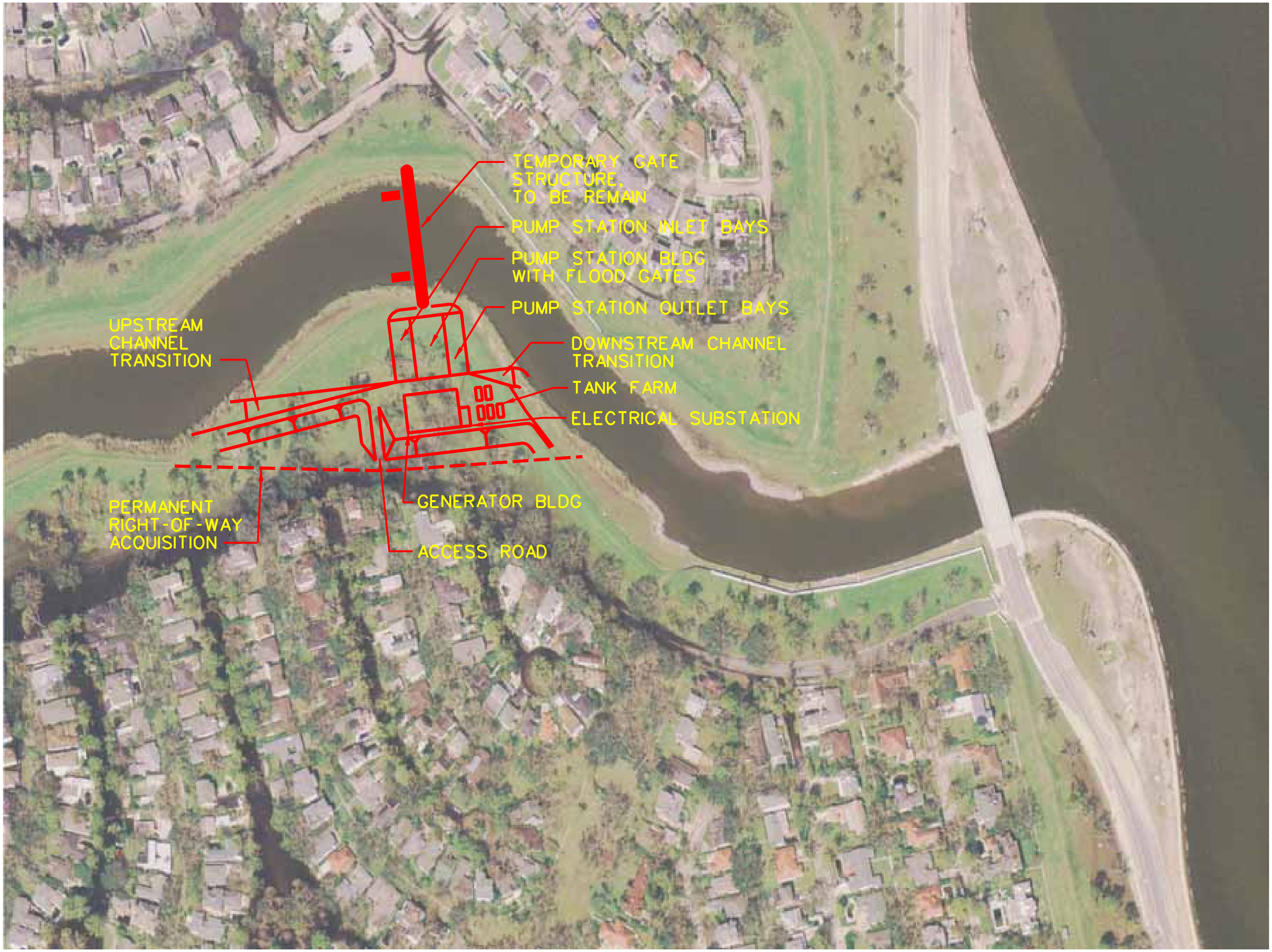

N

100' 50' 0 100'

APPROXIMATE SCALE: 1"= 100'-0"

CONCEPTUAL DESIGN STUDY
PERMANENT FLOOD GATES AND PUMP STATION
ORLEANS AVENUE – OPTION 2A
EXHIBIT 5.4.1.A

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CONCEPTUAL DESIGN STUDY
PERMANENT FLOOD GATES AND PUMP STATION
ORLEANS AVENUE – OPTION 2B
EXHIBIT 5.4.1.B

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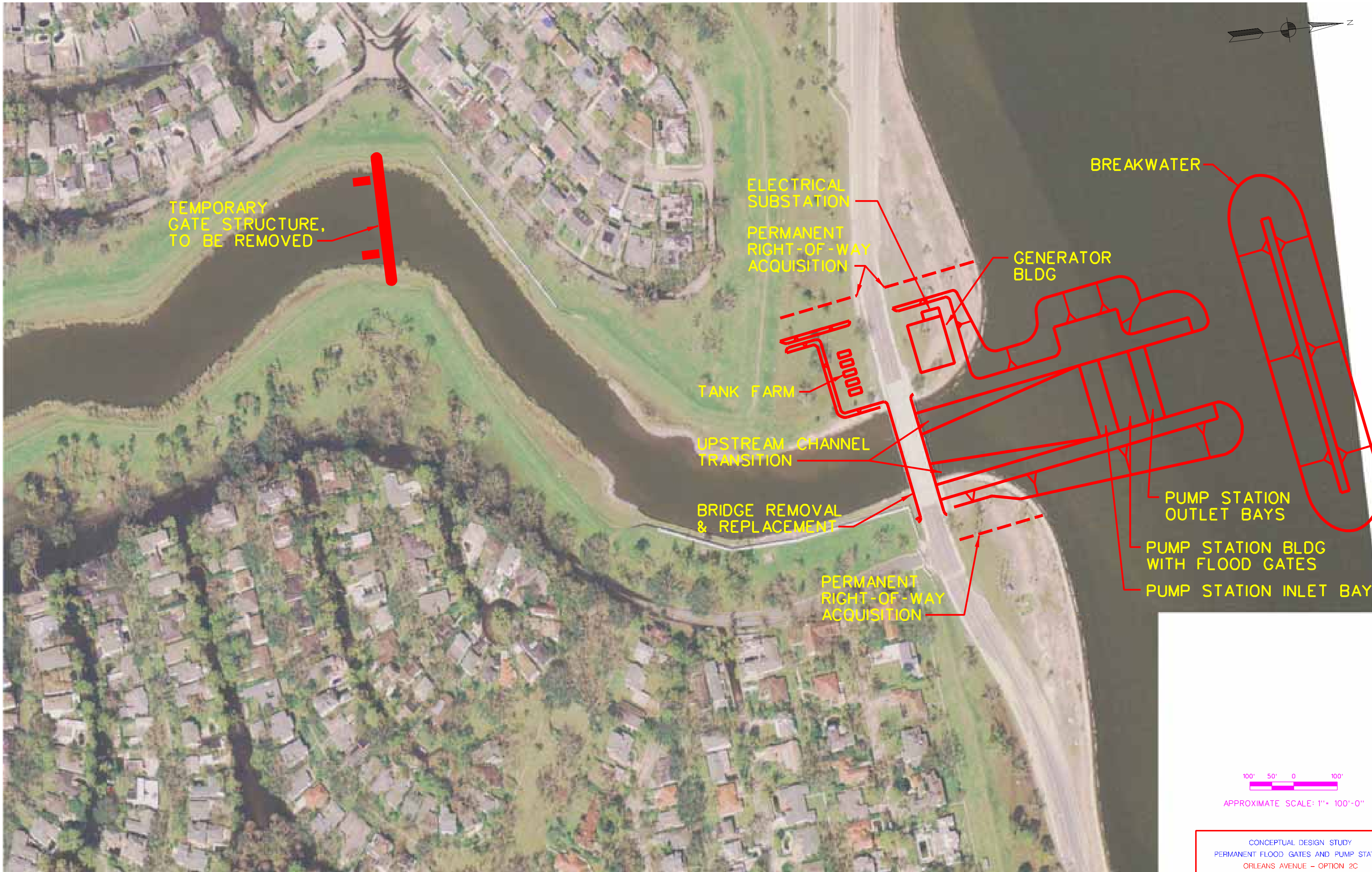

CONCEPTUAL DESIGN STUDY
PERMANENT FLOOD GATES AND PUMP STATION
ORLEANS AVENUE – OPTION 2C
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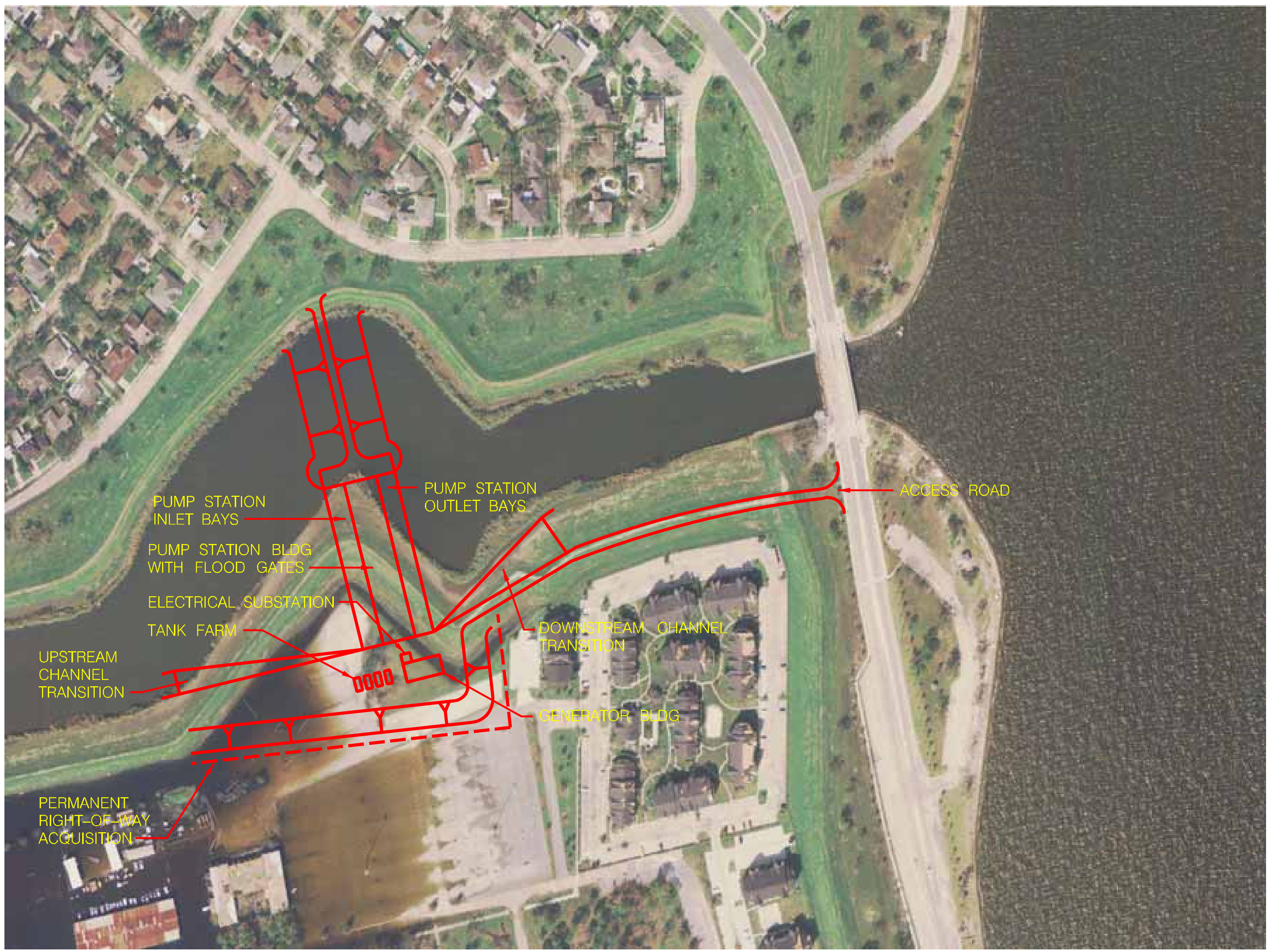
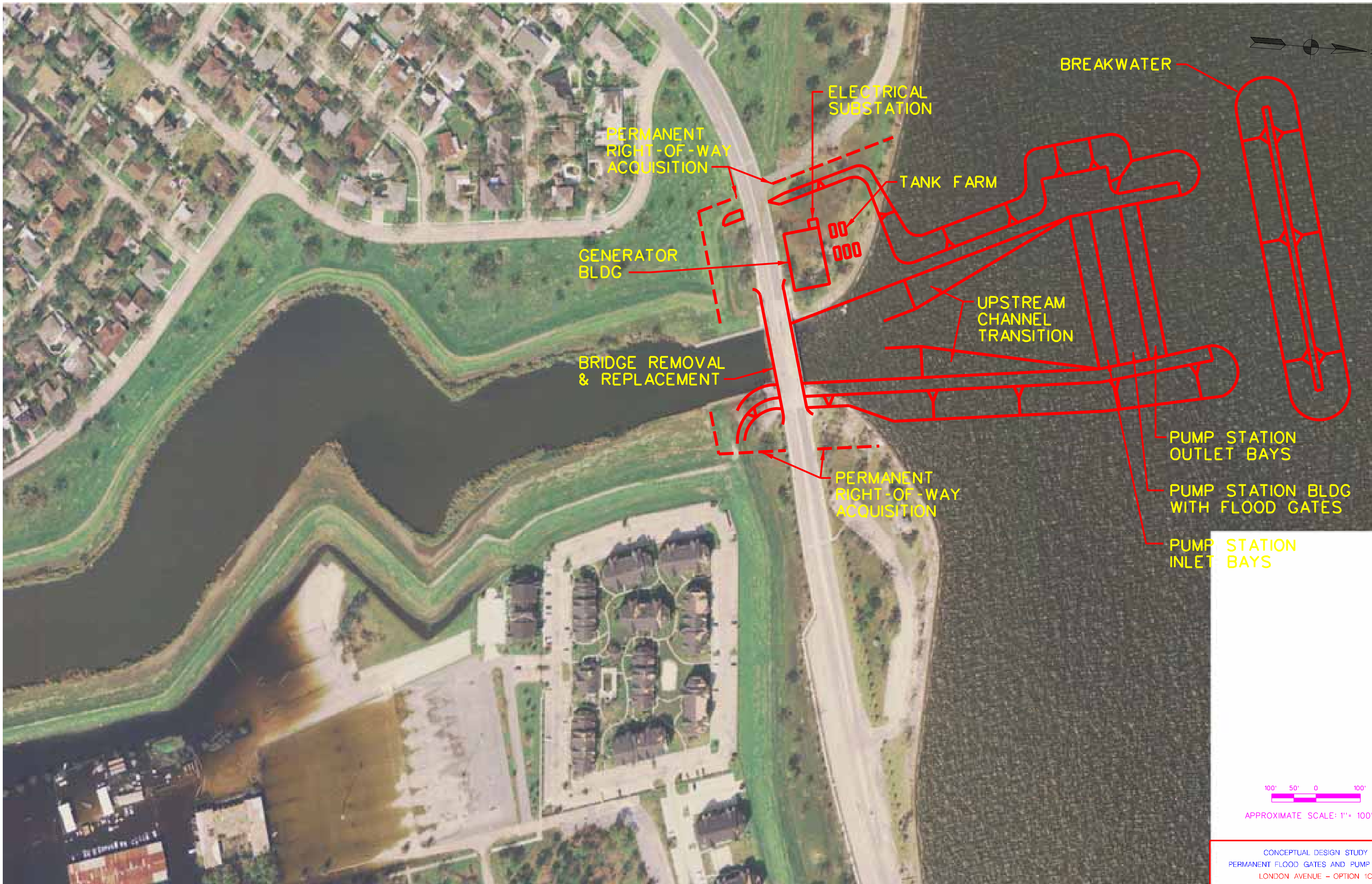


CONCEPTUAL DESIGN STUDY
PERMANENT FLOOD GATES AND PUMP STATION
LONDON AVENUE – OPTION 1A
EXHIBIT 5.5.1.A

CONCEPTUAL DESIGN STUDY
PERMANENT FLOOD GATES AND PUMP STATION
LONDON AVENUE – OPTION 1B
EXHIBIT 5.5.1.B



CONCEPTUAL DESIGN STUDY
PERMANENT FLOOD GATES AND PUMP STATION
LONDON AVENUE – OPTION 1C
EXHIBIT 5.5.1.C

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$FILE$

CONCEPTUAL DESIGN STUDY
PERMANENT FLOOD GATES AND PUMP STATION
LONDON AVENUE – OPTION 2A
EXHIBIT 5.6.1.A

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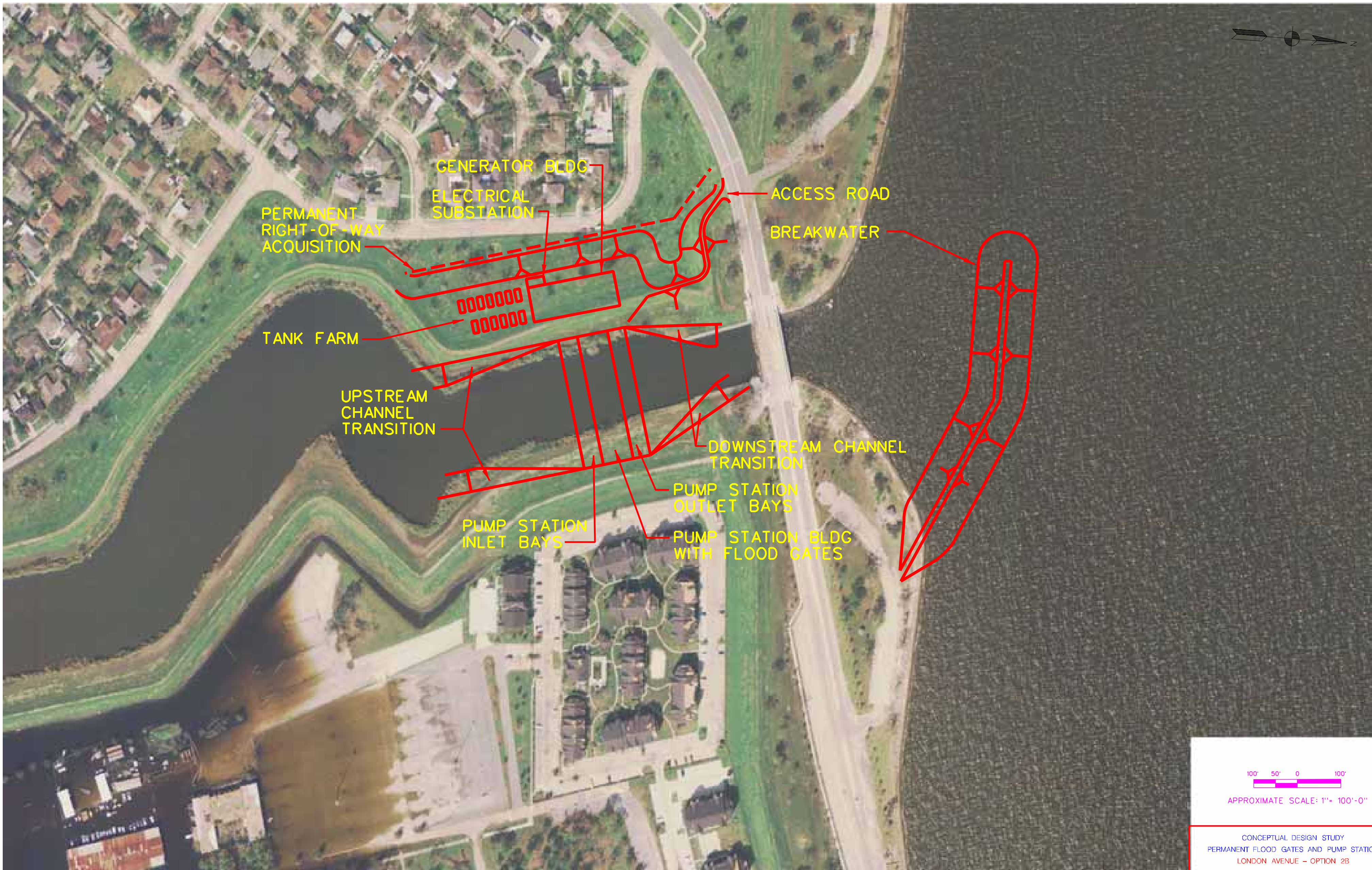

