PUBLIC HEARINGS
H. ITEM 1.

AGENDA REPORT
August 14, 2018

Public Hearing to Consider Approval of Resolution for Adoption of State Revolving Loan Fund (SRF) #WW05113 Wastewater Facility Plan for the Riverside Drive Force Main Replacement project required by the FDEP

SUBJECT:
Public Hearing to Consider Approval of Resolution for Adoption of State Revolving Loan Fund (SRF) #WW05113 Wastewater Facility Plan for the Riverside Drive Force Main Replacement project required by the Florida Department of Environmental Protection. (District 5)

FISCAL IMPACT:
$7,835,040

DEPT/OFFICE:
Utility Services

REQUESTED ACTION:
It is requested that the Board of County Commissioners conduct a public hearing for approval of a resolution adopting State Revolving Fund #WW05113 Wastewater Facility Plan for the Riverside Drive Force Main Replacement project.

SUMMARY EXPLANATION and BACKGROUND:
The Riverside Force Main Replacement Project starts at Eau Gallie Boulevard and runs south along Riverside Drive to Oakland Avenue, for a distance of approximately 3.3 miles. The force main section is in need of replacement. This project will also add surge tanks at the B-19 and B-20 lift stations to further protect the system from pressure spikes and negative pressures. The project is included in the Utility Services adopted Capital Improvements Program.

The proposed force main replacement and surge tanks installation are necessary to eliminate sections of defective pipe which have been prone to failure over the last several years, and to eliminate sanitary sewer overflows which have occurred with prolonged rain events. The Florida Department of Environmental Protection (FDEP) issued a Consent Order in November 2016 which required Brevard County to identify the causes of sanitary sewer discharges and put a plan into place to address the discharge issues. This work plan meets the FDEP requirements.

The Utility Services Department has contracted HDR, Inc. Consulting Engineers to design the project and assist with the process of obtaining funding from the Florida Department of Environmental Protection managed Clean Water State Revolving Fund Program. This funding option affords the Department the opportunity to secure lower interest rates and substantial savings in the financing of the facility improvements and expansion. Recent SRF loans approved by the State have been financed at a rate of less than 1.5% compared with 3% to 4% for traditional bond financing.

Fiscal Impact: This project is expected to cost approximately $7,900,000 and is part of the department's Capital Improvements Program. The project will be budgeted in fund 4163, cost center 365391 when the SRF loan is approved.

Contact: Jim Helmer, Utility Services Director, 321-633-2091, jim.helmer@brevardfl.gov

CLERK TO THE BOARD INSTRUCTIONS:

ATTACHMENTS:

Description

- Resolution: Riverside Force Main
- Riverside Drive Forcemein Replacement Facility Plan

REVIEWERS:

Department	Reviewer
Utility Services	Helmer, Jim

BREVARD County
BOARD OF COUNTY COMMISSIONERS

FLORIDA'S SPACE COAST

Tammy Rowe, Clerk to the Board, 400 South Street • P.O. Box 999, Titusville, Florida 32781-0999

Telephone: (321) 637-2001
Fax: (321) 264-6972
Tammy.Rowe@brevardclerk.us

August 15, 2018

MEMORANDUM

TO: Jim Helmer, Utility Services Director

RE: Item H.1., Resolution for Adoption of State Revolving Fund (SRF) Fund #WW05113 Wastewater Facility Plan for the Riverside Drive Force Main Replacement Project Required by Florida Department of Environmental Protection (FDEP)

The Board of County Commissioners, in regular session on August 14, 2018, conducted a public hearing and adopted Resolution No. 18-105, adopting the State Revolving Fund (SRF) Fund #WW05113 Wastewater Facility Plan for the Riverside Drive Force Main Replacement Project required by Florida Department of Environmental Protection (FDEP). The public hearing process was used to explain the Project and present the financial impacts to the public. The Board unanimously approved the Resolution and there were no speakers or opposition to the Item. Enclosed is fully-executed and two certified copies of the Resolution for your action.

Your continued cooperation is greatly appreciated.

Sincerely yours,

BOARD OF COUNTY COMMISSIONERS
SCOTT ELLIS, CLERK

Tammy Rowe

Tammy Rowe, Deputy Clerk

/ds

Encls. (3)

cc: Finance
Budget

RESOLUTION NO. 2018-105

A RESOLUTION OF THE BOARD OF COUNTY COMMISSIONERS OF BREVARD COUNTY, FLORIDA, APPROVING THE WASTEWATER FACILITY PLAN (2018) AS REQUIRED BY THE FLORIDA DEPARTMENT OF ENVIRONMENTAL PROTECTION STATE REVOLVING FUND PROGRAM AND PROVIDING FOR AN EFFECTIVE DATE.

WHEREAS, local government agencies are allowed under the general laws of Florida to apply for public loans though the State of Florida in order to finance the construction of wastewater treatment facility expansion and improvements, and

WHEREAS, the administrative regulations of the Florida Department of Environmental Protection ("FDEP"), the state agency that oversees the State Revolving Fund Program, requires the Board of County Commissioners, as the local governing body of Brevard County, to formally approve a Wastewater Facility Plan describing the Riverside Drive Force Main Replacement project in order to comply with the funding requirements of the State of Florida; and

WHEREAS, formal approval of a Wastewater Facility Plan is required in order for Brevard County to participate in the State Revolving Fund Program; and

WHEREAS, the Florida Department of Environmental Protection requires a public hearing in conjunction with the adoption of the Wastewater Facility Plan;

WHEREAS, the Board of County Commissioners of Brevard County has held such public hearing on August 14, 2018; and

WHEREAS, there does not appear to be significant public objection to the Plan as presented at the public hearing; and

WHEREAS, the Board of County Commissioners of Brevard County, Florida, desires to formally approve the Wastewater Facility Plan (2018), a copy being attached hereto and incorporated herein by reference, and desires to make certain improvements to its wastewater system as more fully described in the said plan, and

WHEREAS, the Board of County Commissioners concurs with the findings and summary of necessary improvements that are described in the Facility Plan for the purpose of wastewater system improvements.

NOW, THEREFORE, BE IT RESOLVED BY THE BOARD OF COUNTY COMMISSIONERS OF BREVARD COUNTY, FLORIDA, THAT:

Section 1. The foregoing recitals are incorporated herein by reference.

Section 2. The Board of Commissioners of the Brevard County, Florida, is authorized to approve, and does hereby approve, the Wastewater Facility Plan (2018), a copy being attached hereto and incorporated herein by reference. The said Facility Plan is approved pursuant to the State Revolving Fund Program for the purpose of making various improvements to the County's wastewater facilities. A copy of the County's Facility Plan shall be maintained by the County Clerk.

Section 3. The County Manager is hereby authorized and directed to execute the said Facility Plan, including any and all papers and documents necessary and incidental thereto.

Section 4. The County Manager is further designated to be the County's representative who is authorized to provide the assurance and commitments that will be required by the said Facility Plan; and to represent the County in carrying out the County's responsibilities under the said Facility Plan, including the authority to delegate responsibility to appropriate County staff members to carry out the various technical, financial, and administrative activities associated with implementing the said Facility Plan.

Section 5. All Resolutions or part of Resolutions in conflict with any of the provisions of this Resolution are hereby repealed to the extent of such conflict. If any section or portion of a section of this Resolution proves to be invalid, unlawful, or unconstitutional, it shall not be held to invalidate or impair the validity, force, or effect or any other section or part of this Resolution.

Section 6. This Resolution is PASSED AND ADOPTED in Regular Session, this 14th day of August, 2018.

ATTEST:

Scott Ellis, Clerk

BOARD OF COUNTY COMMISSIONERS
OF BREVARD COUNTY, FLORIDA

Rita Pritchett, Chair
Brevard County Commission

(As approved by the Board on August 14, 2018)

RESOLUTION NO. 2018-

A RESOLUTION OF THE BOARD OF COUNTY COMMISSIONERS OF BREVARD COUNTY, FLORIDA, APPROVING THE WASTEWATER FACILITY PLAN (2018) AS REQUIRED BY THE FLORIDA DEPARTMENT OF ENVIRONMENTAL PROTECTION STATE REVOLVING FUND PROGRAM AND PROVIDING FOR AN EFFECTIVE DATE.

WHEREAS, local government agencies are allowed under the general laws of Florida to apply for public loans though the State of Florida in order to finance the construction of wastewater treatment facility expansion and improvements, and

WHEREAS, the administrative regulations of the Florida Department of Environmental Protection ("FDEP"), the state agency that oversees the State Revolving Fund Program, requires the Board of County Commissioners, as the local governing body of Brevard County, to formally approve a Wastewater Facility Plan describing the Riverside Drive Force Main Replacement project in order to comply with the funding requirements of the State of Florida; and

WHEREAS, formal approval of a Wastewater Facility Plan is required in order for Brevard County to participate in the State Revolving Fund Program; and

WHEREAS, the Florida Department of Environmental Protection requires a public hearing in conjunction with the adoption of the Wastewater Facility Plan;

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WHEREAS, there does not appear to be significant public objection to the Plan as presented at the public hearing; and

WHEREAS, the Board of County Commissioners of Brevard County, Florida, desires to formally approve the Wastewater Facility Plan (2018), a copy being attached hereto and incorporated herein by reference, and desires to make certain improvements to its wastewater system as more fully described in the said plan, and

WHEREAS, the Board of County Commissioners concurs with the findings and summary of necessary improvements that are described in the Facility Plan for the purpose of wastewater system improvements.

NOW, THEREFORE, BE IT RESOLVED BY THE BOARD OF COUNTY COMMISSIONERS OF BREVARD COUNTY, FLORIDA, THAT:

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Section 3. The County Manager is hereby authorized and directed to execute the said Facility Plan, including any and all papers and documents necessary and incidental thereto.

Section 4. The County Manager is further designated to be the County's representative who is authorized to provide the assurance and commitments that will be required by the said Facility Plan; and to represent the County in carrying out the County's responsibilities under the said Facility Plan, including the authority to delegate responsibility to appropriate County staff

H.1.

Brevard County Utility Services

WASTEWATER FACILITY PLAN

RIVERSIDE DRIVE FORCE MAIN REPLACEMENT
SRF# WW05113

Brevard
COUNTY

MARCH 2018

Prepared By:

HDR

HDR, Inc.
315 E Robinson Street, Suite 400
Orlando, FL 32801-1949
407-420-4200

HEATH E. HARDY
LICENSE
No 75444
3-21-18
STATE OF
PROFESSIONAL ENGINEER

Heath Hardy, P.E.
FL Registration No. 75444

Contents

Tables

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Appendices

1 Executive Summary

The Brevard County Utility Services Department (USD) provides wastewater service to both residential and commercial customers throughout the County's service area. The existing Riverside Drive force main includes approximately three miles of the county's primary transmission main to the South Beaches Regional Wastewater Treatment Plant.

The proposed force main replacement project is needed to eliminate the repeated force main breaks that have been occurring along this stretch of pipeline and to satisfy the current Florida Department of Environmental Protection (FDEP) consent order that has been issued against the county for these pipe breaks.

The force main replacement project will allow the county to resolve critical failures within their sewer transmission system and continue to provide reliable wastewater collection and treatment to the county's service area.

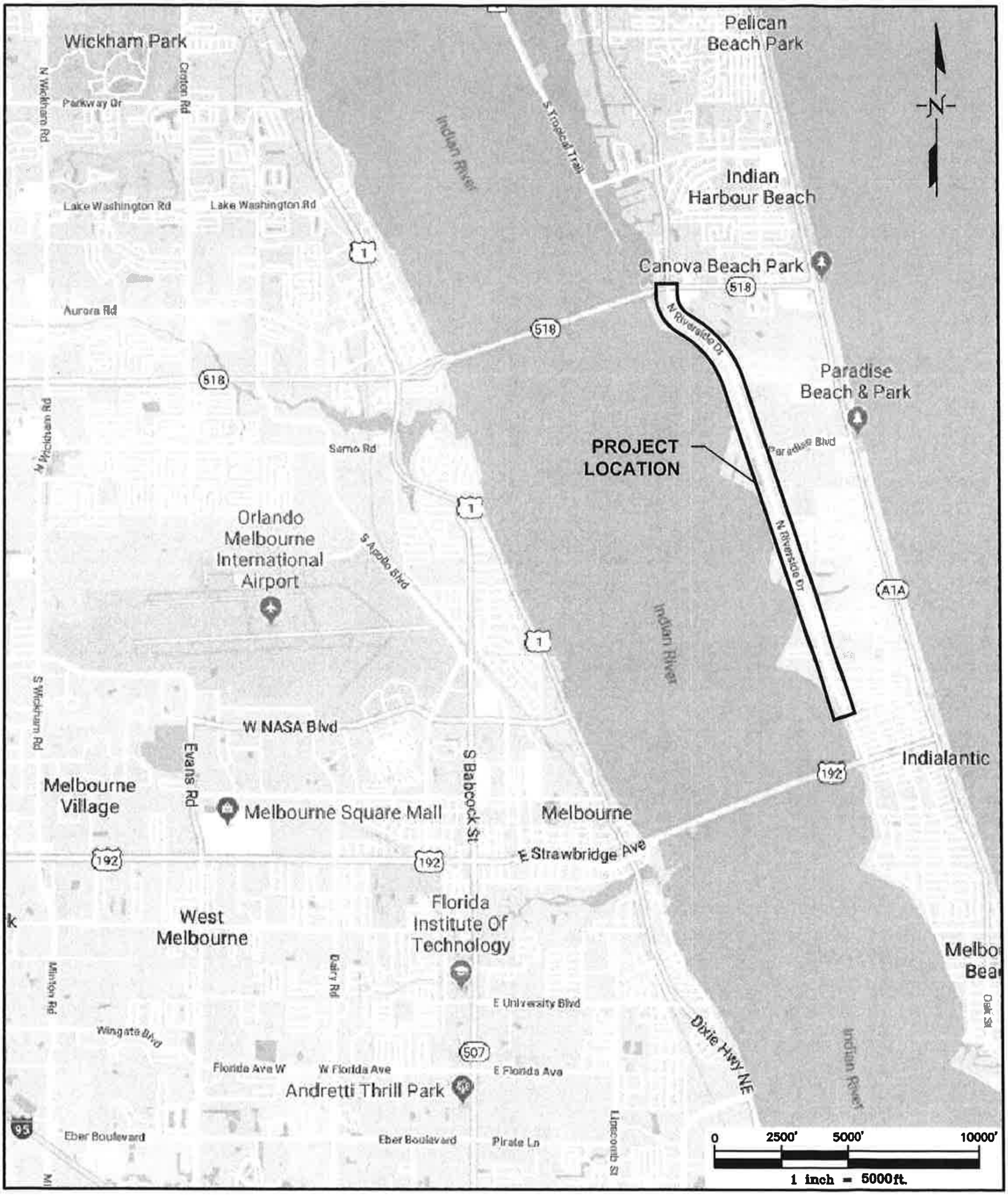
This document demonstrates that the construction and operation of the proposed improvements are in the public interest and will not cause adverse health and environmental impacts. Terrestrial and aquatic communities will not be negatively affected. Threatened or endangered species will be unaffected. This project will not impact any known archeologically sensitive areas. Construction for the new force main will be limited to FDOT and county right-of-way, and will follow the existing force main alignment to minimize any additional conflicts. Furthermore, there are no environmental justice concerns with this project.

The County is familiar with the requirements of the State Revolving Fund (SRF) Program, and has previously gone through the SRF process. They understand the program requirements and will fully comply with the planning and design submittal guidelines.

1.1 Project Description and Location

The County's South Beaches Force Main System consists of roughly 13 miles of 20" and 24" force main that provides collection and transmission from over forty lift stations throughout the South Patrick Shores and Melbourne area, and ultimately terminates at the South Beaches Regional Wastewater Treatment Plant. The force main system is comprised of a combination of ductile iron and polyvinyl-chloride (PVC) pipe. The two sections of PVC force main within this pipe system have been identified as needing replacement. The first segment of PVC force main replacement (1.0 mile long) is from DeSoto Parkway to Banana River Drive and is currently being replaced, with construction expected to be complete by June of 2018. The second area of force main replacement (3.3 miles long) will be the focus of this facility plan and is referred to as the Riverside Drive Force Main replacement project. The Riverside Drive force main segment provides service to roughly 44,000 residential and commercial customers throughout the area. **Figures 1-1** and **1-2** depict the overall location of the project, the existing system limits, and the proposed project layout.

The Riverside Drive Force Main Replacement project starts at Eau Gallie Boulevard and runs south along Riverside Drive all the way to the intersection of Oakland Avenue. The project will include the installation of roughly 3.3 miles of new ductile iron force main, isolation valves, air release valves, pipe fittings, reconnection of existing lift stations within the project area, and the removal/abandonment of the existing 24" PVC force main.



PROJECT TITLE RIVERSIDE DRIVE FORCE MAIN REPLACEMENT

SHEET TITLE
FIGURE:1-1
PROJECT LOCATION MAP

PROJECT NUMBER
10092206

PROJECT MANAGER
H.HARDY

DATE
03/2018

EXHIBIT SCALE
1"=5000'

EXHIBIT NUMBER
1-1

1.2 Justification for Project

The force main replacement project discussed within this document is needed for the following reasons listed below.

- To resolve the recurring pipe breaks in the existing force main
- To eliminate the resultant sewage spills related to pipe failures
- To satisfy the current FDEP consent order against the Brevard County USD

1.2.1 Force Main Breaks and Failures

The South Beaches Force Main System has experienced a significant number of pipe failures over the last few years. The force main failures documented have all been concentrated in the areas where the pipe material is PVC. Various studies have been done on these sections of piping to determine the cause of the pipe failures. **Table 1-1** shows a summary of the pipe failure by type that have occurred to date. The findings from the studies that have been done show that the pipeline was not installed properly due to over deflection of the joints, a lack of pipe restraints, and improper compaction of pipe backfill. In addition, a surge analysis study showed both negative system pressures and pressure surges in the system that would cause weak points in the system to fail. Due to the number of pipe failures and the findings from the force main studies, the complete replacement of the remaining PVC pipe from Eau Gallie Blvd to Oakland Avenue is recommended. **Figures 1-3** and **1-4** show the pipe failure locations within the limits of the Riverside Drive Force Main Replacement project.

Table 1-1: South Beaches Force Main Failures by Type

Type of Break	Occurrences
Bell Split	10
Leak	11
Joint Separation	2
Other	3

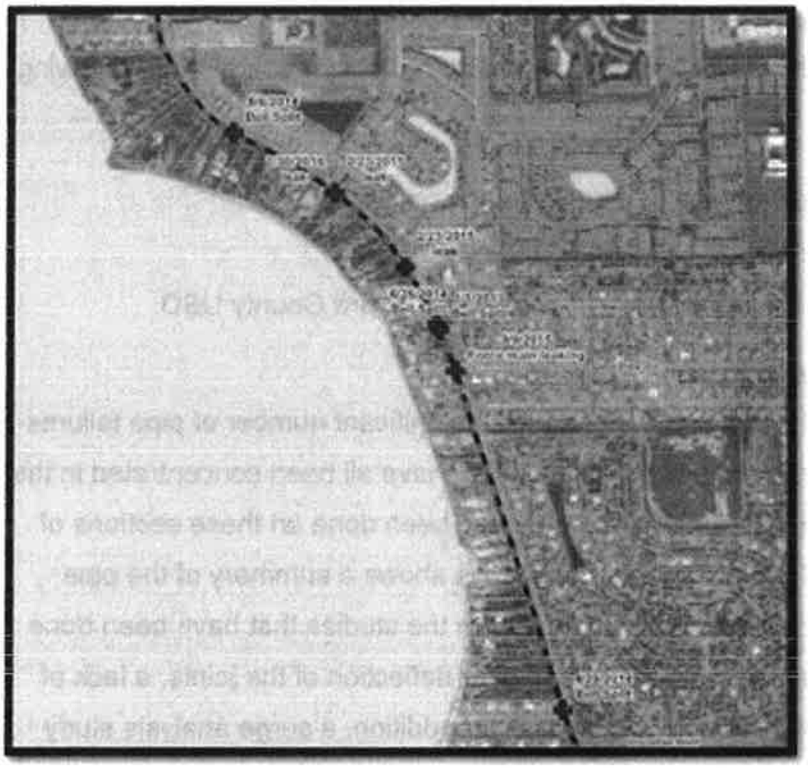


Figure 1-3: Force Main Breaks: Eau Gallie Blvd to Paradise Blvd

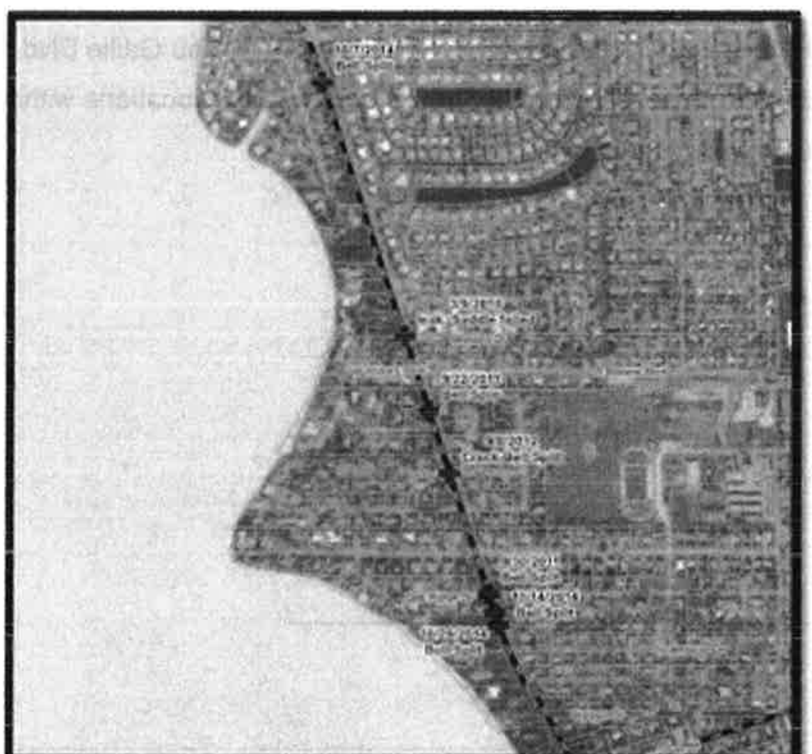


Figure 1-4: Force Main Breaks: Paradise Blvd to Oakland Avenue

1.2.2 Eliminate Sanitary Sewer Spills

The South Beaches Force Main system parallels thirteen miles along the Indian River Lagoon. The Indian River Lagoon is classified under the Florida Administrative Code (FAC) 62-302.700 as an Outstanding Florida Water (OFW) and the Brevard County USD is committed to affording the highest level of protection. Due to the proximity of this force main route to the Indian River Lagoon, any pipe failures resulting in a sanitary sewer overflow (SSO) would pose a direct impact to the lagoon. In addition to the close proximity to the lagoon, the force main system crosses multiple canals and navigable waterways that are directly connected to the Indian River Lagoon. Any pipe failures at these water crossings would result in a direct sewage discharge into the lagoon. By replacing the last section PVC force Main in the system along Riverside Drive, the USD would be able to greatly reduce the risk of future SSO's in the South Beaches area.

1.2.3 Satisfy FDEP Consent Order

In November 2016, the Florida Department of Environmental Protection issued a Consent Order related to sewage spills resulting from these force main breaks, which can be found in **Appendix A**. Included in the Consent Order is the requirement that Brevard County USD conduct a study to determine the cause of the failures within the force main system, and put a plan into place to address the issues. The USD completed a force main system study (**Appendix B**) and a surge analysis study (**Appendix C**), and as a result of the recommendations from these studies the USD determined that the defective pipeline should be replaced. Therefore, to satisfy the current FDEP consent order, the County is required to replace the defective pipe sections as per the recommendations of the South Beaches Force Main Study.

2 Alternatives Comparison

2.1 Force Main Replacement Alternatives

Due to the frequent pipe failures that have occurred over the last few years, the potential public and environmental risks due to SSO's, and the current FDEP consent order against the county, it is evident that action must be taken to address this issue. The following alternatives are evaluated herein:

- Point Repair Existing 24" PVC Force Main
- Complete Replacement with 24" Ductile Iron (DI) Force Main
- Complete Replacement with 30" Ductile Iron (DI) Force Main with Surge Tanks

A comparison of alternatives is based on project cost, system reliability, environmental impact, and future capacity/life. Each of the alternatives are evaluated and ranked form lowest to highest, 1 to 3. The points are totaled and the highest point total is the recommended alternative.

Lowest cost projects have the highest ranking. The most reliable options and the options with least environmental impacts also receive priority ranking. Finally, those options with the most benefits for future lifespan/capacity receive the highest ranking.

2.1.1 Point Repair Existing 24" PVC Force Main

The first alternative is that Brevard County USD can elect to leave the existing 24" PVC force main in place, and perform point repairs based on suspected problems within the system. This option would require the County to develop a list of repairs and improvements based on past pipe failure patterns, information provided from previous system evaluations/studies, and based on additional exploratory efforts. Improvements would include installing joint restraints on the existing force main, replacing pipe segments in areas with multiple failures, replacing pipe segments with excessive joint deflection or settlement, and replacing the segment of pipe under the canal. The primary benefits of this alternative is lower construction costs, and less impact/disruption due to limited pipe replacement. In **Table 2-1**, the conceptual cost estimate for this alternative is roughly $2.1 million. There are several significant disadvantages to this option, which include poor reliability, continued risk to the public/environment, unaddressed system pressure issues, and limited long term lifespan. Since the feasibility of locating and repairing every weak point in this system is not practical, the assumption would have to be made the pipe failures will continue to occur periodically into the foreseeable future. An assumption of two breaks per year, for a period of ten years, was made for the purposes of the cost estimate. In addition each pipe failure that occurs poses a potential risk to both the public health and the surrounding environment, including the Indian River Lagoon. Finally, the long-term lifespan of a pipe system that has been patched in multiple places is not sustainable, and the system would eventually need to be replaced.

Table 2-1: Conceptual Cost Estimate-Point Repair Existing 24" Force Main

ITEM 1 - GENERAL REQUIREMENTS					
1	Mobilization/Demobilization/General Conditions for Infrastructure Imponvements	1	LS	$30,000.00	$30,000
2	Maintenance of Traffic	1	LS	$30,000.00	$30,000
3	Bonds and Insurance Requirements	1	LS	$5,000.00	$5,000
4	Force Main Locating Services	1	LS	$10,000.00	$10,000
5	Professional Videotaping of Site Conditions (Pre/Post Construction)	1	LS	$5,000.00	$5,000
6	Contingency	(20% of Item 2-General Sitework Subtotal)			$339,950
				SUBTOTAL	***$419,950***
ITEM 2 - GENERAL SITEWORK					
7	Remove/Replace 24" PVC Force Main	2655	LF	$150.00	$398,250
8	Post Restrain 24" PVC Force Main	100	EA	$800.00	$80,000
9	Replace 24" Canal Crossing (HDD)	1	LS	$85,000.00	$85,000
10	Utility Fitting - Ductile Iron Elbow - 24"	15	EA	$7,000.00	$105,000
11	Point Repairs (2 per year, for 10 Years)	20	EA	$40,000.00	$800,000
12	Remove and Replace Concrete Sidewalk	1000	SF	$25.00	$25,000
13	Base and Asphalt Trench Repair	800	SY	$50.00	$40,000
14	Asphalt Milling & Overlay	1000	SY	$15.00	$15,000
15	Remove and Replace Concrete Driveway Apron	500	SY	$60.00	$30,000
16	Pavement Striping	1	LS	$5,000.00	$5,000
17	Remove and replace Unsuitable Materials	300	CY	$30.00	$9,000
18	Seeding	2500	SY	$2.00	$5,000
19	Sodding	2500	SY	$5.00	$12,500
20	Landscaping	1	LS	$10,000.00	$10,000
21	Pressure Testing	1	LS	$5,000.00	$5,000
22	Density Testing	1	LS	$25,000.00	$25,000
23	Dewatering	1	LS	$50,000.00	$50,000
				CONSTRUCTION SUBTOTAL	***$1,699,750***
				TOTAL ESTIMATED COST	***$2,119,700***
*Assume 15% of Existing 24" Force Main Needs Replacement.				**LOW (-15%)**	**$1,801,745**
*Assume (2) Point Repairs Required per Year, for 10 Year Period.				**HIGH (+20%)**	**$2,543,640**

2.1.2 Complete Replacement: 24" Ductile Iron Force Main

The second alternative evaluated for this facility plan is the complete replacement of the existing 24" PVC force main with a new 24" ductile iron force main. The primary benefit of this alternative is that all remaining PVC pipe within the South Beaches Force Main system would be replaced with ductile iron. Replacement of the existing 24" PVC force main would eliminate all issues identified in the previous study with regard to improper pipe installation, joint restraints, and compaction. The ductile iron force main would provide a much higher level of protection against future breaks, and therefore remove the public health/environmental impacts. In addition, this alternative would likely extend the life of the pipe system for the next 50+ years, conservatively. The disadvantages of this alternative include construction cost, site construction impacts, and unaddressed issues with pressures in the system. The conceptual cost estimate for this alternative is $6.3 million, as seen in **Table 2-2**, is more expensive that the point repair alternative due to the extent of construction required to replace the entire 3.3 miles of pipeline. The replacement of the entire pipeline would also have more of a construction impact to the community and local residents. Another drawback to this alternative is that it does not address the negative system pressures and pressure surges that were identified in the surge analysis study, and this would require additional action to be taken in the future.

Table 2-2: Conceptual Cost Estimate-24" Force Main Replacement

ITEM 1 - GENERAL REQUIREMENTS					
1	Mobilization/Demobilization/General Conditions for Infrastructure Imporvements	1	LS	$50,000.00	$50,000
2	Maintenance of Traffic	1	LS	$30,000.00	$30,000
3	Bonds and Insurance Requirements	1	LS	$5,000.00	$5,000
4	Survey Layout & Record Drawings	1	LS	$30,000.00	$30,000
5	Professional Videotaping of Site Conditions (Pre/Post Construction)	1	LS	$5,000.00	$5,000
6	Contingency	(20% of Item 2-General Sitework Subtotal)			$1,032,840
				SUBTOTAL	*$1,152,840*
ITEM 2 - GENERAL SITEWORK					
7	Utility Pipe - 24" Ductile Iron Force Main	17700	LF	$200.00	$3,540,000
8	Utility Fitting - Ductile Iron Elbow - 24"	35	EA	$7,000.00	$245,000
9	Utility Fixture - Valve Assembly - 24"	8	EA	$32,000.00	$256,000
10	Utility Fixture - Tapping Sleeve/Valve - 24"	2	EA	$48,000.00	$96,000
11	Utility Fixture - 2" Air Release Valve	6	EA	$6,000.00	$36,000
12	24" HDD Canal Crossing	1	EA	$85,000.00	$85,000
13	Remove and Replace Concrete Sidewalk	15000	SF	$25.00	$375,000
14	Base and Asphalt Trench Repair	800	SY	$50.00	$40,000
15	Asphalt Milling & Overlay	1000	SY	$15.00	$15,000
16	Remove and Replace Concrete Driveway Apron	860	SY	$60.00	$51,600
17	Pavement Striping	1	LS	$10,000.00	$10,000
18	Flowable Fill 24" Force Main	17700	LF	$8.00	$141,600
19	Remove and replace Unsuitable Materials	1000	CY	$30.00	$30,000
20	Seeding	4000	SY	$2.00	$8,000
21	Sodding	7000	SY	$5.00	$35,000
22	Landscaping	1	LS	$20,000.00	$20,000
23	Density Testing	1	LS	$25,000.00	$25,000
24	Pressure Testing	1	LS	$5,000.00	$5,000
25	Dewatering	1	LS	$150,000.00	$150,000
				CONSTRUCTION SUBTOTAL	*$5,164,200*
				TOTAL ESTIMATED COST	*$6,317,040*
				LOW (-15%)	**$5,369,484**
				HIGH (+20%)	**$7,580,448**

2.1.3 Complete Replacement: 30" Ductile Iron Force Main with Surge Tanks

The third alternative considered is the complete replacement of the existing 24" PVC force main with a new 30" ductile iron force main, and the installation of pressure surge tanks at strategic locations. Like the previous alternative, the benefit of this option is that all remaining PVC pipe within the South Beaches Force Main system would be replaced with ductile iron, and this would eliminate all issues identified in the previous study with regard to improper pipe installation, joint restraints, and compaction. The ductile iron force main would provide a much higher level of protection against future breaks, and therefore remove the public health/environmental impacts. This alternative would also extend the life of the pipe system for the next 50+ years, conservatively. An additional benefit of using 30" DI pipe is that this would help alleviate the high pressures seen in the existing force main system. Based on the force may study that was done, upsizing the Riverside Drive Force Main to 30" would lower system pressures, reduce the total dynamic head seen by the pump stations, and ultimately provide added insurance against future pipe failures. Finally, this alternative would provide a solution for excessive pressure fluctuations within the force main system by utilizing pressure surge tanks. Based on the surge analysis report in **Appendix C**, surge tanks are recommended at the existing B-19 and B-20 lift stations to protect the system from both pressure spikes and negative pressures. The disadvantages of this alternative include construction cost and site construction impacts. The conceptual cost estimate for this alternative is $7.8 million, as seen in **Table 2-3**, is more expensive that both the point repair alternative and the 24" pipe replacement alternative.

Table 2-3: Conceptual Cost Estimate-30" Force Main Replacement with Surge Tanks

ITEM NO.	DESCRIPTION	QUANTITY	UNIT	UNIT PRICE	TOTAL
ITEM 1 - GENERAL REQUIREMENTS					
1	Mobilization/Demobilization/General Conditions for Infrastructure Imporvements	1	LS	$50,000.00	$50,000
2	Maintenance of Traffic	1	LS	$30,000.00	$30,000
3	Bonds and Insurance Requirements	1	LS	$5,000.00	$5,000
4	Survey Layout & Record Drawings	1	LS	$30,000.00	$30,000
5	Professional Videotaping of Site Conditions (Pre/Post Construction)	1	LS	$5,000.00	$5,000
6	Contingency	(20% of Item 2-General Sitework Subtotal)			$1,285,840
				SUBTOTAL	*$1,405,840*
ITEM 2 - GENERAL SITEWORK					
7	Utility Pipe - 30" Ductile Iron Force Main	17700	LF	$250.00	$4,425,000
8	Utility Fitting - Ductile Iron Elbow - 30"	35	EA	$8,000.00	$280,000
9	Utility Fixture - Valve Assembly - 30"	8	EA	$40,000.00	$320,000
10	Utility Fixture - Tapping Sleeve/Valve - 30"	2	EA	$56,000.00	$112,000
11	Utility Fixture - 2" Air Release Valve	6	EA	$6,000.00	$36,000
12	30" HDD Canal Crossing	1	EA	$100,000.00	$100,000
13	Force Main Surge Tank with Piping	2	EA	$125,000.00	$250,000
14	Remove and Replace Concrete Sidewalk	15000	SF	$25.00	$375,000
15	Base and Asphalt Trench Repair	800	SY	$50.00	$40,000
16	Asphalt Milling & Overlay	1000	SY	$15.00	$15,000
17	Remove and Replace Concrete Driveway Apron	860	SY	$60.00	$51,600
18	Pavement Striping	1	LS	$10,000.00	$10,000
19	Flowable Fill 24" Force Main	17700	LF	$8.00	$141,600
20	Remove and replace Unsuitable Materials	1000	CY	$30.00	$30,000
21	Seeding	4000	SY	$2.00	$8,000
22	Sodding	7000	SY	$5.00	$35,000
23	Landscaping	1	LS	$20,000.00	$20,000
24	Density Testing	1	LS	$25,000.00	$25,000
25	Pressure Testing	1	LS	$5,000.00	$5,000
26	Dewatering	1	LS	$150,000.00	$150,000
				CONSTRUCTION SUBTOTAL	*$6,429,200*
				TOTAL ESTIMATED COST	*$7,835,040*
				LOW (-15%)	$6,659,784
				HIGH (+20%)	$9,402,048

2.1.4 Conclusions

This evaluation includes a comparison between three different alternatives including point repairing the existing 24" PVC force main, replacing the existing 24" PVC force main with a 24" ductile iron force main, and replacing the existing 24" PVC force main with a 30" ductile iron force main and surge tanks. Additional evaluation criteria are listed below.

Alternative #1 – Point Repair 24" PVC Force Main

<u>Advantages</u>

- Lower construction cost
- Least amount of impact to community

<u>Disadvantages</u>

- Low reliability
- Future pipe failures likely
- Continued SSO risk to public and environment
- Doesn't address pressure issues in force main

Alternative #2 – 24" Ductile Iron Force Main Replacement

<u>Advantages</u>

- Resolves issue with improper PVC pipe installation
- Reduces risk of future pipe failures
- Extends the future lifespan of force main system

Disadvantages

- Higher Construction Cost
- Greater construction impact to community
- Doesn't address high pressures in force main

Alternative #3 – 30" Ductile Iron Force Main Replacement with Surge Tanks

Advantages

- Resolves issue with improper PVC pipe installation
- Resolves the issue of both high and negative pressures in the pipe system
- Greatly reduces risk of future pipe failures
- Extends the future lifespan of force main system

Disadvantages

- Higher Construction Cost
- Greater construction impact

A tabular listing of evaluation criteria for each alternative to address the Riverside Drive Force Main failures is listed below in **Table 2-4**.

Table 2-4: Alternatives Evaluation Criteria

Evaluation Criteria						
No.	Alternative	*Project Cost	System Reliability	Environmental Impact	Future Capacity/Life	Total
1	Point Repair Existing 24" PVC Force Main	3	1	1	1	9
2	Replacement: 24" DI Force Main	2	2	2	2	10
3	Replacement: 30" DI Force Main w/Surge Tanks	1	3	3	3	11
*Cost Factor Multiplied by 2.0						

Based on the evaluation contained herein, Alternative #3, the replacement of the existing 24" force main with a new 30" DI force main and surge tanks is recommended as the preferred option.

3 Environmental Effects

3.1 Physiography

3.1.1 Physiographic Regions

Brevard County has three major physiographic regions: the Atlantic Coastal Ridge/Coastal Zone, the St. Johns Valley, and the Barrier Islands. The project alternatives in this facility plan fall within the Barrier Islands region. The Barrier Island is separated from the Brevard County mainland by the Indian River and is bordered on the east by the Atlantic Ocean. Merritt Island varies in width from less than one mile to about ten miles. The Barrier Islands range in elevation from sea level to about twenty feet.

3.1.2 Soils

The soils survey prepared by the United States Department of Agriculture (USDA) National Resource Conservation Service (NRCS) shows one primary soil associations with four other minor soil associations in the project area. The primary soil classification according to the NRCS soil survey is 71.9% Welaka Sand. The entire soil survey with soil classifications can be found in **Appendix D**.

3.1.3 Water Resources

The Indian River is a major surface water body, which separates the Brevard County mainland from Merritt Island that can be described as a series of shallow poorly flushed, brackish lagoons, separated by manmade barriers in the form of causeways. The river system contains both Class II and Class III waters, as currently classified by the Florida Department of Environmental Protection. The Indian River was given additional protection by legislative act. The river has historically been a very productive estuarine system. Pollution loads cannot be readily assimilated by the river system. This is generally due to the poor water circulation within the system caused by indiscriminate dredging, filling, and bulk heading. The lack of assimilative capacity has resulted in increased stress to the system.

3.1.4 Floodplains

The Federal Emergency Management Agency (FEMA) has established flood plains throughout the country and maintains their Flood Insurance Rate Maps (FIRMs). The force main alignment being evaluated in this facility plan is graphically represented in Figure 3-1. These maps have been reviewed in conjunction with the project alternatives identified in this Facility Plan. There are no detrimental effects to flood plain development resulting from projects in this plan.

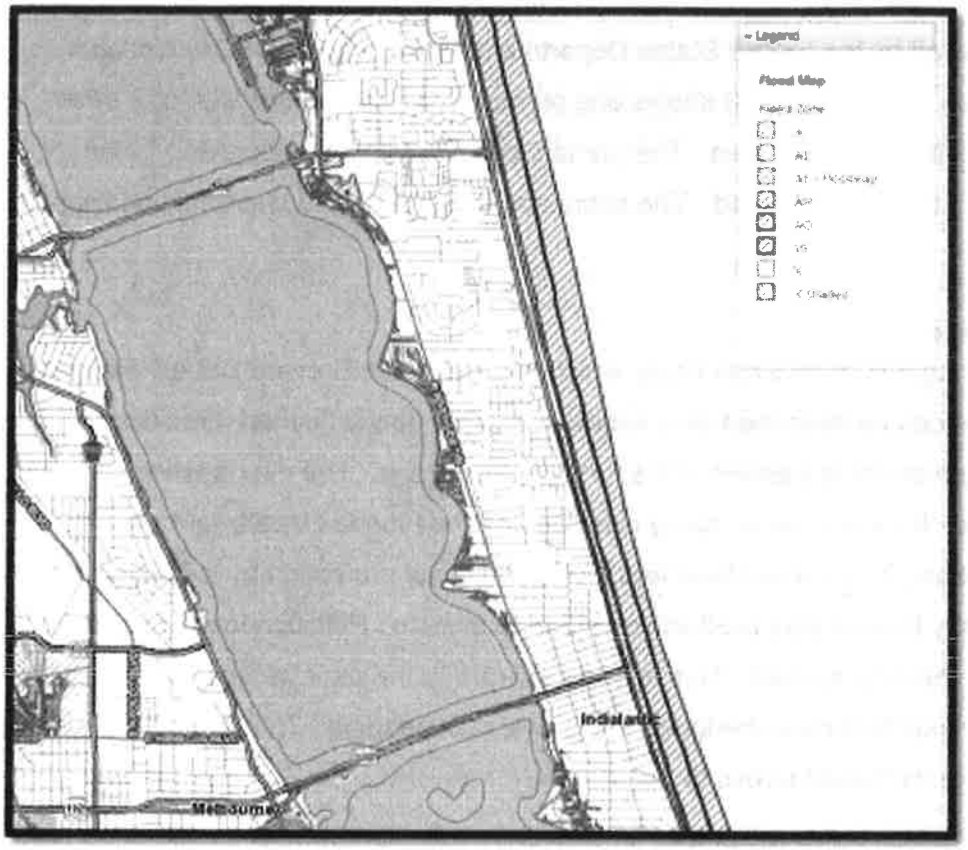


Figure 3-1: FEMA Flood Zones Map

3.1.5 Historic or Archeological Site

Brevard County is planning to repair and/or replace the existing 24" force main along Riverside Drive within the existing boundaries of the current force main limits. There are no known archeological or historic sites that will be affected by the project proposed in this plan.

3.1.6 Wildlife and Endangered Species

There are several mammals, birds, reptiles, fish, and plants listed as threatened, endangered, or of special concern that are present in Brevard County. The proposed project identified in this facility plan will not have any impacts upon the endangered or threatened species in this region. The full list of Florida's federally listed plant species can be found at: https://www.freshfromflorida.com/Divisions-Offices/Florida-Forest-Service/Our-Forests/Forest-Health/Florida-Statewide-Endangered-and-Threatened-Plant-Conservation-Program/Florida-s-Federally-Listed-Plant-Species. A full list of the FWC's listed species may be viewed by visiting the FWC website at: http://myfwc.com/wildlifehabitats/imperiled/profiles/. Some of the listed species can be found in areas nearby the proposed project; however, because the project construction area is within a cleared existing right-of-way along Riverside Drive there are no listed species that would be directly affected by the proposed project. The following **Table 3-1** summarizes the endangered and threatened species found in Brevard County.

Table 3-1: Protection Status of Wildlife Found in Brevard County

Birds - Brevard County USFWC Protected Species	
Common Name	**Status**
Everglades Snail Kite	Endangered
Wood Stork	Threatened
Roseate Spoonbill	Endangered
Little Blue Heron	Endangered
Reddish Egret	Threatened
Snowy Egret	Special Concern
Louisiana Heron	Special Concern
Peregrin Falcon	Special Concern
Southern Kestral	UR2
American Oyster Catcher	Threatened
Southern Bald Eagle	Special Concern
Osprey	Threatened
Brown Pelican	Special Concern
Least Tern	Endangered
Roseate Tern	Endangered
Audubon's Crested Caracara	Threatened
Piping Plover	Threatened
Florida Scrub Jay	Threatened
Red Knot	Threatened

Plants - Brevard County USFWC Protected Species	
Common Name	**Status**
Lewton's Polygala	Endangered
Coastal Vervain	Special Concern
Prickly Pear	Threatened
Sea Lavender	Special Concern
Carter's Mustard	Endangered

Fish - Brevard County USFWC Protected Species	
Common Name	**Status**
Rivulus	Special Concern
Common Snook	Threatened

Mammals - Brevard County USFWC Protected Species	
Common Name	**Status**
West Indian Manatee	Threatened
Florida Mink	Special Concern
Round Tailed Muskrat	UR2
Florida Mouse	UR2
Fox Squirrel	Special Concern
Southeastern Beach Mouse	Threatened

Reptiles - Brevard County USFWC Protected Species	
Common Name	**Status**
American Alligator	Special Concern
Hawksbill Turtle	Endangered
Ridley Turtle	Endangered
Hawksbill Sea Turtle	Endangered
Leatherback Sea Turtle	Endangered
Green Sea Turtle	Threatened
Loggerhead Sea Turtle	Threatened
Atlantic Salt Marsh Snake	Threatened
Eastern Indigo Snake	Threatened
Gopher Tortoise	Candidate

3.2 Environmental/Community Benefits

The repair and/or replacement of the Riverside Drive Force main will greatly reduce the potential for future sewage spills into the environment and reduce construction impacts due to emergency pipe repair. The Riverside Drive Force Main has had multiple failures over the past few years, which have all had an adverse effect on the both the environment and the community. With each pipe failure, there is a usually a corresponding sewage spill that is released into the environment. These spills affect the quality of ground water, soil conditions, and nearby surface water bodies. Depending on the location and extent of the spill, public notices are often issued to warn residents about the potential hazards. In addition to the environmental hazard, the repair of a large diameter force main along a major roadway often causes road closures and traffic delays. By implementing the proposed project in this facility plan, these environmental and community impacts could be mostly eliminated.

3.3 Environmental/Community Impacts

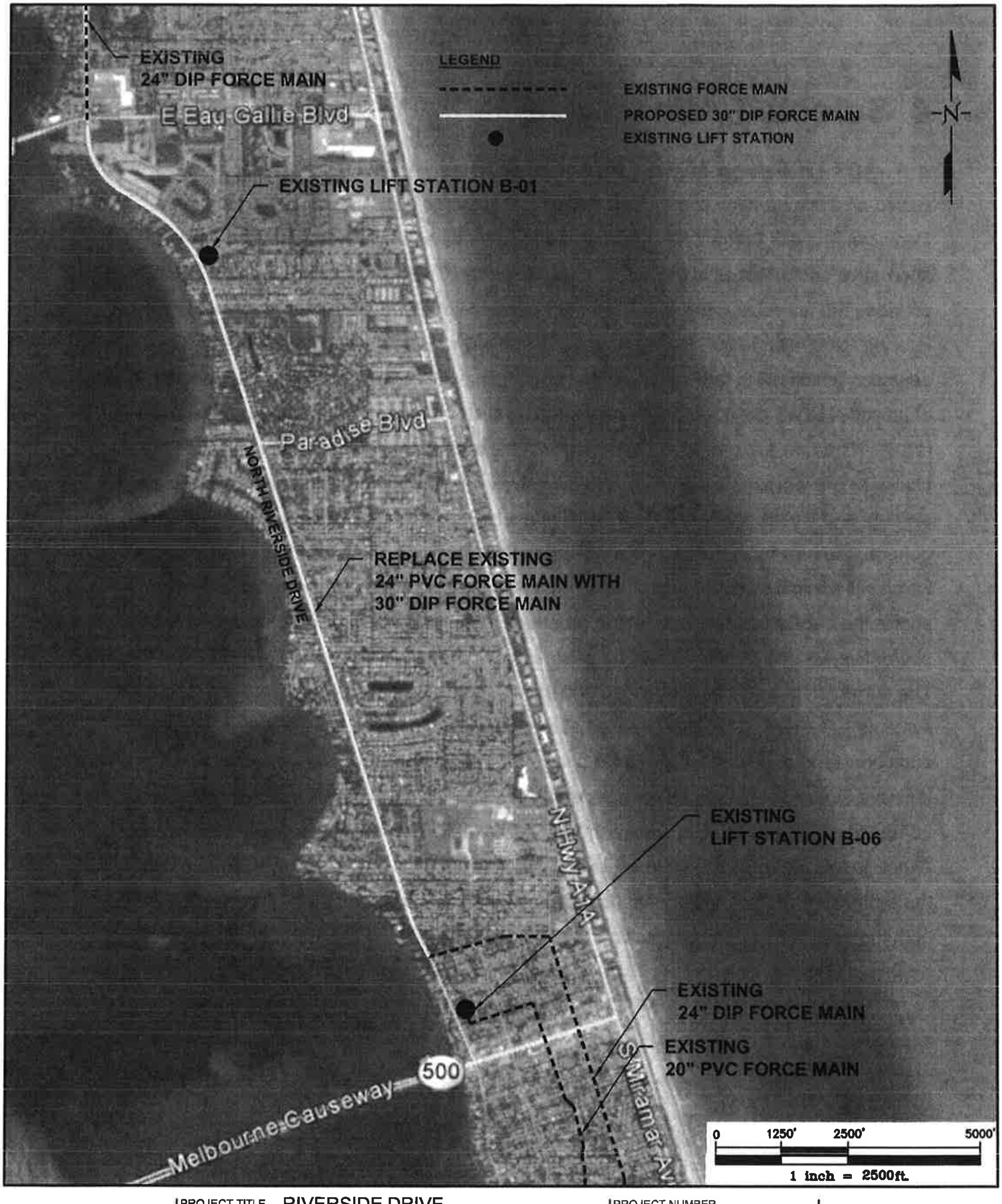
The impacts to the environment and the community as a result of this project are minimal. Community impacts would be limited to temporary lane shifts and lane closures during construction and some traffic delays as a result of the new traffic patterns. The majority of the construction work would occur outside of the travel lanes, and would be in the unpaved right-of-way. Most of the impacts to the travel lanes would be where the new force main crosses the side streets along the proposed alignment.

Environmental impacts would also be very limited due to the fact that the new force main would be placed in the same alignment as the existing force main, and there would be no additional clearing or disruption to the surrounding areas. The proposed project would include a force main crossing underneath a canal. To avoid any environmental impacts at this canal crossing, a trenchless installation method would be used. Horizontal directional drilling would be the preferred installation method, and would eliminate the need to impact the waterway.

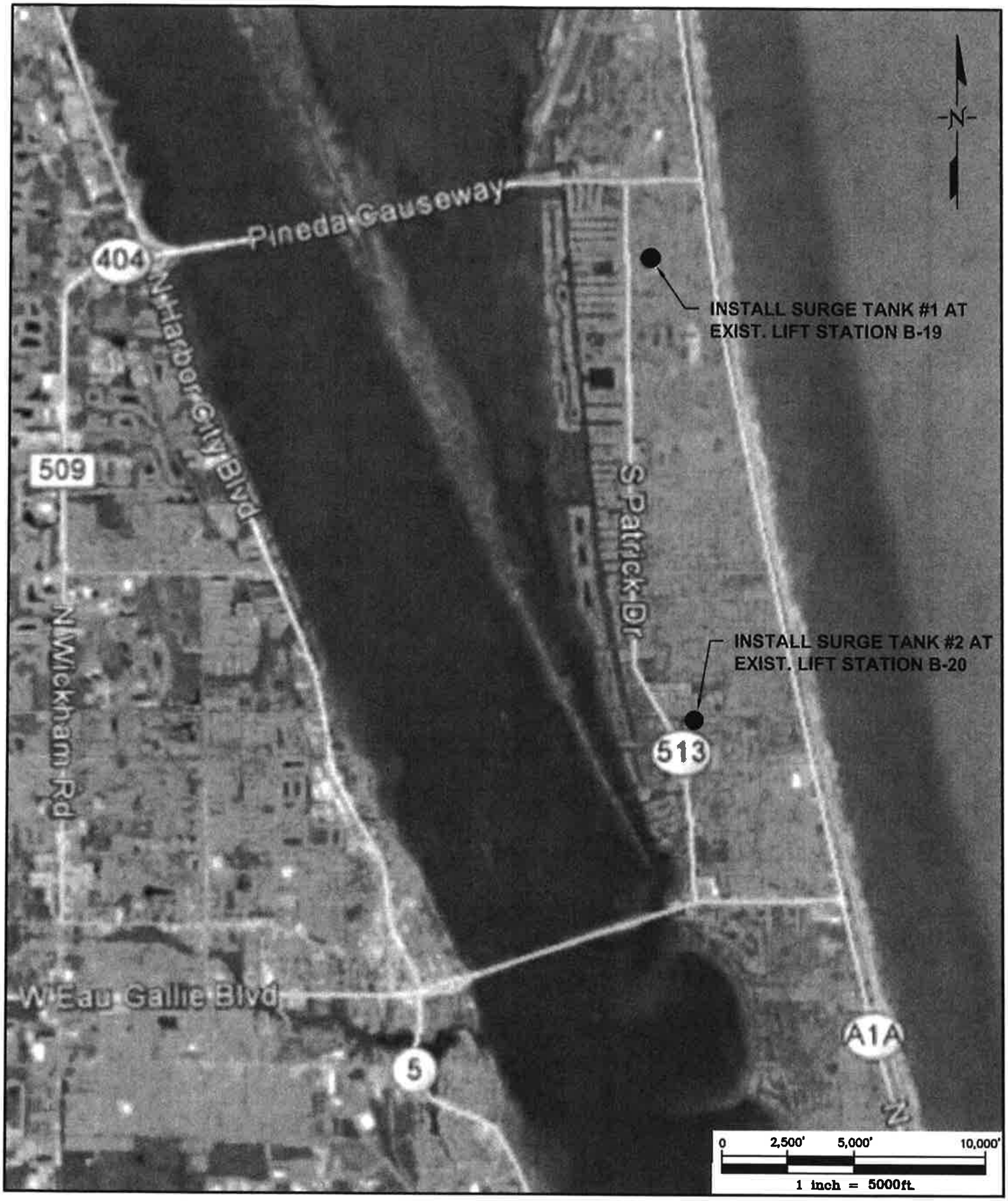
4 Selected Alternative

4.1 30" DI Force Main Replacement with Surge Tanks

Based on the evaluation criteria established in this facility plan, the 30" DI Force Main Replacement with Surge Tank Alternative was determined to be the best option. This alternative addresses all of the issues identified in the South Beaches Force Main Study, and provides the lowest amount of risk for the occurrence of future pipe failures. By upsizing the existing force main to 30", the force main system will see lower pressures in the Riverside Drive segment, which will in turn reduce stress on the pipe. In addition, using ductile iron pipe in lieu of plastic pipe will provide a much stronger pipe system that is less prone to breaks and joint failure. The new system will be designed with the appropriate pipe restraints and/or thrust blocks to prevent joint failures. Air release valves will be installed at the high points in the system, and isolation valves will be installed every 800 to 1,000 feet and at major junctions. In general, the new force main system will follow the alignment of the existing 24" force main, which will minimize the amount of new impacts to the surrounding infrastructure. **Figure 4-1** shows the overall project layout of the proposed force main alternative. Additionally, this alternative will include the installation of pressure surge tanks at the B-19 and B-20 Lift Stations. The surge tanks would be roughly 10,000 gallon vertical surge tanks with internal air bladder systems. The tanks would be installed at existing lift station sites, and would not require any additional land acquisition. **Figure 4-2** shows the proposed surge tank locations, and **Figure 4-3** shows a typical surge tank schematic detail. The conceptual construction cost estimate for the 30" ductile iron force main replacement with surge tanks alternative is roughly $7.8 million. Although this alternative is the most expensive, this option is the only one that addresses all of the system issues that have been identified and therefore provides the highest level of safety for both the environment and the public.



PROJECT TITLE RIVERSIDE DRIVE FORCE MAIN REPLACEMENT	PROJECT NUMBER 10092206	
SHEET TITLE **FIGURE:4-1** PROJECT LAYOUT AERIAL	PROJECT MANAGER H.HARDY	EXHIBIT SCALE 1"=2500'
	DATE 03/2018	EXHIBIT NUMBER 4-1



HDR

PROJECT TITLE RIVERSIDE DRIVE
FORCE MAIN REPLACEMENT

SHEET TITLE
FIGURE:4-2
SURGE TANK LOCATIONS

PROJECT NUMBER
10092206

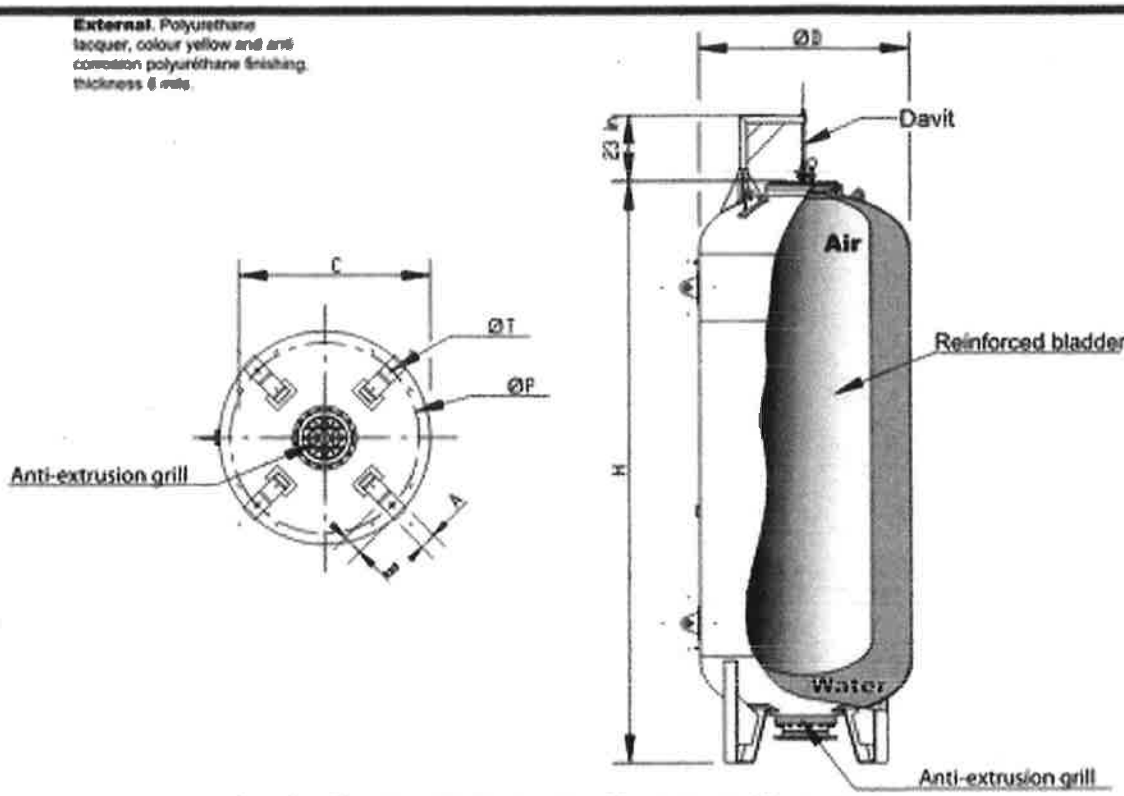
PROJECT MANAGER
H.HARDY

DATE
03/2018

EXHIBIT SCALE
1"=5000'

EXHIBIT NUMBER
4-2

External. Polyurethane lacquer, colour yellow and anti corrosion polyurethane finishing thickness 6 mils.



Capacity gallon	Diameter D	Height H	P	T	Size A	Size B	Size C	Davit
793	47	150	41	1	6	5	54	No
1057	47	161	41	1	6	5	54	No
1321	59	138	51	1	6	7	66	No
1585	59	169	51	1	6	7	66	No
1849	59	189	51	1	6	7	66	No
2113	59	209	51	1	6	7	66	No
2378	75	177	67	1	6	7	81	Yes
2378	83	138	71	1	6	11	89	Yes
2642	75	193	67	1	6	7	81	Yes
2642	83	150	71	1	6	11	89	Yes
3170	75	220	67	1	6	7	81	Yes
3170	83	173	71	1	6	11	89	Yes
3963	83	209	71	1	6	11	89	Yes
5283	98	197	79	1	20	20	105	Yes
6604	98	236	79	1	20	20	105	Yes
7925	118	213	100	1	20	20	125	Yes
9246	118	240	100	1	20	20	125	Yes

Figure 4-3: Surge Tank Schematic Detail

A

Appendix A – FDEP Consent Order

BEFORE THE STATE OF FLORIDA
DEPARTMENT OF ENVIRONMENTAL PROTECTION

STATE OF FLORIDA DEPARTMENT OF ENVIRONMENTAL PROTECTION	)))	IN THE OFFICE OF THE CENTRAL DISTRICT
v.	)	OGC FILE NO. 16-1352
BREVARD COUNTY	))	

CONSENT ORDER

This Consent Order ("Order") is entered into between the State of Florida Department of Environmental Protection ("Department") and Brevard County ("Respondent") to reach settlement of certain matters at issue between the Department and Respondent.

The Department finds and Respondent admits the following:

1. The Department is the administrative agency of the State of Florida having the power and duty to protect Florida's air and water resources and to administer and enforce the provisions of Chapter 403, Florida Statutes ("F.S."), and the rules promulgated and authorized in Title 62, Florida Administrative Code ("F.A.C."). The Department has jurisdiction over the matters addressed in this Order.

2. Respondent is a person within the meaning of Section 403.031(5), F.S.

3. Respondent is the owner and is responsible for the operation of the Brevard County Utilities Department (BCUD) South Beaches wastewater treatment facility (WWTF) and collections system, an 8.0 MGD annual average daily flow (AADF) domestic WWTF facility with surface water discharge (D-001), underground injection (U-001), and land application (R-001) ("Facility"). The Facility is operated under Wastewater Permit No. FL0040622 ("Permit"), which was issued on March 13, 2014 and will expire on March 19, 2019. The Facility is located at 2800 South Highway A1A, Melbourne Beach, FL 32951, in Brevard County, Florida ("Property"). Respondent owns the Property on which the Facility is located.

4. The Department finds that the following violation(s) occurred:

a) A force main break on South Patrick Drive between DeSoto Parkway and Banana River Drive caused three unauthorized discharges of raw wastewater totaling approximately 1,521,000 gallons on September 20, 2016. Two to Class III surface waters, 733,000 gallons and 578,000 gallons, and an additional 210,000 gallons to a stormwater pond. The releases are in violation of 62-604.130 F.A.C., 62-604.500(2) F.A.C., and F.S. Chapter 403.121(3)(b).

Having reached a resolution of the matter Respondent and the Department mutually agree and it is

ORDERED:

5. Respondent shall comply with the following corrective actions within the stated time periods:

a) Within 365 days of the effective date of this Order, replace the 1.1-mile section of the BCUD South Beaches collection system force main along South Patrick Drive between DeSoto Parkway and Banana River Drive with a recent history of line breaks. The Department recognizes that the Respondent has initiated addressing the subject force main as of June 2016. A Task Order was executed to design and construct replacement pipe on August 23, 2016 and the Brevard County Board of Commissioners initiated emergency conditions to waive the bidding process in order to expedite the replacement project on September 27, 2016. Respondent submitted copies of these orders to the Department on October 24, 2016.

b) Within 30 days of the effective date of the Order, provide a plan outlining best management practices (BMPs) and spill prevention and response measures which will be implemented during the replacement of the affected portion of the collection system to prevent any additional unauthorized releases.

c) Within 365 days of the effective date of this Order, complete an evaluation of the 2.8-mile section of the force main on South Patrick Drive/North Riverside Drive between DeSoto Parkway and Sand Dollar Road in accordance with paragraph 6, below.

d) Within 60 days of the completion of paragraph 5c, above, submit the results of the evaluation along with a plan and schedule for corrective action, if applicable.

e) Within 180 days of the effective date of this Order, provide a long-term maintenance plan of the entire BCUD South Beaches collection system to prevent similar discharges in the future. The maintenance plan shall include an up-to-date project list for rehabilitation and maintenance based on the Infrastructure Asset Evaluation completed by Brevard County Utilities in 2013, or any additional more recent evaluations.

f) Every calendar quarter after the effective date of this Order, Respondent shall submit in writing to the Department a report containing information concerning the status and progress of projects being completed under this Order, information as to compliance or noncompliance with the applicable requirements of this Order including construction requirements and effluent limitations, and any reasons for noncompliance. These reports shall also include a projection of the work to be performed pursuant to this Order during the 12-month period which will follow the report. These reports shall be submitted to the Department within 30 days following the end of the quarter.

6. The Respondent shall continue the retained services which commenced on August 23, 2016 of a professional engineer, registered in the State of Florida, to accomplish all of the following:

a) Evaluating the Facility's associated collection system to discover the cause or causes of the noncompliance.

b) Designing modifications of collection systems to ensure the system will function in full and consistent compliance with all applicable rules of the Department.

c) Completing an application for a Department wastewater permit to construct the modifications listed in subparagraph (b) of this paragraph, if such a permit is required.

d) Overseeing the construction of any modifications to the Facility, effluent disposal system, or collection system.

e) Submitting to the Department a Certification of Completion, prepared and sealed by a professional engineer registered in the State of Florida, stating that modifications to the Facility, effluent disposal system, and collection system have been constructed in accordance with the provisions of the Permit.

f) In the event the Department requires additional information to process the permit application described in subparagraph (c) of this paragraph, providing a written response containing the information requested by the Department within 30 days of the date of the request.

7. Within 90 days of the effective date of this Order, Respondent shall submit a complete application for a Department wastewater permit to construct the modifications listed in subparagraph 6b, if such a permit is required.

8. Within 270 days of the date a wastewater permit is issued, or, if no permit is required, within 365 days of the effective date of this Order, Respondent shall complete construction of the modification(s) developed pursuant to paragraph 6 and submit a Certification of Completion, prepared and sealed by a professional engineer registered in the State of Florida, stating that modifications to the Facility, effluent disposal system, and collection system have been constructed in accordance with the provisions of the Permit.

9. Notwithstanding the time periods described in the paragraphs above, Respondent shall complete all corrective actions required by paragraphs 5 and 6 within 365 days of the effective date of this Order and be in full compliance with Rule 62-604, F.A.C., regardless of any intervening events or alternative time frames imposed in this Order, other than those excused delays agreed to by the Department, as described in paragraph 17, below.

10. Within 90 days of the effective date of this Order, Respondent shall submit a written estimate of the total cost of the corrective actions required by this Order to the Department. The written estimate shall identify the information the Respondent relied upon to provide the estimate.

11. Within 45 days of the effective date of this Order, Respondent shall pay the Department $2,750.00 in settlement of the regulatory matters addressed in this Order. This amount includes $2,500.00 for civil penalties and $250.00 for costs and expenses incurred by the Department during the investigation of this matter and the preparation and tracking of this Order. The civil penalties are apportioned as follows: $2,500.00 for violation of Florida Statute (F.S.) 403.121(3)(b).

12. Respondent agrees to pay the Department stipulated penalties in the amount of $100 per day for each and every day Respondent fails to timely comply with any of the

requirements of paragraph(s) 5, 6, 9 of this Order. The Department may demand stipulated penalties at any time after violations occur. Respondent shall pay stipulated penalties owed within 30 days of the Department's issuance of written demand for payment, and shall do so as further described in paragraph 13, below. Nothing in this paragraph shall prevent the Department from filing suit to specifically enforce any terms of this Order. Any stipulated penalties assessed under this paragraph shall be in addition to the civil penalties agreed to in paragraph 11 of this Order.

13. Respondent shall make all payments required by this Order by cashier's check, money order or on-line payment. Cashier's check or money order shall be made payable to the "Department of Environmental Protection" and shall include both the OGC number assigned to this Order and the notation "Water Quality Assurance Trust Fund." Online payments by e-check can be made by going to the DEP Business Portal at: http://www.fldepportal.com/go/pay/. It will take a number of days after this order is final and effective filed with the Clerk of the Department before ability to make online payment is available.

14. Except as otherwise provided, all submittals and payments required by this Order shall be sent to Megan Warr, Environmental Specialist II/Compliance Assurance Program, Department of Environmental Protection, 3319 Maguire Blvd, Suite 232, Orlando, Florida 32803-3767.

15. Respondent shall allow all authorized representatives of the Department access to the Facility and the Property at reasonable times for the purpose of determining compliance with the terms of this Order and the rules and statutes administered by the Department.

16. In the event of a sale or conveyance of the Facility or of the Property upon which the Facility is located, if all of the requirements of this Order have not been fully satisfied, Respondent shall, at least 30 days prior to the sale or conveyance of the Facility or Property, (a) notify the Department of such sale or conveyance, (b) provide the name and address of the purchaser, operator, or person(s) in control of the Facility, and (c) provide a copy of this Order with all attachments to the purchaser, operator, or person(s) in control of the Facility. The sale

or conveyance of the Facility or the Property does not relieve Respondent of the obligations imposed in this Order.

17. If any event, including administrative or judicial challenges by third parties unrelated to Respondent, occurs which causes delay or the reasonable likelihood of delay in complying with the requirements of this Order, Respondent shall have the burden of proving the delay was or will be caused by circumstances beyond the reasonable control of Respondent and could not have been or cannot be overcome by Respondent's due diligence. Neither economic circumstances nor the failure of a contractor, subcontractor, materialman, or other agent (collectively referred to as "contractor") to whom responsibility for performance is delegated to meet contractually imposed deadlines shall be considered circumstances beyond the control of Respondent (unless the cause of the contractor's late performance was also beyond the contractor's control). Upon occurrence of an event causing delay, or upon becoming aware of a potential for delay, Respondent shall notify the Department by the next working day and shall, within seven calendar days notify the Department in writing of (a) the anticipated length and cause of the delay, (b) the measures taken or to be taken to prevent or minimize the delay, and (c) the timetable by which Respondent intends to implement these measures. If the parties can agree that the delay or anticipated delay has been or will be caused by circumstances beyond the reasonable control of Respondent, the time for performance hereunder shall be extended. The agreement to extend compliance must identify the provision or provisions extended, the new compliance date or dates, and the additional measures Respondent must take to avoid or minimize the delay, if any. Failure of Respondent to comply with the notice requirements of this paragraph in a timely manner constitutes a waiver of Respondent's right to request an extension of time for compliance for those circumstances.

18. The Department, for and in consideration of the complete and timely performance by Respondent of all the obligations agreed to in this Order, hereby conditionally waives its right to seek judicial imposition of damages or civil penalties for the violations

described above up to the date of the filing of this Order. This waiver is conditioned upon Respondent's complete compliance with all of the terms of this Order.

19. This Order is a settlement of the Department's civil and administrative authority arising under Florida law to resolve the matters addressed herein. This Order is not a settlement of any criminal liabilities which may arise under Florida law, nor is it a settlement of any violation which may be prosecuted criminally or civilly under federal law. Entry of this Order does not relieve Respondent of the need to comply with applicable federal, state, or local laws, rules, or ordinances.

20. The Department hereby expressly reserves the right to initiate appropriate legal action to address any violations of statutes or rules administered by the Department that are not specifically resolved by this Order.

21. Respondent is fully aware that a violation of the terms of this Order may subject Respondent to judicial imposition of damages, civil penalties up to $10,000.00 per day per violation, and criminal penalties.

22. Respondent acknowledges and waives its right to an administrative hearing pursuant to sections 120.569 and 120.57, F.S., on the terms of this Order. Respondent also acknowledges and waives its right to appeal the terms of this Order pursuant to section 120.68, F.S.

23. Electronic signatures or other versions of the parties' signatures, such as .pdf or facsimile, shall be valid and have the same force and effect as originals. No modifications of the terms of this Order will be effective until reduced to writing, executed by both Respondent and the Department, and filed with the clerk of the Department.

24. The terms and conditions set forth in this Order may be enforced in a court of competent jurisdiction pursuant to sections 120.69 and 403.121, F.S. Failure to comply with the terms of this Order constitutes a violation of section 403.161(1)(b), F.S.

25. This Consent Order is a final order of the Department pursuant to section 120.52(7), F.S., and it is final and effective on the date filed with the Clerk of the Department unless a Petition for Administrative Hearing is filed in accordance with Chapter 120, F.S.

Upon the timely filing of a petition, this Consent Order will not be effective until further order of the Department.

26. Persons who are not parties to this Consent Order, but whose substantial interests are affected by it, have a right to petition for an administrative hearing under sections 120.569 and 120.57, Florida Statutes. Because the administrative hearing process is designed to formulate final agency action, the filing of a petition concerning this Consent Order means that the Department's final action may be different from the position it has taken in the Consent Order.

The petition for administrative hearing must contain all of the following information:

a) The OGC Number assigned to this Consent Order;

b) The name, address, and telephone number of each petitioner; the name, address, and telephone number of the petitioner's representative, if any, which shall be the address for service purposes during the course of the proceeding;

c) An explanation of how the petitioner's substantial interests will be affected by the Consent Order;

d) A statement of when and how the petitioner received notice of the Consent Order;

e) Either a statement of all material facts disputed by the petitioner or a statement that the petitioner does not dispute any material facts;

f) A statement of the specific facts the petitioner contends warrant reversal or modification of the Consent Order;

g) A statement of the rules or statutes the petitioner contends require reversal or modification of the Consent Order; and

h) A statement of the relief sought by the petitioner, stating precisely the action petitioner wishes the Department to take with respect to the Consent Order.

The petition must be filed (received) at the Department's Office of General Counsel, 3900 Commonwealth Boulevard, MS# 35, Tallahassee, Florida 32399-3000 within 21 days of receipt of this notice. A copy of the petition must also be mailed at the time of filing to the Central District Office at 3319 Maguire Blvd, Suite 232, Orlando, FL 32803. Failure to file a petition within the 21-day period constitutes a person's waiver of the right to request an administrative hearing and to participate as a party to this proceeding under sections 120.569 and 120.57, Florida Statutes. Before the deadline for filing a petition, a person whose substantial interests are affected by this Consent Order may choose to pursue mediation as an alternative remedy under section 120.573, Florida Statutes. Choosing mediation will not adversely affect such person's right to request an administrative hearing if mediation does not result in a settlement. Additional information about mediation is provided in section 120.573, Florida Statutes and Rule 62-110.106(12), Florida Administrative Code.

27. Rules referenced in this Order are available at http://www.dep.state.fl.us/legal/Rules/rulelist.htm

FOR THE RESPONDENT:

Jim Barfield
Chairman, Board of County Commissioners of Brevard County, Florida

11-17-16
Date

DONE AND ORDERED this 18th day of November 2016, in Orange County, Florida.

STATE OF FLORIDA DEPARTMENT
OF ENVIRONMENTAL PROTECTION

Jeff Prather
Director, Central District

Filed, on this date, pursuant to section 120.52, F.S., with the designated Department Clerk, receipt of which is hereby acknowledged.

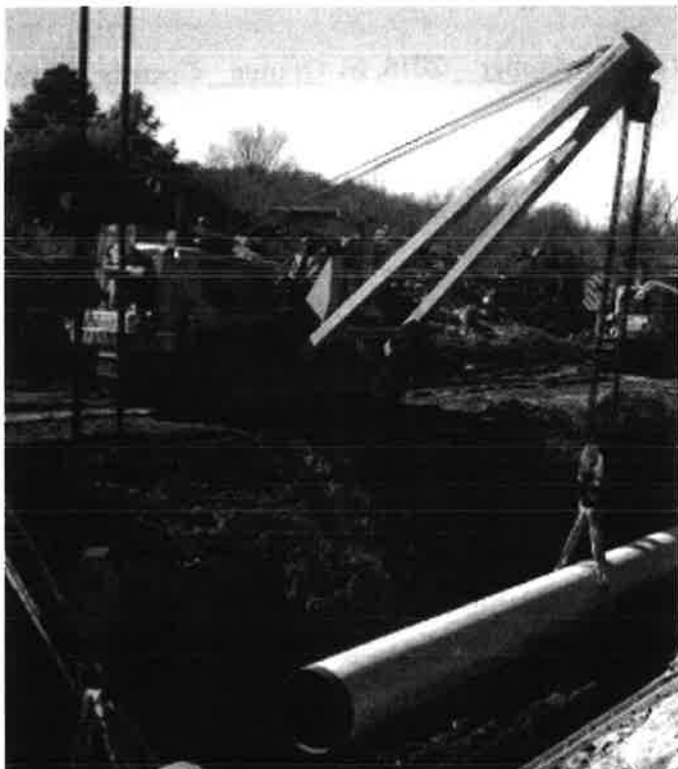
Clerk

November 18, 2016
Date

Copies furnished to:

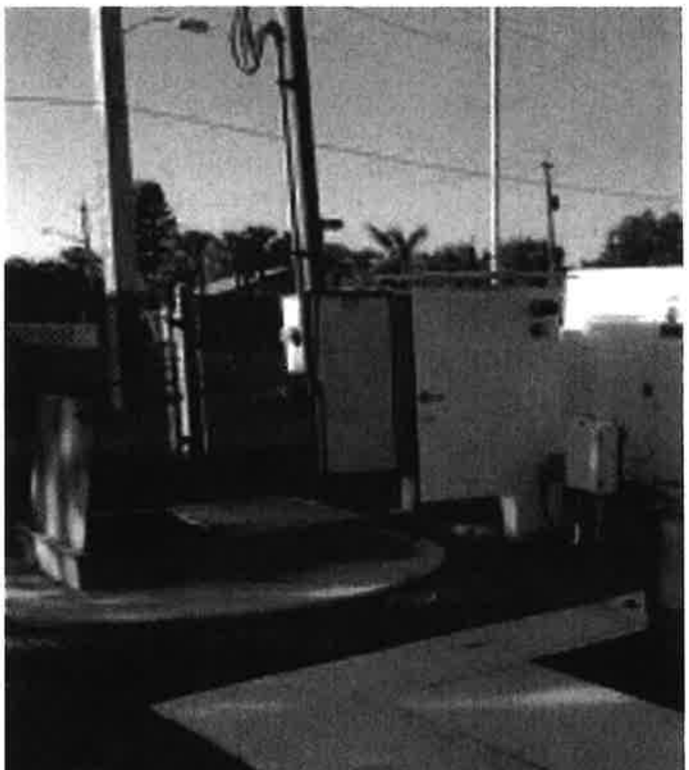
Lea Crandall, Agency Clerk
Mail Station 35

DW_CO (REV. 06/08)



B

Appendix B – South Beaches Force Main Study



Brevard County

South Beaches Force Main Analysis

November 14, 2017

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Introduction

Brevard County Utility Services (USD) has requested HDR Inc. to evaluate the 'South Beaches Force Main System'. Specifically, USD has identified the need to investigate the cause of repeated force main breaks along S. Patrick Drive and Riverside Drive, which are part of the force main system from Lift Station #B-19 to the South Beaches WWTP. The force main system is comprised of a combination of ductile iron pipe and PVC pipe. The pipe sizes on the main force main system vary between 20" and 24". The three main pump stations that contribute the majority of the flow to this system are Lift Stations B-19, B-20, and B-01.

The system has experienced a significant number of breaks over the last few years, concentrated in the areas where the pipe material is PVC. In November 2016, the Florida Department of Environmental Protection issued a Consent Order related to sewage spills resulting from these force main breaks. Included in the Consent Order is the requirement that USD study the force main system and the cause of the pipeline failures.

This report will consider the cause of the breaks and provide analysis of options for system upgrades that would minimize spills to the Indian River Lagoon. The Indian River Lagoon is an Outstanding Florida Water (OFW) and the USD is committed to reducing SSO's for this system that could negatively impact this water body.

A portion of this force main system from DeSoto Parkway to Banana River Drive is currently being replaced, with construction expected to be complete by the end of 2017. This portion of PVC pipe was deemed inadequate because of the large number of breaks and lack of proper restraints, so the USD decided to replace this concurrent with studying the remainder of the system.

2.0 Review of Pipe Break Data and Prior Surge Reports

A review of the existing data indicates evidence of improperly installed pipe in at least two of the pipeline segments. USD has had the pipe manufacturer investigate the installed PVC in one segment to determine if a manufacturing issue was the cause. The force main system is 13 miles in length and operates at relatively high pressures. Pressure surges are thought to be a contributing issue to the high frequency of breaks. Lack of joint restraints, inadequate compaction and pipe bedding are also thought to be contributing factors to the frequency of the breaks. PBS&J performed a surge analysis of the system in 2005, and in 2010, CH2M Hill performed an analysis of the B-6 force main system which also included data and observations about the B-19 system.

Table 1 shows a summary of the force main breaks, categorized by the type of break. Although there have been four types of breaks, nearly 40% of the pipe breaks have been split pipe at the bell of the pipe.

Table 1. Pipe Breaks

Type of Break	Occurrences
Bell Split	10
Leak	11
Joint Separation	2
Other	3

Figures 1 – 3 show the force main breaks along the pipeline route. Figure 1 shows the breaks in the run of pipeline from Desoto Parkway to Banana River Drive. This section is currently under construction and should be complete by the end of 2017. This section is being replaced with 20" and 24" DIP. Further analysis of the pipeline system will focus on the areas shown in Figures 2 – 3.

HDR was tasked to evaluate the PBS&J and the CH2M reports, the Diamond Plastics pipe reports, as well as the data provided in Task 2A, with regard to operating issues and documented failures.