CONCEPTUAL DESIGN STUDY
PERMANENT FLOOD GATES AND PUMP STATION
LONDON AVENUE – OPTION 2B
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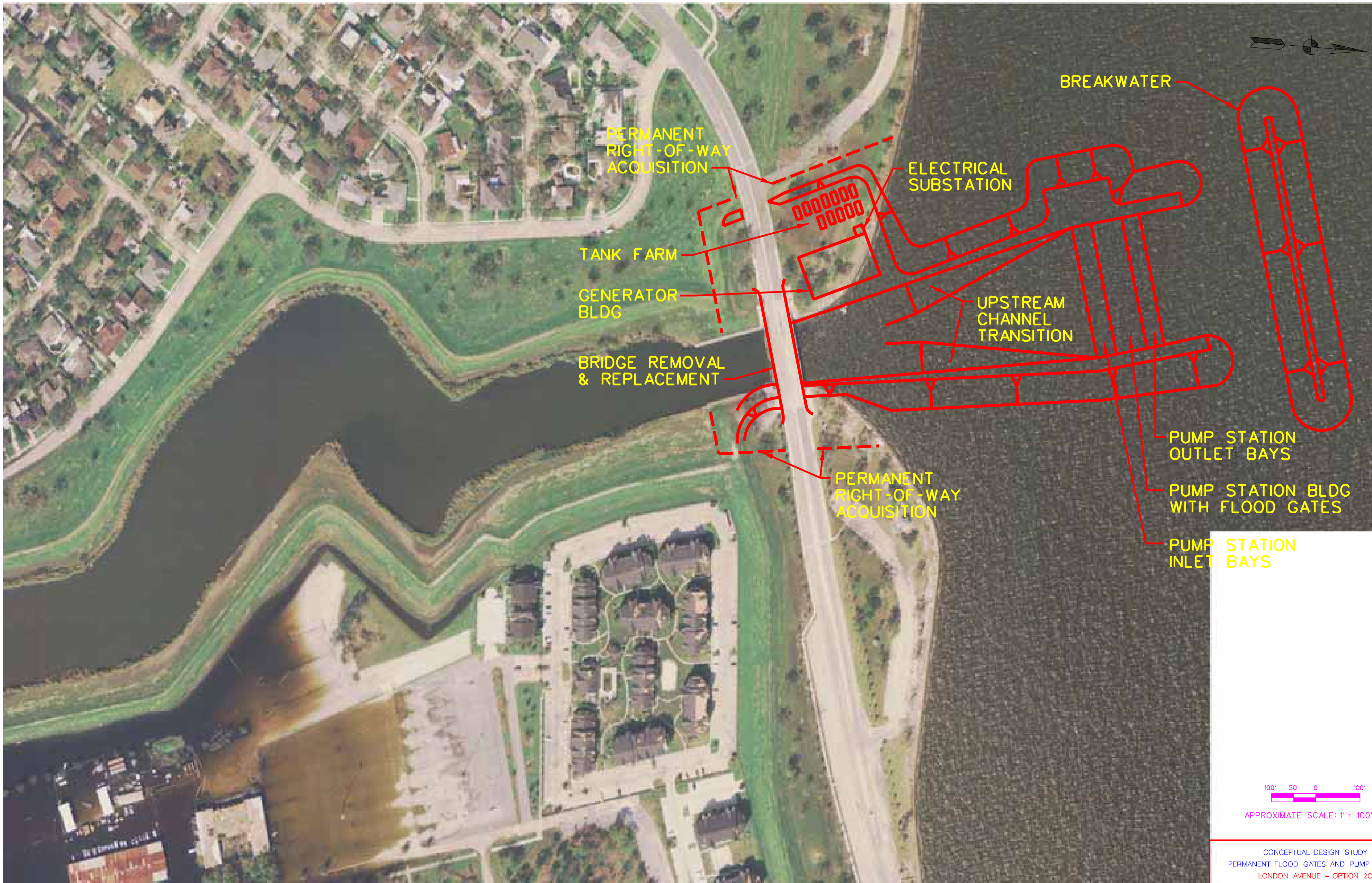

CONCEPTUAL DESIGN STUDY
PERMANENT FLOOD GATES AND PUMP STATION
LONDON AVENUE – OPTION 2C
EXHIBIT 5.6.1.C

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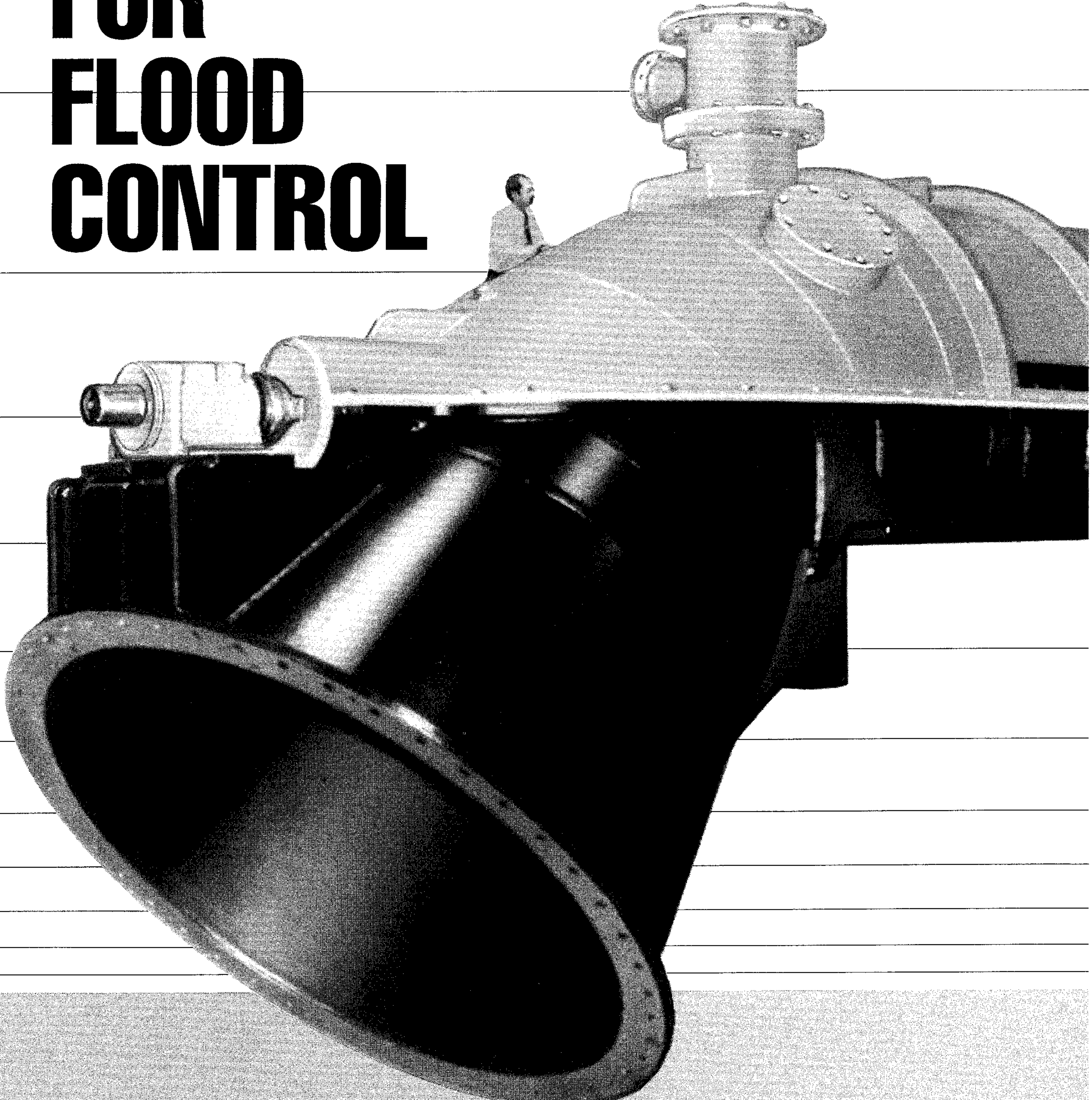
Appendix D

MECHANICAL

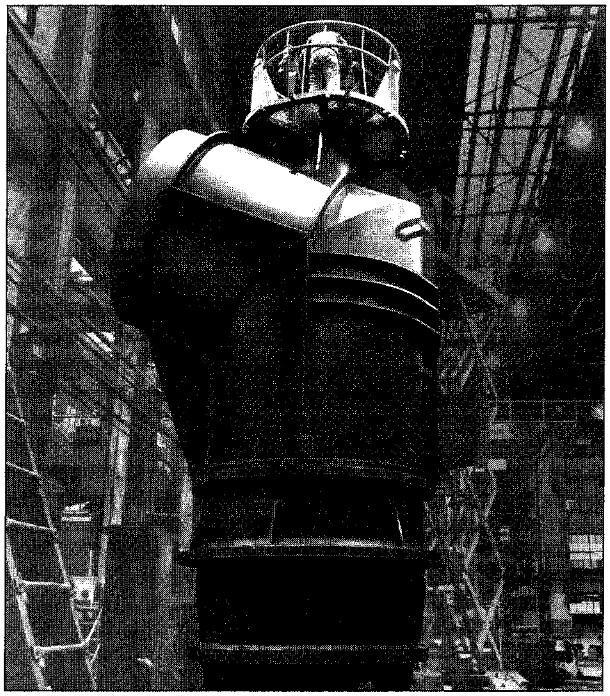
ITT A-C Pump
ITT Fluid Technology Corporation

PUMPS FOR FLOOD CONTROL



VERTICAL WET PIT COLUMN PUMP

Types WCAX, YDD and WCA



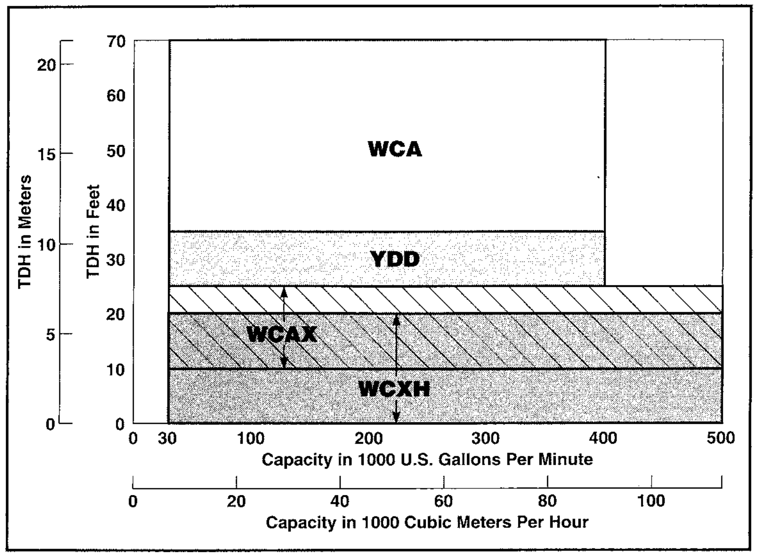
Vertical Wet Pit Pumps Offer Maximum Flexibility

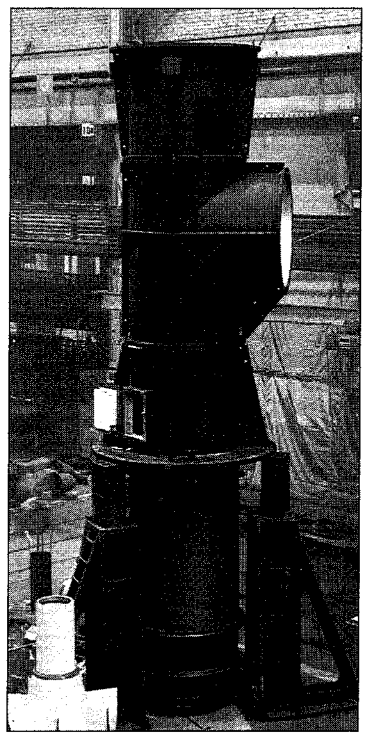
The vertical wet pit column pump is the backbone of flood control applications. It has the capability of operating over a wide range of heads, varying suction water levels, and takes a minimum of floor space.

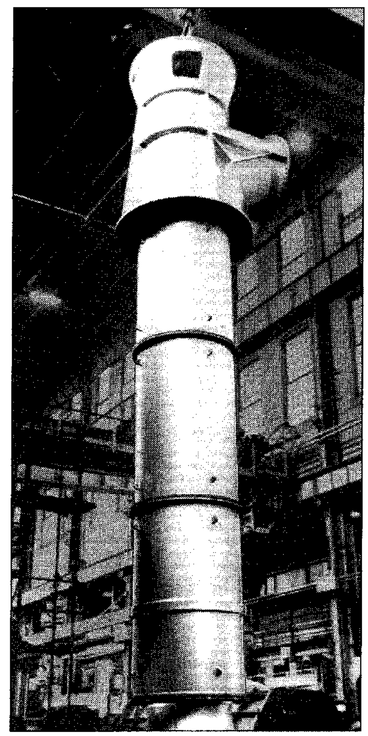
ITT A-C Pump offers several specific speed designs in the axial and mixed flow range to meet a broad range of customer requirements. Mechanical designs are HEAVY-DUTY for long life and reliability.

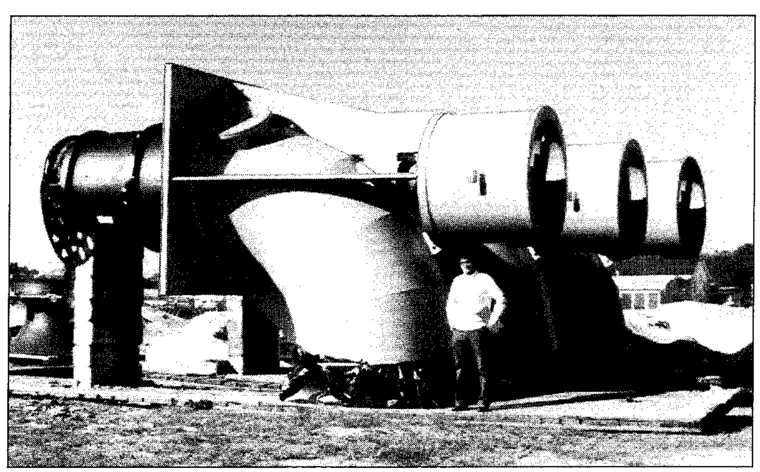
ITT A-C Pump offers a full range of pump materials of construction including either cast or fabricated bowls to handle fresh, brackish or sea water. Typically the discharge elbow and column are constructed of fabricated steel and the bowl components are cast for maximum hydraulic performance. Our computer finite element stress analysis programs are used to determine required wall thickness and rib location for maximum rigidity on fabricated components. Bearings are available in water lubricated fluted rubber or grease lubricated bronze designs. When the design requires intermediate bearings, they are rigidly supported by spiders fitted to the column pipe. Bearing lengths and spans are optimized through computerized lateral and torsional critical speed analysis. Shaft protecting sleeves are located along the pump shaft at the bearings and the stuffing box for ease of maintenance and long pump life.

Capacity and Head Range









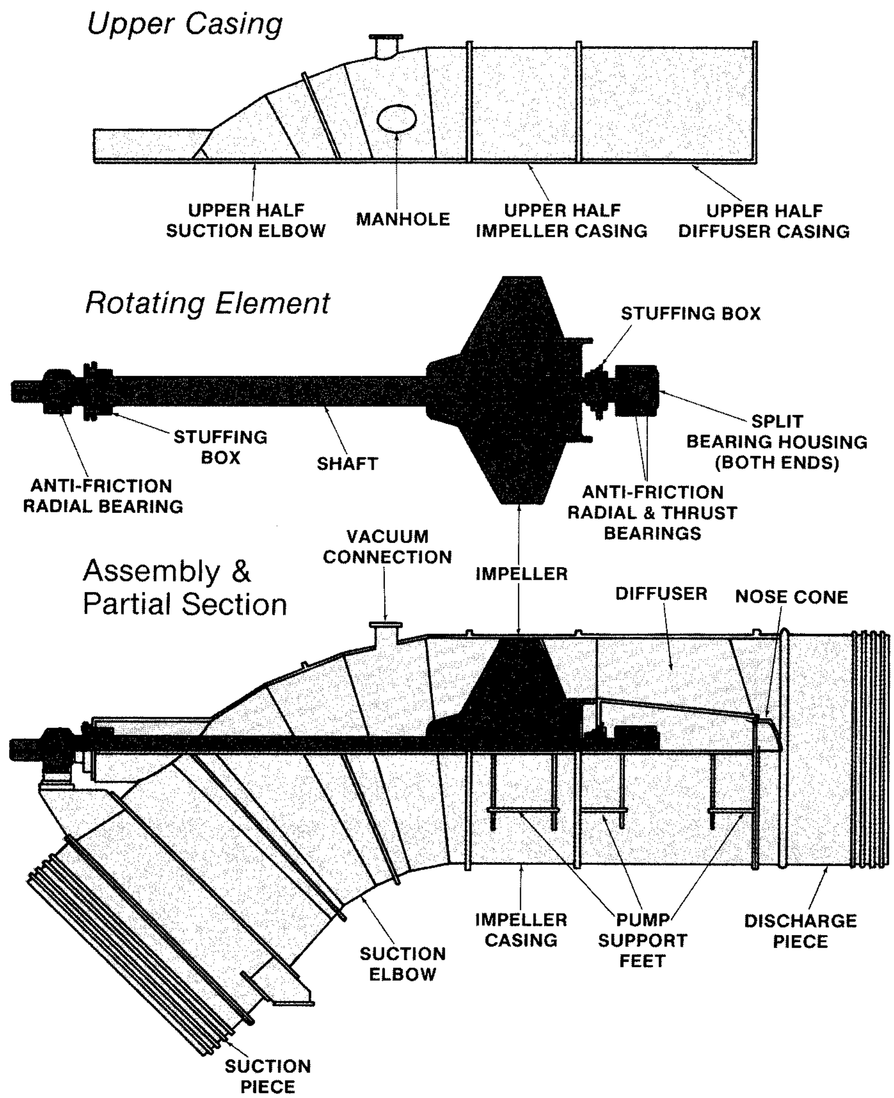
HORIZONTAL PUMP

Type WCXH

The original design of this pump has been proven over many years of service. The horizontal arrangement has been used extensively in New Orleans, Louisiana, where flood control is a way of life.

Advantages of the WCXH Pump

The major advantage of this type of pump is the rotating element sits "high and dry" when the pump is not in use. In addition, the casing is split horizontally for easy access to the removable rotating assembly. In contrast the vertical wet pit bowl assembly is submerged, being subject to the corrosive effects of the pumped water while sitting idle in the standby condition. Thus the horizontal design offers maximum life and reliability as well as ease of maintenance. Because the horizontal pump sits out of the water, sump excavation is reduced. With the horizontal arrangement a vacuum system is used to prime the pump during start-up.



ITT A-C PUMP WCXH CONSTRUCTION FEATURES

The **top casing half** is removable exposing complete rotating assembly for ease of maintenance and removal.

The **impeller** is single suction, open type, offering excellent suction lift characteristics, and is available in a variety of cast alloys.

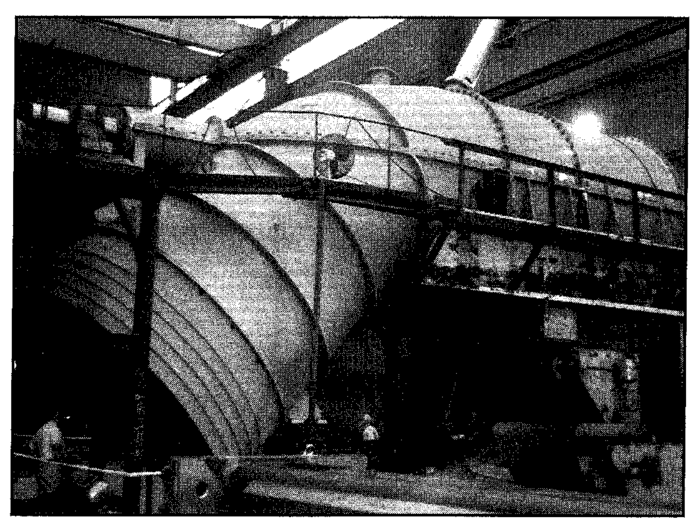
The **casing** is heavy-walled fabricated steel. Suction elbow, impeller casing and diffuser section are all flanged horizontally and vertically.

The **bearings** are self oil lubricated anti-friction type for maximum life. Bearing housings are horizontally split for bearing inspection and maintenance. The nose cone of the diffuser is removable to provide access to the inboard bearing assembly. The outboard radial bearing housing is supported at the suction elbow.

The **shaft** is precisely machined from alloy steel to receive the impeller, bearings, sleeves and coupling. It is conservatively sized to transmit the maximum required power exhibiting lateral and torsional critical speeds safely above the maximum rotating speed of the machine.

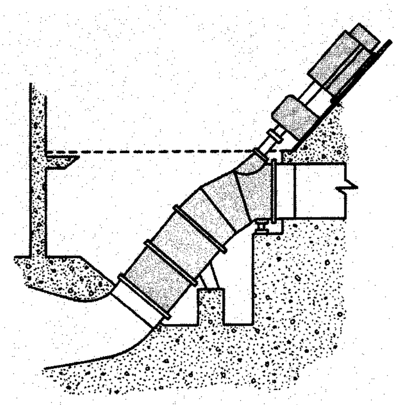
Shaft sleeves protect the shaft where it passes through the stuffing boxes (or at fluted rubber bearings when applied). 400 series stainless steel, hardened to 500 BHN minimum is used for extended life.

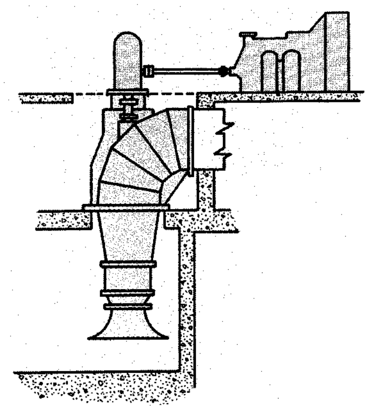
Stuffing boxes are located at the inboard bearing housing to seal the inboard bearing chamber from process water and at the shaft exit through the suction elbow to control leakage.

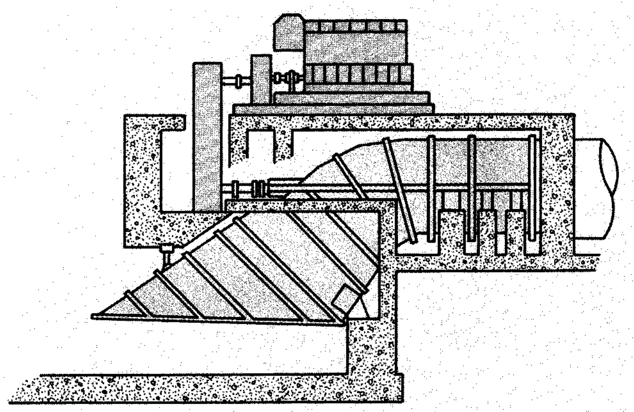


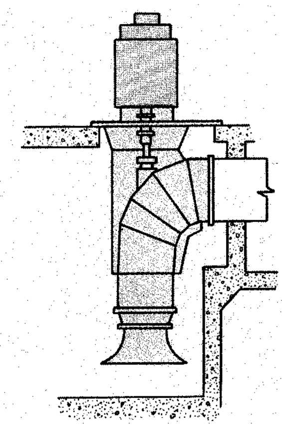
A 144" x 132" WCXH Pump rated 516,000 GPM at 9.5 Ft. TDH.

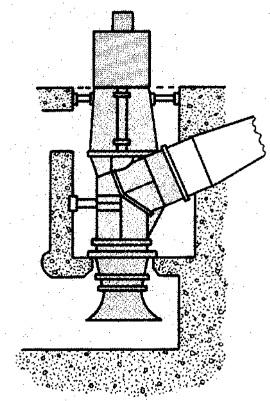
PUMP EXPERIENCE PUMP KNOWLEDGE



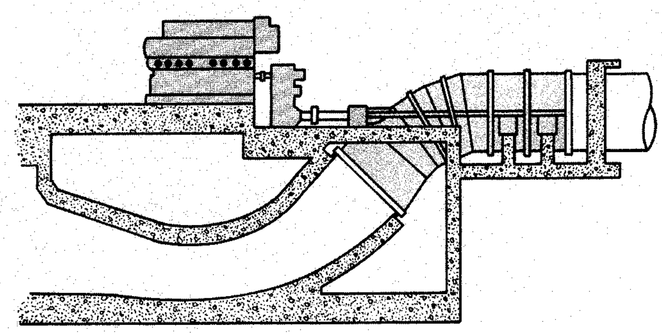








Experienced Custom Designs



Topography, variable suction and discharge water levels, available space...all vary so widely from one site to another that each flood control application is a unique engineering proposition.

Vertical, horizontal or angle flow type pumps; we have them all. Turnkey equipment packaging with drivers, pumps and valves; we have the experience. Applications, Engineering and Project Management; we have the talent. All backed by 120 years of pump experience, and a users list that gets longer every year.

ITT A-C Pump

A unit of ITT Corporation

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Pewaukee, WI 53072 USA
Telephone: 414/548-8181
Fax: 414/548-8170

INTERNATIONAL SALES
445 Godwin Avenue
Midland Park, NJ 07432 USA
Telephone: 201/444-6030
Fax: 201/444-0124

ITT A-C PUMP
1150 Tennessee Avenue
Cincinnati, OH 45229 USA
Telephone: 513/482-2500
Fax: 513/482-2569

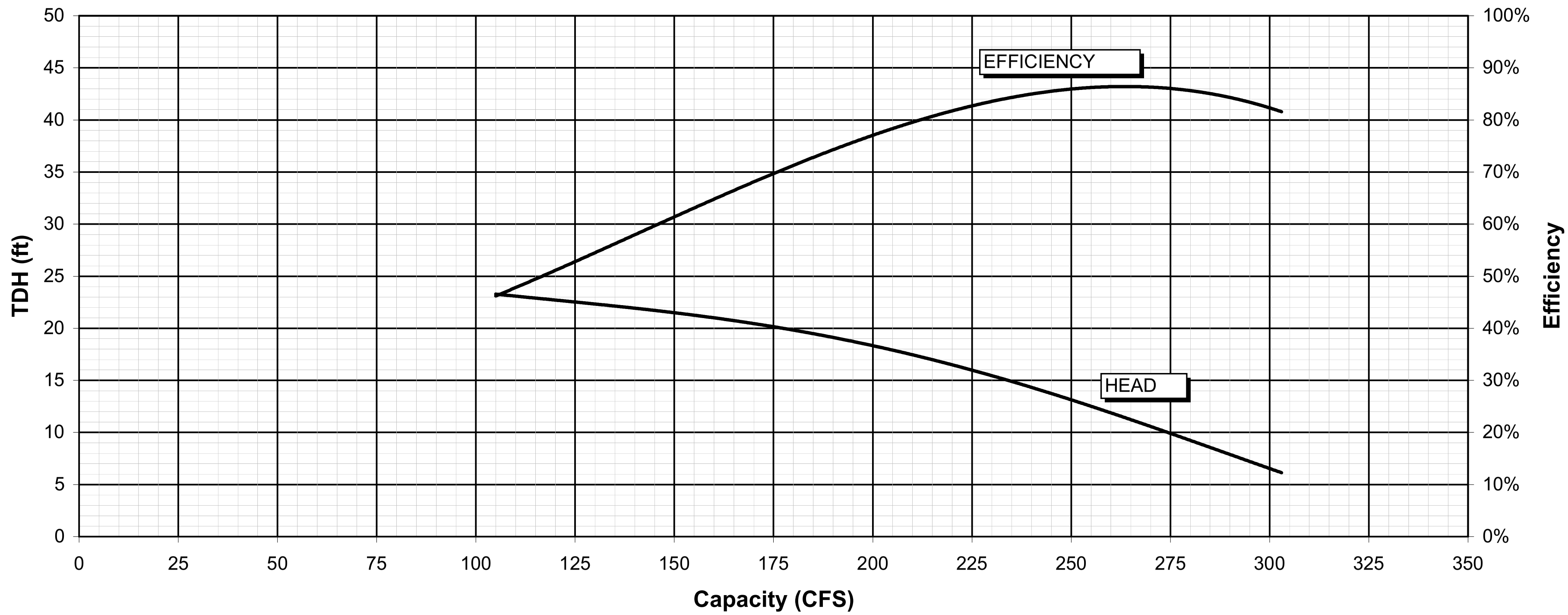
08B3000

Proposed Performance

Order Number	WFLT-20334	**Liquid**	Water	**Speed (RPM)**	264	**Capacity (CFS)**	250
Customer	B&V	**Temperature (°F)**		**Pump type**	64APS	**TDH (ft)**	
Service		**Viscosity (Cp)**		**Pump Serial No.**		**Efficiency**	
Date	7/21/2006	**Specific Gravity**	1			**NPSHR (ft)**	
	250CFS Option #1						

Rev. A

Curves are approximate. Pump is guaranteed for one set of conditions. Capacity, head, and efficiency guarantees are based on shop test and when handling clear, cold, fresh water at a temperature of not over 85 degrees.

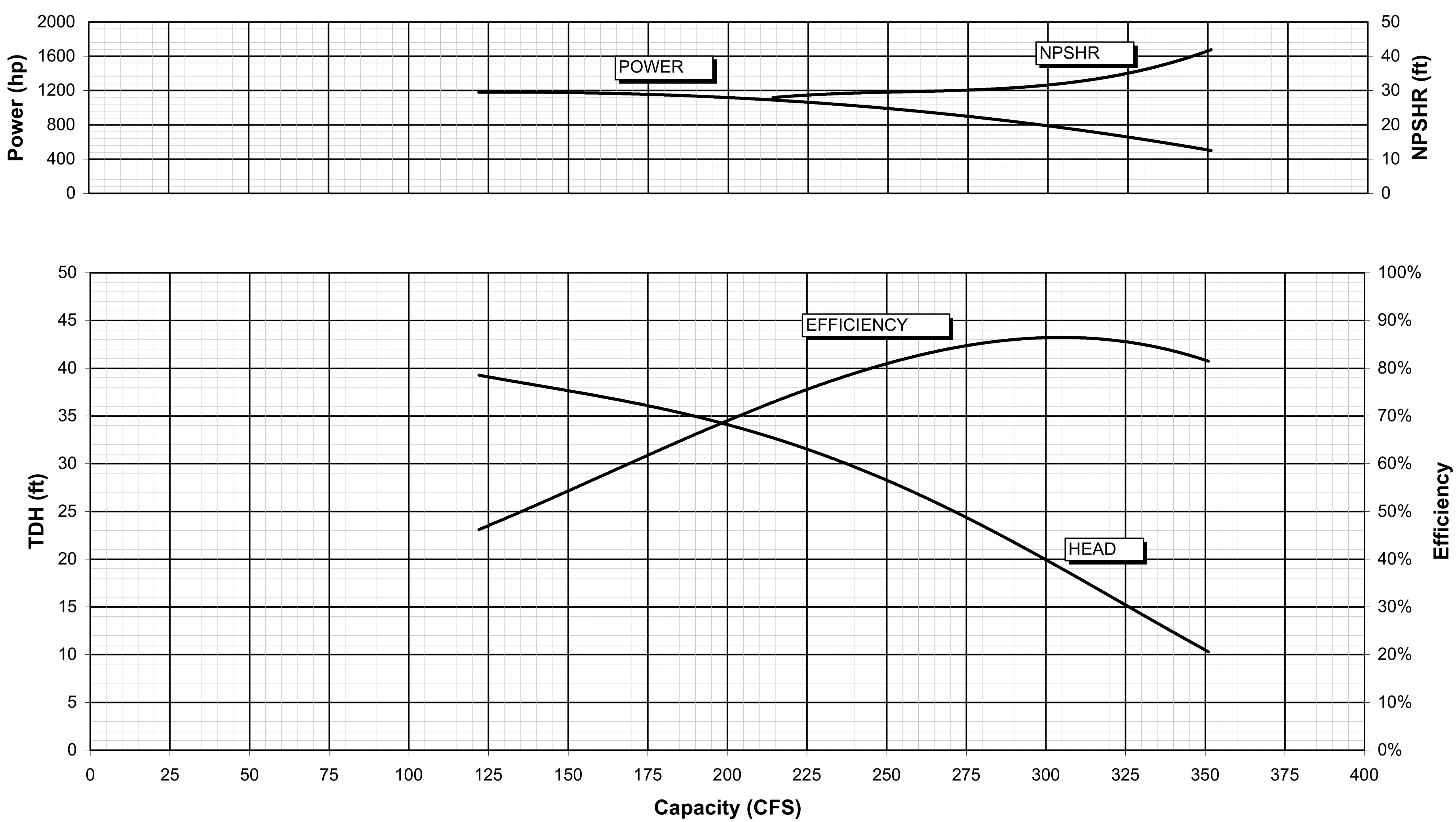


Proposed Performance

Order Number	WFLT-20334	**Liquid**	Water	**Speed (RPM)**	362	**Capacity (CFS)**	250
Customer	B&V	**Temperature (°F)**		**Pump type**	60APS	**TDH (ft)**	
Service		**Viscosity (Cp)**		**Pump Serial No.**		**Efficiency**	
Date	7/21/2006	**Specific Gravity**	1			**NPSHR (ft)**	
	250CFS Option #2						

Rev. A

Curves are approximate. Pump is guaranteed for one set of conditions. Capacity, head, and efficiency guarantees are based on shop test and when handling clear, cold, fresh water at a temperature of not over 85 degrees.

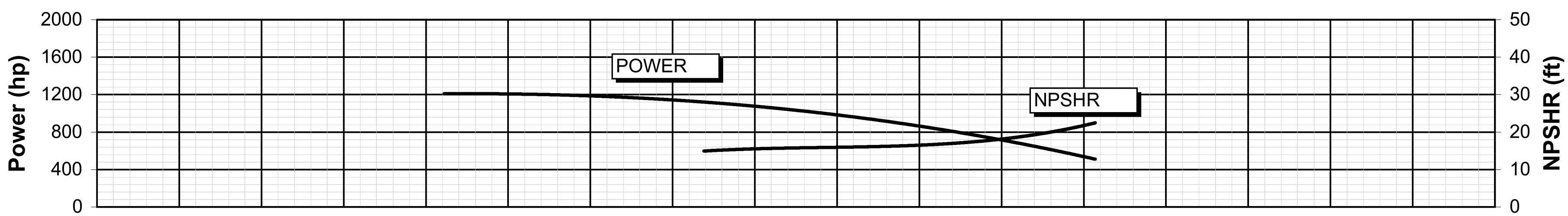


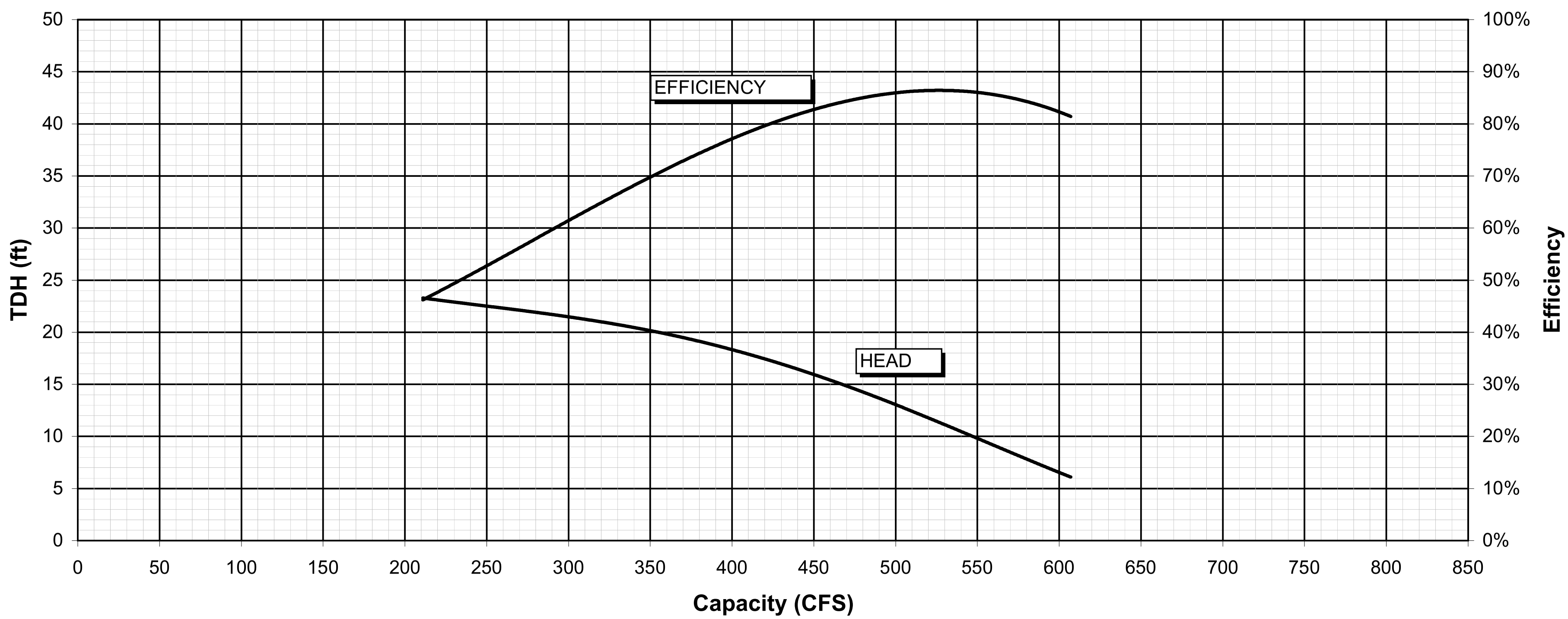
Proposed Performance

Order Number	WFLT-20334	**Liquid**	Water	**Speed (RPM)**	187	**Capacity (CFS)**	500
Customer	B&V	**Temperature (°F)**		**Pump type**	90APS	**TDH (ft)**	
Service		**Viscosity (Cp)**		**Pump Serial No.**		**Efficiency**	
Date	7/21/2006	**Specific Gravity**	1			**NPSHR (ft)**	
	500CFS Option 1						

Rev. A

Curves are approximate. Pump is guaranteed for one set of conditions. Capacity, head, and efficiency guarantees are based on shop test and when handling clear, cold, fresh water at a temperature of not over 85 degrees.