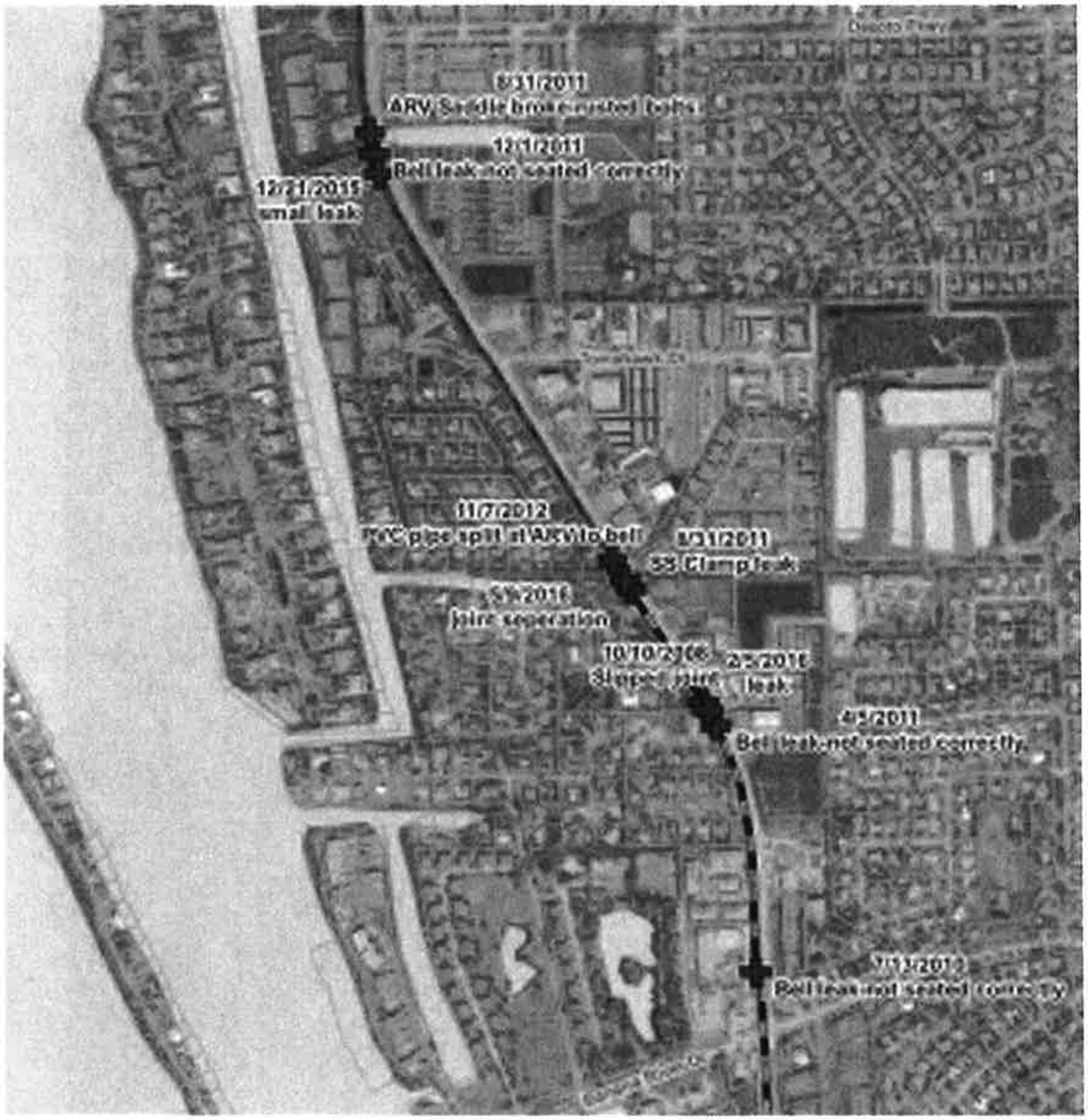
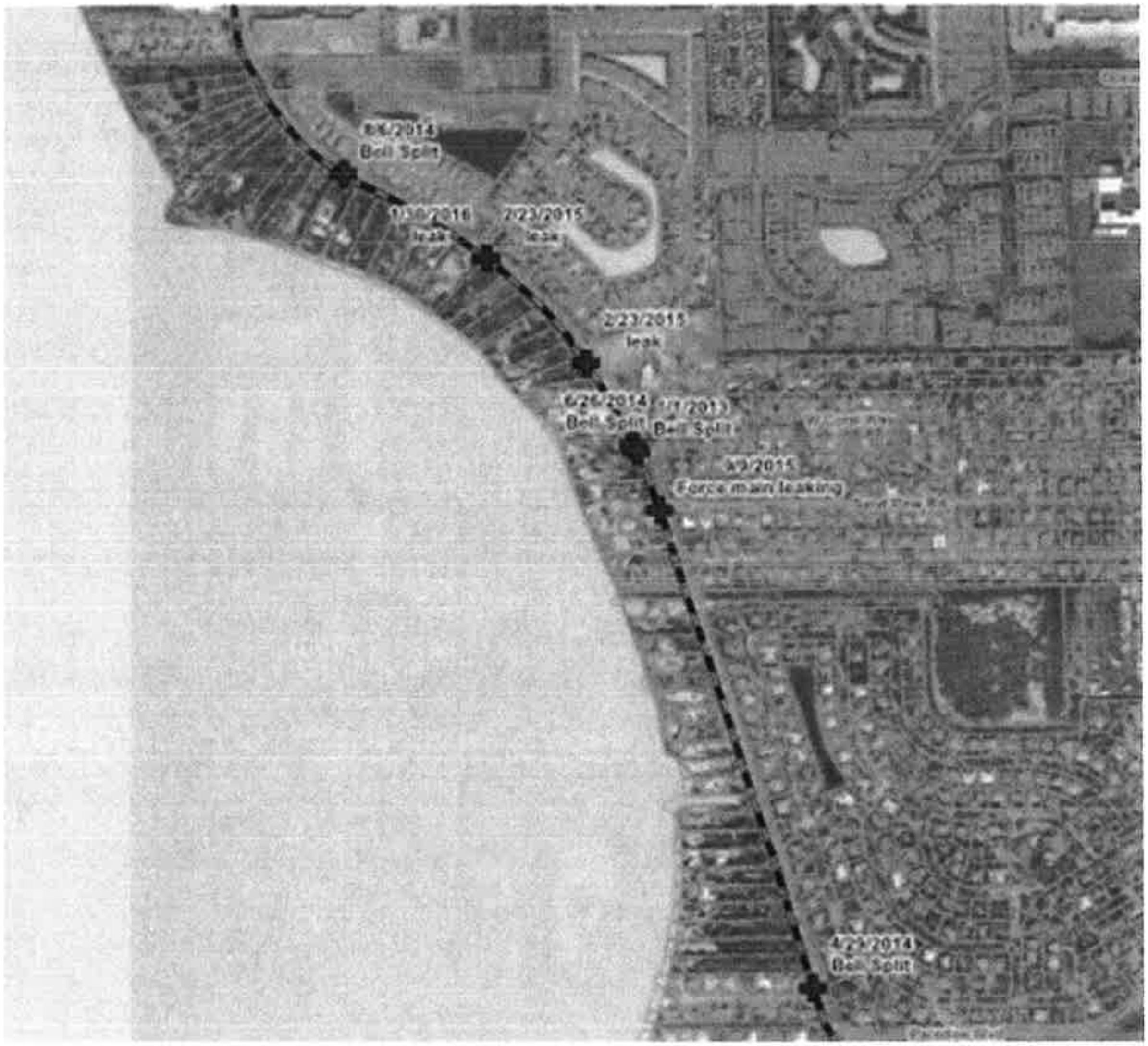


Figure 1. Force Main on S. Patrick from Desoto Pkwy to Banana River Dr.



<u>Figure 2. Riverside Dr. from Eau Gallie Blvd. to Paradise Blvd.</u>

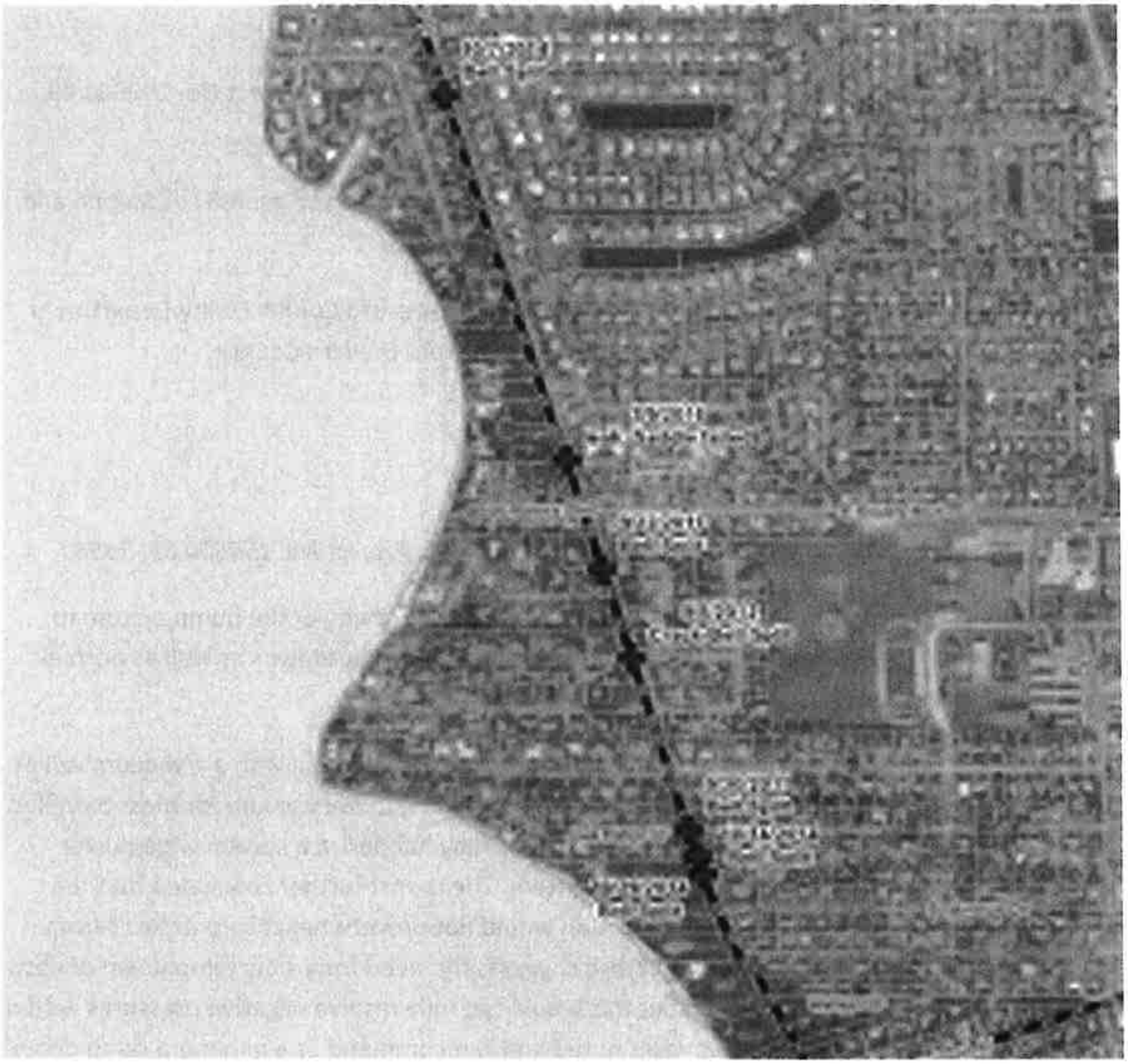


Figure 3. Riverside Dr. from Paradise Blvd. to Oakland Ave.

Reviewed Reports:

- South Beaches, Sewer Force Main Draft Surge Analysis, October 2005, Project No. 150500.09, PBS&J
- Expert Opinion of Scott C. Williams, PE on a Construction Project Known as: B-6 Lift Station and Force Main Modifications Brevard County, FL, February 16, 2010
- Diamond Plastics Corporation Field Report No. FR 3395 Re: Brevard County Utility Department AWWA C 905 DR 25 PVC Sewer Force Main Pipe Samples from Brevard County

Findings from Reviewed Reports:

South Beaches, Sewer Force Main Draft Surge Analysis, October 2005, Project No. 150500.09, PBS&J

The PBS&J report utilized a hydraulic transient model to assess the sensitivity of the pump system to adverse hydraulic transient pressures resulting from uncontrolled pump shutdowns as well as normal shutdown and startup procedures.

The PBS&J recommendations suggested that installing 'surge relief' valves along with air/vacuum valves at each pump station would prevent negative pressures from dropping to full vacuum for most modeled cases. However, when all pump stations fail simultaneously, as may happen in a system wide power outage, full vacuum was predicted to occur within the system. The report further concluded that the 'surge relief' valve and air/vacuum valve surge mitigation would not prevent negative pressures from occurring in the majority of the system. The report also suggests the need for a slow ramp down of each pump to help mitigate the negative transients, but this would not fully resolve negative pressures within the modeled system. A ramp down of 60 seconds is noted and is recommend as a minimum ramp down time in the report.

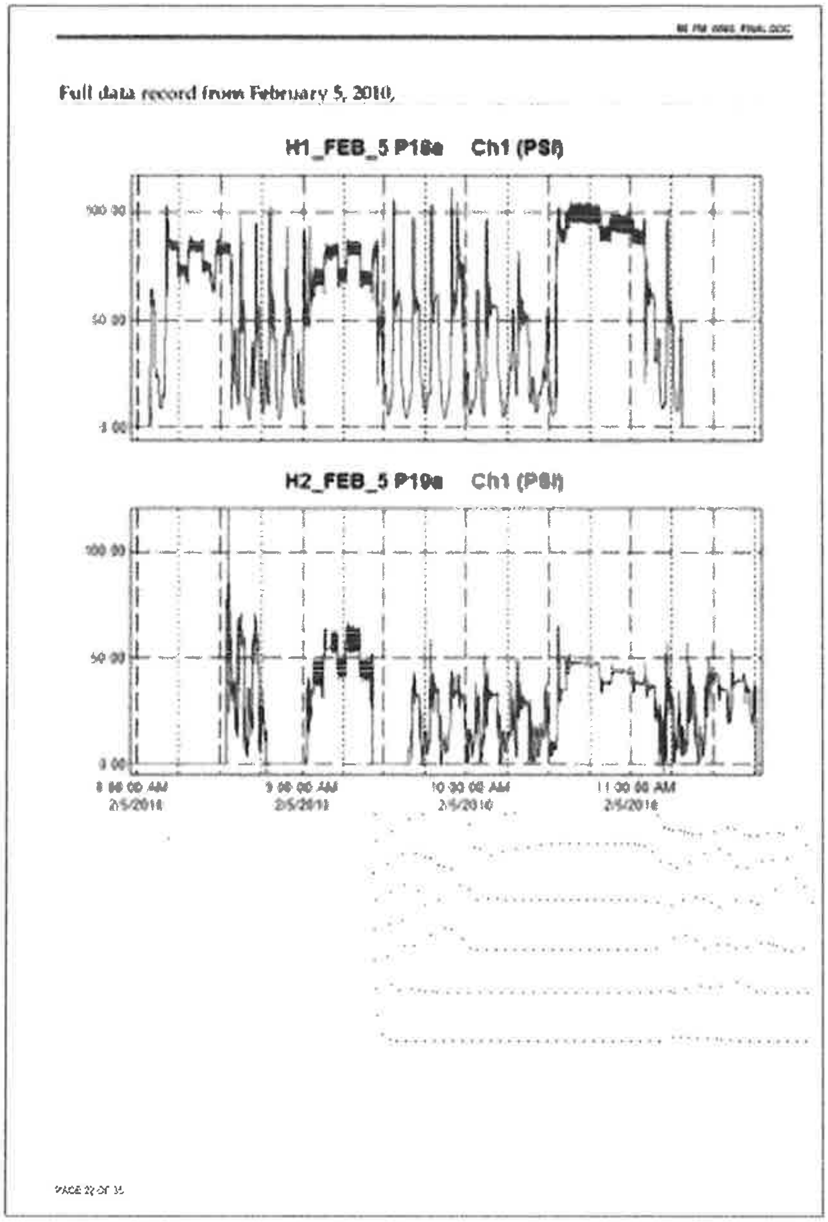
Ultimately the PBS&J report recommends the installation of a 2,000 gallon pressurized bladder surge tank at each of the pump stations B-19, B-20, B-1 and B-6 to prevent the formation of the negative pressures and vapor cavities within the force main. The report goes on to say that installation of bladder surge tanks would adequately mitigate adverse transient pressures during normal startup and shutdown of the pumps.

Expert Opinion of Scott C. Williams, PE on a Construction Project Known as: B-6 Lift Station and Force Main Modifications Brevard County, FL, February 16, 2010

Scott Williams of CH2MHill was asked to evaluate the B-6 Lift Station and Force Main. As part of this evaluation Mr. Williams installed high frequency pressure recorders within the system and recorded a series of pump starts and stops to evaluate the system's transient response. These recordings are critical to understanding the role of adverse transient pressures and the potential for the transient pressures to cause damage or failure to the force main system. Below is a list of findings drawn directly from Mr. Williams report:

- Page 4 of 35, Last Sentence, "Measured pressure data on the B-19 Force Main did not reveal unusual transient pressures or signals"
- Page 11 of 35, First Bulleted Item, "Estimated pump operating points based on measure discharge heads indicate that the pumps are operating normally, and that trapped air/gas pockets are not creating "air binding" on the pipeline."
- Page 19 of 35, Second Paragraph, Last Sentence, "Still, no usual pressure excursions indicating hydraulic shock were measured in either line."
- Page 19 of 35, Last Bulleted Item, "Direct observation of B-19 Force Main in operation at pump stations, valve vaults, and air release manholes on February 5, 2010 did not reveal any auditory or visual indicators or hydraulic shock events within or along the pipeline."

Also from this report the following chart was published showing the recordings of the high frequency transient monitors for February 5, 2010. The H1_FEB_5 P18a recorder was located at the discharge of Lift Station B-19 and monitor H2_FEB_5 P19a was located at various locations along the force main.



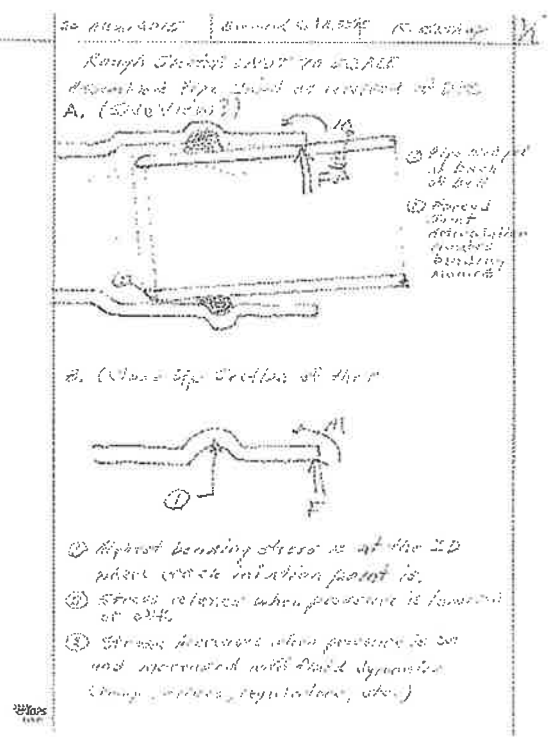
The P18a monitor located at the B-19 Lift Station shows a pressure fluctuation from 0 to 100 psig. This fluctuation in pressure is significant and will cause repetitive dynamic loads (cyclic loads) within the system. However, while the pressure alone is within the allowable limits of the design, the frequency of the pressure fluctuation should be noted because thermoplastic elastomeric materials such as PVC and HDPE are susceptible to fatigue and embrittlement due to work hardening. Cyclic loading also worsens the propagation of cracks within thermoplastic pipe.

Diamond Plastics Corporation Field Report No. FR 3395 Re: Brevard County Utility Department AWWA C 905 DR 25 PVC Sewer Force Main Pipe Samples from Brevard County

The Field Report 3395 submitted by Ron Bishop of Diamond Plastics is an assessment of two failed pipe sections that were shipped to Diamond Plastics for evaluation. The report is summarized on Page 11:

"From our examination of the two samples described in this letter report and the attached formal Field Report #3395, the direct failure is clearly a cyclic fatigue failure. The most likely direct cause of the fractures is the presence of repeated cyclic pressures that combined with a bending force applied at the bell entrance lip by the over assembled spigot that was placed in an angled articulation of the spigot end of the pipe relative to the bell end in order to accommodate curvature to the pipe line as it was installed. A less likely second alternative is that the angle of the change in direction at the joint could have been increased due to localized settlement of the joint if it had not been placed on firm bedding prior to placing the pipe zone material and the subsequent backfill over the pipe before surface loads were applied over the top of pipe."

Mr. Bishop also provided the following sketch as a supplement to the Field Report 3395:



The sketch represents a description of the vertically deflected joint as it was received for testing. The sketch also identifies the force and moment that was identified as the reason for the Bell Split failure.

Pipe Break Analysis

In Figures 1-3, several types of breaks were identified within the evaluated section of force main. The majority of the breaks were identified as a Bell Split. This type of break is identified in the Diamond Plastic Field Report. The Diamond Plastic Field report describes how over-deflection, over-insertion, and applied cyclic loading resulted in the Bell Split failures. However, the reason why the joints were over deflected to a point in which a bending moment was introduced may vary for the following reasons:

- The installer deflected the joint to produce a vertical or horizontal curve to avoid a conflict with a utility, obtain a required lay elevation, or make an alignment change
- The fill material providing structural support around the joint was displaced by fluctuating ground water and migration of fines that weaken the soil support and allowed localized differential settlement
- The fill material was poorly compacted causing differential settlement and over deflection of the joint.
- With three feet of minimum cover the impact loading from heavy equipment could have introduced differential loading resulting in deflection.

In the design of pressurized pipelines utilizing push-on or gasketed joints, it is common to limit the deflection within the pipeline by designing straight runs of pipe on constant slope with varying degrees of fill above the pipe crown to limit the vertical undulation of the pipeline along the alignment. It is also common to limit the allowable joint deflection to some percent (i.e. 50% or 75%) of the manufacturer's specified allowable deflection limit. In design, this ensures that the vertical and horizontal curves designed along the alignment will not introduce excess stress to the push on joint, and as necessary, modifications to the horizontal or vertical alignment during construction may be made. By using 50% of the allowable deflection in design, variations in the alignment can be made in construction while maintaining the deflections within the manufacturer's allowable limits. Two additional types of failures were noted in Figures 1-3. These were "Joint Separation / Slipped Joint" and "Saddle Failed"

The joint separation/slipped joint failures are most likely a result of the following:

- This may be an installation issue resulting from under stabbed or inserted joints installed in hot temperature and covered without allowing for temperature adjustments. Once covered the reduction in temperature will result in significant contraction of PVC pipe. This contraction may result in the pipe pulling away from the joint.
- Gasket or seal failure with longitudinal movement or thrust at unrestrained joints.
    - Shrinkage as described above
    - Differential settlements
    - Movement resulting from thrust at unrestrained joints

Possible reasons for the saddle failures include the following:

- Seal or gasket failure on saddle
- Corrosion of saddle material or bolting assembly causing saddle to release or separate
- Stress concentration at pressure tap leading to longitudinal or fracture 'coupon' failure along pipe segment

<u>Pipe Break Conclusions</u>

Through evaluation of the various reports and record drawings, HDR, Inc has concluded that a portion of the pipeline system should be replaced regardless of the surge modeling results. The multiple Bell Split failures resulting from a combination of over-inserted, over-deflected joints in the presence of cyclic loading will continue to result in failure along the pipeline alignment. Additional surge mitigation may be necessary and may reduce the frequency of failures, but will not resolve the fundamental problem that is resulting in the failures. Consequently, the number of failures over time may be the same even if additional surge mitigation is installed.

HDR, Inc. recommends that USD replace the section of PVC force main on Riverside Drive from Eau Gallie Boulevard to Oakland Avenue. For this replacement the use of USD's standard material for force mains is appropriate. In design of this replacement pipe HDR, Inc. recommends that the following conservative design methodology is used. This includes:

- Limit the allowable deflection in design to 50% of the manufacture's allowable joint deflection,
- Limit the number of vertical and horizontal deflections by using constant pipe slopes and allowing the depth of cover to vary along the alignment,
- Use fittings as necessary and design restraint systems for the working plus transient pressure as a minimum at every joint,
- Provide well-compacted and bedding material for the pipe and consider the use of geo-membrane to maintain the integrity of the soil envelope around the pipe. This will help limit differential settlement from rising and falling groundwater and migration of fines into the pipe zone.

It is also recommended that during construction that experienced inspectors are used to ensure design information and manufacturer's allowable limits are maintained during construction. Considering these conservative design methods and providing detailed inspection during construction will limit the leakage in this replacement.

3.0 Surge Analysis

HDR has collected field data on the operation of the force main system by installing pressure recorders at strategic locations. The recorders were installed at the B-19 lift station and the B-01 lift station. The data collection took place in September and October 2017.

HDR is developing a hydraulic transient model to further analyze issues identified in previous tasks. HDR will provide recommendations and develop an implementation plan based on a hierarchy of importance/priority/cost. The surge data is shown in Appendix A.

4.0 Options Analysis

This report will analyze options for upgrades to the force main system, construction of a re-pump station along the force main route, construction of a new WWTP, and construction of raw wastewater holding tanks. These options will be considered along with conceptual cost estimates for comparison purposes. It is apparent that with some of these long-term options, repair and replacement of the force main on Riverside Drive will still have to be performed. These options could be combined as appropriate as well.

4.1 Force Main Upgrades and B-01 Re-alignment

The section considers upgrades to the portions of the force main system that are PVC and have been experiencing breaks. The USD is completing the replacement of the force main on S. Patrick Drive between Desoto Parkway and Banana River drive in 2017, so that portion is not considered for further analysis. This analysis is focused on the portion of force main on Riverside Drive between Eau Gallie Blvd and Oakland Avenue.

The USD has a current capital improvement project to rehabilitate Lift Station B-01. B-01 is located downstream of B-19 and B-20. The scope includes replacement of existing pumps with larger pumps as well as other miscellaneous modifications. It has been proposed to replace the existing pumps with new pumps that have a design condition of 800 gpm at 191 feet of head. The current pumps are not able to pump down the wet well when lift stations B-19 and/or B-20 are running because the head conditions are too high.

The USD also has a future capital improvement project to take the flow from B-01 off of the B-19 force main system and install a new 24-inch force main to tie into the Lift Station B-06/B-07 force main system. The USD is also considering pipeline improvements upstream and downstream of B-01. Accordingly, the USD has asked HDR to consider the pump upgrades at B-01 in relation to the downstream pipeline improvements.

The following options for the force main system improvements have been evaluated:

1. Increase head on B-01 pumps with no change in downstream force main size
2. Increase head on B-01 pumps, increase force main size to 30-inch when existing 24-inch PVC force main sections on Riverside Drive are replaced
3. Install new 20-inch force main to B-06 as an interconnect to remove flow from S. Patrick force main system
4. Install a parallel 12" force to B-06 to send the B-01 flow to the B-06 system.

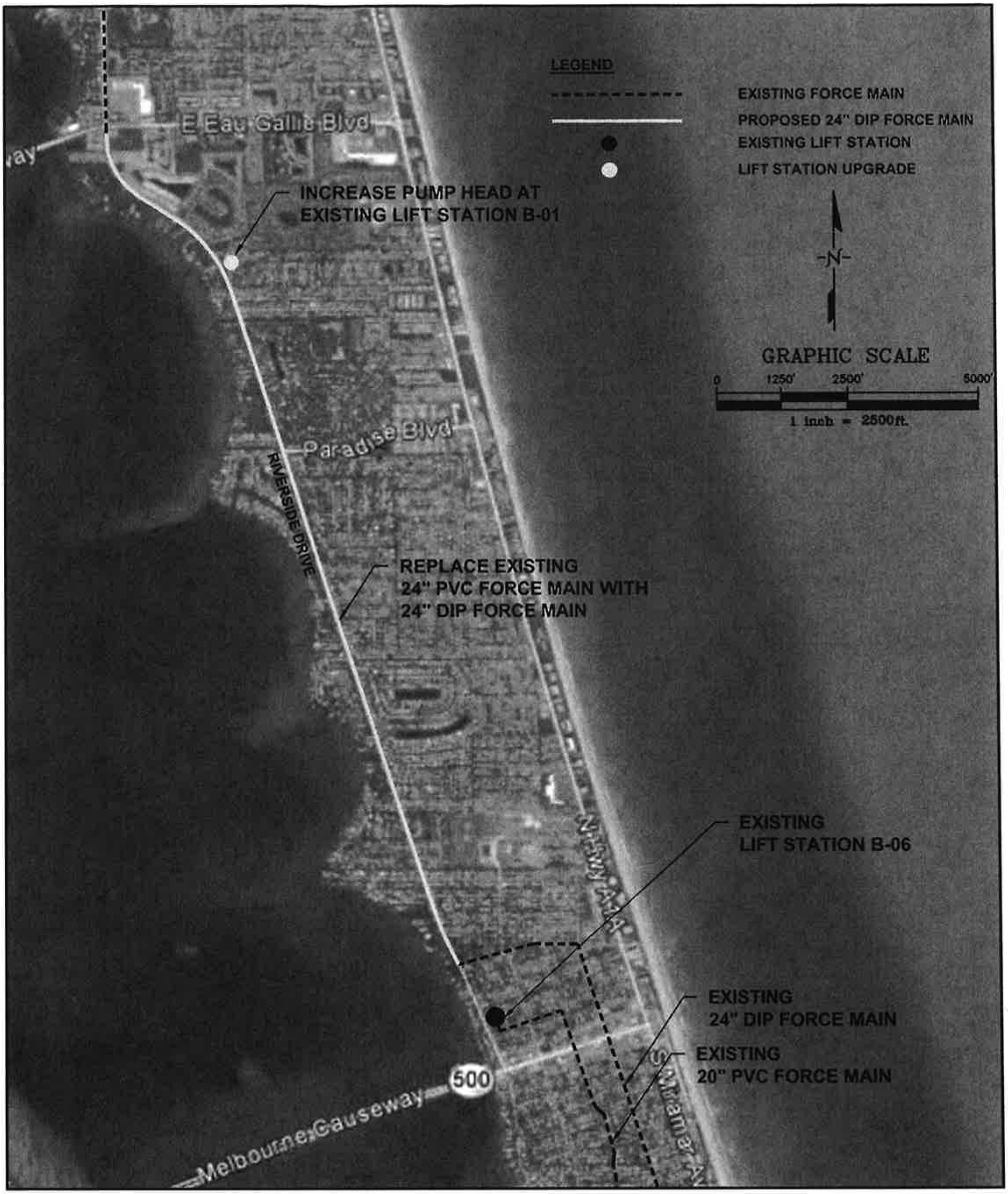
To determine the hydraulic consequences of the above scenarios, HDR has created a hydraulic model of the B-19/B-20/B-01, as well as, the B-06/B-07 force main system. HDR chose EPANET for the model because of the relative simplicity of the system and because it is non-proprietary software. The model assumptions are summarized in Appendix A. The following is a description of each of the four options mentioned above:

- Option 1 – Installing new pumps at the B-01 LS that are able to operate at higher head conditions will allow the lift station to pump down more often. See Figure 4 for location map of this option. The USD has indicated that the current scenario is that the pump station can only get into the system when B-19 or B-20 is off. These higher head pumps at B-01 may cause the existing pumps at B-19 and B-20 to operate at a point on their curve to the left of current operation (slightly less flow, higher head). Overall there would be no significant negative system impacts.

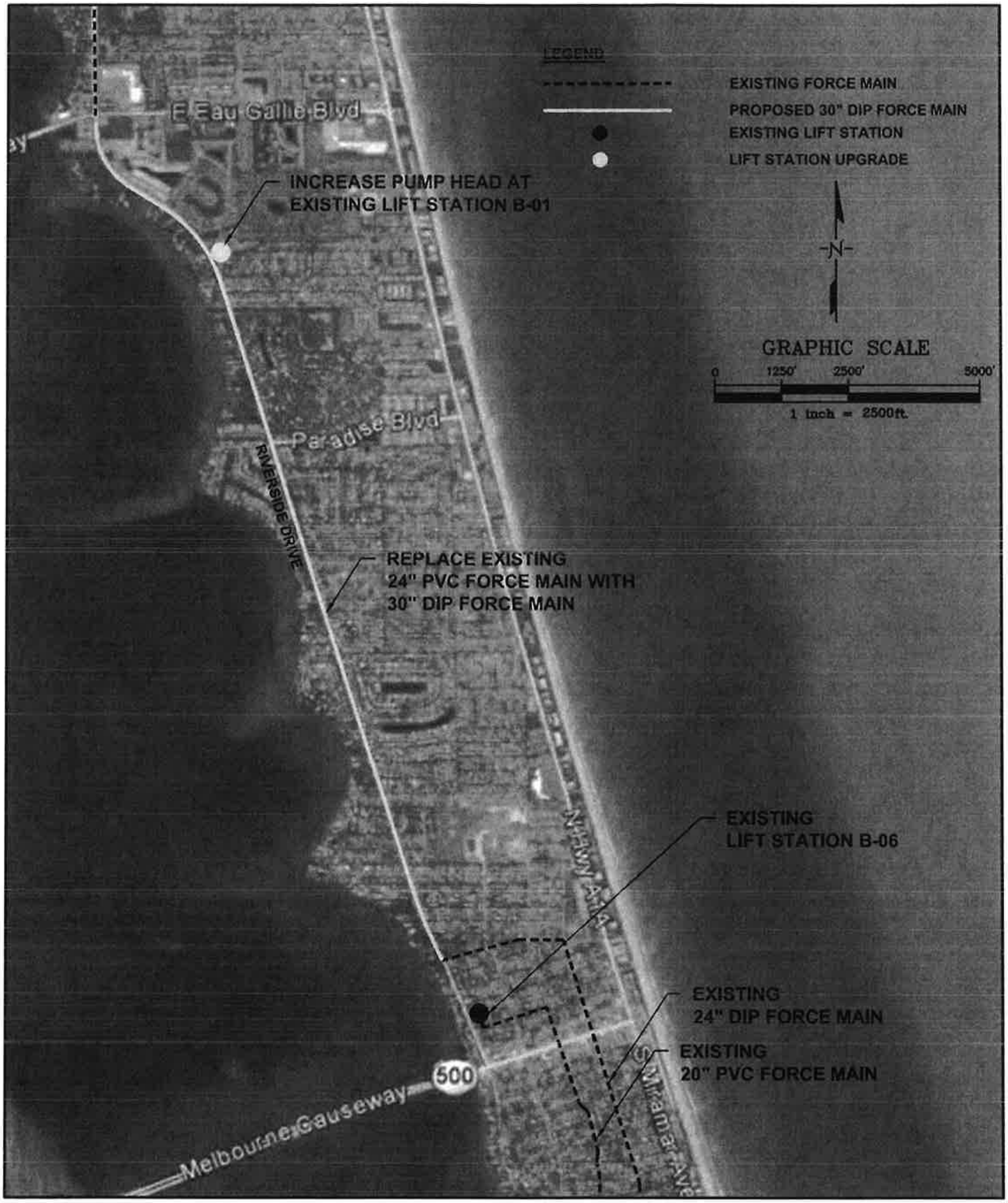
- Option 2- Install new pumps with increased head at B-01 and replace the existing 24-inch PVC force main with 30-inch ductile iron pipe (DIP) on Riverside Drive. This accomplishes Option 1 (allow the B-01 lift station to pump down more often), and also reduces the velocity and head (pressure) in the long run of force main. The run of pipe expected to be replaced runs from Banana River Drive south to the intersection of Oakland Ave and Riverside Drive, a distance of approximately 3.5 miles. Figure 5 shows this pipeline alternative. This would result in lower velocities in this new stretch of force main, and this could translate to somewhat reduced pumping costs. However, the remainder of the force main downstream of this section of proposed 30-inch force main that runs to the wastewater treatment plant would still be 24-inch, and reducing the diameter of a force main on the downstream side is not standard practice.

- Option 3 – Install a new 20-inch force main section from the B-19/B-20/B-01 force main near Oakland Ave to the B-06/B-07 force main system, just downstream of the B-06 pump station, in order to reduce the flow in the B-19 force main system by directing it into the B-06/B-07 force main system. This would reduce the amount of flow in the B-19/B-20/B-01 force main system, which would lessen the velocities in the force main system thereby reducing losses to friction. This would translate in to some energy savings in terms of pumping operations; however, to accomplish this a new 20-inch force main 1,200 feet in length would have to be constructed. . As an Option 3 alternative, the 20-inch force main connection between the B-19/B-20/B-01 force main system and the B-0/B-07 force main system could be moved slightly downstream to connect from the B-19/B-20/B-01 system's 24-inch on N Shannon Ave, run along 2nd Avenue approximately 680 feet, and connect to the B-06/B-07 system's 20-inch force main as it turns south at N Palm Ave. Both of these options are shown in Figure 7. This alternative would shorten the 20-inch interconnect between the two force mains and move the construction from a major road, N Riverside Dr, and move it to a smaller residential street that may permit closure to through traffic during construction.

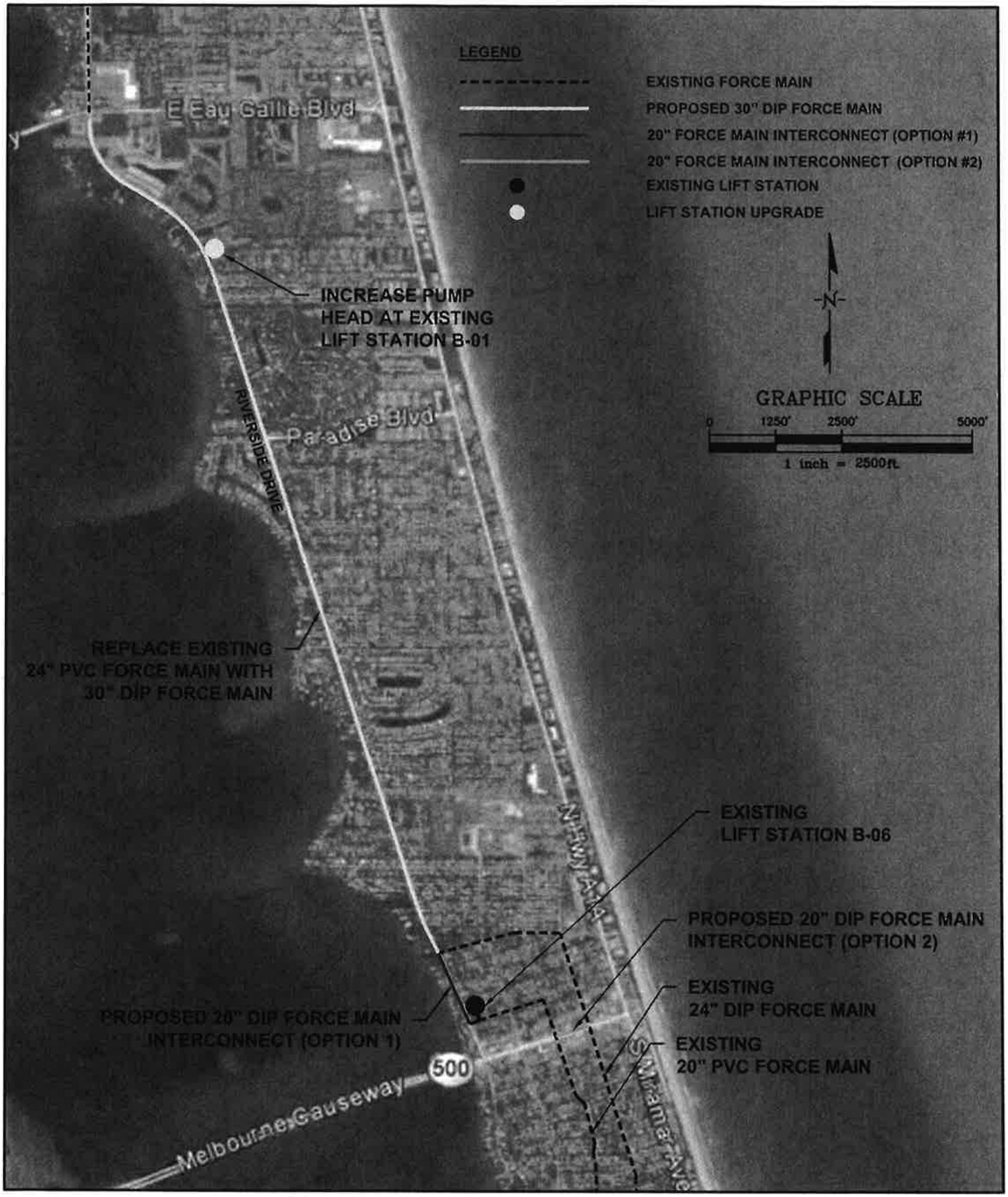
- Option 4 - Install new 12" force main from B-01 Lift Station to B-06 force main system in order to remove flow from the B-19 force main system. This option is shown in Figure 6. This would reduce the amount of flow in the B-19 force main system, which would lessen the velocities in the force main system thereby reducing losses to friction. This would translate in to some



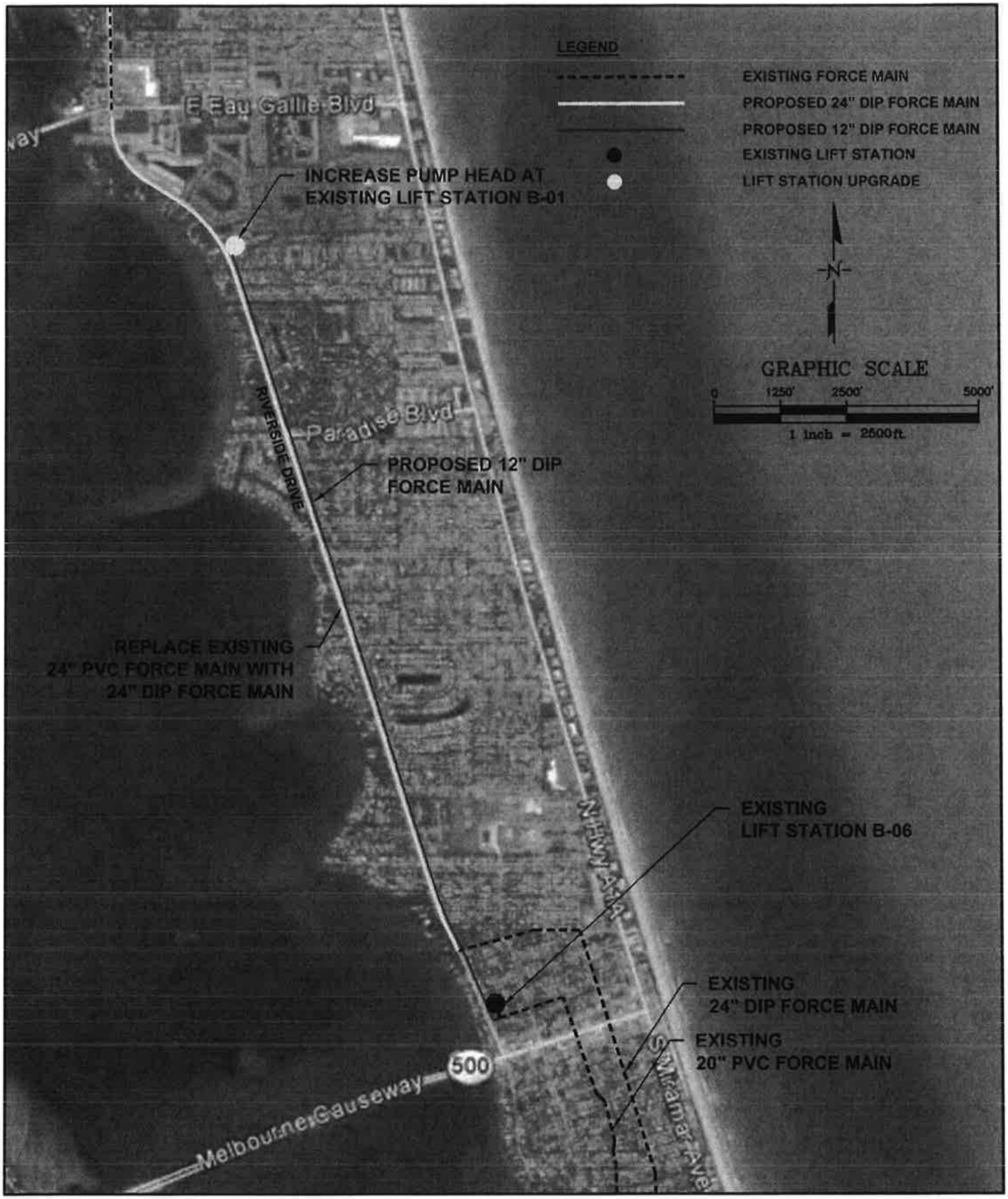
HDR	PROJECT TITLE SOUTH PATRICK RIVERSIDE FORCE MAIN IMPROVEMENTS	PROJECT NUMBER 10049527	REFERENCE SHEET -
	SHEET TITLE **FIGURE-4** UPGRADE PUMPS AT B-01 PUMP STATION	PROJECT MANAGER J.CRIGLER	REFERENCE DOCUMENT -
		DATE 11/17	EXHIBIT NUMBER 4



HDR	PROJECT TITLE SOUTH PATRICK RIVERSIDE FORCE MAIN IMPROVEMENTS	PROJECT NUMBER 10049527	REFERENCE SHEET -
	SHEET TITLE **FIGURE-5** UPGRADE PUMPS AT B-01 PUMP STATION AND UPSIZE FORCE MAIN TO 30"	PROJECT MANAGER J.CRIGLER	REFERENCE DOCUMENT -
		DATE 11/17	EXHIBIT NUMBER 5



PROJECT TITLE SOUTH PATRICK RIVERSIDE FORCE MAIN IMPROVEMENTS	PROJECT NUMBER 10049527	REFERENCE SHEET -
SHEET TITLE **FIGURE-6** UPGRADE PUMPS AT B-01 PUMP STATION, UPSIZE FORCE MAIN TO 30", AND INSTALL 20" INTERCONNECT FORCE MAIN	PROJECT MANAGER J.CRIGLER	REFERENCE DOCUMENT -
	DATE 11/17	EXHIBIT NUMBER 6



HDR	PROJECT TITLE SOUTH PATRICK RIVERSIDE FORCE MAIN IMPROVEMENTS	PROJECT NUMBER 10049527	REFERENCE SHEET -
	SHEET TITLE **FIGURE-7** UPGRADE PUMPS AT B-01 PUMP STATION AND INSTALL 12" FORCE MAIN TO B-06	PROJECT MANAGER J.CRIGLER	REFERENCE DOCUMENT -
		DATE 11/17	EXHIBIT NUMBER 7

energy savings in terms of pumping operations; however, to accomplish this a new 12" force main 3 miles long would have to be built at a cost of approximately $1.5 million. This large capital expenditure would not justify the minor energy savings realized by the new 12" force main. The cost mentioned above does not include any upgrade costs at B-06 or B-07. This option was not modeled and is considered cost prohibitive because of the marginal benefits obtained at a substantial capital cost.

System Scenarios

Both high head and low head conditions (scenarios) were analyzed in order to determine the possible operational total dynamic head (TDH) range of the B01 pump station system. The TDH range was utilized to select efficient pumps to operate at the desired flow rate. Design flow rates of 800 gpm and 1,000 gpm were analyzed for the high head conditions. The design flow rate of 800 gpm was analyzed for the low head conditions, as this would generate a lower head than the 1,000 gpm alternative. Three system alternatives were analyzed:

1. Existing Condition,
2. Upsize of the 24-inch PVC force main sections to 30-inch,
3. Upsize of the 24-inch PVC force main sections to 30-inch with a 20-inch interconnect to B-06/B-07 force main system.

Existing Conditions

The existing condition scenarios represent the system as it is today with B-01 contributing flow changed to 800 gpm and 1,000 gpm for respective scenarios. In this scenario, the existing 24-inch PVC sections along N Riverside Dr are kept as is. For one sub-scenario, both B19 and B20 triplex stations were allowed to operate with two pumps running, each, and, thus, generated flow based on the head condition of the system. For comparison, another sub-scenario was run where the B-19 and B-20 pump stations were only permitted to generate flow equal to their design flow and no more, imitating variable frequency drive condition limit.

Future: 30-inch Force Main

Under this system scenario, the existing 24-inch PVC sections along N Riverside Dr are upsized to 30-inch ductile iron pipe (DIP). A total of 19,712 linear feet (LF) of 24-inch PVC was replaced with 30-inch DIP in this scenario. Both B19 and B20 triplex stations were allowed to operate with two pumps running, each, and, thus, generated flow based on the head condition of the system.

Future: 30-inch Force Main with 20-inch Interconnect

Under this system scenario, the existing 24-inch PVC sections along N Riverside Dr are upsized to 30-inch ductile iron pipe (DIP). A total of 19,712 linear feet (LF) of 24-inch PVC was replaced with 30-inch DIP in this scenario. Additionally, a 1,200 LF, 20-inch force main was added along N Riverside Dr from Oakland

Avenue to Second Avenue connecting the B-19/B-20/B-01 24-inch force main to the B-06/B07 20-inch force main. Both B19 and B20 triplex stations were allowed to operate with only one pump running, each, and, thus, generated flow based on the head condition of the system. Only one pump, each, was allowed to operate at B-19 and B-20 such that the stations would generate flow similar to their design condition. The lower head conditions at these stations resulting from the system improvements permitted the stations to run on one pump and still produce close to if not more than the design flow. For the high head scenarios, B-06 and B-07 pump stations were constrained to pump at their design flow rates of 1,940 gpm and 1,375 gpm, respectively. The inclusion of the 20-inch interconnect results in additional flow being conveyed through the B-06/B-7 force main system, the head condition at these pump stations would not allow the existing pumps to generate their design flow. If the interconnect is implemented in the County's system an in depth review of these stations will be needed.

An alternative system scenario was analyzed where the 20-inch interconnect alignment was moved from N Riverside Dr to 2nd Avenue. The 680 LF, 20-inch interconnect is further downstream from the alignment proposed along N Riverside Dr and thus the overall head condition as seen from the B-19/B-20/B-01 force main system pump stations are slightly higher, however, the difference is fairly negligible.

Results

The results of the EPAnet modeled scenarios described below are provided in Table 2 and Table 3 for high head and low head scenarios analyzed, respectively. As would be expected, replacing the 24-inch PVC force main sections in the B-19/B-20/B-01 force main system lowers the system head allowing the existing pumps at the stations to pump a greater flow. With the interconnect in place B-19/B-20/B-01 force main system head drops even lower, allowing for the shutdown of one pump at each of the major stations (B-19 and B-20) while allowing them to pump at or near their design flow rate. Under existing conditions where B-01 is designed to convey 800 gpm the B-01 proposed pumps would have to be able to pump at between 87 and 218 ft of TDH. Likewise, where the 24-inch PVC force main sections are replaced with 30-inch DIP the TDH range as seen at the B-01 pump station could range between 79 and 186 ft at a flow rate of 800 gpm. With the inclusion of the 20-inch interconnect to the B-06/B-07 force main system along N Riverside Dr, the system head at B-01 drops even further to a range of between 62 and 138 ft of TDH. Head conditions for the other major stations, B-19, B-20, B-06, and B-07 are also provided in Table 2 and Table 3, as well as, the flow conveyed from the B-19/B-20/B-01 force main system into the B-06/B-07 force main system via the 20-inch interconnect.

Future Surge Analysis

As the B-01 pump station is upgraded, it is recommended that the pressure monitoring and surge modeling be performed with the new pumps selected for B-01, and taking in to account the pipeline recommendations mentioned in this report. This will provide the most accurate representation of future surge conditions.

Brevard County Riverside Drive 24" FM Replacement
0% Conceptual Cost Estimate

ITEM Number	SUMMARY OF PAY ITEMS	QUANTITY	UNIT	AVERAGE UNIT PRICE	TOTAL AMOUNT
1	Mobilization	1	LS	$80,000.00	$80,000.00
2	Layout and Asbuilt	1	LS	$41,760.00	$41,760.00
3	Erosion Control	1	LS	$17,500.00	$17,500.00
4	Dewatering	1	LS	$100,000.00	$100,000.00
5	24" Ductile Iron FM	17700	LF	$230.00	$4,071,000.00
6	24" x 24" Tapping Sleeve & Valve	2	EA	$50,000.00	$100,000.00
7	24" Gate Valve	8	EA	$32,000.00	$256,000.00
8	2" Air Release Valve	4	EA	$11,300.00	$45,200.00
9	24" 45° DI Elbow	20	EA	$7,000.00	$140,000.00
10	24" 11.25° DI Elbow	8	EA	$6,500.00	$52,000.00
11	Canal/Ditch Crossing	2	EA	$80,000.00	$160,000.00
12	Lift Station Connections	4	EA	$45,000.00	$180,000.00
13	Demo and Remove Concrete Sidewalk	20000	SF	$1.25	$25,000.00
14	Sidewalk Replacement	20000	SF	$5.50	$110,000.00
15	Roadway Cut/Patch	1200	SY	$110.00	$132,000.00
16	Concrete Driveway	600	SY	$50.00	$30,000.00
17	Construct Curbing	800	LF	$40.00	$32,000.00
18	Striping	1	LS	$4,000.00	$4,000.00
19	Abandon 24" PVC FM	17700	LF	$6.00	$106,200.00
20	Remove and Replace Pipe Bedding	300	CY	$24.00	$7,200.00
21	Remove and Replace Pipe Backfill	300	CY	$26.00	$7,800.00
22	Seeding	3000	SY	$2.00	$6,000.00
23	Sodding	7000	SY	$4.00	$28,000.00
24	Landscaping	1	LS	$50,000.00	$50,000.00
25	Density Testing	1	LS	$27,200.00	$27,200.00
26	Pressure Testing	1	LS	$12,000.00	$12,000.00
27	Maintenance of Traffic	1	LS	$17,500.00	$17,500.00
	Sub-total				$5,838,360.00
	Contingency (20%)				$1,167,672.00
	TOTAL*				**$7,006,032.00**

Brevard County Riverside Drive 30" FM Replacement
0% Conceptual Cost Estimate

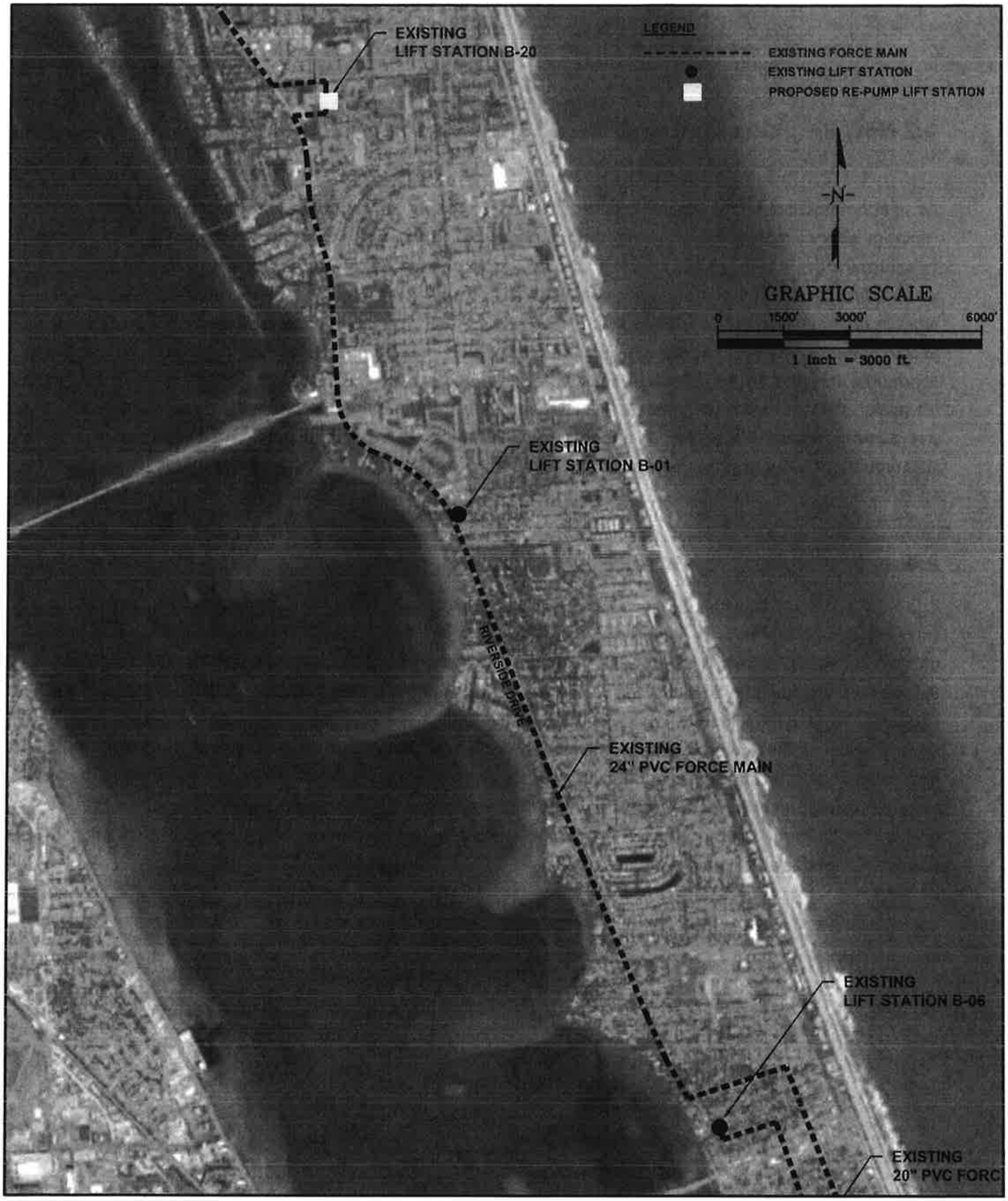
ITEM Number	SUMMARY OF PAY ITEMS	QUANTITY	UNIT	AVERAGE UNIT PRICE	TOTAL AMOUNT
1	Mobilization	1	LS	$40,000.00	$40,000.00
2	Layout and Asbuilt	1	LS	$41,760.00	$41,760.00
3	Erosion Control	1	LS	$12,500.00	$12,500.00
4	Dewatering	1	LS	$110,000.00	$110,000.00
5	30" Ductile Iron FM	17700	LF	$245.00	$4,336,500.00
6	30" x 24" Tapping Sleeve & Valve	2	EA	$56,000.00	$112,000.00
7	30" Gate Valve	8	EA	$42,000.00	$336,000.00
8	2" Air Release Valve	4	EA	$9,000.00	$36,000.00
9	30" 45° DI Elbow	20	EA	$8,500.00	$170,000.00
10	30" 11.25° DI Elbow	8	EA	$7,500.00	$60,000.00
11	Canal/Ditch Crossing	2	EA	$90,000.00	$180,000.00
12	Lift Station Connections	4	EA	$55,000.00	$220,000.00
13	Demo and Remove Concrete Sidewalk	20000	SF	$1.25	$25,000.00
14	Sidewalk Replacement	20000	SF	$5.50	$110,000.00
15	Roadway Cut/Patch	1200	SY	$110.00	$132,000.00
16	Concrete Driveway	600	SY	$50.00	$30,000.00
17	Construct Curbing	800	LF	$40.00	$32,000.00
18	Striping	1	LS	$4,000.00	$4,000.00
19	Abandon 24" PVC FM	17700	LF	$8.00	$141,600.00
20	Remove and Replace Pipe Bedding	300	CY	$24.00	$7,200.00
21	Remove and Replace Pipe Backfill	300	CY	$26.00	$7,800.00
22	Seeding	3000	SY	$2.00	$6,000.00
23	Sodding	7000	SY	$4.00	$28,000.00
24	Landscaping	1	LS	$50,000.00	$50,000.00
25	Density Testing	1	LS	$27,000.00	$27,000.00
26	Pressure Testing	1	LS	$12,000.00	$12,000.00
27	Maintenance of Traffic	1	LS	$17,500.00	$17,500.00
	Sub-total				$6,284,860.00
	Contingency (20%)				$1,256,972.00
	TOTAL*				**$7,541,832.00**

4.2 New Re-pump Station

As an alternative option to reducing the force main system pressure, consideration was given to replacing either Lift Station B-20 or Lift Station B-01 and construction a new re-pump station. In order to determine the viability of this option, a site analysis was done first to determine if adequate space was available to accommodate a larger pump station and an electrical building. Based on the initial site analysis the determination was made that the B-01 lift station site lacks the space needed to accommodate a new re-pump station. In addition, the B-01 lift station site is located in a visible residential area and the aesthetics of the new station would most likely cause issues. The existing B-20 lift station site was evaluated and determined to have the space required to consider the construction of a re-pump station for the system. In addition, the B-20 site is located off of the main road in an area that would not cause aesthetic concerns.

B-20 Re-Pump Station Consideration

Currently, the B-20 lift station discharges through a 20" force main and manifolds in to the existing 24" force main trunk line that run down South Patrick drive. Approximately 3.5 miles upstream of the B-20 lift station tie-in location is Lift Station B-19. If B-20 were converted into a re-pump station then Lift Station B-19 would pump directly into the new B-20 Lift Station. The new B-20 lift station would then re-pump all flow approximately 9 miles south to the South Beaches Regional WWTP. The potential benefit of having the flow from B-19 re-pumped from Lift Station B-19 would be the reduction in total head seen at both lift stations, reduction in overall energy consumption, and the reduction of overall system pressure. In order to determine the extent of these benefits, a system model is being developed. Once the modeling results are complete a determination can be made as to whether or not the benefits of this alternative is viable.



PROJECT TITLE SOUTH PATRICK RIVERSIDE FORCE MAIN IMPROVEMENTS	PROJECT NUMBER 10049527	REFERENCE SHEET -
SHEET TITLE **FIGURE-8** CONVERT LIFT STATION B-20 TO RE-PUMP LIFT STATION	PROJECT MANAGER J.CRIGLER	REFERENCE DOCUMENT -
	DATE 11/17	EXHIBIT NUMBER 8

4.3 New WWTP at B-20 Site

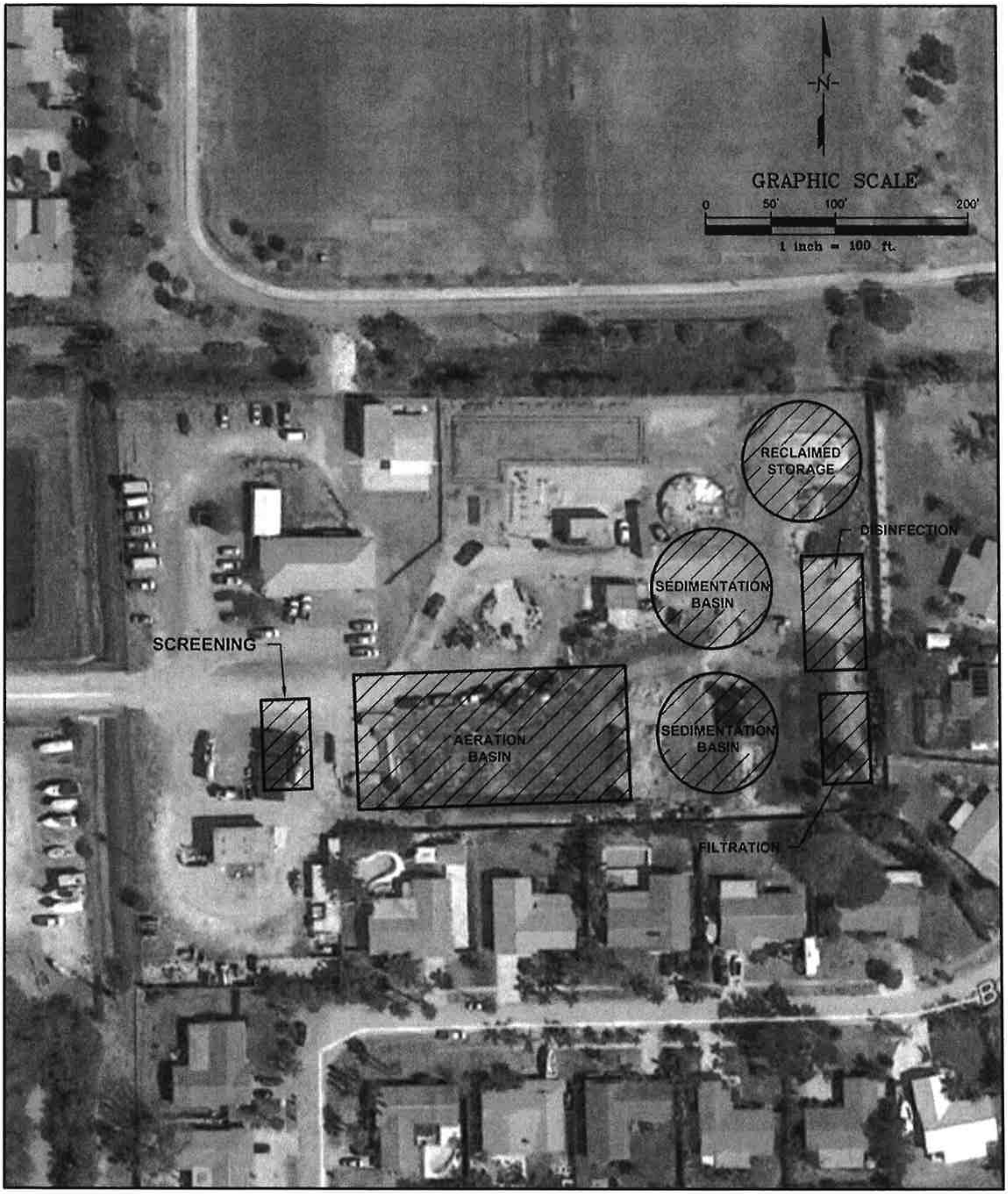
The USD has considered the option of constructing a new wastewater treatment plant in the South Beaches service area that could produce reclaimed water for the customers located on the island, as well as eliminate the need for a pipeline carrying raw wastewater. The concept is to develop the property currently owned by USD at the B-20 lift station site into a new wastewater treatment plant (WWTP) with the capability to produce reclaimed water as an effluent disposal method. The treated effluent would then be pumped into the existing wastewater pipelines from B-20 to the South Beaches WWTP. The reclaimed water could be utilized by customers along that route. Any reclaimed wastewater not delivered to customers would be sent to the South Beaches WWTP for disposal in the existing deep injection well(s). The following system improvements would be required for this alternative:

- Construct 12" FM from B-01 to B-06 FM (in existing 24" HDPE conduit). This would allow the raw wastewater from the B-01 basin to get to the South Beached WWTP
- Construct 6.0 MDG treatment plant at IHB site using similar design/layout from South Central WWTP. This will provide consistency with operators and plant personnel
- Divert flows from B-19 and B-20 to new plant. The flow for B-20 already comes to the proposed WWTP site, and all the flow from B-19 could be diverted with a short force main connection from S. Patrick Drive to the new plant influent
- Pump reclaimed water using the existing 24" Riverside FM to the South Beaches WWTP. While this pipeline currently has issues with main breaks, it would operate at a lower pressure than current operating pressure. In addition, spills from a reclaimed line into the Indian River Lagoon would be less harmful than a raw wastewater spill.
- Connect Riverside FM to existing injection well. Most reclaimed water is injected into a 2900 ft deep injection well at the treatment plant site
- Connect S-06 & S-07 pumping stations to the B-19 FM eliminating re-pumping, and eliminating need for upgrades to FM from S-06 to B-19
- Repurpose B-20 lift station to reclaimed water pump station. The influent

Currently Seven million gallons of wastewater is reclaimed every day at the South Beach plant and fewer than two million gallons are used for beneficial purposes. Additional capacity at South Beach plant provides opportunity for sewer service all the way to Sebastian Inlet (septic tank elimination). The following table lists some advantages/disadvantages of a new WWTP at this location.

Advantage	Disadvantages
Makes reclaimed water available from Satellite Beach to South Melbourne Beach	High Capital Cost
Lowers pressures on beachside force mains	Proximity to ocean
Adds treatment capacity to beachside system (4.5 MGD required to complete service area)	Vulnerable to Storms & Storm Surge
Reclaimed water trunk line and reclaimed pumping station already in place (Repurposed Riverside FM and B-20 pumping station)	Elevation concerns (flooding)
Creates "fail-safe" for Riverside force main break (reclaimed water – not sewage)	No pipelines to distribute the water to users

A preliminary layout of a new WWTP is shown in Figure 9. This layout is similar to the technology used at the S. Beaches plant, which will allow ease of operation for existing staff. Also, a conceptual cost estimate has been developed and is shown in Table 6.



PROJECT TITLE SOUTH PATRICK RIVERSIDE FORCE MAIN IMPROVEMENTS	PROJECT NUMBER 10049527	REFERENCE SHEET -
SHEET TITLE **FIGURE-9** B-20 LIFT STATION SITE WASTEWATER TREATMENT PLANT OPTION	PROJECT MANAGER J.CRIGLER	REFERENCE DOCUMENT -
	DATE 11/17	EXHIBIT NUMBER 9

New WWTP at B-20 Site
0% Conceptual Cost Estimate

ITEM Number	SUMMARY OF PAY ITEMS	QUANTITY	UNIT	AVERAGE UNIT PRICE	TOTAL AMOUNT
1	Design 6.0 MGD Plant	1	LS	$1,600,000.00	$1,600,000.00
2	6.0 MGD Plant	1	LS	$21,000,000.00	$21,000,000.00
3	12" FM from B-01 to B-06	1	LS	$1,800,000.00	$1,800,000.00
4	Piping at South Beaches Plant	1	LS	$350,000.00	$350,000.00
5	Piping Changes at B-20	1	LS	$150,000.00	$150,000.00
6	S-6 & S-17 Piping to B-19 FM	1	LS	$200,000.00	$200,000.00
7	B-20 conversion to Reclaimed Water PS	1	LS	$250,000.00	$250,000.00
	Sub-total				$25,350,000.00
	Contingency (15%)				$3,802,500.00
	TOTAL*				**$29,152,500.00**

4.4 Off-Line Wastewater Storage at B-20

The South Beaches collection system experiences a large amount of inflow/infiltration. The gravity collection system contains significant amounts of vitrified clay gravity pipe that has reached its design life. Significant storm events place a burden on the system, and the existing pump station and force main network becomes overburdened during large storm events. This can lead to SSO's. In addition to the system upgrades mentioned as options in this report, off-line storage of raw wastewater at the B-20 lift station site would provide some operational flexibility during periods of high flows and could mitigate SSO's daring these events. It would also provide additional flexibility when the station or force main systems requires being taken out of service for maintenance or repair. The B-20 lift station has insufficient wetwell storage, and the USD indicated that it overflows within 20 minutes of outage

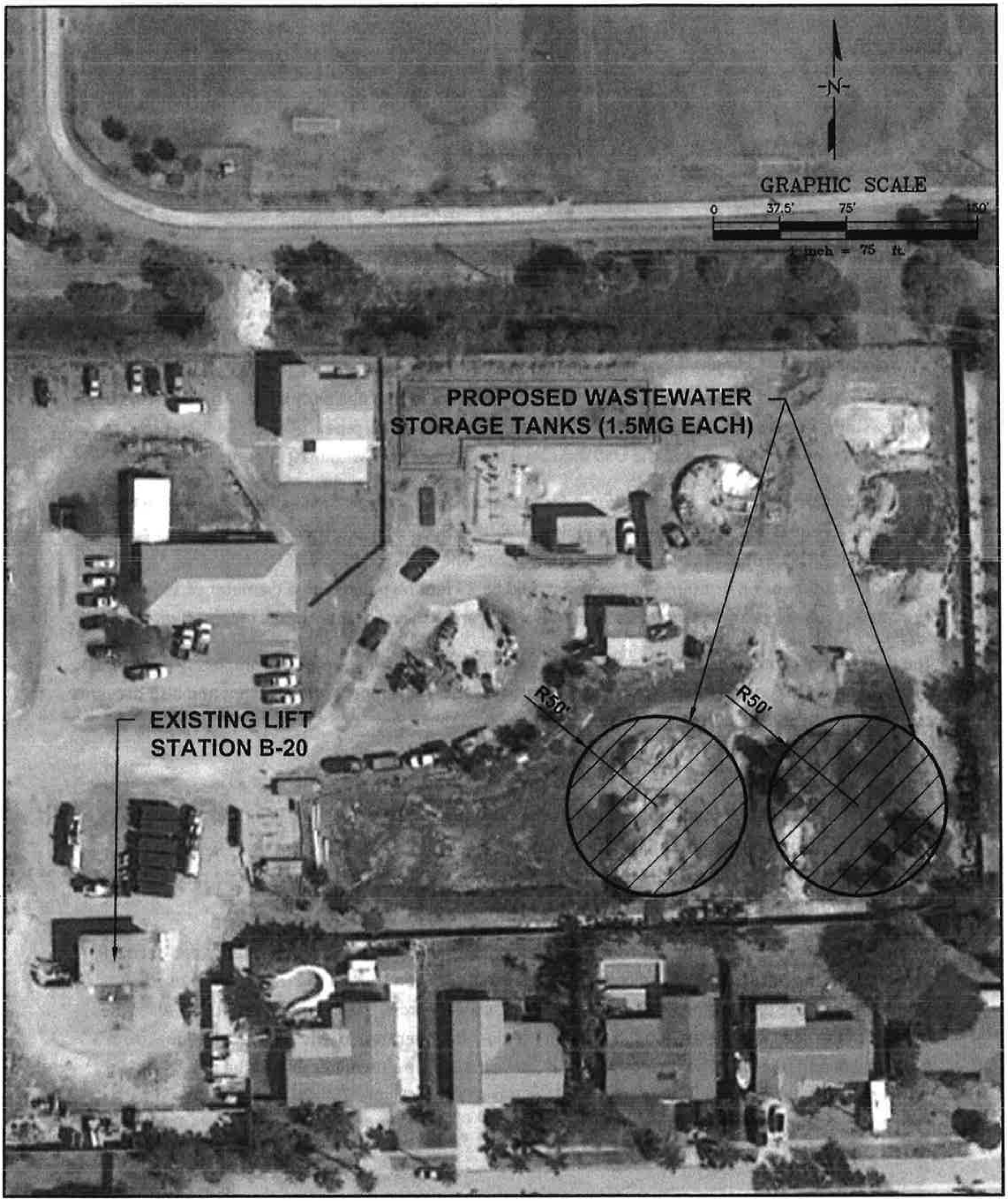
The B-20 lift station site has adequate room for construction of storage holding tanks. Based on the flows to B-19 and B-20 lift stations, two holding tanks of 1.5 million gallons each could be constructed on the B-20 site. This would provide approximately one to one and an half day of storage. The approximate dimension s of tanks of this size would be 25 feet high and with a diameter of 100 feet. The tanks would be pre=stressed concrete tanks which are proven for the storage of wastewater.

The operational protocol for these tanks would be that they are always emptied completely each time they are used. This ensures that storage capacity is available for the next rainfall event and also prevents consolidation of sludge in the base of the tank.

Such tanks should incorporate the following features:

- Facilities for re-suspension of solids as the tanks are drawn down,
- Tank mixer unit or air/water scour pumps;
- Facilities for cleaning of the tank floor on draw-down of the contents, so sludge can be automatically drawn off from the conical sump at the invert of the tank.

Raw wastewater storage will present some fundamental challenges, including aesthetics and securing support from the neighborhoods adjacent to the B-20 site. Even though the tanks will be used infrequently, these types of projects can be met with concern from the nearby residents. Also, the location of the site, while large enough for construction and operation of the tanks, is not ideal from a vulnerability perspective. The proximity to the ocean makes the site vulnerable to the effects of hurricanes and storm surge. A proposed site layout is shown in Figure 10, and a conceptual cost estimate has been developed and is shown in Table 7 .



HDR	PROJECT TITLE SOUTH PATRICK RIVERSIDE FORCE MAIN IMPROVEMENTS	PROJECT NUMBER 10049527	REFERENCE SHEET -
	SHEET TITLE **FIGURE-10** B-20 LIFT STATION SITE WASTEWATER STORAGE TANKS	PROJECT MANAGER J.CRIGLER	REFERENCE DOCUMENT -
		DATE 11/17	EXHIBIT NUMBER 10

Wastewater Storage at B-20
0% Conceptual Estimate

ITEM Number	SUMMARY OF PAY ITEMS	QUANTITY	UNIT	AVERAGE UNIT PRICE	TOTAL AMOUNT
1	Earthwork	1	LS	$500,000.00	$500,000.00
2	1.5 Million Gallon Concrete Tank	2	EA	$1,750,000.00	$3,500,000.00
3	Site Piping	1	LS	$450,000.00	$450,000.00
4	Valves	1	LS	$250,000.00	$250,000.00
5	Site Work	1	LS	$175,000.00	$175,000.00
	Sub-total				$4,875,000.00
	Contingency (15%)				$731,250.00
	TOTAL*				**$5,606,250.00**

5.0 Recommendations

The proposed re-pump station is a viable alternative if Lift Station B-20 were at the end of its design life. This scenario will lower the head in the system somewhat, but does not negate the need to upgrade the force main on Riverside Drive. Any energy savings realized by lower head pumps at B-19 would be offset by the energy requirement to re-pump the wastewater at B-20.

Constructing a new WWTP at the B-20 site would provide beneficial reuse to customers at the beach, but the high capital cost of this option makes it impractical at this time. There are also concerns about locating another WWTP in an area vulnerable to storms and storm surge.

Raw wastewater storage tanks at the B-20 site would provide additional flexibility for operation of the system during significant rain events and for maintenance/repair, but there are significant challenges to the siting of these tanks relative to the surrounding neighborhoods and the vulnerability of the site location. Furthermore these tanks would not negate the need to upgrade the force main on Riverside Drive either.

Therefore, based on the analysis of the pipe break data and the hydraulic modeling results, HDR recommends at this time that the **USD replace the section of 24" force main on Riverside Drive from Banana River Drive south to the intersection of Oakland Ave and Riverside Drive, with 30" DI force main. In addition, based on the results of the surge analysis report pressure surge tanks should be installed at Lift Station B-19 and Lift Station B-20.** If additional funding becomes available, the USD should re-visit the options for off-line storage or a re-pump station, as these would improve operational flexibility.

Furthermore, if additional operational flexibility is desired, USD should consider constructing the interconnect between the two force main systems. This would require the upgrade of Lift Stations B-06 and B-07 but would improve the flow conditions in the B-19 system, while also providing operational flexibility for main breaks or force main maintenance/repair. The interconnect could be operated in either the normally closed or normally open condition, depending on the desired outcome. If intended to alleviate high pressures in the B-19 system, it could be operated in the normally open condition, whereas if it is intended to be used as an alternate transmission main in certain scenarios then the valve would be normally closed.

APPENDIX

Existing System Scenario: Major Components

Links: Pipes and Pumps

<u>Pipes:</u>

- B19_InfluenPipe: Gravity line flowing into B19. The length value of 1,000 LF does not affect calculations.
- 20" DIP: Force main just down stream of B19 and runs from B19 and south along Riverside Drive.
- 20" PVC: Force main section down stream of 20" DIP and runs south along Riverside Drive.
- B20_Inflow_16": Gravity line flowing into B20. The length value of 1,000 LF does not affect calculations.
- B20_18"FM: Force main section from B20 that ties into the 20"/24" at Riverside Drive.
- 24"_PVC_1: Force main section down stream of B20 tie in and runs south along Riverside Drive.
- 24"_DIP: Force main section down stream of 24"_PVC_1 and runs south along Riverside Drive.
- 24"_PVC_2: Force main section down stream of 24"_DIP and runs south along Riverside Drive.
- B1_18"GravitySewer: Gravity line flowing into B1. The length value of 1,000 LF does not affect calculations.
- 12"DIP_B1: Force main section from B1 that ties into the 20"/24" at Riverside Drive.
- 24"_PVC_3: Force main section down stream of 24"_PVC_2 and runs south along Riverside Drive.
- 24"_DIP_2: Force main section down stream of 24"_PVC_3 and runs south along Riverside Drive to the SBWRF.

Table 1 Force Main Attributes

Force Main Section	Minor Loss Coefficient (K)	Hazen-William Friction Coefficient (C)	Material	Diameter (inch)	Length (Ft), Model
20" DIP	12.00	100	DIP	20	15,491
20" PVC	1.75	110	PVC	20	2,981
B20_18"FM	0.00	100	DIP	18	1,000
24"_PVC_1	2.75	110	PVC	24	2,205
24"_DIP	2.75	100	DIP	24	5,144
24"_PVC_2	3.00	110	PVC	24	3,590
12"DIP_B1	0.00	100	DIP	12	150
24"_PVC_3	10.50	110	PVC	24	13,917
24"_DIP_2	19.50	100	DIP	24	25,730

Note: Influent gravity lines excluded from table, attributes are non-real world and do not affect model results.

Pumps:

- B1_P1: Lead pump for B1 pump station ABS AFP 1501310 mm, set open.
- B1_P2: Lag pump for B1 pump station, ABS AFP 1501310 mm, set closed as B1 is a duplex station.
- B19_P1: Lead pump for B19 pump station FLYGT C3231-63-430, set open.
- B19_P2: Lag pump for B19 pump station FLYGT C3231-63-430, set open as B19 is a triplex station.
- B19_P3: Lag-lag pump for B19 pump station FLYGT C3231-63-430, set open as B19 is a triplex station.
- B20_P1: Lead pump for B20 pump station FLYGT C3231-63-430, set open.
- B20_P2: Lag pump for B20 pump station FLYGT C3231-63-430, set open as B20 is a triplex station.
- B20_P3: Lag-lag pump for B20 pump station FLYGT C3231-63-430, set open as B20 is a triplex station

Wet Wells:

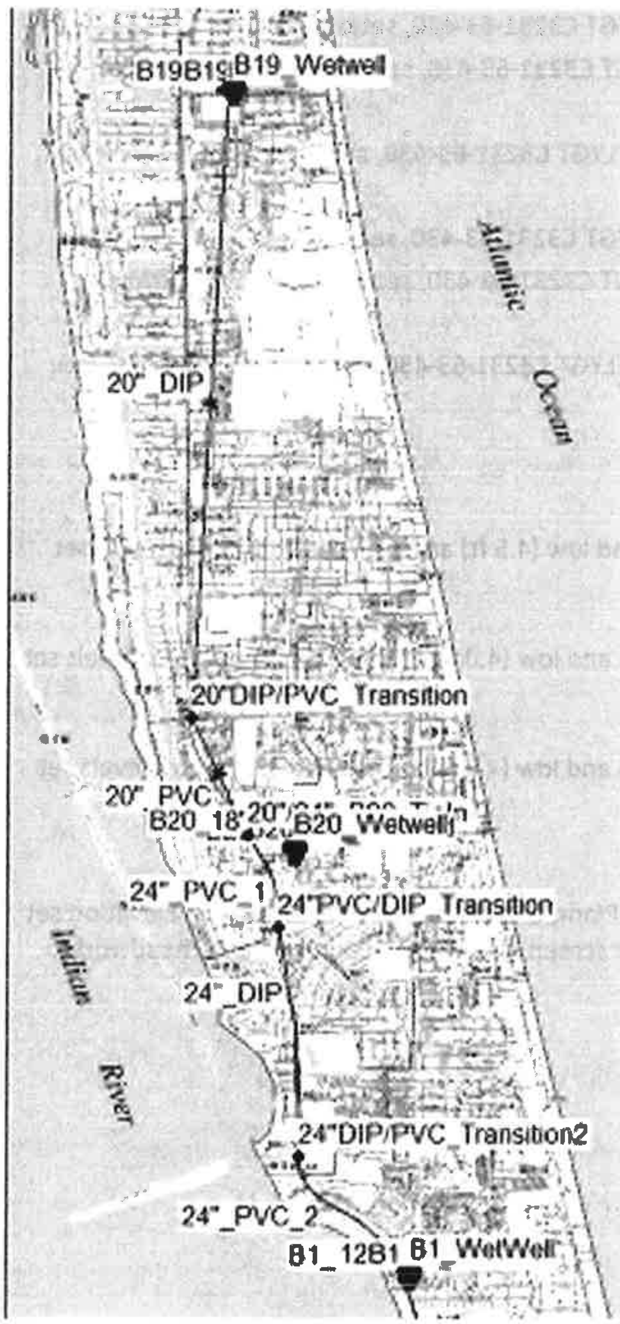
- B1_WetWell: B1 wet well, depth elevation and low (4.5 ft) and high (11.5 ft) water levels set based on asbuilts.
- B19_WetWell: B19 wet well, depth elevation and low (4.0 ft) and high (11.0 ft) water levels set based on asbuilts.
- B20_WetWell: B19 wet well, depth elevation and low (4.4 ft) and high (8.1 ft) water levels set based on asbuilts.

Reservoirs:

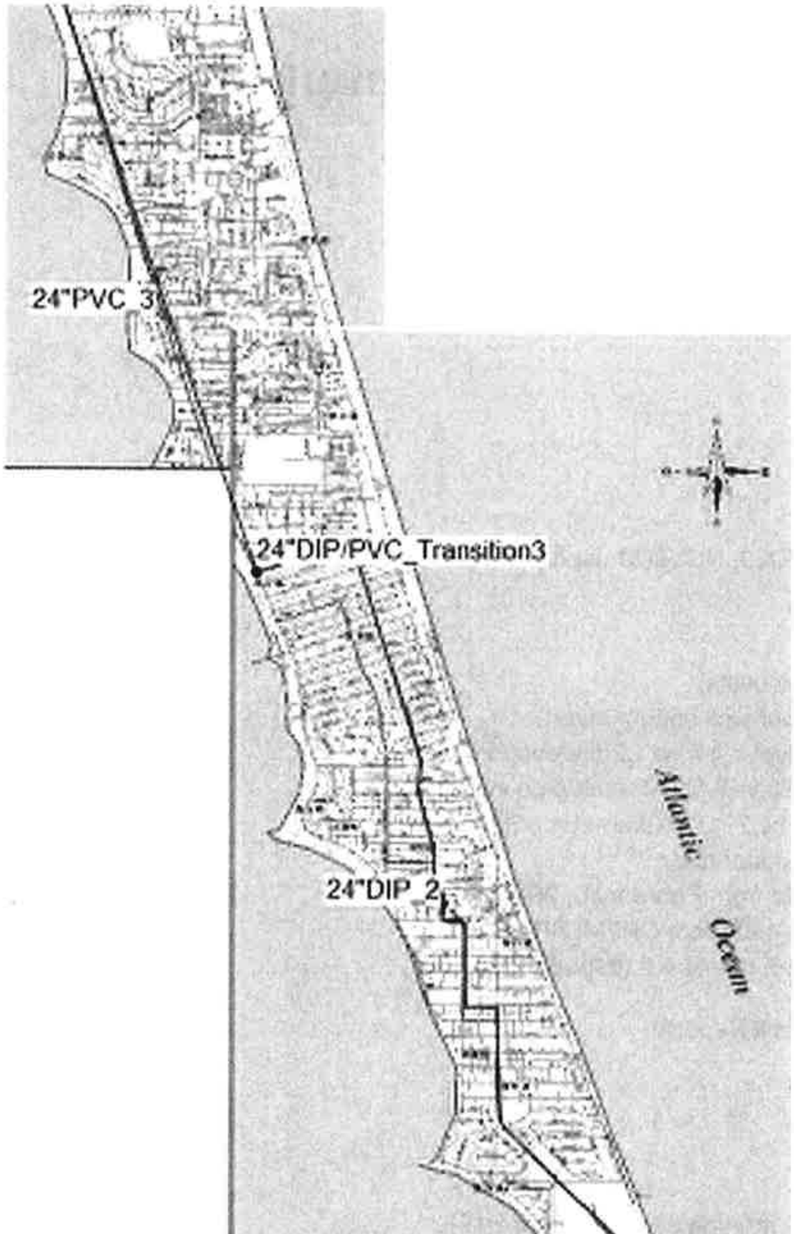
- SBWRF: South Beach Wastewater Treatment Plant, end of the force main system. Elevation set based on County provided information on bar screen high water elevation at the headworks.

EPAnet Network Map

North Section:



South Section:



Existing System: EPAnet Model Assumptions, Inputs and Notes

EPAnet Version 2.0, Build 2.00.12

Friction Equation: Hazen Williams

Flow: GPM

Energy Price: $0.06 / kw-hr

Assume elevations on asbuilts are NGVD29, NGVD29 used in model.

B19 Lift Station:

1. Wet well (information from asbuilts)
    a. Elevation = -15.2 (wet well bottom invert)
    b. Initial / Minimum Level = 4 ft, -11.2 ft elevation
    c. Maximum = 7.7 ft, Elev -7.5 ft 16-inch pipe influent invert
    d. dimensions: 12.5 ft by 29.0 ft (diameter equivalent = 21.5 ft)
2. Triplex with two running, one standby
3. Inflow = 2,000 gpm (estimate from February 6, 2010 Report)
4. Pumps = FLYGT C3231-63-430 (from CH2MHill Report)
5. Pipe Discharge Center Line Elev = 4.4 ft (asbuilts)

20" DIP&PVC = 18,473 LF (from CH2MHill Report)

20" DIP FM

1. Length: 15,491 LF
2. Top of Pipe Elevation =
    a. 2.0 ft (from asbuilts, starting elevation near B19)
    b. 0.0 ft (from asbuilts, elevation at Desoto Pkwy)
3. C = 100
4. Diameter = 20 inches
5. Minor Losses: (Coeff from Sanks)
    a. Assume 1 fitting per 1000 feet for a total of 16, Coeff 0.25 each
    b. Assume 1 isolation vale per 2000 feet for a total of 8, Coeff 1.00 each
    c. Total Minor Loss Coeff Assumption = 12

20" PVC FM

1. Length: 2,981 LF (based on Jeff C markups of PVC length on map)
2. Top of Pipe Elevation = 0 ft (from asbuilts, tie-in elevation with B20)

3. C = 110
4. Diameter = 20 inches
5. Minor Losses: (Coeff from Sanks)
    a. Assume 1 fitting per 1000 feet for a total of 3, Coeff 0.25 each
    b. Assume 1 isolation vale per 2000 feet for a total of 1, Coeff 1.00 each
    c. Total Minor Loss Coeff Assumption = 1.75

B20 Lift Station:

1. Wet well (from asbuilts)
    a. Elevation = -14.6 (wet well bottom invert)
    b. Initial / Minimum Level = 4.4 ft, -10.2 ft elevation
    c. Maximum = 8.1 ft, -6.5 ft 16-inch pipe influent invert
    d. dimensions: 12.5 ft by 29.0 ft (diameter equivalent = 21.5 ft)
2. Triplex with two running, one standby
3. Inflow = 3,500 gpm (estimate from CH2MHill Report)
4. Pumps = FLYGT C3231-63-430 (from CH2MHill Report)
5. 18" DIP from B20 to tie in with 24" FM, 1000 LF approximate, 5.0 ft elev, approximate

24" PVC FM

1. Length: 2,205 LF (based on Jeff C markups of PVC length on map)
2. Top of Pipe Elevation = -2.0 ft (from asbuilts, near Banana River Dr)
3. C = 110
4. Diameter = 24 inches
5. Minor Losses: (Coeff from Sanks)
    a. Assume 1 fitting per 1000 feet for a total of 3, Coeff 0.25 each
    b. Assume 1 isolation vale per 2000 feet for a total of 1, Coeff 1.00 each
    c. Total Minor Loss Coeff Assumption = 2.75

24" DIP FM (north segment)

1. Length: 5144 LF (based on Jeff C markups of DIP length on map)
2. Top of Pipe Elevation = 2.0 ft (from asbuilts, near bend in roadway)
3. C = 100
4. Diameter = 24 inches
5. Minor Losses: (Coeff from Sanks)
    a. Assume 1 fitting per 1000 feet for a total of 5, Coeff 0.25 each
    b. Assume 1 isolation vale per 2000 feet for a total of 1, Coeff 1.00 each
    c. Total Minor Loss Coeff Assumption = 2.75

24" PVC FM (from 24" DIP north section to B1 tie in)

1. Length: 3,590 LF (based on Jeff C markups of PVC length on map)
2. Top of Pipe Elevation = 2.0 ft (from asbuilts, near Coral Way West)
3. C = 110
4. Diameter = 24 inches
5. Minor Losses: (Coeff from Sanks)
    a. Assume 1 fitting per 1000 feet for a total of 4, Coeff 0.25 each
    b. Assume 1 isolation vale per 2000 feet for a total of 2, Coeff 1.00 each

    c. Total Minor Loss Coeff Assumption = 3.00

B1 Lift Station:

1. Wet well (information from asbuilts)
    a. Elevation = -15.40 (wet well bottom invert)
    b. Initial / Minimum Level = 4.5 ft, -10.9 ft elevation
    c. Maximum = 11.5 ft, Elev -3.9 ft 18-inch pipe influent invert
    d. dimensions: 10 ft diameter
2. Duplex with one running, one standby
3. Inflow = 600 gpm (estimate from CH2MHill Report); 800 gpm and 1,000 gpm (analyzed)
4. Pumps = ABS AFP 1501 310 mm (from CH2MHill Report)
5. Pipe Discharge Center Line Elev of 12-inch = 4.25 ft (asbuilts)

24" PVC FM (from B1 tie in to 24" DIP Transition)

1. Length: 13,917 LF (based on Jeff C markups of PVC length on map and CH2MHill Length)
2. Top of Pipe Elevation = 2.0 ft (from asbuilts, near Coral Way West)
3. C = 110
4. Diameter = 24 inches
5. Minor Losses: (Coeff from Sanks)
    a. Assume 1 fitting per 1000 feet for a total of 14, Coeff 0.25 each
    b. Assume 1 isolation vale per 2000 feet for a total of 7, Coeff 1.00 each
    c. Total Minor Loss Coeff Assumption = 10.50

24" DIP FM (southern segment to plant SBWRF)

1. Length: 25,730 LF (based on CH2MHill Length)
2. Top of Pipe Elevation = 2.0 ft (from PBSJ asbuilt at Oakland)
3. C = 100
4. Diameter = 24 inches
5. Minor Losses: (Coeff from Sanks)
    a. Assume 1 fitting per 1000 feet for a total of 26, Coeff 0.25 each
    b. Assume 1 isolation vale per 2000 feet for a total of 13, Coeff 1.00 each
    c. Total Minor Loss Coeff Assumption = 19.50

SBWRF: Elevation 33.814 ft (32.4 ft NAVD88)

To note, only pump stations noted above were incorporated into the model. "Smaller" stations were not included in the model or analysis.

C

Appendix C – South Beaches Surge Analysis Study

HydraTek

TECHNICAL MEMORANDUM

Date: 15 December 2017

Revision: Version 2 (Draft)

To: Jeff Crigler (HDR)

From: Johnathan D. Nault and Djordje Radulj (HydraTek)

Subject: Hydraulic Transient (Surge) Analysis
South Beaches Lift Stations and Forcemains, Brevard County

INTRODUCTION

HydraTek & Associates (HydraTek) has been retained by HDR Inc. (HDR) on behalf of Brevard County Utility Services (County) to conduct a hydraulic transient (surge) analysis of the South Beaches sewage forcemain system located in Brevard County, Florida. The objectives of the analysis are to (i) support an investigation into previous forcemain failures, including reviewing transient pressure monitoring field data, and (ii) support the design of system upgrades while recommending alternatives for reducing the hydraulic transient (surge) risk in the system.

The South Beaches system features two separate forcemains – the larger east forcemain and the smaller west forcemain. The east system has roughly 69,000 ft of 20-inch to 24-inch diameter forcemain with a combination of PVC and ductile iron pipe, and it is supplied by a series of large and small pumping stations, the key ones of which are the larger Lift Stations B-19, B-20, and B-1. The west forcemain system comprises nearly 25,000 ft of 20-inch diameter PVC pipe and 24-inch diameter ductile iron pipe, and it too is supplied by a combination of large and small lift stations with Lift Stations B-6 and B-7 representing the primary supply. Both systems convey wastewater to the South Beaches Wastewater Treatment Plant (WWTP) located in Melbourne Beach at the south end of the County.

This work primarily concerns the east system that has, in recent years, experienced a number of forcemain (pipe) breaks. Most of these have been concentrated at the PVC sections of the forcemain, and previous work by others suggests that the breaks are predominantly caused by improperly installed pipe. Transient (surge) pressures resulting from pump start and stop operations are thought to be contributing to the failures. To better understand the likely source of the forcemain failures and assess whether transient pressures are related thereto, HDR was retained by the County to review previous studies of the system, conduct a transient pressure monitoring exercise (performed by Blacoh Industries; Blacoh), and analyze alternatives for improving the system.

Based on the previously completed review, HDR has proposed that approximately 17,800 ft of 24-inch diameter PVC forcemain along North Riverside Drive from Eau Gallie Boulevard to Oakland Avenue be replaced with 30-inch diameter ductile iron pipe and that Lift Station B-1 be upgraded in order to better operate with the larger pumps at Lift Stations B-19 and B-20. An interconnection between the east and west forcemain systems is also being considered along either Riverside Drive or along 2nd Avenue (though this has yet to be finalized). In facilitating the design of the proposed upgrades while supporting HDR's investigation, this report presents the results of a hydraulic transient (surge) analysis of the existing South Beaches forcemain system and proposed upgrades.

REFERENCE INFORMATION

Hydraulic steady-state and hydraulic transient (surge) models were developed to facilitate the hydraulic transient analysis using the following information provided by HDR:

Plan and Profile Drawings

- South Patrick and Indian Harbour Beach Force Main Interconnection record drawings showing original forcemain installation by Stottler Stagg & Associates, dated February 1986;
- South Patrick Drive Forcemain Relocation as-built drawings showing 20-inch and 24-inch PVC forcemain installation by Brevard County Water Resources Department, dated January 1997;
- South Beaches 24-inch HDPE Force Main Replacement record drawings showing 24-inch PVC and DI forcemain installation by PBS&J, dated June 2006; and
- South Beaches drawing index sketch, received from HDR on 03 June 2017.

Lift Station Drawings

- South Patrick Regional Pumping Station (Lift Station B-19) record drawings by Stottler Stagg & Associates, dated February 1986;
- Indian Harbour Beach Regional Pumping Station (Lift Station B-20) record drawings by Stottler Stagg & Associates, dated February 1986;
- South Beaches Lift Station B-01 Reconstruction and Connection to Regional Force Main final design drawings by Brevard County, dated 15 October 1992; and
- Lift Station B-1 60% Design Submission Drawings by HDR, dated November 2017.

Pump Performance Curves

- Flygt C3231-63-430 pump curves for Lift Stations B-19 and B-20, dated 02 March 2002;
- ABS AFP 1501 60 Hz pump curves for existing Lift Station B-01, dated 28 September 2007; and
- Xylem NP 3202 HT 3-466 pump curves for proposed Lift Station B-01, received from HDR on 16 November 2017.

Reports and Other Data

- South Beaches Regional Wastewater Treatment Facility Hydraulic Profile (sheet M-7) record drawing by Stottler Stagg & Associates, dated 20 July 1988;
- South Beaches Sewer Force Main Draft Surge Analysis report by PBS&J, dated October 2005;
- Expert Opinion of Scott C. Williams, PE, on a Construction Project Known as B-6 Lift Station and Force Main Modifications by CH2M Hill, dated 16 February 2010;
- Brevard County Utility Services Department AWWA C905 DR25 PVC Force Main Pipe Samples failure review report by Diamond Plastics, dated 18 August 2015;
- Pipe break map for the east forcemain system by Brevard County, dated 02 June 2016;
- Review of Previous Pipe Failure Data and Surge Reports Technical Memorandum by HDR, dated 07 April 2017;

- B01 Pump Station Analysis EPANET Model, Results, and Assumptions report by HDR, dated 15 June 2017;
- South Beaches Wastewater Treatment Plant service area schematic showing lift stations and typical flows, received from HDR on 31 October 2017;
- South Beaches System Map by Brevard County, received from HDR on 03 June 2017; and
- Transient pressure monitoring data at LSs B-19 and B-01 for the period of 26 September 2017 to 25 October 2017 by Blacoh.

SYSTEM DESCRIPTION AND ANALYSIS METHODOLOGY

The following sections describe the South Beaches forcemain system and how they are modeled. The system is described first, followed by a discussion of the model development. In developing the hydraulic transient model, some assumptions were made of the key factors that affect the response of the system: these are discussed where appropriate. Finally, the transient analysis scenarios are listed before presenting and discussing the results.

East and West Forcemain Systems

Figure 1 on the following page illustrates a schematic of the South Beaches system (excluding the smaller lift stations). The larger east forcemain spans Sea Park Boulevard at the north to the South Beaches WWTP at the south, and it is primarily supplied by three lift stations. Lift Station B-19 (the South Patrick Regional Pumping Station) is the north-most facility, and it has three dry-pit centrifugal pumps, each with a motor soft-starter. Lift Station B-20 (the Indian Harbour Beach Regional Pumping Station) is situated in the middle. Like Lift Station B-19, lift station B-20 features a similar configuration with three dry-pit pumps with motor soft-starters. Lastly for the east system, the somewhat smaller Lift Station B-1 is the south-most facility with two submersible wet-pit pumps. Following the proposed upgrades, Lift Station B-1 will still have two larger pumps and a similar arrangement. All pumps at all lift stations are constant speed pumps.

Figure 2 on the subsequent page shows the centerline profile for the east forcemain system, including the location of the proposed 30-inch forcemain upgrade from Eau Gallie Boulevard to Oakland Avenue. The west forcemain system spans 2nd Avenue at the north to the WWTP at the south, and it is supplied by Lift Stations B-6 and B-7, both of which feature similar configurations to Lift Station B-1 and each have two submersible wet-pit centrifugal pumps. All of the pumps at each lift station are equipped with a tilted disc check valve.

Operationally, the number of pumps on at each facility is controlled by the level in the facility's wet well. Table 1 lists the elevations and corresponding levels for each lift station based on the information provided by HDR. Each pump starts against a closed check valve, and the pump's flow develops as the system's static head is overcome. For the pumps at Lift Stations B-19 and B-20, the previous hydraulic transient analysis by PBS&J notes that the pump's motor soft-starters have a 30 s ramp duration for pump starts and stops. Motor soft-starters provide electrical control and, depending on the drive type, occasionally some albeit limited hydraulic control for pump starts and stops by limiting either the motor voltage or current. Though each station is designed to provide its nominal rated flow at firm capacity (that is, with all pumps on except for the largest pump), it is understood that, on occasion, a station may operate at full capacity with all pumps on (e.g., during periods with large inflow and infiltration from heavy rainfall, Lift Station B-19 may operate with three pumps on).

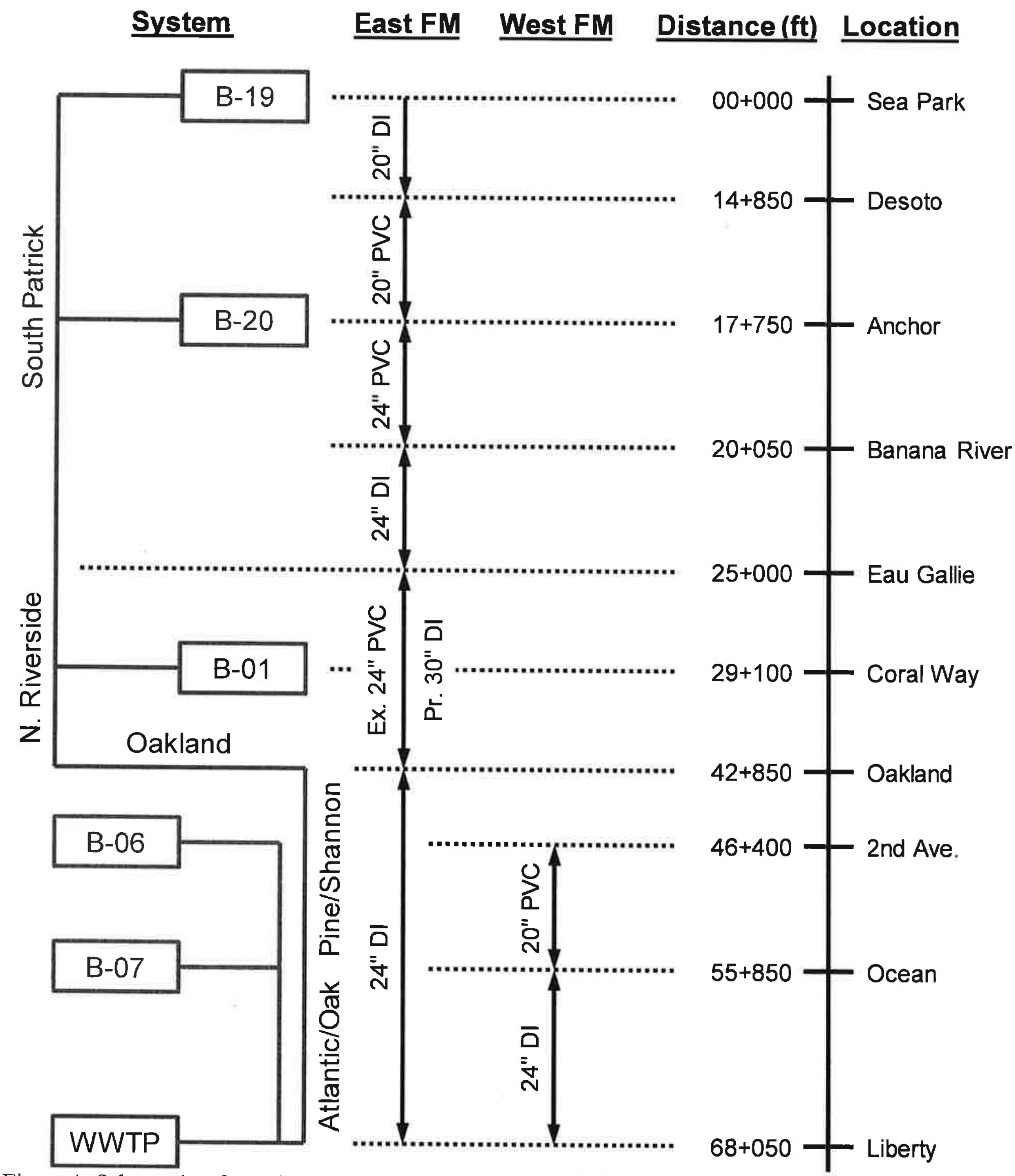


Figure 1: Schematic of South Beaches east and west forcemain systems

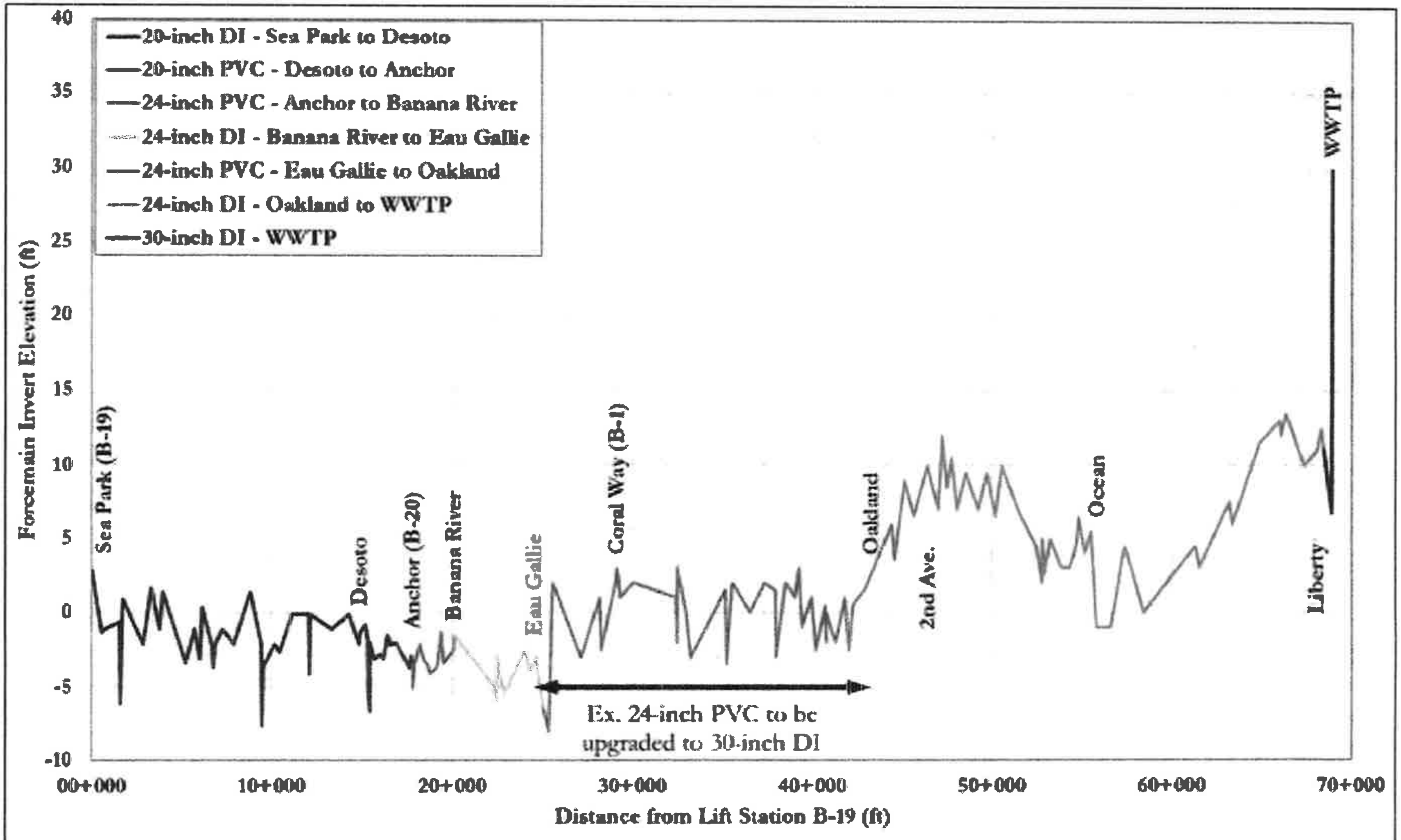


Figure 2: Centerline profile for the South Beaches existing east forcemain system

Table 1: Lift station wet well elevations and levels

Location	Elevation (ft)					Level (ft)				
	B-19	B-20	B-1	B-6*	B-7*	B-10	B-20	B-1	B-6*	B-7*
Wet well invert	-15.2	-14.6	-15.4	-15.4	-15.4	0.0	0.0	0.0	0.0	0.0
Pumps off	-9.2	-8.6	-12.4	-12.4	-12.4	6.0	6.0	3.0	3.0	3.0
Inlet pipe invert	-7.5	-6.5	-9.4	-9.4	-9.4	7.7	8.1	6.0	6.0	6.0
Lead pump	-6.2	-6.1	-6.4	-6.4	-6.4	9.0	8.5	9.0	9.0	9.0
Lag pump 1	-4.2	-5.1	-5.4	-5.4	-5.4	11.0	9.5	10.0	10.0	10.0
Lag pump 2	-3.2	-4.5	-	-	-	12.0	10.1	-	-	-
High alarm	-2.2	-4.1	-2.4	-2.4	-2.4	13.0	10.5	13.0	13.0	13.0
Top of wet well	7.7	7.1	8.5	8.5	8.5	22.9	21.7	23.9	23.9	23.9

* Elevations for B-6 and B-7 assumed to be as shown in HDR's EPANET model, and levels assumed as per those for B-1.

Hydraulic Model Development

The hydraulic transient (surge) analysis was performed using Bentley's HAMMER v10 software package. HAMMER is based on the proven method of characteristics solution of the governing water hammer equations, and it has been used for numerous projects globally for over a decade. To facilitate the numerical modeling analysis, a hydraulic model (shown in Figure 3) was developed using the information provided by HDR, namely the forcemain as-built drawings, pump performance curves, and previous analysis reports. Intermediate nodes were also added at local high points, local low points, and other key locations, such as transitions between pipe materials. Because the east forcemain system is the primary subject system (the west system is only concerned for scenarios with the east-west system interconnections), intermediate nodes at points of interest along the west forcemain were not modeled.

In developing the hydraulic transient model, some key assumptions were made to facilitate the analysis. The system pipes were modeled using their nominal diameters. The smaller lift stations (which are not shown in Figure 1) were omitted from the analysis, for they represent rather small flows compared to the larger Lift Stations B-19, B-20, and B-1 (and similarly B-6 and B-7 for the west system). A Hazen-Williams roughness (C-factor) of 110 was used for all existing piping due to their age, and a roughness factor of 140 was used for all new piping – the latter assumption is generally conservative for hydraulic transient analyses due to simulating larger initial flows and thus larger changes in hydraulic states. Further, the lift stations' levels were considered as either the low water level (LWL) and the high water level (HWL): the former corresponds to the "pumps off" level listed in Table 1, while the HWL values correspond to the higher lag pump start elevation (e.g., -3.2 ft for Lift Station B-19). Regarding the downstream boundary condition, the WWTP hydraulic profile schematic shows a submerged inlet condition for the forcemain. The WWTP is thus represented as a constant head reservoir with an elevation of 30.5 ft based on the LWL from the plant's hydraulic profile. Further assumptions were made in addition to those discussed above; these pertain to the pump properties, system wave speeds, and surge protection, which are discussed in the following sections.

Pump Properties

Pump properties such as size and speed are key parameters that directly affect the steady and transient hydraulics (flows and pressures) of a pumping system. Table 2 lists the properties of the pumps at each lift station. The combined rotational mass moment of inertia of a pump and its motor are the other key factor that affects the transient response of a pumping system. Under a power failure condition at a pump station, a pump with greater total inertia will take a longer time to completely rundown (i.e., push water through the system) than will a pump with a lower total inertia. The manufacturer values for pump inertia were not readily available for this analysis, so the radius-based (WR^2) pump inertia was estimated for each pump based on empirical relations - these values are also listed in Table 2. Because the total moment of inertia is occasionally a critical parameter to the hydraulic transient response of a system, a sensitivity analysis of these values is conducted.

Table 2: South Beaches forcemain system lift station pump properties

System	Facility	Pump ID	Status	Manufacturer and Model	Rated Flow* (GPM)	Rated Head* (ft)	Rated Power (hp)	Rated Speed (rpm)	Estimated Total Inertia (lb-ft^2)
East	B-19	1	Existing	Flygt C3231-63-430	3,643	147.0	185	1,780	75.8
		2	Existing	Flygt C3231-63-430	3,643	147.0	185	1,780	75.8
		3	Existing	Flygt C3231-63-430	3,643	147.0	185	1,780	75.8
	B-20	1	Existing	Flygt C3231-63-430	3,643	147.0	185	1,780	75.8
		2	Existing	Flygt C3231-63-430	3,643	147.0	185	1,780	75.8
		3	Existing	Flygt C3231-63-430	3,643	147.0	185	1,780	75.8
	B-1	1	Existing	ABS AFP 1501, 310 mm	1,698	108.0	70	1,785	20.4
		1	Proposed	Flygt NP 3202 HT 3~466	1,093	142.7	70	1,775	16.7
		2	Existing	ABS AFP 1501, 310 mm	1,698	108.0	70	1,785	20.4
		2	Proposed	Flygt NP 3202 HT 3~466	1,093	142.7	70	1,775	16.7
West	B-6	1	Existing	ABS AFP 1501, M430/4-44	1,600	90.6	60	1,775	15.6
		2	Existing	ABS AFP 1501, M430/4-44	1,600	90.6	60	1,775	15.6
	B-7	1	Existing	ABS AFP 1501 R25076-42.50	1,125	41.0	20	1,180	9.1
		2	Existing	ABS AFP 1501 R25076-42.50	1,125	41.0	20	1,180	9.1

* At the best efficiency point (BEP) on the pump's performance curve.
† Based on the empirical relations presented in Fluid Transients in Pipeline Systems, 2nd Ed. (Thorley, 2004).

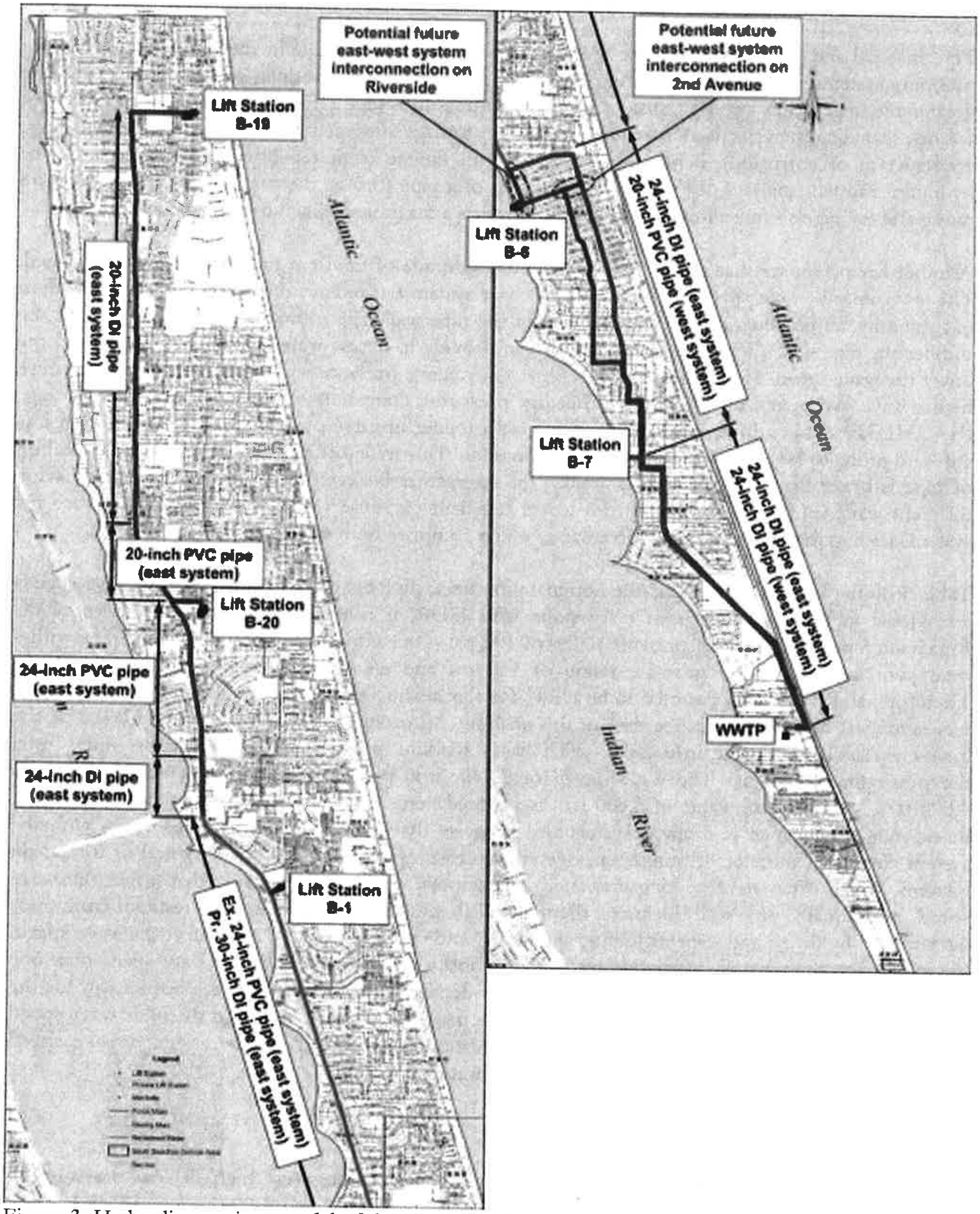


Figure 3: Hydraulic transient model of the South Beaches east and west forcemain systems

Pipe Class and Wave Speed

Pipe material and pipe class (i.e., pressure rating) play an important role in the transient response of pumping systems. The higher the rating of the pipe, the greater its working pressure rating, and the greater the upper limit against transient (i.e., surge) pressures. Pipe rating is affected by a number of factors, including imperfections from manufacturing, damage sustained during installation or use (e.g., construction or corrosion), depth of installation, and fatigue from repetitive pressure cycling. For hydraulic transient analyses, the total pressure rating of a pipe (that is, the sum of its working pressure rating and the pipe's surge allowance) is typically used as a maximum design limit or upper bound.

Another key parameter that affects the nature and magnitude of transient pressures is the wave speed. The wave speed (sonic speed through a liquid) of a system depends on both the nature of the fluid (e.g., density, air content) and on the elasticity of the pipe wall. The more elastic the material and the thinner the pipe wall, the more storage a pipe can provide to excess water being compressed and the lower the wave speed. Materials such as ductile iron, concrete pressure pipe, and steel typically produce higher wave speeds and more significant upsurge pressures; comparatively, more elastic materials (e.g., PVC, HDPE) tend to have lower wave speeds with milder upsurge pressures, yet they are often less stiff and prone to longer downsurge pressure durations. This trade-off means that neither classification of pipe is better than the other. Additionally, the interaction between different pipe materials having different wave speeds can also lead to a worse transient response – this is particularly the case for systems such as the South Beaches forcemains, which comprise both ductile iron and PVC pipe.

Table 3 on the following page lists the nominal pipe sizes, pipe materials and classes, and wave speeds considered in this analysis. From discussions with HDR, it is understood that the existing PVC forcemain pipe has a working pressure rating of 100 psi – this corresponds to DR41 PVC pipe with a total (working plus surge) pressure rating of 160 psi and an estimated wave speed of 850 ft/s. Therefore, all PVC pipe is assumed to be DR41 for this analysis. The pressure class of the ductile iron forcemain was not available at the time of this analysis. Accordingly, it is assumed to be Class 150, the lowest available for ductile iron pipe, which has a working pressure rating of 150 psi and a total pressure rating of 250 psi. The wave speed for ductile iron pipe typically ranges from 3,600 ft/s to 4,000 ft/s, so an average value of 3,800 ft/s is adopted here. A review of Blacoh's transient pressure monitoring data (which is discussed later) also suggests that the system wave speed varies with the system condition (number of pumps on, operating pressures and flows) – this is typical of forcemain systems, which often have varying amounts of entrapped and dissolved gases that affect the wave speed. Additionally, pipe wall thickness, the material Poisson ratio, joints, and the restraint conditions (as imposed by thrust and joint restraints, anchoring, and expansion joints) also affect the wave speed. For instance, a pipe that is sufficiently restrained at both ends will have a higher wave speed than one with anchoring at one end or none at all. The exact details of these properties are not known for the South Beaches system though, so no adjustments are made to consider these and the basic wave speed assumptions are utilized. Furthermore, sensitivity tests are performed for the assumed wave speed values to explore whether they are indeed a critical parameter.

Existing Surge Protection

Surge protection devices are intended to alleviate otherwise excessively high or low pressures in pressurized pipe systems. The as-built drawings for Lift Stations B-19 and B-20 show that each has a 6-inch surge control valve located on the discharge header, though the specific type and function of the valves is not known. In addition to the surge valves, the as-built drawings for the forcemains show air valves at a small number of high points from Lift Station B-19 to the WWTP. From discussions with HDR, the surge valves at Lift Stations B-19 and B-20 are presently isolated and thus not used, and the air valves along the forcemains are 2-inch nominal diameter air release valves – that is, the air valves are small outflow only air valves that do not provide vacuum protection. Altogether, the existing South Beaches forcemain system effectively does not feature dedicated surge protection.

Table 3: South Beaches forcemain pipe pressure ratings and wave speeds

System	Section	Nominal Diameter (in)	Pipe Material and Class	Working Pressure Rating (psi)	Total Pressure Rating (psi)	Wave Speed (ft/s)
East	Lift Station B-19 to Desoto	20	Ductile Iron, Class 150*	150*	250*	3,800
	Desoto to Anchor	20	PVC, DR41+	100	160	850
	Anchor to Banana River	24	PVC, DR41+	100	160	850
	Banana River to Eau Gallie	24	Ductile Iron, Class 150*	150*	150*	3,800
	Eau Gallie to Oakland (Existing)	24	PVC, DR41+	100	160	850
	Eau Gallie to Oakland (Proposed)	30	Ductile Iron, class to be determined	150*	250*	3,800
	Oakland to WWTP	24	Ductile Iron, Class 150*	150*	250*	3,800
West	Lift Station B-6 to 1st Avenue	20	PVC, DR41+	100	160	850
	1st Avenue to WWTP	24	Ductile Iron, Class 150*	150*	250*	3,800

* Because the class of ductile iron pipe is not known for this system, it is assumed to be class 150 with a working pressure rating of 150 psi and total (operating plus surge) pressure rating of 250 psi.

+From discussions with HDR, it is understood that the existing PVC pipe has a working pressure rating of 100 psi. This corresponds to DR41 PVC pipe, which is assumed for all PVC pipe in the system.

It is also worth briefly mentioning the previous hydraulic transient (surge) analysis performed by PBS&J in 2005. At the time of PBS&J's analysis, the status of the existing surge valves at Lift Stations B-19 and B-20 is not clear, and PBS&J's report does not list any other forms of existing surge protection in the system. Results from the analysis suggest that, without dedicated surge protection, negative pressures reaching full vacuum are problematic along the forcemain. To alleviate such conditions, PBS&J discusses two alternatives: installing combination air valves (CAVs) along the forcemain at critical locations, or installing surge tanks at the lift stations. From discussions with HDR, it is understood that neither of these recommendations has been implemented.

Transient Events

While most systems are often subjected to a variety of risks resulting from hydraulic transients, the key for the analysis and design against unsteady flow (surge, transient, or water hammer conditions) comes down to the selection of an appropriate design event or design events. In this system, the primary transient conditions arise from three events: operational changes (i.e., routine pump or valve operations), pipe failures (i.e., forcemain breaks), and power failures at the lift stations. The risks from routine pump or valve changes and other planned events (e.g., filling or draining) can be minimized and accommodated through proper operator training and by ensuring that these changes evolve over a moderate period of time. Pipe breaks are almost impossible to predict in any detail, are more difficult to control, and can result in negative pressure surges that yield subsequent pipe breaks. The key to minimizing the impact of such an event (in both water supply and wastewater systems) is providing adequate system storage, or in very large systems, pressurized vessels (i.e., surge tanks). Power failure events are often the most problematic. They can occur under almost any operating condition; as a result, the key transient protection challenge likely comes down to a power failure event at the source lift stations (i.e., Lift Stations B-19, B-20, B-1, B-6, and B-7). This design event is a suitable choice because power failures are, by their very nature, unpredictable. They are inevitable over the long life of pumping systems, and they are excellent surrogate design events for gauging a system's response to routine and non-routine pump trips, starts, and stops, and in some cases, pipe breaks.

When a power failure occurs, there is an immediate deceleration of the pump rotor and a decrease in the rate that fluid is forced into the associated pipe segment. This reduction of inflow creates a mass imbalance in the associated pipe, since the downstream end is not immediately informed of the loss of power. The net effect is a low-pressure wave that propagates through the pipe system until it is eventually reflected back at the downstream end. The reflected wave essentially sends a signal back to the pump and repeats the cycle, a process that eventually and gradually brings about a new equilibrium in the system. This is essentially a "conversation" between the ends of the system that takes place through a series of pressure wave propagations. Eventually, these pressure waves are dissipated by friction and other energy losses, and a new steady-state is established.

From a design perspective, there are three immediate challenges associated with power failure events. The first is the downsurge that immediately follows pump stoppage: this low-pressure wave can potentially collapse a "weak" or under-designed pipeline or allow an intrusion event to occur at susceptible points in the system. The second risk arises from the return surge that is generated from the reflected wave at the downstream end of a system. This reflected wave, referred to as a "return upsurge", actually increases pressures in the line, possibly to values that can considerably exceed the original steady-state or equilibrium levels experienced before the power failure. Unless properly controlled, the resulting transient pressures create either a risk of pipe failure or a need for much stronger pipes (i.e., thicker pipe walls, resulting in a higher pipe pressure rating and greater cost). Lastly, power failures can also create an upsurge event at the upstream (suction) line of the source pumping station. The effect of the upsurge at the suction side generally depends on the type of pumping station. In a "true" pumping station (such as Lift Station B-19), the upsurge is typically mitigated by the presence of a wet well and/or a short length of pipe.

Although power failures are likely the governing transient event, significant transient pressures can still arise from other source events. If not adequately controlled, the transient pressures resulting from routine pump starts and stops, which are more frequent than power failures, may be considerable; thus, it is worthwhile verifying that power failures are indeed the governing event. Under normal operating conditions, routine pump starts and stops at Lift Stations B-19 and B-20 are performed using the pumps' motor soft-starters. Based on information provided by HDR, pump start (and stop) operations involve starting (or stopping) a pump using its motor soft-starter with a 30 s ramp duration. Again, motor soft-starters can provide some (albeit limited) hydraulic control for pump start and stop operations depending on the drive type, so here routine starts and stops are represented using a linear torque ramp. The pumps at the remaining Lift Stations B-1, B6, and B-7 are simply started (and stopped) as an across-the-line operation with no hydraulic nor electrical control.

From the above, the following events are considered in the hydraulic transient analysis:

- Global power failure of all lift stations,
- Local power failure of select lift stations, and
- Routine pump starts and pump stops.

The pump start and stop operations at Lift Stations B-19 and B-20 are represented by a linear torque ramp having the same 30 s duration as the motor soft-starters. Soft-starters limit either current or voltage, thereby allowing power and thus torque to gradually adjust to the operating condition.

Analysis Scenarios

A total of 35 scenarios were simulated to explore the hydraulic transient (surge) response of the system under different conditions. They consider various combinations of the system state (existing, proposed upgrades, and potential east-west system interconnection), number of pumps initially on, wet well levels, transient event (global or local power failure, pump operations), and surge protection.

The first group of scenarios (Scenarios 1A through 5B; Table 4) concerns the existing east forcemain. These scenarios are intended to support the investigation into previous east forcemain failures, and they explore the system response to global power failure events under full capacity (Scenarios 1A and 1B), local power failures at Lift Stations B-19 (Scenario 2) and B-20 (Scenario 3), and routine pump operations at Lift Stations B-19 (Scenarios 4A and 5A) and B-20 (Scenarios 4B and 5B). Scenarios 2 through 5B consider HWLs at the lift stations, for this yields the largest initial flow and thus likely the greatest potential for extreme pressures.

Table 4: List of hydraulic transient analysis scenarios for existing conditions without surge protection

Scenario	Parent Scenario	No. Pumps					Well Levels	System	Event+
		B-19	B-20	B-1	B-6	B-7			
1A	-	3	3	2	-	-	HWL	Existing	Global Failure
1B	1A	3	3	2	-	-	LWL	Existing	Global Failure
2	-	3	0	0	-	-	HWL	Existing	Local Failure
3	-	0	3	0	-	-	HWL	Existing	Local Failure
4A	-	1*	0	0	-	-	HWL	Existing	B-19 Pump Start
4B	4A	0	1*	0	-	-	HWL	Existing	B-20 Pump Start
5A	-	1	0	0	-	-	HWL	Existing	B-19 Pump Stop
5B	5A	0	1	0	-	-	HWL	Existing	B-20 Pump Stop

* Pump start with pump initially off.

The second set of simulations (Scenarios 6A to 17B; Table 5 on the following page) explores the hydraulic transient response of the system with the proposed 30-inch ductile iron forcemain replacement, proposed Lift Station B-1 upgrades, and no surge protection. Scenarios 10 and 11 in particular consider interconnections between the east and west forcemain systems on North Riverside Drive and 2nd Avenue, respectively; the rational for these two scenarios is to test the potential hydraulic transient benefits (or detriments) of interconnecting the two systems. Scenarios 12 through 15 are sensitivity tests of key parameters, including wave speed, pump inertia, and vapor cavity formation. Additional simulations (Scenarios 16A to 17B) were performed for routine pump starts and stops at Lift Stations B-19 and B-20.

Table 5: List of hydraulic transient analysis scenarios for proposed conditions without surge protection

Scenario	Parent Scenario	No. of Pumps					Well Levels	System	Event+	Sensitivity
		B-19	B-20	B-1	B-6	B-7				
6A	-	3	3	2	-	-	HWL	Pr. Upgrades	GPF	-
6B	6A	2	2	1	-	-	HWL	Pr. Upgrades	GPF	-
7A	-	3	3	2	-	-	LWL	Pr. Upgrades	GPF	-
7B	7A	2	2	1	-	-	LWL	Pr. Upgrades	GPF	-
8A	-	3	0	0	-	-	LWL	Pr. Upgrades	GPF	-
8B	8A	2	0	0	-	-	LWL	Pr. Upgrades	GPF	-
9A	-	0	3	0	-	-	LWL	Pr. Upgrades	GPF	-
9B	9A	0	2	0	-	-	LWL	Pr. Upgrades	GPF	-
10	7A	3	3	2	2	2	LWL	Riverside Interconnection	GPF	-
11	7A	3	3	2	2	2	LWL	Oakland Interconnection	GPF	-
12	Worst GPF	0	3	0	-	-	LWL	Pr. Upgrades	GPF	+10% Wave Speed
13	Worst GPF	0	3	0	-	-	LWL	Pr. Upgrades	GPF	-10% Wave Speed
14	Worst GPF	0	3	0	-	-	LWL	Pr. Upgrades	GPF	50% Pump Inertia
15	Worst GPF	0	3	0	-	-	LWL	Pr. Upgrades	GPF	No Vapor Cavities
16A	-	1*	0	0	-	-	HWL	Pr. Upgrades	Pump Start	-
16B	-	0	1*	0	-	-	HWL	Pr. Upgrades	Pump Start	-
16C	-	0	0	1*	-	-	HWL	Pr. Upgrades	Pump Start	-
17A	-	1	0	0	-	-	HWL	Pr. Upgrades	Pump Stop	-
17B	-	0	1	0	-	-	HWL	Pr. Upgrades	Pump Stop	-
17C	-	0	0	1	-	-	HWL	Pr. Upgrades	Pump Start	-

* Pump start with pump initially off.
+GPF = global power failure, LPF = local power failure.

The aforementioned scenarios (Scenarios 1A through 17B) conservatively consider no dedicated surge protection. The remaining Scenarios 18A to 20 (Table 6 on the following page) explore the potential benefits of installing dedicated surge protection, namely CAVs at critical locations to alleviate the formation of vapor cavities (Scenarios 18A to 19B) and surge tanks (i.e., hydropneumatic air chambers or HACs) at the larger Lift Stations B-19 and B-20 (Scenarios 20A to 20D). The latter set of scenarios are mainly intended to demonstrate the potential benefits of installing HACs with bladders.

Table 6: List of hydraulic transient analysis scenarios for proposed conditions with surge protection

Scenario	Parent Scenario	System	Power Failure Event	Surge Protection
18A	Worst Base No. 1	Pr. Upgrades	Global	2-inch Standard CAVs†
18B	Worst Base No. 2	Pr. Upgrades	Global	2-inch Standard CAVs†
19A	Worst Base No. 1	Pr. Upgrades	Global	2-inch 3-Stage CAVs†
19B	Worst Base No. 2	Pr. Upgrades	Global	2-inch 3-Stage CAVs†
20A	Worst Base No. 1	Pr. Upgrades	Global	HAC+ at B-19
20B	Worst Base No. 1	Pr. Upgrades	Global	HAC+ at B-20
20C	Worst Base No. 1	Pr. Upgrades	Global	HAC+ at B-1
20D	Worst Base No. 1	Pr. Upgrades	Global	HACs at B-19 and B-20

+GPF = global power failure, HAC = hydropneumatic air chamber (pressurized surge tank).
† CAVs at select critical locations.

ANALYSIS RESULTS AND DISCUSSION

The following sections present and discuss the hydraulic transient (surge) analysis results. The initial discussion concerns the field monitoring data and simulation results for the existing east forcemain system, while subsequent sections discuss the results for the proposed upgrades, including alternatives for surge protection. Further, Appendix A contains a tabular summary of the modeling results, including initial lift station flows, the maximum predicted pressures, and the maximum predicted air/vapor cavity volumes; graphical results in the form of hydraulic grade line (HGL) profiles are also provided in Appendix B for the forcemain path from Lift Station B-19 to the WWTP. Lastly, Appendix C contains excerpt plots of the field monitoring data with Blacoh's full report provided in Appendix D.

Transient Pressure Monitoring Data

The transient pressure monitoring exercise was undertaken by Blacoh from 26 September 2017 to 25 October 2017. One high-frequency pressure logger was installed on the discharge side of Lift Station B-19 from 26 September 2017 to 25 October 2017, and another was installed on the discharge side of Lift Station B-1 from 26 September 2017 to 12 October 2017. Both installations were downstream of the pump check valves. Normal and transient pressures were recorded, the latter at up to 100 samples per second during transient (surge) events. Because the exercise was passive in nature (that is, no coordinated field tests were performed), the data presumably represents typical system conditions and routine operations. It is also worth noting that some transient pressure loggers suffer from time drift, which otherwise impacts the ability to correlate data between loggers and against other data. Consequently, the monitoring data cannot be directly compared against the modeling data due to uncertainties of the following: the precise pressure measurement elevations, boundary conditions (e.g., wet well and WWTP levels, active pumps), the precise operational changes (and nature thereof), and the system properties. Despite the aforementioned limitations, inferences and observations can be drawn from the field data for the existing east forcemain system. Sample monitoring results in the form of pressure histories are provided in Appendix C for 30 September 2017 (Figure 47 to Figure 50) and 05 October 2017 (Figure 51 to Figure 55), two dates where monitoring data are available at both locations. Table 7 below presents an overall summary of the pressure monitoring data.

Table 7: Summary of transient pressure monitoring data at Lift Stations B-19 and B-1

Lift Station	Monitoring Period		Pressure Summary (psi)				
	Start	End	Minimum	Average Low	Average	Average High	Maximum
B-19	26 Sep 2017	25 Oct 2017	-8	39	65	77	148
B-1	26 Sep 2017	12 Oct 2017	-10	27	44	54	130

From a review of the transient pressure monitoring data, the following general observations are made:

- Transient pressures at both monitoring locations regularly reach partial vacuum conditions. The minimum recorded pressure at Lift Station B-19 is -8 psi, and that at Lift Station B-1 is -10 psi. The minimum pressures are likely lower than -10 psi at forcemain high points, and these locations are therefore likely where full vacuum pressures and vapor cavity formation occurs. Given the low and even negative transient pressures, the system is at risk of experiencing vapor cavity formation and collapse, which can create significant transient pressures, extreme stresses on forcemain pipes, and exacerbate failures. The monitoring data in Figure 48 (Appendix C) shows what may even be the result of rapid air expulsion or vapor cavity collapse.

- From the transient pressure field monitoring data, the wave travel time from Lift Station B-19 to B-1 ranges from 30 s to 35 s. Given that the distance between these two stations is roughly 29,100 ft, this corresponds to an aggregate wave speed of 830 ft/s to 970 ft/s. This is less than that expected based on the estimated wave speeds for PVC and ductile iron (Table 3), though the field estimate is subject to the precise time synchronization of the two transient pressure loggers.
- Regarding pressures, the maximum pressures recorded at Lift Stations B-19 and B-1 are 148 psi and 130 psi, respectively. These are less than the assumed total (operating plus surge) pressure ratings for the ductile iron pipe (250 psi) and PVC pipe (160 psi). The peak pressure at Lift Station B-19 arose from what appears to be a single pump start at Lift Station B-19 following a period with all east system pumps off: Figure 56 and Figure 57 in Appendix C plot the pressure time series for this. The maximum pressure recorded at Lift Station B-1 arose from what is likely check valve slam following a pump stop at Lift Station B-1, and this is shown in Figure 55.
- In general, the system operation during the day time appears to have fewer 0-to-1 pump starts and fewer 1-to-0 pump stops. For example, Figure 47 and Figure 51 show frequent periods with all system pumping off over night, while Figure 49 and Figure 53 show fewer operations during the day time. This is likely due to more consistent wet well inflows during the day time.
- Following what is presumably full pump stoppage for the system (i.e., all pumps at the major lift stations are turned off), the monitoring data shows rapid and extreme pressure fluctuations at Lift Station B-19 (see Figure 48, Figure 50, and Figure 52 in Appendix C). The precise cause of this behavior cannot be ascertained from the data alone, though it is quite likely the result of either or a combination of (i) vapor cavity formation and collapse, (ii) rapid air accumulation, expulsion, or intake at leaky points in the system, or (iii) part of the forcemain draining.
- It is understood that the pumps at Lift Stations B-19 and B-20 are equipped with motor soft-starters that have ramp up and ramp down durations of 30 s. From the pressure monitoring data, the soft-starters at Lift Station B-19 do not appear to provide meaningful hydraulic control, particularly for pump stop operations. This may indicate that either (i) the soft-starters are not configured properly with respect to hydraulic control or (ii) the soft-starters simply provide no hydraulic control for pump start and stop operations.
- The field data for Lift Station B-1 shows check valve slam following routine pump stops at Lift Station B-1. This can be seen in Figure 54, and a closer view is provided in Figure 55. Check valve slam is caused by a reflected downsurge wave returning before a check valve has time to close, thereby slamming the valve closed. Check valve slam is detrimental to the local piping and valve itself, as it creates excessive pressures and wears the valve seating and seal, potentially leading to valve failure and an improperly seating (leaky) valve. The pumps at Lift Station B-1 are started and stopped with no hydraulic control (other than that provided by the check valves themselves). Consequently and following a pump stop at Lift Station B-1, the downsurge wave travels away from the facility along the relatively short 12-inch discharge line, reflects off of the existing 24-inch forcemain, and returns back to the pumps slamming the check valve closed.

Existing East Forcemain System

Table 9 in Appendix A summarizes the simulation results for the existing east forcemain system without the proposed upgrades nor any surge protection (Scenarios 1A through 5B). Scenarios 1A to 3 consider global and local power failures with the system initially operating at full capacity (that is, with all pumps on at an active facility under large inflow conditions). In each case, the system response is rather varied with a number of intermediate reflections, and these are caused by the use of both less elastic ductile iron and more elastic PVC pipe materials along the forcemain. The tabular results in Table 9 and graphical results in Appendix B also show partial to full vacuum conditions (negative pressures) along the forcemain from Lift Station B-19 to the WWTP – for example, the HGL envelope along the east forcemain for Scenario 1B (Figure 7 in Appendix B) shows negative pressures at nearly all locations. Negative pressures were simulated as persisting for up to roughly 30 s. Moreover, the full vacuum conditions are predicted to give rise to vapor cavity formation and collapse at both the lift stations and along the forcemain, as indicated by the non-zero vapor volumes.[1] Of Scenarios 1A to 3, Scenario 1B yields the most critical response with the largest predicted vapor cavities, and this is likely due to (i) the system operating at full capacity and with high initial velocities and (ii) the wet wells being at LWL, which yields a slightly lower initial HGL and thus less ability to accommodate downsurges. Overall, the long wave travel time for the PVC sections of the forcemain also prolong negative pressure durations, thus exacerbating concerns of vapor cavities.

The remaining Scenarios 4A through 5B explore the system response to routine single pump starts and stops at Lift Stations B-19 and B-20. The scenarios are based on an initial condition of no pumping at other locations, thereby yielding the largest flow change for the pump start and stop. The results (Figure 10 to Figure 13 in Appendix B) show rather high upsurge pressures (up to 95 psi at Lift Station B-19); for comparison, Figure 4 plots the results for Scenario 4A against the field monitoring data for what appears to be a routine pump start at Lift Station B-19. These data compare reasonably well for the initial upsurge; however, there are clear differences thereafter. Moreover, compared to the field monitoring data, the numerical modeling results predict lower maximum transient pressures. The difference between the model and field data is largely due to how pump start operations are actually performed. For this analysis, a 30 s linear torque ramp is used for pump starts, whereas the actual field conditions likely vary from this depending on the soft-starters' control and type (that is, whether the soft-starters limit voltage or current, and whether this translates to effective torque and thus hydraulic control). The differences are also affected by uncertainties of the boundary conditions (wet well levels, number of pumps on) and system properties. The results for Scenarios 5A and 5B show negative pressures and even full vacuum conditions following pump stop operations, which generally agrees with the low field monitoring pressures observed following full pump stoppage in the system.

Both the field monitoring data and numerical modeling results suggest that the typical pressures under steady and transient (surge) conditions are within the assumed total (operating plus surge) pressure ratings of the system's ductile iron and PVC pipes. There are, however, issues with check valve slam at Lift Station B-1, and Lift Stations B-19 and B-1 regularly experience negative pressures. Further, it is quite likely that negative pressures routinely occur throughout the entire forcemain, and this is likely due to a combination of the system having a mixture of pipe materials, a lack of dedicated surge protection, possible vapor cavity formation and line draining, and frequent pump start and stop operations with insufficient hydraulic control to minimize transient pressures. Regarding the latter, motor soft-starters alone generally provide inadequate (if any) hydraulic control, which is particularly important at the larger Lift Stations B-19 and B-20. Compared to motor soft-starters, hydraulic control valves and variable frequency drives (VFDs; also referred to as variable speed drives) tend to provide much better hydraulic control with greater capability for controlling transient pressures resulting from routine operations.

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[1] Indeed, vapor cavity formation is important, but it is important to note that the ability of numerical models to accurately simulate vapor cavity formation and collapse is difficult. Furthermore, the nature of the fluid (i.e., wastewater) will likely limit the duration of potential vapor cavities due to the presence of dissolved gases and the high number of nucleation sites. Accordingly, the actual system response is likely to differ from the simulation results, though negative pressures and vapor cavity formation are nonetheless problematic.

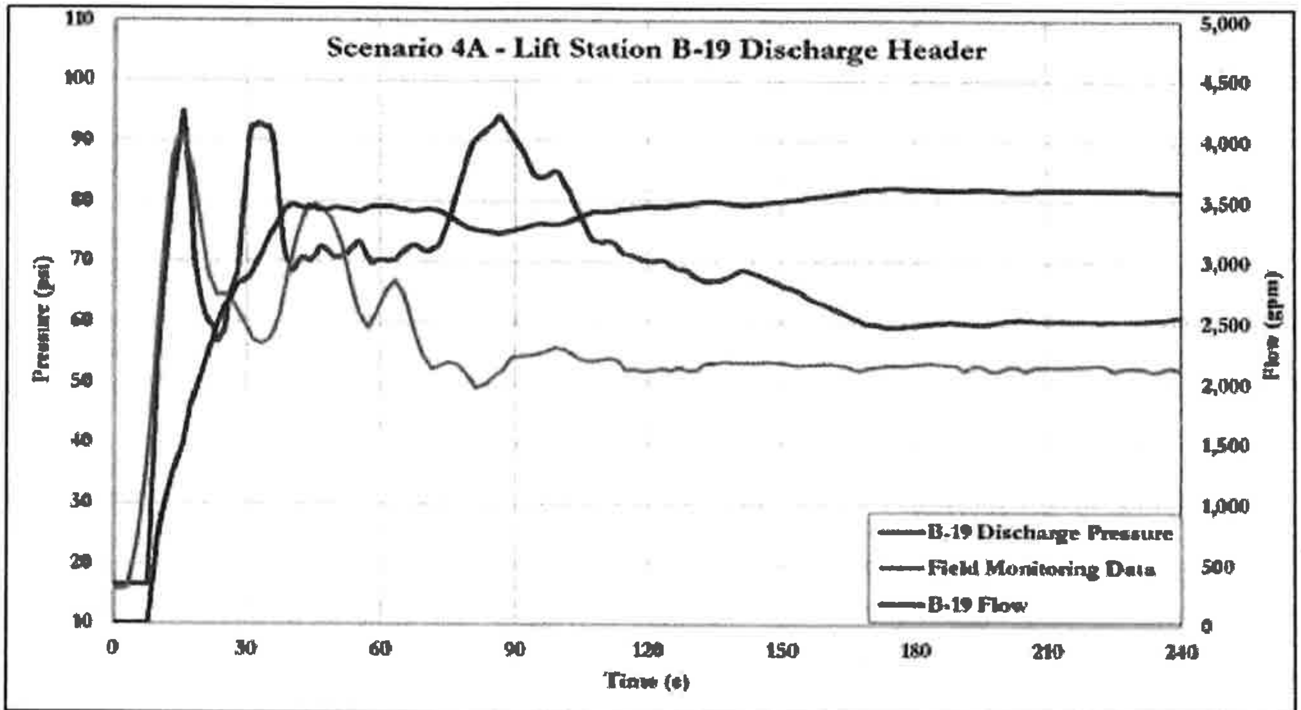


Figure 4: Comparison of simulation results for Scenario 4A (existing system, HWL, single pump start at Lift Station B-19) against Lift Station B-19 monitoring data on 30 September 2017 at 12:44 PM

Proposed Upgrades without Dedicated Surge Protection

Table 10 in Appendix A summarizes the simulation results for the east forcemain system with the proposed 30-inch ductile iron forcemain and upgrades to Lift Station B-1 (Scenarios 6A through 17B). As with the existing conditions, power failures are expected to create marginal upsurge pressures, yet negative pressures reaching full vacuum conditions are still predicted all along the forcemain from Lift Station B-19 to the WWTP. The negative pressures persist for up to roughly 10 s, which is less than that simulated for the existing conditions, likely due to the proposed 30-inch ductile iron forcemain having a higher wave speed than the existing PVC pipe. The simulation results show vapor cavities forming due to the negative pressures, and the subsequent vapor cavity collapse is predicted to create pressure spikes and high local stresses. Of the power failure scenarios for the potential system upgrades, Scenarios 7A and 9A are generally the worst: the former considers a system-wide power failure under full capacity pumping, while the latter scenario 9A considers a local power failure with only Lift Station B-20 on and at full capacity. Both scenarios result in larger vapor cavities forming compared to other scenarios. The total flow is high for Scenario 7A, so the resulting flow change for the forcemain is rather large, thus yielding extreme pressure drops. Scenario 9A features a high local flow at Lift Station B-20 due to no other system pumps being on, and this leads to an extreme local response that propagates to the rest of the system. Herein Scenarios 7A and 9A are considered as the base worst-case events due to having more severe responses, with both having overall greater vapor cavity volumes predicted along the forcemain compared to all other scenarios discussed thus far.

Scenarios 10 and 11, each based on Scenario 7A, test the system response with the proposed upgrades and potential future east-west forcemain interconnections on North Riverside Drive and along 2nd Avenue, respectively. The interconnections increase the downstream system capacity; consequently, the total upstream flow and thus initial upstream velocity are higher than for Scenario 7A. Though increased flows are beneficial for capacity, the results for Scenarios 10 (Figure 22) and 11 (Figure 23) suggest that the higher initial flows yield worse hydraulic transient responses with larger vapor cavities forming in the upstream segment of the forcemain.

Because a number of assumptions were made in developing the hydraulic transient model, sensitivity tests were performed to explore the effect of varying the wave speed (Scenarios 12 and 13), pump inertia (Scenario 14), and vapor cavity formation (Scenario 15). Scenario 9A was used as the base scenario for these due to having large predicted vapor cavity formation. In general, a comparison of the results for Scenarios 12 through 14 (Figure 23 to Figure 26 in Appendix B) against those of Scenario 9A shows that the wave speed and pump inertia indeed affect the results but not significantly. Regarding Scenario 15 (based on Scenario 9A without vapor cavity formation), Figure 27 in Appendix B shows a much lower minimum HGL envelope without vapor cavity formation, which suggests that vapor cavity formation and collapse is certainly possible for the system.

Lastly, Scenarios 16A through 17C test the system response to routine single pump start and stop operations with the proposed upgrades. Scenarios 16A, 16B, and 16C consider single pump starts at Lift Stations B-19, B-20, and B-1, respectively, while Scenarios 17A, 17B, and 17C consider single pump stops at the same respective facilities. From the results for the pump start scenarios (Figure 28 and Figure 33 in Appendix B), rather large upsurge pressures are predicted, with that at Lift Station B-19 reaching an estimated 120 psi and that at Lift Station B-20 reaching an estimated 110 psi. As with Scenarios 5A and 5B for the existing system, the results for the pump stop tests with the proposed upgrades (Scenarios 17A and 17B; Figure 31 and Figure 32 in Appendix C) show negative pressures following full pump stoppage in the system, with negative pressures reaching partial vacuum conditions along much of the east forcemain.

Proposed Upgrades with Surge Protection

From the simulation results for Scenarios 1A through 17B, the key hydraulic transients challenge in the South Beaches forcemain system (with or without the proposed upgrades) concerns negative pressures. Negative pressures reaching full vacuum are predicted throughout the system under a power failure condition. This can give rise to vapor cavity formation and collapse, which itself creates extreme local pressures and stresses that exacerbate pipe wear, ultimately and likely leading to pipe failures. The risk of negative pressures has also been confirmed via the parallel field pressure monitoring study. Two alternatives for controlling negative pressures that are considered as part of this analysis are (i) installing CAVs[2] at critical high points that will allow air to enter the forcemain under vacuum conditions to prevent local vapor cavity formation or (ii) introducing pressurized storage (e.g., surge tanks, aka HACs) that alleviates downsurges and thus negative pressures.

Scenarios 18A through 20C test the benefits of installing dedicated surge protection. The scenarios are based on the worst-case Scenarios 7A and 9A, which feature the largest predicted vapor cavity volumes. Scenarios 18A to 19B test the potential benefits of installing CAVs at forcemain high points located roughly 11,300 ft from Lift Station B-19 (near South Patrick Drive at Island Drive), 19,330 ft from Lift Station B-19 (near South Patrick Drive at Indrio Boulevard), and 47,210 ft from Lift Station B-19 (near North Shannon Avenue at Fifth Avenue) where previous scenarios show large vapor cavities forming. Scenarios 20A to 20C explore the system response with HACs (i.e., pressurized surge tanks) installed at Lift Stations B-19 and B-20. Table 8 lists the surge protection details.

[2] Combination air valves (CAVs) are a dual function air valve. Under pressurized conditions, they allow the controlled release of accumulated air through a small (and large) orifice, and under vacuum conditions, air enters the pipe through a larger orifice to alleviate negative pressures.

Table 8: List of surge protection simulated in Scenarios 18A to 20C

Scenario	Parent Scenario	Conditions	Surge Protection
18A	7A	Full Capacity, LWL, GPF+	2-inch standard CAVs at select locations*
18B	9A	Full Capacity at B-20, LWL, LPF+	2-inch standard CAVs at select locations*
19A	7A	Full Capacity, LWL, GPF+	2-inch three-stage CAVs at select locations*
19B	9A	Full Capacity at B-20, LWL, LPF+	2-inch three-stage CAVs at select locations*
20A	7A	Full Capacity, LWL, GPF+	4,000 gal HAC at B-19
20B	7A	Full Capacity, LWL, GPF+	10,000 gal HAC at B-20
20C	7A	Full Capacity, LWL, GPF+	4,000 gal HAC† at B-1
20D	7A	Full Capacity, LWL, GPF+	3,000 gal HAC† at B-19 6,000 gal HAC† at B-20

* CAVs were modeled at roughly 11,300 ft (near South Patrick Drive at Island Drive), 19,330 ft (near South Patrick Drive at Indrio Boulevard), and 47,210 ft (near North Shannon Avenue at Fifth Avenue) from Lift Station B-19.

+GPF = global power failure and LPF = local power failure.

† HAC = hydropneumatic air chamber with a bladder separating the liquid and gas phases. For Scenario 20A, the Lift Station B-19 HAC was modeled with a 5-inch connection, and for Scenario 20B the Lift Station B-20 HAC was modeled with an 8-inch connection. The Lift Stations B-19 and B-20 HACs were modeled with 4-inch and 6-inch connections, respectively, for Scenario 20C. The bladders were modeled with a pre-charge pressure of 10 psi, yielding initial air volumes of 22% and 24% for Lift Stations B-19 and B-20, respectively.

Table 11 in Appendix A summarizes the simulation results for Scenarios 18A through 20C. In general, the Scenarios 18A to 19B results (Figure 34 to Figure 37 in Appendix B) show that CAVs help alleviate the formation of some vapor cavities. However, negative pressures are still problematic along the entire forcemain, as the CAVs mostly provide localized benefits in their vicinity. Furthermore, reliance on multiple CAVs for surge protection requires all critical CAVs to be operational and in working order, and this can seldom be ensured for wastewater systems where solids in the fluid can damage CAVs.

Scenario 20A tests the benefits of installing a 4,000 gal HAC only at Lift Station B19. Similarly, Scenario 20B tests a 10,000 gal HAC only at Lift Station B-20. Tank sizes of 2,000 gal were initially tested (the results are not reported here) based on PBS&J's surge analysis report, and the current sizes were determined as the minimum necessary to keep the tanks from draining. The Scenario 20A results (Figure 38 and Figure 39 in Appendix B) show that an HAC at Lift Station B-19 helps reduce the formation of vapor cavities, though there are still some along the forcemain. Comparatively, the results for Scenario 20B with an HAC at Lift Station B-20 (Figure 40 and Figure 41 in Appendix B) show greater improvement over the parent Scenario 7A as well as Scenario 20A, with negative pressures eliminated along a majority of the east forcemain. To explore the system response with an HAC installed at Lift Station B-1, Scenario 20C tests a 4,000 gal HAC (the minimum size from Scenarios 20A and 20B). The results (Figure 42 and Figure 43 in Appendix B) show little change in the overall system response, indicating that a larger HAC is required and that Lift Station B-1 is likely not an optimal location due to being a smaller lift station and thus less severe source of transient pressures. Lastly, the results for Scenario 20D (Figure 44, Figure 45, and Figure 46 in Appendix B) suggest that installing an HAC at each of Lift Station B-19 (3,000 gal) and Lift Station B-20 (6,000 gal) nearly eliminates negative pressures altogether, with a few sections experiencing pressures down to -5 psi. These results indicate that HACs are the most effective alternative for controlling transient pressures in the system, and though not tested here, they would also benefit the system for routine pump starts and stops should no changes be made to the pump start-stop controls. The HAC sizes listed above are the minimum necessary to prevent draining, and larger sizes will provide an even greater benefit to the system in controlling transient pressures and likely even eliminating the occurrence of negative pressures.

CONCLUSIONS AND RECOMMENDATIONS

In response to previous failures in the South Beaches forcemain system, HDR was retained by Brevard County to review previous studies, conduct a transient pressure monitoring exercise, and propose alternatives for improving the system. In support of these, HydraTek performed a review of the field monitoring data, and a numerical hydraulic transient (surge) analysis was completed parallel thereto to analyze the existing system, proposed upgrades, and identify potential improvements. As with most flat forcemains, the key hydraulic transient challenge for the South Beaches system concerns negative pressures and frequent pump start-stop operations; moreover, the system response is complex and largely influenced by the mixed use of ductile iron and PVC pipes. Based on the review of the field monitoring data and numerical modeling analysis, the following concluding remarks and recommendations are provided to support HDR's work:

1. From a review of the transient pressure field monitoring data, the maximum pressures recorded at Lift Stations B-19 and B-1 are 148 psi and 130 psi, respectively. The average high pressures recorded at Lift Stations B-19 and B-20 are also 77 psi and 54 psi, respectively. At the time of this analysis, the working and total (operating plus surge) pressure ratings of the system's ductile iron and PVC pipes were not available; accordingly, the actual working and total pressure ratings of the ductile iron and PVC forcemain sections should be confirmed and compared against the aforementioned values.

2. Although the maximum recorded transient pressures are likely within the forcemain's upper pressure limit for the PVC and ductile iron pipe, it is quite possible that the combined effect of frequent pump start-stop events; a lack of suitable hydraulic control during such operations; and potential forcemain draining, vapor cavity collapse, or rapid air expulsion contributed to or exacerbated the previous failures.

3. The numerically predicted negative pressures should be avoided for the sake of maintaining joint seals – see ASTM D3139-98 (Standard Specification for Elastomeric Seals (Gaskets) for Joining Plastic Pipes), which recommends a minimum partial vacuum pressure of -10 psi; otherwise, repeated positive-negative pressure cycles can wear the seals, leading to leakage and ultimately joint failure.

4. In addition to the above, a fatigue loading analysis can be performed for the PVC section of the forcemain to determine whether repetitive pressure cycling from frequent pump start-stop operations is contributing to the forcemain failures. Such an analysis should consider a pressure range of -10 psi to 90 psi (the average recorded upsurge pressure) and a frequency of 30 complete cycles per system per day.

5. The field monitoring data for Lift Station B-1 shows pressure traces that are characteristic of check valve slam following pump stops. Check valve slam can create significant pressure spikes, and it causes high local stresses that can lead to or cause pipe and valve failure. The design of the proposed upgrades for Lift Station B-1 should consider check valve slam and the selection of better quality check valves. This may be accomplished by using valves with shorter closure strokes/angles, and either check valve accelerators (to close the valve faster before the arrival of a return wave) or dampeners (to cushion and slow the valve's closure).

6. It appears that the motor soft-starters at Lift Stations B-19 and B-20 provide less than adequate hydraulic control for pump starts (for example, as shown in Figure 57). There is also inadequate control for pump stops, and the field data shows that Lift Stations B-19, B-1, and likely B-20 routinely experience negative pressures reaching partial vacuum conditions following full pump stoppage for the system. The resulting peak pressures are also on the order of two to three times as high as the average operating pressures. Furthermore and as is typical of sewage forcemain systems, pump start and stop operations are rather frequent and likely contributing to cyclic fatigue of the piping and exacerbating failures. Consideration should thus be given to improving hydraulic control for pump starts and stops to better control transient pressures.

7. Two alternatives for better controlling pump operations are discharge control valves and VFDs. Discharge control valves provide direct hydraulic control by modulating the head and flow of individual pumps, whereas VFDs control the operating speed of a pump and thus its pump curve and operating point. Using either alternative, operations can be performed more slowly during pump starts and stops, thereby minimizing transient pressures and the shock to the system. Between control valves and VFDs, the former tend to be less costly and provide more direct control, though they are less common in wastewater applications due to the nature of the fluid. Comparatively, VFDs, while more expensive, can be used to operate the pumps at lower speeds during low inflow periods to minimize unnecessary pump on-off operations while improving operational efficiency. Another option is to stagger the pump start-stop wet well levels by a larger distance than is presently set to reduce the frequency of start-stop changes.

8. The data for Lift Station B-19 shows high frequency and large transient pressure fluctuations following full pump stoppage for the system (i.e., with no pumping from Lift Stations B19, B-20, and B-01). This is likely the result of either or a combination of: (i) sections of the forcemain draining, refilling, and re-pressurizing, (ii) rapid air intake, accumulation, or expulsion, or (iii) vapor cavity formation and collapse. The cause of these pressure fluctuations should be further investigated, starting with an investigation as to why the system pressures are low and even negative following complete stoppage. This may be tested by fully stopping all pumps for a period of at least 5 minutes (e.g., during low flow periods overnight), wet well levels permitting, and recording the system pressures at Lift Stations B-19, B-20, and B-1 to explore what the static system hydraulics are without pumping (and in comparison to an expected static HGL based on the WWTP outlet elevation of 30.5 ft.) A flow meter suited to wastewater applications can also be used to monitor potential backflow and forcemain draining. If not properly mitigated, this phenomenon may further contribute to failures by causing rapid and significant stress changes.

9. Further to the above, the original design and forcemain should be reviewed to inspect whether there are existing in-line check valves at any location, including at the WWTP. If not, consideration should be given to exploring the possibility of installing in-line check valves along the forcemain to prevent it from draining and depressurizing under conditions with no pumping.

10. The field monitoring data should be compared against the County's SCADA data for the South Beaches system to correlate operational changes to observed events. Furthermore, the system response and behavior can be further investigated with a coordinated transient pressure monitoring investigation, whereby more pressure loggers are installed and controlled, planned tests (such as pump emergency stops) are performed.

11. The proposed 30-inch ductile iron forcemain replacement and Lift Station B-1 upgrade will increase the system's capacity and thus maximum total flow. Despite this, the system response is expected to remain comparable to its current state, featuring negative pressures reaching full vacuum conditions along the forcemain and potentially with vapor cavity formation and collapse. Negative pressures can damage pipe seals, particularly for the PVC sections of the forcemain, and vapor cavity collapse creates high localized pressures and extreme stresses on the forcemain.

    Regarding the proposed 30-inch ductile iron forcemain replacement, the corresponding pipe class can be chosen according to the maximum expected operating pressure rather than transient pressure. For the Lift Station B-1 upgrades, check valve slam should be appropriately considered with design pressures based on the field monitoring data. It is also recommended that consideration be given to installing either discharge control valves or VFDs for new pump installations and modifications (e.g., at Lift Station B-1) to improve the hydraulic control of routine pump start and stop operations and thus minimize the transient pressures and shock to the system. All piping and appurtenances should also be designed to withstand full negative pressures reaching up to -14.7 psi, as well as have adequate bedding and structural support to accommodate frequent loading cycles.

12. Presently the South Beaches forcemain does not have any dedicated surge protection. The surge valves at each of Lift Stations B-19 and B-20 are currently isolated, and all air valves installed along the forcemain are air release (outflow only) valves. From the field data, numerical modeling results, and history of failures, the South Beaches system would benefit from the installation of dedicated surge protection. Two alternatives for this are (i) installing combination air valves (CAVs) along the forcemains at critical locations, namely local high points, or (ii) installing hydropneumatic air chambers (HACs; i.e., surge tanks) at Lift Stations B-19 and B-20.

    The analysis explored the potential benefits of installing CAVs at select locations on the east forcemain. The results suggest that CAVs help alleviate the formation and collapse of some vapor cavities, including the subsequent pressure shocks and high local pipe stresses that may otherwise exacerbate or cause failures. Though beneficial, CAVs are expected to only provide localized benefits to the system where they are installed, and negative pressures remain problematic along nearly all of the forcemain. Moreover, reliance on multiple CAVs for surge protection requires that all of the CAVs be maintained, operational, and reliable, and CAVs are typically less reliable than surge tanks and for wastewater applications. Thus, CAVs alone are insufficient unless combined with other forms of surge protection and better hydraulic control for pump start and stop operations.

    Numerical simulations were also performed to assess the benefits of installing HACs (i.e., pressurized surge tanks) at each of the larger Lift Stations B-19 and B-20. The model results suggest that suitably sized HACs (in the range of 4,000 gallons to 10,000 gallons per facility) can eliminate nearly all negative pressures along the forcemain. This is highly beneficial for avoiding vapor cavity formation, frequent and rapid pressure cycling, and wear on joint seals. Given the system's history of failures, this option should be given further consideration. The current study only explored a limited number of HAC combinations to demonstrate their potential benefits, with the sizes mainly determined to avoid tank draining. A more detailed analysis can be conducted to optimize the number of HACs, their sizes and locations, and performance – for instance, designing the tanks to not only avoid draining but also have a minimum allowable water volume/content under transient conditions (e.g., a minimum 10% of the overall tank volume) and/or a minimum water depth (i.e., to avoid vortex action at the outlet).