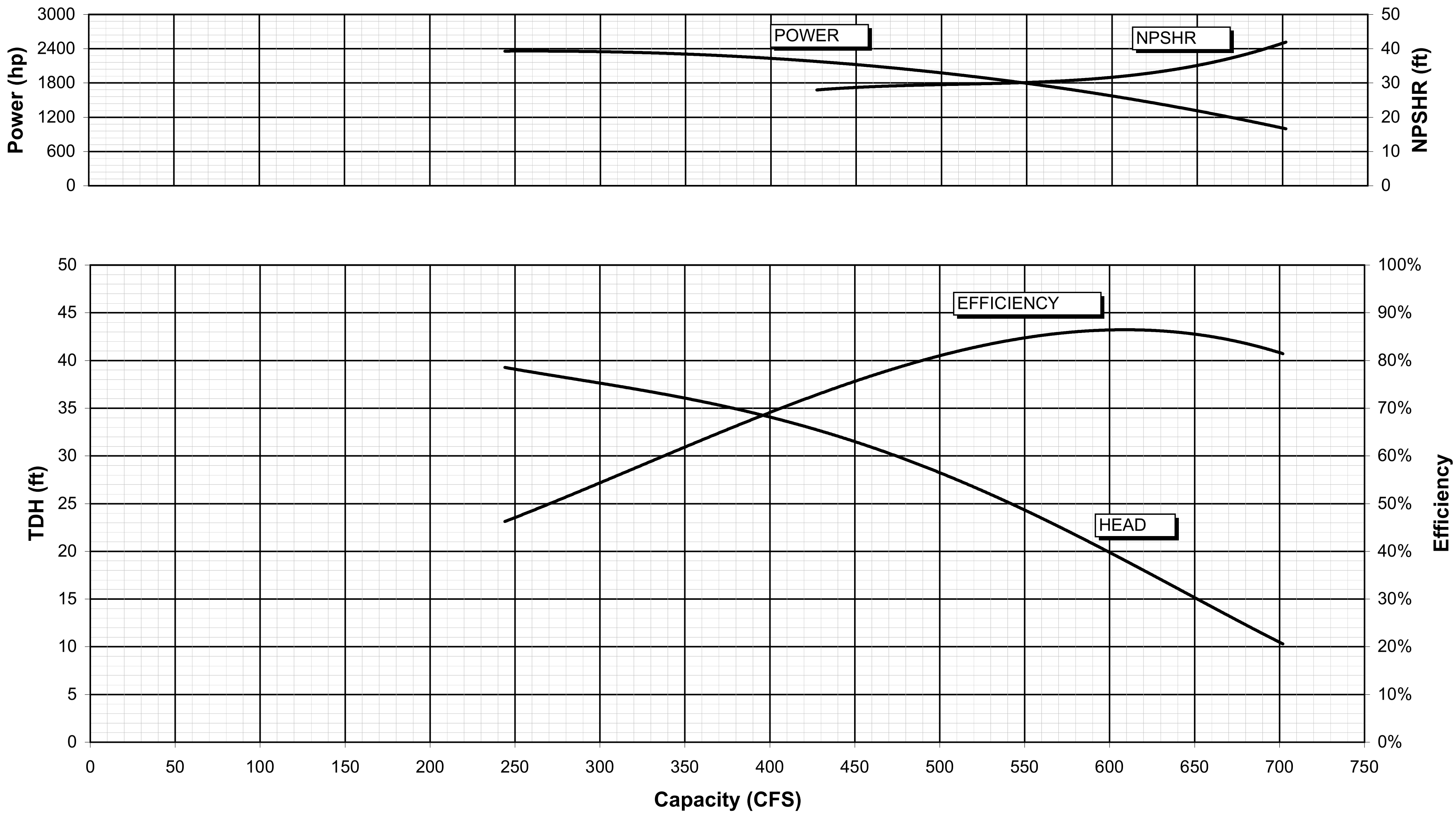


Proposed Performance

Order Number	WFLT-20334	**Liquid**	Water	**Speed (RPM)**	256	**Capacity (CFS)**	500
Customer	B&V	**Temperature (°F)**		**Pump type**	85APS	**TDH (ft)**	
Service		**Viscosity (Cp)**		**Pump Serial No.**		**Efficiency**	
Date	7/21/2006	**Specific Gravity**	1			**NPSHR (ft)**	
	500CFS Option #2						

Rev. A

Curves are approximate. Pump is guaranteed for one set of conditions. Capacity, head, and efficiency guarantees are based on shop test and when handling clear, cold, fresh water at a temperature of not over 85 degrees.

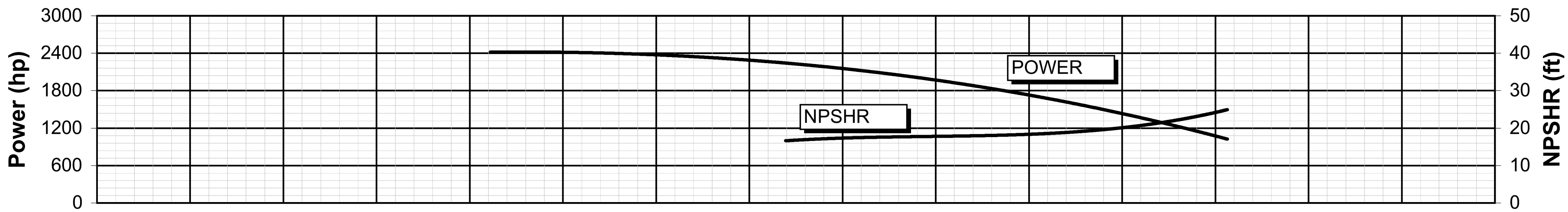


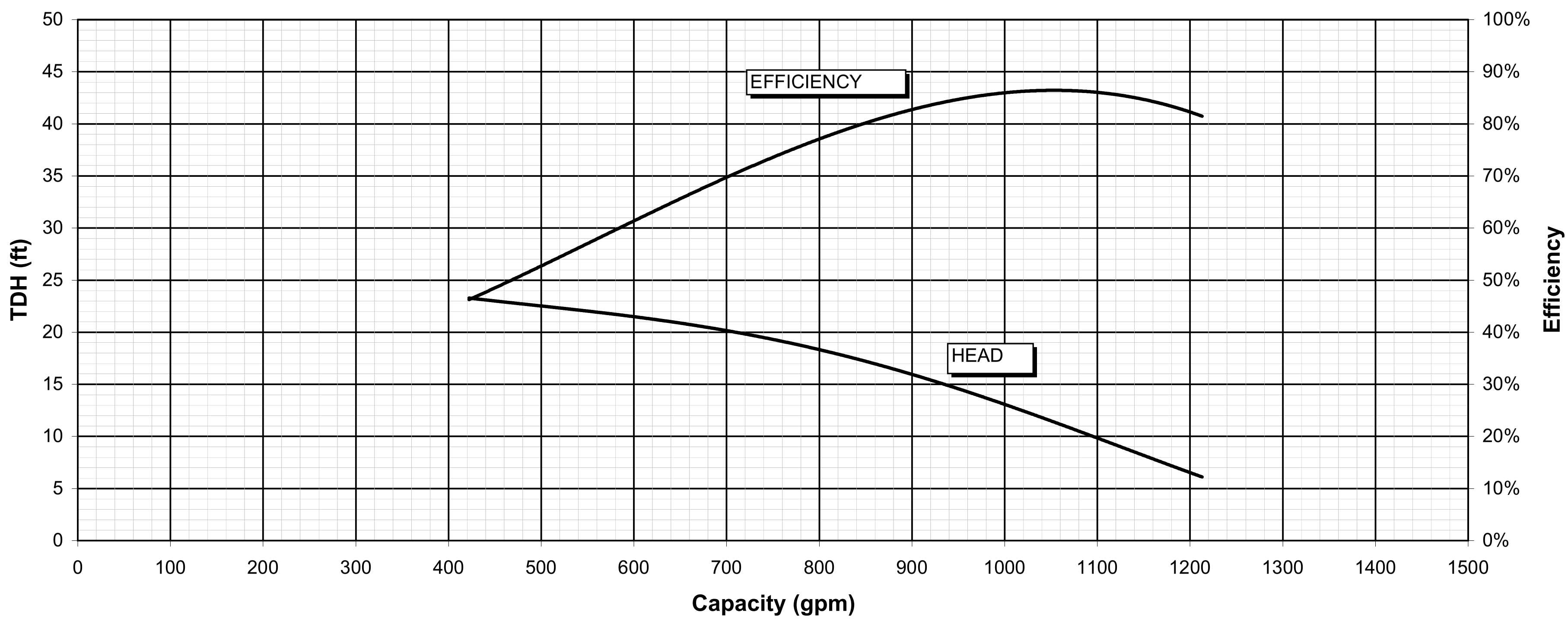
Proposed Performance

Order Number	WFLT-20334	**Liquid**	Water	**Speed (RPM)**	132	**Capacity (CFS)**	1000
Customer	B&V	**Temperature (°F)**		**Pump type**	127APS	**TDH (ft)**	
Service		**Viscosity (Cp)**		**Pump Serial No.**		**Efficiency**	
Date	7/19/2006	**Specific Gravity**	1			**NPSHR (ft)**	
	1000CFS Option #1						

Rev. A

Curves are approximate. Pump is guaranteed for one set of conditions. Capacity, head, and efficiency guarantees are based on shop test and when handling clear, cold, fresh water at a temperature of not over 85 degrees.



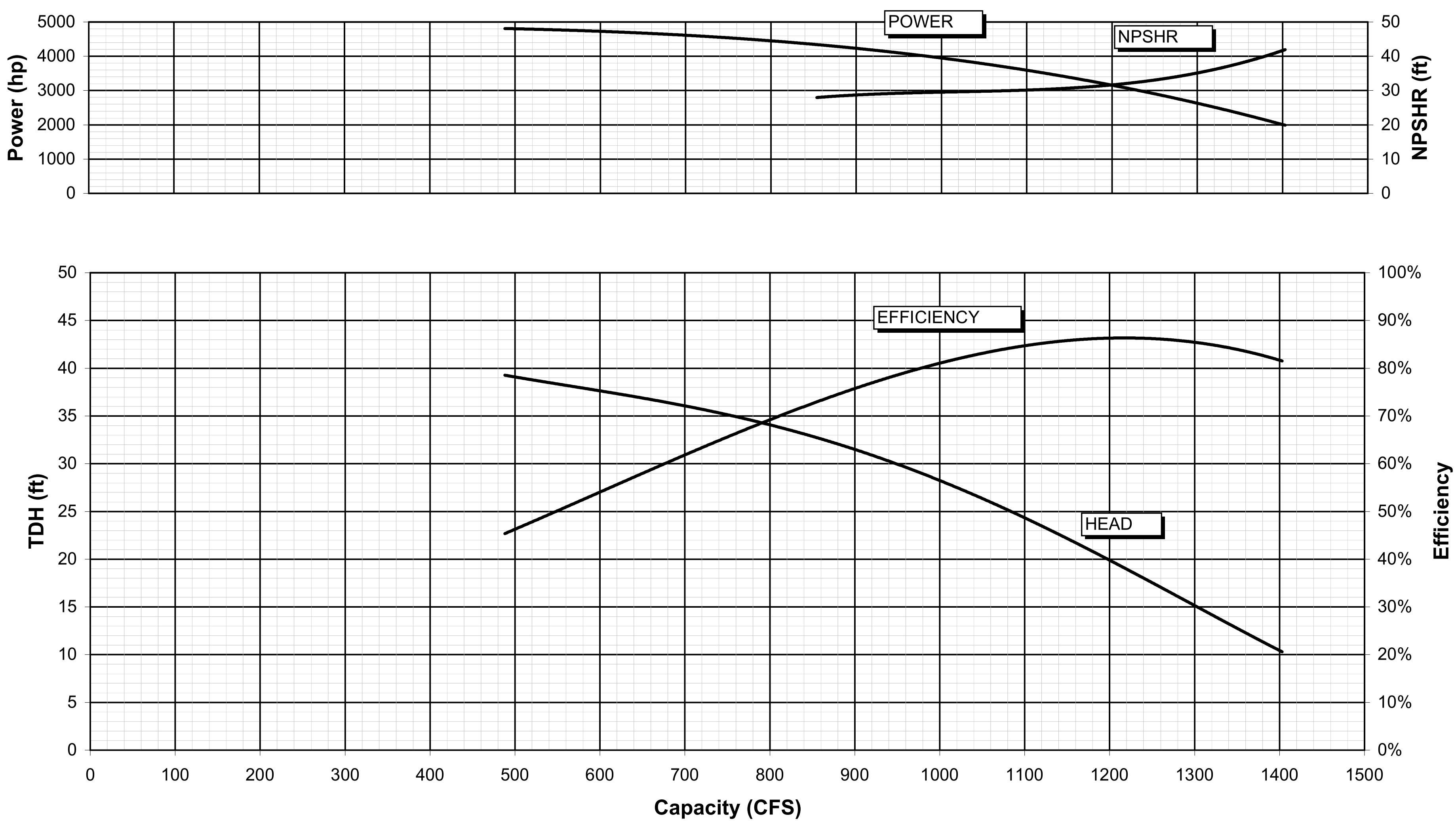


Proposed Performance

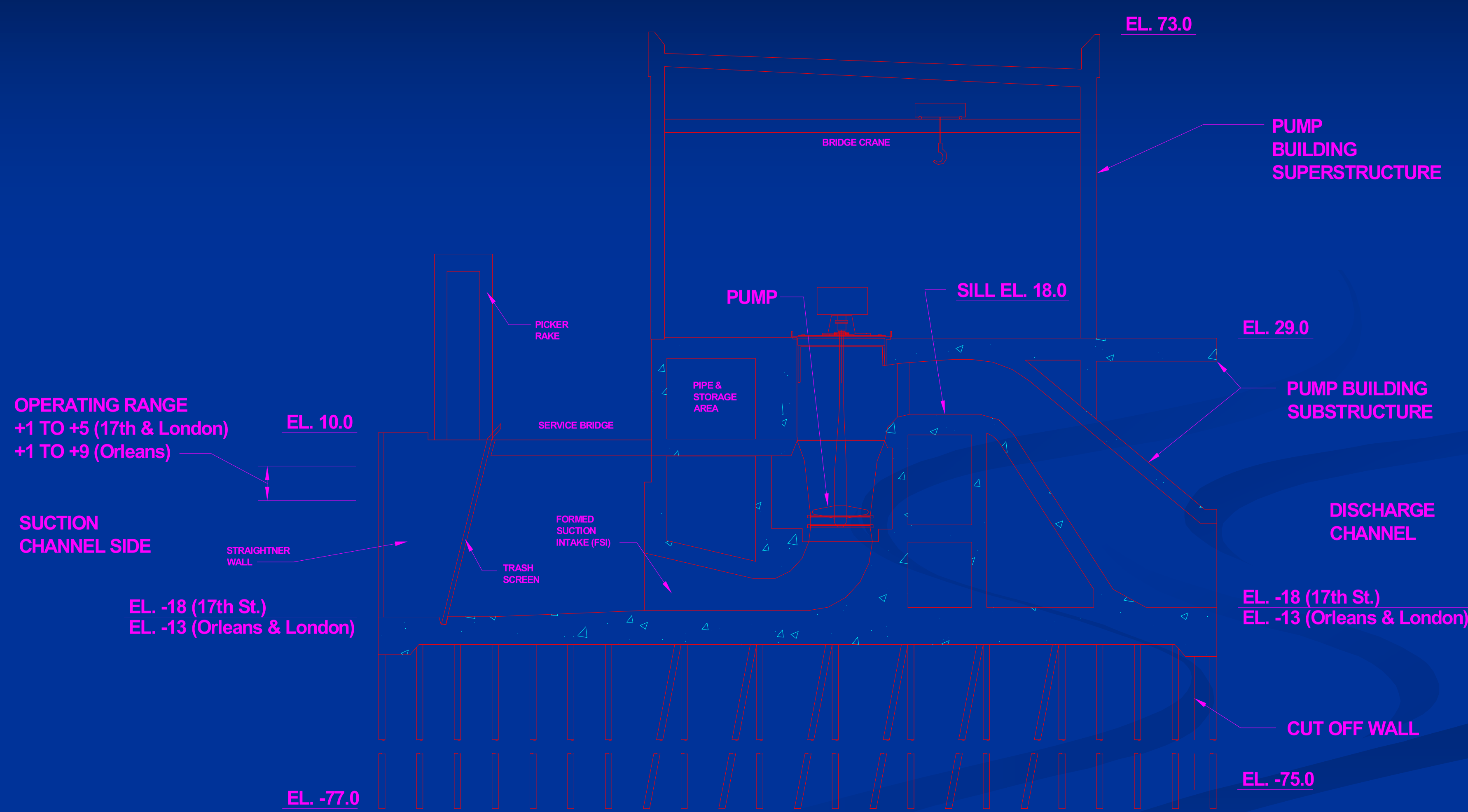
Order Number	WFLT-20334	**Liquid**	Water	**Speed (RPM)**	181	**Capacity (CFS)**	1000
Customer	B&V	**Temperature (°F)**		**Pump type**	120APS	**TDH (ft)**	
Service		**Viscosity (Cp)**		**Pump Serial No.**		**Efficiency**	
Date	7/20/2006	**Specific Gravity**	1			**NPSHR (ft)**	
	1000 CFS Option #2						

Curves are approximate. Pump is guaranteed for one set of conditions. Capacity, head, and efficiency guarantees are based on shop test and when handling clear, cold, fresh water at a temperature of not over 85 degrees.

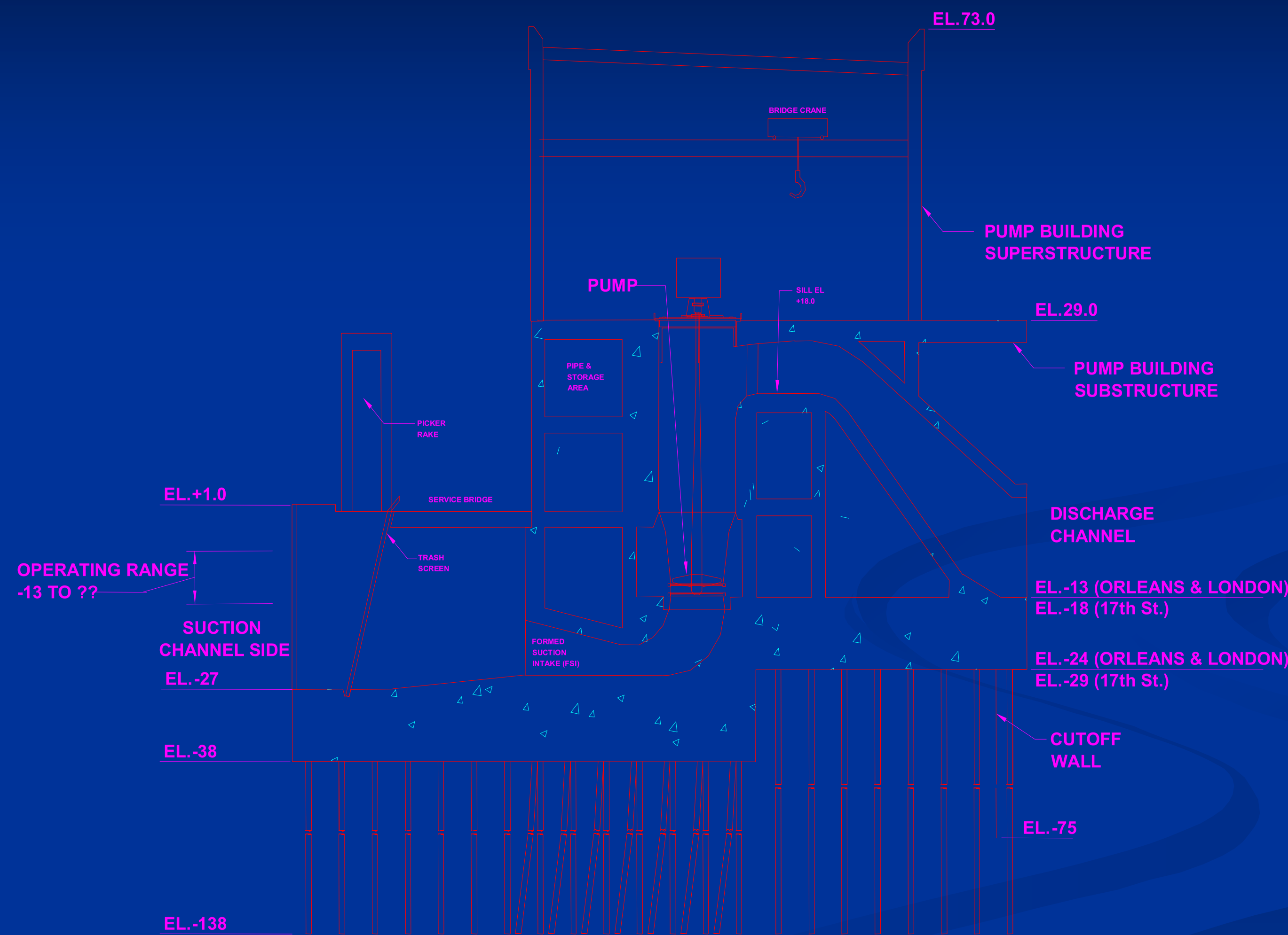
Rev. A



Pump Station – Option 1



Pump Station - Option 2



Concept Study for Permanent Flood Gates and Pump Stations

Gulf Engineers & Consultants

Appendix E

ELECTRICAL

Central Plant Evaluation

This evaluation is to perform the early stages of analysis to consider the option of a Central Power Plant in lieu of local standby power generation at each pump station. Because of the magnitude of the power requirements, many power generation and supply alternatives must be considered with the decision factors including costs, reliability, environmental, plant location, permanent easement for plant and distribution lines, power company's willingness to provide equipment or enter into power purchase/sell, as well as many other factors that affect the feasibility and the ultimate cost of an approach. This evaluation is not intended to determine the feasibility of an approach, but rather to start discussions with the local power company and the stakeholders on the project that may eliminate some approaches without requiring further analysis. For the approaches that are not screened out, further analysis is needed to define feasible approaches for a more refined evaluation. This appendix presents load requirements and rough-order-of-magnitude (ROM) costs for the Central Plant option. Note that this evaluation did not factor into the power generation and supply for Options 1 and 2.

Concept

The Central Power Plant alternative assumes all electric-driven pumps to reflect the most appropriate scenario considering economics. If a central plant were constructed, it would be most economical to drive all the pumps with electric motors. Utility power will also be brought to the Central Plant to provide power for the normal pump loads. Because the utility feed and the standby power system share the same busses and conductors, the system will always be energized. The utility service and power generation system will operate at 13.2kV to match the standard Entergy utility power distribution voltage. Some of the advantages of using a Central Plant are listed below:

- Engine operation and maintenance will be in one location
- Fuel storage will be in one location
- Pumps will all be electric-driven
- Less real estate will be required at each pump station site

The standby power generation units will be configured with an N + 1 design such that if a standby power generation unit goes off line or one is down for maintenance, the full load of all the pump stations will still be supplied with standby power.

13.2kV electrical distribution circuits from the Central Plant to each pump station will be routed underground in concrete-encased ductbank. For additional reliability, multiple dedicated feeders will be routed to each pump station. If not cost prohibitive, the feeders will be routed in separate ductbanks to each pump station.

For the Central Plant option, indoor substations for each pump station feeder will be required at each pump station to step the voltage down from the 13.2kV distribution voltage to 4160V for the pump motors.

Option 1 Evaluation

Load calculations for the Option 1 Central Plant are shown below based on anticipated motor sizes and assumed diversity. The load calculation assumptions include 1 hp = 1 kVA and the pump building load is approximately 5% of the pump motor load.

17th Street Canal – Option 1						
Pump	**Driver**	**Motor hp**	**Motor hp**	**Source**	**Utilization**	
cfs	**bhp**	**Nameplate**	**Load**		**Grid**	**Ind**
1000	1052	2750	1800	Genset		1%
1000	1052	2750	1800	Genset		1%
1000	1052	2750	1800	Genset		1%
1000	1052	2750	1800	Genset		1%
1000	1052	2750	1800	Genset		1%
1000	1052	2750	1800	Genset		1%
1000	1052	2750	1800	Genset		1%
1000	1052	2750	1800	Genset		1%
1000	1052	2750	1800	Genset		1%
1000	1052	2750	1800	Genset		1%
1000	1052	2750	1800	Grid/Gen	1%	1%
500	526	1500	900	Grid/Gen	5%	1%
500	526	1500	900	Grid/Gen	10%	1%
250	263	750	450	Grid/Gen	50%	1%
250	263	750	450	Grid/Gen	100%	1%
12500	**13150**		**22500**	**Total hp (1hp=1kVA)**		
			1125	Bldg Load (kVA) 5% of motor load		
			23625	Total Load (kVA)		
			21.3	MW @ 0.9 PF		
			1033	Amps @ 13.2 kV		
			3281	Amps @ 4160V		
			5625	Utility kVA		
			246	Utility Amps @ 13.2 kV		

Orleans Canal – Option 1						
Pump	**Driver**	**Motor hp**	**Motor hp**	**Source**	**Utilization**	
cfs	**bhp**	**Nameplate**	**Load**		**Grid**	**Ind**
1000	1052	2750	1800	Genset		1%
1000	1052	2750	1800	Genset		1%
500	526	1500	900	Grid/Gen	1%	1%
500	526	1500	900	Grid/Gen	5%	1%
250	263	750	450	Grid/Gen	50%	1%
150	158	500	300	Grid/Gen	100%	1%
3400	**3577**		**6150**	**Total hp (1hp=1kVA)**		
			308	Bldg Load (kVA) 5% of motor load		
			6458	Total Load (kVA)		
			5.8	MW @ 0.9 PF		
			282	Amps @ 13.2 kV		
			897	Amps @ 4160V		
			4658	Utility kVA		
			204	Utility Amps @ 13.2 kV		

London Canal – Option 1						
Pump	**Driver**	**Motor hp**	**Motor hp**	**Source**	**Utilization**	
cfs	**bhp**	**Nameplate**	**Load**		**Grid**	**Ind**
1000	1052	2750	1800	Genset		1%
1000	1052	2750	1800	Genset		1%
1000	1052	2750	1800	Genset		1%
1000	1052	2750	1800	Genset		1%
1000	1052	2750	1800	Genset		1%
1000	1052	2750	1800	Genset		1%
1000	1052	2750	1800	Genset		1%
1000	1052	2750	1800	Grid/Gen	1%	1%
500	526	1500	900	Grid/Gen	10%	1%
250	263	750	450	Grid/Gen	50%	1%
250	263	750	450	Grid/Gen	100%	1%
9000	**9468**		**16200**	**Total hp (1hp=1kVA)**		
			810	Bldg Load (kVA) 5% of motor load		
			17010	Total Load (kVA)		
			15.3	MW @ 0.9 PF		
			744	Amps @ 13.2 kV		
			2363	Amps @ 4160V		
			6210	Utility kVA		
			272	Utility Amps @ 13.2 kV		

The following table is a summary total load requirement of the Option 1 pump stations to be supplied by the Central Plant. Based on a load of 42.4 MW, a 50 MW plant is recommended.

Totals for the Central Plant – 50 MW (Option 1)	
44850	Total motor hp (1hp=1kVA)
2243	Building Loads (kVA)
47093	Total Load (kVA)
42.4	**MW @ 0.9 PF**
2060	Amps @ 13.2 kV
16493	Utility kVA
721	Utility Amps @ 13.2 kV

Because of the size of the plant, two readily apparent options for a 50 MW Central Plant include using standard 2500 kW standby diesel generators and 15 MW dual fuel (natural gas and diesel) turbine generator sets were evaluated and presented below. Although various plant sizes are available, sizes were selected either based on maximum size available or a size that is the best fit to provide firm capacity. Note that the available power for the 15 MW gas turbines is derated to approximately 13 MW at 86 degrees F. In addition, large diesel generators (greater than or equal to 5 MW) could be considered in the next step.

2500 kW Diesel Generators				**15 MW Gas Turbines**			
Item	**Qty**	**Unit Price**	**Total Price**	**Item**	**Qty**	**Unit Price**	**Total Price**
2500 kW Diesel Gen.	20	$850,000	$17.0 M	15 MW Gas Turbine	5	$7.5 M	$37.5 M
Building	32K SF	$750/SF	$24.0 M	Building	14K SF	$750/SF	$10.5 M
U/G Ductbank	7 miles	---	$5.9 M	U/G Ductbank	7 miles	---	$5.9 M
Pump Bldg Substations	8	---	$0.9 M	Pump Bldg Substations	8	---	$0.9 M
Subtotal			$47.8 M	Subtotal			$54.8 M
Gen. req. & contingency		40%	$19.1 M	Gen. req. & contingency		40%	$21.9 M
Total			**$66.9 M**	**Total**			**$76.7 M**

Option 2 Evaluation

Load calculations for the Option 2 Central Plant are shown below based on anticipated motor sizes and assumed diversity. The load calculation assumptions include 1 hp = 1 kVA and the pump building load is approximately 5% of the pump motor load.

17th Street Canal – Option 2						
Pump	**Driver**	**Motor hp**	**Motor hp**	**Source**	**Utilization**	
cfs	**bhp**	**Nameplate**	**Load**		**Grid**	**Ind**
1000	2719	4550	3825	Genset		1%
1000	2719	4550	3825	Genset		1%
1000	2719	4550	3825	Genset		1%
1000	2719	4550	3825	Genset		1%
1000	2719	4550	3825	Genset		1%
1000	2719	4550	3825	Genset		1%
1000	2719	4550	3825	Genset		1%
1000	2719	4550	3825	Genset		1%
1000	2719	4550	3825	Genset		1%
1000	2719	4550	3825	Genset		1%
1000	2719	4550	3825	Grid/Gen	1%	1%
500	1359	2250	1925	Grid/Gen	5%	1%
500	1359	2250	1925	Grid/Gen	10%	1%
250	680	1250	950	Grid/Gen	50%	1%
250	680	1250	950	Grid/Gen	100%	1%
12500	**33987**		**47825**	**Total hp (1hp=1kVA)**		
			2391	Bldg Load (kVA) 5% of motor load		
			50216	Total Load (kVA)		
			45.2	MW @ 0.9 PF		
			2196	Amps @ 13.2 kV		
			6974	Amps @ 4160V		
			11966	Utility kVA		
			523	Utility Amps @ 13.2 kV		

Orleans Canal – Option 2						
Pump	**Driver**	**Motor hp**	**Motor hp**	**Source**	**Utilization**	
cfs	**bhp**	**Nameplate**	**Load**		**Grid**	**Ind**
1000	2719	4550	3825	Genset		1%
1000	2719	4550	3825	Genset		1%
500	1359	2250	1925	Grid/Gen	1%	1%
500	1359	2250	1925	Grid/Gen	5%	1%
250	680	1250	950	Grid/Gen	50%	1%
150	408	1000	700	Grid/Gen	100%	1%
3400	**9244**		**13150**	**Total hp (1hp=1kVA)**		
			658	Bldg Load (kVA) 5% of motor load		
			13808	Total Load (kVA)		
			12.4	MW @ 0.9 PF		
			604	Amps @ 13.2 kV		
			1918	Amps @ 4160V		
			9983	Utility kVA		
			437	Utility Amps @ 13.2 kV		

London Canal – Option 2						
Pump	**Driver**	**Motor hp**	**Motor hp**	**Source**	**Utilization**	
cfs	**bhp**	**Nameplate**	**Load**		**Grid**	**Ind**
1000	2719	4550	3825	Genset		1%
1000	2719	4550	3825	Genset		1%
1000	2719	4550	3825	Genset		1%
1000	2719	4550	3825	Genset		1%
1000	2719	4550	3825	Genset		1%
1000	2719	4550	3825	Genset		1%
1000	2719	4550	3825	Genset		1%
1000	2719	4550	3825	Grid/Gen	1%	1%
500	1359	2250	1925	Grid/Gen	10%	1%
250	680	1250	950	Grid/Gen	50%	1%
250	680	1250	950	Grid/Gen	100%	1%
9000	**24471**		**34425**	**Total hp (1hp=1kVA)**		
			1721	Bldg Load (kVA) 5% of motor load		
			36146	Total Load (kVA)		
			32.5	MW @ 0.9 PF		
			1581	Amps @ 13.2 kV		
			5020	Amps @ 4160V		
			13196	Utility kVA		
			577	Utility Amps @ 13.2 kV		

The following table is a summary total load requirement of the Option 1 pump stations to be supplied by the Central Plant. Based on a load of 90.2 MW, a 100 MW plant is recommended.

Totals for the Central Plant – 100 MW (Option 2)	
95400	Total motor hp (1hp=1kVA)
4770	Building Loads (kVA)
100170	Total Load (kVA)
90.2	**MW @ 0.9 PF**
4381	Amps @ 13.2 kV
13913	Utility kVA
609	Utility Amps @ 13.2 kV

An option for a 100 MW Central Plant includes using 15 MW dual fuel (natural gas and diesel) turbine generator sets as shown below. Due to the large generating capacity required and considering the largest current diesel generator set capacity is less than 5MW, diesel generator were not considered viable for this option. Larger diesel generators (greater than or equal to 5 MW) are currently being developed and should be considered in the follow-on central plant evaluation.

15 MW Gas Turbines			
Item	**Qty**	**Unit Price**	**Total Price**
15 MW Gas Turbine	9	$7,500,000	$67.5 M
Building	22.4K SF	$750/SF	$16.8 M
U/G Ductbank	7 miles	---	$9.3 M
Pump Bldg Substations	12	---	$1.5 M
Subtotal			$95.1 M
Gen. req. & contingency		40%	$38.0 M
Total			**$133.1 M**

Additional Considerations

Additional considerations that should be evaluated in the follow-on evaluation:

- Location for Central Plant
- Route for underground ductbanks from Central Plant to pump stations
- Real estate acquisition requirements
- Feasibility of connecting to the local power grid and selling commercial power
- Air quality permit requirements
- Reliability of natural gas for fuel source
- On-site fuel storage requirements

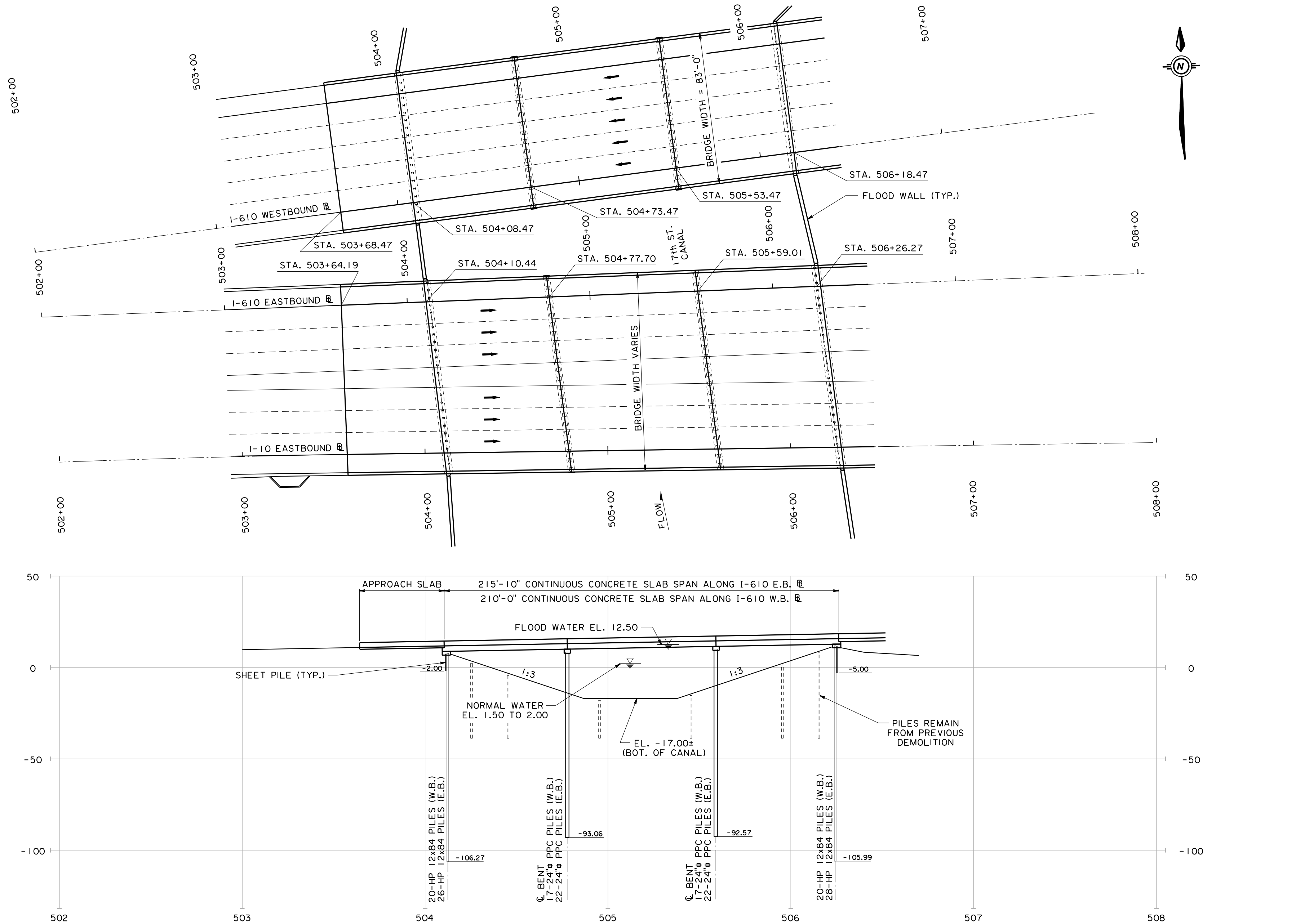
Conclusions

The next step to determine if a Central Plant is preferred over local standby generation power at each pump station is to decide if the Central Plant will be standby only, or if the power can be sold for commercial use and some return of investment on the Central Plant can be obtained. The outcome of this decision will direct the next step. If the decision is to provide standby power for the pump stations only, then a cost comparison can be made of using a Central Plant with all-electric motors vs. a distributed system with a combination of electric-driven and engine-driven pumps. If the decision is to connect to the local power grid and sell power, then a Central Plant will provide standby power to the pump stations with dedicated feeders and local standby power generation will not be needed at the pump stations.