13. During and following commissioning of the proposed upgrades, transient pressure monitoring can be carried out to further identify and monitor the overall risk to the system. This would require the installation of high frequency transient pressure monitors similar to those used by Blacoh and ideally ones that can remotely transmit the data for ongoing review and confirmation.

14. Following a power failure event (or emergency pump stop or trip), Lift Stations B-19, B-20, and B-1 should stay idle for a period of at least 3 minutes in order to allow the transient pressure waves to dissipate. Therefore, the pumps should not be allowed to start (via utility or backup power) within this designated standby period unless the wet well level is at the high-high alarm. The mixed pipe material nature of this overall system further increases the risk of pressure wave superimposition and amplification.

APPENDIX A: TABULAR SUMMARY OF NUMERICAL MODELING RESULTS

Table 9: Hydraulic transient simulation results for existing conditions – Scenarios 1A to 5B

Scenario	Parent Scenario	Initial Lift Station Flows (gpm)					20-inch Forcemain			24/30-inch Force	
		B19	**B20**	**B01**	**B06**	**B07**	**Min. Pressure (psi)**	**Max. Pressure (psi)**	**Max. Vapour Vol. (gal)**	**Min. Pressure (psi)**	**Max. Pressure (psi)**
1A	-	2,289	3,359	962	-	-	**-14**	**108**	3	**-14**	**97**
1B	1A	2,241	3,380	889	-	-	**-14**	106	5	**-14**	95
2	-	4,529	0	0	-	-	**-14**	96	**14**	**-14**	58
3	-	0	5,597	0	-	-	**-14**	81	9	**-14**	79
4A	-	0	0	0	-	-	11	95	0	0	70
4B	-	0	0	0	-	-	11	88	0	0	82
5A	-	3,565	0	0	-	-	-10	68	0	-10	43
5B	-	0	4,143	0	-	-	-12	53	0	**-14**	52
Min.	-	-	-	-	-	-	-14	-	-	-14	-
Max.	-	4,529	5,597	962	-	-	-	108	14	-	97

Table 10: Hydraulic transient simulation results for proposed conditions without surge protection – Scenarios 6A tc

Scenario	Parent Scenario	Initial Lift Station Flows (gpm)					20-inch Forcemain			24/30-inch Forc	
		B19	B20	B01	B06	B07	Min. Pressure (psi)	Max. Pressure (psi)	Max. Vapour Vol. (gal)	Min. Pressure (psi)	Max. Pressure (psi)
6A	-	2,406	4,302	1,051	-	-	-14	107	6	-14	95
6B	6A	2,528	3,945	763	-	-	-14	100	8	-14	86
7A	-	2,355	4,318	973	-	-	-14	105	8	-14	93
7B	7A	2,477	3,939	717	-	-	-14	98	10	-14	84
8A	-	4,800	0	0	-	-	-14	92	25	-14	50
8B	8A	4,459	0	0	-	-	-14	82	20	-14	57
9A	-	0	6,758	0	-	-	-14	82	41	-14	80
9B	9A	0	6,137	0	-	-	-14	79	31	-14	70
10	7A	2,755	5,390	1,694	978	794	-14	103	34	-14	87
11	7A	2,635	5,060	1,469	1,480	844	-14	104	31	-14	89
12	9A	0	6,758	0	-	-	-14	82	40	-14	80
13	9A	0	6,758	0	-	-	-14	82	22	-14	80
14	9A	0	6,758	0	-	-	-14	82	**46**	-14	80
15	9A	0	6,758	0	-	-	-55	82	0	-41	80
16A	-	3,768*	0	0	-	-	11	120	0	0	92
16B	-	0	4,687*	0	-	-	11	**122**	0	0	**110**
16C	-	0	0		-	-	-11	62	0	-14	60
17A	-	3,768	0	0	-	-	-10	65	0	-10	38
17B	-	0	4,687	0	-	-	-11	49	0	-11	48
17C	-				-	-	-14	40	0	-12	32
Min.	-	-	-	-	-	-	-55	-	-	-41	-
Min.+	-	-	-	-	-	-	-14	-	-	-14	-
Max.	-	4,800	6,758	1,694	1,480	844	-	122	46	-	110

+Excluding the results for Scenario 15 with no vapor cavity formation.

Table 11: Hydraulic transient simulation results for proposed conditions with surge protection – Scenarios 18A to 2(

Scenario	Parent Scenario	Initial Lift Station Flows (gpm)					20-inch Forcemain			24/30-inch Force		
		B19	B20	B01	B06	B07	Min. Pressure (psi)	Max. Pressure (psi)	Max. Vapour Vol. (gal)	Min. Pressure (psi)	Max. Pressure (psi)	
18A	7A	2,355	4,318	973	-	-	**-14**	160	2,627	-14	102	
18B	9A	0	6,758	0	-	-	**-14**	**169**	**3,003**	-14	88	
19A	7A	2,355	4,318	973	-	-	**-14**	128	2,298	-14	93	
19B	9A	0	6,758	0	-	-	**-14**	114	2,612	-14	80	
20A	7A	2,355	4,318	973	-	-	**-14**	105	8	-14	93	
20B	7A	2,355	4,318	973	-	-	-13	105	0	-7	93	
20C	7A	2,355	4,318	973	-	-	-14	105	19	-14	93	
20D	7A	2,355	4,318	973	-	-	-5	105	0	-3	93	
Min.	-	-	-	-	-	-	-14	-	-	-14	-	
Max.	-	2,355	6,758	973	-	-	-	169	3,003	-	102	

APPENDIX B: GRAPHICAL HYDRAULIC TRANSIENT ANALYSIS RESULTS

This appendix presents the numerical hydraulic transient analysis results for the South Beaches east forcemain system via transient pressure envelope figures. The transient pressure envelope figures are plotted as hydraulic head (pressure) versus distance along the forcemain profile path from Lift Station B-19 to the South Beaches Wastewater Treatment Plant (WWTP). The profile figures should be read from left to right. The left side corresponds to the location of Lift Station B-19, and the right side corresponds to the South Beaches WWTP (refer to Figure 5 on the following page). Lift Stations B-20 and B-1 are located at distances of approximately 17,750 ft and 29,100 ft from Lift Station B-19, respectively. The hydraulic head profiles are representative of that along the red-highlighted line in Figure 5 on the following page.

Unless otherwise noted, all of the values are presented in the units of feet (ft) and all figures can be deciphered using the following legend:

Red Line:	Estimated vapor or air cavity volume in gallons
Light Blue Line:	Steady-state (operating) hydraulic grade line (HGL)
Blue Lines:	Maximum and minimum transient pressure envelope (HGLs)
Green Line:	Pipe profile (elevation)

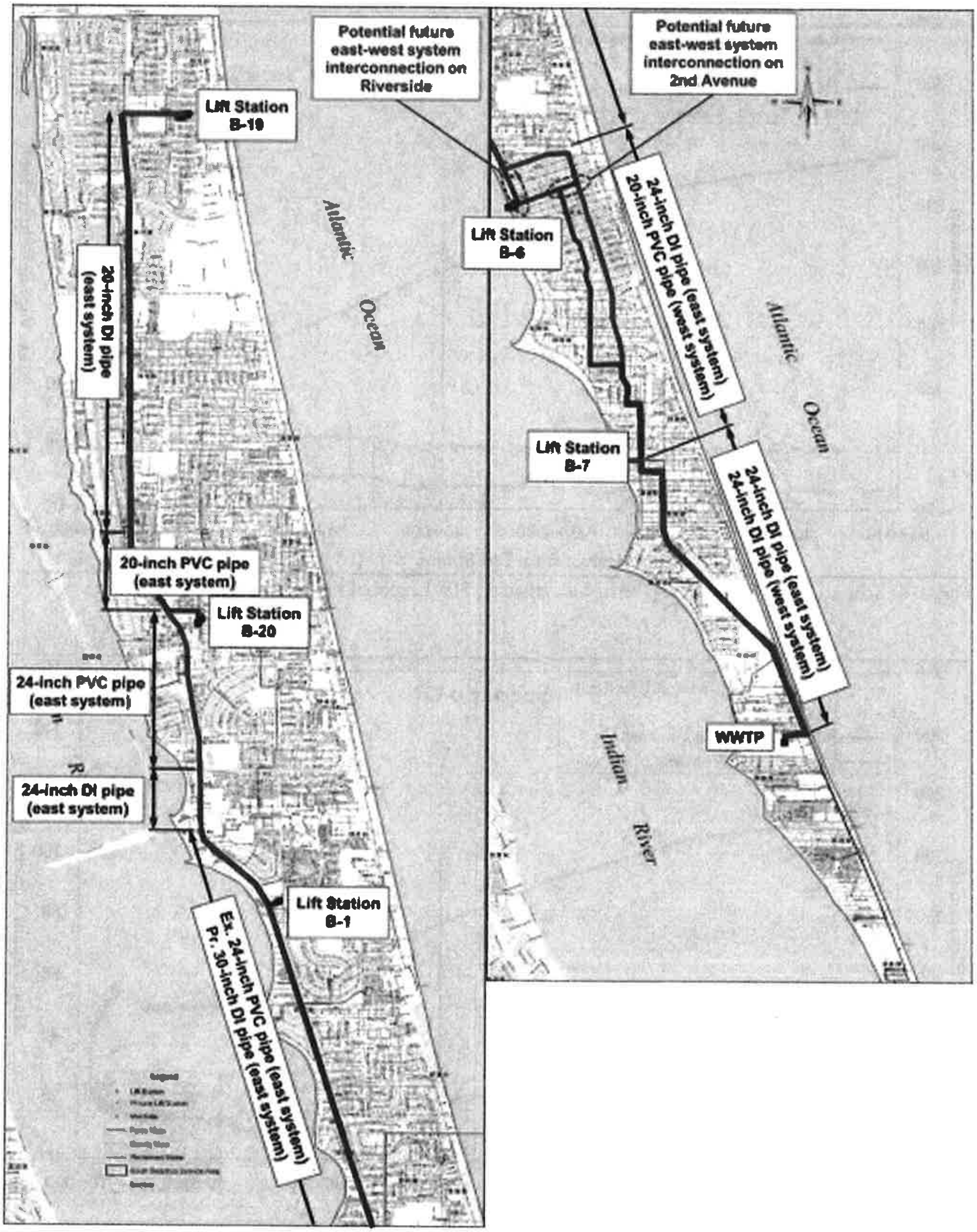


Figure 5: Profile path for analysis results, from Lift Station B-19 (top-left) to the South Beaches WWTP (bottom-right)

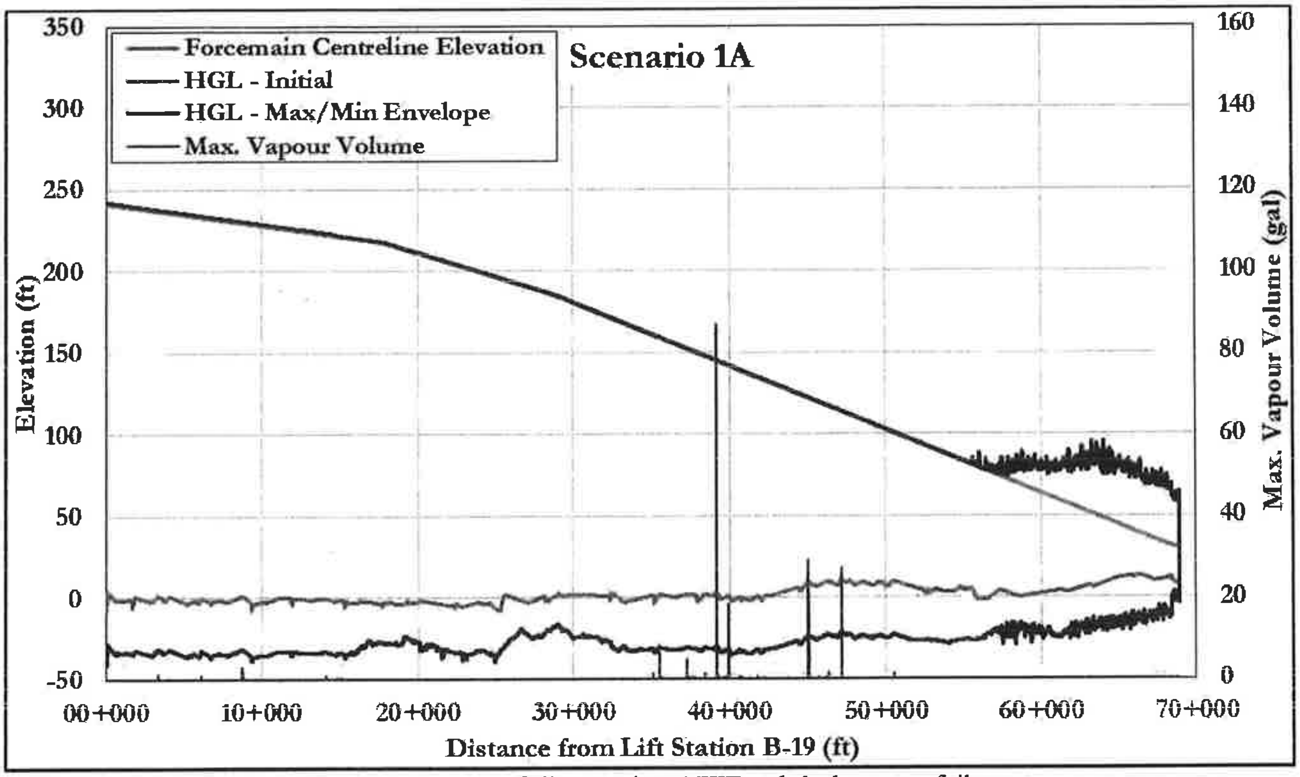


Figure 6: Scenario 1A – Existing system, full capacity, HWL, global power failure

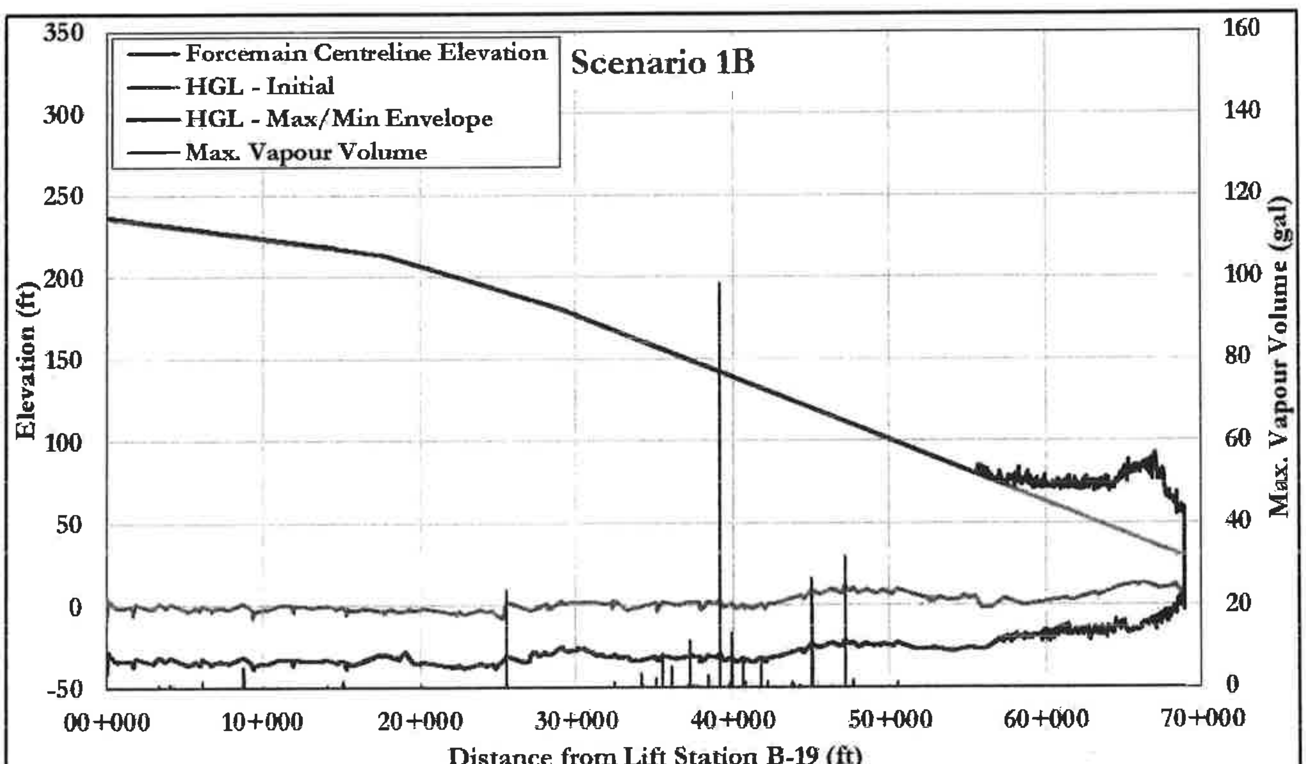


Figure 7: Scenario 1B – Existing system, full capacity, LWL, global power failure

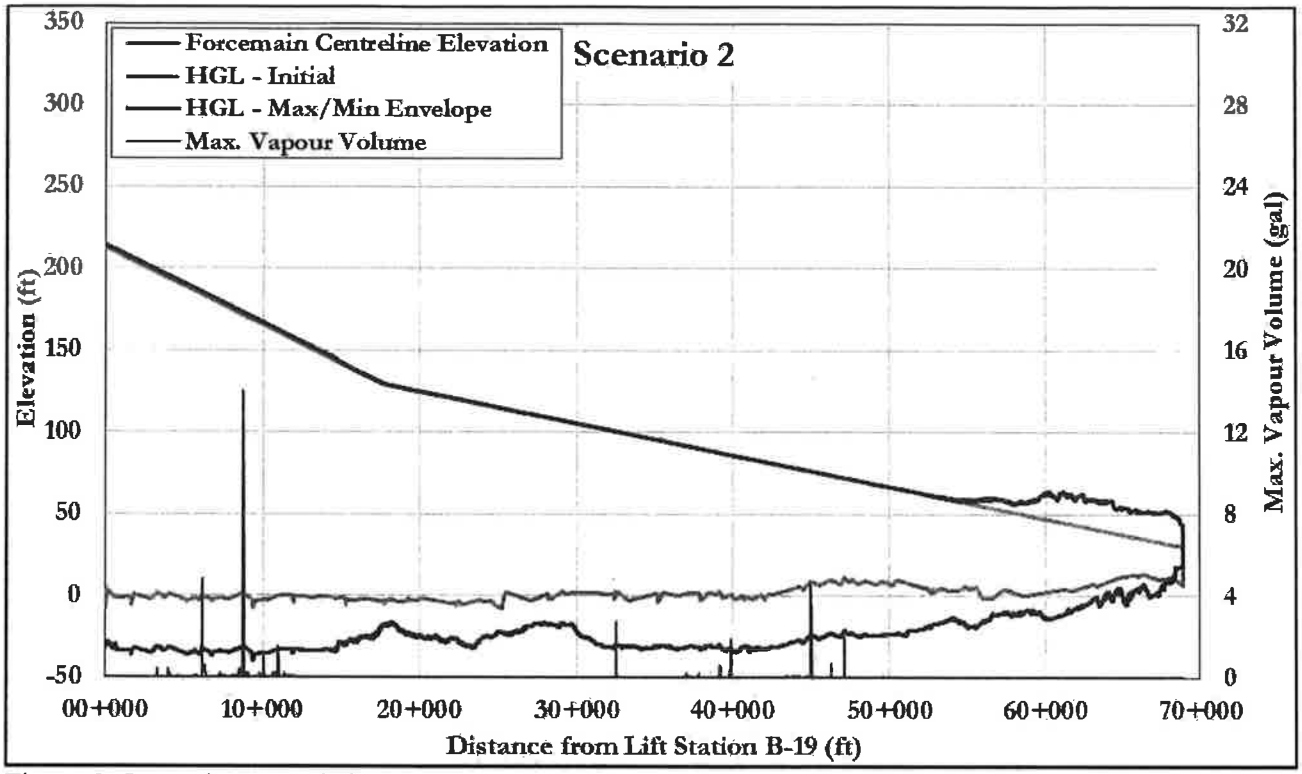


Figure 8: Scenario 2 – Existing system, only B-19 on at full capacity, HWL, local power failure

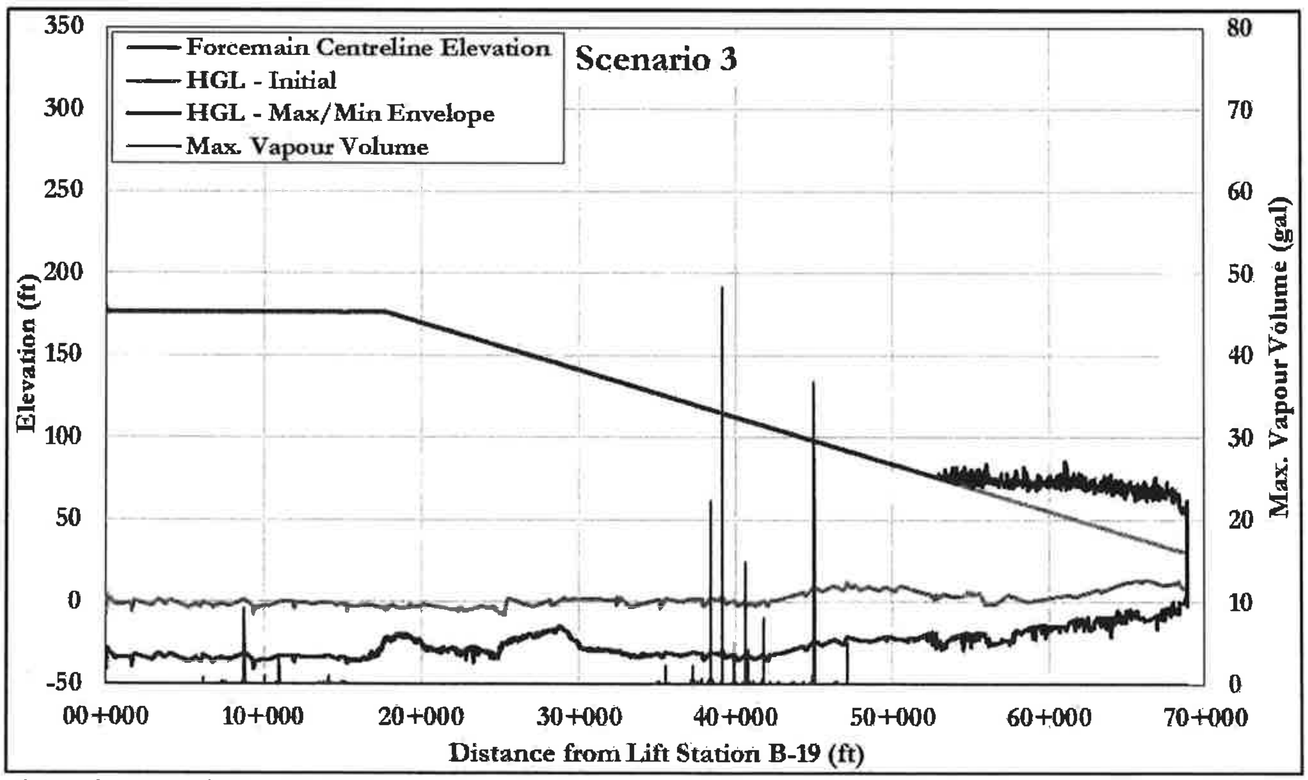


Figure 9: Scenario 3 – Existing system, only B-20 on at full capacity, HWL, local power failure

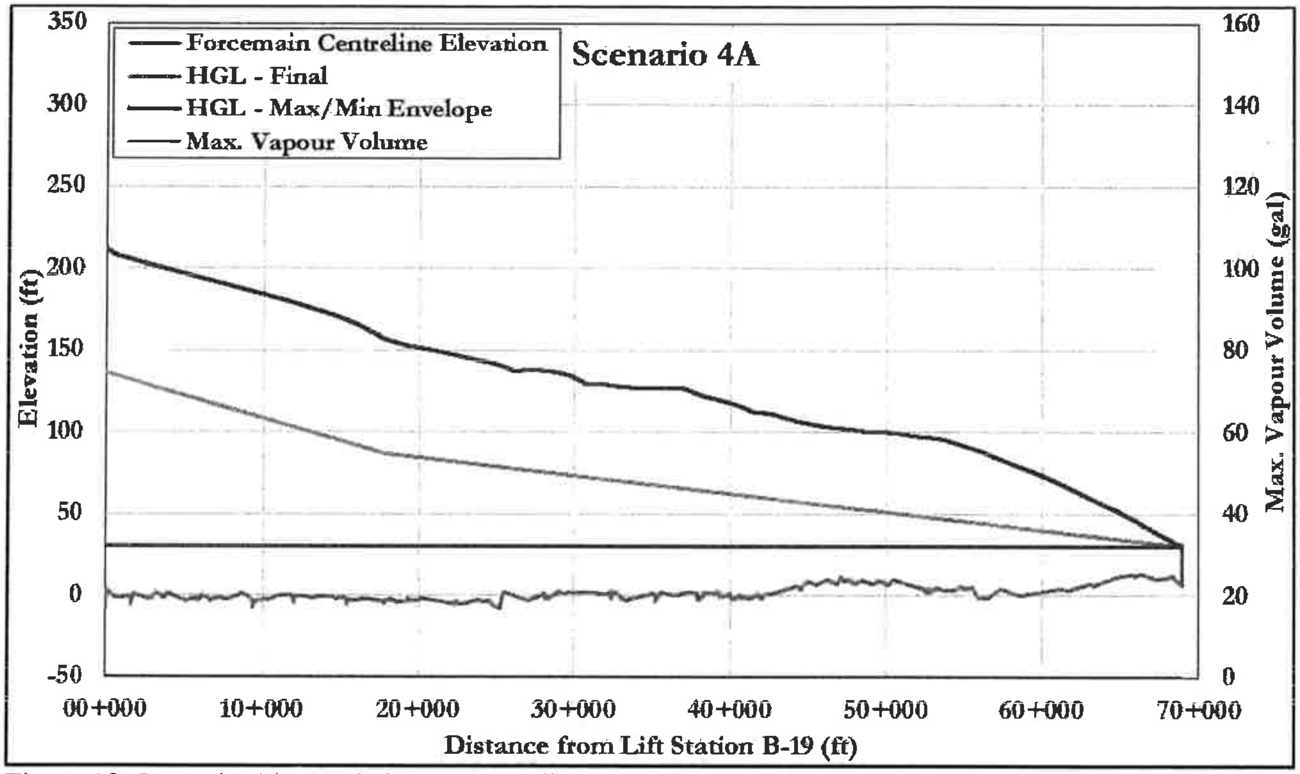


Figure 10: Scenario 4A – Existing system, all pumps off, HWL, single pump start at B-19

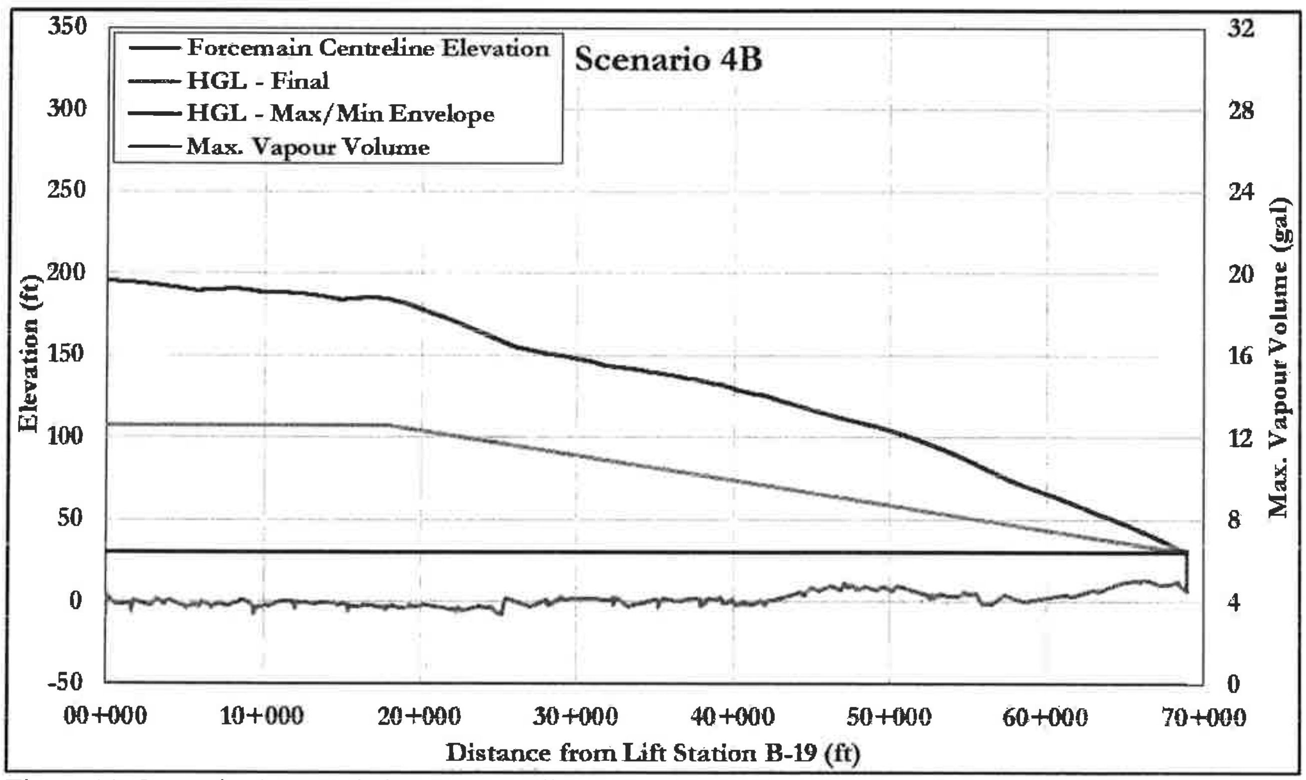


Figure 11: Scenario 4B – Existing system, all pumps off, HWL, single pump start at B-20

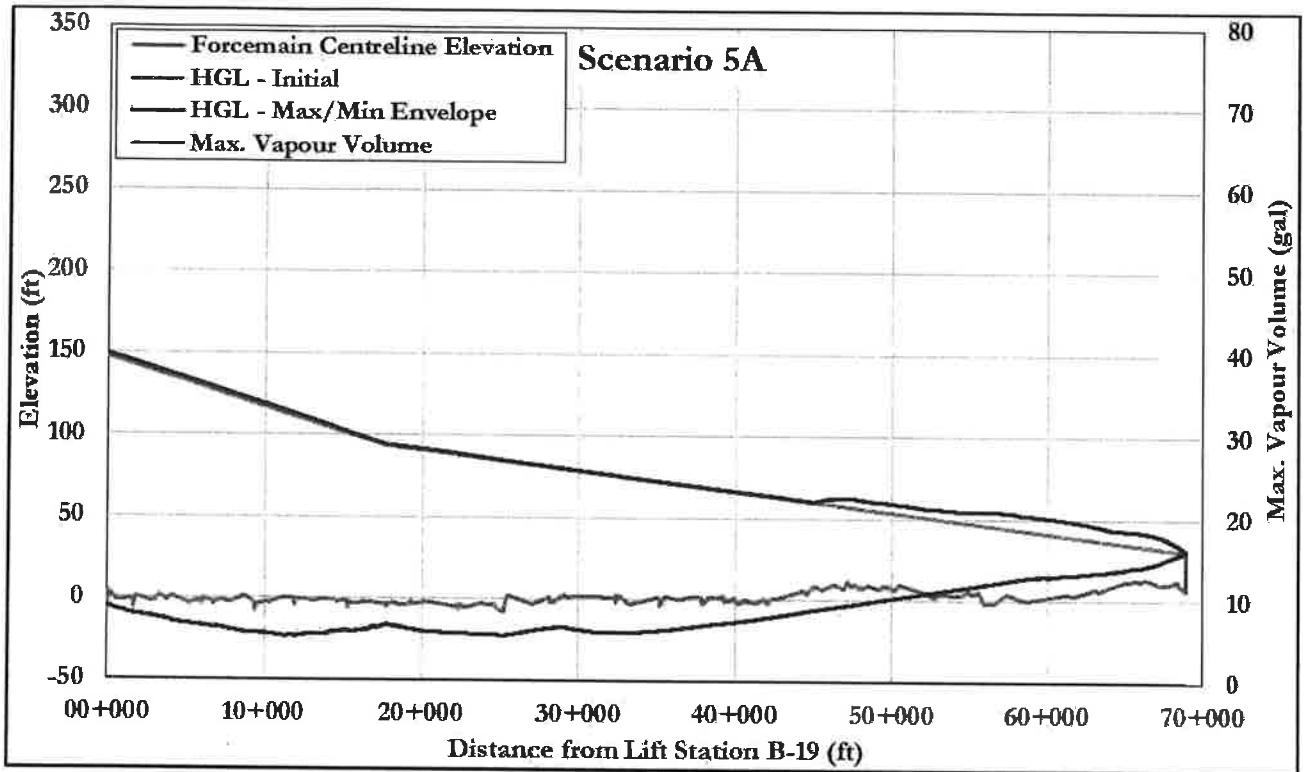


Figure 12: Scenario 5A – Existing system, HWL, single pump stop at B-19

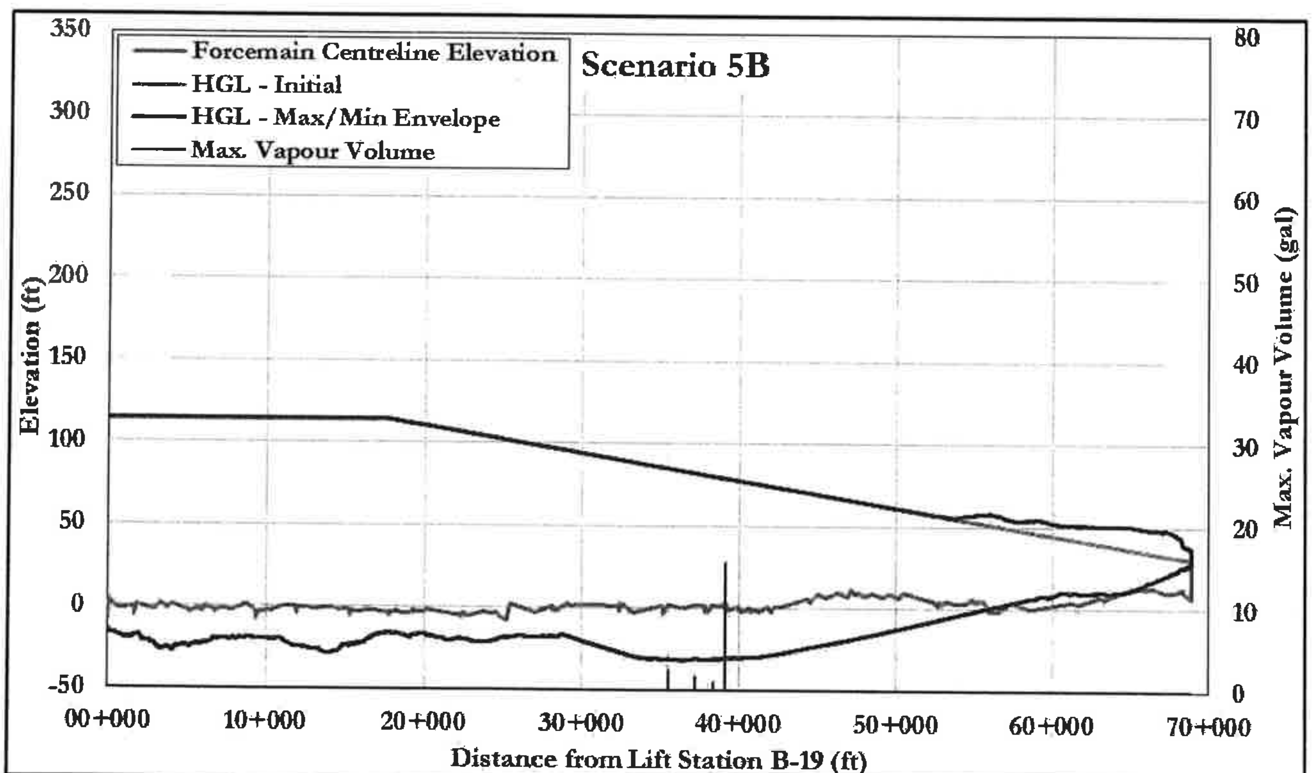


Figure 13: Scenario 5B – Existing system, HWL, single pump stop at B-19

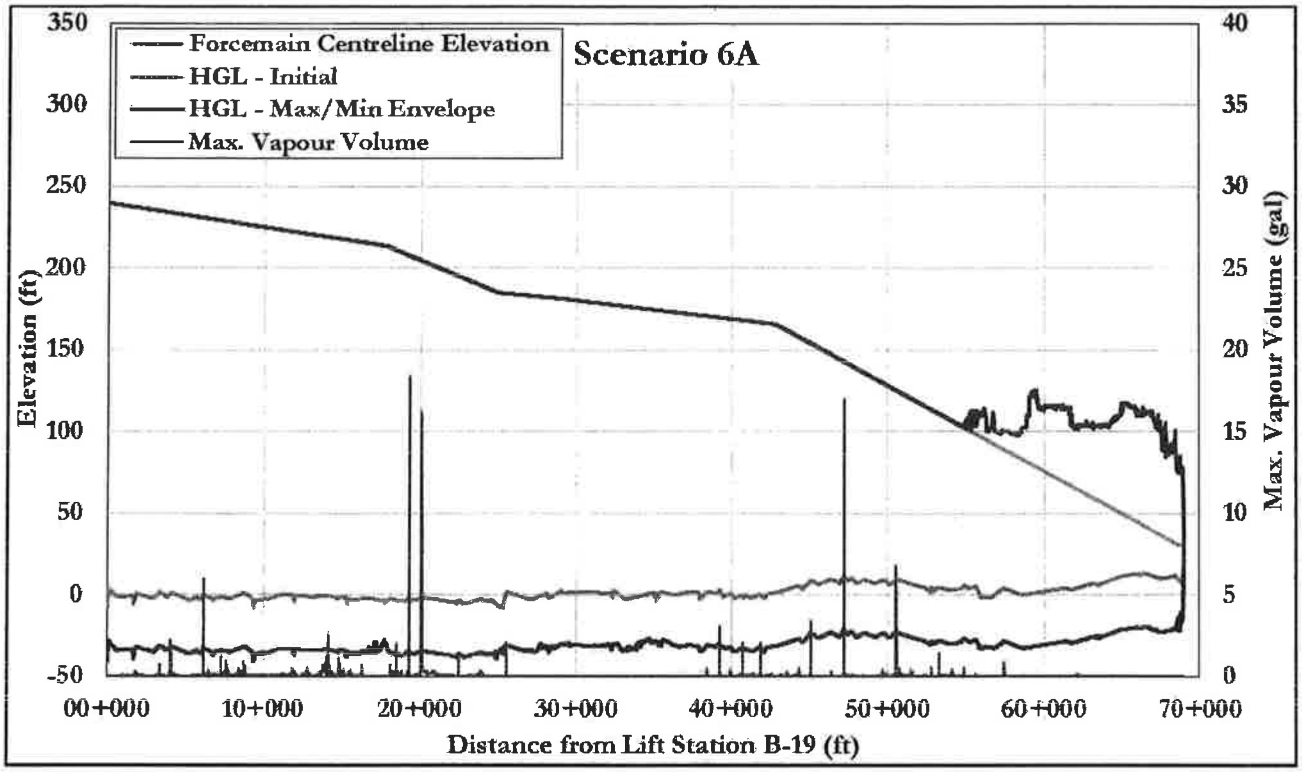


Figure 14: Scenario 6A – Proposed upgrades, full capacity pumping, HWL, global power failure

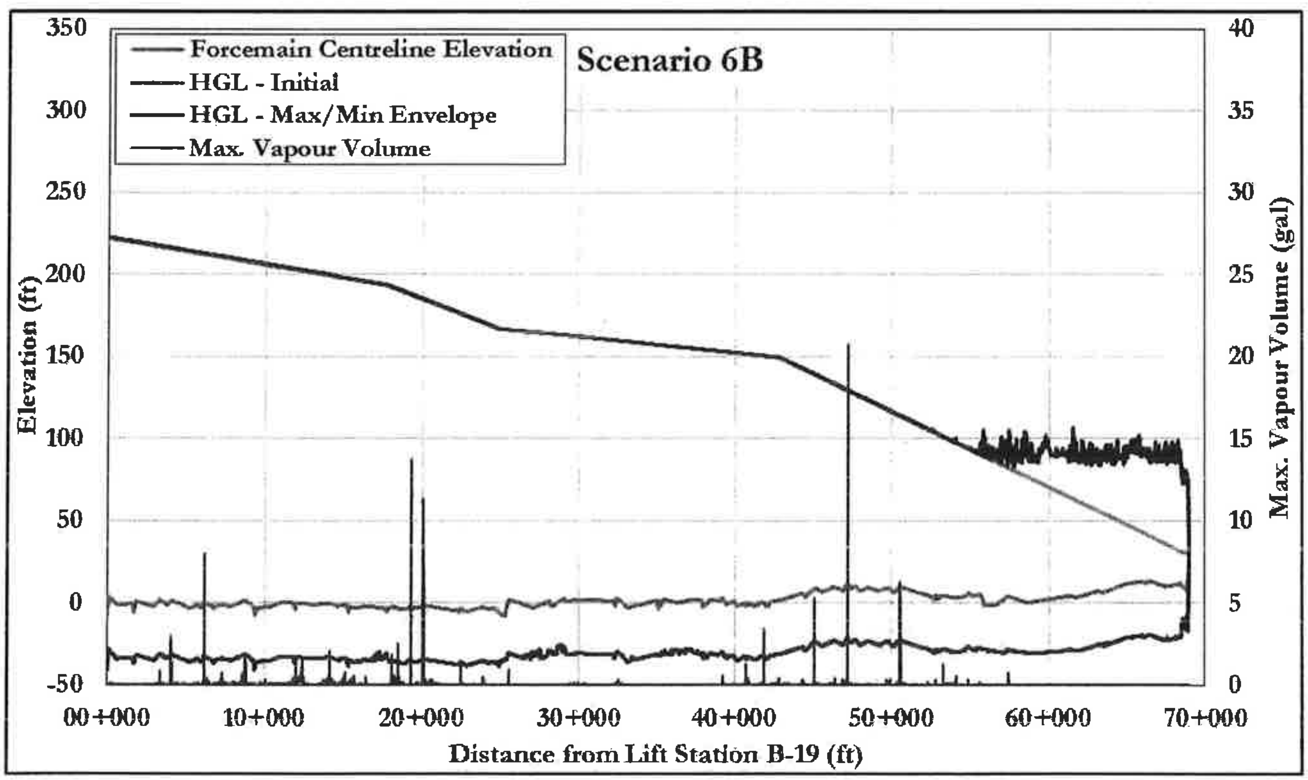


Figure 15: Scenario 6B – Proposed upgrades, firm capacity pumping, HWL, global power failure

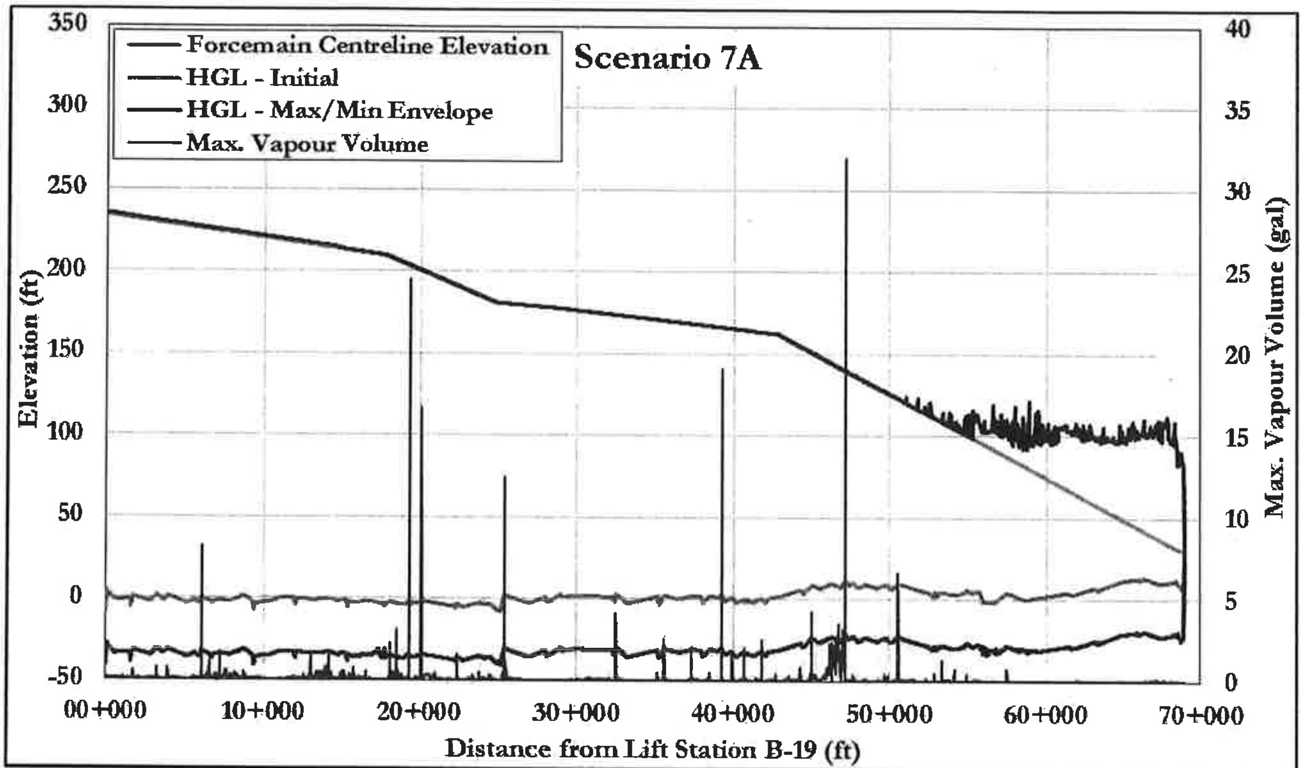


Figure 16: Scenario 7A – Proposed upgrades, full capacity pumping, LWL, global power failure

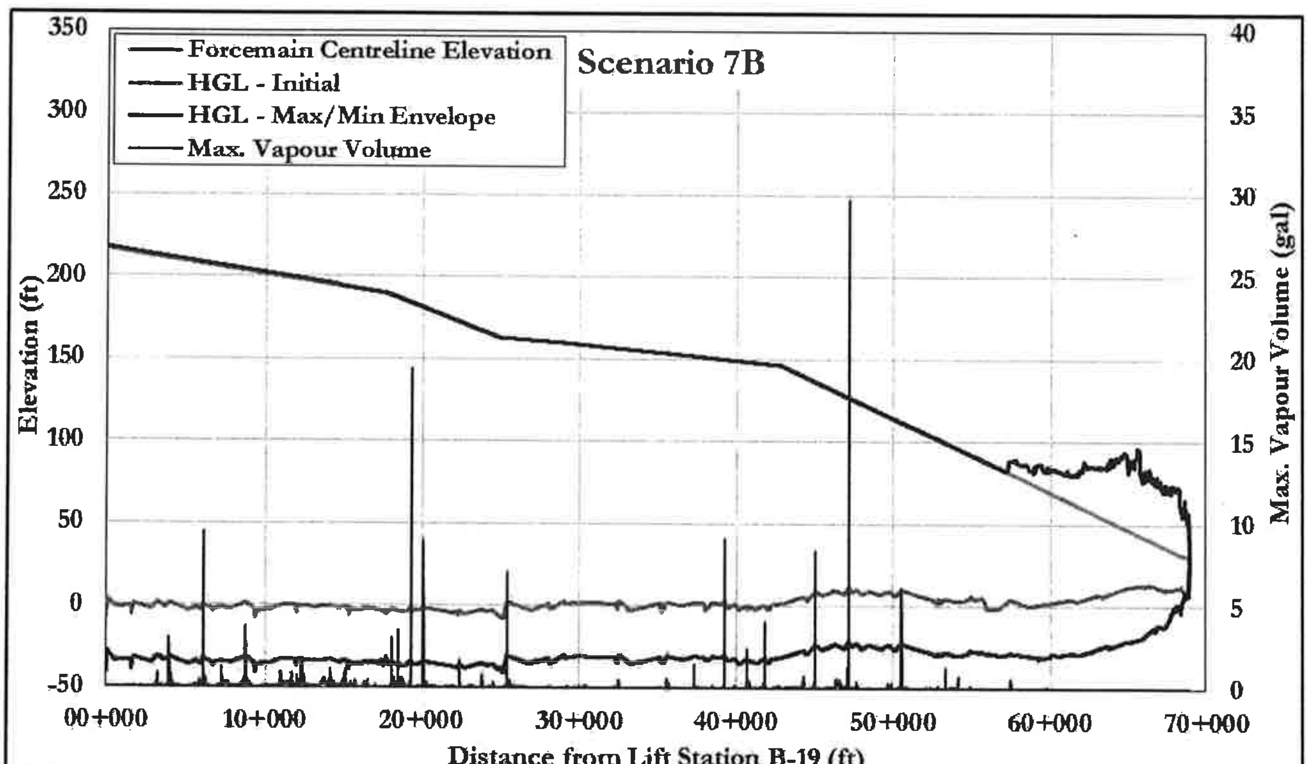


Figure 17: Scenario 7B – Proposed upgrades, firm capacity pumping, LWL, global power failure

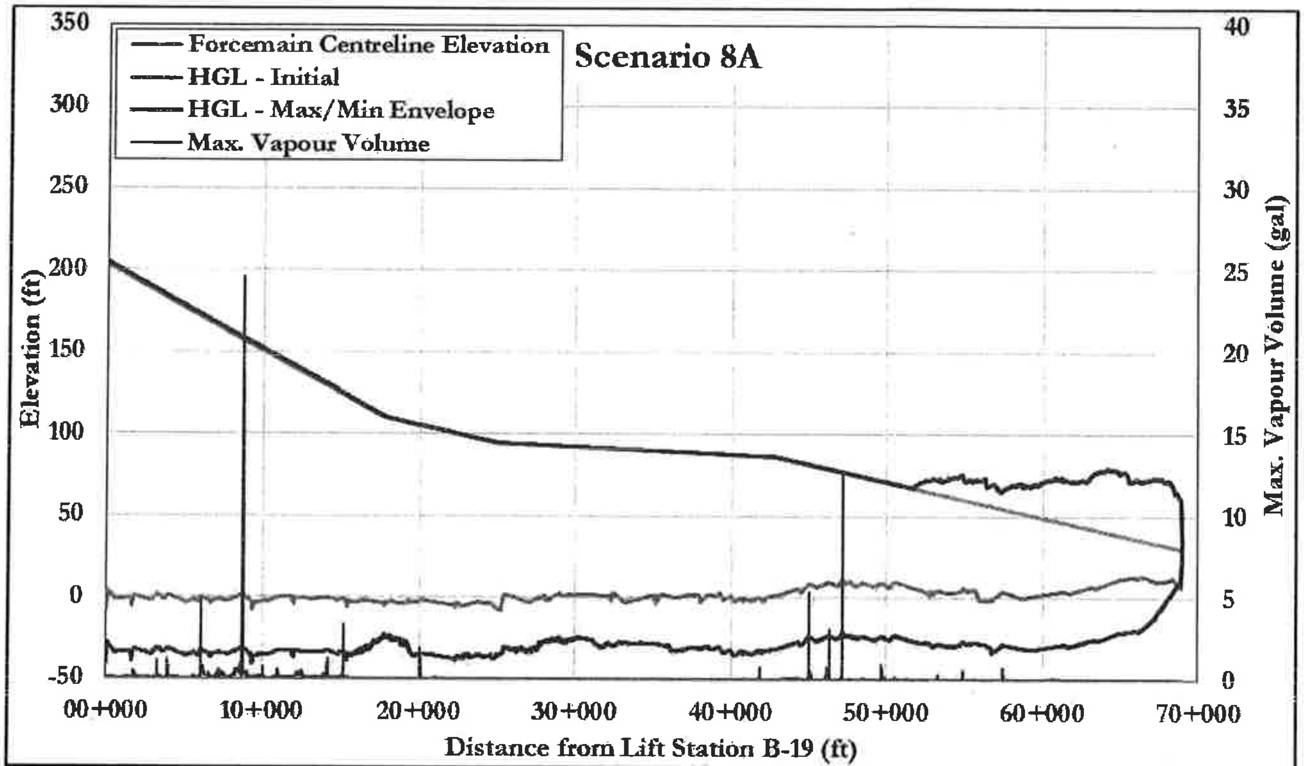


Figure 18: Scenario 8A – Proposed upgrades, full capacity at B-19 only, LWL, local power failure

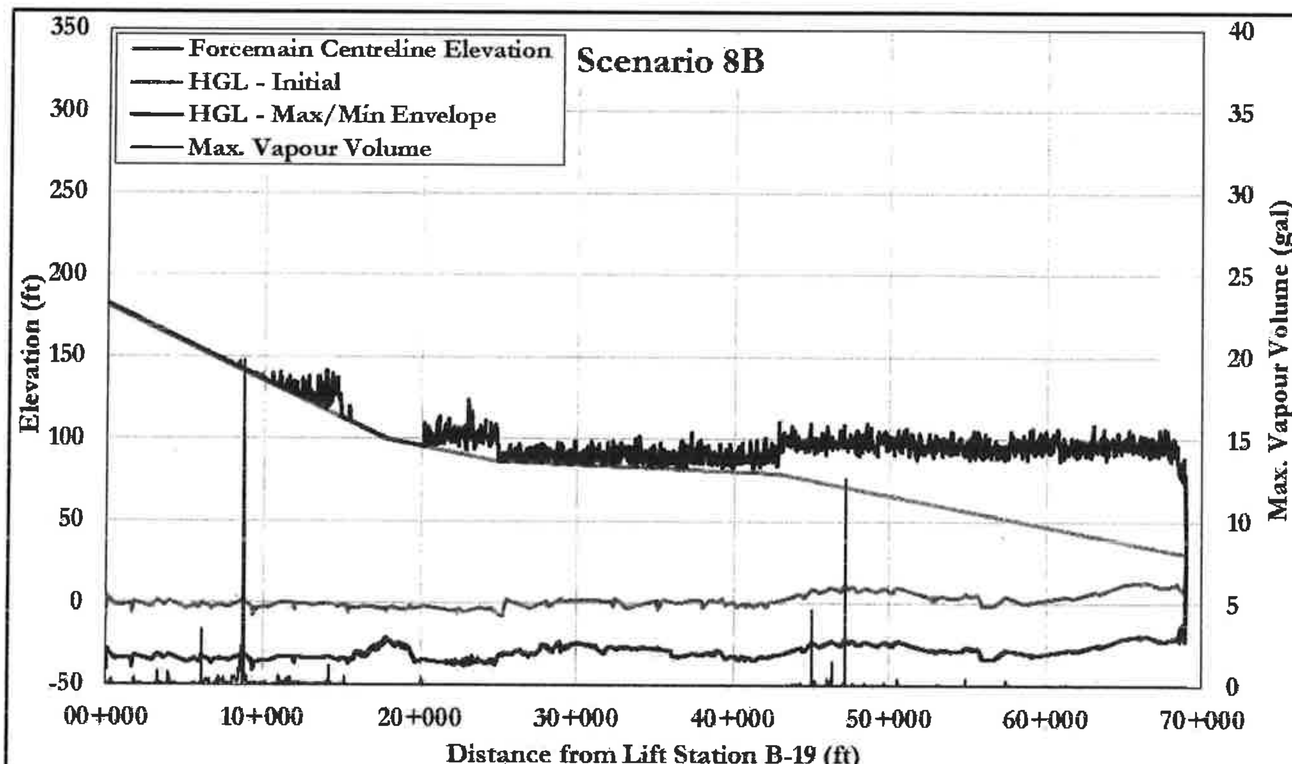


Figure 19: Scenario 8B – Proposed upgrades, firm capacity at B-19 only, LWL, local power failure

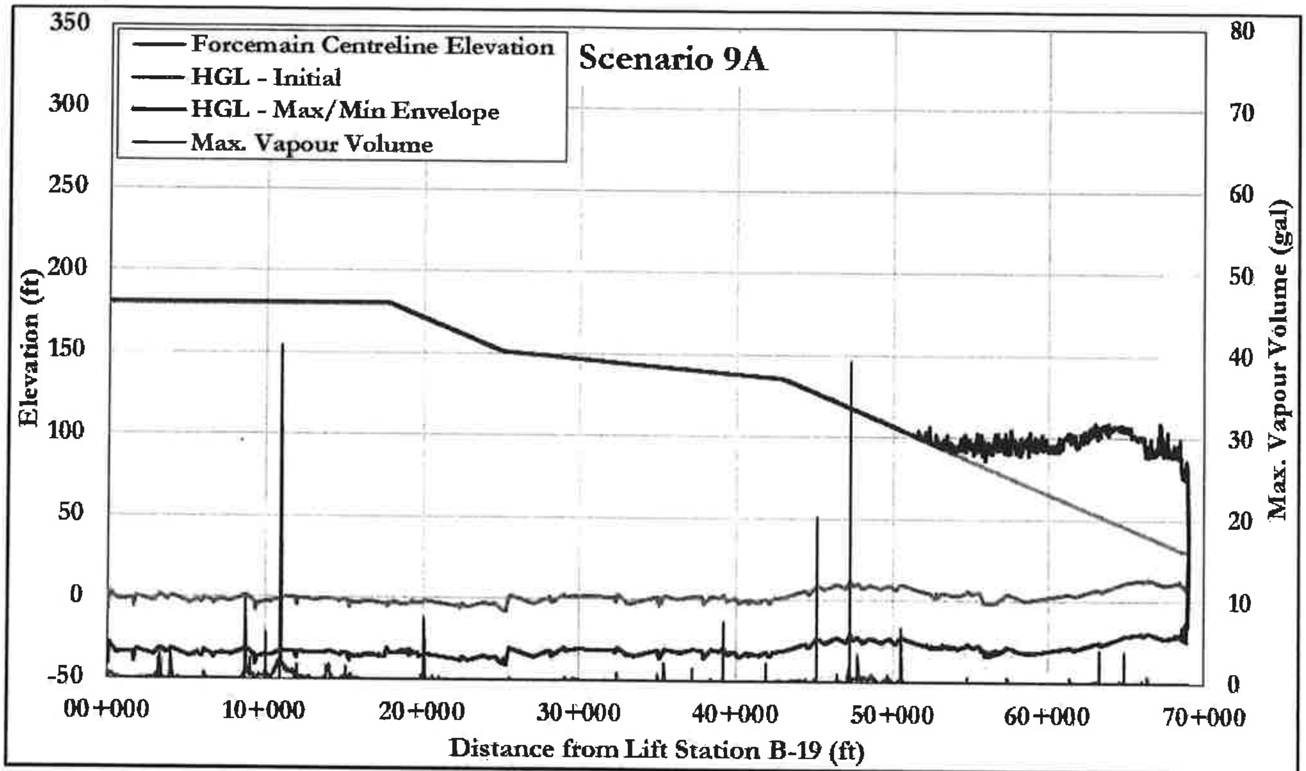


Figure 20: Scenario 9A – Proposed upgrades, full capacity at B-20 only, LWL, local power failure

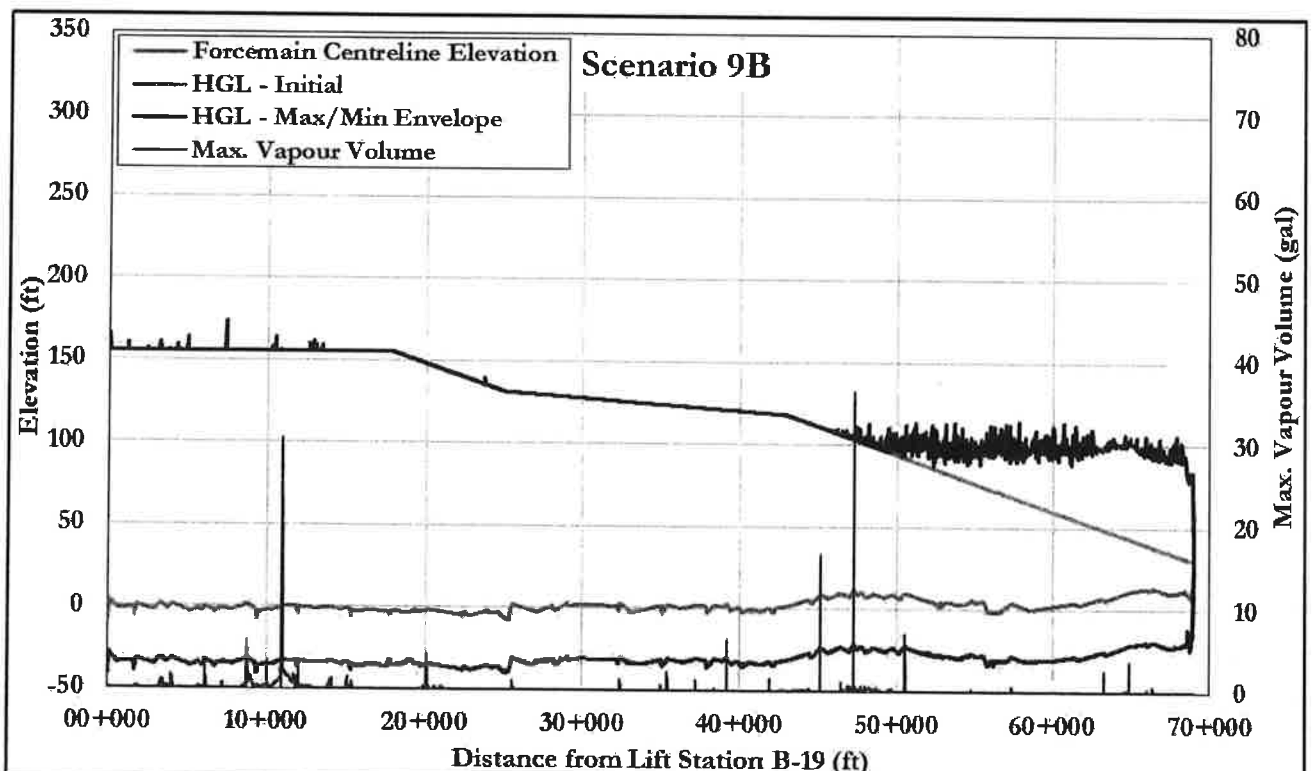


Figure 21: Scenario 9B – Proposed upgrades, firm capacity at B-20 only, LWL, local power failure

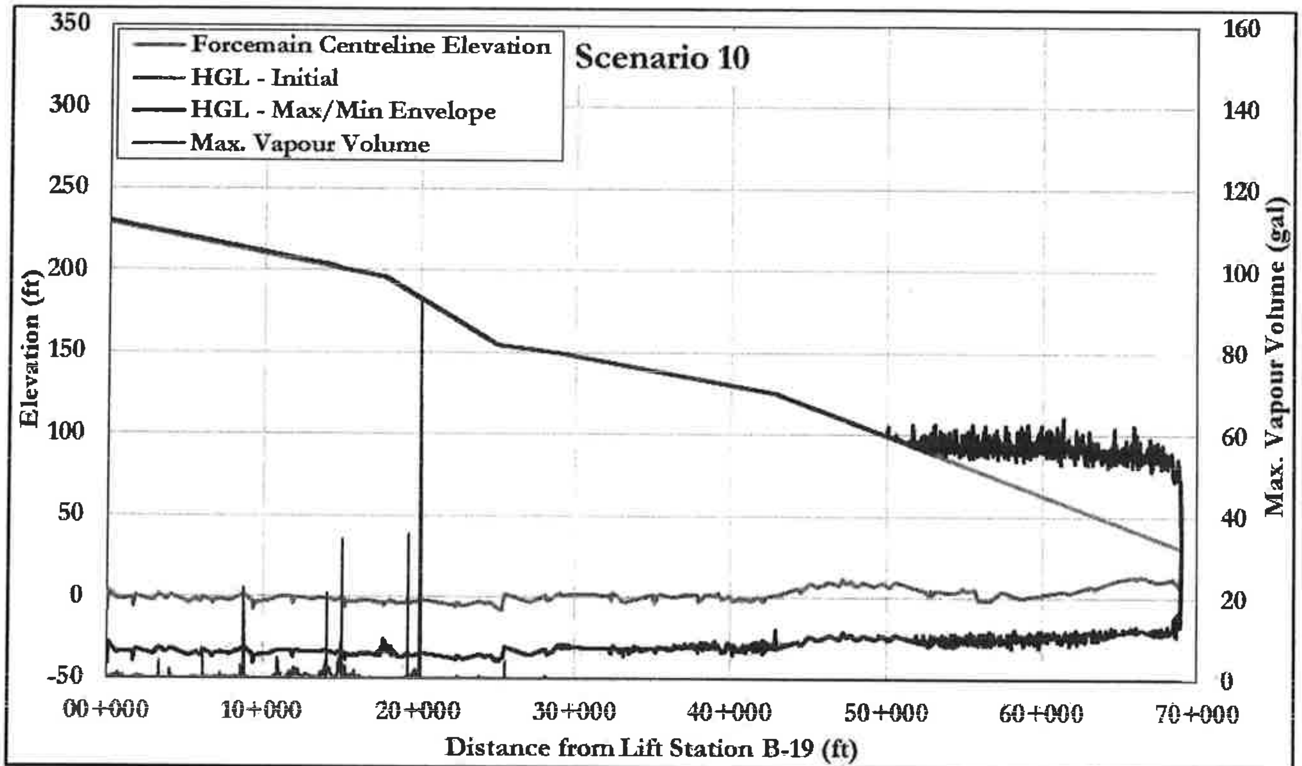


Figure 22: Scenario 10 – Proposed upgrades, full capacity, LWL, east-west forcemain interconnection at Riverside Drive, global power failure

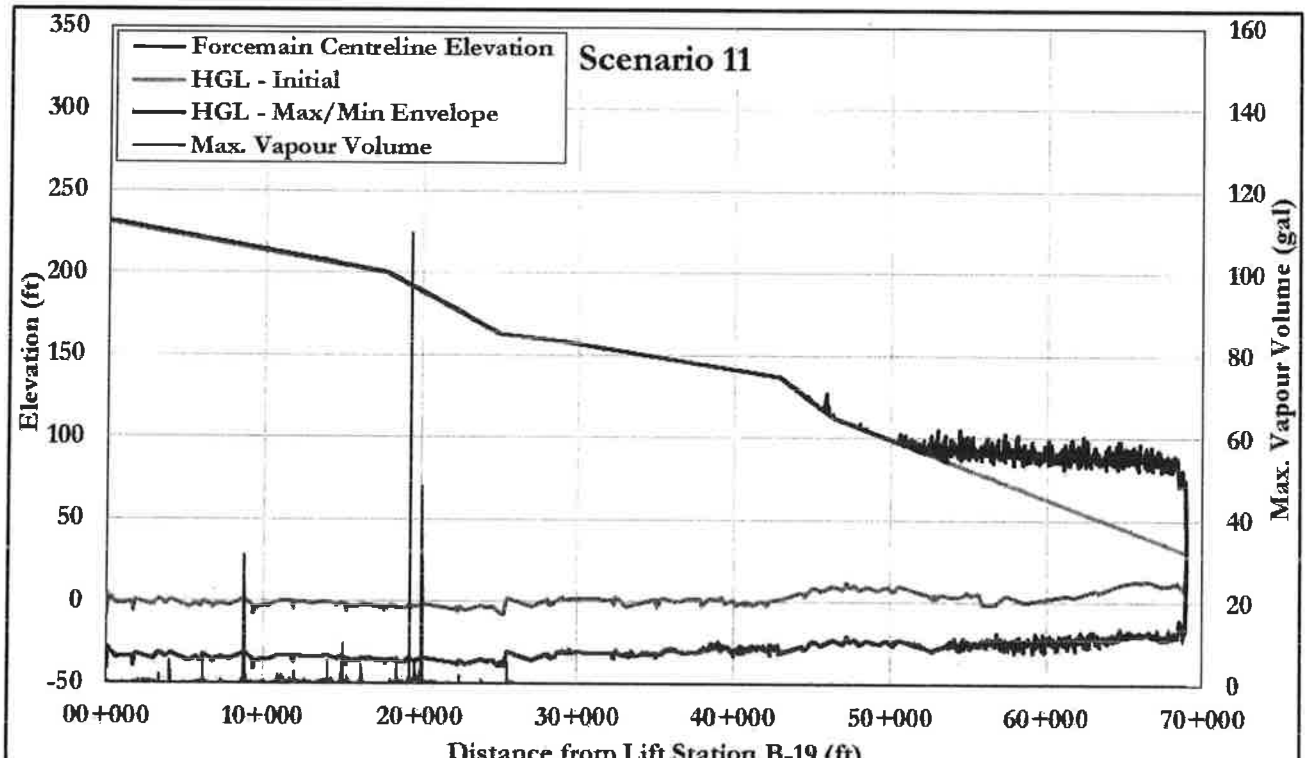


Figure 23: Scenario 11 – Proposed upgrades, full capacity, LWL, east-west forcemain interconnection at Oakland Avenue, global power failure

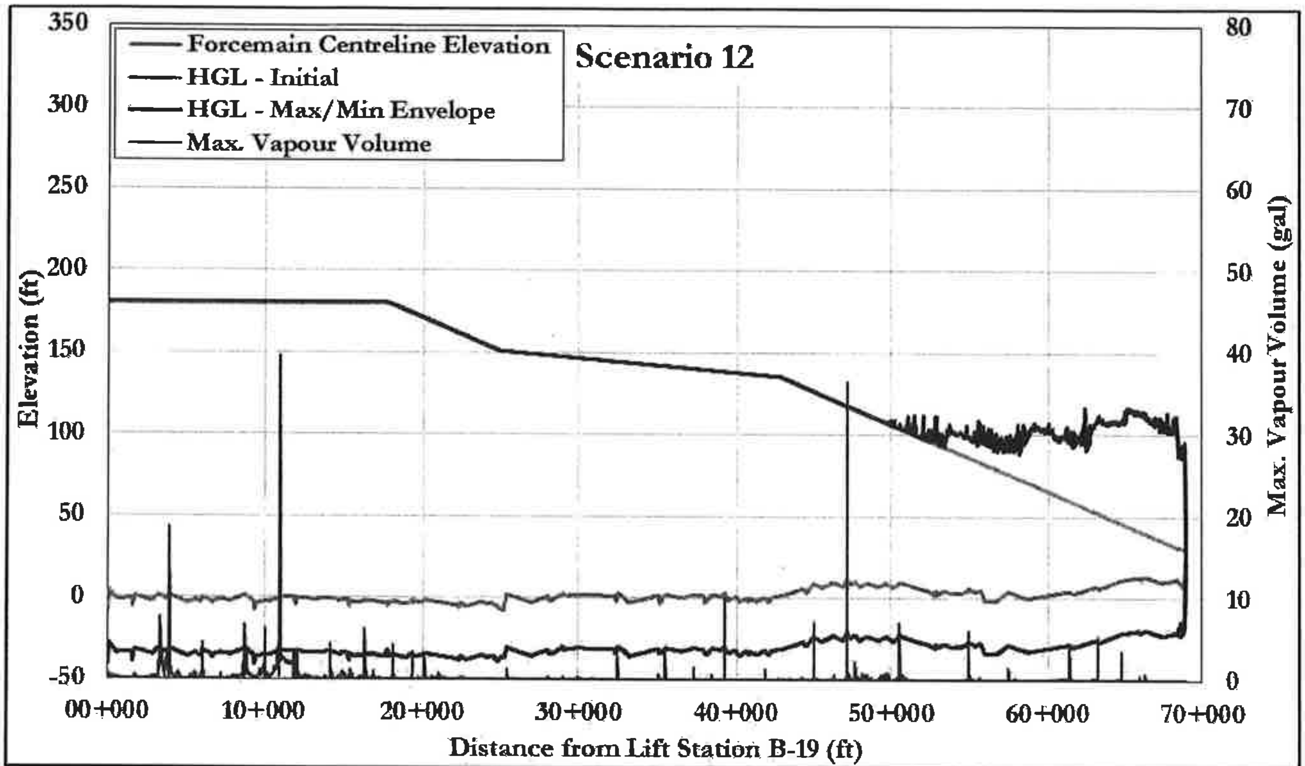


Figure 24: Scenario 12 – Scenario 9A (proposed upgrades, full capacity at B-20 only, LWL, global power failure) with +10% wave speed

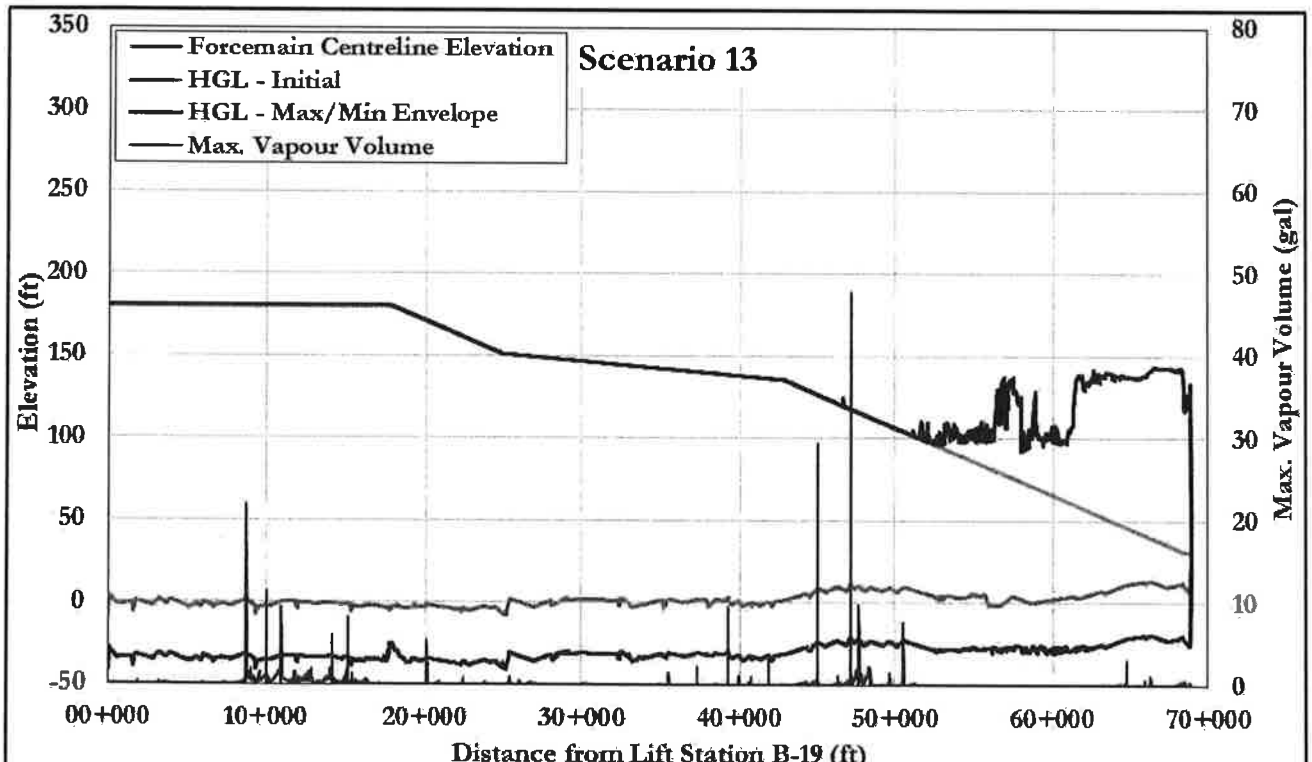


Figure 25: Scenario 13 – Scenario 9A (proposed upgrades, full capacity at B-20 only, LWL, global power failure) with -10% wave speed

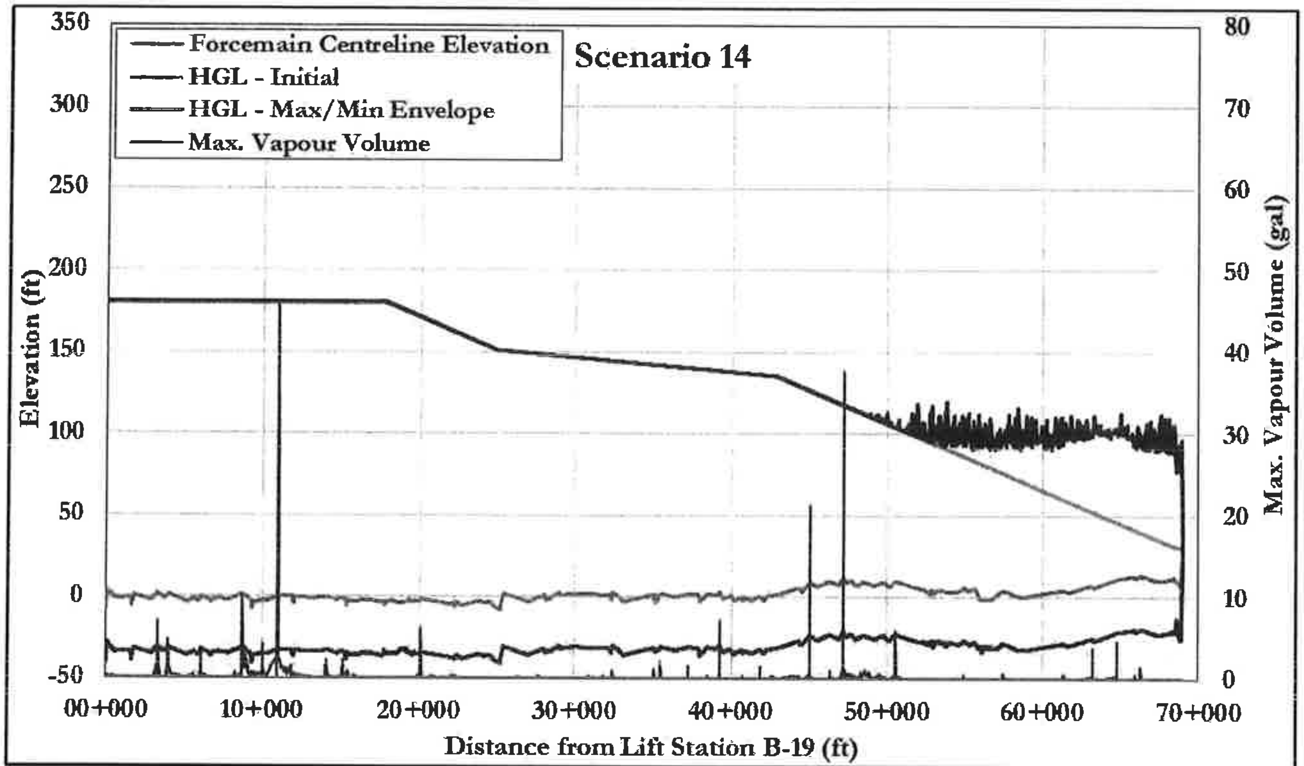


Figure 26: Scenario 14 – Scenario 9A (proposed upgrades, full capacity at B-20 only, LWL, global power failure) with 50% pump inertia

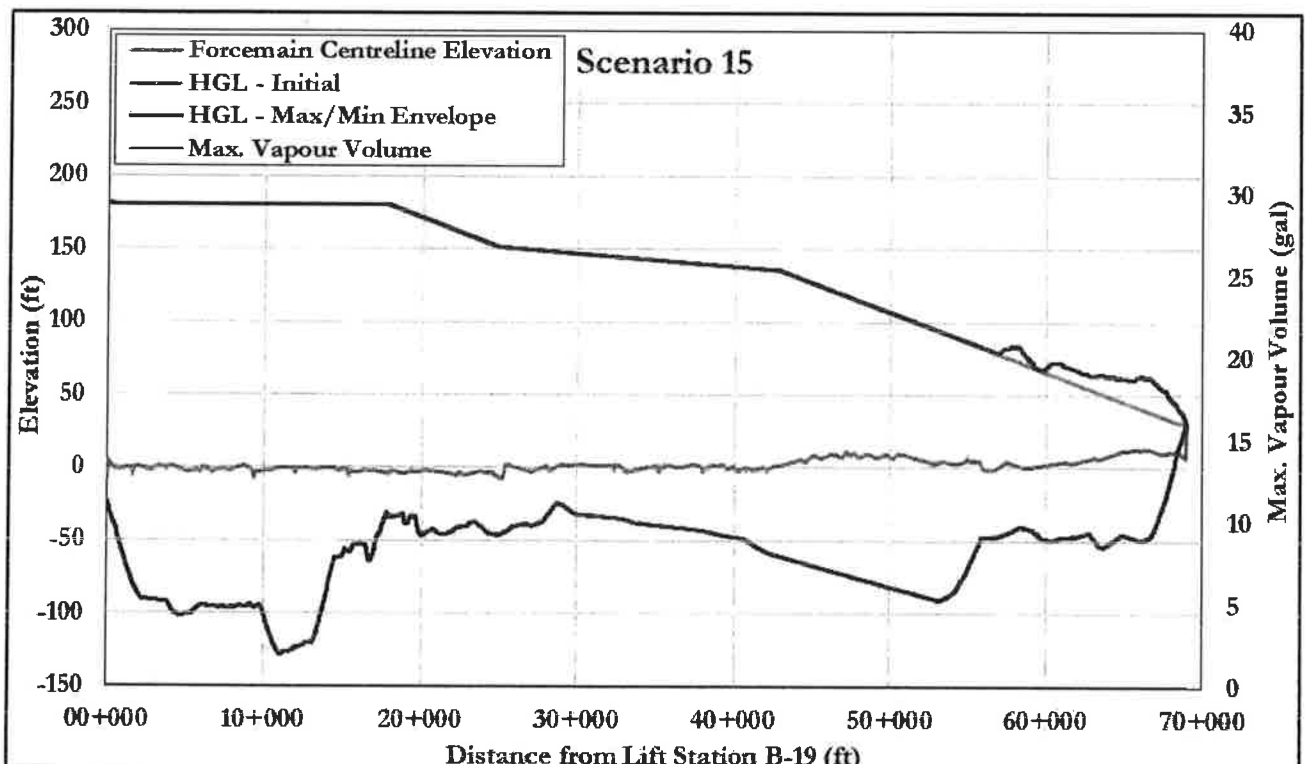


Figure 27: Scenario 15 – Scenario 9A (proposed upgrades, full capacity at B-20 only, LWL, global power failure) with no vapor cavities

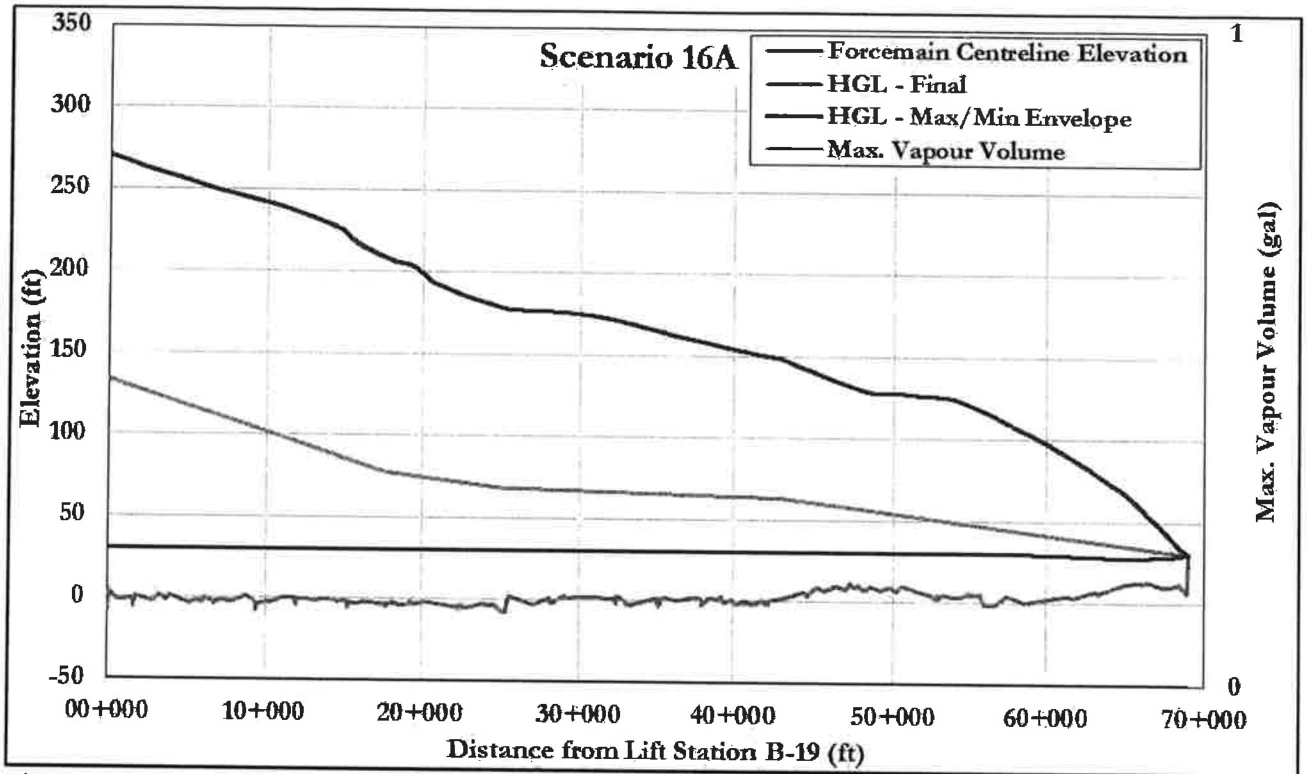


Figure 28: Scenario 16A – Proposed upgrades, all pumps off, HWL, single pump start at B-19

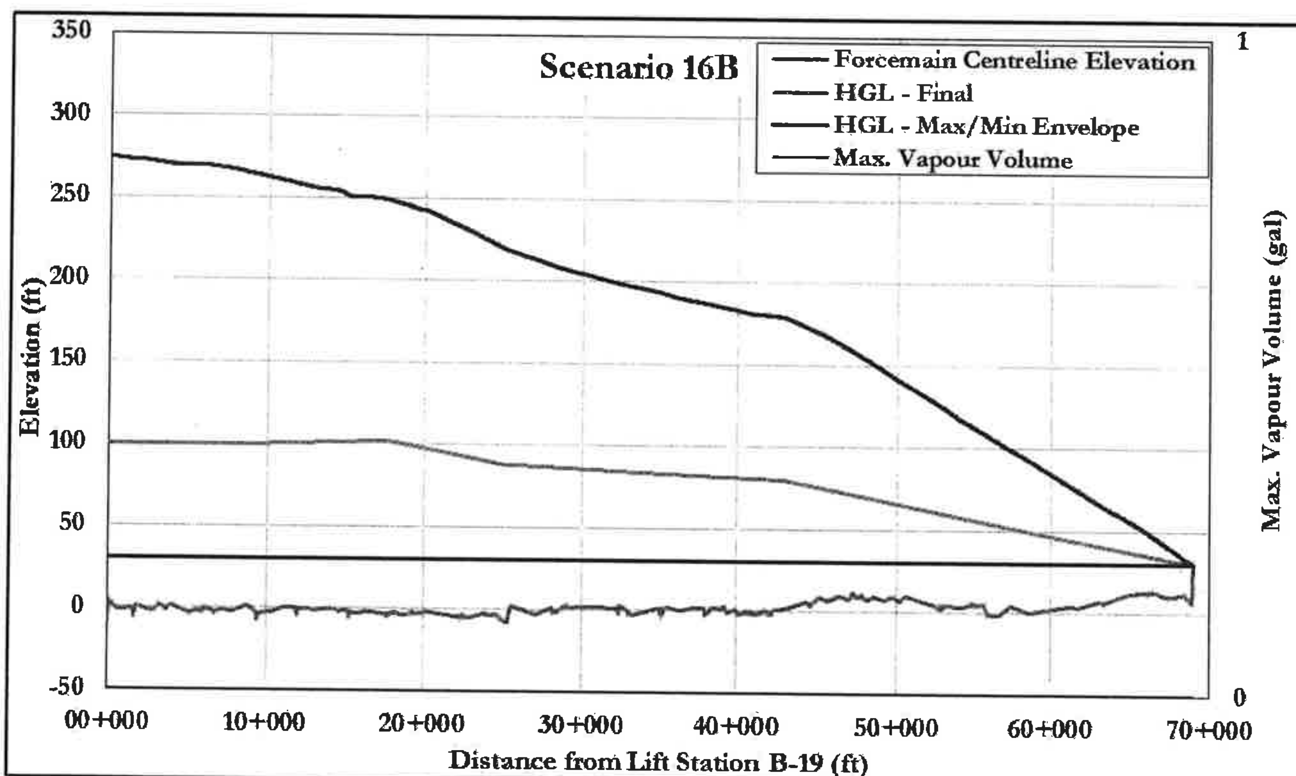


Figure 29: Scenario 16B – Proposed upgrades, all pumps off, HWL, single pump start at B-20

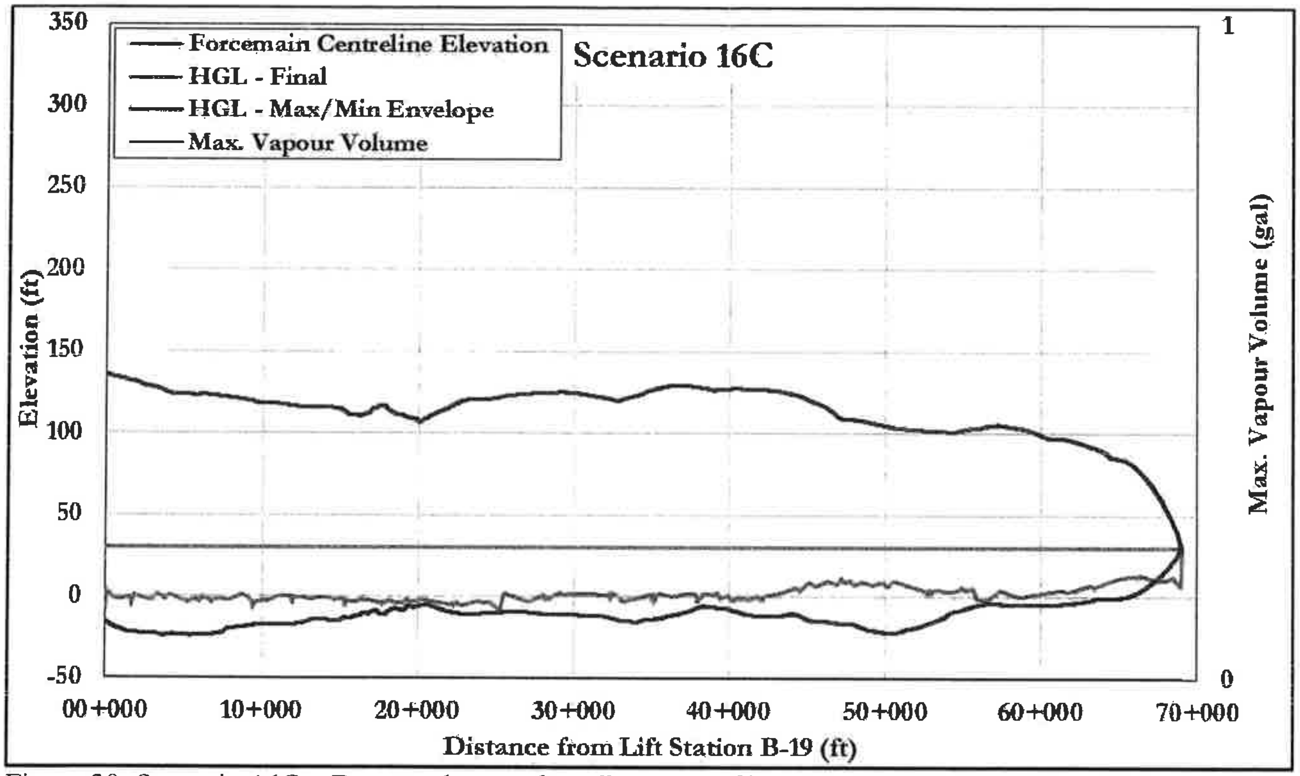


Figure 30: Scenario 16C – Proposed upgrades, all pumps off, HWL, single pump start at B-1

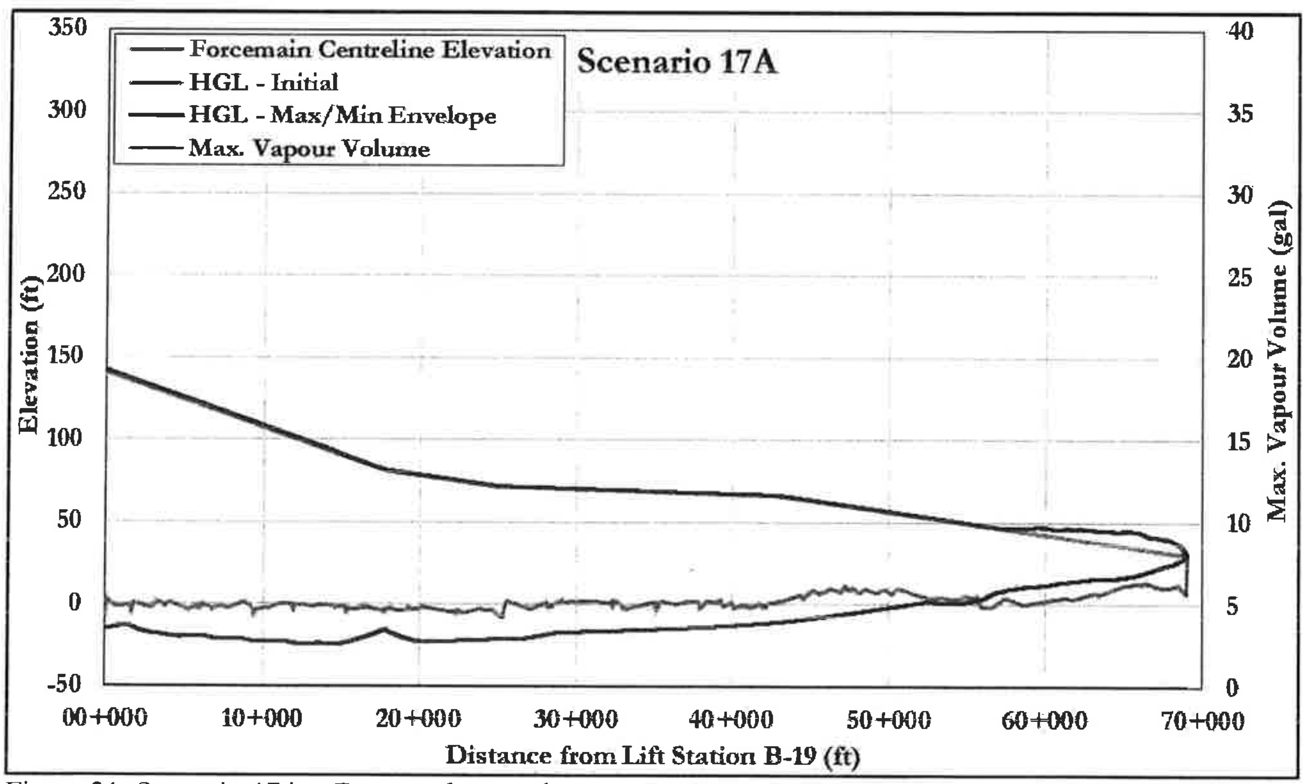


Figure 31: Scenario 17A – Proposed upgrades, one B-19 pump on, HWL, single pump stop at B-19

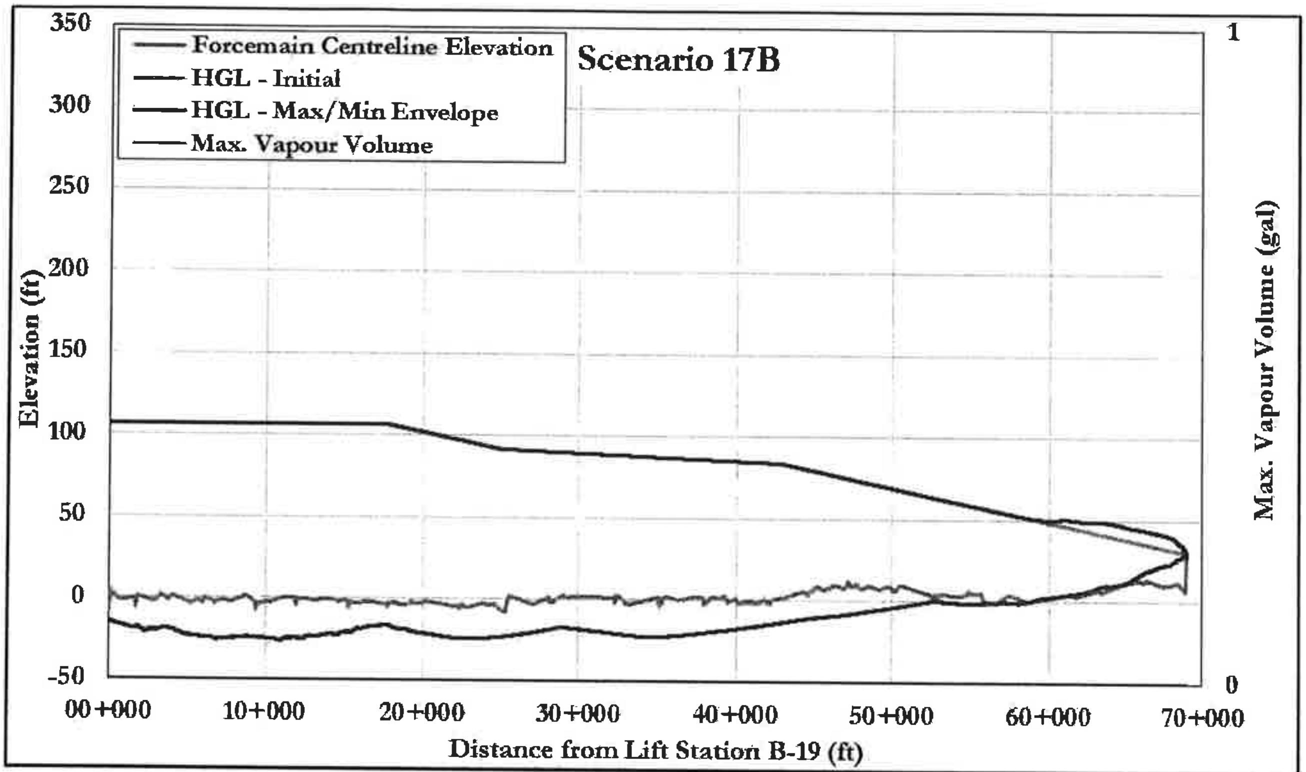


Figure 32: Scenario 17B – Proposed upgrades, one B-20 pump on, HWL, single pump stop at B-20

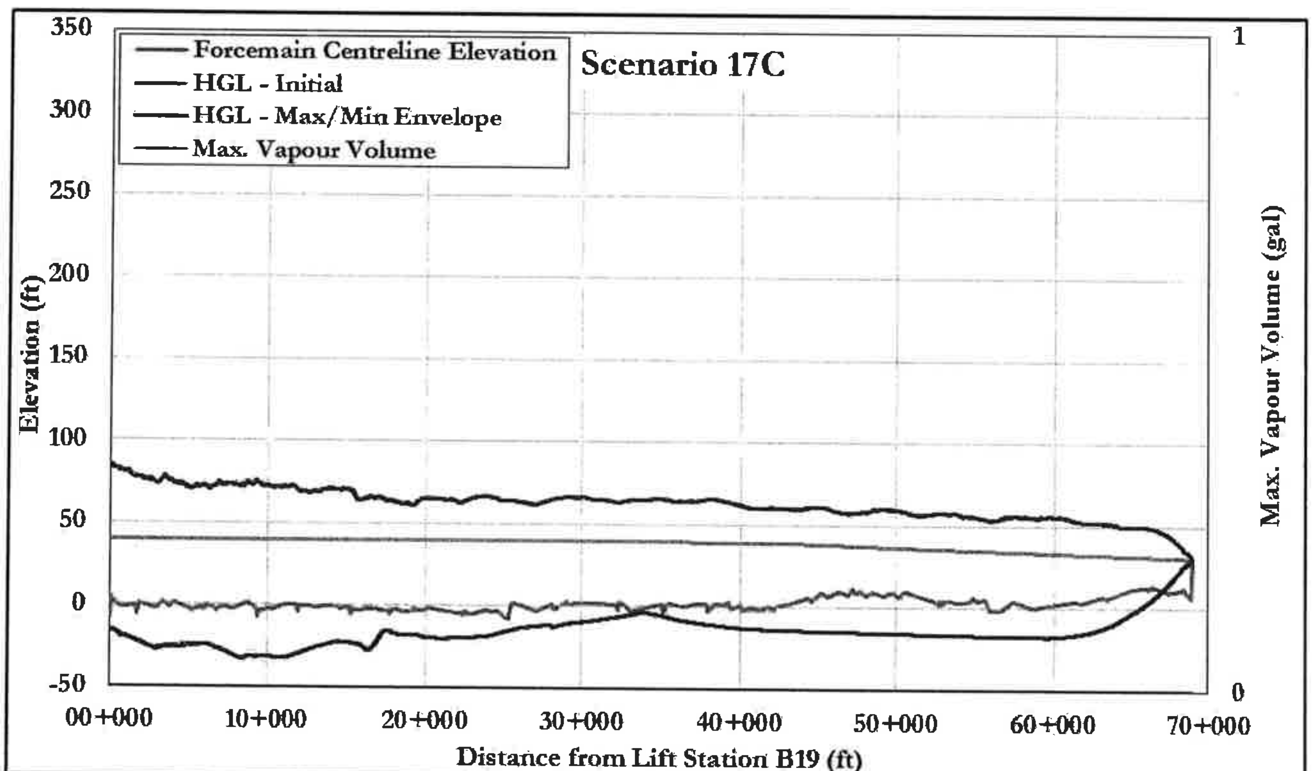


Figure 33: Scenario 17C – Proposed upgrades, one B-1 pump on, HWL, single pump stop at B-1

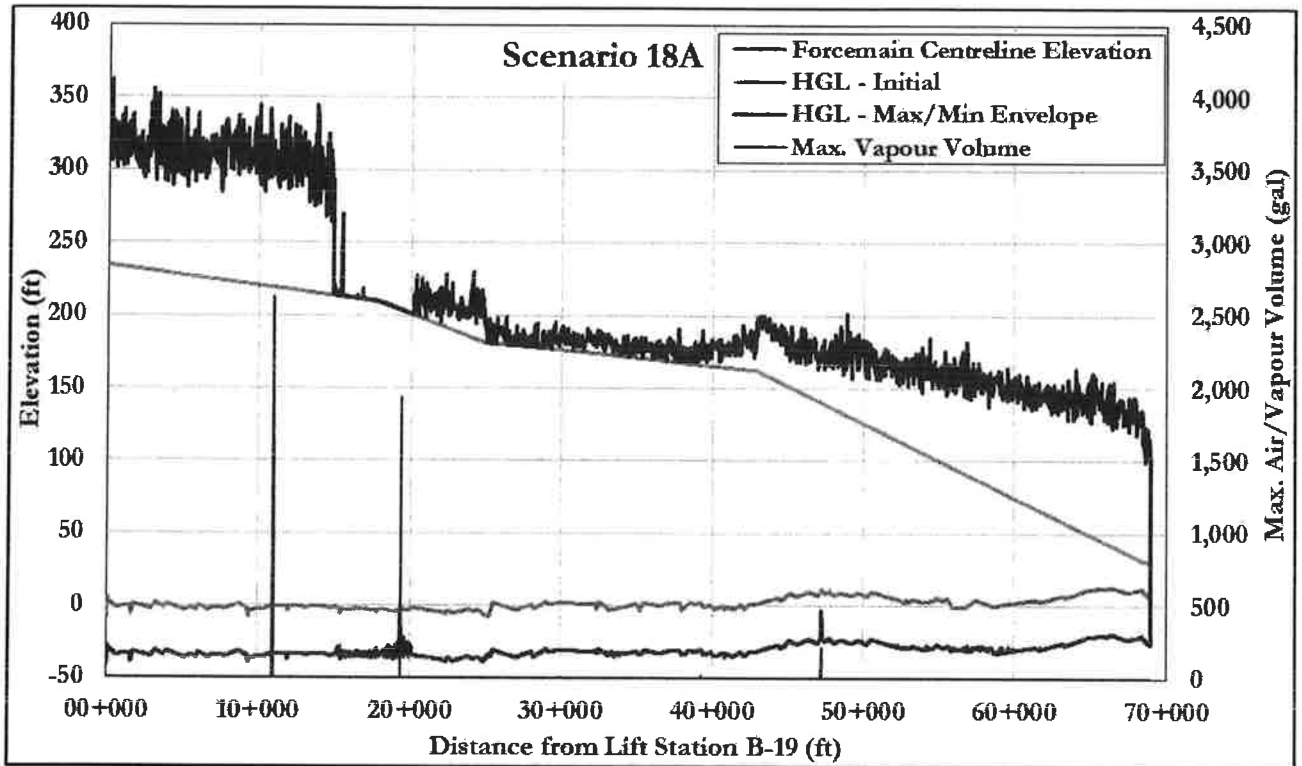


Figure 34: Scenario 18A – Scenario 7A (proposed upgrades, full capacity, LWL, global power failure) with 2-inch standard CAVs at select locations

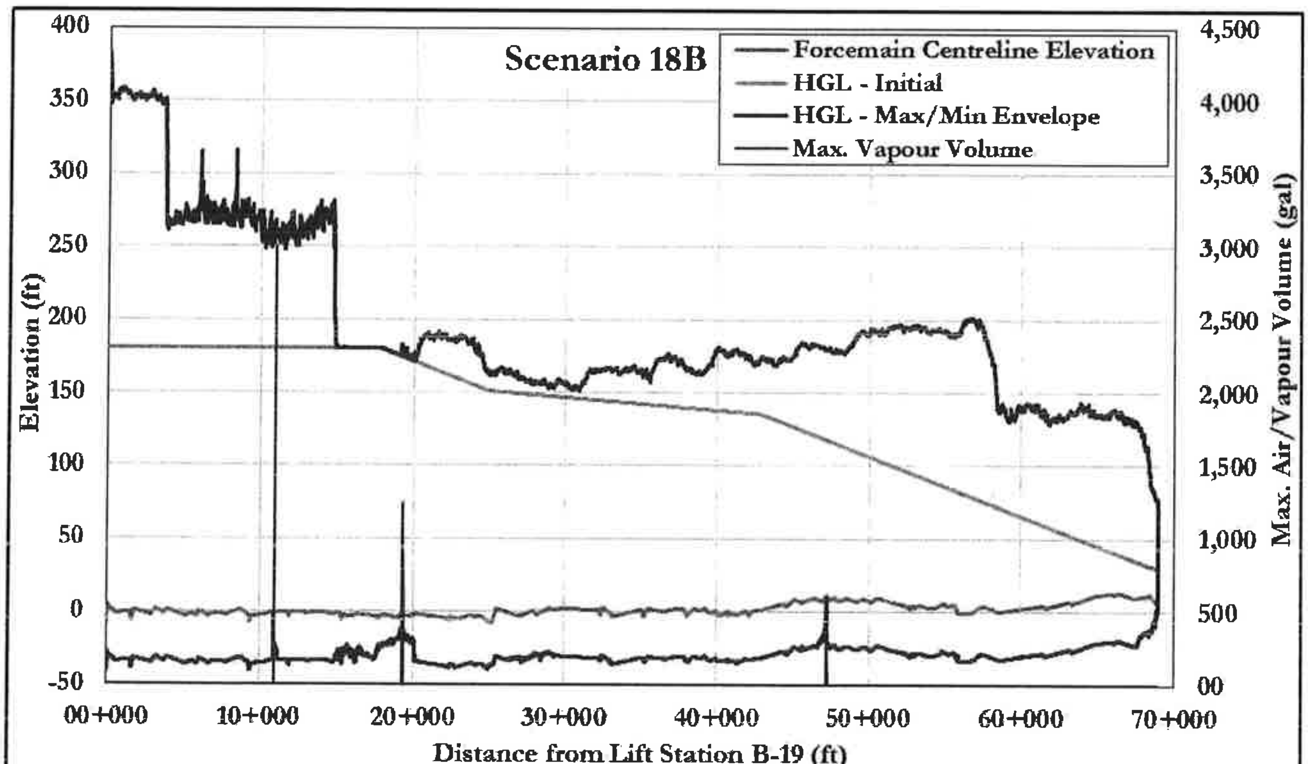


Figure 35: Scenario 18B – Scenario 9A (Proposed upgrades, full capacity at B-20 only, LWL, global power failure) with 2-inch standard CAVs at select locations

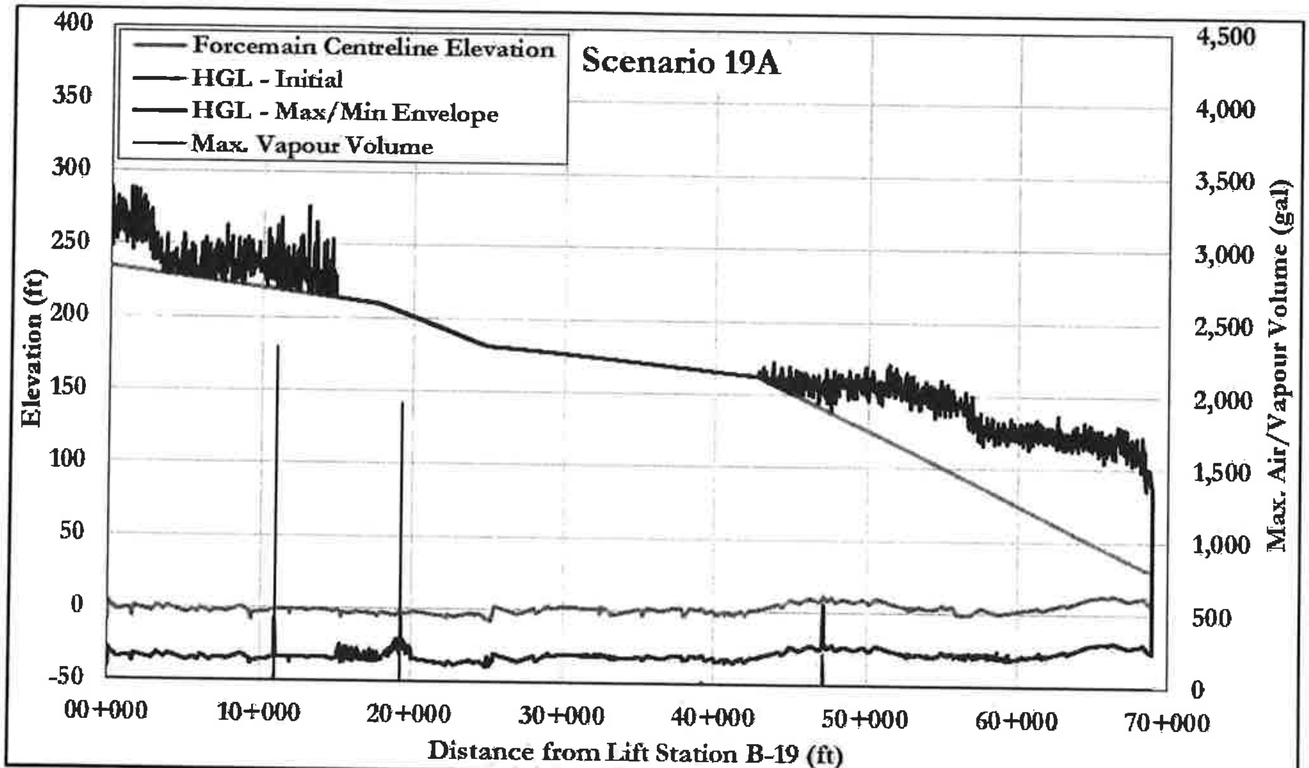


Figure 36: Scenario 19A – Scenario 7A (proposed upgrades, full capacity, LWL, global power failure) with 2-inch three-stage CAVs at select locations

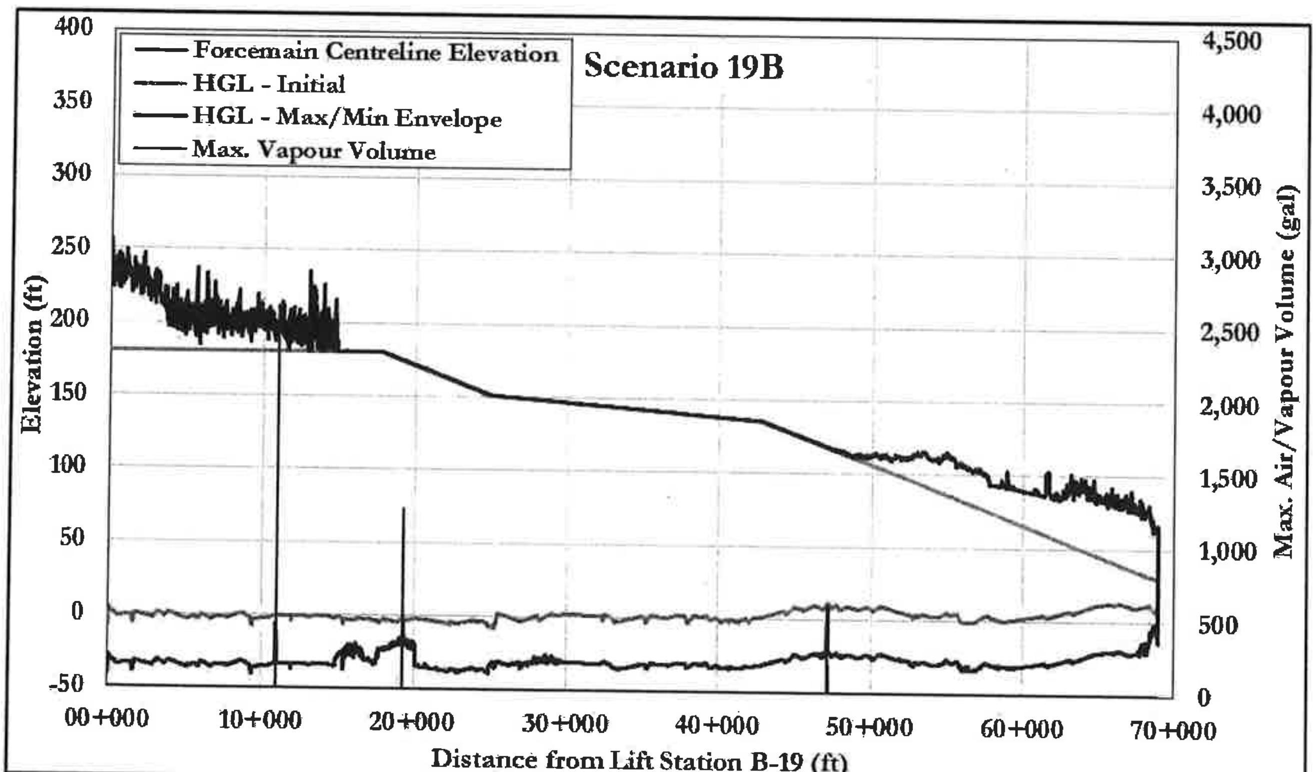


Figure 37: Scenario 19B – Scenario 9A (Proposed upgrades, full capacity at B-20 only, LWL, global power failure) with 2-inch three-stage CAVs at select locations

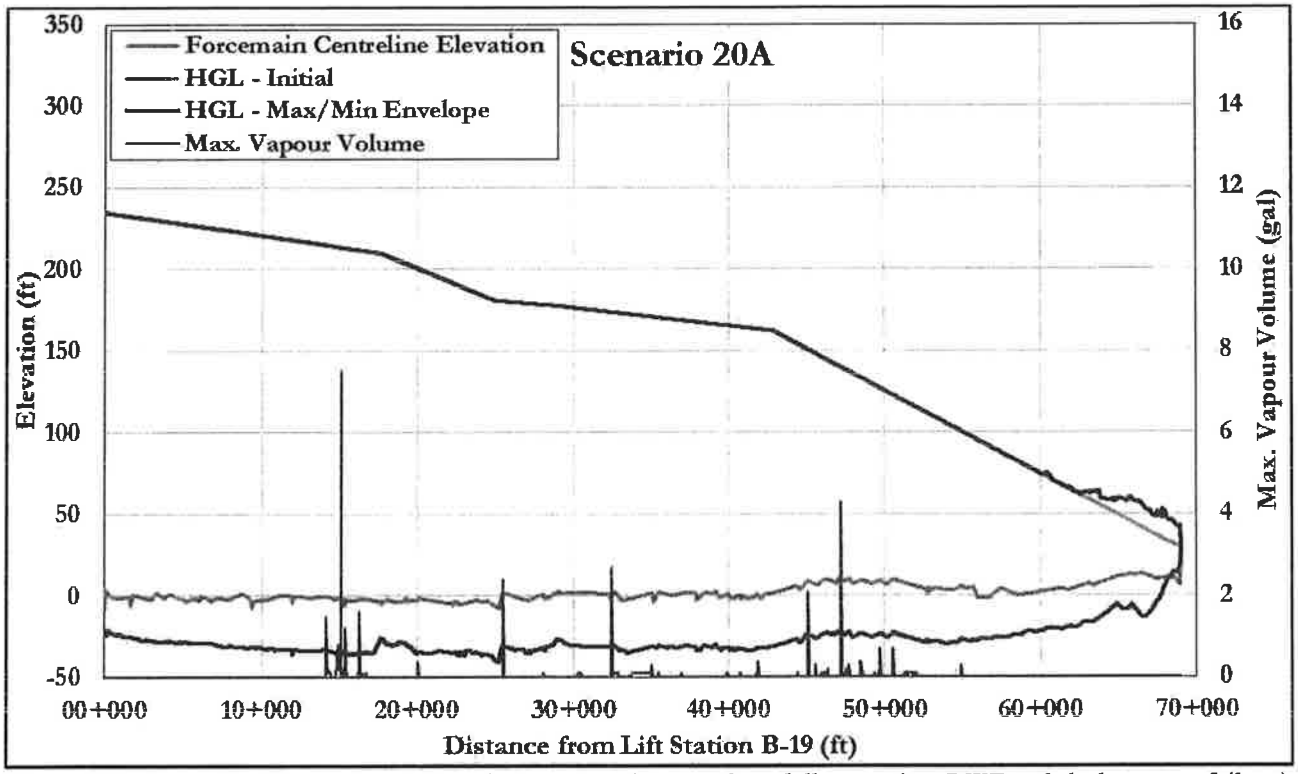


Figure 38: Scenario 20A – Scenario 7A (proposed upgrades, full capacity, LWL, global power failure) with a 4,000 gal HAC at Lift Station B-19

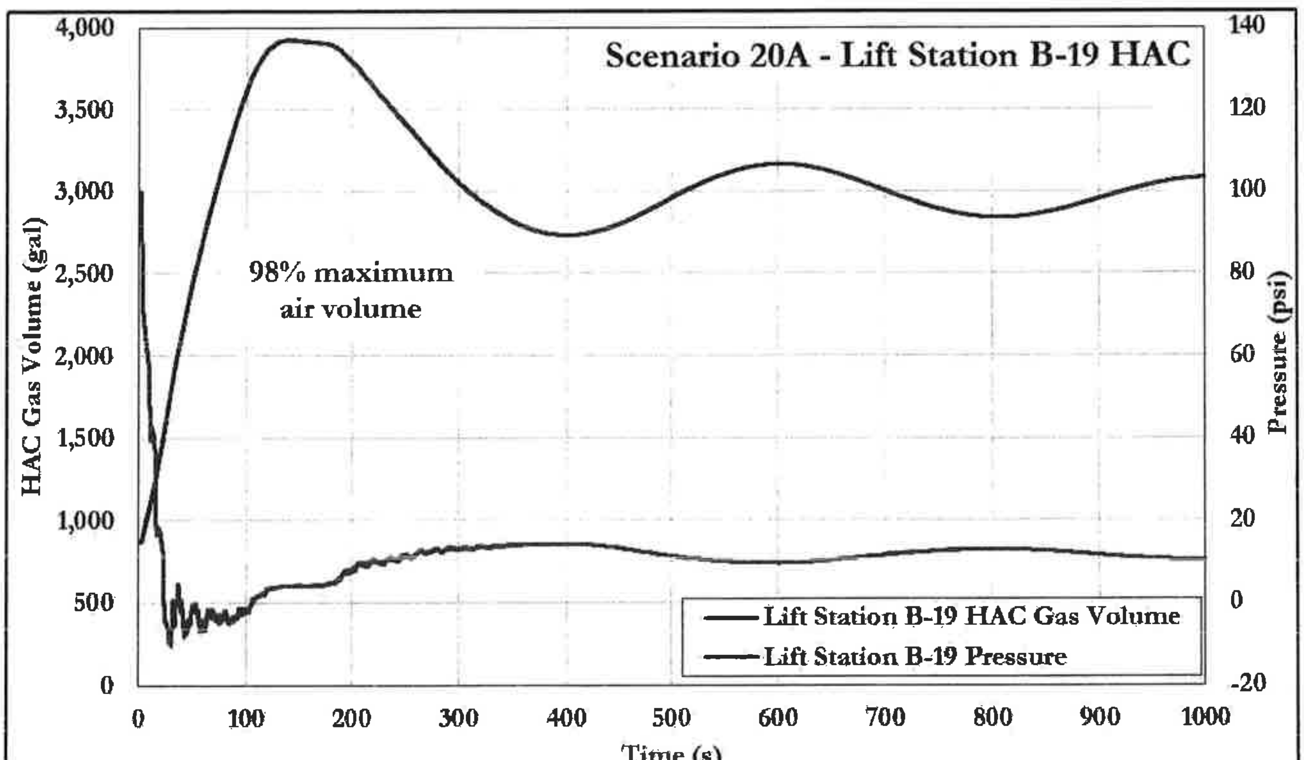


Figure 39: Scenario 20A – HAC gas volume and pressure for 4,000 gal HAC at Lift Station B-19

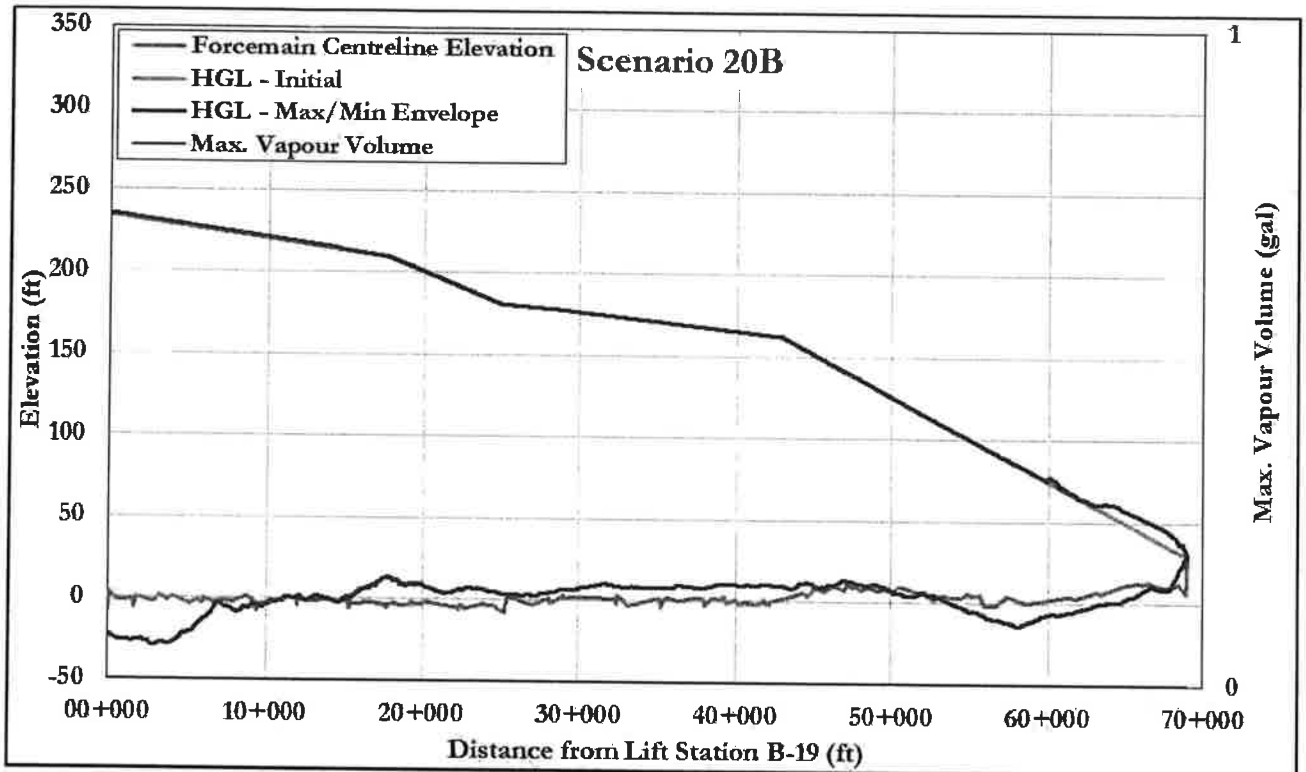


Figure 40: Scenario 20B – Scenario 7A (proposed upgrades, full capacity, LWL, global power failure) with a 10,000 gal HAC at Lift Station B-20

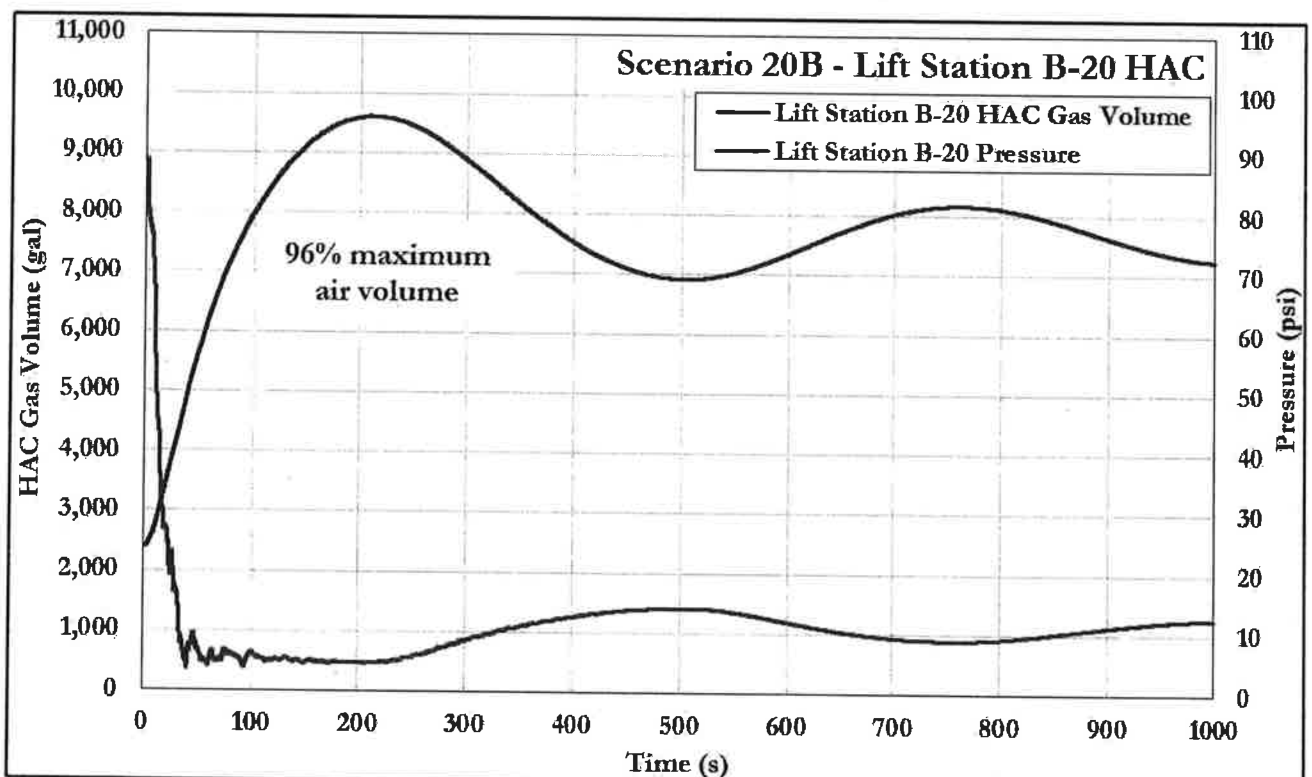


Figure 41: Scenario 20B – HAC gas volume and pressure for 10,000 gal HAC at Lift Station B-20

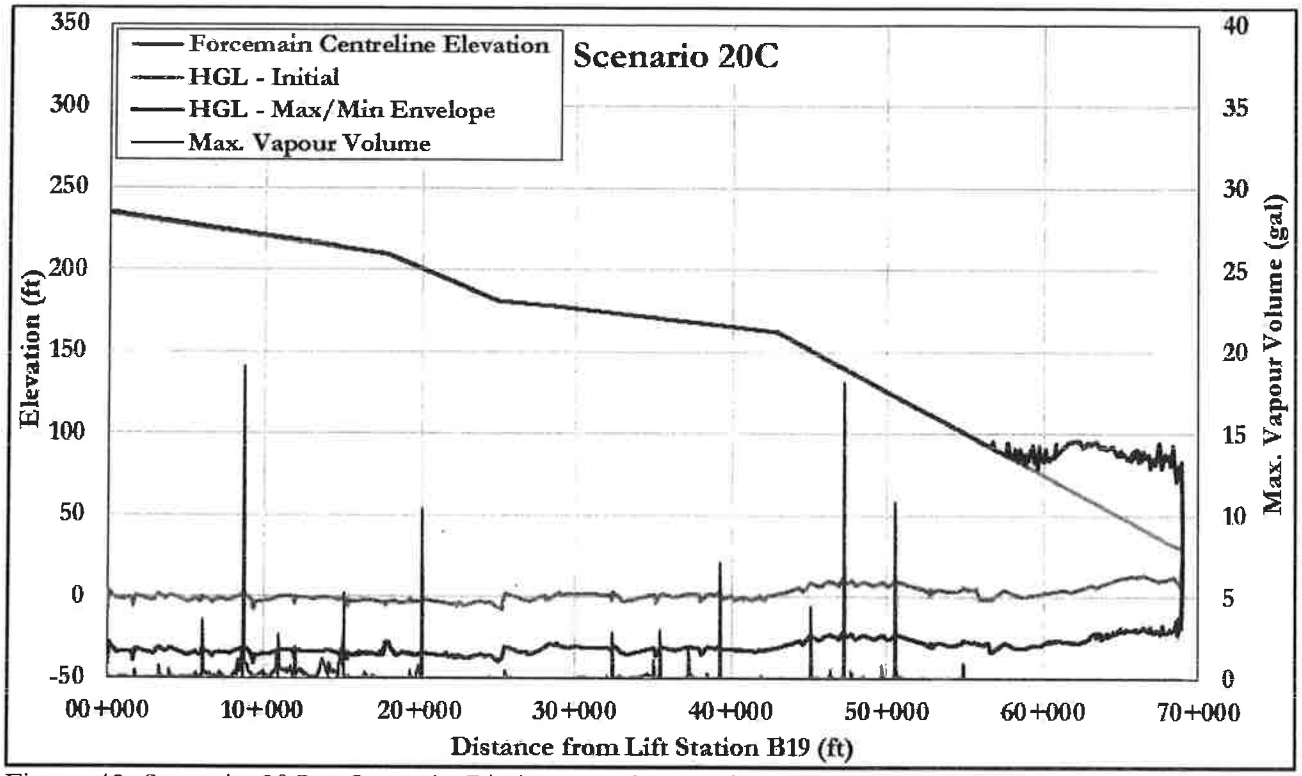


Figure 42: Scenario 20C – Scenario 7A (proposed upgrades, full capacity, LWL, global power failure) with a 4,000 gal HAC at Lift Station B-1

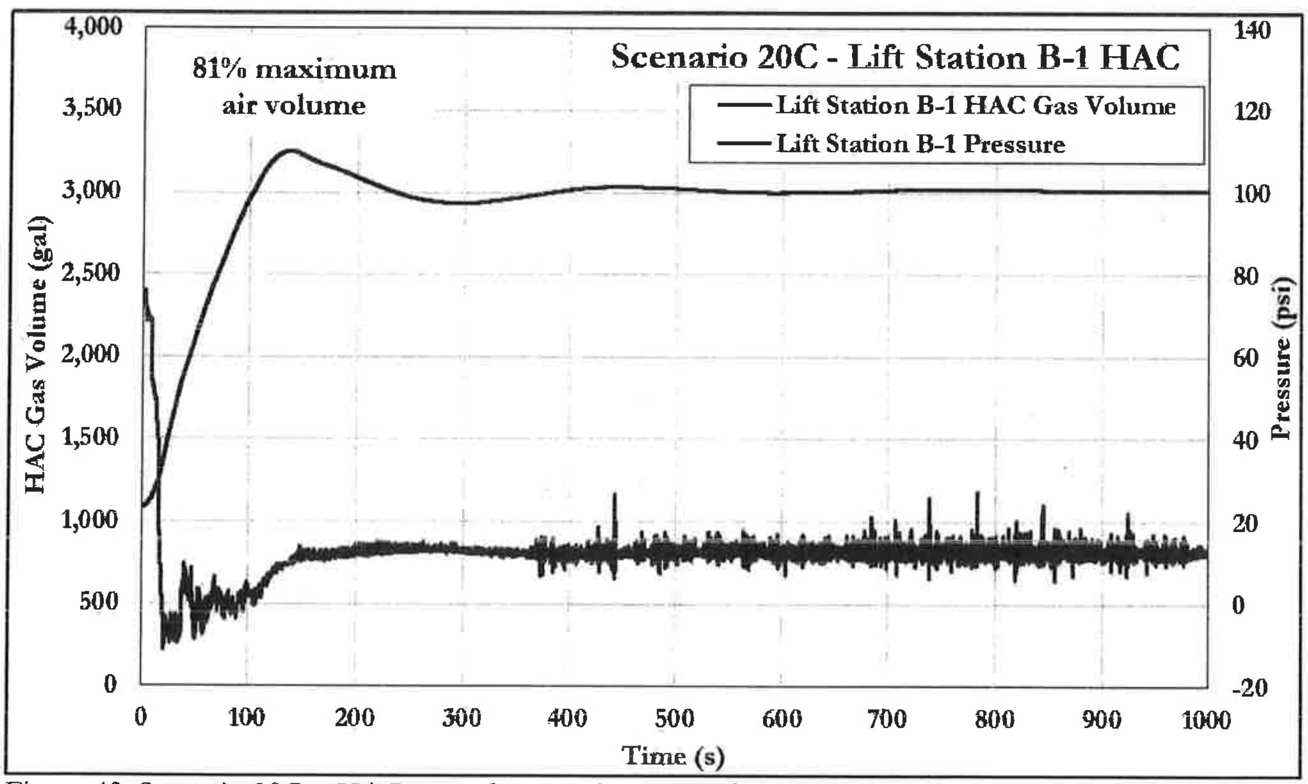


Figure 43: Scenario 20C – HAC gas volume and pressure for 4,000 gal HAC at Lift Station B-1

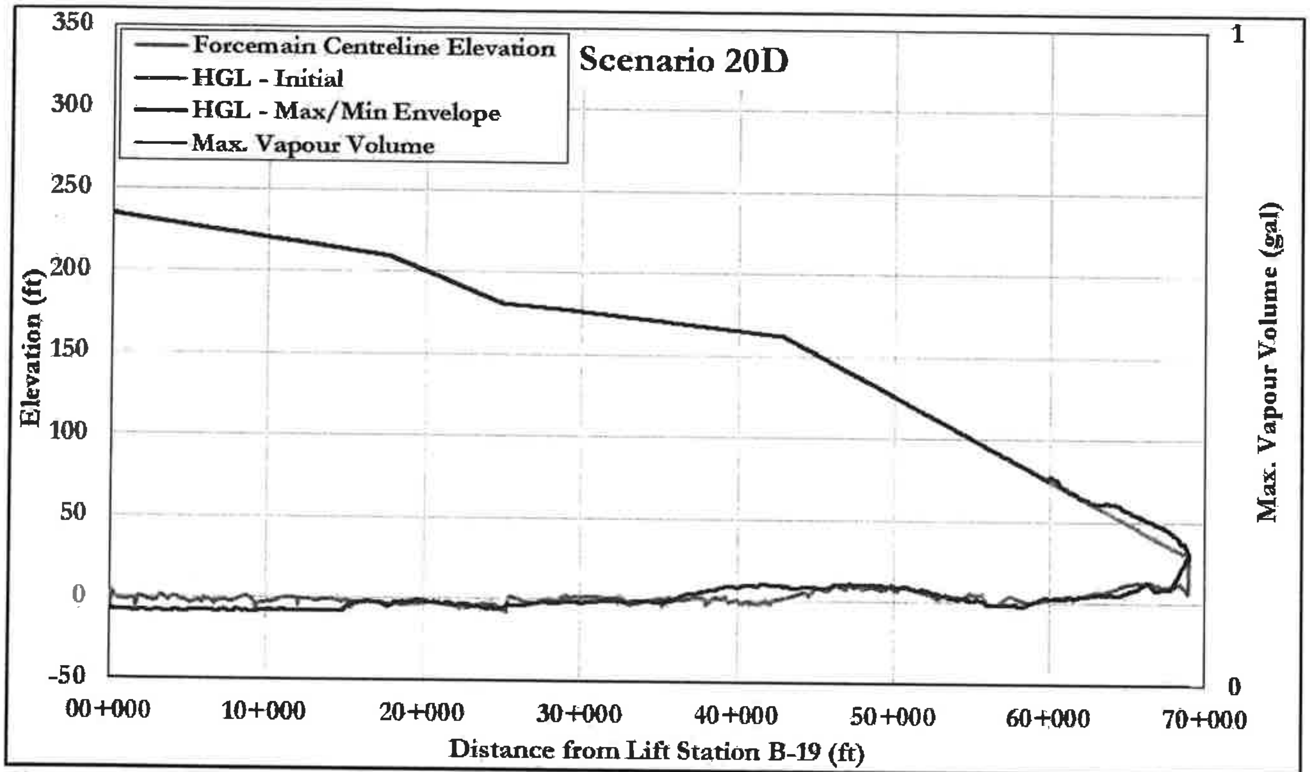


Figure 44: Scenario 20D – Scenario 7A (proposed upgrades, full capacity, LWL, global power failure) with a 3,000 gal HAC at Lift Station B-19 and a 6,000 gal HAC at Lift Station B-20

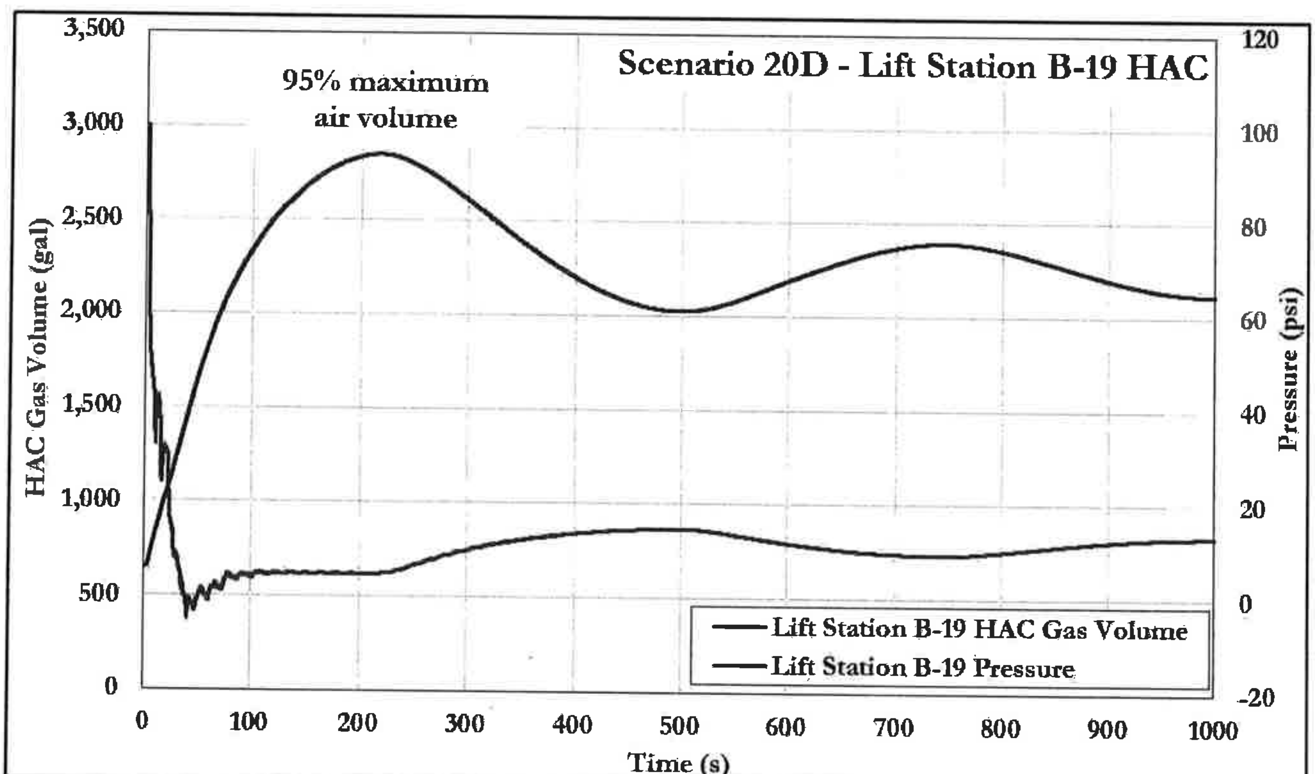


Figure 45: Scenario 20D – HAC gas volume and pressure for 3,000 gal HAC at Lift Station B-19 (with a 6,000 gal HAC at Lift Station B-20)

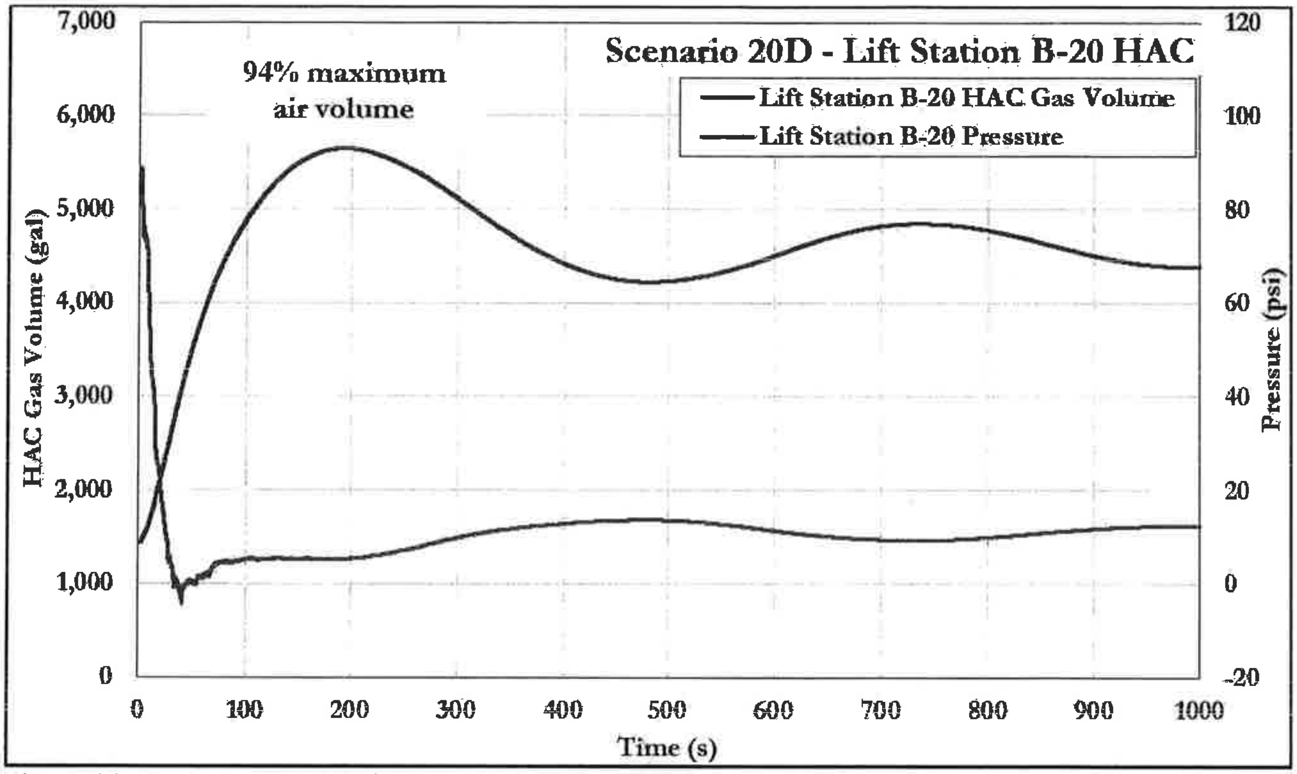


Figure 46: Scenario 20D – HAC gas volume and pressure for 6,000 gal HAC at Lift Station B-20 (with a 3,000 gal HAC at Lift Station B-19)

APPENDIX C: SAMPLE PLOTS OF TRANSIENT PRESSURE MONITORING DATA AT LIFT STATIONS B-19 AND B-1

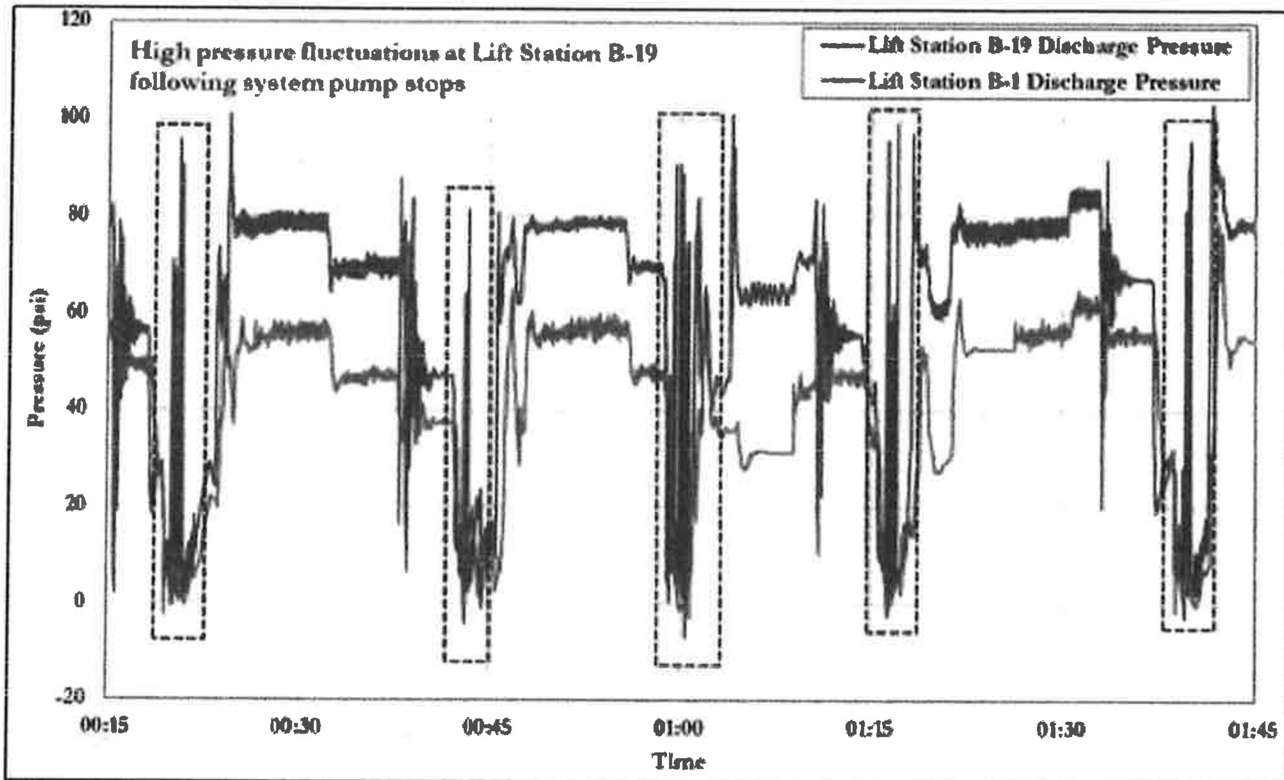


Figure 47: Transient pressure monitoring data from 12:15 AM to 1:45 AM on 30 September 2017 showing high pressure fluctuations at Lift Station B-19 following pump stop operations

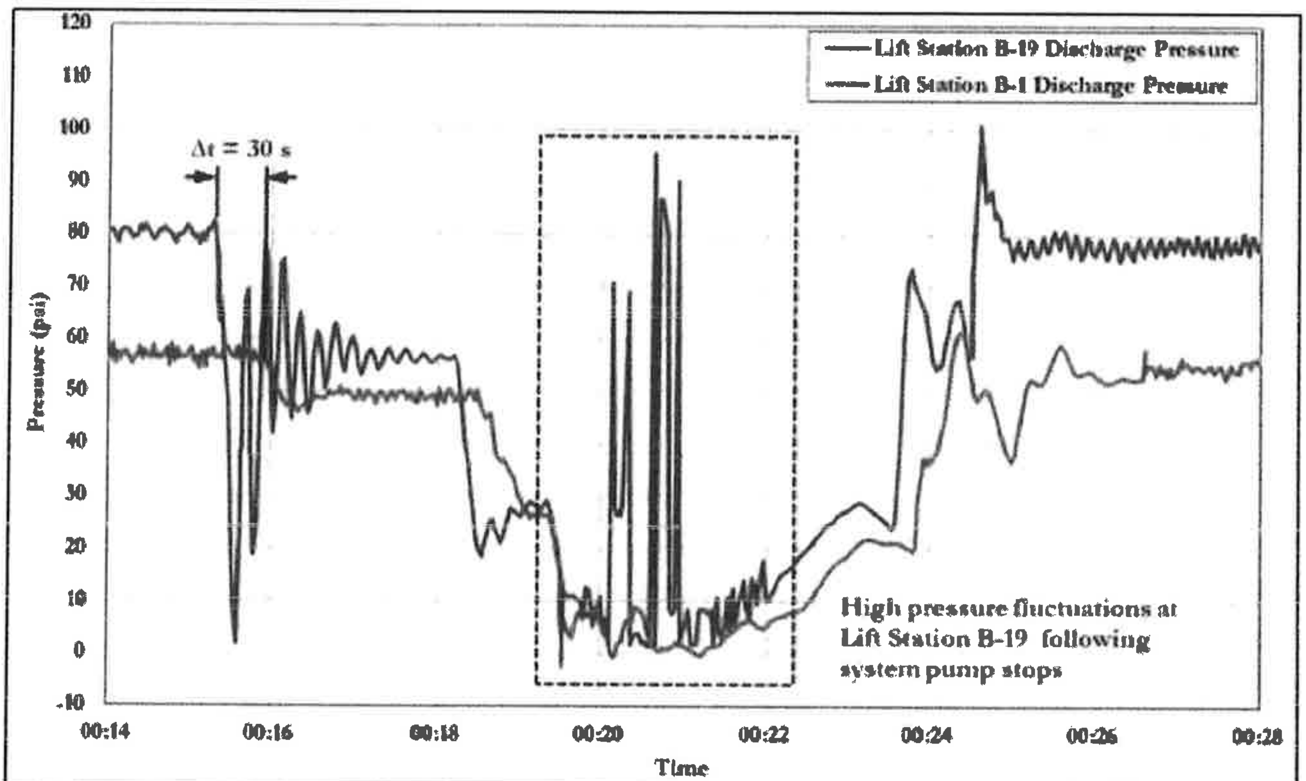


Figure 48: Transient pressure monitoring data from 12:14 AM to 12:28 AM on 30 September 2017 showing high pressure fluctuations at Lift Station B-19 following pump stop operations

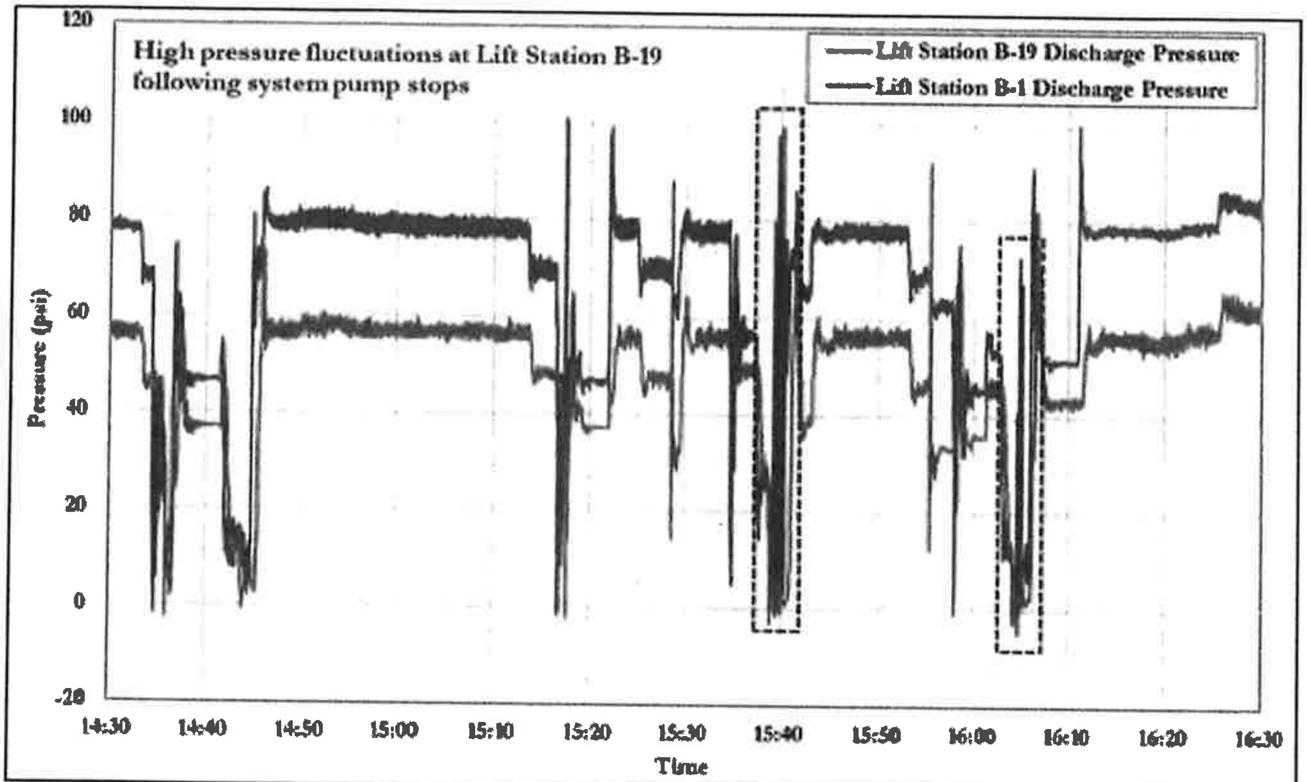


Figure 49: Transient pressure monitoring data from 2:30 PM to 4:30 PM on 30 September 2017 showing high pressure fluctuations at Lift Station B-19 following pump stop operations

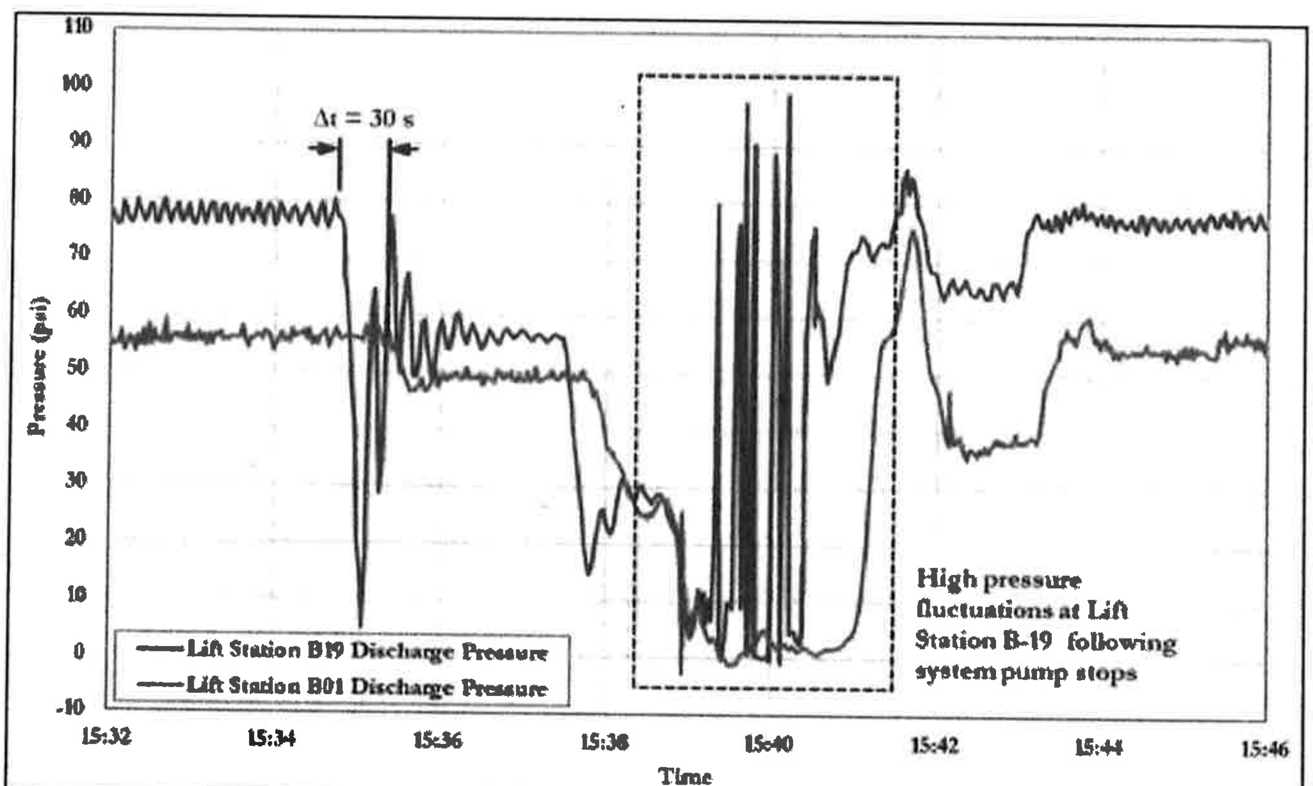


Figure 50: Transient pressure monitoring data from 3:32 PM to 3:46 PM on 30 September 2017 showing high pressure fluctuations at Lift Station B-19 following pump stop operations

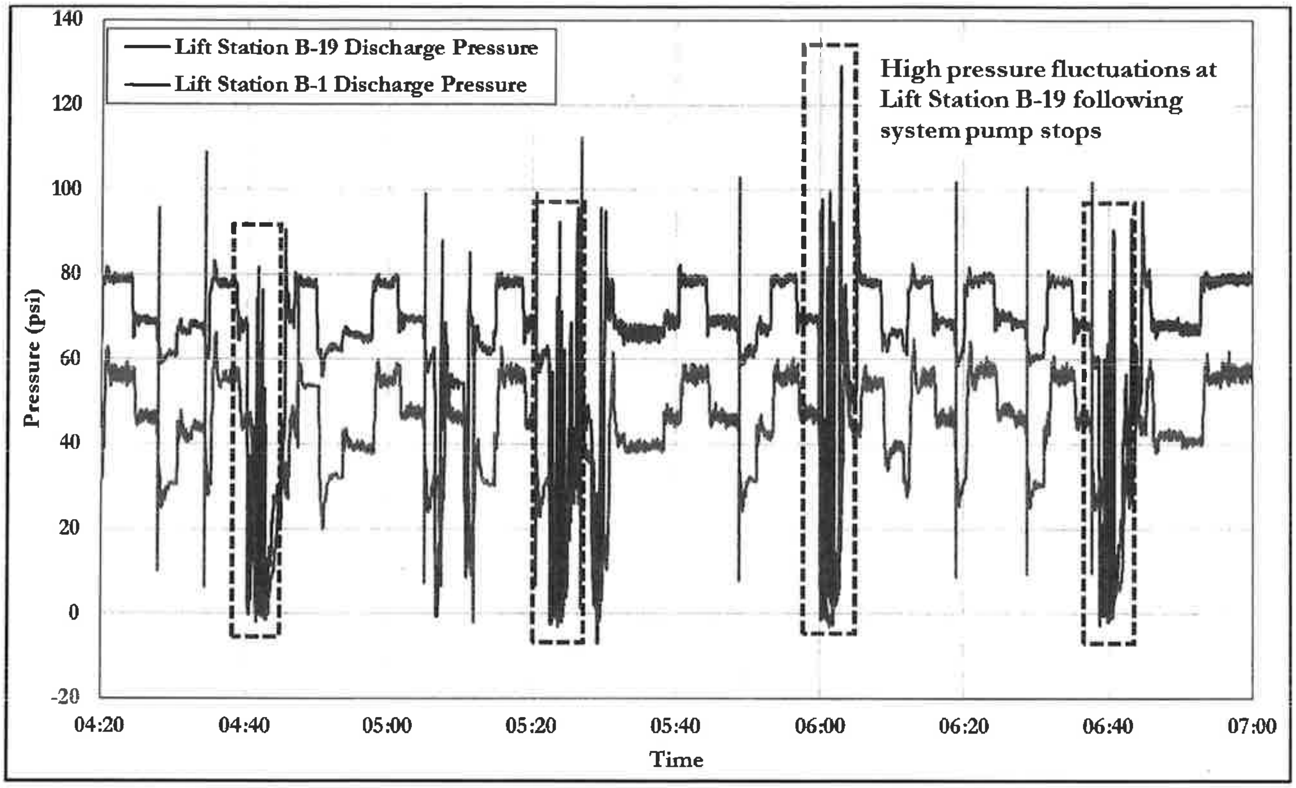


Figure 51: Transient pressure monitoring data from 4:20 AM to 7:00 AM on 05 October 2017 showing high pressure fluctuations at Lift Station B-19 following pump stop operations

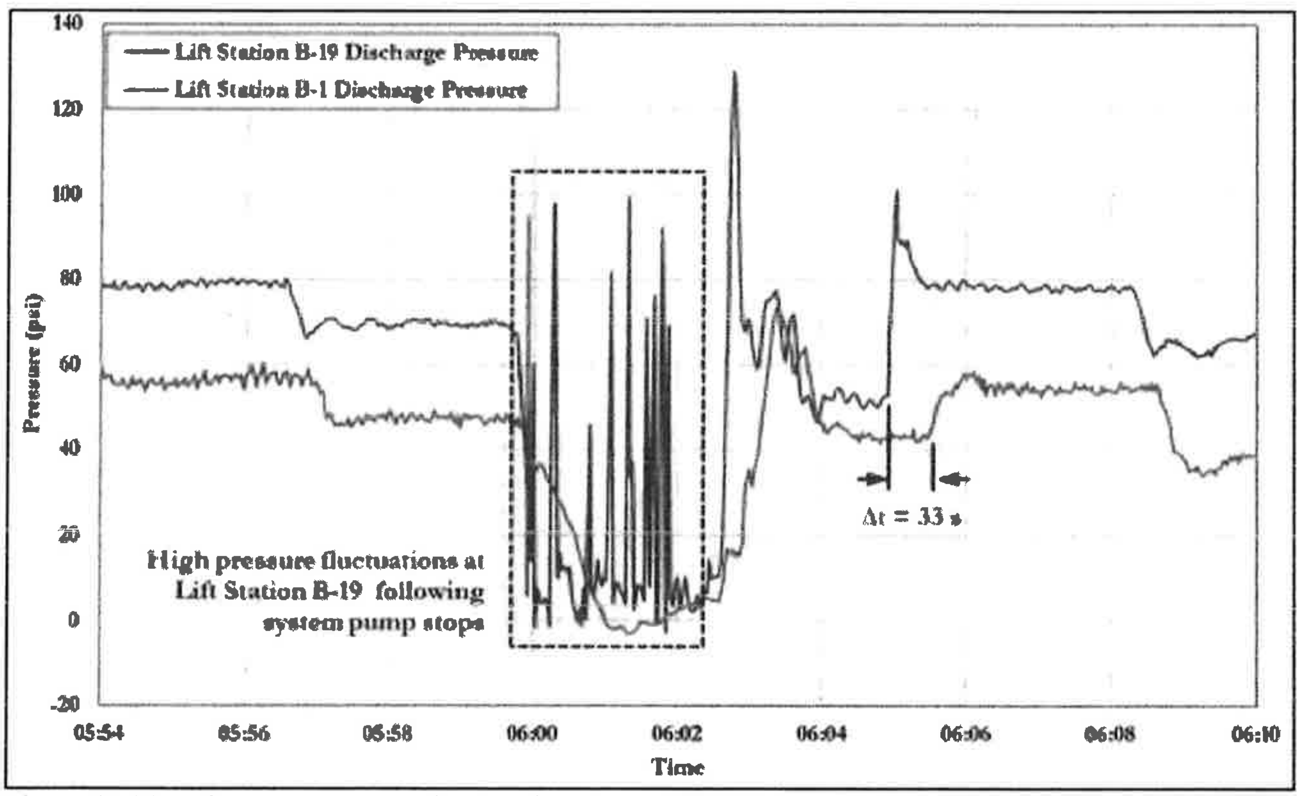


Figure 52: Transient pressure monitoring data from 5:54 AM to 6:10 AM on 05 October 2017 showing high pressure fluctuations at Lift Station B-19 following pump stop operations