Appendix F

BRIDGES

I-10 AT 17th STREET CANAL

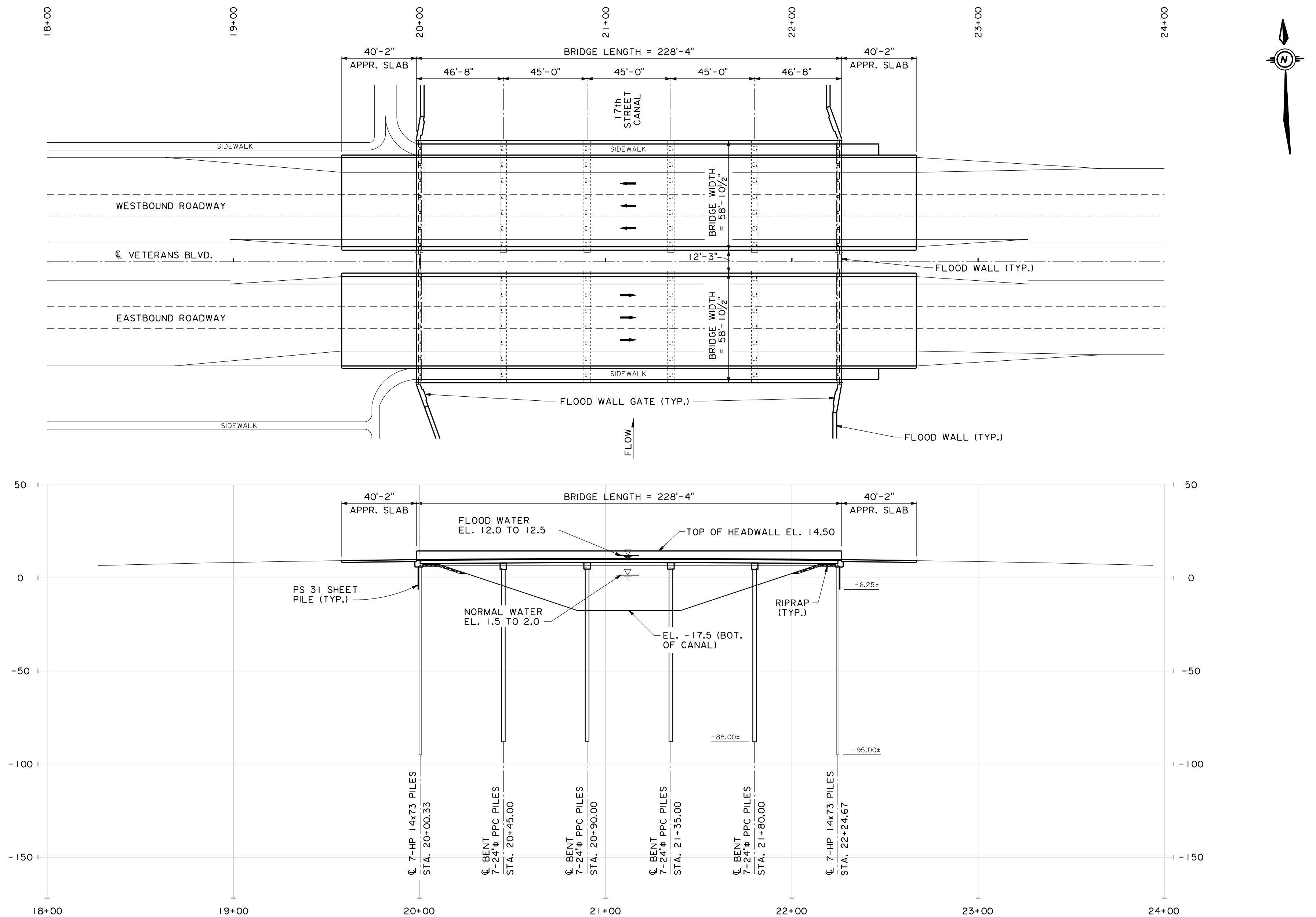


GEC Gulf Engineers & Consultants

BLACK & VEATCH Corporation

EXHIBIT 5.1.2.2A

VETERANS BOULEVARD AT 17th STREET CANAL



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BLACK & VEATCH Corporation

EXHIBIT 5.1.2.2B

HAMMOND HIGHWAY AT 17th STREET CANAL

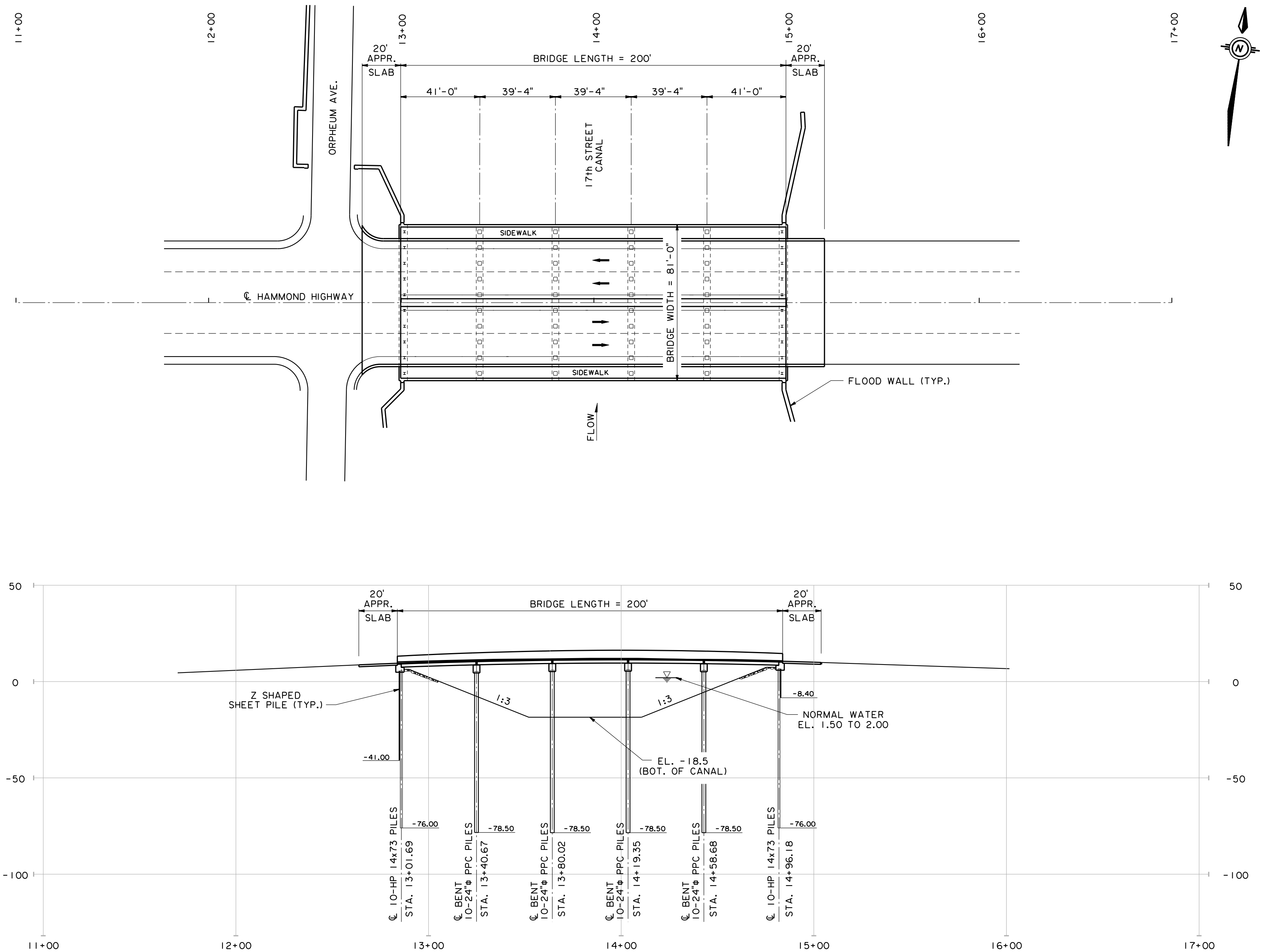

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EXHIBIT 5.1.2.2C

I-610 AT ORLEANS AVENUE CANAL

EXHIBIT
5.3.2.2A

GEC Gulf Engineers & Consultants

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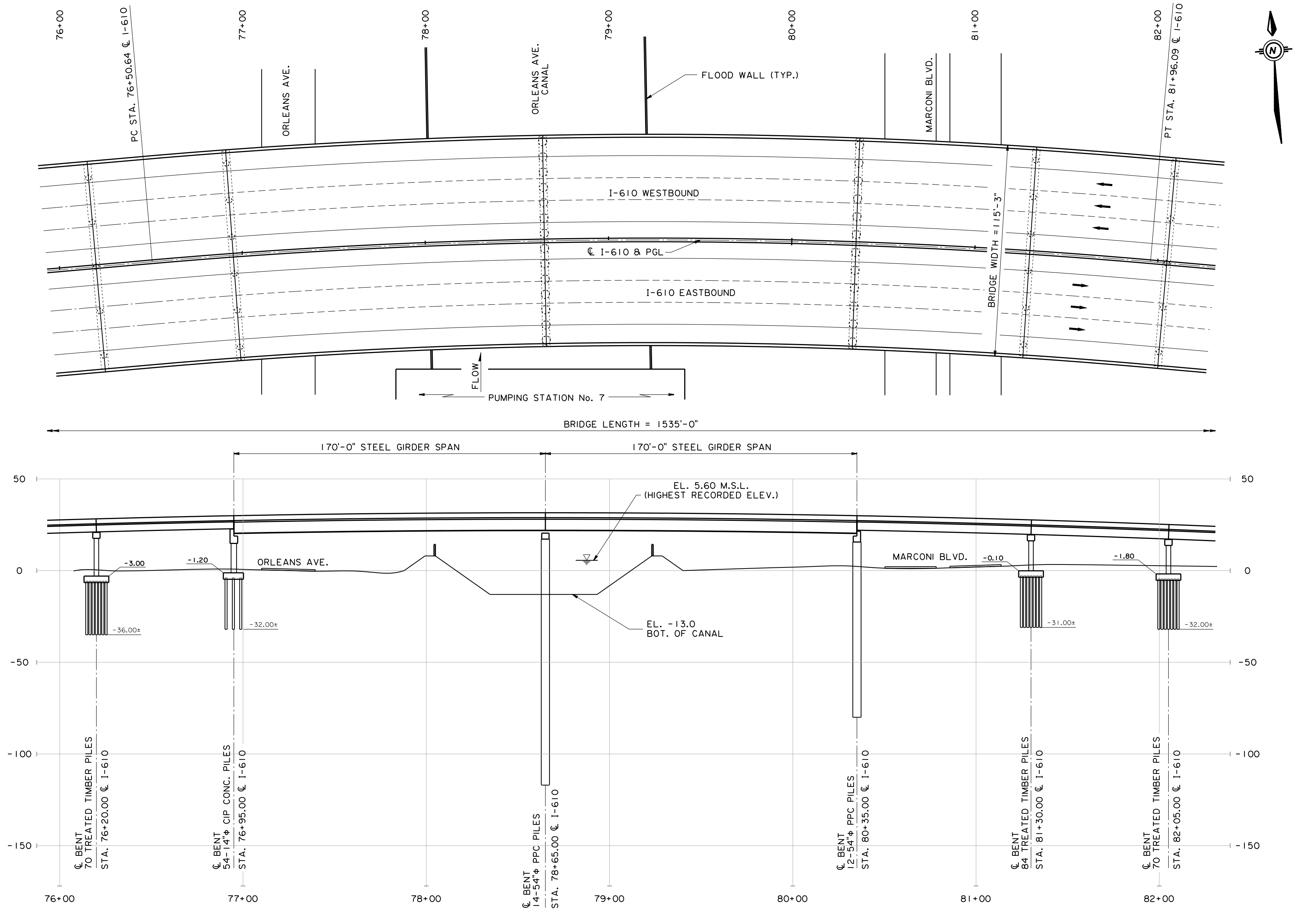

HARRISON AVE. AT ORLEANS AVE. CANAL

EXHIBIT 5.3.2.2B

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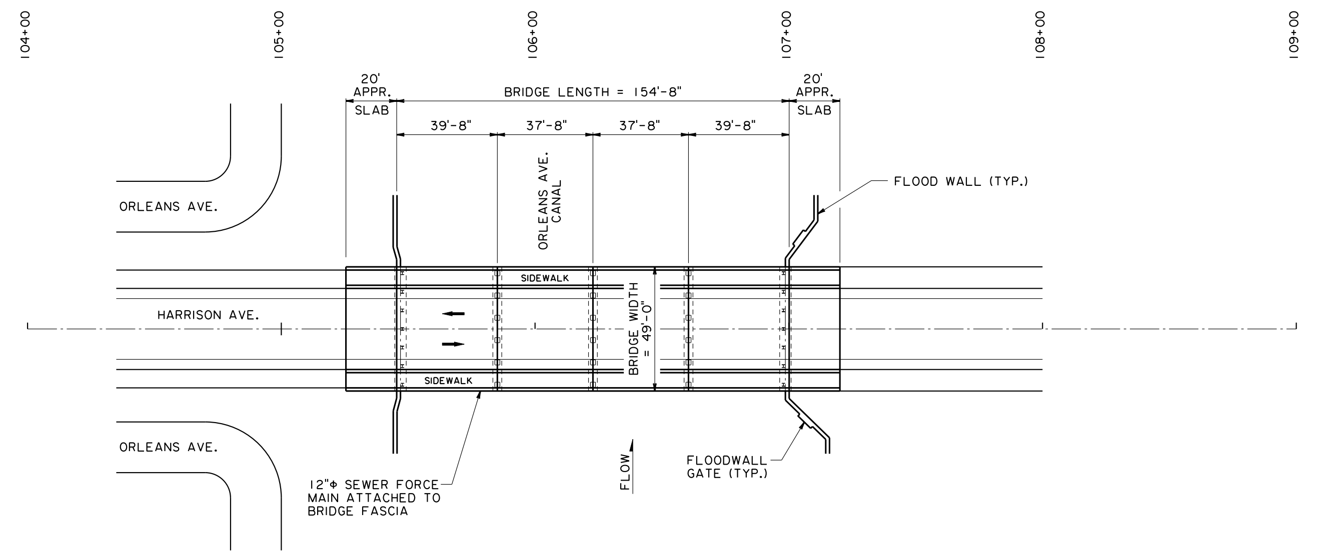