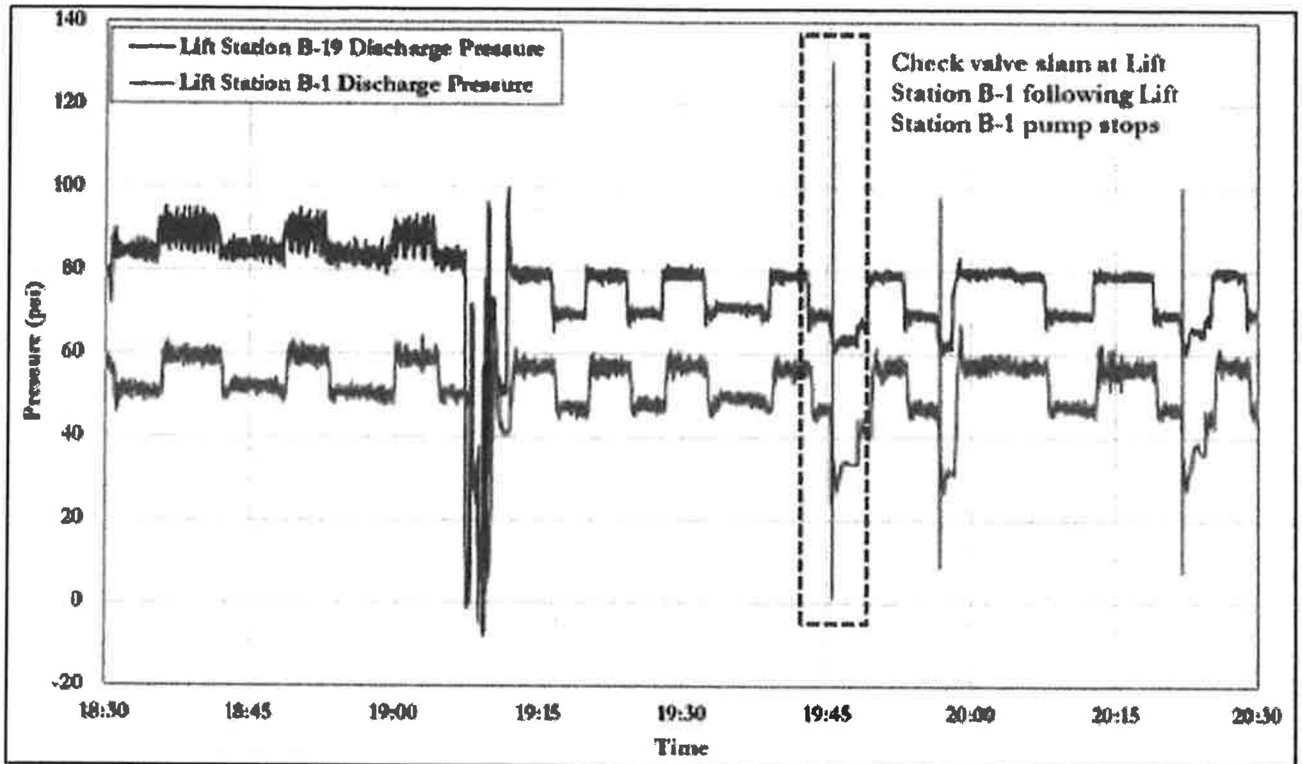


Figure 53: Transient pressure monitoring data from 6:30 PM to 8:30 PM on 05 October 2017 showing normal pump on-off operations and check valve slam at Lift Station B-1

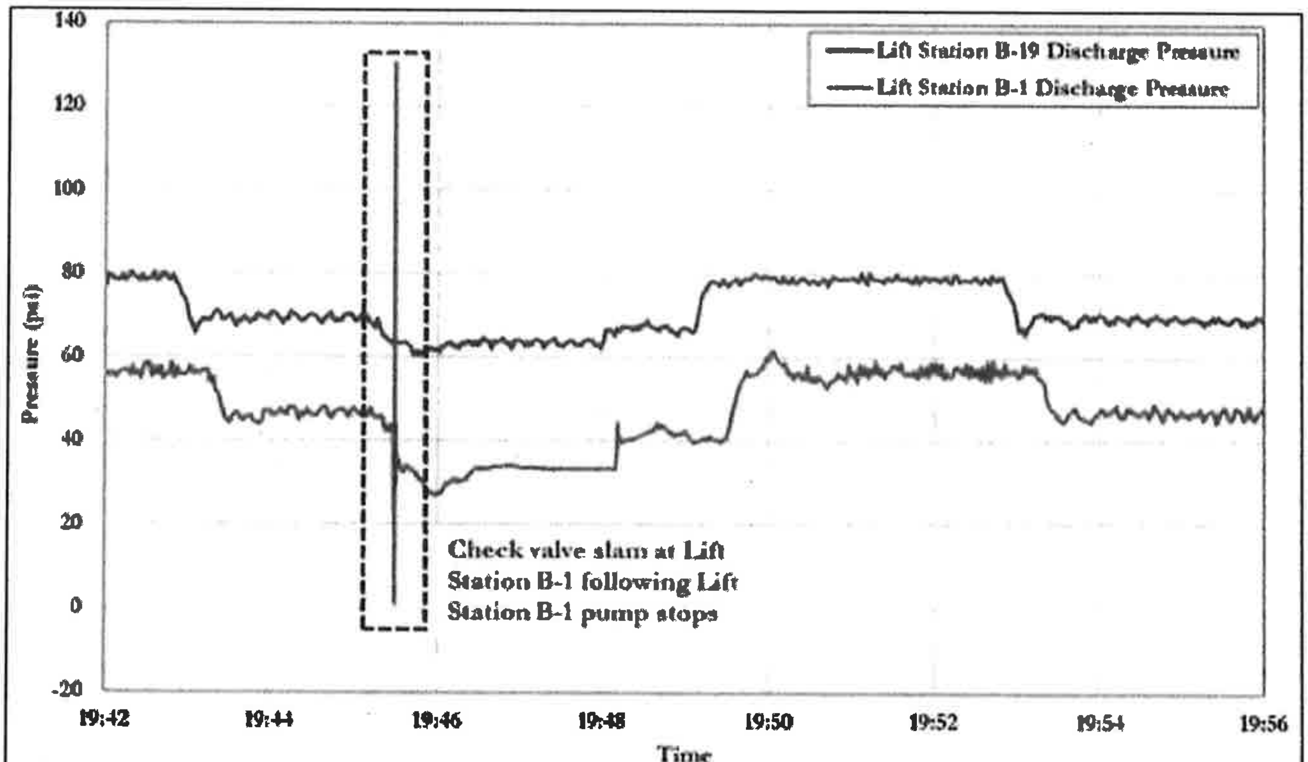


Figure 54: Transient pressure monitoring data from 7:42 PM to 7:56 PM on 05 October 2017 showing normal pump on-off operations and check valve slam at Lift Station B-1

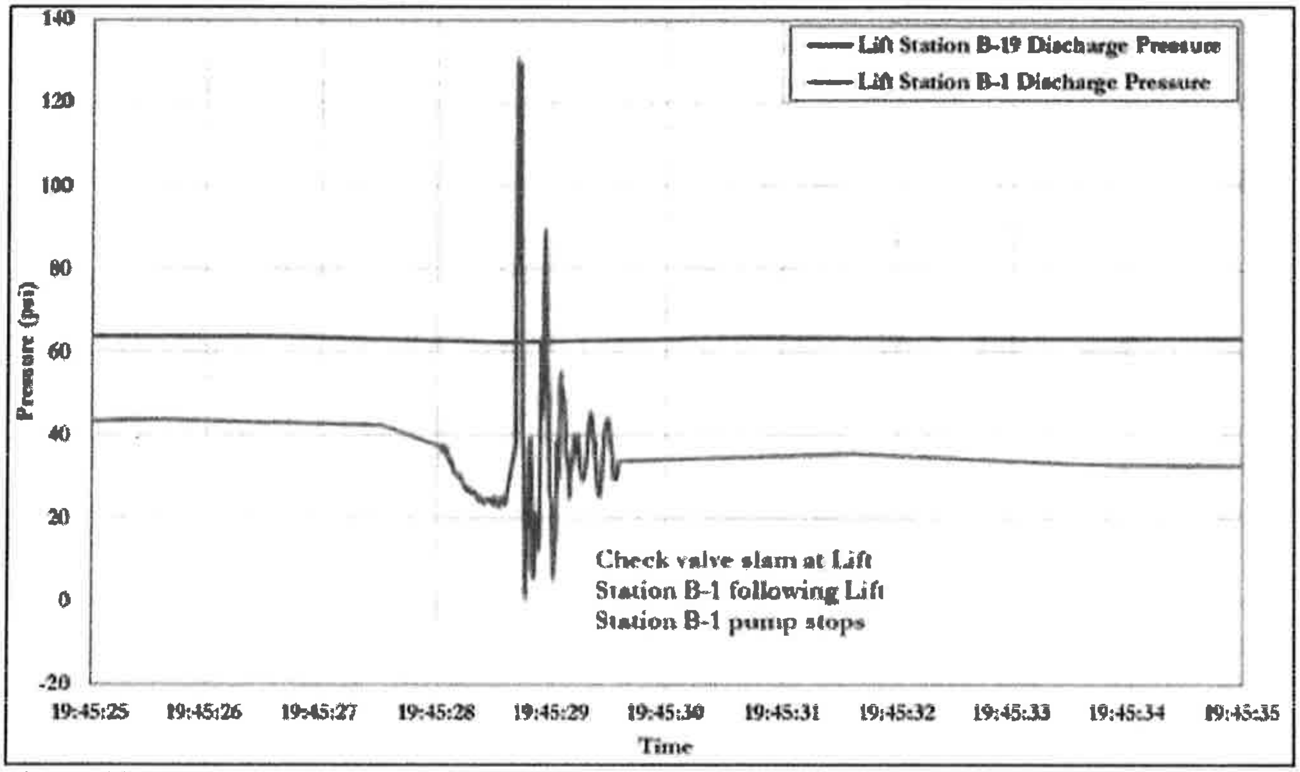


Figure 55: Transient pressure monitoring data at 7:45 PM on 05 October 2017 showing check valve slam at Lift Station B-1

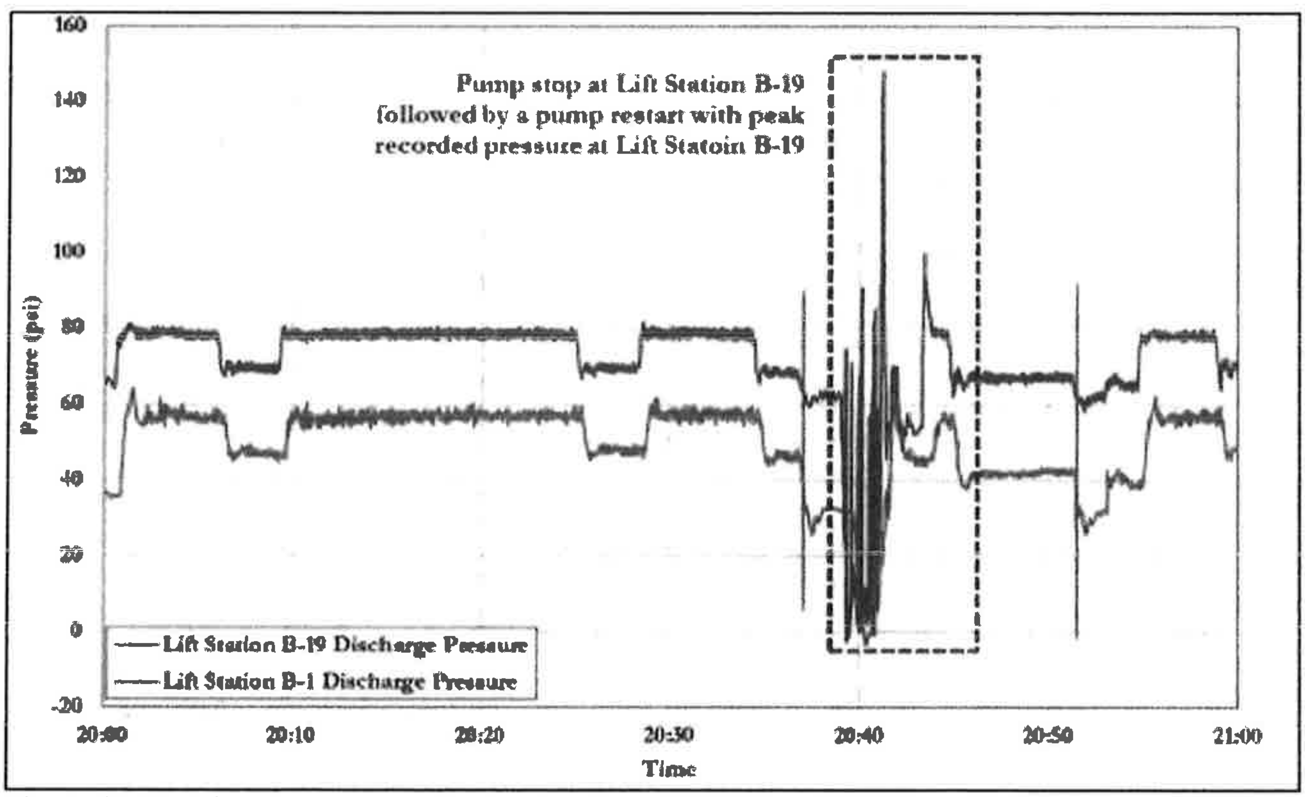


Figure 56: Transient pressure monitoring data from 8:00 PM to 9:00 PM on 08 October 2017 showing peak pressure recorded at Lift Station B-19

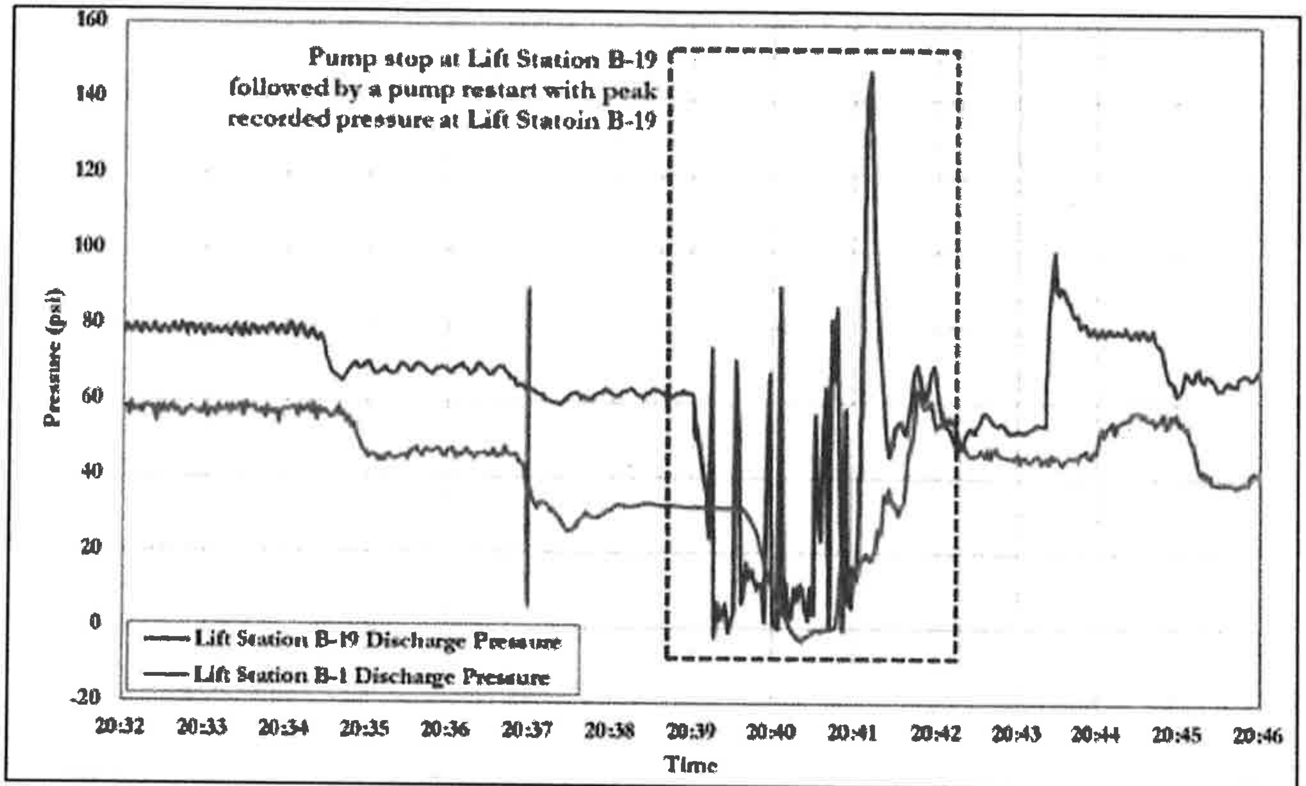


Figure 57: Transient pressure monitoring data from 8:32 PM to 8:36 PM on 08 October 2017 showing peak pressure recorded at Lift Station B-19

APPENDIX D: BLACOH COMPLETE TRANSIENT PRESSURE DATA PLOTS

HDR – B19

September 26, 2017 – October 25, 2017

Figure 1: September 26, 2017 – October 25, 2017

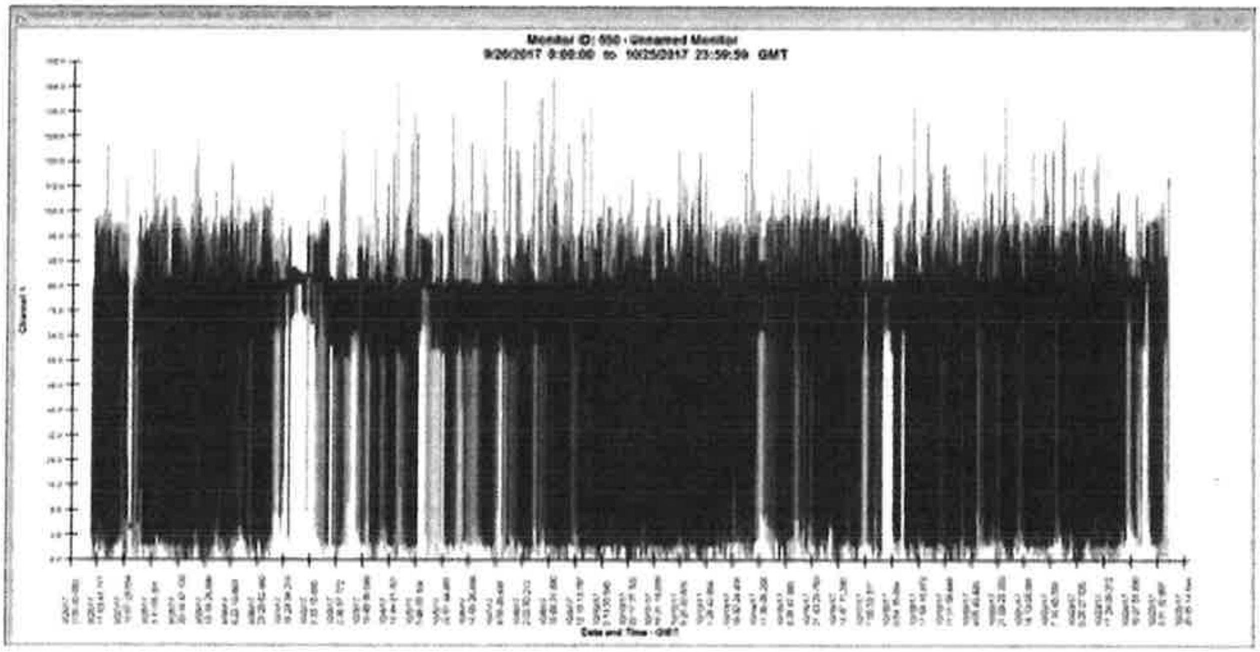


Figure 2: Highest Recorded Data Point (October 8, 2017)

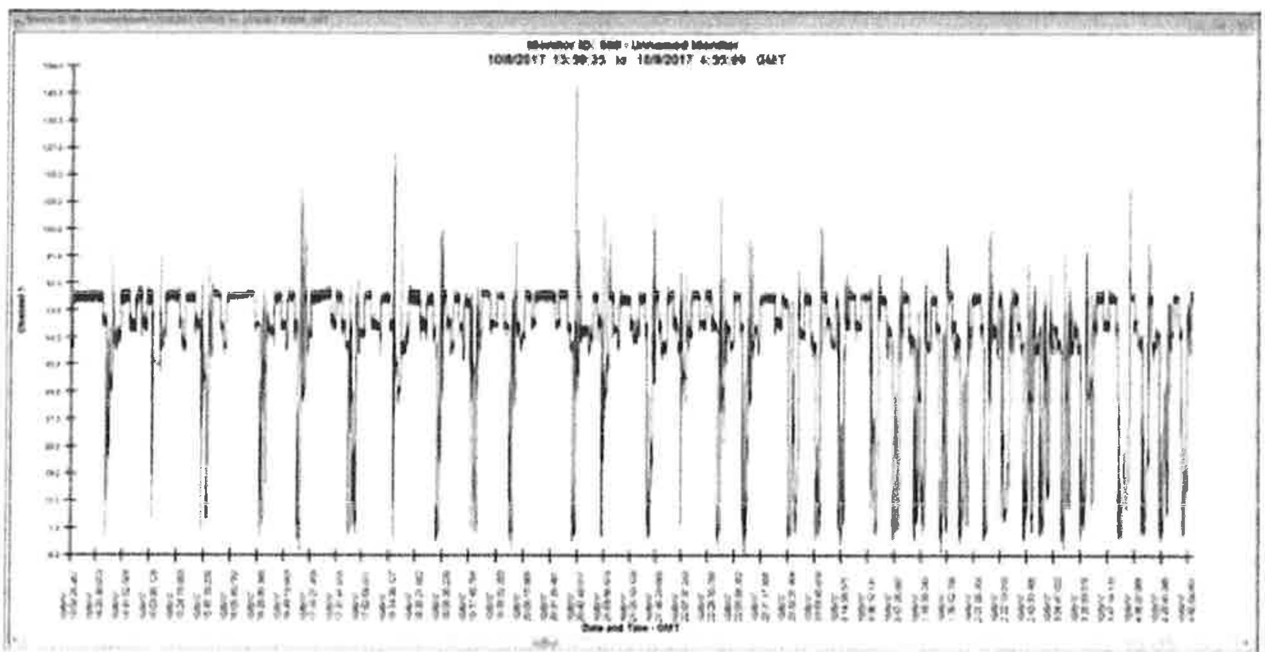


Figure 3: Highest Recorded Data Point (Zoom Level A)

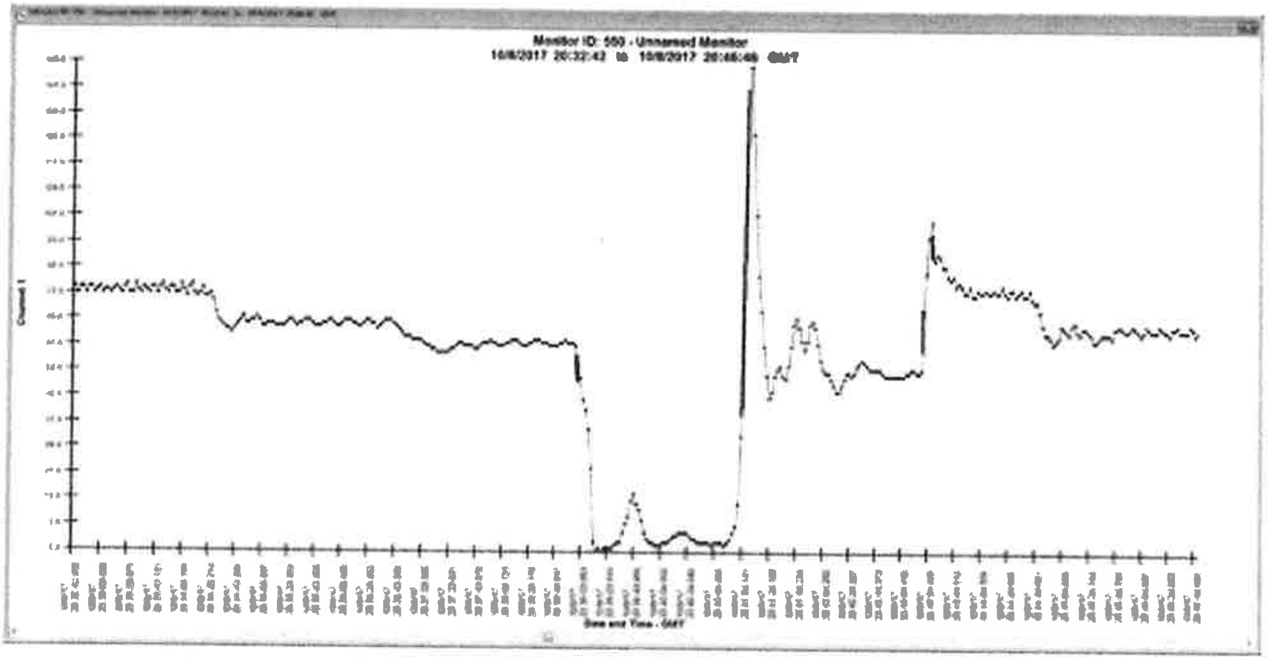


Figure 4: Highest Recorded Data Point (Zoom Level B)

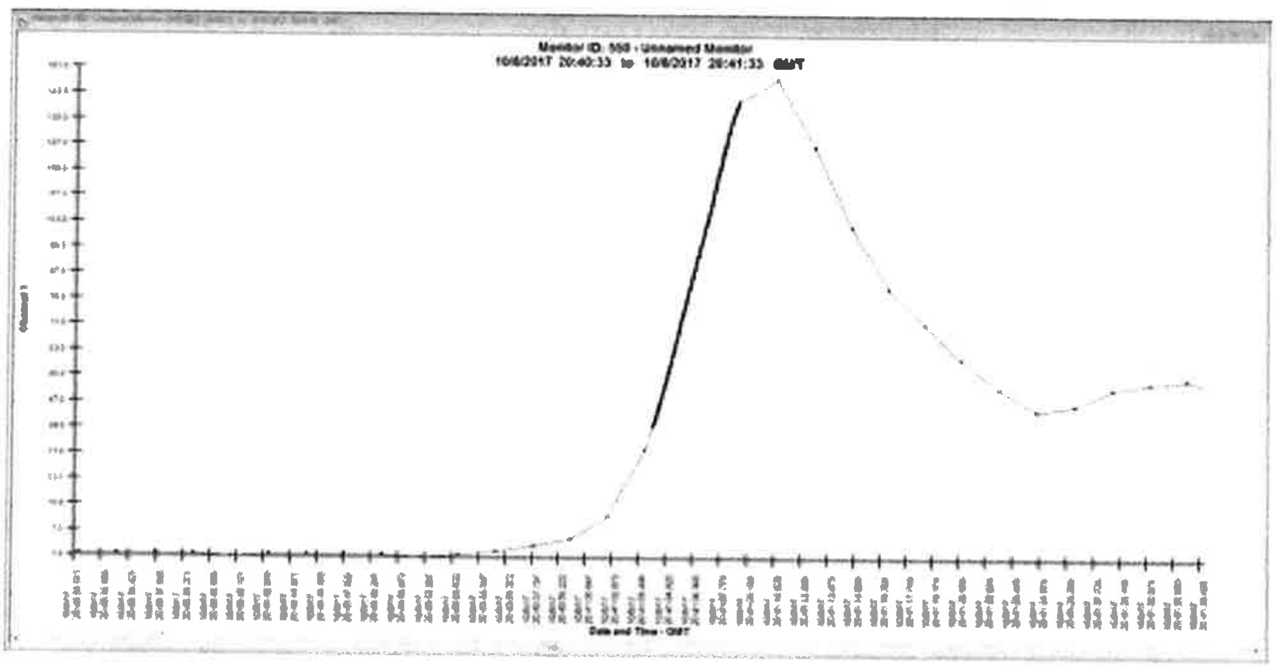


Figure 5: Lowest Recorded Data Point ***(One instance)***

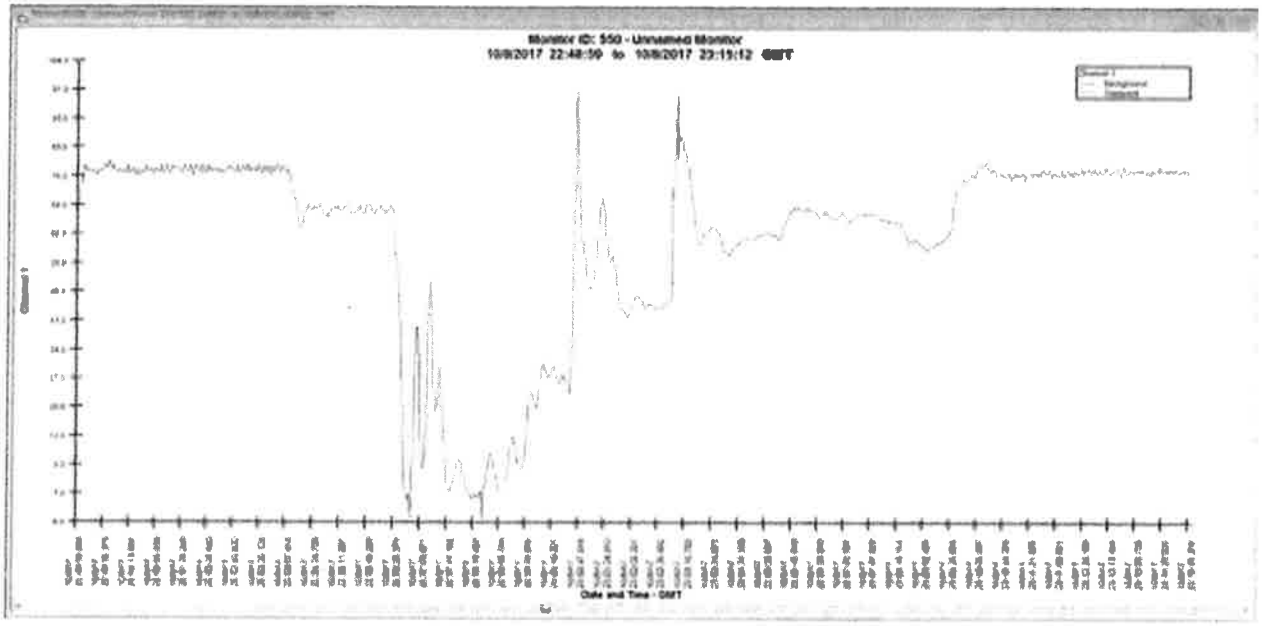


Figure 6: Lowest Recorded Data Point (Zoom Level A)

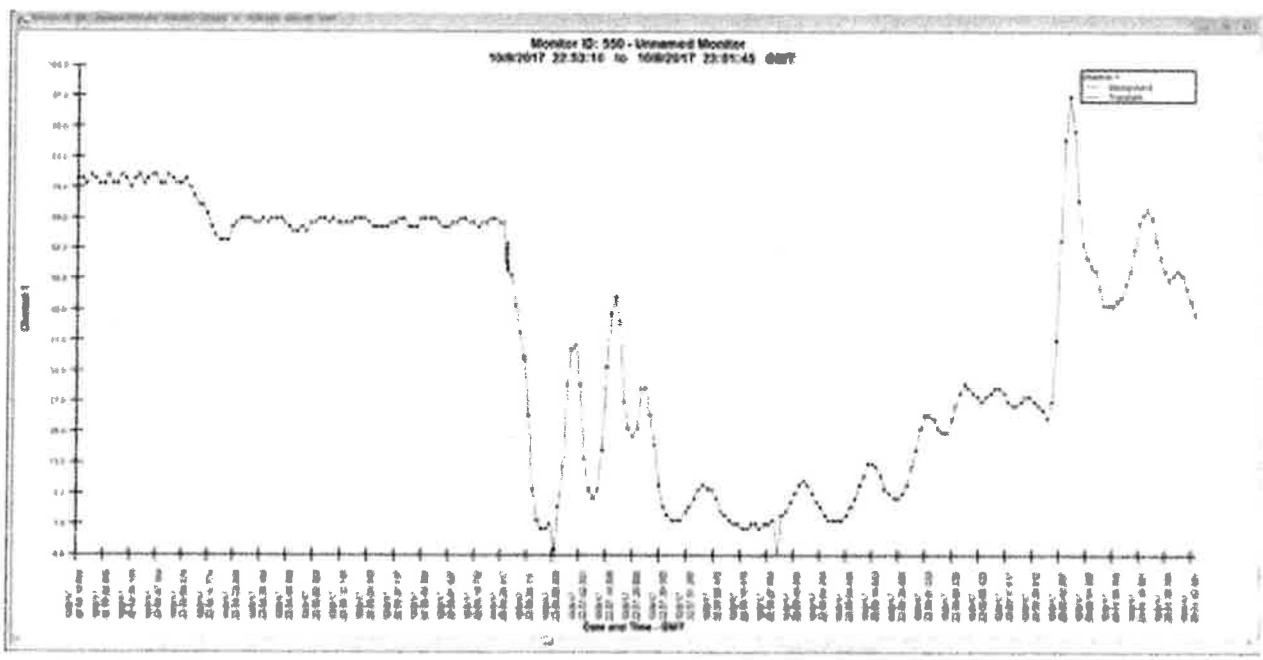


Figure 7: Lowest Recorded Data Point (Zoom Level B)

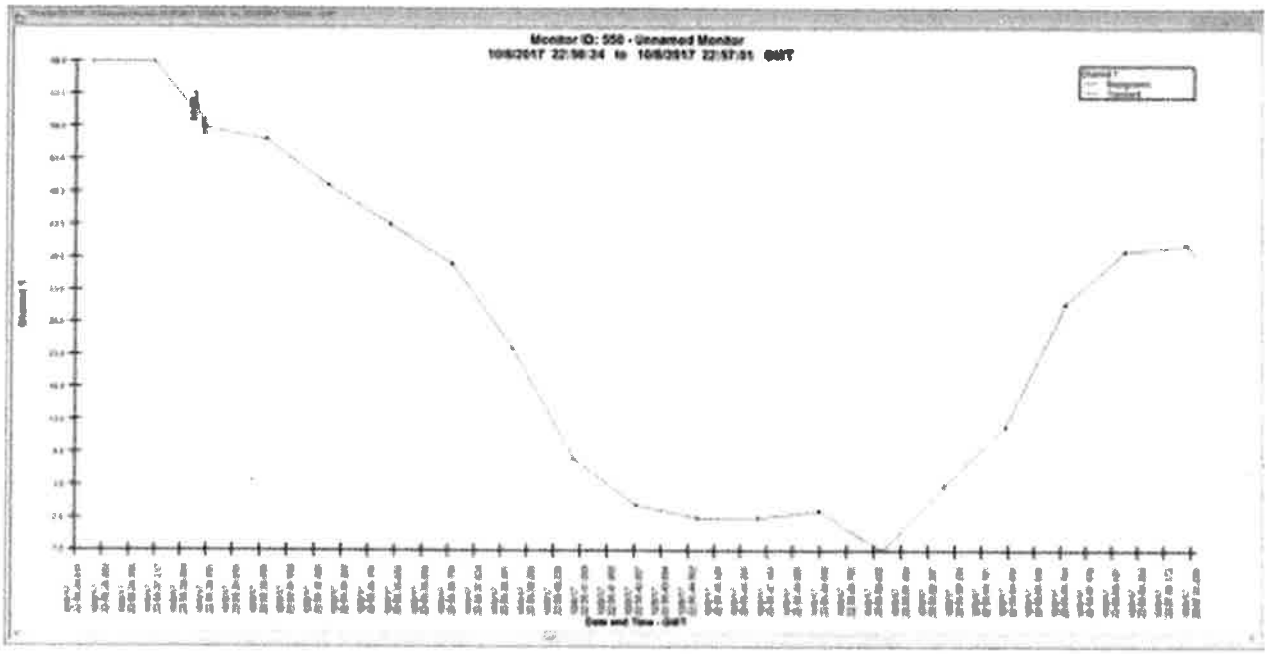


Figure 8: September 26, 2017

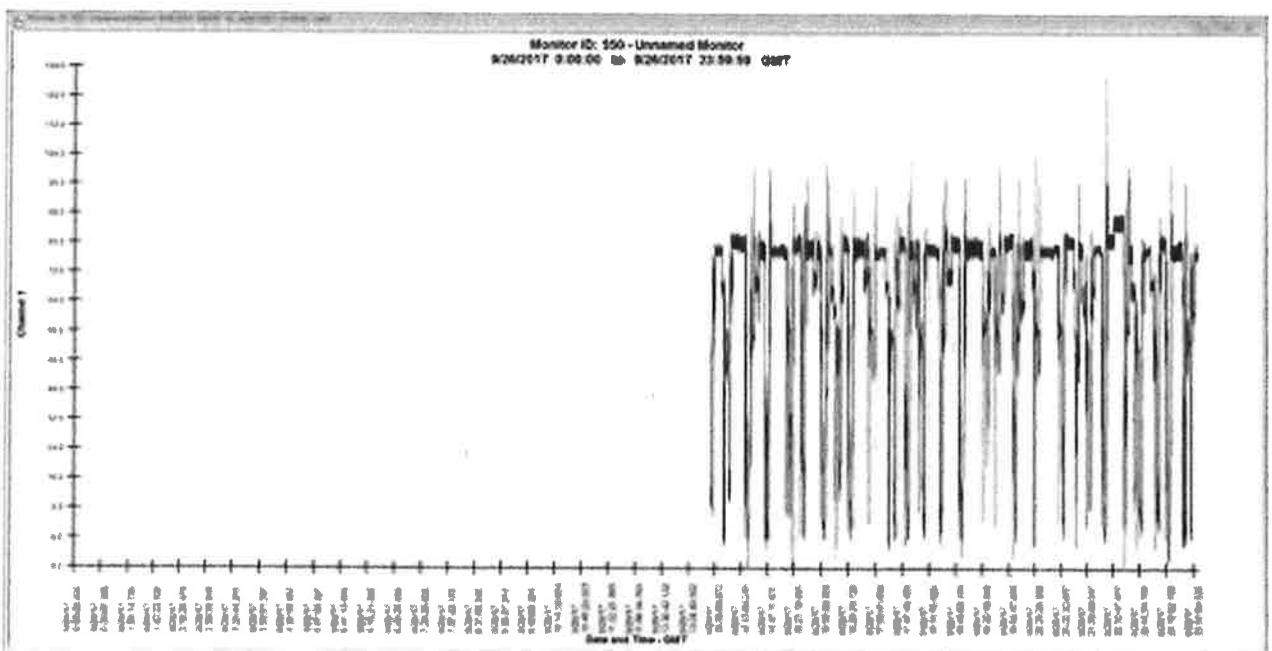


Figure 9: September 27, 2017

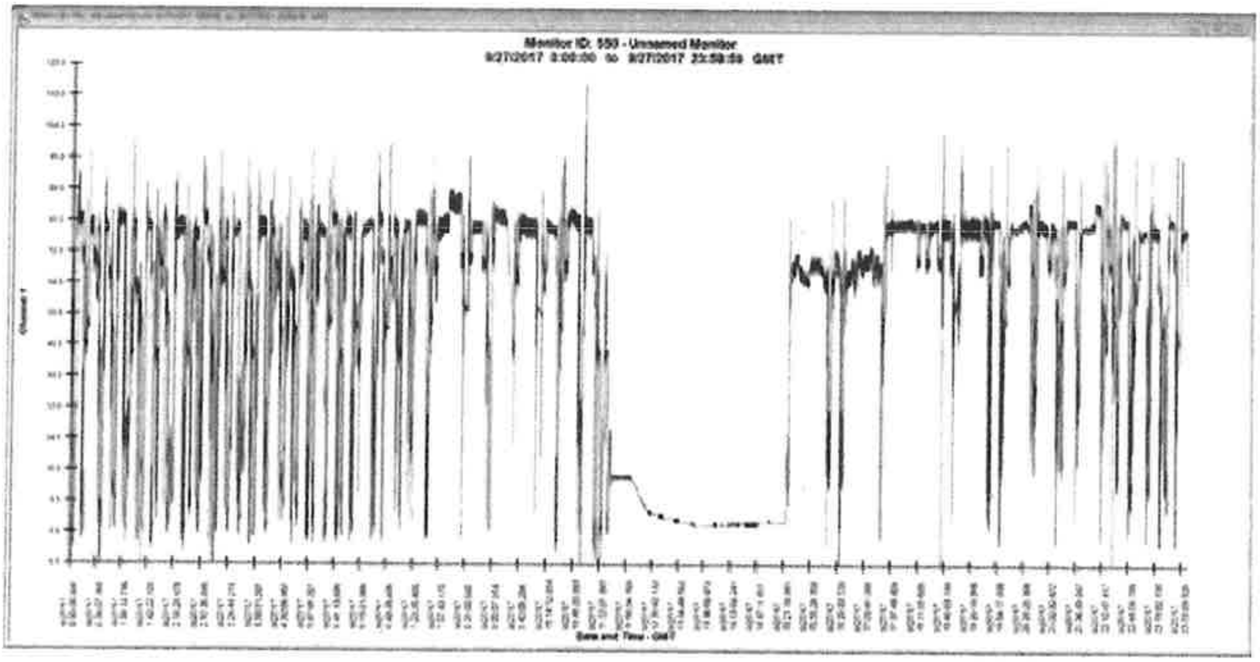


Figure 10: September 28, 2017

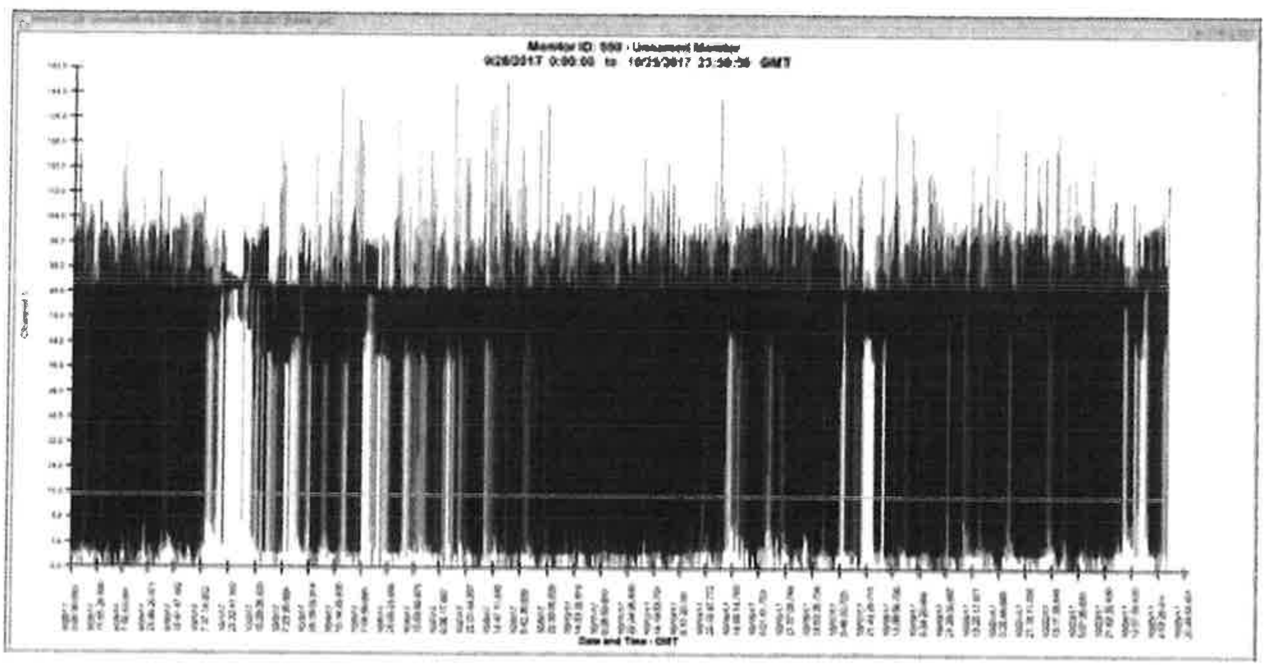


Figure 11: September 29, 2017

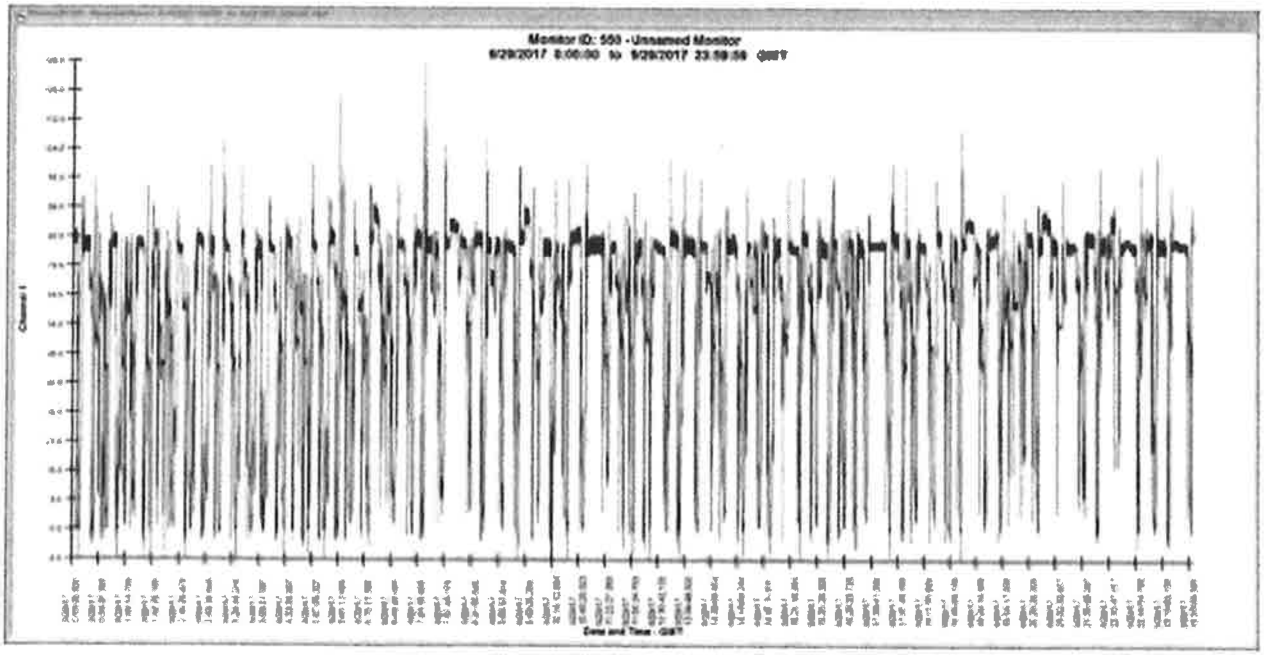


Figure 12: September 30, 2017

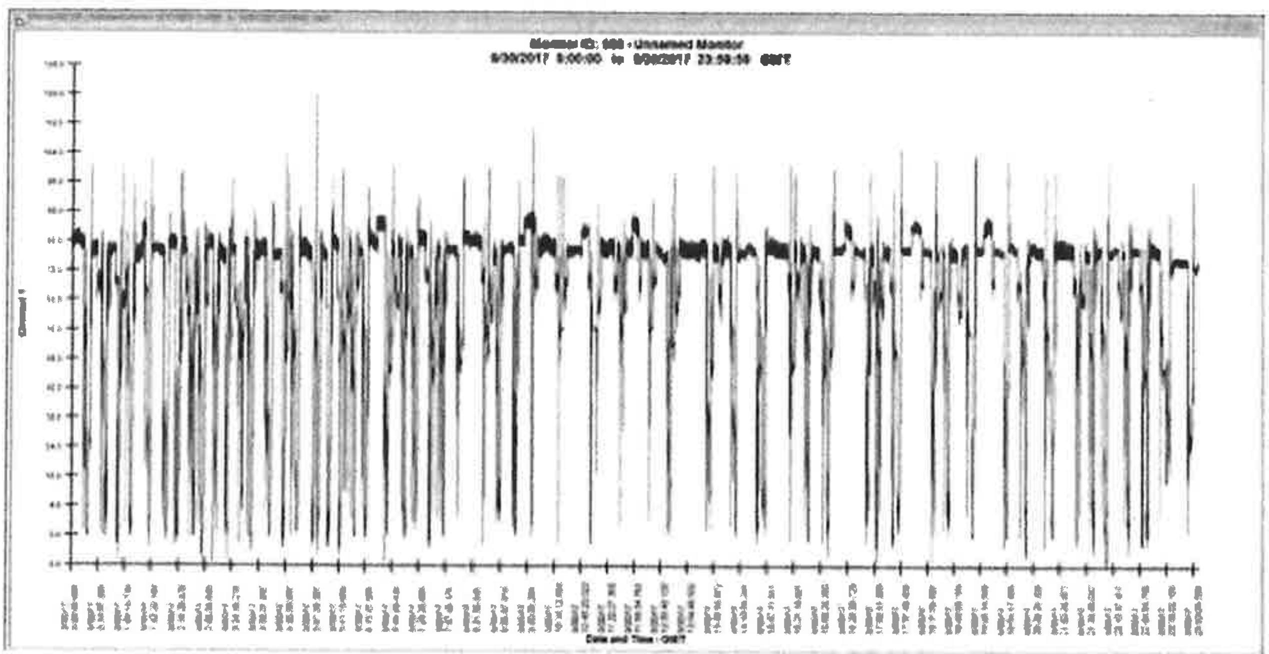


Figure 13: October 1, 2017

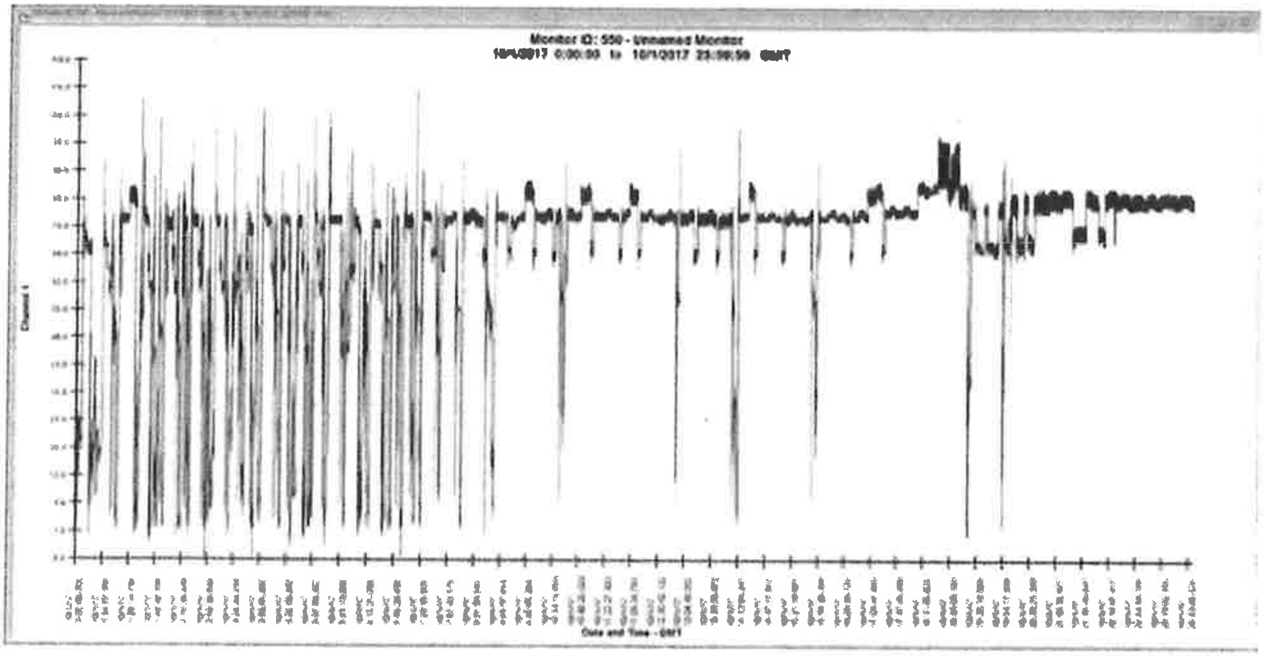


Figure 14: October 2, 2017

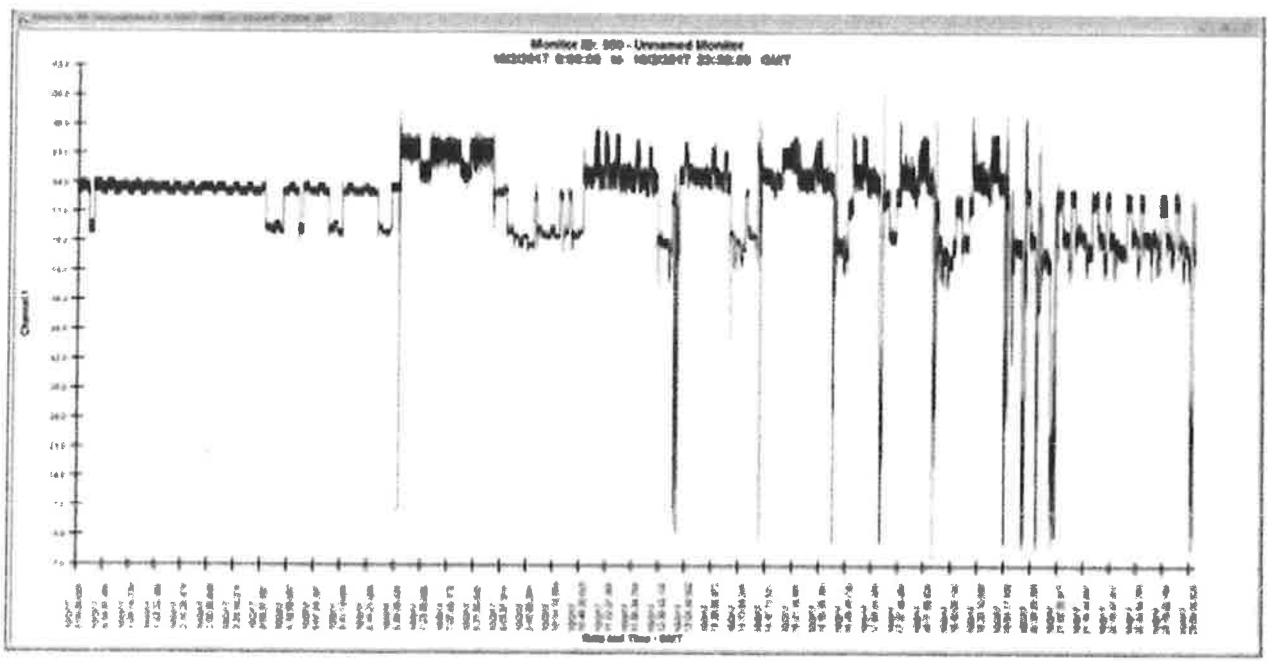


Figure 15: October 3, 2017

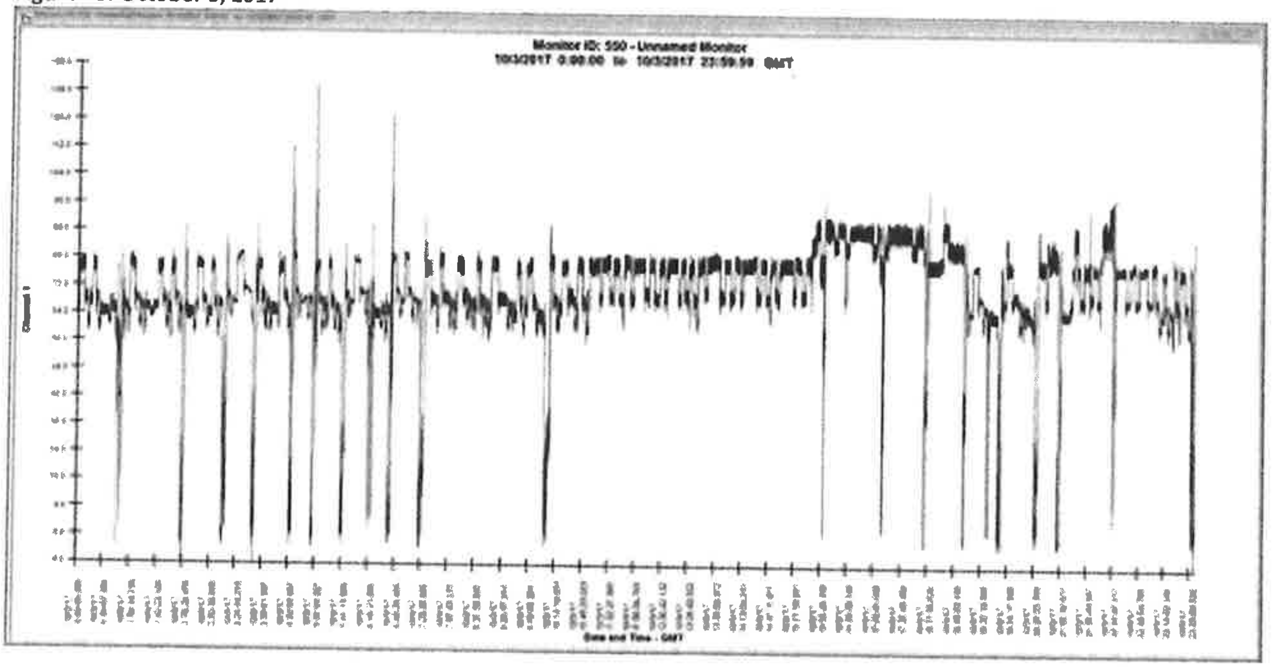


Figure 16: October 4, 2017

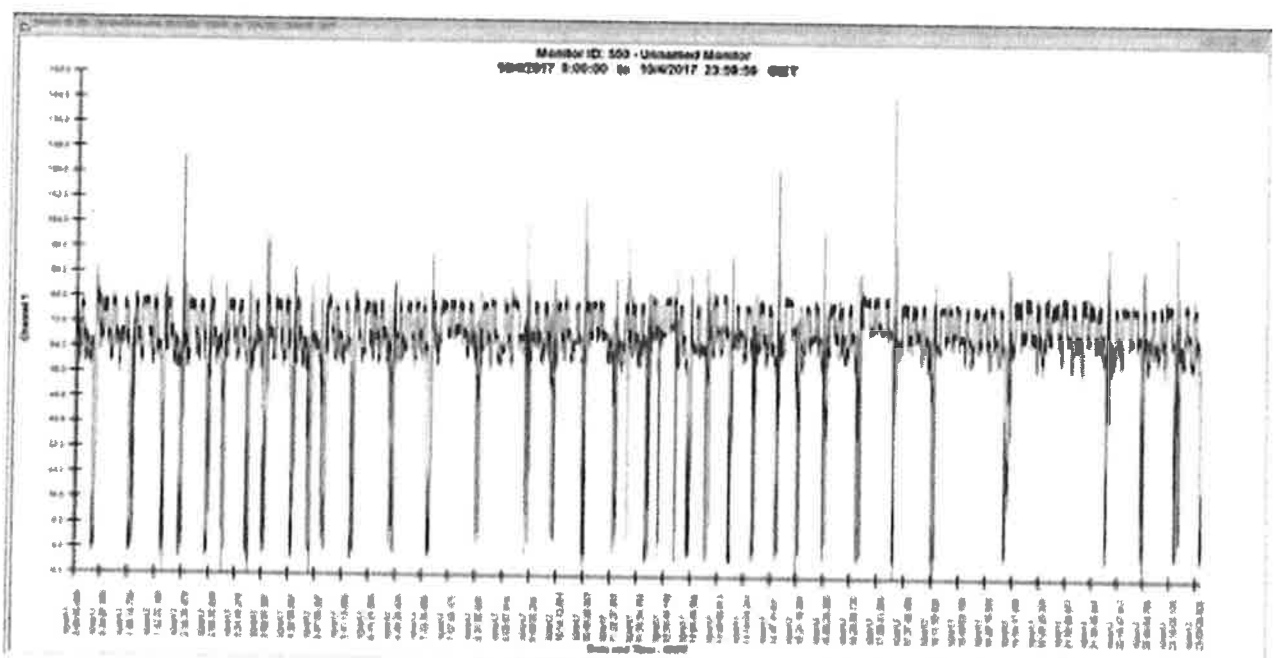


Figure 17: October 5, 2017

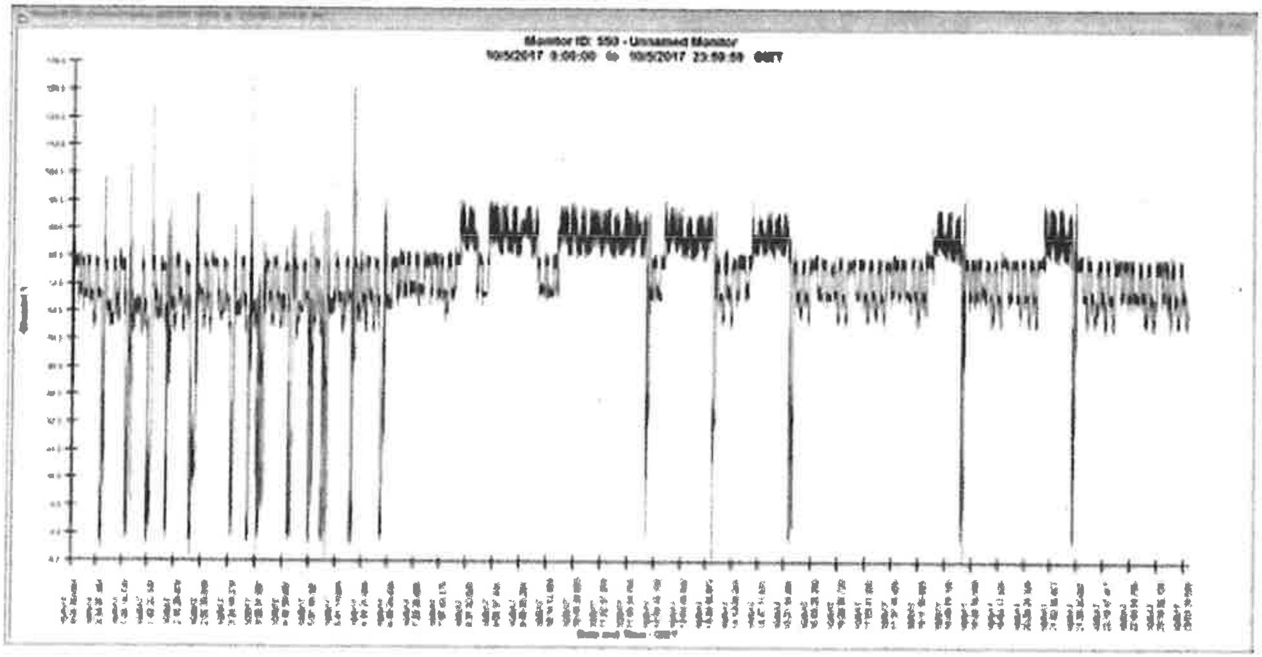


Figure 18: October 6, 2017

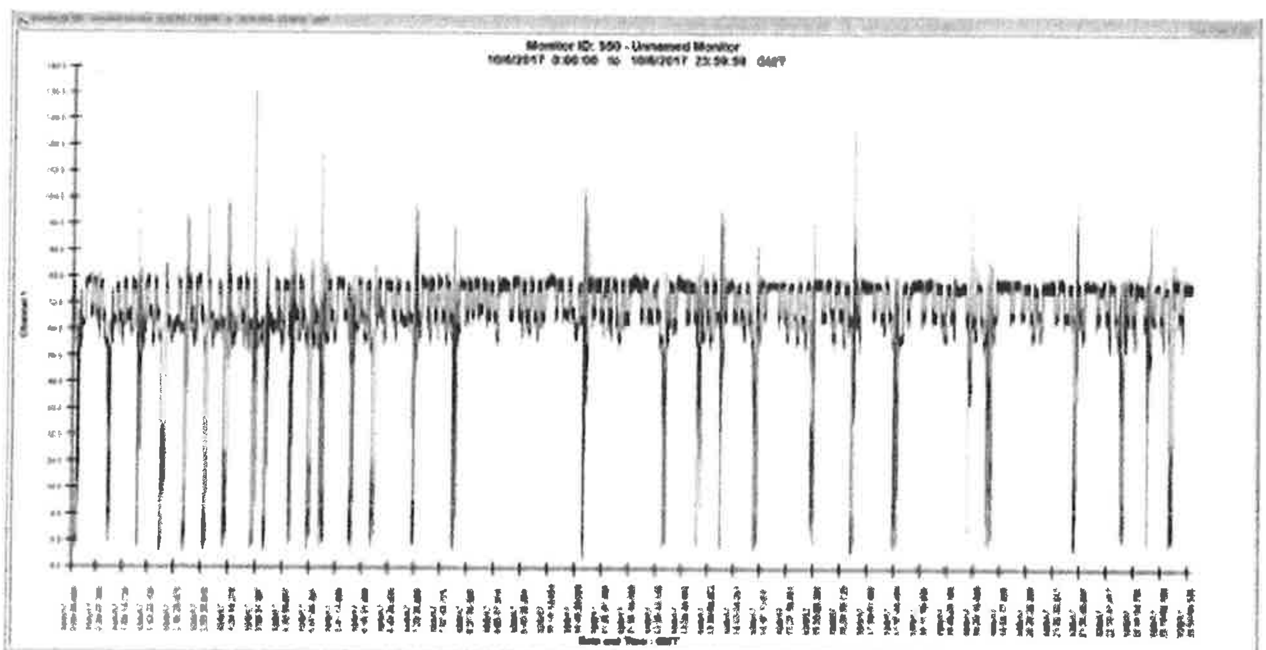


Figure 19: October 7, 2017

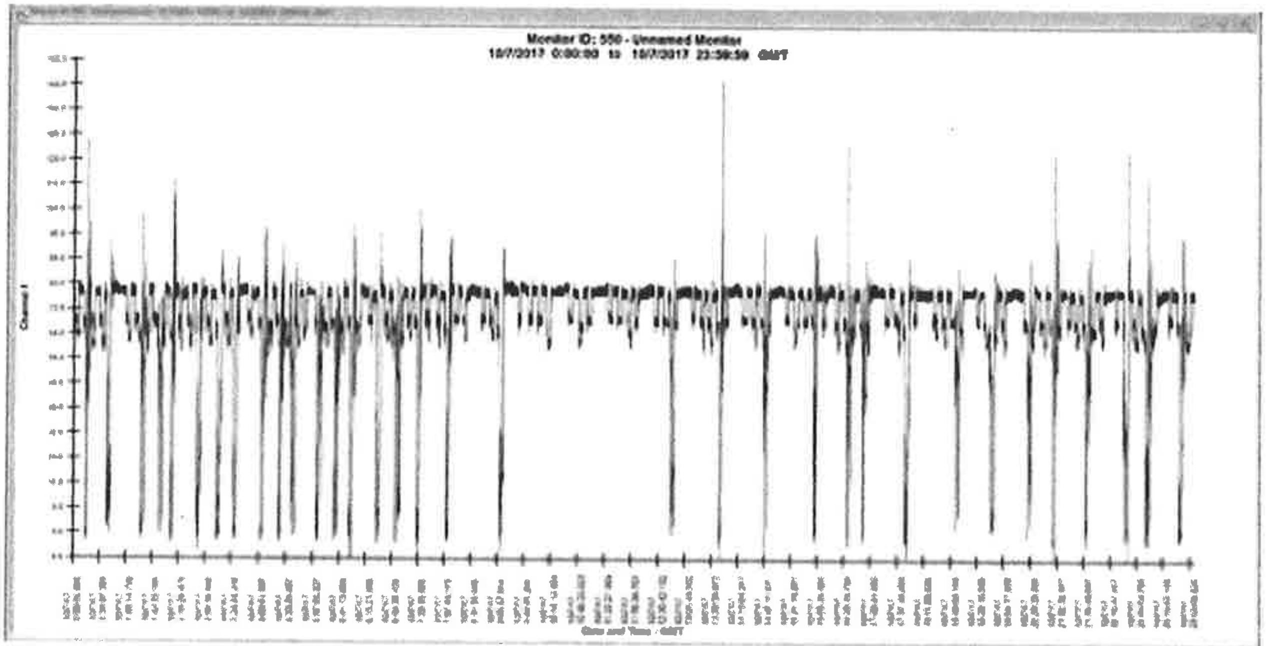


Figure 20: October 8, 2017

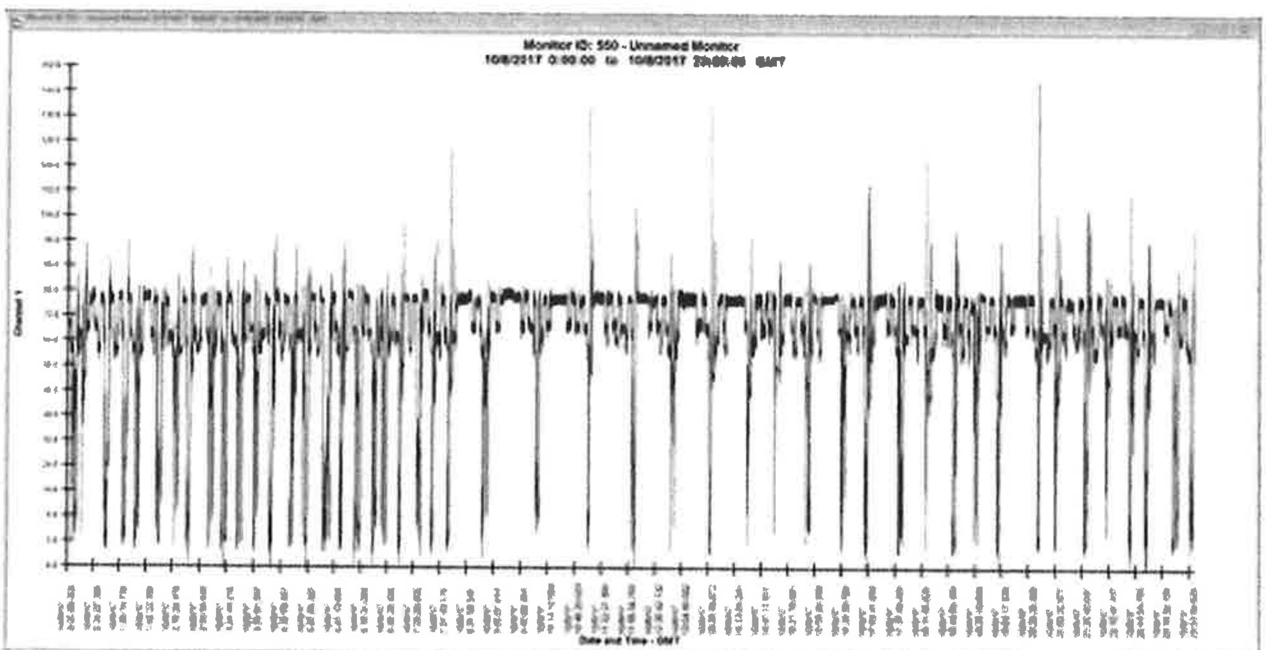


Figure 21: October 9, 2017

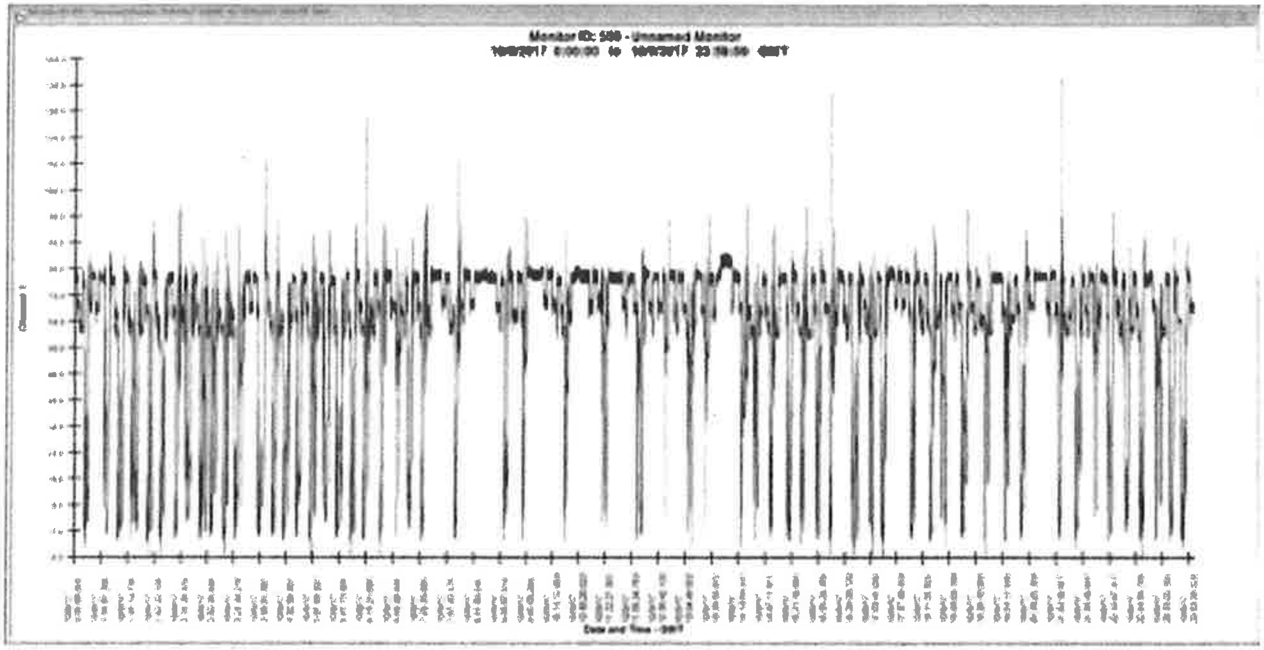


Figure 22: October 10, 2017

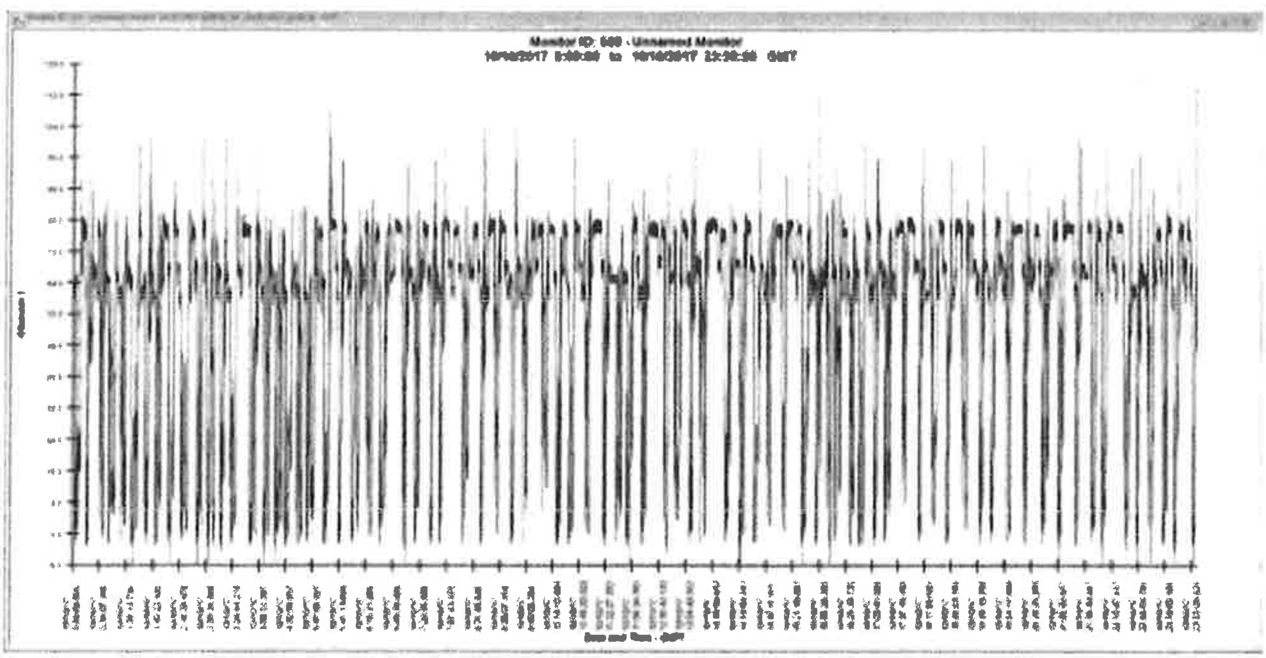


Figure 23: October 11, 2017

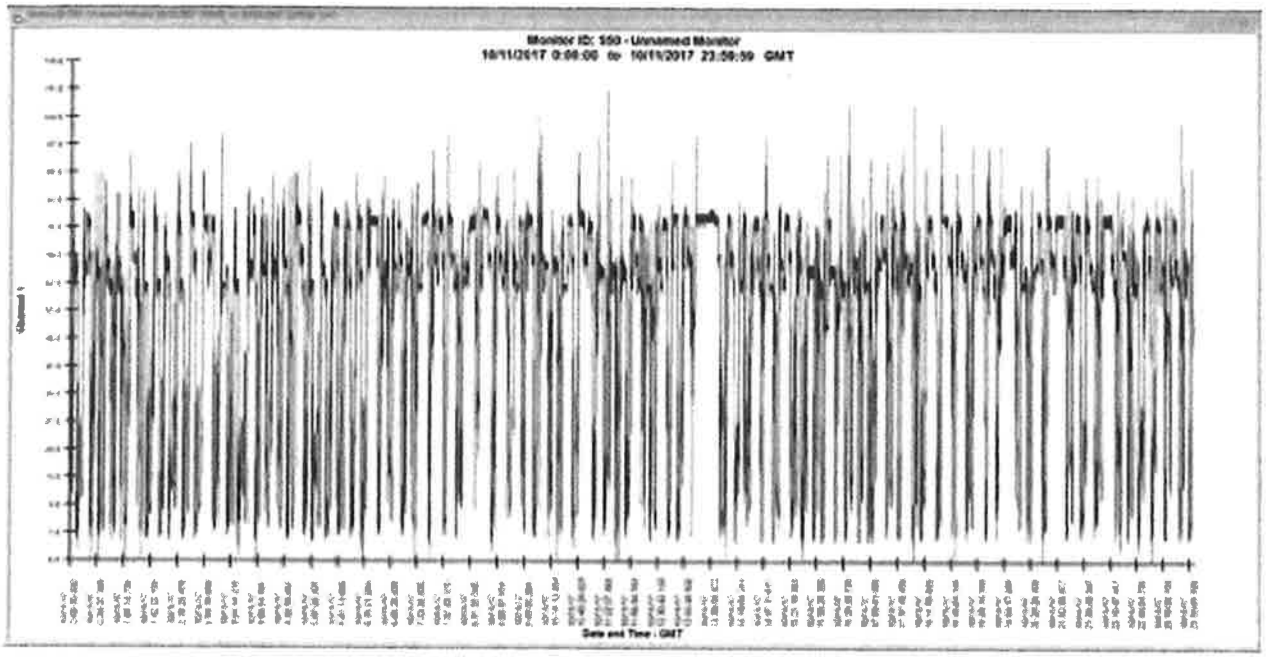


Figure 24: October 12, 2017

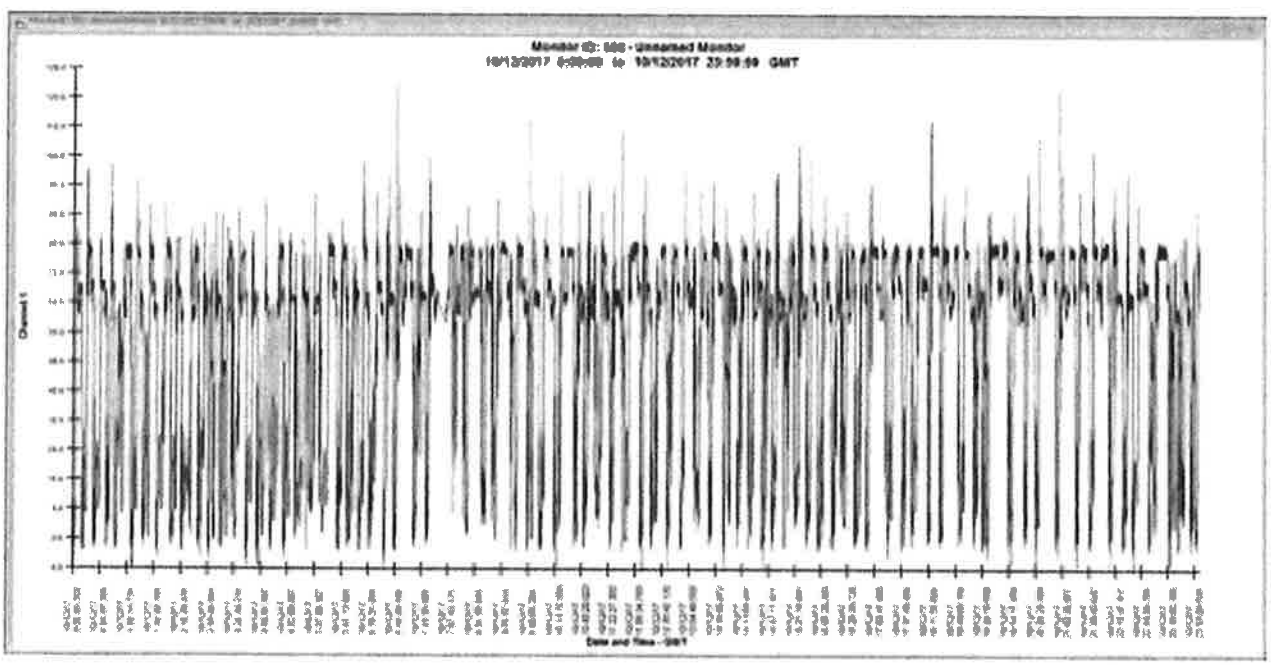


Figure 25: October 13, 2017

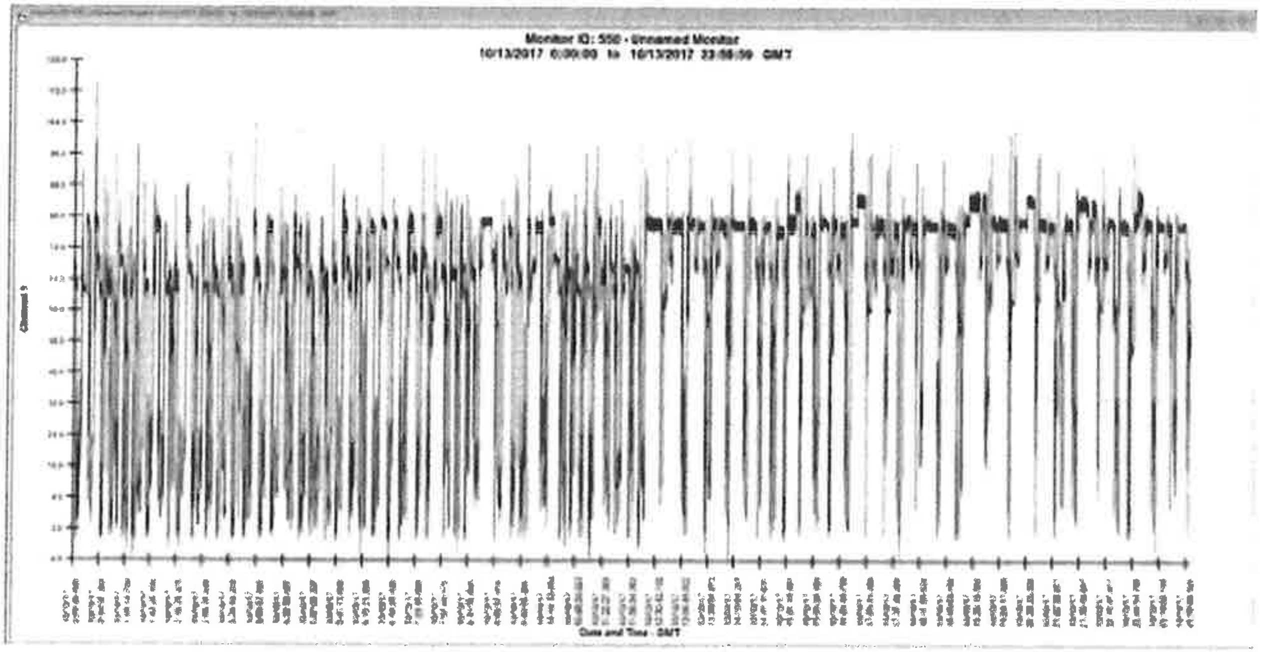


Figure 26: October 14, 2017

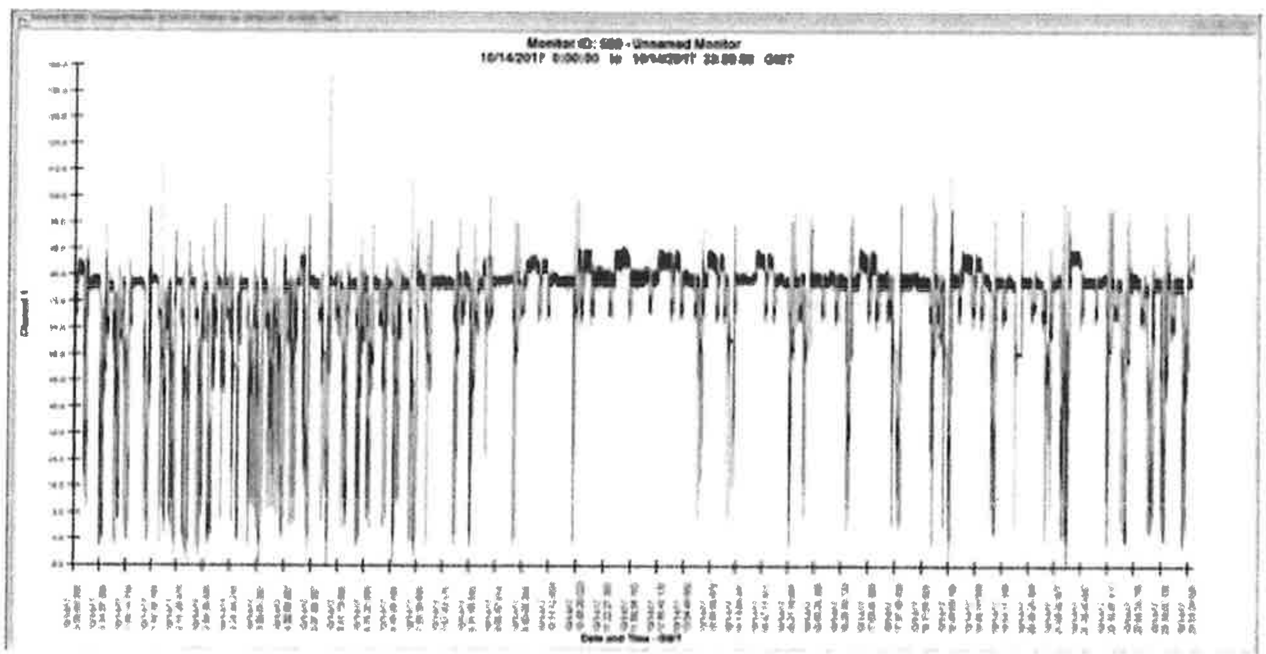


Figure 27: October 15, 2017

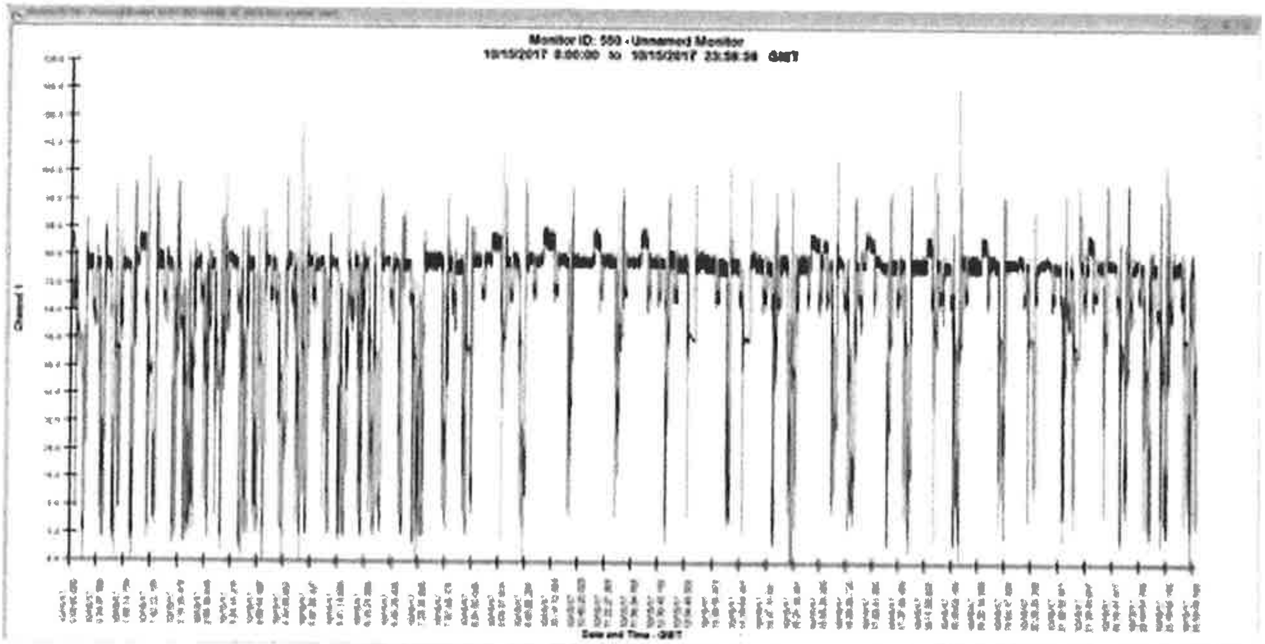


Figure 28: October 16, 2017

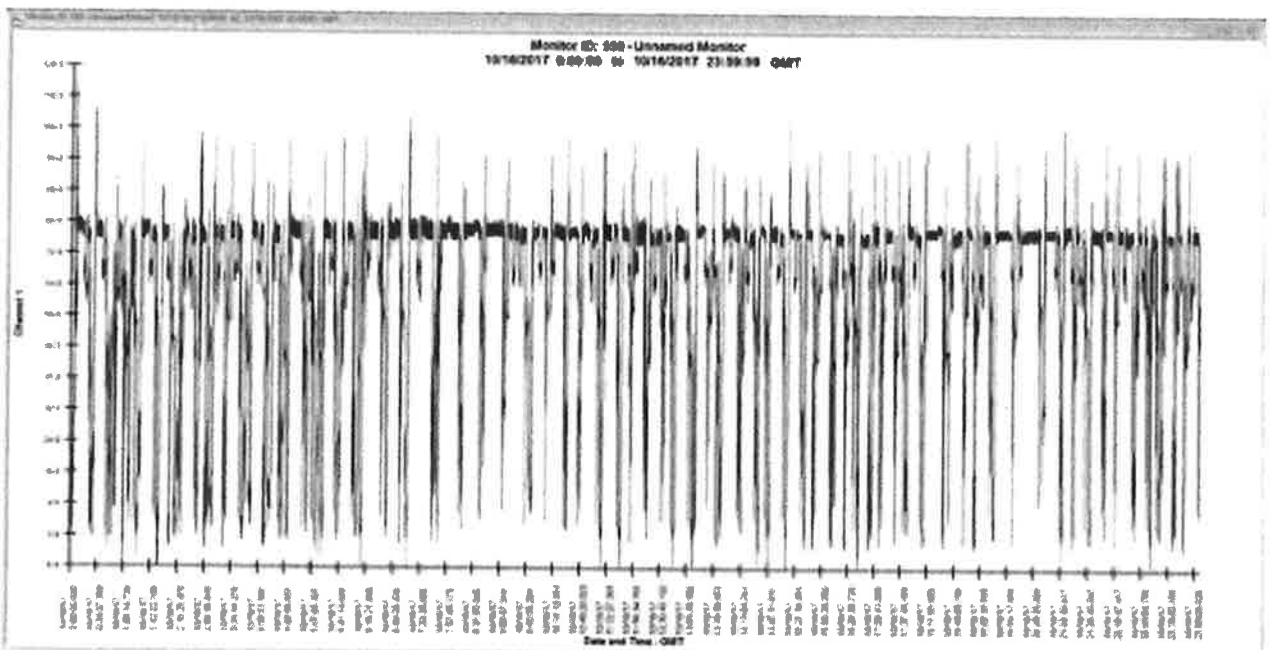


Figure 29: October 17, 2017

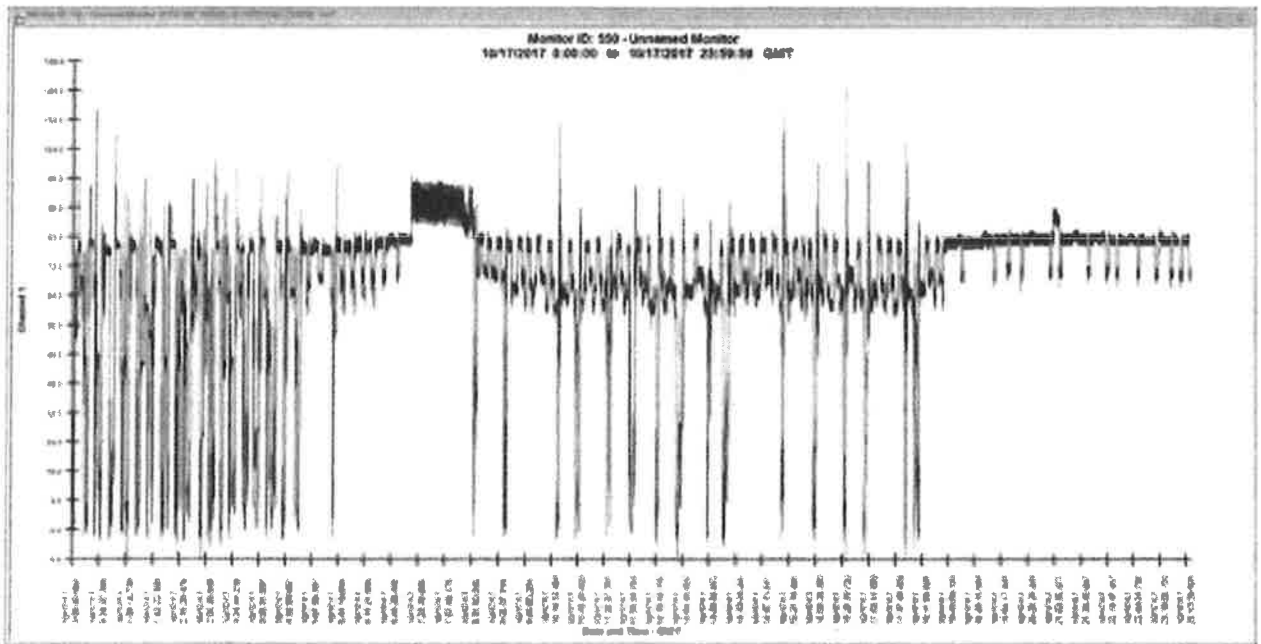


Figure 30: October 18, 2017

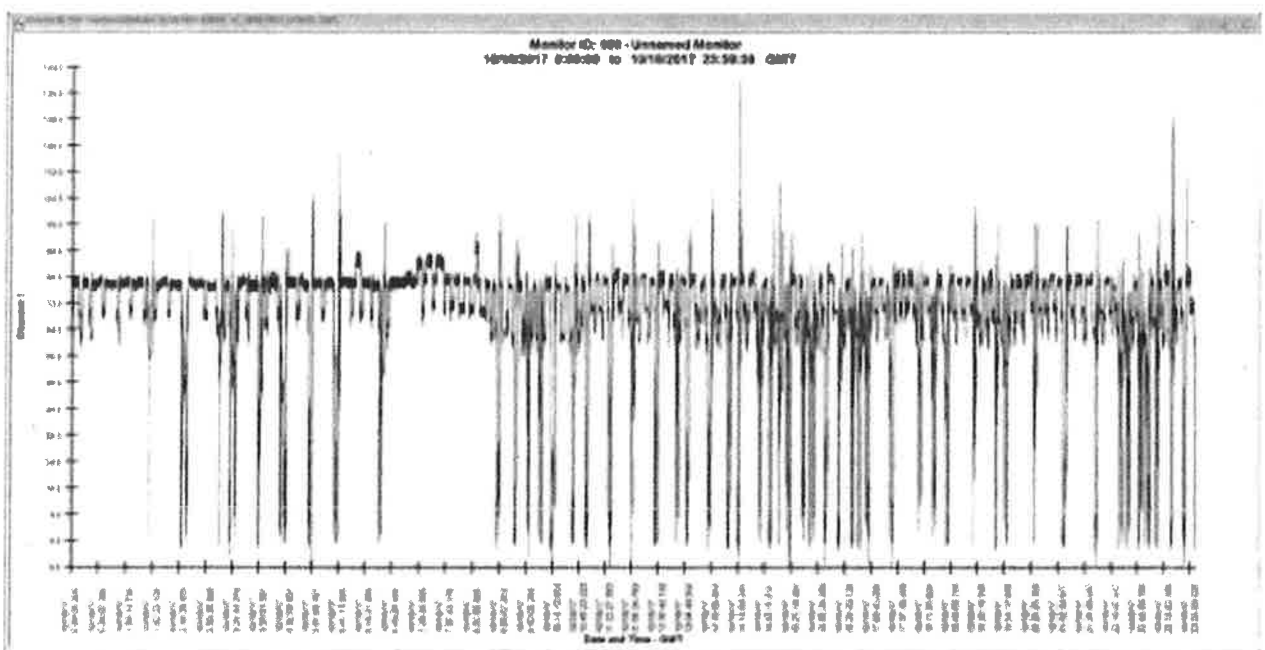


Figure 31: October 19, 2017

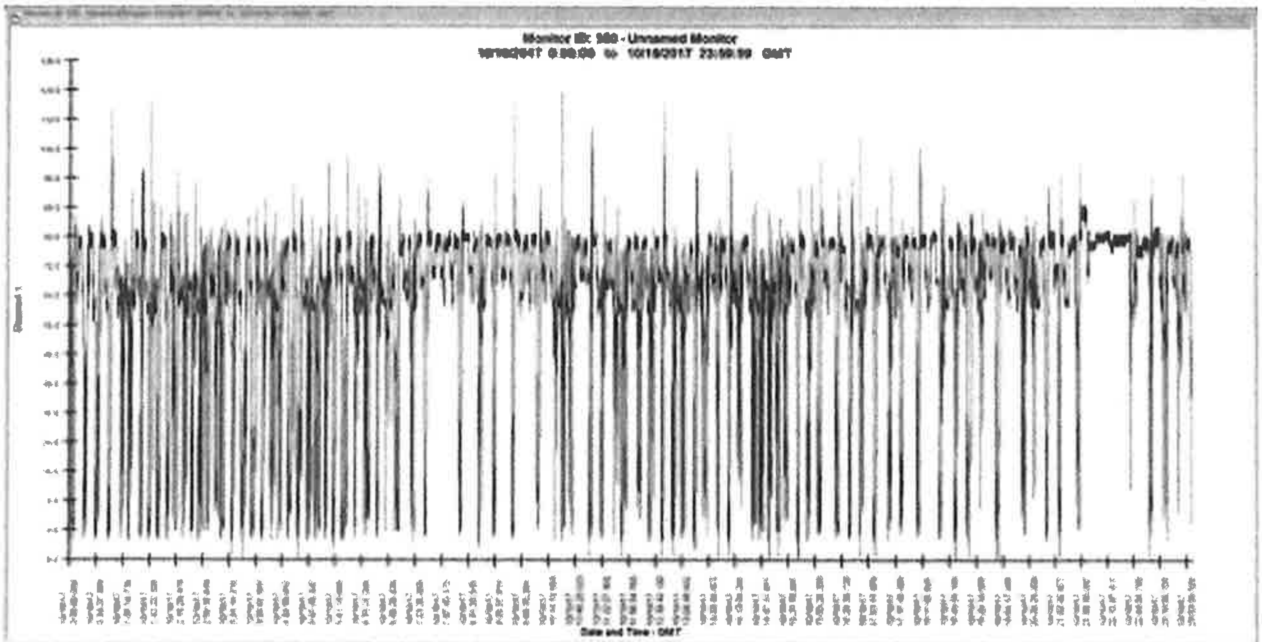


Figure 32: October 20, 2017

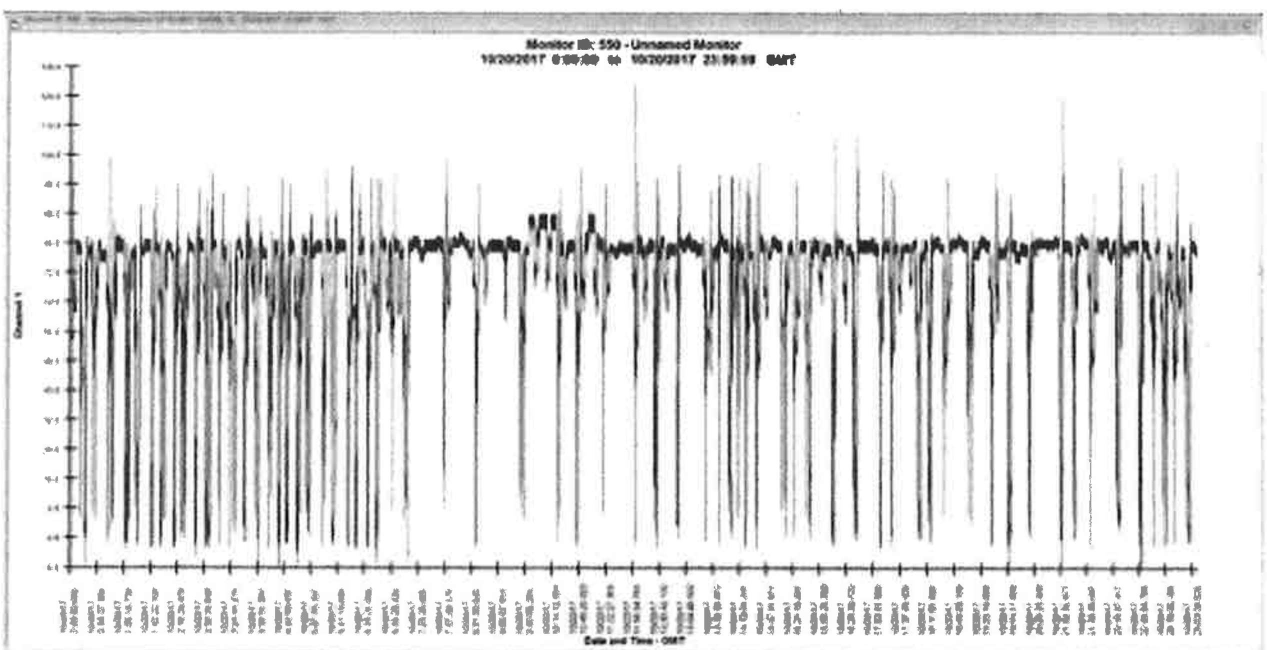


Figure 33: October 21, 2017

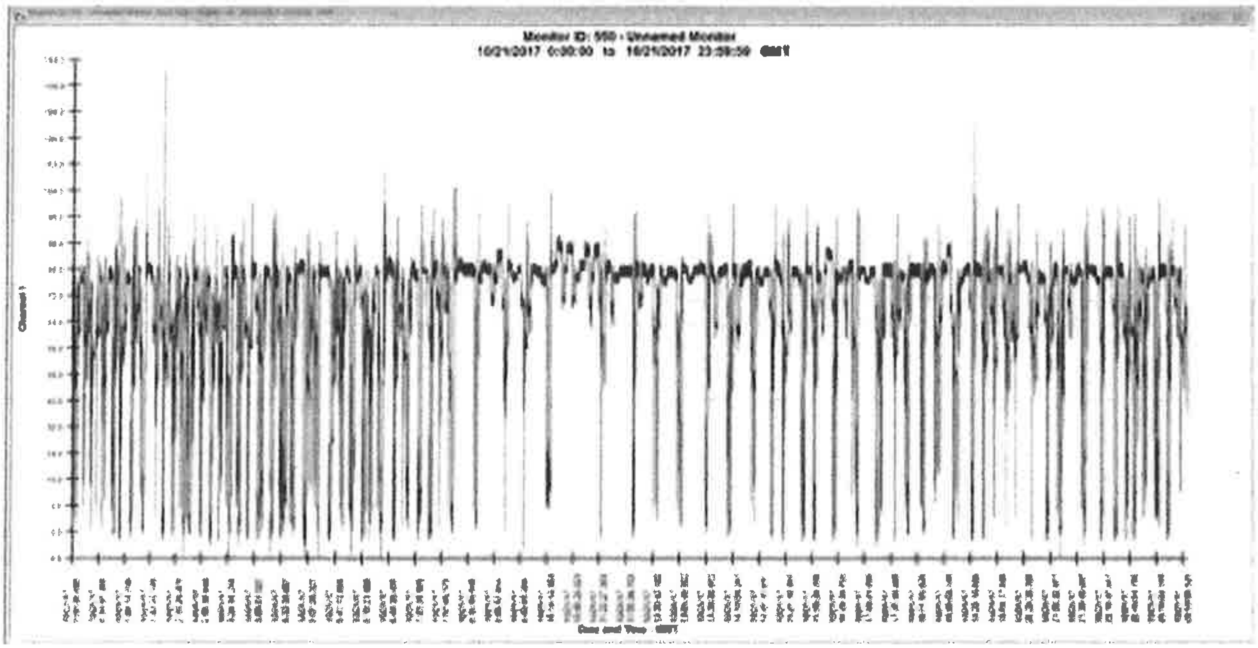


Figure 34: October 22, 2017

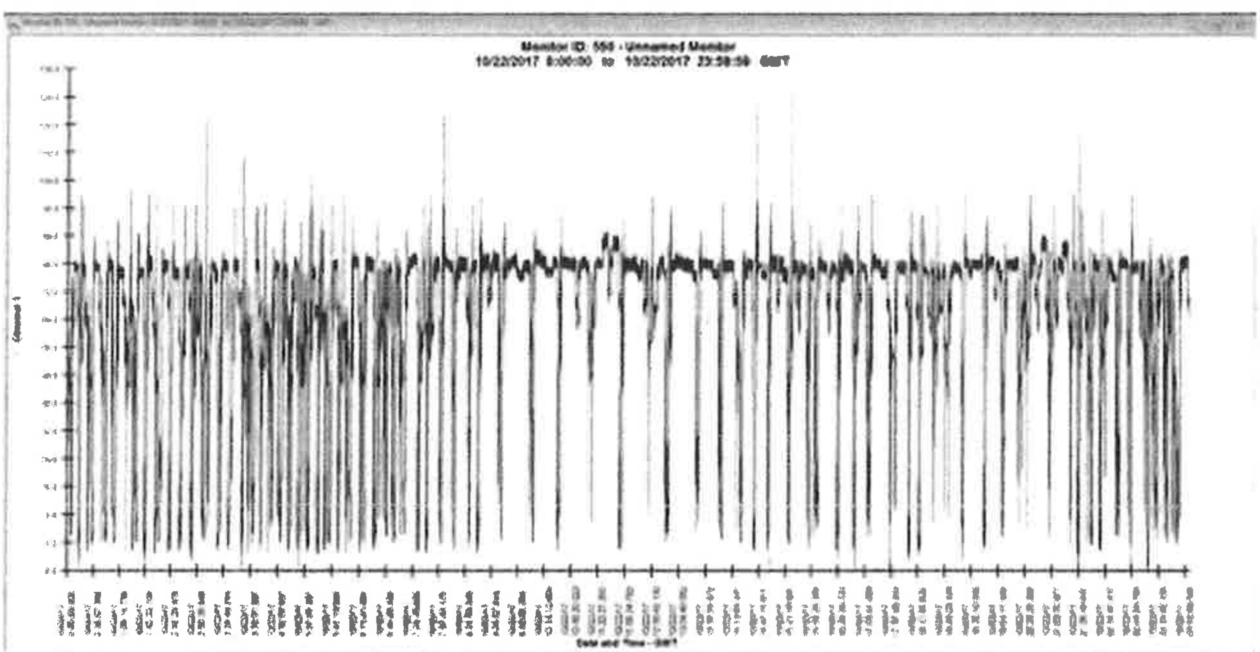


Figure 35: October 23, 2017

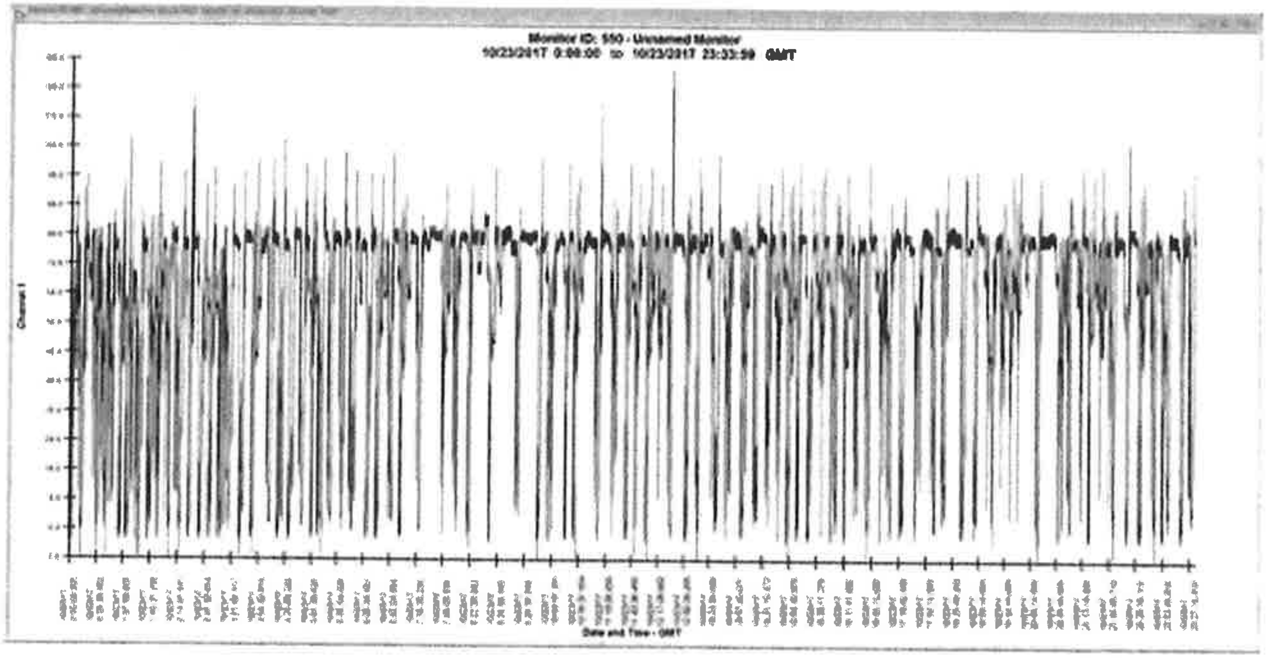


Figure 36: October 24, 2017

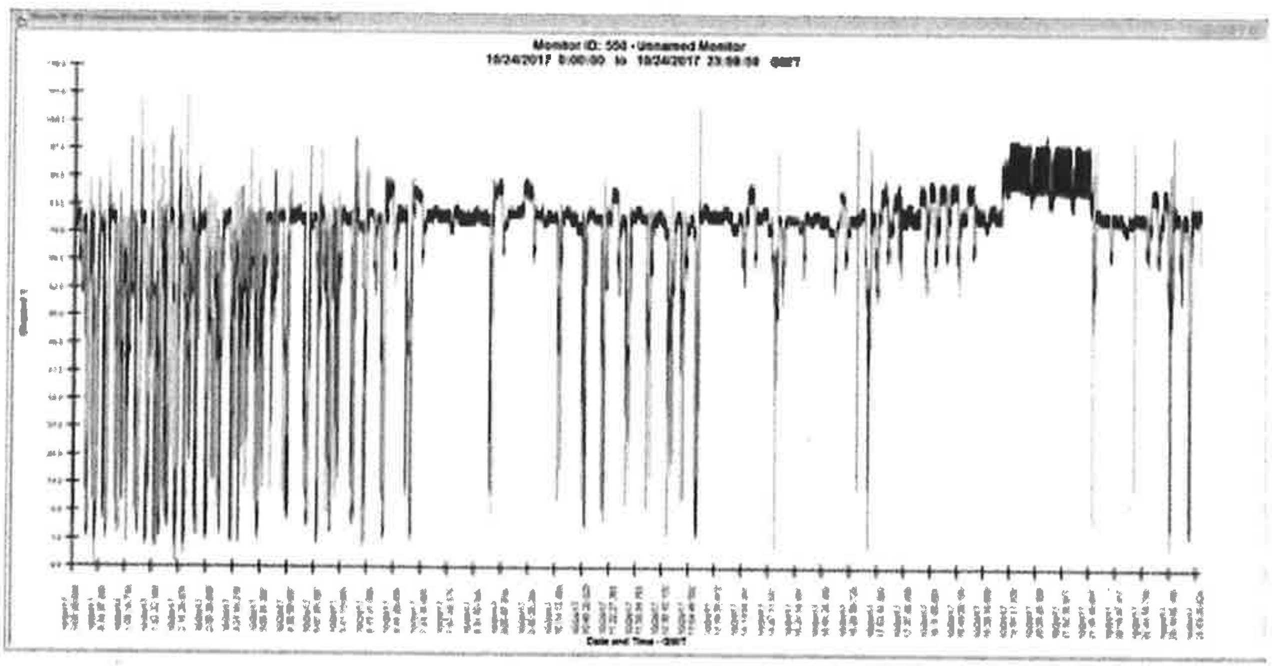
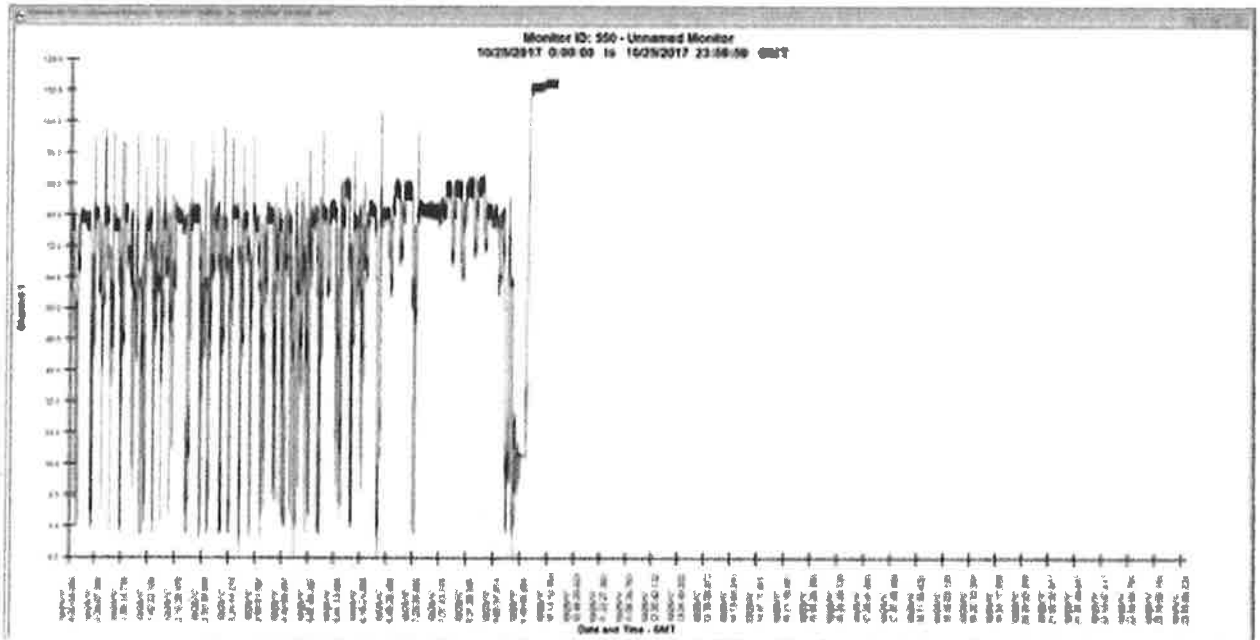


Figure 37: October 25, 2017



D

Appendix D – USDA NRCS Soil Survey

USDA United States Department of Agriculture

NRCS Natural Resources Conservation Service

A product of the National Cooperative Soil Survey, a joint effort of the United States Department of Agriculture and other Federal agencies, State agencies including the Agricultural Experiment Stations, and local participants

Custom Soil Resource Report for

Brevard County, Florida

Riverside Drive Force Main

February 16, 2018

Preface

Soil surveys contain information that affects land use planning in survey areas. They highlight soil limitations that affect various land uses and provide information about the properties of the soils in the survey areas. Soil surveys are designed for many different users, including farmers, ranchers, foresters, agronomists, urban planners, community officials, engineers, developers, builders, and home buyers. Also, conservationists, teachers, students, and specialists in recreation, waste disposal, and pollution control can use the surveys to help them understand, protect, or enhance the environment.

Various land use regulations of Federal, State, and local governments may impose special restrictions on land use or land treatment. Soil surveys identify soil properties that are used in making various land use or land treatment decisions. The information is intended to help the land users identify and reduce the effects of soil limitations on various land uses. The landowner or user is responsible for identifying and complying with existing laws and regulations.

Although soil survey information can be used for general farm, local, and wider area planning, onsite investigation is needed to supplement this information in some cases. Examples include soil quality assessments (http://www.nrcs.usda.gov/wps/portal/nrcs/main/soils/health/) and certain conservation and engineering applications. For more detailed information, contact your local USDA Service Center (https://offices.sc.egov.usda.gov/locator/app?agency=nrcs) or your NRCS State Soil Scientist (http://www.nrcs.usda.gov/wps/portal/nrcs/detail/soils/contactus/?cid=nrcs142p2_053951).

Great differences in soil properties can occur within short distances. Some soils are seasonally wet or subject to flooding. Some are too unstable to be used as a foundation for buildings or roads. Clayey or wet soils are poorly suited to use as septic tank absorption fields. A high water table makes a soil poorly suited to basements or underground installations.

The National Cooperative Soil Survey is a joint effort of the United States Department of Agriculture and other Federal agencies, State agencies including the Agricultural Experiment Stations, and local agencies. The Natural Resources Conservation Service (NRCS) has leadership for the Federal part of the National Cooperative Soil Survey.

Information about soils is updated periodically. Updated information is available through the NRCS Web Soil Survey, the site for official soil survey information.

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Contents

Soil Map

The soil map section includes the soil map for the defined area of interest, a list of soil map units on the map and extent of each map unit, and cartographic symbols displayed on the map. Also presented are various metadata about data used to produce the map, and a description of each soil map unit.

Soil Map

Map Scale: 1:29,300 if printed on A portrait (8.5" x 11") sheet.

N

0 400 800 1600 2400 Meters

0 1000 2000 4000 6000 Feet

Map projection: Web Mercator Corner coordinates: WGS84 Edge tics: UTM Zone 17N WGS84

MAP LEGEND

Area of Interest (AOI)
- Area of Interest (AOI)

Soils
- Soil Map Unit Polygons
- Soil Map Unit Lines
- Soil Map Unit Points

Special Point Features
- Blowout
- Borrow Pit
- Clay Spot
- Closed Depression
- Gravel Pit
- Gravelly Spot
- Landfill
- Lava Flow
- Marsh or swamp
- Mine or Quarry
- Miscellaneous Water
- Perennial Water
- Rock Outcrop
- Saline Spot
- Sandy Spot
- Severely Eroded Spot
- Sinkhole
- Slide or Slip
- Sodic Spot
- Spoil Area
- Stony Spot
- Very Stony Spot
- Wet Spot
- Other
- Special Line Features

Water Features
- Streams and Canals

Transportation
- Rails
- Interstate Highways
- US Routes
- Major Roads
- Local Roads

Background
- Aerial Photography

MAP INFORMATION

The soil surveys that comprise your AOI were mapped at 1:24,000.

Please rely on the bar scale on each map sheet for map measurements.

Source of Map: Natural Resources Conservation Service
Web Soil Survey URL:
Coordinate System: Web Mercator (EPSG:3857)

Maps from the Web Soil Survey are based on the Web Mercator projection, which preserves direction and shape but distorts distance and area. A projection that preserves area, such as the Albers equal-area conic projection, should be used if more accurate calculations of distance or area are required.

This product is generated from the USDA-NRCS certified data as of the version date(s) listed below.

Soil Survey Area: Brevard County, Florida
Survey Area Data: Version 16, Sep 26, 2017

Soil map units are labeled (as space allows) for map scales 1:50,000 or larger.

Date(s) aerial images were photographed: Mar 12, 2011—Mar 16, 2014

The orthophoto or other base map on which the soil lines were compiled and digitized probably differs from the background imagery displayed on these maps. As a result, some minor shifting of map unit boundaries may be evident.

Map Unit Legend

Map Unit Symbol	Map Unit Name	Acres in AOI	Percent of AOI
9	Canaveral-Anclote complex, gently undulating	2.3	0.8%
25	Canaveral-Palm Beach-Urban land complex	27.1	9.4%
36	Myakka sand, 0 to 2 percent slopes	1.6	0.6%
49	Pomello sand	18.2	6.3%
51	Pompano sand, 0 to 2 percent slopes	9.0	3.1%
66	Bessie muck, tidal	0.7	0.3%
69	Urban land	19.4	6.8%
72	Welaka sand	206.5	71.9%
100	Waters of the Atlantic Ocean	2.4	0.8%
Totals for Area of Interest		**287.3**	**100.0%**

Map Unit Descriptions

The map units delineated on the detailed soil maps in a soil survey represent the soils or miscellaneous areas in the survey area. The map unit descriptions, along with the maps, can be used to determine the composition and properties of a unit.

A map unit delineation on a soil map represents an area dominated by one or more major kinds of soil or miscellaneous areas. A map unit is identified and named according to the taxonomic classification of the dominant soils. Within a taxonomic class there are precisely defined limits for the properties of the soils. On the landscape, however, the soils are natural phenomena, and they have the characteristic variability of all natural phenomena. Thus, the range of some observed properties may extend beyond the limits defined for a taxonomic class. Areas of soils of a single taxonomic class rarely, if ever, can be mapped without including areas of other taxonomic classes. Consequently, every map unit is made up of the soils or miscellaneous areas for which it is named and some minor components that belong to taxonomic classes other than those of the major soils.

Most minor soils have properties similar to those of the dominant soil or soils in the map unit, and thus they do not affect use and management. These are called noncontrasting, or similar, components. They may or may not be mentioned in a particular map unit description. Other minor components, however, have properties and behavioral characteristics divergent enough to affect use or to require different management. These are called contrasting, or dissimilar, components. They generally are in small areas and could not be mapped separately because of the scale used. Some small areas of strongly contrasting soils or miscellaneous areas are identified by a special symbol on the maps. If included in the database for a given area, the contrasting minor components are identified in the map unit descriptions along with some characteristics of each. A few areas of minor components may not have been observed, and consequently they are not mentioned in the descriptions, especially where the pattern was so complex that it was impractical to make enough observations to identify all the soils and miscellaneous areas on the landscape.

The presence of minor components in a map unit in no way diminishes the usefulness or accuracy of the data. The objective of mapping is not to delineate pure taxonomic classes but rather to separate the landscape into landforms or landform segments that have similar use and management requirements. The delineation of such segments on the map provides sufficient information for the development of resource plans. If intensive use of small areas is planned, however, onsite investigation is needed to define and locate the soils and miscellaneous areas.

An identifying symbol precedes the map unit name in the map unit descriptions. Each description includes general facts about the unit and gives important soil properties and qualities.

Soils that have profiles that are almost alike make up a *soil series*. Except for differences in texture of the surface layer, all the soils of a series have major horizons that are similar in composition, thickness, and arrangement.

Soils of one series can differ in texture of the surface layer, slope, stoniness, salinity, degree of erosion, and other characteristics that affect their use. On the basis of such differences, a soil series is divided into *soil phases*. Most of the areas shown on the detailed soil maps are phases of soil series. The name of a soil phase commonly indicates a feature that affects use or management. For example, Alpha silt loam, 0 to 2 percent slopes, is a phase of the Alpha series.

Some map units are made up of two or more major soils or miscellaneous areas. These map units are complexes, associations, or undifferentiated groups.

A *complex* consists of two or more soils or miscellaneous areas in such an intricate pattern or in such small areas that they cannot be shown separately on the maps. The pattern and proportion of the soils or miscellaneous areas are somewhat similar in all areas. Alpha-Beta complex, 0 to 6 percent slopes, is an example.

An *association* is made up of two or more geographically associated soils or miscellaneous areas that are shown as one unit on the maps. Because of present or anticipated uses of the map units in the survey area, it was not considered practical or necessary to map the soils or miscellaneous areas separately. The pattern and relative proportion of the soils or miscellaneous areas are somewhat similar. Alpha-Beta association, 0 to 2 percent slopes, is an example.

An *undifferentiated group* is made up of two or more soils or miscellaneous areas that could be mapped individually but are mapped as one unit because similar interpretations can be made for use and management. The pattern and proportion of the soils or miscellaneous areas in a mapped area are not uniform. An area can be made up of only one of the major soils or miscellaneous areas, or it can be made up of all of them. Alpha and Beta soils, 0 to 2 percent slopes, is an example.

Some surveys include *miscellaneous areas*. Such areas have little or no soil material and support little or no vegetation. Rock outcrop is an example.

Brevard County, Florida

9—Canaveral-Anclote complex, gently undulating

Map Unit Setting

National map unit symbol: 1lg2n
Elevation: 10 to 60 feet
Mean annual precipitation: 49 to 57 inches
Mean annual air temperature: 68 to 75 degrees F
Frost-free period: 350 to 365 days
Farmland classification: Not prime farmland

Map Unit Composition

Canaveral and similar soils: 60 percent
Anclote and similar soils: 30 percent
Minor components: 10 percent
Estimates are based on observations, descriptions, and transects of the mapunit.

Description of Canaveral

Setting

Landform: Ridges on marine terraces, dunes on marine terraces
Landform position (three-dimensional): Interfluve
Down-slope shape: Convex
Across-slope shape: Linear
Parent material: Sandy marine deposits

Typical profile

A - 0 to 6 inches: sand
C - 6 to 12 inches: sand
C - 12 to 80 inches: coarse sand

Properties and qualities

Slope: 0 to 5 percent
Depth to restrictive feature: More than 80 inches
Natural drainage class: Somewhat poorly drained
Runoff class: Low
Capacity of the most limiting layer to transmit water (Ksat): Very high (19.98 to 50.02 in/hr)
Depth to water table: About 12 to 36 inches
Frequency of flooding: None
Frequency of ponding: None
Calcium carbonate, maximum in profile: 15 percent
Salinity, maximum in profile: Nonsaline to very slightly saline (0.0 to 2.0 mmhos/cm)
Sodium adsorption ratio, maximum in profile: 6.0
Available water storage in profile: Very low (about 1.4 inches)

Interpretive groups

Land capability classification (irrigated): None specified
Land capability classification (nonirrigated): 6s
Hydrologic Soil Group: A/D
Other vegetative classification: Forage suitability group not assigned (G156BC999FL)
Hydric soil rating: No

Description of Anclote

Setting

Landform: Flats on marine terraces
Landform position (three-dimensional): Talf
Down-slope shape: Linear
Across-slope shape: Linear
Parent material: Sandy marine deposits

Typical profile

A - 0 to 19 inches: sand
Cg - 19 to 72 inches: sand

Properties and qualities

Slope: 0 to 2 percent
Depth to restrictive feature: More than 80 inches
Natural drainage class: Very poorly drained
Runoff class: Very high
Capacity of the most limiting layer to transmit water (Ksat): High to very high (5.95 to 19.98 in/hr)
Depth to water table: About 0 to 6 inches
Frequency of flooding: None
Frequency of ponding: None
Salinity, maximum in profile: Nonsaline to very slightly saline (0.0 to 2.0 mmhos/cm)
Sodium adsorption ratio, maximum in profile: 4.0
Available water storage in profile: Low (about 5.3 inches)

Interpretive groups

Land capability classification (irrigated): None specified
Land capability classification (nonirrigated): 3w
Hydrologic Soil Group: A/D
Other vegetative classification: Sandy soils on flats of mesic or hydric lowlands (G156BC141FL)
Hydric soil rating: Yes

Minor Components

Pomello

Percent of map unit: 5 percent
Landform: Rises on marine terraces, flats on marine terraces
Landform position (three-dimensional): Interfluve
Down-slope shape: Convex
Across-slope shape: Linear
Other vegetative classification: Sand Pine Scrub (R155XY001FL), Sandy soils on rises and knolls of mesic uplands (G156BC131FL)
Hydric soil rating: No

Palm beach

Percent of map unit: 5 percent
Landform: Dunes on marine terraces
Landform position (three-dimensional): Interfluve
Down-slope shape: Convex
Across-slope shape: Linear
Other vegetative classification: Sandy soils on ridges and dunes of xeric uplands (G156BC111FL)

Hydric soil rating: No

25—Canaveral-Palm Beach-Urban land complex

Map Unit Setting

National map unit symbol: 1lg35
Elevation: 10 to 20 feet
Mean annual precipitation: 49 to 57 inches
Mean annual air temperature: 68 to 75 degrees F
Frost-free period: 350 to 365 days
Farmland classification: Not prime farmland

Map Unit Composition

Canaveral and similar soils: 31 percent
Palm beach and similar soils: 30 percent
Urban land: 29 percent
Minor components: 10 percent
Estimates are based on observations, descriptions, and transects of the mapunit.

Description of Canaveral

Setting

Landform: Flats on marine terraces, ridges on marine terraces
Landform position (three-dimensional): Interfluve
Down-slope shape: Convex
Across-slope shape: Linear
Parent material: Sandy marine deposits

Typical profile

A - 0 to 6 inches: sand
C - 6 to 12 inches: sand
C - 12 to 80 inches: coarse sand

Properties and qualities

Slope: 0 to 2 percent
Depth to restrictive feature: More than 80 inches
Natural drainage class: Somewhat poorly drained
Runoff class: Low
Capacity of the most limiting layer to transmit water (Ksat): Very high (19.98 to 50.02 in/hr)
Depth to water table: About 12 to 36 inches
Frequency of flooding: None
Frequency of ponding: None
Calcium carbonate, maximum in profile: 15 percent
Salinity, maximum in profile: Nonsaline to very slightly saline (0.0 to 2.0 mmhos/cm)
Sodium adsorption ratio, maximum in profile: 6.0
Available water storage in profile: Very low (about 1.4 inches)

Interpretive groups

Land capability classification (irrigated): None specified
Land capability classification (nonirrigated): 6s
Hydrologic Soil Group: A/D
Other vegetative classification: Forage suitability group not assigned (G156BC999FL)
Hydric soil rating: No

Description of Palm Beach

Setting

Landform: Flats on marine terraces
Landform position (three-dimensional): Talf
Down-slope shape: Convex
Across-slope shape: Linear
Parent material: Shells and sandy marine deposits

Typical profile

A - 0 to 3 inches: sand
C - 3 to 80 inches: sand

Properties and qualities

Slope: 0 to 2 percent
Depth to restrictive feature: More than 80 inches
Natural drainage class: Excessively drained
Runoff class: Negligible
Capacity of the most limiting layer to transmit water (Ksat): Very high (19.98 to 50.02 in/hr)
Depth to water table: More than 80 inches
Frequency of flooding: None
Frequency of ponding: None
Calcium carbonate, maximum in profile: 30 percent
Salinity, maximum in profile: Nonsaline to very slightly saline (0.0 to 2.0 mmhos/cm)
Sodium adsorption ratio, maximum in profile: 4.0
Available water storage in profile: Very low (about 1.8 inches)

Interpretive groups

Land capability classification (irrigated): None specified
Land capability classification (nonirrigated): 7s
Hydrologic Soil Group: A
Other vegetative classification: Forage suitability group not assigned (G156BC999FL)
Hydric soil rating: No

Description of Urban Land

Setting

Landform: Marine terraces
Landform position (three-dimensional): Interfluve, talf
Down-slope shape: Linear
Across-slope shape: Linear
Parent material: No parent material

Interpretive groups

Land capability classification (irrigated): None specified

Other vegetative classification: Forage suitability group not assigned (G156BC999FL)
Hydric soil rating: Unranked

Minor Components

Paola

Percent of map unit: 4 percent
Landform: Flats on marine terraces, rises on marine terraces
Landform position (three-dimensional): Interfluve
Down-slope shape: Convex
Across-slope shape: Linear
Other vegetative classification: Sand Pine Scrub (R155XY001FL), Forage suitability group not assigned (G156BC999FL)
Hydric soil rating: No

Pomello

Percent of map unit: 3 percent
Landform: Flats on marine terraces, rises on marine terraces
Landform position (three-dimensional): Interfluve
Down-slope shape: Convex
Across-slope shape: Linear
Other vegetative classification: Sand Pine Scrub (R155XY001FL), Forage suitability group not assigned (G156BC999FL)
Hydric soil rating: No

Welaka

Percent of map unit: 3 percent
Landform: Rises on marine terraces
Landform position (three-dimensional): Interfluve
Down-slope shape: Convex
Across-slope shape: Linear
Other vegetative classification: Sand Pine Scrub (R155XY001FL), Forage suitability group not assigned (G156BC999FL)
Hydric soil rating: No

36—Myakka sand, 0 to 2 percent slopes

Map Unit Setting

National map unit symbol: 2twt9
Elevation: 10 to 130 feet
Mean annual precipitation: 43 to 62 inches
Mean annual air temperature: 64 to 77 degrees F
Frost-free period: 280 to 365 days
Farmland classification: Farmland of unique importance

Map Unit Composition

Myakka and similar soils: 85 percent

Minor components: 15 percent
Estimates are based on observations, descriptions, and transects of the mapunit.

Description of Myakka

Setting

Landform: Flatwoods on marine terraces
Landform position (three-dimensional): Tread, talf
Down-slope shape: Convex
Across-slope shape: Linear
Parent material: Sandy marine deposits

Typical profile

A - 0 to 6 inches: sand
E - 6 to 20 inches: sand
Bh - 20 to 36 inches: sand
C - 36 to 80 inches: sand

Properties and qualities

Slope: 0 to 2 percent
Depth to restrictive feature: More than 80 inches
Natural drainage class: Poorly drained
Runoff class: High
Capacity of the most limiting layer to transmit water (Ksat): Moderately high to high (0.57 to 5.95 in/hr)
Depth to water table: About 6 to 18 inches
Frequency of flooding: None
Frequency of ponding: None
Salinity, maximum in profile: Nonsaline to very slightly saline (0.0 to 2.0 mmhos/cm)
Sodium adsorption ratio, maximum in profile: 4.0
Available water storage in profile: Low (about 5.0 inches)

Interpretive groups

Land capability classification (irrigated): None specified
Land capability classification (nonirrigated): 4w
Hydrologic Soil Group: A/D
Other vegetative classification: South Florida Flatwoods (R155XY003FL), Sandy soils on flats of mesic or hydric lowlands (G155XB141FL)
Hydric soil rating: No

Minor Components

Basinger

Percent of map unit: 5 percent
Landform: Drainageways on marine terraces
Landform position (three-dimensional): Dip
Down-slope shape: Convex, linear
Across-slope shape: Linear, concave
Other vegetative classification: Sandy soils on flats of mesic or hydric lowlands (G155XB141FL)
Hydric soil rating: Yes

Oldsmar

Percent of map unit: 5 percent
Landform: Flatwoods on marine terraces
Landform position (three-dimensional): Talf

Down-slope shape: Convex
Across-slope shape: Linear
Other vegetative classification: South Florida Flatwoods (R155XY003FL), Sandy soils on flats of mesic or hydric lowlands (G155XB141FL)
Hydric soil rating: No

Valkaria

Percent of map unit: 5 percent
Landform: Drainageways on marine terraces
Landform position (three-dimensional): Dip
Down-slope shape: Convex, linear
Across-slope shape: Linear, concave
Other vegetative classification: Slough (R155XY011FL), Sandy soils on flats of mesic or hydric lowlands (G155XB141FL)
Hydric soil rating: Yes

49—Pomello sand

Map Unit Setting

National map unit symbol: 1lg3y
Mean annual precipitation: 49 to 57 inches
Mean annual air temperature: 68 to 75 degrees F
Frost-free period: 350 to 365 days
Farmland classification: Not prime farmland

Map Unit Composition

Pomello and similar soils: 90 percent
Minor components: 10 percent
Estimates are based on observations, descriptions, and transects of the mapunit.

Description of Pomello

Setting

Landform: Rises on marine terraces, flats on marine terraces
Landform position (three-dimensional): Interfluve
Down-slope shape: Convex
Across-slope shape: Linear
Parent material: Sandy marine deposits

Typical profile

A - 0 to 3 inches: sand
E - 3 to 50 inches: sand
Bh - 50 to 62 inches: sand
Cg - 62 to 80 inches: sand

Properties and qualities

Slope: 0 to 2 percent
Depth to restrictive feature: More than 80 inches
Natural drainage class: Moderately well drained
Runoff class: Negligible

Capacity of the most limiting layer to transmit water (Ksat): High (1.98 to 5.95 in/hr)
Depth to water table: About 24 to 42 inches
Frequency of flooding: None
Frequency of ponding: None
Salinity, maximum in profile: Nonsaline to very slightly saline (0.0 to 2.0 mmhos/cm)
Sodium adsorption ratio, maximum in profile: 4.0
Available water storage in profile: Low (about 3.4 inches)

Interpretive groups
Land capability classification (irrigated): None specified
Land capability classification (nonirrigated): 6s
Hydrologic Soil Group: A
Other vegetative classification: Sand Pine Scrub (R155XY001FL), Sandy soils on rises and knolls of mesic uplands (G156BC131FL)
Hydric soil rating: No

Minor Components

Myakka
Percent of map unit: 5 percent
Landform: Flats on marine terraces
Landform position (three-dimensional): Talf
Down-slope shape: Convex
Across-slope shape: Linear
Other vegetative classification: South Florida Flatwoods (R155XY003FL), Sandy soils on flats of mesic or hydric lowlands (G156BC141FL)
Hydric soil rating: No

Immokalee
Percent of map unit: 5 percent
Landform: Flats on marine terraces
Landform position (three-dimensional): Talf
Down-slope shape: Convex
Across-slope shape: Linear
Other vegetative classification: South Florida Flatwoods (R155XY003FL), Sandy soils on flats of mesic or hydric lowlands (G156BC141FL)
Hydric soil rating: No

51—Pompano sand, 0 to 2 percent slopes

Map Unit Setting
National map unit symbol: 2tzw4
Elevation: 0 to 40 feet
Mean annual precipitation: 44 to 58 inches
Mean annual air temperature: 68 to 77 degrees F
Frost-free period: 350 to 365 days
Farmland classification: Not prime farmland

Map Unit Composition

Pompano and similar soils: 82 percent
Minor components: 18 percent
Estimates are based on observations, descriptions, and transects of the mapunit.

Description of Pompano

Setting

Landform: Drainageways on flatwoods on marine terraces
Landform position (three-dimensional): Tread, dip, talf
Down-slope shape: Linear
Across-slope shape: Linear, concave
Parent material: Sandy marine deposits

Typical profile

A - 0 to 6 inches: sand
C - 6 to 80 inches: sand

Properties and qualities

Slope: 0 to 2 percent
Depth to restrictive feature: More than 80 inches
Natural drainage class: Poorly drained
Runoff class: Very high
Capacity of the most limiting layer to transmit water (Ksat): High to very high (6.00 to 20.00 in/hr)
Depth to water table: About 0 to 12 inches
Frequency of flooding: None
Frequency of ponding: None
Salinity, maximum in profile: Nonsaline to very slightly saline (0.0 to 2.0 mmhos/cm)
Sodium adsorption ratio, maximum in profile: 4.0
Available water storage in profile: Very low (about 2.4 inches)

Interpretive groups

Land capability classification (irrigated): None specified
Land capability classification (nonirrigated): 4w
Hydrologic Soil Group: A/D
Other vegetative classification: Slough (R155XY011FL), Sandy soils on flats of mesic or hydric lowlands (G155XB141FL)
Hydric soil rating: Yes

Minor Components

Myakka

Percent of map unit: 8 percent
Landform: Drainageways on flatwoods on marine terraces
Landform position (three-dimensional): Tread, dip, talf
Down-slope shape: Linear
Across-slope shape: Concave, linear
Other vegetative classification: South Florida Flatwoods (R155XY003FL), Sandy soils on flats of mesic or hydric lowlands (G155XB141FL)
Hydric soil rating: No

Holopaw

Percent of map unit: 4 percent
Landform: — error in exists on —

Landform position (three-dimensional): Tread, dip, talf
Down-slope shape: Convex, concave, linear
Across-slope shape: Linear, concave
Other vegetative classification: Slough (R155XY011FL), Sandy soils on flats of mesic or hydric lowlands (G155XB141FL)
Hydric soil rating: No

Hallandale

Percent of map unit: 4 percent
Landform: Flatwoods on marine terraces
Landform position (three-dimensional): Tread, talf
Down-slope shape: Linear
Across-slope shape: Linear
Other vegetative classification: South Florida Flatwoods (R155XY003FL), Sandy soils on flats of mesic or hydric lowlands (G155XB141FL)
Hydric soil rating: Yes

Samsula

Percent of map unit: 2 percent
Landform: Depressions on marine terraces
Landform position (three-dimensional): Tread, dip
Down-slope shape: Concave
Across-slope shape: Concave
Other vegetative classification: Freshwater Marshes and Ponds (R155XY010FL), Organic soils in depressions and on flood plains (G155XB645FL)
Hydric soil rating: Yes

66—Bessie muck, tidal

Map Unit Setting

National map unit symbol: 1lg4g
Mean annual precipitation: 49 to 57 inches
Mean annual air temperature: 68 to 75 degrees F
Frost-free period: 350 to 365 days
Farmland classification: Not prime farmland

Map Unit Composition

Bessie, tidal, and similar soils: 100 percent
Estimates are based on observations, descriptions, and transects of the mapunit.

Description of Bessie, Tidal

Setting

Landform: Tidal marshes on marine terraces
Landform position (three-dimensional): Dip
Down-slope shape: Linear
Across-slope shape: Concave
Parent material: Organic material over clayey and sandy marine deposits

Typical profile

Oa - 0 to 18 inches: muck

2Cg - 18 to 44 inches: sandy clay
3Cg - 44 to 80 inches: loamy fine sand

Properties and qualities

Slope: 0 to 1 percent
Depth to restrictive feature: More than 80 inches
Natural drainage class: Very poorly drained
Runoff class: Very high
Capacity of the most limiting layer to transmit water (Ksat): Moderately low to moderately high (0.06 to 0.20 in/hr)
Depth to water table: About 0 inches
Frequency of flooding: Very frequent
Frequency of ponding: Frequent
Calcium carbonate, maximum in profile: 20 percent
Salinity, maximum in profile: Strongly saline (16.0 to 32.0 mmhos/cm)
Sodium adsorption ratio, maximum in profile: 60.0
Available water storage in profile: High (about 11.1 inches)

Interpretive groups

Land capability classification (irrigated): None specified
Land capability classification (nonirrigated): 8
Hydrologic Soil Group: C/D
Other vegetative classification: Forage suitability group not assigned (G156BC999FL)
Hydric soil rating: Yes

69—Urban land

Map Unit Composition

Urban land: 100 percent
Estimates are based on observations, descriptions, and transects of the mapunit.

Description of Urban Land

Setting

Landform: Marine terraces
Landform position (three-dimensional): Interfluve, talf
Down-slope shape: Linear
Across-slope shape: Linear
Parent material: No parent material

Interpretive groups

Land capability classification (irrigated): None specified
Other vegetative classification: Forage suitability group not assigned (G156BC999FL)
Hydric soil rating: Unranked

72—Welaka sand

Map Unit Setting

National map unit symbol: 1lg4n
Mean annual precipitation: 49 to 57 inches
Mean annual air temperature: 68 to 75 degrees F
Frost-free period: 350 to 365 days
Farmland classification: Not prime farmland

Map Unit Composition

Welaka and similar soils: 90 percent
Minor components: 10 percent
Estimates are based on observations, descriptions, and transects of the mapunit.

Description of Welaka

Setting

Landform: Rises on marine terraces
Landform position (three-dimensional): Interfluve
Down-slope shape: Convex
Across-slope shape: Linear
Parent material: Sandy marine deposits

Typical profile

A - 0 to 3 inches: sand
E - 3 to 18 inches: sand
Bw - 18 to 55 inches: sand
2C - 55 to 80 inches: gravelly fine sand

Properties and qualities

Slope: 0 to 2 percent
Depth to restrictive feature: More than 80 inches
Natural drainage class: Well drained
Runoff class: Negligible
Capacity of the most limiting layer to transmit water (Ksat): Very high (19.98 to 50.02 in/hr)
Depth to water table: More than 80 inches
Frequency of flooding: None
Frequency of ponding: None
Calcium carbonate, maximum in profile: 5 percent
Salinity, maximum in profile: Nonsaline to very slightly saline (0.0 to 2.0 mmhos/cm)
Sodium adsorption ratio, maximum in profile: 4.0
Available water storage in profile: Very low (about 2.0 inches)

Interpretive groups

Land capability classification (irrigated): None specified
Land capability classification (nonirrigated): 6s
Hydrologic Soil Group: A
Other vegetative classification: Sand Pine Scrub (R155XY001FL), Sandy soils on ridges and dunes of xeric uplands (G156BC111FL)
Hydric soil rating: No

Minor Components

Pomello

Percent of map unit: 5 percent
Landform: Flats on marine terraces, rises on marine terraces
Landform position (three-dimensional): Interfluve
Down-slope shape: Convex
Across-slope shape: Linear
Other vegetative classification: Sand Pine Scrub (R155XY001FL), Sandy soils on rises and knolls of mesic uplands (G156BC131FL)
Hydric soil rating: No

Paola

Percent of map unit: 5 percent
Landform: Flats on marine terraces, rises on marine terraces
Landform position (three-dimensional): Interfluve
Down-slope shape: Convex
Across-slope shape: Linear
Other vegetative classification: Sand Pine Scrub (R155XY001FL), Sandy soils on ridges and dunes of xeric uplands (G156BC111FL)
Hydric soil rating: No

100—Waters of the Atlantic Ocean

Map Unit Composition

Waters of the atlantic ocean: 100 percent
Estimates are based on observations, descriptions, and transects of the mapunit.

Description of Waters Of The Atlantic Ocean

Interpretive groups

Land capability classification (irrigated): None specified
Other vegetative classification: Forage suitability group not assigned (G156BC999FL)
Hydric soil rating: Unranked

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Brevard County Utility Services

WASTEWATER FACILITY PLAN

RIVERSIDE DRIVE FORCE MAIN REPLACEMENT
SRF# WW05113

MARCH 2018

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1 Executive Summary

The Brevard County Utility Services Department (USD) provides wastewater service to both residential and commercial customers throughout the County's service area. The existing Riverside Drive force main includes approximately three miles of the county's primary transmission main to the South Beaches Regional Wastewater Treatment Plant.

The proposed force main replacement project is needed to eliminate the repeated force main breaks that have been occurring along this stretch of pipeline and to satisfy the current Florida Department of Environmental Protection (FDEP) consent order that has been issued against the county for these pipe breaks.

The force main replacement project will allow the county to resolve critical failures within their sewer transmission system and continue to provide reliable wastewater collection and treatment to the county's service area.

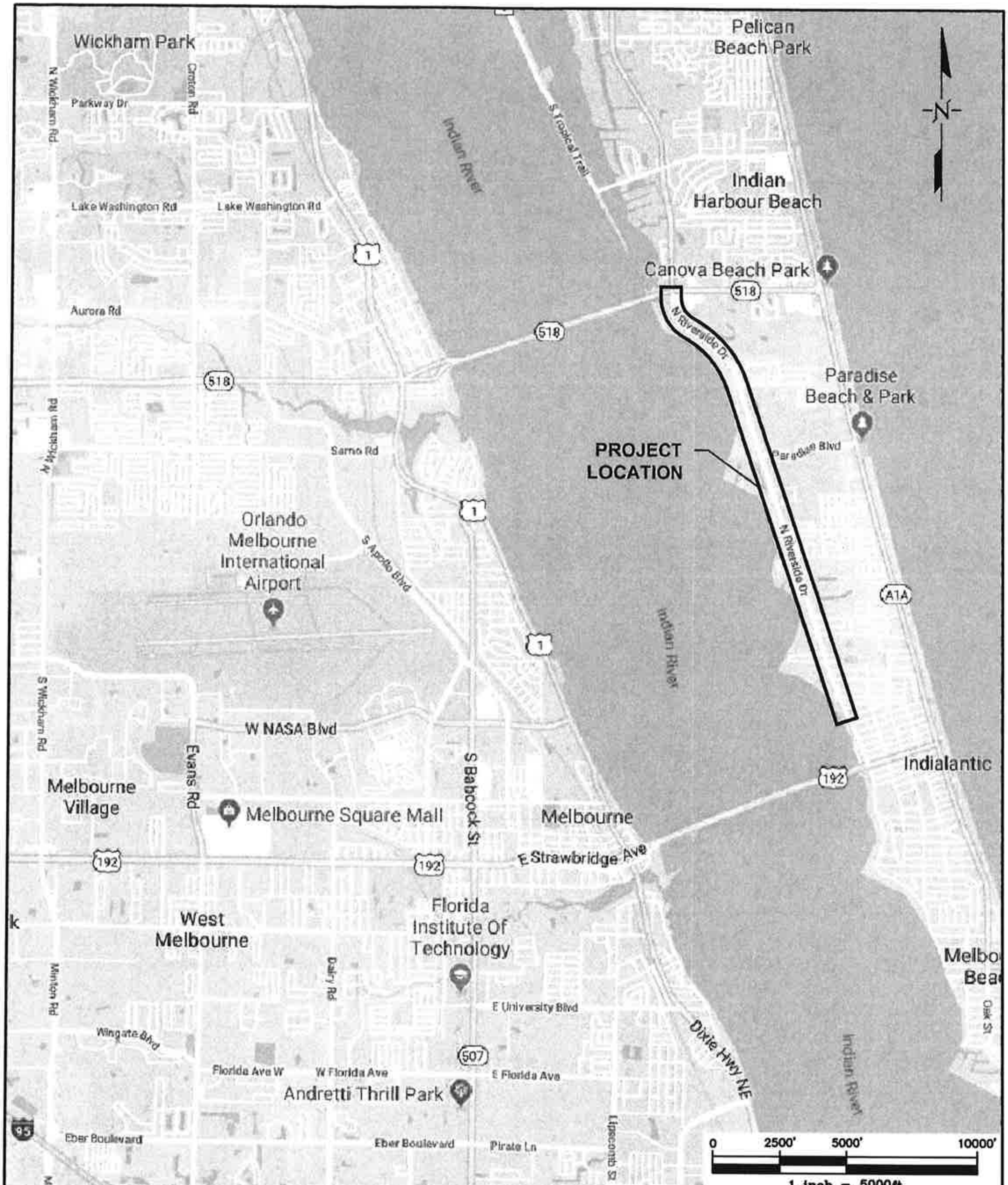
This document demonstrates that the construction and operation of the proposed improvements are in the public interest and will not cause adverse health and environmental impacts. Terrestrial and aquatic communities will not be negatively affected. Threatened or endangered species will be unaffected. This project will not impact any known archeologically sensitive areas. Construction for the new force main will be limited to FDOT and county right-of-way, and will follow the existing force main alignment to minimize any additional conflicts. Furthermore, there are no environmental justice concerns with this project.

The County is familiar with the requirements of the State Revolving Fund (SRF) Program, and has previously gone through the SRF process. They understand the program requirements and will fully comply with the planning and design submittal guidelines.

1.1 Project Description and Location

The County's South Beaches Force Main System consists of roughly 13 miles of 20" and 24" force main that provides collection and transmission from over forty lift stations throughout the South Patrick Shores and Melbourne area, and ultimately terminates at the South Beaches Regional Wastewater Treatment Plant. The force main system is comprised of a combination of ductile iron and polyvinyl-chloride (PVC) pipe. The two sections of PVC force main within this pipe system have been identified as needing replacement. The first segment of PVC force main replacement (1.0 mile long) is from DeSoto Parkway to Banana River Drive and is currently being replaced, with construction expected to be complete by June of 2018. The second area of force main replacement (3.3 miles long) will be the focus of this facility plan and is referred to as the Riverside Drive Force Main replacement project. The Riverside Drive force main segment provides service to roughly 44,000 residential and commercial customers throughout the area. **Figures 1-1** and **1-2** depict the overall location of the project, the existing system limits, and the proposed project layout.

The Riverside Drive Force Main Replacement project starts at Eau Gallie Boulevard and runs south along Riverside Drive all the way to the intersection of Oakland Avenue. The project will include the installation of roughly 3.3 miles of new ductile iron force main, isolation valves, air release valves, pipe fittings, reconnection of existing lift stations within the project area, and the removal/abandonment of the existing 24" PVC force main.



HDR	PROJECT TITLE RIVERSIDE DRIVE FORCE MAIN REPLACEMENT	PROJECT NUMBER 10092206	
	SHEET TITLE **FIGURE:1-1** PROJECT LOCATION MAP	PROJECT MANAGER H.HARDY	EXHIBIT SCALE 1"=5000'
		DATE 03/2018	EXHIBIT NUMBER 1-1

1.2 Justification for Project

The force main replacement project discussed within this document is needed for the following reasons listed below.

- To resolve the recurring pipe breaks in the existing force main
- To eliminate the resultant sewage spills related to pipe failures
- To satisfy the current FDEP consent order against the Brevard County USD

1.2.1 Force Main Breaks and Failures

The South Beaches Force Main System has experienced a significant number of pipe failures over the last few years. The force main failures documented have all been concentrated in the areas where the pipe material is PVC. Various studies have been done on these sections of piping to determine the cause of the pipe failures. **Table 1-1** shows a summary of the pipe failure by type that have occurred to date. The findings from the studies that have been done show that the pipeline was not installed properly due to over deflection of the joints, a lack of pipe restraints, and improper compaction of pipe backfill. In addition, a surge analysis study showed both negative system pressures and pressure surges in the system that would cause weak points in the system to fail. Due to the number of pipe failures and the findings from the force main studies, the complete replacement of the remaining PVC pipe from Eau Gallie Blvd to Oakland Avenue is recommended. **Figures 1-3** and **1-4** show the pipe failure locations within the limits of the Riverside Drive Force Main Replacement project.

Table 1-1: South Beaches Force Main Failures by Type

Type of Break	Occurrences
Bell Split	10
Leak	11
Joint Separation	2
Other	3

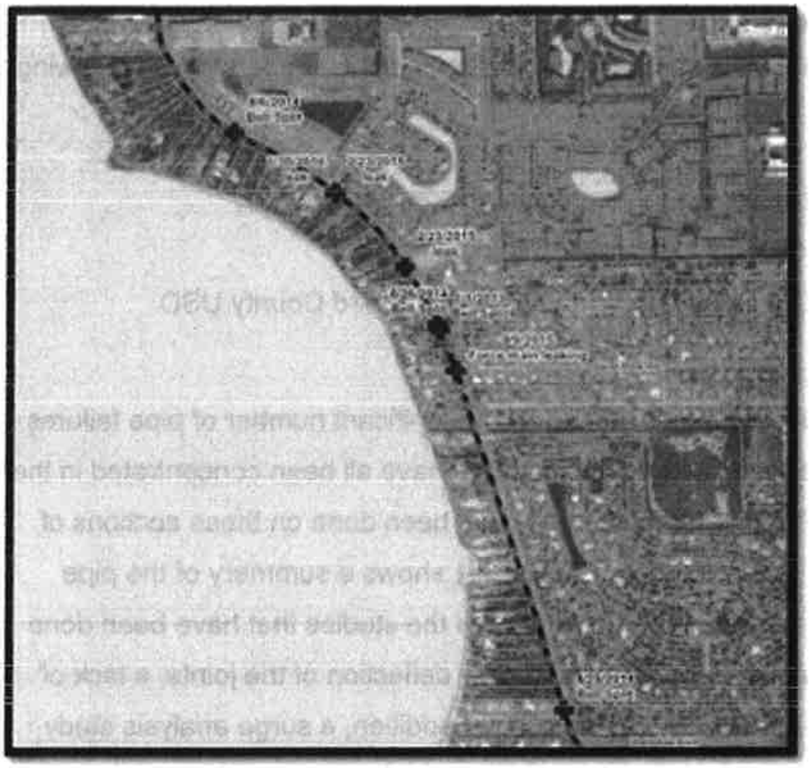


Figure 1-3: Force Main Breaks: Eau Gallie Blvd to Paradise Blvd

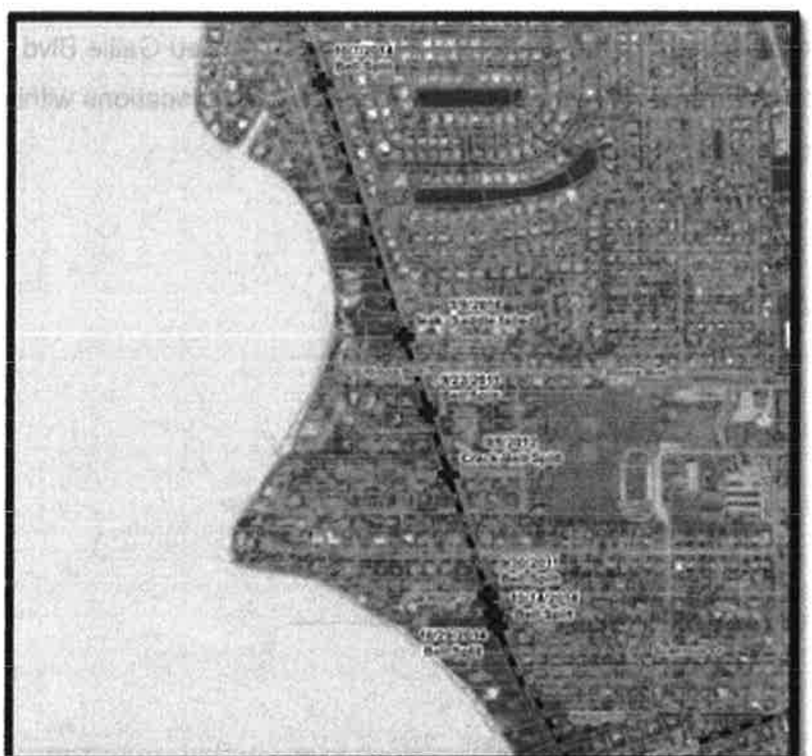


Figure 1-4: Force Main Breaks: Paradise Blvd to Oakland Avenue

1.2.2 Eliminate Sanitary Sewer Spills

The South Beaches Force Main system parallels thirteen miles along the Indian River Lagoon. The Indian River Lagoon is classified under the Florida Administrative Code (FAC) 62-302.700 as an Outstanding Florida Water (OFW) and the Brevard County USD is committed to affording the highest level of protection. Due to the proximity of this force main route to the Indian River Lagoon, any pipe failures resulting in a sanitary sewer overflow (SSO) would pose a direct impact to the lagoon. In addition to the close proximity to the lagoon, the force main system crosses multiple canals and navigable waterways that are directly connected to the Indian River Lagoon. Any pipe failures at these water crossings would result in a direct sewage discharge into the lagoon. By replacing the last section PVC force Main in the system along Riverside Drive, the USD would be able to greatly reduce the risk of future SSO's in the South Beaches area.

1.2.3 Satisfy FDEP Consent Order

In November 2016, the Florida Department of Environmental Protection issued a Consent Order related to sewage spills resulting from these force main breaks, which can be found in **Appendix A**. Included in the Consent Order is the requirement that Brevard County USD conduct a study to determine the cause of the failures within the force main system, and put a plan into place to address the issues. The USD completed a force main system study (**Appendix B**) and a surge analysis study (**Appendix C**), and as a result of the recommendations from these studies the USD determined that the defective pipeline should be replaced. Therefore, to satisfy the current FDEP consent order, the County is required to replace the defective pipe sections as per the recommendations of the South Beaches Force Main Study.

2 Alternatives Comparison

2.1 Force Main Replacement Alternatives

Due to the frequent pipe failures that have occurred over the last few years, the potential public and environmental risks due to SSO's, and the current FDEP consent order against the county, it is evident that action must be taken to address this issue. The following alternatives are evaluated herein:

- Point Repair Existing 24" PVC Force Main
- Complete Replacement with 24" Ductile Iron (DI) Force Main
- Complete Replacement with 30" Ductile Iron (DI) Force Main with Surge Tanks

A comparison of alternatives is based on project cost, system reliability, environmental impact, and future capacity/life. Each of the alternatives are evaluated and ranked form lowest to highest, 1 to 3. The points are totaled and the highest point total is the recommended alternative.

Lowest cost projects have the highest ranking. The most reliable options and the options with least environmental impacts also receive priority ranking. Finally, those options with the most benefits for future lifespan/capacity receive the highest ranking.

2.1.1 Point Repair Existing 24" PVC Force Main

The first alternative is that Brevard County USD can elect to leave the existing 24" PVC force main in place, and perform point repairs based on suspected problems within the system. This option would require the County to develop a list of repairs and improvements based on past pipe failure patterns, information provided from previous system evaluations/studies, and based on additional exploratory efforts. Improvements would include installing joint restraints on the existing force main, replacing pipe segments in areas with multiple failures, replacing pipe segments with excessive joint deflection or settlement, and replacing the segment of pipe under the canal. The primary benefits of this alternative is lower construction costs, and less impact/disruption due to limited pipe replacement. In **Table 2-1**, the conceptual cost estimate for this alternative is roughly $2.1 million. There are several significant disadvantages to this option, which include poor reliability, continued risk to the public/environment, unaddressed system pressure issues, and limited long term lifespan. Since the feasibility of locating and repairing every weak point in this system is not practical, the assumption would have to be made the pipe failures will continue to occur periodically into the foreseeable future. An assumption of two breaks per year, for a period of ten years, was made for the purposes of the cost estimate. In addition each pipe failure that occurs poses a potential risk to both the public health and the surrounding environment, including the Indian River Lagoon. Finally, the long-term lifespan of a pipe system that has been patched in multiple places is not sustainable, and the system would eventually need to be replaced.

Table 2-1: Conceptual Cost Estimate-Point Repair Existing 24" Force Main

ITEM 1 - GENERAL REQUIREMENTS					
1	Mobilization/Demobilization/General Conditions for Infrastructure Imporvements	1	LS	$30,000.00	$30,000
2	Maintenance of Traffic	1	LS	$30,000.00	$30,000
3	Bonds and Insurance Requirements	1	LS	$5,000.00	$5,000
4	Force Main Locating Services	1	LS	$10,000.00	$10,000
5	Professional Videotaping of Site Conditions (Pre/Post Construction)	1	LS	$5,000.00	$5,000
6	Contingency	(20% of Item 2-General Sitework Subtotal)			$339,950
				SUBTOTAL	***$419,950***
ITEM 2 - GENERAL SITEWORK					
7	Remove/Replace 24" PVC Force Main	2655	LF	$150.00	$398,250
8	Post Restrain 24" PVC Force Main	100	EA	$800.00	$80,000
9	Replace 24" Canal Crossing (HDD)	1	LS	$85,000.00	$85,000
10	Utility Fitting - Ductile Iron Elbow - 24"	15	EA	$7,000.00	$105,000
11	Point Repairs (2 per year, for 10 Years)	20	EA	$40,000.00	$800,000
12	Remove and Replace Concrete Sidewalk	1000	SF	$25.00	$25,000
13	Base and Asphalt Trench Repair	800	SY	$50.00	$40,000
14	Asphalt Milling & Overlay	1000	SY	$15.00	$15,000
15	Remove and Replace Concrete Driveway Apron	500	SY	$60.00	$30,000
16	Pavement Striping	1	LS	$5,000.00	$5,000
17	Remove and replace Unsuitable Materials	300	CY	$30.00	$9,000
18	Seeding	2500	SY	$2.00	$5,000
19	Sodding	2500	SY	$5.00	$12,500
20	Landscaping	1	LS	$10,000.00	$10,000
21	Pressure Testing	1	LS	$5,000.00	$5,000
22	Density Testing	1	LS	$25,000.00	$25,000
23	Dewatering	1	LS	$50,000.00	$50,000
				CONSTRUCTION SUBTOTAL	***$1,699,750***
				TOTAL ESTIMATED COST	***$2,119,700***
*Assume 15% of Existing 24" Force Main Needs Replacement.				**LOW (-15%)**	**$1,801,745**
*Assume (2) Point Repairs Required per Year, for 10 Year Period.				**HIGH (+20%)**	**$2,543,640**

2.1.2 Complete Replacement: 24" Ductile Iron Force Main

The second alternative evaluated for this facility plan is the complete replacement of the existing 24" PVC force main with a new 24" ductile iron force main. The primary benefit of this alternative is that all remaining PVC pipe within the South Beaches Force Main system would be replaced with ductile iron. Replacement of the existing 24" PVC force main would eliminate all issues identified in the previous study with regard to improper pipe installation, joint restraints, and compaction. The ductile iron force main would provide a much higher level of protection against future breaks, and therefore remove the public health/environmental impacts. In addition, this alternative would likely extend the life of the pipe system for the next 50+ years, conservatively. The disadvantages of this alternative include construction cost, site construction impacts, and unaddressed issues with pressures in the system. The conceptual cost estimate for this alternative is $6.3 million, as seen in **Table 2-2**, is more expensive that the point repair alternative due to the extent of construction required to replace the entire 3.3 miles of pipeline. The replacement of the entire pipeline would also have more of a construction impact to the community and local residents. Another drawback to this alternative is that it does not address the negative system pressures and pressure surges that were identified in the surge analysis study, and this would require additional action to be taken in the future.