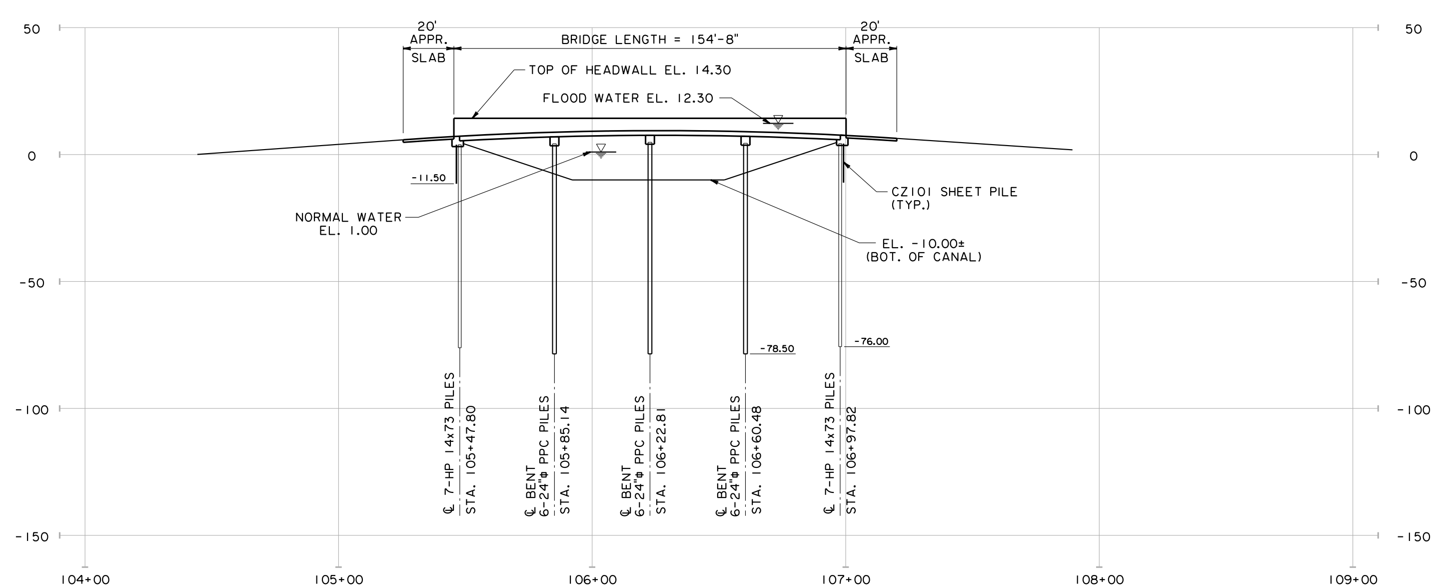

FILMORE AVE. AT ORLEANS AVE. CANAL

GEC Gulf Engineers & Consultants

BLACK & VEATCH Corporation

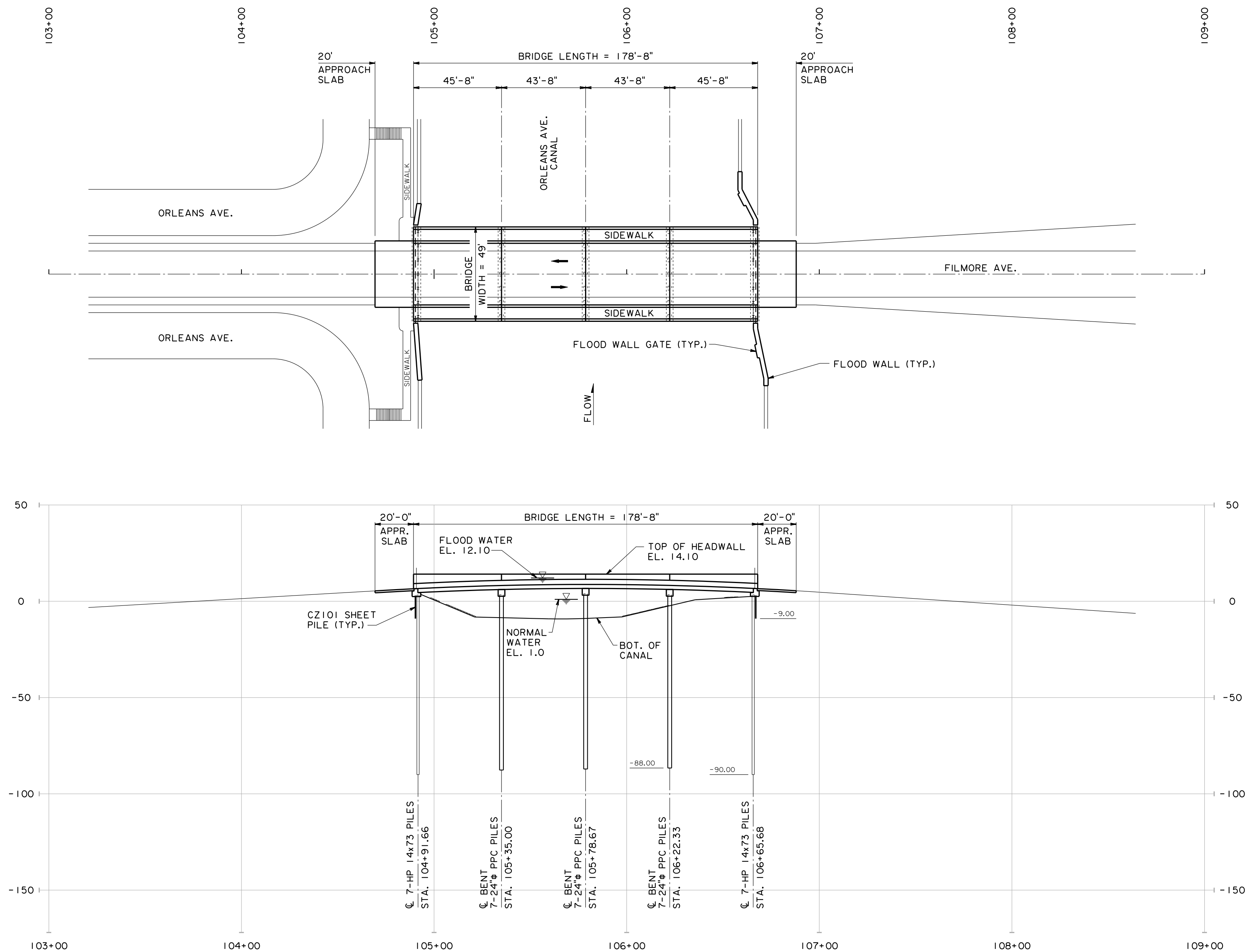

EXHIBIT 5.3.2.2C

ROBERT E. LEE BLVD. AT ORLEANS AVE. CANAL

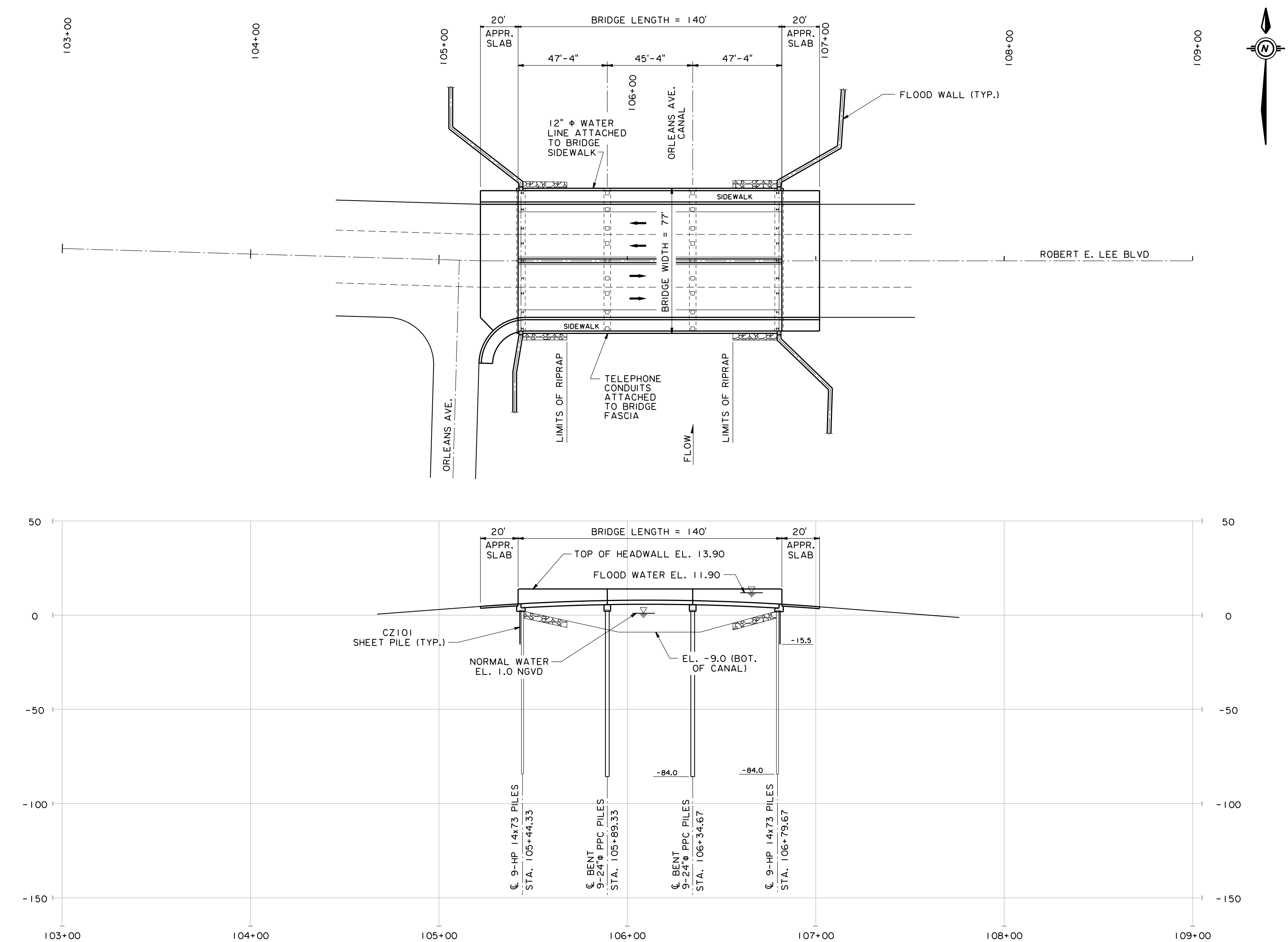

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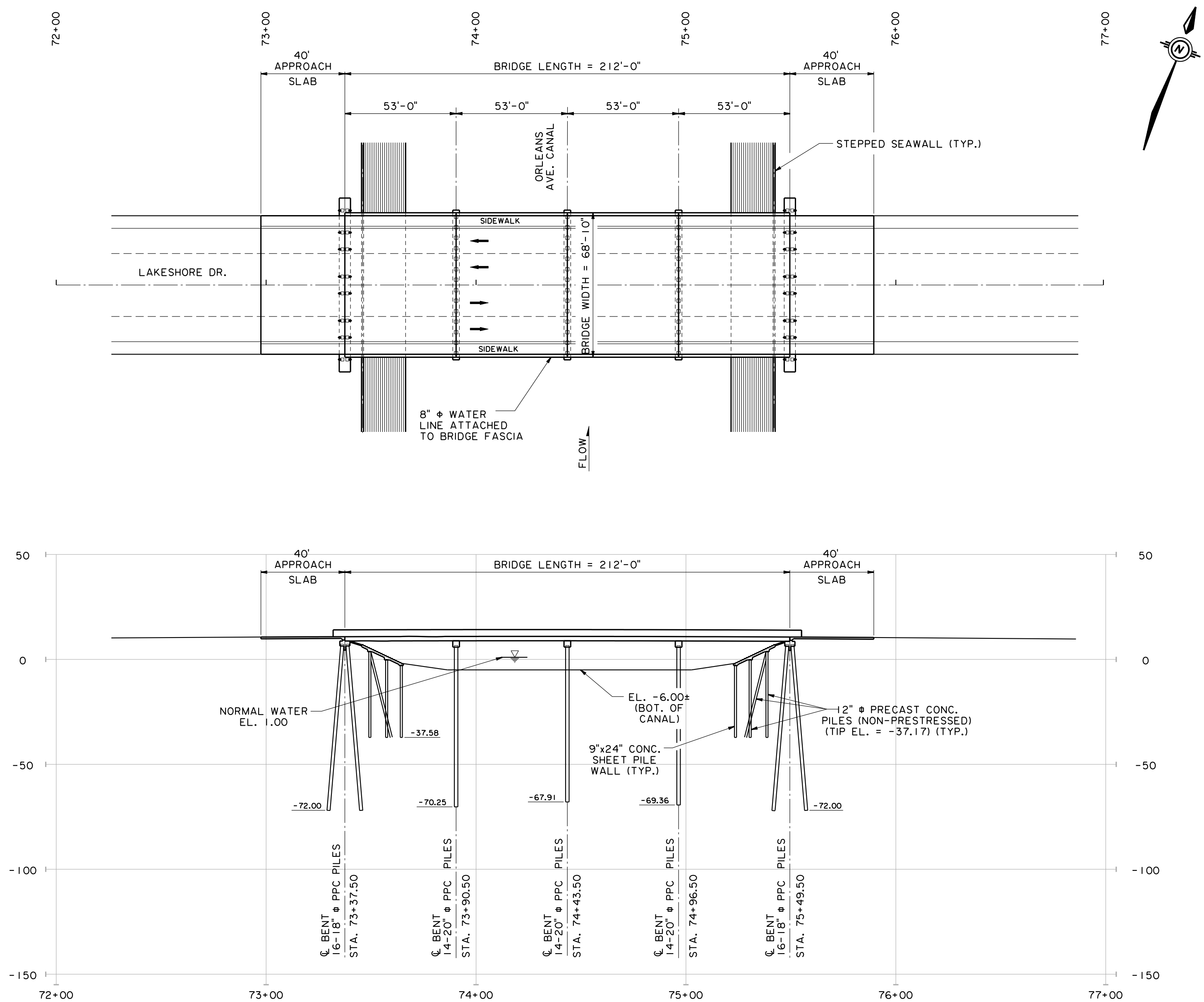
EXHIBIT 5.3.2.2D

LAKESHORE DR. AT ORLEANS AVE. CANAL



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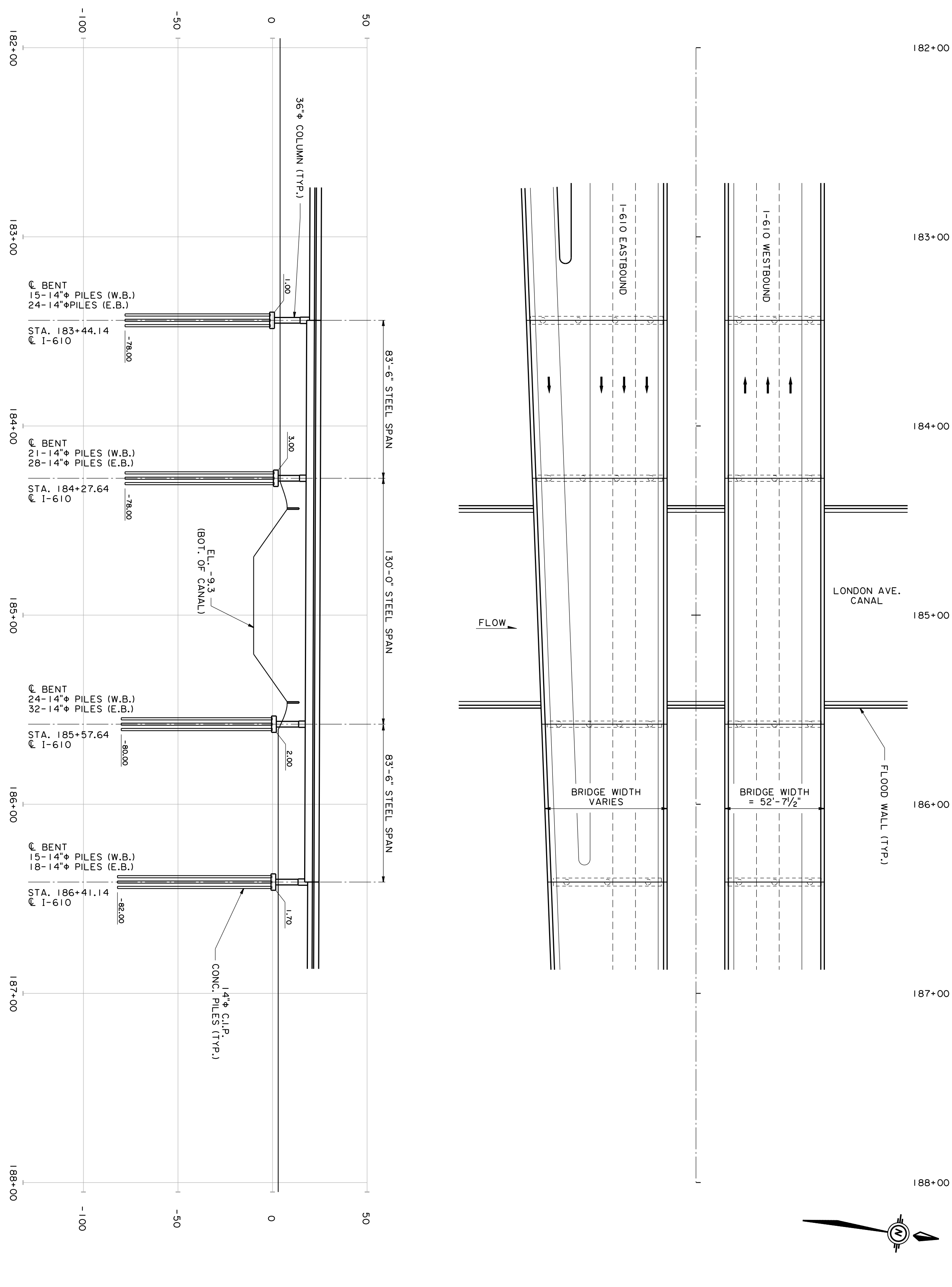

I-610 AT LONDON AVENUE CANAL

EXHIBIT
5.5.2.2A

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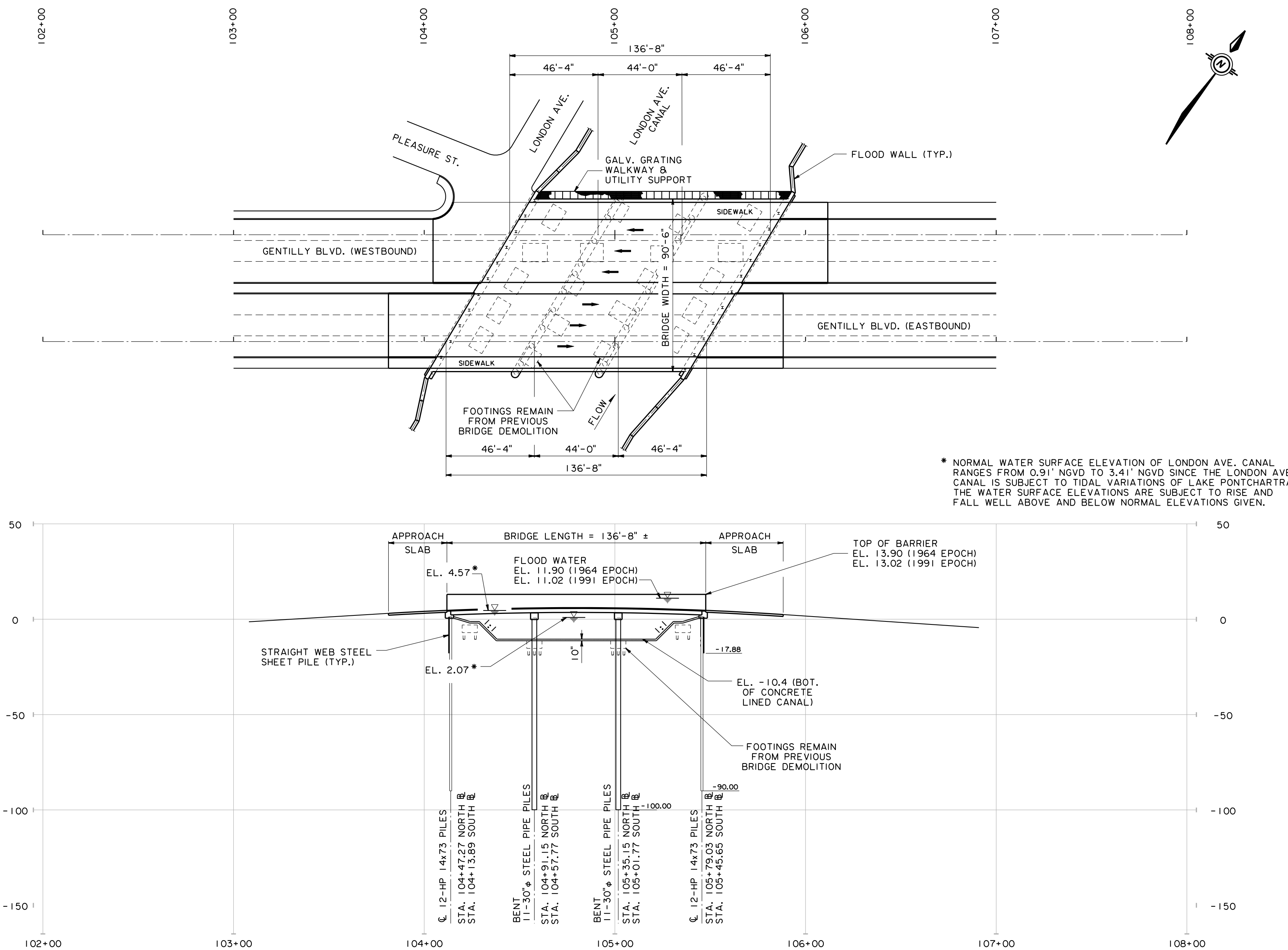
GENTILLY BLVD. AT LONDON AVE. CANAL



* NORMAL WATER SURFACE ELEVATION OF LONDON AVE. CANAL RANGES FROM 0.91' NGVD TO 3.41' NGVD SINCE THE LONDON AVE. CANAL IS SUBJECT TO TIDAL VARIATIONS OF LAKE PONTCHARTRAIN, THE WATER SURFACE ELEVATIONS ARE SUBJECT TO RISE AND FALL WELL ABOVE AND BELOW NORMAL ELEVATIONS GIVEN.

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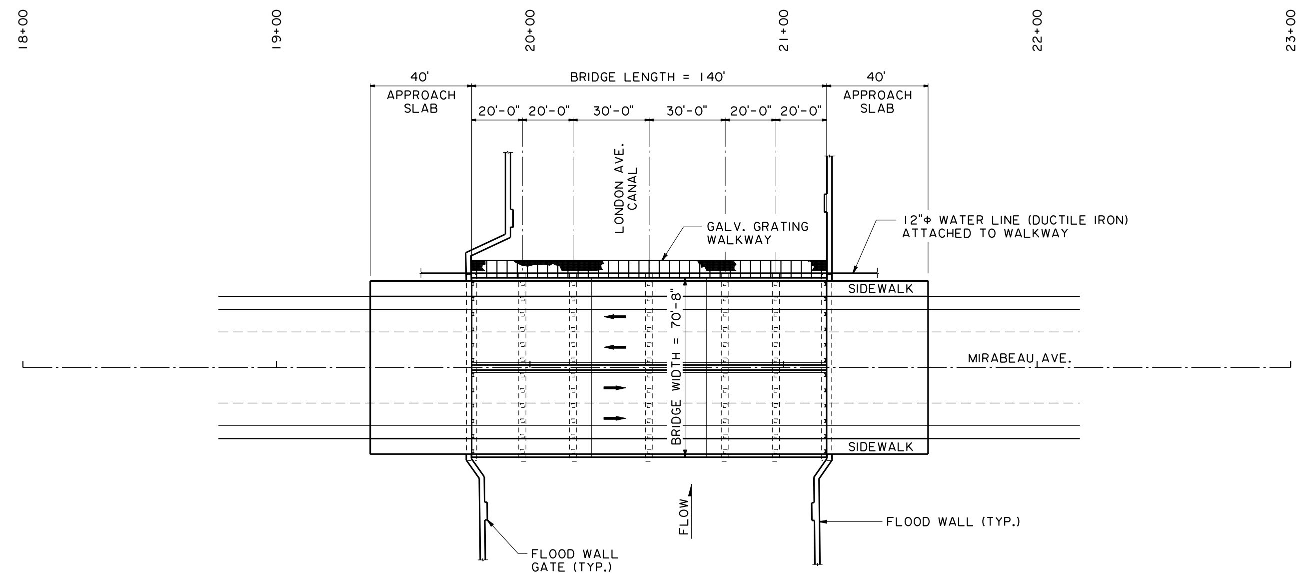
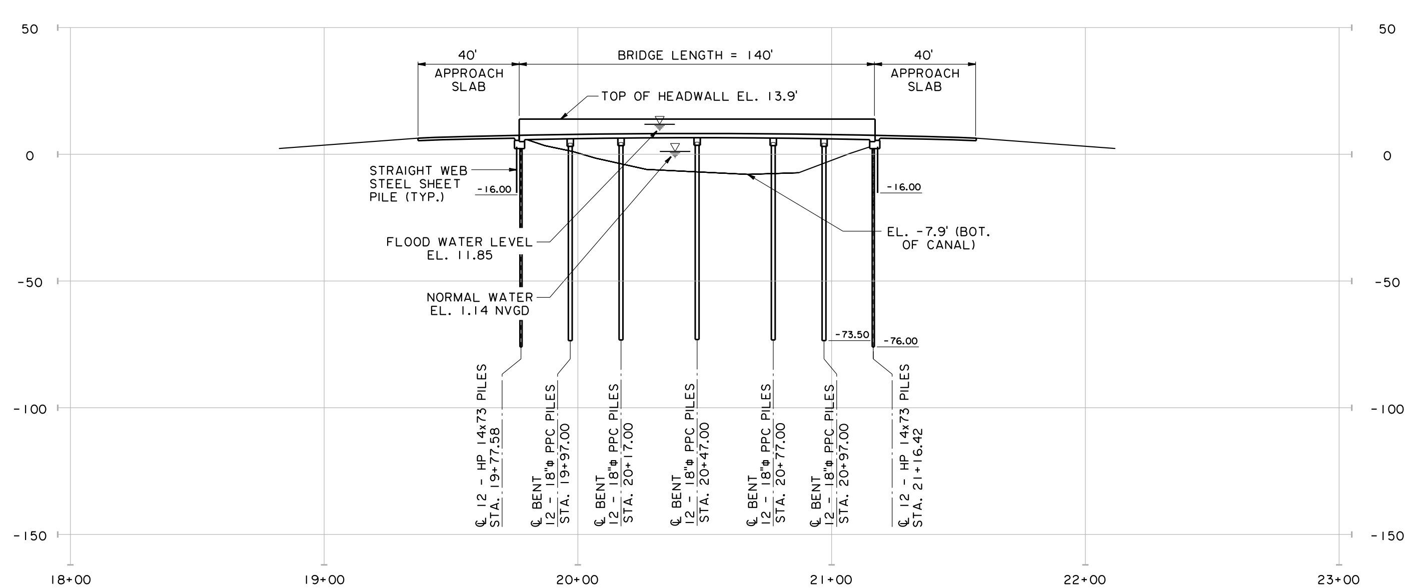
EXHIBIT 5.5.2.2B





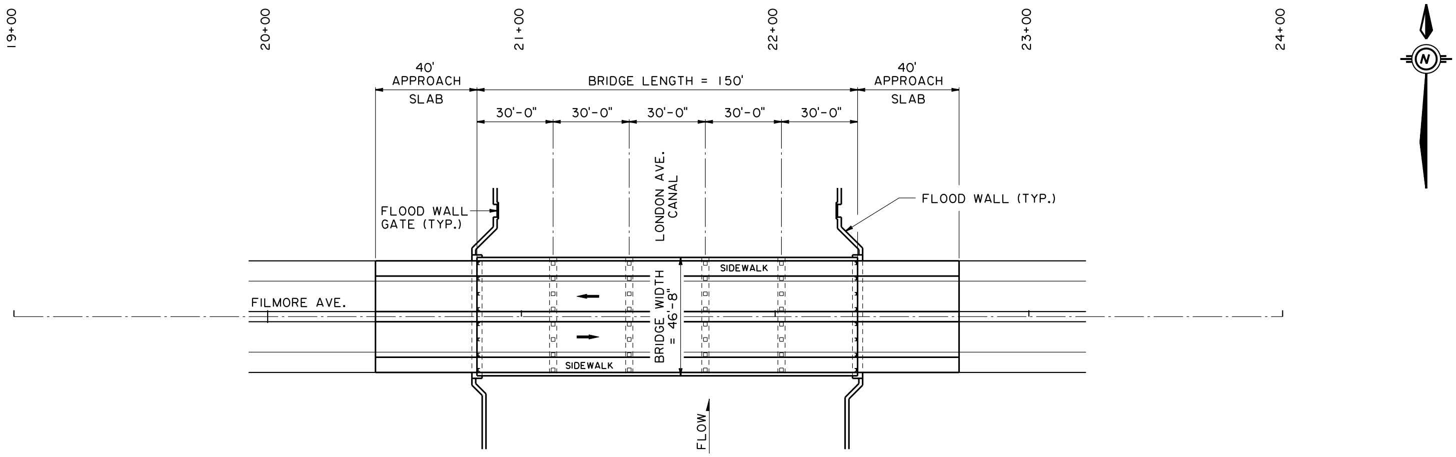
MIRABEAU AVE. AT LONDON AVE. CANAL

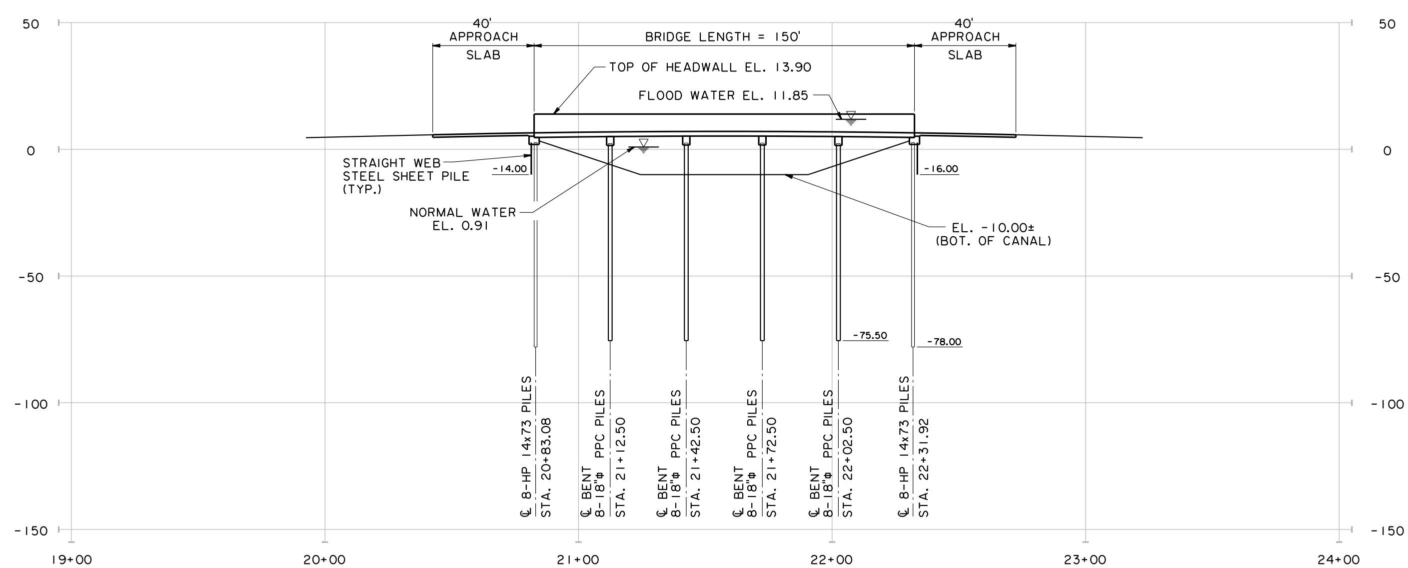
GEC Gulf Engineers & Consultants

BLACK & VEATCH Corporation

EXHIBIT 5.5.2.2C

FILMORE AVE. AT LONDON AVE. CANAL





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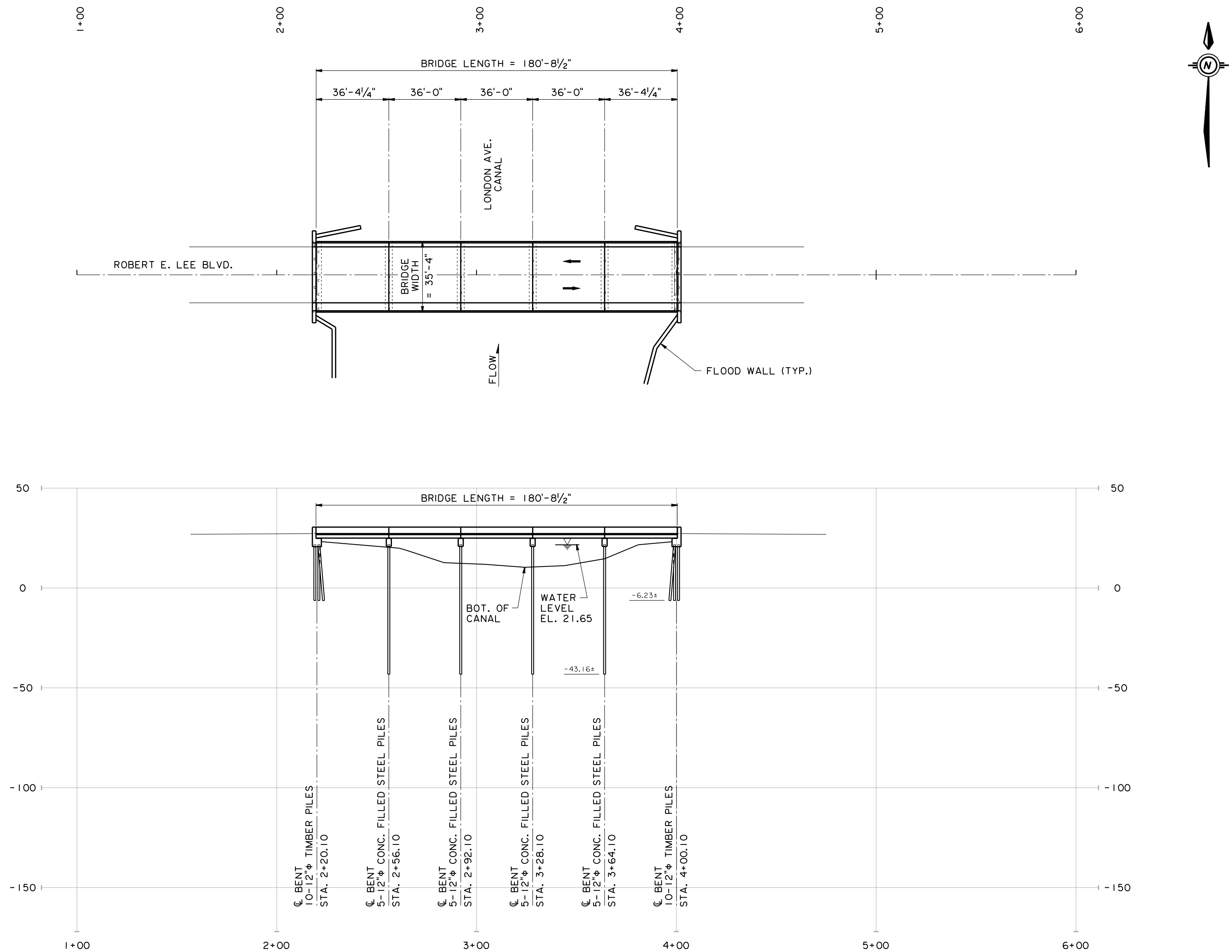

ROBERT E. LEE BLVD. AT LONDON AVE. CANAL

GEC Gulf Engineers & Consultants

BLACK & VEATCH Corporation

EXHIBIT 5.5.2.2E

LEON C. SIMON BLVD. BRIDGE AT LONDON AVE. CANAL

EXHIBIT 5.5.2.2F

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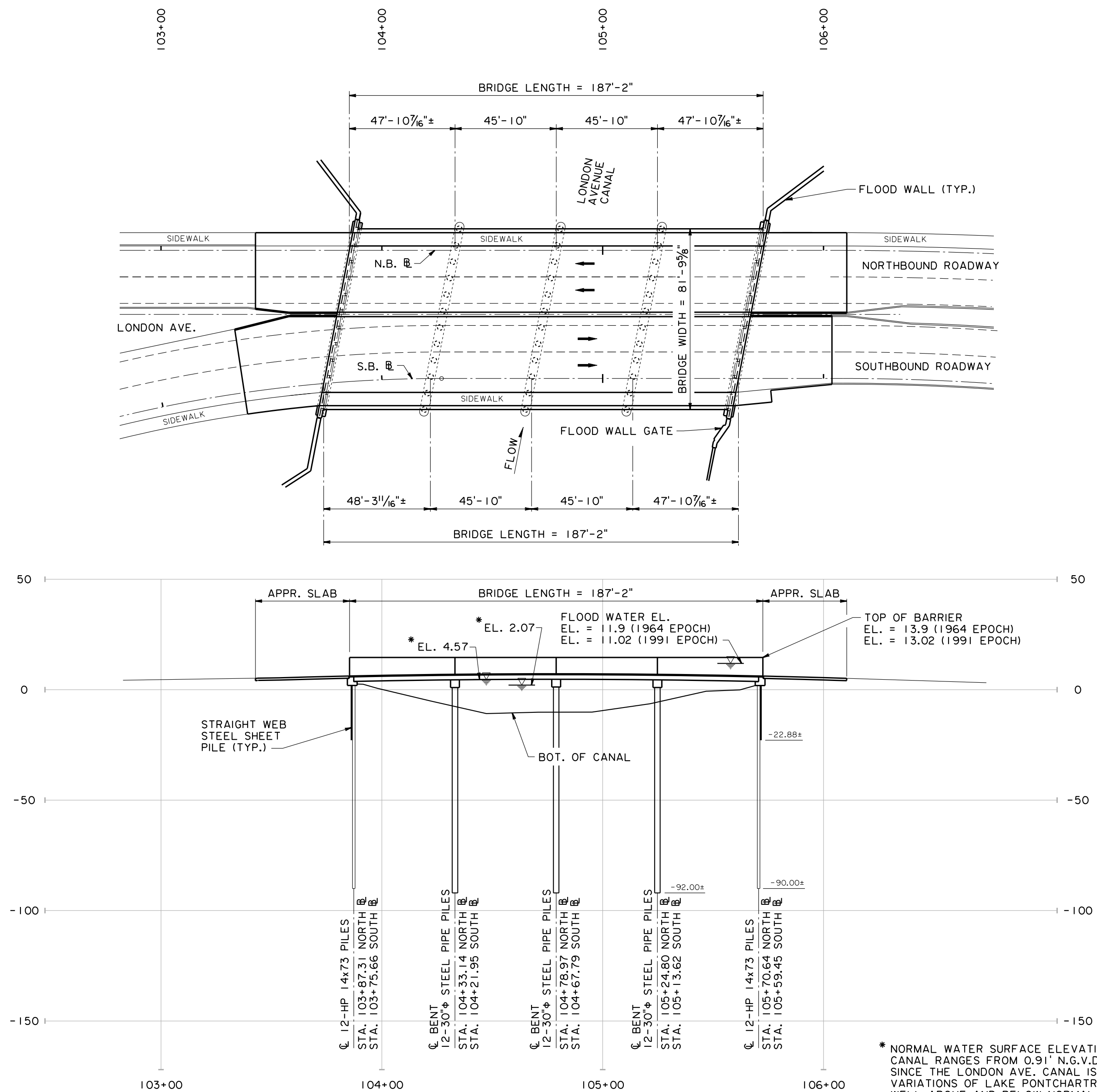
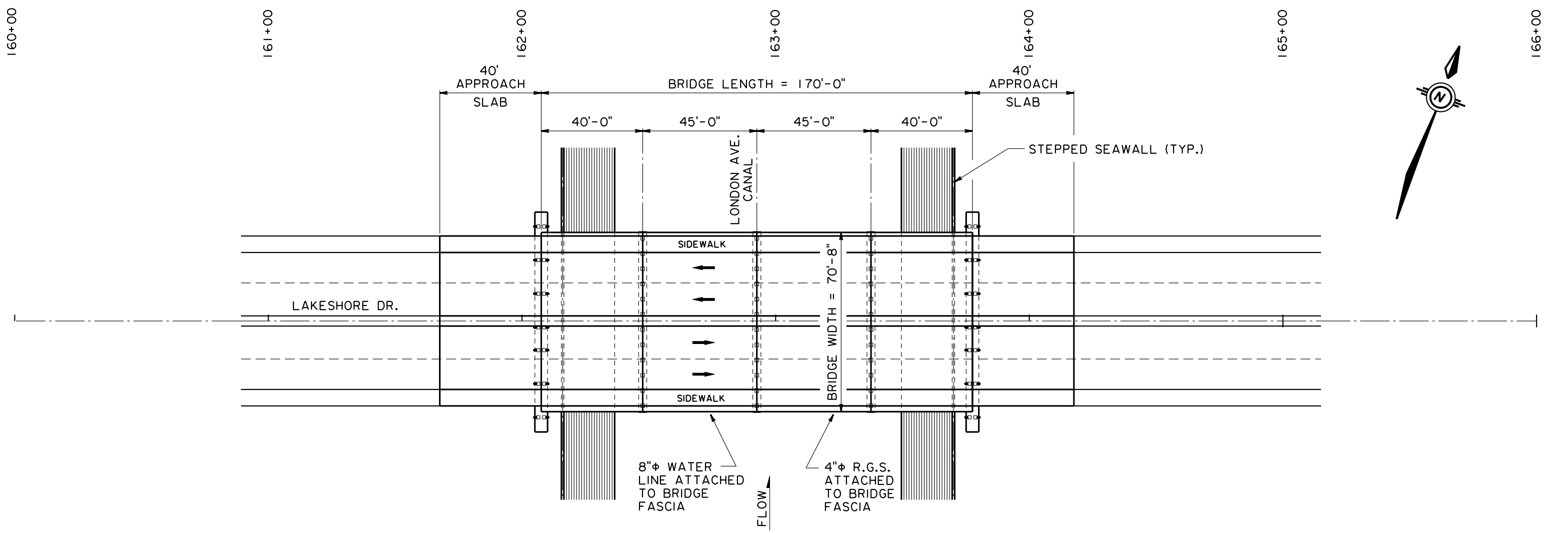


* NORMAL WATER SURFACE ELEVATION OF LONDON AVE. CANAL RANGES FROM 0.91' N.G.V.D. TO 3.41' N.G.V.D. SINCE THE LONDON AVE. CANAL IS SUBJECT TO TIDAL VARIATIONS OF LAKE PONTCHARTRAIN, TO RISE & FALL WELL ABOVE AND BELOW NORMAL ELEVATION GIVEN

LAKESHORE DR. AT LONDON AVE. CANAL



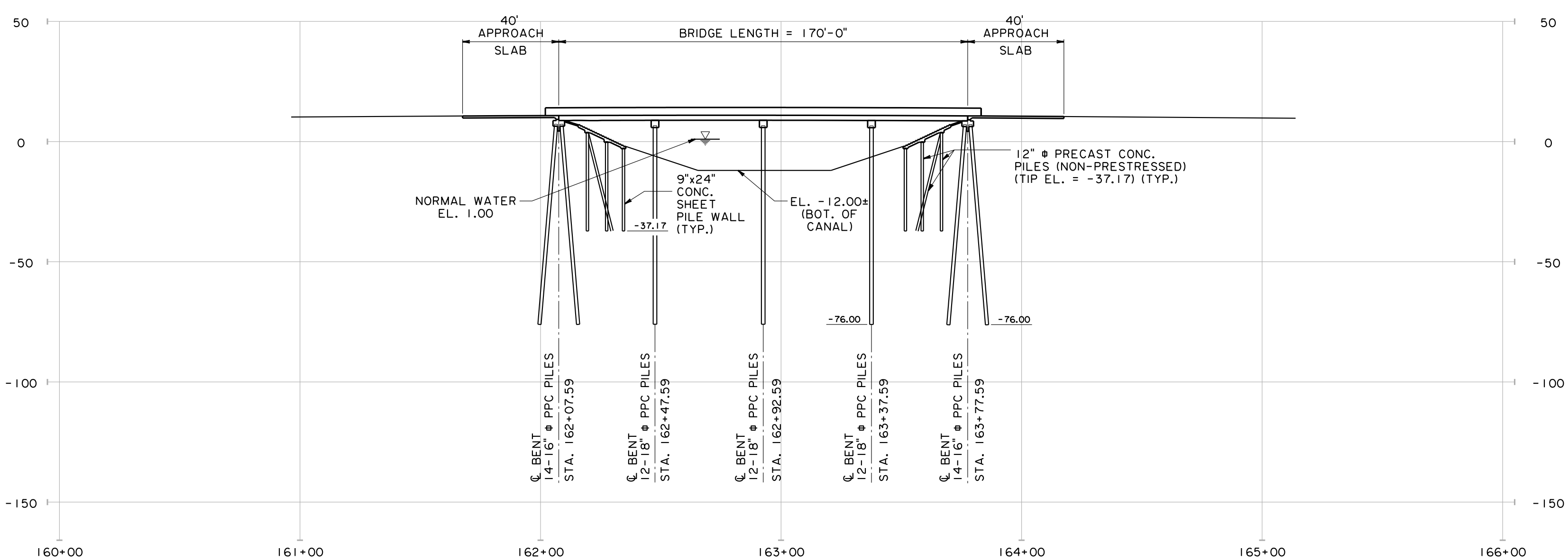


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EXHIBIT 5.5.2.2G

Appendix G

UTILITIES

Conceptual Design Services for Permanent Flood Gates
And Pump Stations Project
Utility Conflicts on the Three Canals
Relocation/Adjustment Cost Estimates
July 25, 2006

<u>17th Street Canal</u>

<u>Option 1</u>

- No utility relocation costs involved.

<u>Option 2 (from P.S. No. 6 to Lake Pontchartrain)</u>

- 12 Electric transmission poles (pile supported) are located within the canal between the railroad and Veterans Blvd. and must be relocated at an estimated cost of \$1,080,000.

<u>Orleans Avenue Canal</u>

<u>Option 1</u>

- In Alternative C, an 8" dia. water line, attached to the Lakeshore Dr. bridge deck, must be replaced along with the bridge. \$12,600

<u>Option 2 (from P.S. No. 7 to Lake Pontchartrain)</u>

- Bragg St. - 30" dia. water line crossing the canal at at water surface grade, w/ 3 pile supports in the canal. \$50,000

- Robert E. Lee Blvd. bridge (south side) –

  - BellSouth submarine duct bank w/ 144k & 36k fiber optic cables enclosed in steel conduit (attached to bridge deck). \$150,000

Orleans Avenue Canal, Option 2 (cont.)

- Robert E. Lee Blvd. bridge (north side) –

    - 12” dia. water line (attached to bridge deck) $10,400
    - 10’ x 7’ box drainage siphon $210,000
    - 24” dia. gravity sewer – (will require a new $300k sewer pump station & $112,000 for subaqueous gravity sewer)
    - 8” dia. high pressure gas line $56,000
    - Entergy submarine primary electric feed $37,500

- Lakeshore Dr. bridge –

    - 8” dia. water line (attached to bridge deck) (same as per Option 1C) $12,600

London Avenue Canal

Option 1

- In Alternative C, an 6” dia. water line, attached to the Lakeshore Dr. bridge deck, must be replaced along with the bridge. $4,800. Also, a 6” dia. submarine high pressure gas line located about 30’ south of this bridge must be replaced/relocated. $38,400

Option 2 (from P.S. No. 3 to Lake Pontchartrain)

- In the P.S. No. 3 discharge basin –

    - 48” dia. drainage pressure pipe crossing at water surface grade and supported by 14 pile supports in the canal. $100,000

- Mirabeau Avenue bridge –

    - 8” dia. submarine high pressure gas line crossing about 50’ south of the bridge. $44,800
    - 12” dia. water line (attached to the bridge deck) on the north side. $10,400

London Avenue Canal, Option 2 (cont.)

- Fillmore Avenue bridge (south side) –
  - 4” dia. submarine high pressure gas line crossing about 15’ from bridge. $40,000
  - 50” dia. water line crossing at water surface grade and supported by 4 piles located in the canal 10’ from bridge. $60,000

- Prentiss Avenue/Pump Station No. 4
  - BellSouth duct bank w/ 48k, 72k, and 144k fiber optic cables enclosed crossing at water surface grade. Duct bank supported by 2 piles in canal. $150,000
  - 120” dia. and 48” dia. drainage pressure pipes cross the canal at water surface grade on one set of 3 pile supports located in the canal (about 60’ north of the BellSouth duct bank). $100,000

- Robert E. Lee Blvd. bridge (north side) –
  - Entergy submarine crossing primary electric feed (approximately 10’ from bridge) $54,000

- Lakeshore Dr. bridge –
  - 6” dia. submarine crossing high pressure gas line (about 30’ south of the bridge) (same as per Option 1C) $38,400
  - 6” dia. water line (attached to bridge deck) (same as per Option 1C) $4,800

Summary of Conflicting Utilities Adjustment/Relocation Cost Estimates

Sub-total for 17th Street Canal -	Option 1: $0	Option 2: $1,080,000
Sub-total for Orleans Ave. Canal-	Option 1: $12,600	Option 2: $938,500
Sub-total for London Ave. Canal-	Option 1: $43,200	Option 2: $602,400
Sub-total for project -	Option 1: $55,800	Option 2: $2,620,900
20% Construction Contingencies-	Option 1: $11,160	Option 2: $524,180
TOTAL Construction Cost:	**Option 1: $66,960**	**Option 2: $3,145,080**

Appendix H

COST ESTIMATES

Appendix H contains proprietary information and has been redacted.