Table 2-2: Conceptual Cost Estimate-24" Force Main Replacement

ITEM 1 - GENERAL REQUIREMENTS					
1	Mobilization/Demobilization/General Conditions for Infrastructure Imporvements	1	LS	$50,000.00	$50,000
2	Maintenance of Traffic	1	LS	$30,000.00	$30,000
3	Bonds and Insurance Requirements	1	LS	$5,000.00	$5,000
4	Survey Layout & Record Drawings	1	LS	$30,000.00	$30,000
5	Professional Videotaping of Site Conditions (Pre/Post Construction)	1	LS	$5,000.00	$5,000
6	Contingency	(20% of Item 2-General Sitework Subtotal)			$1,032,840
				SUBTOTAL	$1,152,840
ITEM 2 - GENERAL SITEWORK					
7	Utility Pipe - 24" Ductile Iron Force Main	17700	LF	$200.00	$3,540,000
8	Utility Fitting - Ductile Iron Elbow - 24"	35	EA	$7,000.00	$245,000
9	Utility Fixture - Valve Assembly - 24"	8	EA	$32,000.00	$256,000
10	Utility Fixture - Tapping Sleeve/Valve - 24"	2	EA	$48,000.00	$96,000
11	Utility Fixture - 2" Air Release Valve	6	EA	$6,000.00	$36,000
12	24" HDD Canal Crossing	1	EA	$85,000.00	$85,000
13	Remove and Replace Concrete Sidewalk	15000	SF	$25.00	$375,000
14	Base and Asphalt Trench Repair	800	SY	$50.00	$40,000
15	Asphalt Milling & Overlay	1000	SY	$15.00	$15,000
16	Remove and Replace Concrete Driveway Apron	860	SY	$60.00	$51,600
17	Pavement Striping	1	LS	$10,000.00	$10,000
18	Flowable Fill 24" Force Main	17700	LF	$8.00	$141,600
19	Remove and replace Unsuitable Materials	1000	CY	$30.00	$30,000
20	Seeding	4000	SY	$2.00	$8,000
21	Sodding	7000	SY	$5.00	$35,000
22	Landscaping	1	LS	$20,000.00	$20,000
23	Density Testing	1	LS	$25,000.00	$25,000
24	Pressure Testing	1	LS	$5,000.00	$5,000
25	Dewatering	1	LS	$150,000.00	$150,000
				CONSTRUCTION SUBTOTAL	$5,164,200
				TOTAL ESTIMATED COST	$6,317,040
				LOW (-15%)	$5,369,484
				HIGH (+20%)	$7,580,448

2.1.3 Complete Replacement: 30" Ductile Iron Force Main with Surge Tanks

The third alternative considered is the complete replacement of the existing 24" PVC force main with a new 30" ductile iron force main, and the installation of pressure surge tanks at strategic locations. Like the previous alternative, the benefit of this option is that all remaining PVC pipe within the South Beaches Force Main system would be replaced with ductile iron, and this would eliminate all issues identified in the previous study with regard to improper pipe installation, joint restraints, and compaction. The ductile iron force main would provide a much higher level of protection against future breaks, and therefore remove the public health/environmental impacts. This alternative would also extend the life of the pipe system for the next 50+ years, conservatively. An additional benefit of using 30" DI pipe is that this would help alleviate the high pressures seen in the existing force main system. Based on the force may study that was done, upsizing the Riverside Drive Force Main to 30" would lower system pressures, reduce the total dynamic head seen by the pump stations, and ultimately provide added insurance against future pipe failures. Finally, this alternative would provide a solution for excessive pressure fluctuations within the force main system by utilizing pressure surge tanks. Based on the surge analysis report in **Appendix C**, surge tanks are recommended at the existing B-19 and B-20 lift stations to protect the system from both pressure spikes and negative pressures. The disadvantages of this alternative include construction cost and site construction impacts. The conceptual cost estimate for this alternative is $7.8 million, as seen in **Table 2-3**, is more expensive that both the point repair alternative and the 24" pipe replacement alternative.

Table 2-3: Conceptual Cost Estimate-30" Force Main Replacement with Surge Tanks

ITEM NO.	DESCRIPTION	QUANTITY	UNIT	UNIT PRICE	TOTAL
ITEM 1 - GENERAL REQUIREMENTS					
1	Mobilization/Demobilization/General Conditions for Infrastructure Imporvements	1	LS	$50,000.00	$50,000
2	Maintenance of Traffic	1	LS	$30,000.00	$30,000
3	Bonds and Insurance Requirements	1	LS	$5,000.00	$5,000
4	Survey Layout & Record Drawings	1	LS	$30,000.00	$30,000
5	Professional Videotaping of Site Conditions (Pre/Post Construction)	1	LS	$5,000.00	$5,000
6	Contingency	(20% of Item 2-General Sitework Subtotal)			$1,285,840
				SUBTOTAL	*$1,405,840*
ITEM 2 - GENERAL SITEWORK					
7	Utility Pipe - 30" Ductile Iron Force Main	17700	LF	$250.00	$4,425,000
8	Utility Fitting - Ductile Iron Elbow - 30"	35	EA	$8,000.00	$280,000
9	Utility Fixture - Valve Assembly - 30"	8	EA	$40,000.00	$320,000
10	Utility Fixture - Tapping Sleeve/Valve - 30"	2	EA	$56,000.00	$112,000
11	Utility Fixture - 2" Air Release Valve	6	EA	$6,000.00	$36,000
12	30" HDD Canal Crossing	1	EA	$100,000.00	$100,000
13	Force Main Surge Tank with Piping	2	EA	$125,000.00	$250,000
14	Remove and Replace Concrete Sidewalk	15000	SF	$25.00	$375,000
15	Base and Asphalt Trench Repair	800	SY	$50.00	$40,000
16	Asphalt Milling & Overlay	1000	SY	$15.00	$15,000
17	Remove and Replace Concrete Driveway Apron	860	SY	$60.00	$51,600
18	Pavement Striping	1	LS	$10,000.00	$10,000
19	Flowable Fill 24" Force Main	17700	LF	$8.00	$141,600
20	Remove and replace Unsuitable Materials	1000	CY	$30.00	$30,000
21	Seeding	4000	SY	$2.00	$8,000
22	Sodding	7000	SY	$5.00	$35,000
23	Landscaping	1	LS	$20,000.00	$20,000
24	Density Testing	1	LS	$25,000.00	$25,000
25	Pressure Testing	1	LS	$5,000.00	$5,000
26	Dewatering	1	LS	$150,000.00	$150,000
				CONSTRUCTION SUBTOTAL	*$6,429,200*
				TOTAL ESTIMATED COST	*$7,835,040*
				LOW (-15%)	**$6,659,784**
				HIGH (+20%)	**$9,402,048**

2.1.4 Conclusions

This evaluation includes a comparison between three different alternatives including point repairing the existing 24" PVC force main, replacing the existing 24" PVC force main with a 24" ductile iron force main, and replacing the existing 24" PVC force main with a 30" ductile iron force main and surge tanks. Additional evaluation criteria are listed below.

Alternative #1 – Point Repair 24" PVC Force Main

<u>Advantages</u>

- Lower construction cost
- Least amount of impact to community

<u>Disadvantages</u>

- Low reliability
- Future pipe failures likely
- Continued SSO risk to public and environment
- Doesn't address pressure issues in force main

Alternative #2 – 24" Ductile Iron Force Main Replacement

<u>Advantages</u>

- Resolves issue with improper PVC pipe installation
- Reduces risk of future pipe failures
- Extends the future lifespan of force main system

Disadvantages
- Higher Construction Cost
- Greater construction impact to community
- Doesn't address high pressures in force main

Alternative #3 – 30" Ductile Iron Force Main Replacement with Surge Tanks
Advantages
- Resolves issue with improper PVC pipe installation
- Resolves the issue of both high and negative pressures in the pipe system
- Greatly reduces risk of future pipe failures
- Extends the future lifespan of force main system

Disadvantages
- Higher Construction Cost
- Greater construction impact

A tabular listing of evaluation criteria for each alternative to address the Riverside Drive Force Main failures is listed below in **Table 2-4**.

Table 2-4: Alternatives Evaluation Criteria

Evaluation Criteria						
No.	Alternative	*Project Cost	System Reliability	Environmental Impact	Future Capacity/Life	Total
1	Point Repair Existing 24" PVC Force Main	3	1	1	1	9
2	Replacement: 24" DI Force Main	2	2	2	2	10
3	Replacement: 30" DI Force Main w/Surge Tanks	1	3	3	3	11
*Cost Factor Multiplied by 2.0						

Based on the evaluation contained herein, Alternative #3, the replacement of the existing 24" force main with a new 30" DI force main and surge tanks is recommended as the preferred option.

3 Environmental Effects

3.1 Physiography

3.1.1 Physiographic Regions

Brevard County has three major physiographic regions: the Atlantic Coastal Ridge/Coastal Zone, the St. Johns Valley, and the Barrier Islands. The project alternatives in this facility plan fall within the Barrier Islands region. The Barrier Island is separated from the Brevard County mainland by the Indian River and is bordered on the east by the Atlantic Ocean. Merritt Island varies in width from less than one mile to about ten miles. The Barrier Islands range in elevation from sea level to about twenty feet.

3.1.2 Soils

The soils survey prepared by the United States Department of Agriculture (USDA) National Resource Conservation Service (NRCS) shows one primary soil associations with four other minor soil associations in the project area. The primary soil classification according to the NRCS soil survey is 71.9% Welaka Sand. The entire soil survey with soil classifications can be found in **Appendix D**.

3.1.3 Water Resources

The Indian River is a major surface water body, which separates the Brevard County mainland from Merritt Island that can be described as a series of shallow poorly flushed, brackish lagoons, separated by manmade barriers in the form of causeways. The river system contains both Class II and Class III waters, as currently classified by the Florida Department of Environmental Protection. The Indian River was given additional protection by legislative act. The river has historically been a very productive estuarine system. Pollution loads cannot be readily assimilated by the river system. This is generally due to the poor water circulation within the system caused by indiscriminate dredging, filling, and bulk heading. The lack of assimilative capacity has resulted in increased stress to the system.

3.1.4 Floodplains

The Federal Emergency Management Agency (FEMA) has established flood plains throughout the country and maintains their Flood Insurance Rate Maps (FIRMs). The force main alignment being evaluated in this facility plan is graphically represented in Figure 3-1. These maps have been reviewed in conjunction with the project alternatives identified in this Facility Plan. There are no detrimental effects to flood plain development resulting from projects in this plan.

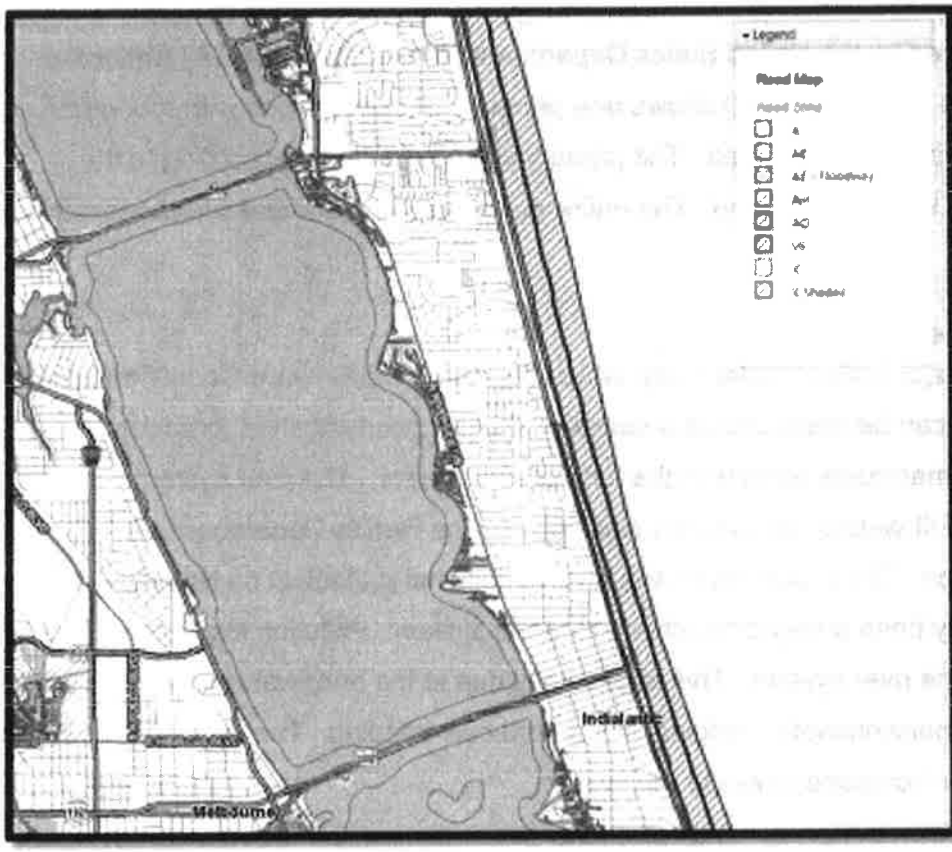


Figure 3-1: FEMA Flood Zones Map

3.1.5 Historic or Archeological Site

Brevard County is planning to repair and/or replace the existing 24" force main along Riverside Drive within the existing boundaries of the current force main limits. There are no known archeological or historic sites that will be affected by the project proposed in this plan.

3.1.6 Wildlife and Endangered Species

There are several mammals, birds, reptiles, fish, and plants listed as threatened, endangered, or of special concern that are present in Brevard County. The proposed project identified in this facility plan will not have any impacts upon the endangered or threatened species in this region. The full list of Florida's federally listed plant species can be found at: https://www.freshfromflorida.com/Divisions-Offices/Florida-Forest-Service/Our-Forests/Forest-Health/Florida-Statewide-Endangered-and-Threatened-Plant-Conservation-Program/Florida-s-Federally-Listed-Plant-Species. A full list of the FWC's listed species may be viewed by visiting the FWC website at: http://myfwc.com/wildlifehabitats/imperiled/profiles/. Some of the listed species can be found in areas nearby the proposed project; however, because the project construction area is within a cleared existing right-of-way along Riverside Drive there are no listed species that would be directly affected by the proposed project. The following **Table 3-1** summarizes the endangered and threatened species found in Brevard County.

Table 3-1: Protection Status of Wildlife Found in Brevard County

Birds - Brevard County USFWC Protected Species	
Common Name	**Status**
Everglades Snail Kite	Endangered
Wood Stork	Threatened
Roseate Spoonbill	Endangered
Little Blue Heron	Endangered
Reddish Egret	Threatened
Snowy Egret	Special Concern
Louisiana Heron	Special Concern
Peregrin Falcon	Special Concern
Southern Kestral	UR2
American Oyster Catcher	Threatened
Southern Bald Eagle	Special Concern
Osprey	Threatened
Brown Pelican	Special Concern
Least Tern	Endangered
Roseate Tern	Endangered
Audubon's Crested Caracara	Threatened
Piping Plover	Threatened
Florida Scrub Jay	Threatened
Red Knot	Threatened

Plants - Brevard County USFWC Protected Species	
Common Name	**Status**
Lewton's Polygala	Endangered
Coastal Vervain	Special Concern
Prickly Pear	Threatened
Sea Lavender	Special Concern
Carter's Mustard	Endangered

Fish - Brevard County USFWC Protected Species	
Common Name	**Status**
Rivulus	Special Concern
Common Snook	Threatened

Mammals - Brevard County USFWC Protected Species	
Common Name	**Status**
West Indian Manatee	Threatened
Florida Mink	Special Concern
Round Tailed Muskrat	UR2
Florida Mouse	UR2
Fox Squirrel	Special Concern
Southeastern Beach Mouse	Threatened

Reptiles - Brevard County USFWC Protected Species	
Common Name	**Status**
American Alligator	Special Concern
Hawksbill Turtle	Endangered
Ridley Turtle	Endangered
Hawksbill Sea Turtle	Endangered
Leatherback Sea Turtle	Endangered
Green Sea Turtle	Threatened
Loggerhead Sea Turtle	Threatened
Atlantic Salt Marsh Snake	Threatened
Eastern Indigo Snake	Threatened
Gopher Tortoise	Candidate

3.2 Environmental/Community Benefits

The repair and/or replacement of the Riverside Drive Force main will greatly reduce the potential for future sewage spills into the environment and reduce construction impacts due to emergency pipe repair. The Riverside Drive Force Main has had multiple failures over the past few years, which have all had an adverse effect on the both the environment and the community. With each pipe failure, there is a usually a corresponding sewage spill that is released into the environment. These spills affect the quality of ground water, soil conditions, and nearby surface water bodies. Depending on the location and extent of the spill, public notices are often issued to warn residents about the potential hazards. In addition to the environmental hazard, the repair of a large diameter force main along a major roadway often causes road closures and traffic delays. By implementing the proposed project in this facility plan, these environmental and community impacts could be mostly eliminated.

3.3 Environmental/Community Impacts

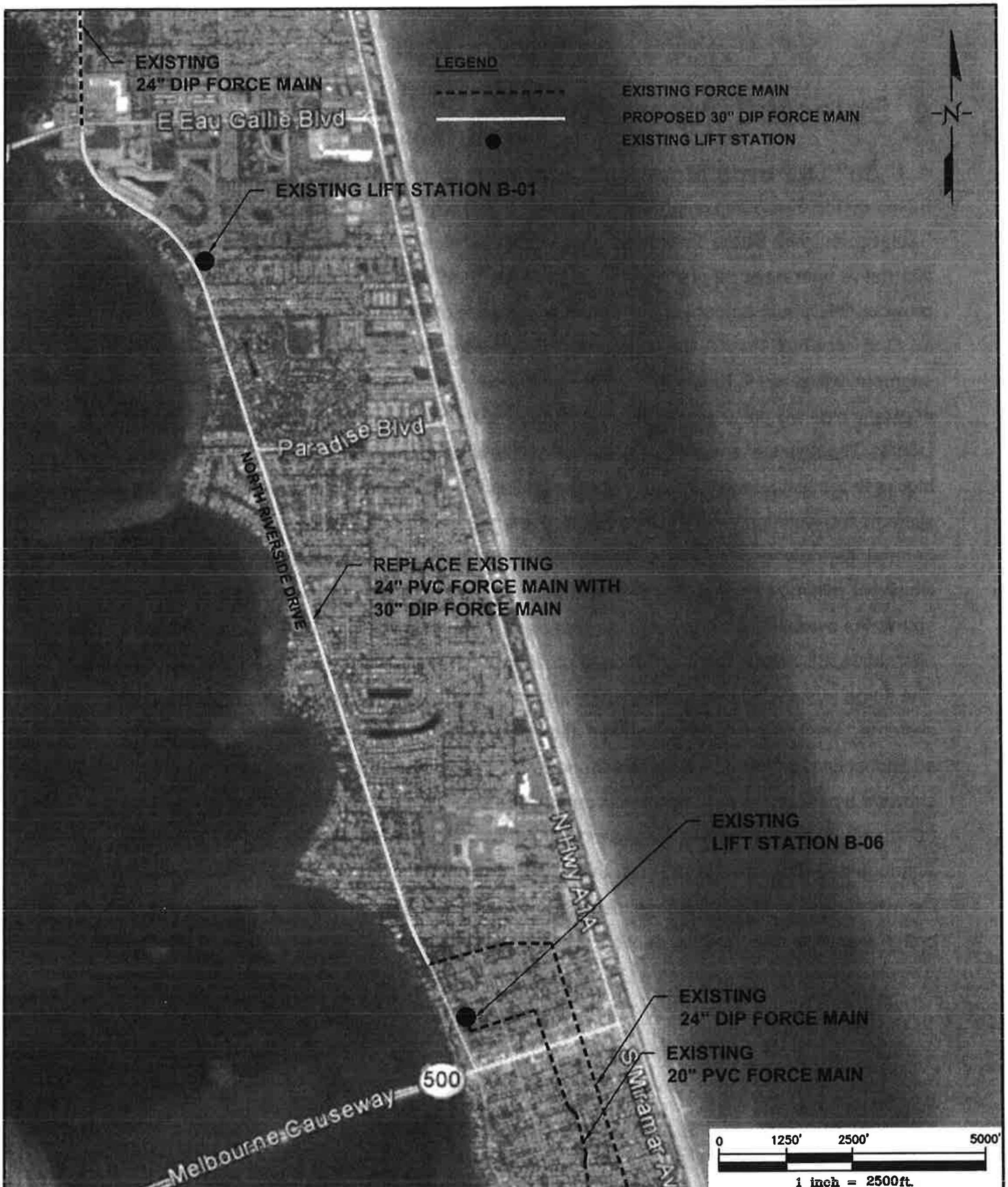
The impacts to the environment and the community as a result of this project are minimal. Community impacts would be limited to temporary lane shifts and lane closures during construction and some traffic delays as a result of the new traffic patterns. The majority of the construction work would occur outside of the travel lanes, and would be in the unpaved right-of-way. Most of the impacts to the travel lanes would be where the new force main crosses the side streets along the proposed alignment.

Environmental impacts would also be very limited due to the fact that the new force main would be placed in the same alignment as the existing force main, and there would be no additional clearing or disruption to the surrounding areas. The proposed project would include a force main crossing underneath a canal. To avoid any environmental impacts at this canal crossing, a trenchless installation method would be used. Horizontal directional drilling would be the preferred installation method, and would eliminate the need to impact the waterway.

4 Selected Alternative

4.1 30" DI Force Main Replacement with Surge Tanks

Based on the evaluation criteria established in this facility plan, the 30" DI Force Main Replacement with Surge Tank Alternative was determined to be the best option. This alternative addresses all of the issues identified in the South Beaches Force Main Study, and provides the lowest amount of risk for the occurrence of future pipe failures. By upsizing the existing force main to 30", the force main system will see lower pressures in the Riverside Drive segment, which will in turn reduce stress on the pipe. In addition, using ductile iron pipe in lieu of plastic pipe will provide a much stronger pipe system that is less prone to breaks and joint failure. The new system will be designed with the appropriate pipe restraints and/or thrust blocks to prevent joint failures. Air release valves will be installed at the high points in the system, and isolation valves will be installed every 800 to 1,000 feet and at major junctions. In general, the new force main system will follow the alignment of the existing 24" force main, which will minimize the amount of new impacts to the surrounding infrastructure. **Figure 4-1** shows the overall project layout of the proposed force main alternative. Additionally, this alternative will include the installation of pressure surge tanks at the B-19 and B-20 Lift Stations. The surge tanks would be roughly 10,000 gallon vertical surge tanks with internal air bladder systems. The tanks would be installed at existing lift station sites, and would not require any additional land acquisition. **Figure 4-2** shows the proposed surge tank locations, and **Figure 4-3** shows a typical surge tank schematic detail. The conceptual construction cost estimate for the 30" ductile iron force main replacement with surge tanks alternative is roughly $7.8 million. Although this alternative is the most expensive, this option is the only one that addresses all of the system issues that have been identified and therefore provides the highest level of safety for both the environment and the public.



HDR

PROJECT TITLE RIVERSIDE DRIVE FORCE MAIN REPLACEMENT

SHEET TITLE

FIGURE:4-1

PROJECT LAYOUT AERIAL

PROJECT NUMBER
10092206

PROJECT MANAGER
H.HARDY

DATE
03/2018

EXHIBIT SCALE
1"=2500'

EXHIBIT NUMBER
4-1

HDR

PROJECT TITLE RIVERSIDE DRIVE
FORCE MAIN REPLACEMENT

SHEET TITLE
FIGURE:4-2
SURGE TANK LOCATIONS

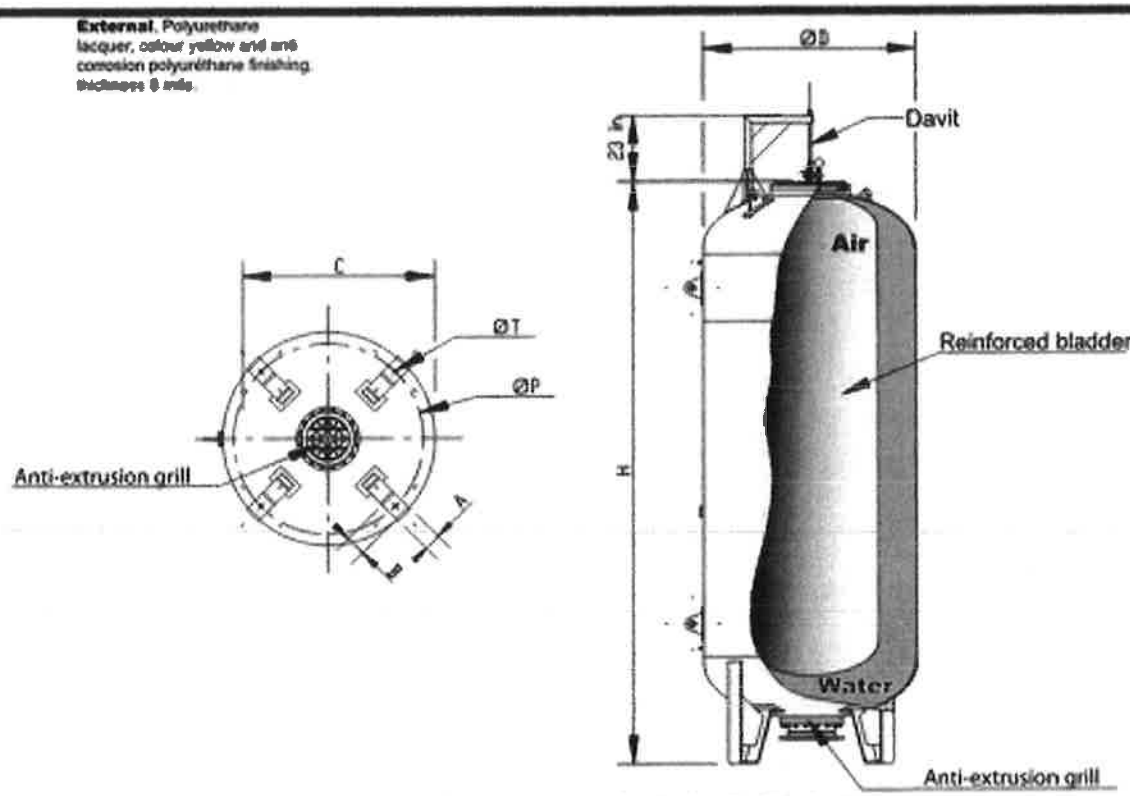
PROJECT NUMBER
10092206

PROJECT MANAGER
H.HARDY

DATE
03/2018

EXHIBIT SCALE
1"=5000'

EXHIBIT NUMBER
4-2



Capacity gallon	Diameter D	Height H	P	T	Size A	Size B	Size C	Davit
793	47	150	41	1	6	5	54	No
1057	47	161	41	1	6	5	54	No
1321	59	138	51	1	6	7	66	No
1585	59	169	51	1	6	7	66	No
1849	59	189	51	1	6	7	66	No
2113	59	209	51	1	6	7	66	No
2378	75	177	67	1	6	7	81	Yes
2378	83	138	71	1	6	11	89	Yes
2642	75	193	67	1	6	7	81	Yes
2642	83	150	71	1	6	11	89	Yes
3170	75	220	67	1	6	7	81	Yes
3170	83	173	71	1	6	11	89	Yes
3963	83	209	71	1	6	11	89	Yes
5283	98	197	79	1	20	20	105	Yes
6604	98	236	79	1	20	20	105	Yes
7925	118	213	100	1	20	20	125	Yes
9246	118	240	100	1	20	20	125	Yes

Figure 4-3: Surge Tank Schematic Detail

A

Appendix A – FDEP Consent Order

BEFORE THE STATE OF FLORIDA
DEPARTMENT OF ENVIRONMENTAL PROTECTION

STATE OF FLORIDA DEPARTMENT OF ENVIRONMENTAL PROTECTION	)))	IN THE OFFICE OF THE CENTRAL DISTRICT
v.	)	OGC FILE NO. 16-1352
BREVARD COUNTY	))	

CONSENT ORDER

This Consent Order ("Order") is entered into between the State of Florida Department of Environmental Protection ("Department") and Brevard County ("Respondent") to reach settlement of certain matters at issue between the Department and Respondent.

The Department finds and Respondent admits the following:

1. The Department is the administrative agency of the State of Florida having the power and duty to protect Florida's air and water resources and to administer and enforce the provisions of Chapter 403, Florida Statutes ("F.S."), and the rules promulgated and authorized in Title 62, Florida Administrative Code ("F.A.C."). The Department has jurisdiction over the matters addressed in this Order.

2. Respondent is a person within the meaning of Section 403.031(5), F.S.

3. Respondent is the owner and is responsible for the operation of the Brevard County Utilities Department (BCUD) South Beaches wastewater treatment facility (WWTF) and collections system, an 8.0 MGD annual average daily flow (AADF) domestic WWTF facility with surface water discharge (D-001), underground injection (U-001), and land application (R-001) ("Facility"). The Facility is operated under Wastewater Permit No. FL0040622 ("Permit"), which was issued on March 13, 2014 and will expire on March 19, 2019. The Facility is located at 2800 South Highway A1A, Melbourne Beach, FL 32951, in Brevard County, Florida ("Property"). Respondent owns the Property on which the Facility is located.

4. The Department finds that the following violation(s) occurred:

a) A force main break on South Patrick Drive between DeSoto Parkway and Banana River Drive caused three unauthorized discharges of raw wastewater totaling approximately 1,521,000 gallons on September 20, 2016. Two to Class III surface waters, 733,000 gallons and 578,000 gallons, and an additional 210,000 gallons to a stormwater pond. The releases are in violation of 62-604.130 F.A.C., 62-604.500(2) F.A.C., and F.S. Chapter 403.121(3)(b).

Having reached a resolution of the matter Respondent and the Department mutually agree and it is

ORDERED:

5. Respondent shall comply with the following corrective actions within the stated time periods:

a) Within 365 days of the effective date of this Order, replace the 1.1-mile section of the BCUD South Beaches collection system force main along South Patrick Drive between DeSoto Parkway and Banana River Drive with a recent history of line breaks. The Department recognizes that the Respondent has initiated addressing the subject force main as of June 2016. A Task Order was executed to design and construct replacement pipe on August 23, 2016 and the Brevard County Board of Commissioners initiated emergency conditions to waive the bidding process in order to expedite the replacement project on September 27, 2016. Respondent submitted copies of these orders to the Department on October 24, 2016.

b) Within 30 days of the effective date of the Order, provide a plan outlining best management practices (BMPs) and spill prevention and response measures which will be implemented during the replacement of the affected portion of the collection system to prevent any additional unauthorized releases.

c) Within 365 days of the effective date of this Order, complete an evaluation of the 2.8-mile section of the force main on South Patrick Drive/North Riverside Drive between DeSoto Parkway and Sand Dollar Road in accordance with paragraph 6, below.

d) Within 60 days of the completion of paragraph 5c, above, submit the results of the evaluation along with a plan and schedule for corrective action, if applicable.

e) Within 180 days of the effective date of this Order, provide a long-term maintenance plan of the entire BCUD South Beaches collection system to prevent similar discharges in the future. The maintenance plan shall include an up-to-date project list for rehabilitation and maintenance based on the Infrastructure Asset Evaluation completed by Brevard County Utilities in 2013, or any additional more recent evaluations.

f) Every calendar quarter after the effective date of this Order, Respondent shall submit in writing to the Department a report containing information concerning the status and progress of projects being completed under this Order, information as to compliance or noncompliance with the applicable requirements of this Order including construction requirements and effluent limitations, and any reasons for noncompliance. These reports shall also include a projection of the work to be performed pursuant to this Order during the 12-month period which will follow the report. These reports shall be submitted to the Department within 30 days following the end of the quarter.

6. The Respondent shall continue the retained services which commenced on August 23, 2016 of a professional engineer, registered in the State of Florida, to accomplish all of the following:

a) Evaluating the Facility's associated collection system to discover the cause or causes of the noncompliance.

b) Designing modifications of collection systems to ensure the system will function in full and consistent compliance with all applicable rules of the Department.

c) Completing an application for a Department wastewater permit to construct the modifications listed in subparagraph (b) of this paragraph, if such a permit is required.

d) Overseeing the construction of any modifications to the Facility, effluent disposal system, or collection system.

e) Submitting to the Department a Certification of Completion, prepared and sealed by a professional engineer registered in the State of Florida, stating that modifications to the Facility, effluent disposal system, and collection system have been constructed in accordance with the provisions of the Permit.

f) In the event the Department requires additional information to process the permit application described in subparagraph (c) of this paragraph, providing a written response containing the information requested by the Department within 30 days of the date of the request.

7. Within 90 days of the effective date of this Order, Respondent shall submit a complete application for a Department wastewater permit to construct the modifications listed in subparagraph 6b, if such a permit is required.

8. Within 270 days of the date a wastewater permit is issued, or, if no permit is required, within 365 days of the effective date of this Order, Respondent shall complete construction of the modification(s) developed pursuant to paragraph 6 and submit a Certification of Completion, prepared and sealed by a professional engineer registered in the State of Florida, stating that modifications to the Facility, effluent disposal system, and collection system have been constructed in accordance with the provisions of the Permit.

9. Notwithstanding the time periods described in the paragraphs above, Respondent shall complete all corrective actions required by paragraphs 5 and 6 within 365 days of the effective date of this Order and be in full compliance with Rule 62-604, F.A.C., regardless of any intervening events or alternative time frames imposed in this Order, other than those excused delays agreed to by the Department, as described in paragraph 17, below.

10. Within 90 days of the effective date of this Order, Respondent shall submit a written estimate of the total cost of the corrective actions required by this Order to the Department. The written estimate shall identify the information the Respondent relied upon to provide the estimate.

11. Within 45 days of the effective date of this Order, Respondent shall pay the Department $2,750.00 in settlement of the regulatory matters addressed in this Order. This amount includes $2,500.00 for civil penalties and $250.00 for costs and expenses incurred by the Department during the investigation of this matter and the preparation and tracking of this Order. The civil penalties are apportioned as follows: $2,500.00 for violation of Florida Statute (F.S.) 403.121(3)(b).

12. Respondent agrees to pay the Department stipulated penalties in the amount of $100 per day for each and every day Respondent fails to timely comply with any of the

requirements of paragraph(s) 5, 6, 9 of this Order. The Department may demand stipulated penalties at any time after violations occur. Respondent shall pay stipulated penalties owed within 30 days of the Department's issuance of written demand for payment, and shall do so as further described in paragraph 13, below. Nothing in this paragraph shall prevent the Department from filing suit to specifically enforce any terms of this Order. Any stipulated penalties assessed under this paragraph shall be in addition to the civil penalties agreed to in paragraph 11 of this Order.

13. Respondent shall make all payments required by this Order by cashier's check, money order or on-line payment. Cashier's check or money order shall be made payable to the "Department of Environmental Protection" and shall include both the OGC number assigned to this Order and the notation "Water Quality Assurance Trust Fund." Online payments by e-check can be made by going to the DEP Business Portal at: http://www.fldepportal.com/go/pay/. It will take a number of days after this order is final and effective filed with the Clerk of the Department before ability to make online payment is available.

14. Except as otherwise provided, all submittals and payments required by this Order shall be sent to Megan Warr, Environmental Specialist II/Compliance Assurance Program, Department of Environmental Protection, 3319 Maguire Blvd, Suite 232, Orlando, Florida 32803-3767.

15. Respondent shall allow all authorized representatives of the Department access to the Facility and the Property at reasonable times for the purpose of determining compliance with the terms of this Order and the rules and statutes administered by the Department.

16. In the event of a sale or conveyance of the Facility or of the Property upon which the Facility is located, if all of the requirements of this Order have not been fully satisfied, Respondent shall, at least 30 days prior to the sale or conveyance of the Facility or Property, (a) notify the Department of such sale or conveyance, (b) provide the name and address of the purchaser, operator, or person(s) in control of the Facility, and (c) provide a copy of this Order with all attachments to the purchaser, operator, or person(s) in control of the Facility. The sale

or conveyance of the Facility or the Property does not relieve Respondent of the obligations imposed in this Order.

17. If any event, including administrative or judicial challenges by third parties unrelated to Respondent, occurs which causes delay or the reasonable likelihood of delay in complying with the requirements of this Order, Respondent shall have the burden of proving the delay was or will be caused by circumstances beyond the reasonable control of Respondent and could not have been or cannot be overcome by Respondent's due diligence. Neither economic circumstances nor the failure of a contractor, subcontractor, materialman, or other agent (collectively referred to as "contractor") to whom responsibility for performance is delegated to meet contractually imposed deadlines shall be considered circumstances beyond the control of Respondent (unless the cause of the contractor's late performance was also beyond the contractor's control). Upon occurrence of an event causing delay, or upon becoming aware of a potential for delay, Respondent shall notify the Department by the next working day and shall, within seven calendar days notify the Department in writing of (a) the anticipated length and cause of the delay, (b) the measures taken or to be taken to prevent or minimize the delay, and (c) the timetable by which Respondent intends to implement these measures. If the parties can agree that the delay or anticipated delay has been or will be caused by circumstances beyond the reasonable control of Respondent, the time for performance hereunder shall be extended. The agreement to extend compliance must identify the provision or provisions extended, the new compliance date or dates, and the additional measures Respondent must take to avoid or minimize the delay, if any. Failure of Respondent to comply with the notice requirements of this paragraph in a timely manner constitutes a waiver of Respondent's right to request an extension of time for compliance for those circumstances.

18. The Department, for and in consideration of the complete and timely performance by Respondent of all the obligations agreed to in this Order, hereby conditionally waives its right to seek judicial imposition of damages or civil penalties for the violations

described above up to the date of the filing of this Order. This waiver is conditioned upon Respondent's complete compliance with all of the terms of this Order.

19. This Order is a settlement of the Department's civil and administrative authority arising under Florida law to resolve the matters addressed herein. This Order is not a settlement of any criminal liabilities which may arise under Florida law, nor is it a settlement of any violation which may be prosecuted criminally or civilly under federal law. Entry of this Order does not relieve Respondent of the need to comply with applicable federal, state, or local laws, rules, or ordinances.

20. The Department hereby expressly reserves the right to initiate appropriate legal action to address any violations of statutes or rules administered by the Department that are not specifically resolved by this Order.

21. Respondent is fully aware that a violation of the terms of this Order may subject Respondent to judicial imposition of damages, civil penalties up to $10,000.00 per day per violation, and criminal penalties.

22. Respondent acknowledges and waives its right to an administrative hearing pursuant to sections 120.569 and 120.57, F.S., on the terms of this Order. Respondent also acknowledges and waives its right to appeal the terms of this Order pursuant to section 120.68, F.S.

23. Electronic signatures or other versions of the parties' signatures, such as .pdf or facsimile, shall be valid and have the same force and effect as originals. No modifications of the terms of this Order will be effective until reduced to writing, executed by both Respondent and the Department, and filed with the clerk of the Department.

24. The terms and conditions set forth in this Order may be enforced in a court of competent jurisdiction pursuant to sections 120.69 and 403.121, F.S. Failure to comply with the terms of this Order constitutes a violation of section 403.161(1)(b), F.S.

25. This Consent Order is a final order of the Department pursuant to section 120.52(7), F.S., and it is final and effective on the date filed with the Clerk of the Department unless a Petition for Administrative Hearing is filed in accordance with Chapter 120, F.S.

Upon the timely filing of a petition, this Consent Order will not be effective until further order of the Department.

26. Persons who are not parties to this Consent Order, but whose substantial interests are affected by it, have a right to petition for an administrative hearing under sections 120.569 and 120.57, Florida Statutes. Because the administrative hearing process is designed to formulate final agency action, the filing of a petition concerning this Consent Order means that the Department's final action may be different from the position it has taken in the Consent Order.

The petition for administrative hearing must contain all of the following information:

a) The OGC Number assigned to this Consent Order;

b) The name, address, and telephone number of each petitioner; the name, address, and telephone number of the petitioner's representative, if any, which shall be the address for service purposes during the course of the proceeding;

c) An explanation of how the petitioner's substantial interests will be affected by the Consent Order;

d) A statement of when and how the petitioner received notice of the Consent Order;

e) Either a statement of all material facts disputed by the petitioner or a statement that the petitioner does not dispute any material facts;

f) A statement of the specific facts the petitioner contends warrant reversal or modification of the Consent Order;

g) A statement of the rules or statutes the petitioner contends require reversal or modification of the Consent Order; and

h) A statement of the relief sought by the petitioner, stating precisely the action petitioner wishes the Department to take with respect to the Consent Order.

The petition must be filed (<u>received</u>) at the Department's Office of General Counsel, 3900 Commonwealth Boulevard, MS# 35, Tallahassee, Florida 32399-3000 within <u>21 days</u> of receipt of this notice. A copy of the petition must also be mailed at the time of filing to the Central District Office at 3319 Maguire Blvd, Suite 232, Orlando, FL 32803. Failure to file a petition within the 21-day period constitutes a person's waiver of the right to request an administrative hearing and to participate as a party to this proceeding under sections 120.569 and 120.57, Florida Statutes. Before the deadline for filing a petition, a person whose substantial interests are affected by this Consent Order may choose to pursue mediation as an alternative remedy under section 120.573, Florida Statutes. Choosing mediation will not adversely affect such person's right to request an administrative hearing if mediation does not result in a settlement. Additional information about mediation is provided in section 120.573, Florida Statutes and Rule 62-110.106(12), Florida Administrative Code.

27. Rules referenced in this Order are available at http://www.dep.state.fl.us/legal/Rules/rulelist.htm

FOR THE RESPONDENT:

11-17-16

Jim Barfield
Chairman, Board of County Commissioners of Brevard County, Florida

Date

DONE AND ORDERED this 18th day of November 2016, in Orange County, Florida.

STATE OF FLORIDA DEPARTMENT
OF ENVIRONMENTAL PROTECTION

Jeff Prather
Director, Central District

Filed, on this date, pursuant to section 120.52, F.S., with the designated Department Clerk, receipt of which is hereby acknowledged.

November 18, 2016

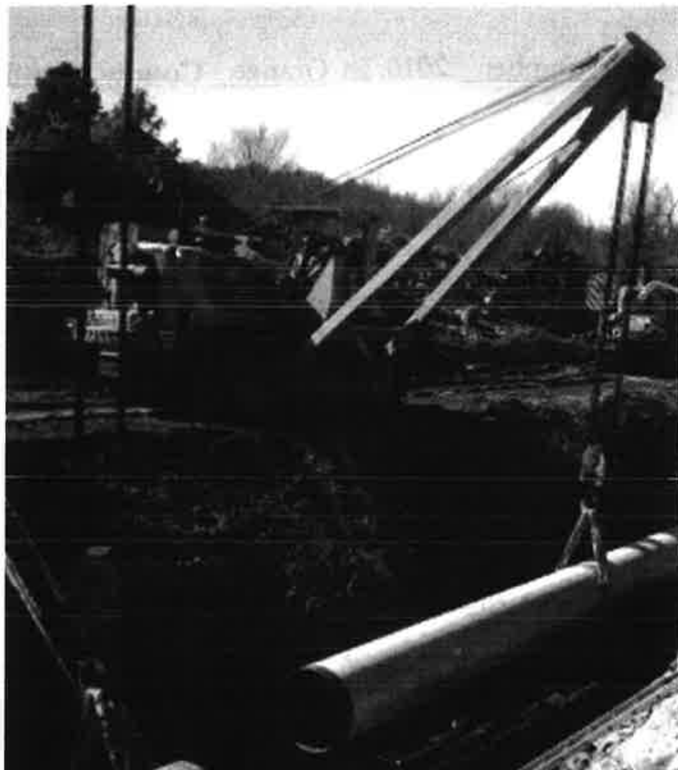
Clerk

Date

Copies furnished to:

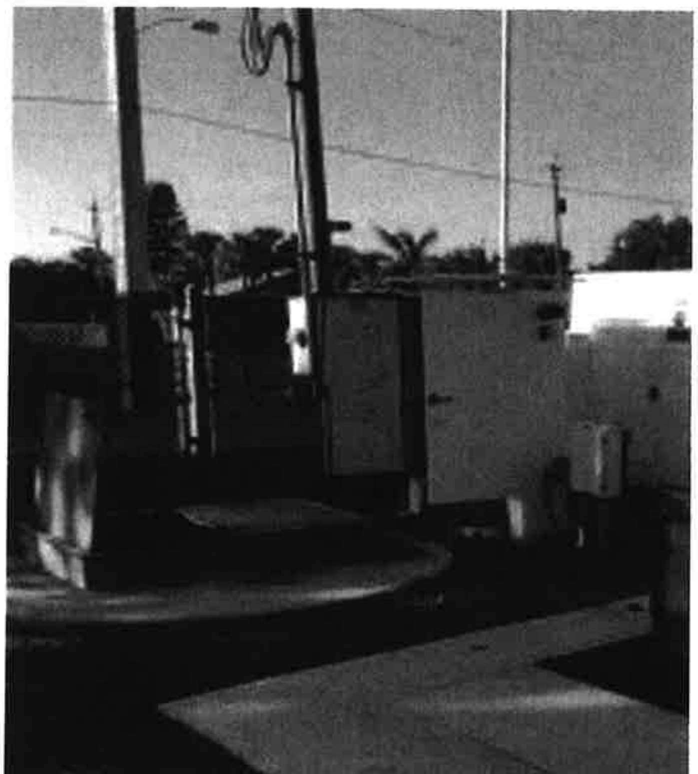
Lea Crandall, Agency Clerk
Mail Station 35

DW_CO (REV. 06/09)



B

Appendix B – South Beaches Force Main Study



Brevard County

South Beaches Force Main Analysis

November 14, 2017

Table of Contents

Introduction

Brevard County Utility Services (USD) has requested HDR Inc. to evaluate the 'South Beaches Force Main System'. Specifically, USD has identified the need to investigate the cause of repeated force main breaks along S. Patrick Drive and Riverside Drive, which are part of the force main system from Lift Station #B-19 to the South Beaches WWTP. The force main system is comprised of a combination of ductile iron pipe and PVC pipe. The pipe sizes on the main force main system vary between 20" and 24". The three main pump stations that contribute the majority of the flow to this system are Lift Stations B-19, B-20, and B-01.

The system has experienced a significant number of breaks over the last few years, concentrated in the areas where the pipe material is PVC. In November 2016, the Florida Department of Environmental Protection issued a Consent Order related to sewage spills resulting from these force main breaks. Included in the Consent Order is the requirement that USD study the force main system and the cause of the pipeline failures.

This report will consider the cause of the breaks and provide analysis of options for system upgrades that would minimize spills to the Indian River Lagoon. The Indian River Lagoon is an Outstanding Florida Water (OFW) and the USD is committed to reducing SSO's for this system that could negatively impact this water body.

A portion of this force main system from DeSoto Parkway to Banana River Drive is currently being replaced, with construction expected to be complete by the end of 2017. This portion of PVC pipe was deemed inadequate because of the large number of breaks and lack of proper restraints, so the USD decided to replace this concurrent with studying the remainder of the system.

2.0 Review of Pipe Break Data and Prior Surge Reports

A review of the existing data indicates evidence of improperly installed pipe in at least two of the pipeline segments. USD has had the pipe manufacturer investigate the installed PVC in one segment to determine if a manufacturing issue was the cause. The force main system is 13 miles in length and operates at relatively high pressures. Pressure surges are thought to be a contributing issue to the high frequency of breaks. Lack of joint restraints, inadequate compaction and pipe bedding are also thought to be contributing factors to the frequency of the breaks. PBS&J performed a surge analysis of the system in 2005, and in 2010, CH2M Hill performed an analysis of the B-6 force main system which also included data and observations about the B-19 system.

Table 1 shows a summary of the force main breaks, categorized by the type of break. Although there have been four types of breaks, nearly 40% of the pipe breaks have been split pipe at the bell of the pipe.

Table 1. Pipe Breaks

Type of Break	Occurrences
Bell Split	10
Leak	11
Joint Separation	2
Other	3

Figures 1 – 3 show the force main breaks along the pipeline route. Figure 1 shows the breaks in the run of pipeline from Desoto Parkway to Banana River Drive. This section is currently under construction and should be complete by the end of 2017. This section is being replaced with 20" and 24" DIP. Further analysis of the pipeline system will focus on the areas shown in Figures 2 – 3.

HDR was tasked to evaluate the PBS&J and the CH2M reports, the Diamond Plastics pipe reports, as well as the data provided in Task 2A, with regard to operating issues and documented failures.

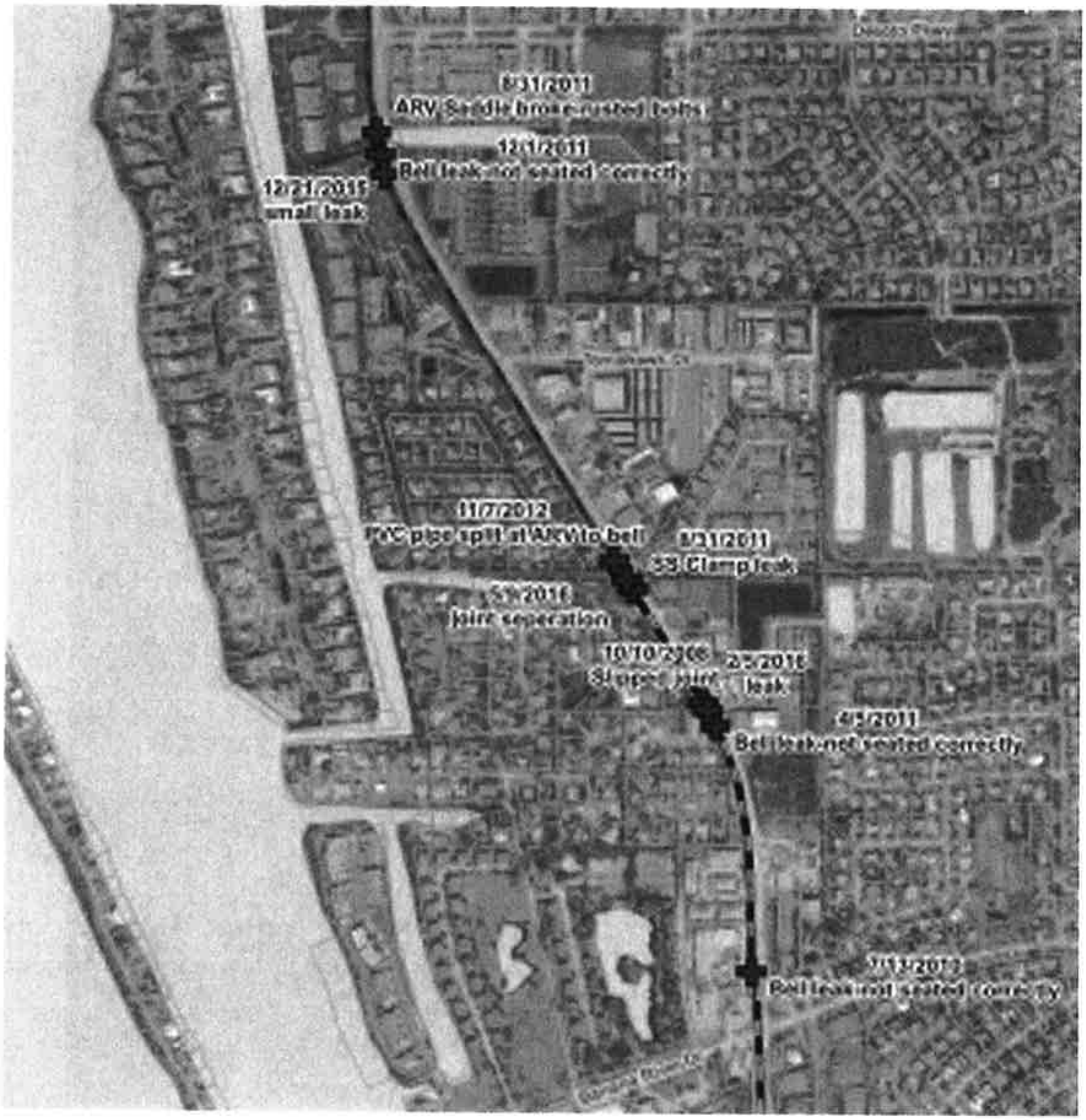


Figure 1. Force Main on S. Patrick from Desoto Pkwy to Banana River Dr.

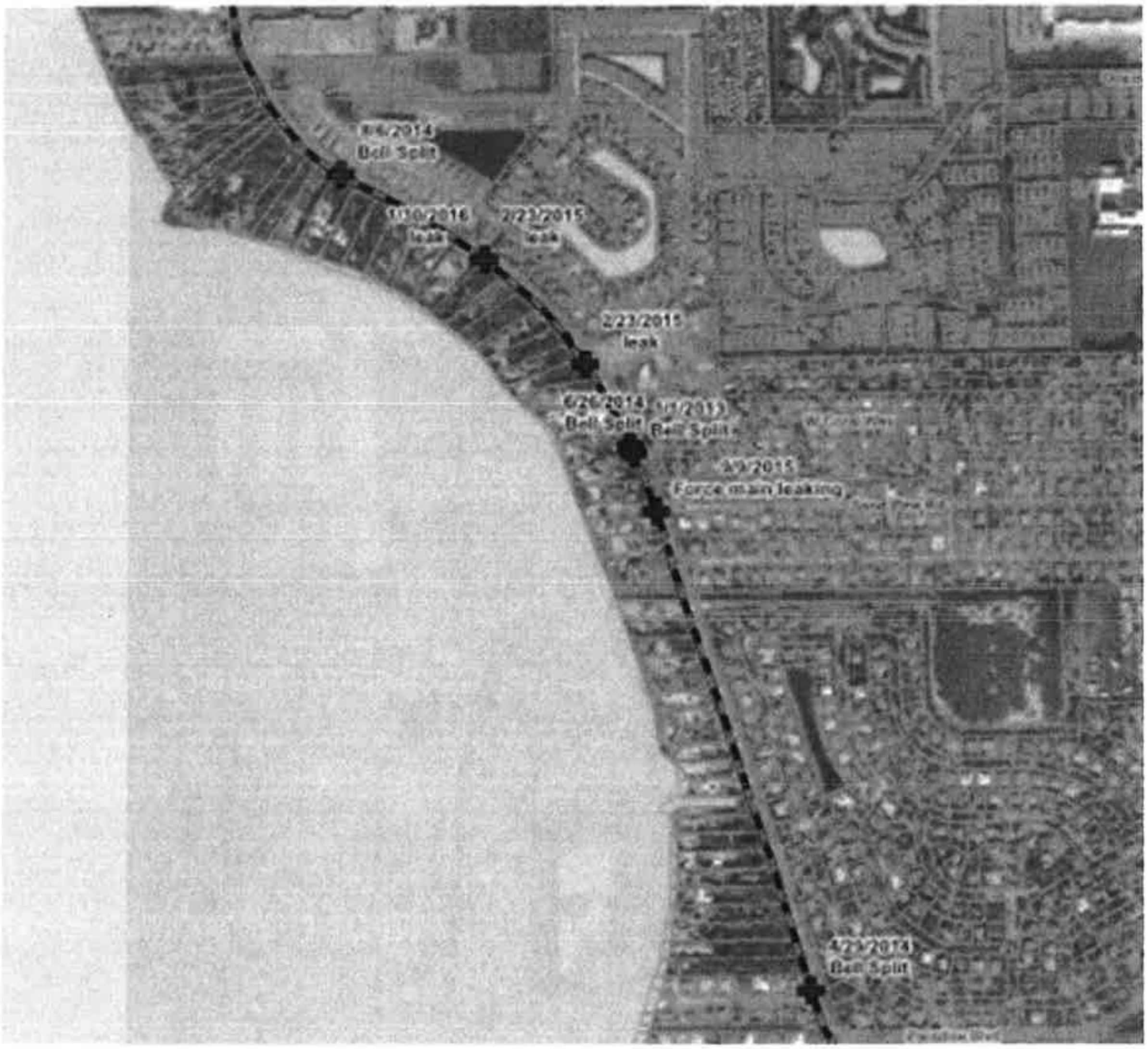


Figure 2. Riverside Dr. from Eau Gallie Blvd. to Paradise Blvd.

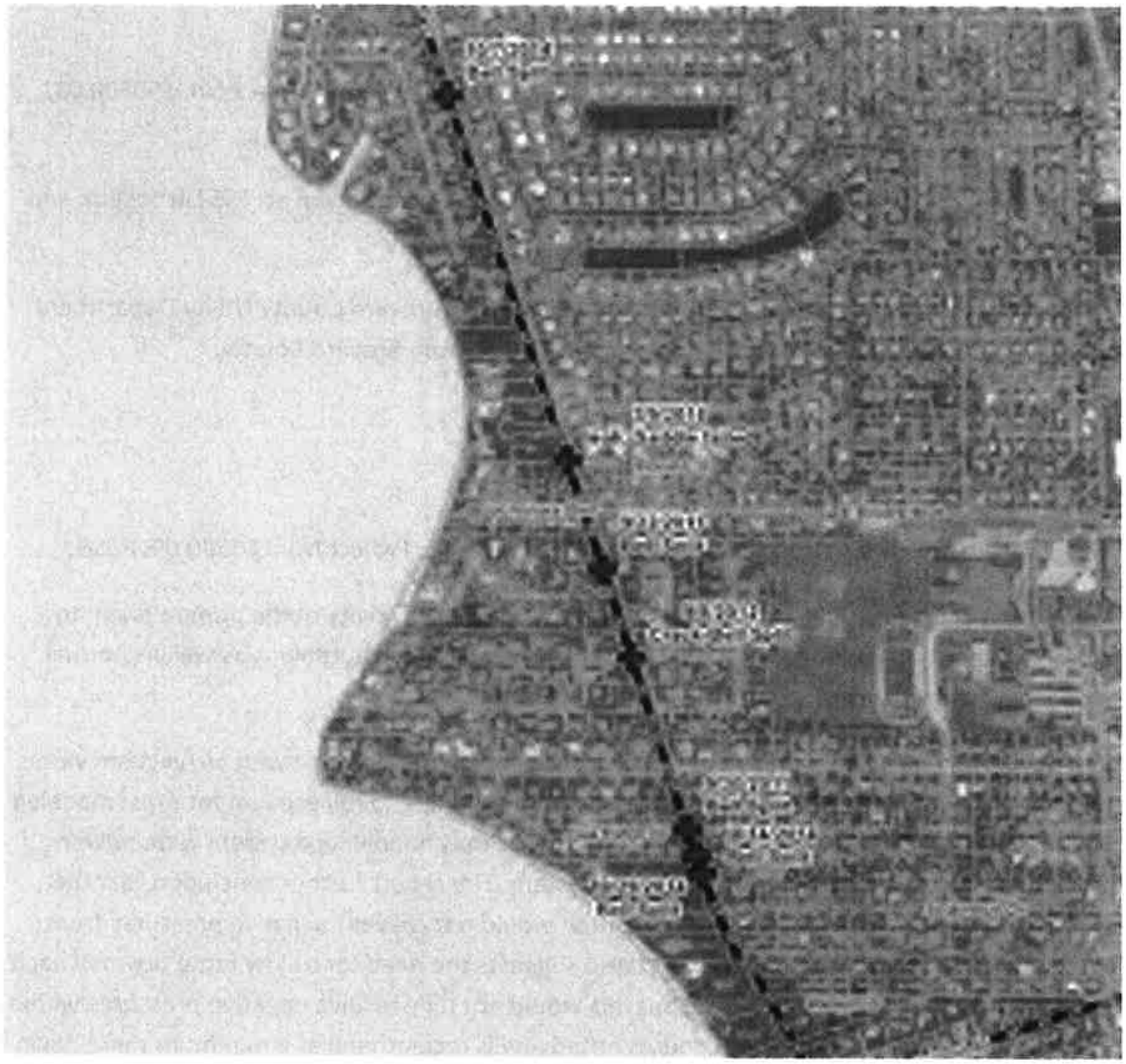


Figure 3. Riverside Dr. from Paradise Blvd. to Oakland Ave.

Reviewed Reports:

- South Beaches, Sewer Force Main Draft Surge Analysis, October 2005, Project No. 150500.09, PBS&J
- Expert Opinion of Scott C. Williams, PE on a Construction Project Known as: B-6 Lift Station and Force Main Modifications Brevard County, FL, February 16, 2010
- Diamond Plastics Corporation Field Report No. FR 3395 Re: Brevard County Utility Department AWWA C 905 DR 25 PVC Sewer Force Main Pipe Samples from Brevard County

Findings from Reviewed Reports:

South Beaches, Sewer Force Main Draft Surge Analysis, October 2005, Project No. 150500.09, PBS&J

The PBS&J report utilized a hydraulic transient model to assess the sensitivity of the pump system to adverse hydraulic transient pressures resulting from uncontrolled pump shutdowns as well as normal shutdown and startup procedures.

The PBS&J recommendations suggested that installing 'surge relief' valves along with air/vacuum valves at each pump station would prevent negative pressures from dropping to full vacuum for most modeled cases. However, when all pump stations fail simultaneously, as may happen in a system wide power outage, full vacuum was predicted to occur within the system. The report further concluded that the 'surge relief' valve and air/vacuum valve surge mitigation would not prevent negative pressures from occurring in the majority of the system. The report also suggests the need for a slow ramp down of each pump to help mitigate the negative transients, but this would not fully resolve negative pressures within the modeled system. A ramp down of 60 seconds is noted and is recommend as a minimum ramp down time in the report.

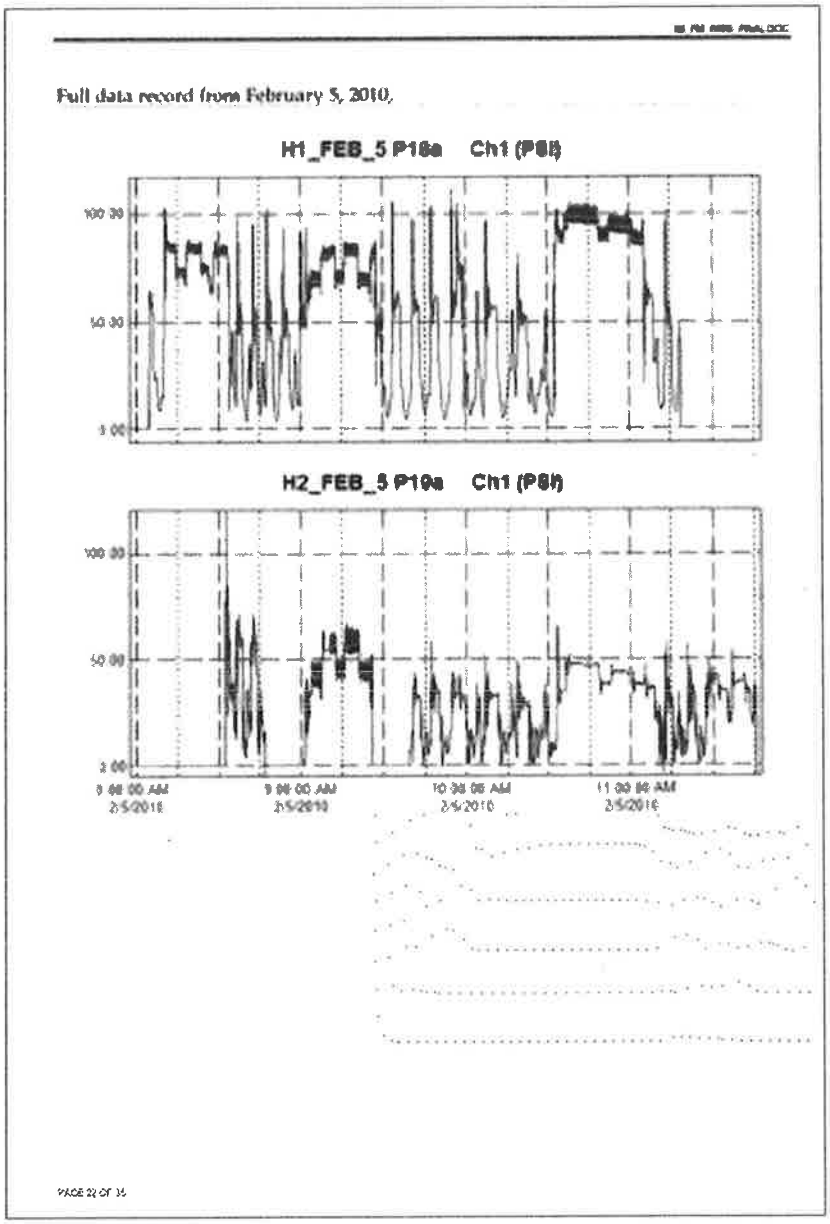
Ultimately the PBS&J report recommends the installation of a 2,000 gallon pressurized bladder surge tank at each of the pump stations B-19, B-20, B-1 and B-6 to prevent the formation of the negative pressures and vapor cavities within the force main. The report goes on to say that installation of bladder surge tanks would adequately mitigate adverse transient pressures during normal startup and shutdown of the pumps.

Expert Opinion of Scott C. Williams, PE on a Construction Project Known as: B-6 Lift Station and Force Main Modifications Brevard County, FL, February 16, 2010

Scott Williams of CH2MHill was asked to evaluate the B-6 Lift Station and Force Main. As part of this evaluation Mr. Williams installed high frequency pressure recorders within the system and recorded a series of pump starts and stops to evaluate the system's transient response. These recordings are critical to understanding the role of adverse transient pressures and the potential for the transient pressures to cause damage or failure to the force main system. Below is a list of findings drawn directly from Mr. Williams report:

- Page 4 of 35, Last Sentence, "Measured pressure data on the B-19 Force Main did not reveal unusual transient pressures or signals"
- Page 11 of 35, First Bulleted Item, "Estimated pump operating points based on measure discharge heads indicate that the pumps are operating normally, and that trapped air/gas pockets are not creating "air binding" on the pipeline."
- Page 19 of 35, Second Paragraph, Last Sentence, "Still, no usual pressure excursions indicating hydraulic shock were measured in either line."
- Page 19 of 35, Last Bulleted Item, "Direct observation of B-19 Force Main in operation at pump stations, valve vaults, and air release manholes on February 5, 2010 did not reveal any auditory or visual indicators or hydraulic shock events within or along the pipeline."

Also from this report the following chart was published showing the recordings of the high frequency transient monitors for February 5, 2010. The H1_FEB_5 P18a recorder was located at the discharge of Lift Station B-19 and monitor H2_FEB_5 P19a was located at various locations along the force main.



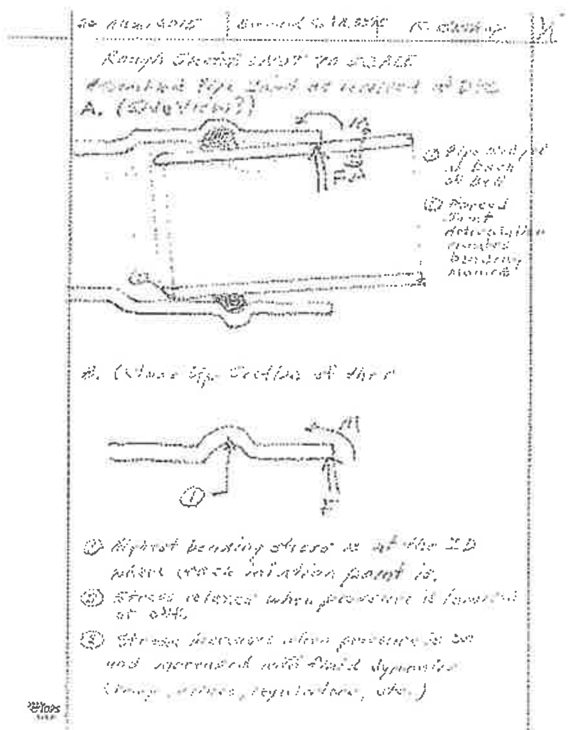
The P18a monitor located at the B-19 Lift Station shows a pressure fluctuation from 0 to 100 psig. This fluctuation in pressure is significant and will cause repetitive dynamic loads (cyclic loads) within the system. However, while the pressure alone is within the allowable limits of the design, the frequency of the pressure fluctuation should be noted because thermoplastic elastomeric materials such as PVC and HDPE are susceptible to fatigue and embrittlement due to work hardening. Cyclic loading also worsens the propagation of cracks within thermoplastic pipe.

Diamond Plastics Corporation Field Report No. FR 3395 Re: Brevard County Utility Department AWWA C 905 DR 25 PVC Sewer Force Main Pipe Samples from Brevard County

The Field Report 3395 submitted by Ron Bishop of Diamond Plastics is an assessment of two failed pipe sections that were shipped to Diamond Plastics for evaluation. The report is summarized on Page 11:

"From our examination of the two samples described in this letter report and the attached formal Field Report #3395, the direct failure is clearly a cyclic fatigue failure. The most likely direct cause of the fractures is the presence of repeated cyclic pressures that combined with a bending force applied at the bell entrance lip by the over assembled spigot that was placed in an angled articulation of the spigot end of the pipe relative to the bell end in order to accommodate curvature to the pipe line as it was installed. A less likely second alternative is that the angle of the change in direction at the joint could have been increased due to localized settlement of the joint if it had not been placed on firm bedding prior to placing the pipe zone material and the subsequent backfill over the pipe before surface loads were applied over the top of pipe."

Mr. Bishop also provided the following sketch as a supplement to the Field Report 3395:



The sketch represents a description of the vertically deflected joint as it was received for testing. The sketch also identifies the force and moment that was identified as the reason for the Bell Split failure.

Pipe Break Analysis

In Figures 1-3, several types of breaks were identified within the evaluated section of force main. The majority of the breaks were identified as a Bell Split. This type of break is identified in the Diamond Plastic Field Report. The Diamond Plastic Field report describes how over-deflection, over-insertion, and applied cyclic loading resulted in the Bell Split failures. However, the reason why the joints were over deflected to a point in which a bending moment was introduced may vary for the following reasons:

- The installer deflected the joint to produce a vertical or horizontal curve to avoid a conflict with a utility, obtain a required lay elevation, or make an alignment change
- The fill material providing structural support around the joint was displaced by fluctuating ground water and migration of fines that weaken the soil support and allowed localized differential settlement
- The fill material was poorly compacted causing differential settlement and over deflection of the joint.
- With three feet of minimum cover the impact loading from heavy equipment could have introduced differential loading resulting in deflection.

In the design of pressurized pipelines utilizing push-on or gasketed joints, it is common to limit the deflection within the pipeline by designing straight runs of pipe on constant slope with varying degrees of fill above the pipe crown to limit the vertical undulation of the pipeline along the alignment. It is also common to limit the allowable joint deflection to some percent (i.e. 50% or 75%) of the manufacturer's specified allowable deflection limit. In design, this ensures that the vertical and horizontal curves designed along the alignment will not introduce excess stress to the push on joint, and as necessary, modifications to the horizontal or vertical alignment during construction may be made. By using 50% of the allowable deflection in design, variations in the alignment can be made in construction while maintaining the deflections within the manufacturer's allowable limits. Two additional types of failures were noted in Figures 1-3. These were "Joint Separation / Slipped Joint" and "Saddle Failed"

The joint separation/slipped joint failures are most likely a result of the following:

- This may be an installation issue resulting from under stabbed or inserted joints installed in hot temperature and covered without allowing for temperature adjustments. Once covered the reduction in temperature will result in significant contraction of PVC pipe. This contraction may result in the pipe pulling away from the joint.
- Gasket or seal failure with longitudinal movement or thrust at unrestrained joints.
    - Shrinkage as described above
    - Differential settlements
    - Movement resulting from thrust at unrestrained joints

Possible reasons for the saddle failures include the following:

- Seal or gasket failure on saddle
- Corrosion of saddle material or bolting assembly causing saddle to release or separate
- Stress concentration at pressure tap leading to longitudinal or fracture 'coupon' failure along pipe segment

Pipe Break Conclusions

Through evaluation of the various reports and record drawings, HDR, Inc has concluded that a portion of the pipeline system should be replaced regardless of the surge modeling results. The multiple Bell Split failures resulting from a combination of over-inserted, over-deflected joints in the presence of cyclic loading will continue to result in failure along the pipeline alignment. Additional surge mitigation may be necessary and may reduce the frequency of failures, but will not resolve the fundamental problem that is resulting in the failures. Consequently, the number of failures over time may be the same even if additional surge mitigation is installed.

HDR, Inc. recommends that USD replace the section of PVC force main on Riverside Drive from Eau Gallie Boulevard to Oakland Avenue. For this replacement the use of USD's standard material for force mains is appropriate. In design of this replacement pipe HDR, Inc. recommends that the following conservative design methodology is used. This includes:

- Limit the allowable deflection in design to 50% of the manufacture's allowable joint deflection,
- Limit the number of vertical and horizontal deflections by using constant pipe slopes and allowing the depth of cover to vary along the alignment,
- Use fittings as necessary and design restraint systems for the working plus transient pressure as a minimum at every joint,
- Provide well-compacted and bedding material for the pipe and consider the use of geo-membrane to maintain the integrity of the soil envelope around the pipe. This will help limit differential settlement from rising and falling groundwater and migration of fines into the pipe zone.

It is also recommended that during construction that experienced inspectors are used to ensure design information and manufacturer's allowable limits are maintained during construction. Considering these conservative design methods and providing detailed inspection during construction will limit the leakage in this replacement.

3.0 Surge Analysis

HDR has collected field data on the operation of the force main system by installing pressure recorders at strategic locations. The recorders were installed at the B-19 lift station and the B-01 lift station. The data collection took place in September and October 2017.

HDR is developing a hydraulic transient model to further analyze issues identified in previous tasks. HDR will provide recommendations and develop an implementation plan based on a hierarchy of importance/priority/cost. The surge data is shown in Appendix A.

4.0 Options Analysis

This report will analyze options for upgrades to the force main system, construction of a re-pump station along the force main route, construction of a new WWTP, and construction of raw wastewater holding tanks. These options will be considered along with conceptual cost estimates for comparison purposes. It is apparent that with some of these long-term options, repair and replacement of the force main on Riverside Drive will still have to be performed. These options could be combined as appropriate as well.

4.1 Force Main Upgrades and B-01 Re-alignment

The section considers upgrades to the portions of the force main system that are PVC and have been experiencing breaks. The USD is completing the replacement of the force main on S. Patrick Drive between Desoto Parkway and Banana River drive in 2017, so that portion is not considered for further analysis. This analysis is focused on the portion of force main on Riverside Drive between Eau Gallie Blvd and Oakland Avenue.

The USD has a current capital improvement project to rehabilitate Lift Station B-01. B-01 is located downstream of B-19 and B-20. The scope includes replacement of existing pumps with larger pumps as well as other miscellaneous modifications. It has been proposed to replace the existing pumps with new pumps that have a design condition of 800 gpm at 191 feet of head. The current pumps are not able to pump down the wet well when lift stations B-19 and/or B-20 are running because the head conditions are too high.

The USD also has a future capital improvement project to take the flow from B-01 off of the B-19 force main system and install a new 24-inch force main to tie into the Lift Station B-06/B-07 force main system. The USD is also considering pipeline improvements upstream and downstream of B-01. Accordingly, the USD has asked HDR to consider the pump upgrades at B-01 in relation to the downstream pipeline improvements.

The following options for the force main system improvements have been evaluated:

1. Increase head on B-01 pumps with no change in downstream force main size
2. Increase head on B-01 pumps, increase force main size to 30-inch when existing 24-inch PVC force main sections on Riverside Drive are replaced
3. Install new 20-inch force main to B-06 as an interconnect to remove flow from S. Patrick force main system
4. Install a parallel 12" force to B-06 to send the B-01 flow to the B-06 system.

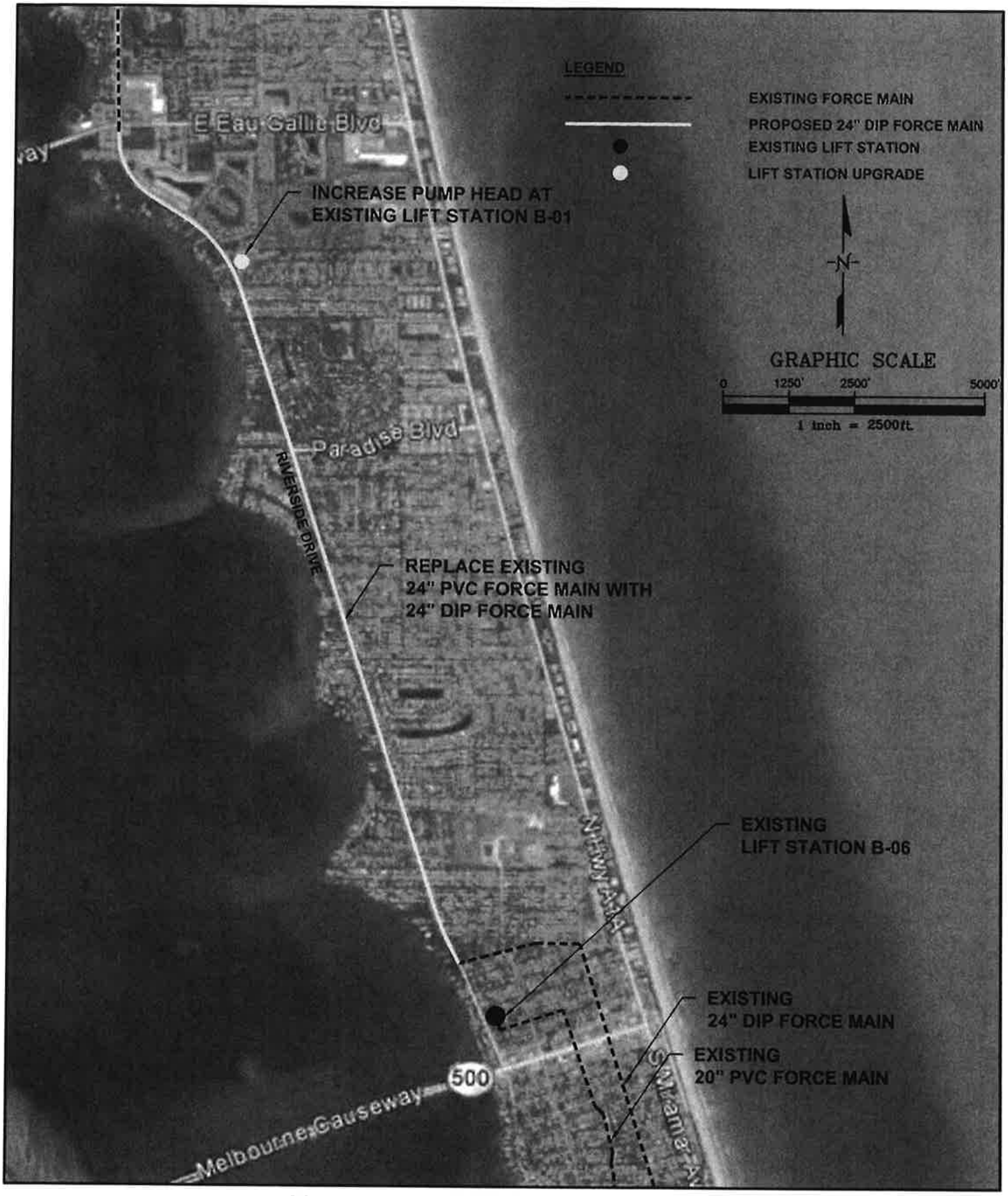
To determine the hydraulic consequences of the above scenarios, HDR has created a hydraulic model of the B-19/B-20/B-01, as well as, the B-06/B-07 force main system. HDR chose EPANET for the model because of the relative simplicity of the system and because it is non-proprietary software. The model assumptions are summarized in Appendix A. The following is a description of each of the four options mentioned above:

- Option 1 – Installing new pumps at the B-01 LS that are able to operate at higher head conditions will allow the lift station to pump down more often. See Figure 4 for location map of this option. The USD has indicated that the current scenario is that the pump station can only get into the system when B-19 or B-20 is off. These higher head pumps at B-01 may cause the existing pumps at B-19 and B-20 to operate at a point on their curve to the left of current operation (slightly less flow, higher head). Overall there would be no significant negative system impacts.

- Option 2- Install new pumps with increased head at B-01 and replace the existing 24-inch PVC force main with 30-inch ductile iron pipe (DIP) on Riverside Drive. This accomplishes Option 1 (allow the B-01 lift station to pump down more often), and also reduces the velocity and head (pressure) in the long run of force main. The run of pipe expected to be replaced runs from Banana River Drive south to the intersection of Oakland Ave and Riverside Drive, a distance of approximately 3.5 miles. Figure 5 shows this pipeline alternative. This would result in lower velocities in this new stretch of force main, and this could translate to somewhat reduced pumping costs. However, the remainder of the force main downstream of this section of proposed 30-inch force main that runs to the wastewater treatment plant would still be 24-inch, and reducing the diameter of a force main on the downstream side is not standard practice.

- Option 3 – Install a new 20-inch force main section from the B-19/B-20/B-01 force main near Oakland Ave to the B-06/B-07 force main system, just downstream of the B-06 pump station, in order to reduce the flow in the B-19 force main system by directing it into the B-06/B-07 force main system. This would reduce the amount of flow in the B-19/B-20/B-01 force main system, which would lessen the velocities in the force main system thereby reducing losses to friction. This would translate in to some energy savings in terms of pumping operations; however, to accomplish this a new 20-inch force main 1,200 feet in length would have to be constructed. . As an Option 3 alternative, the 20-inch force main connection between the B-19/B-20/B-01 force main system and the B-0/B-07 force main system could be moved slightly downstream to connect from the B-19/B-20/B-01 system's 24-inch on N Shannon Ave, run along 2nd Avenue approximately 680 feet, and connect to the B-06/B-07 system's 20-inch force main as it turns south at N Palm Ave. Both of these options are shown in Figure 7. This alternative would shorten the 20-inch interconnect between the two force mains and move the construction from a major road, N Riverside Dr, and move it to a smaller residential street that may permit closure to through traffic during construction.

- Option 4 - Install new 12" force main from B-01 Lift Station to B-06 force main system in order to remove flow from the B-19 force main system. This option is shown in Figure 6. This would reduce the amount of flow in the B-19 force main system, which would lessen the velocities in the force main system thereby reducing losses to friction. This would translate in to some



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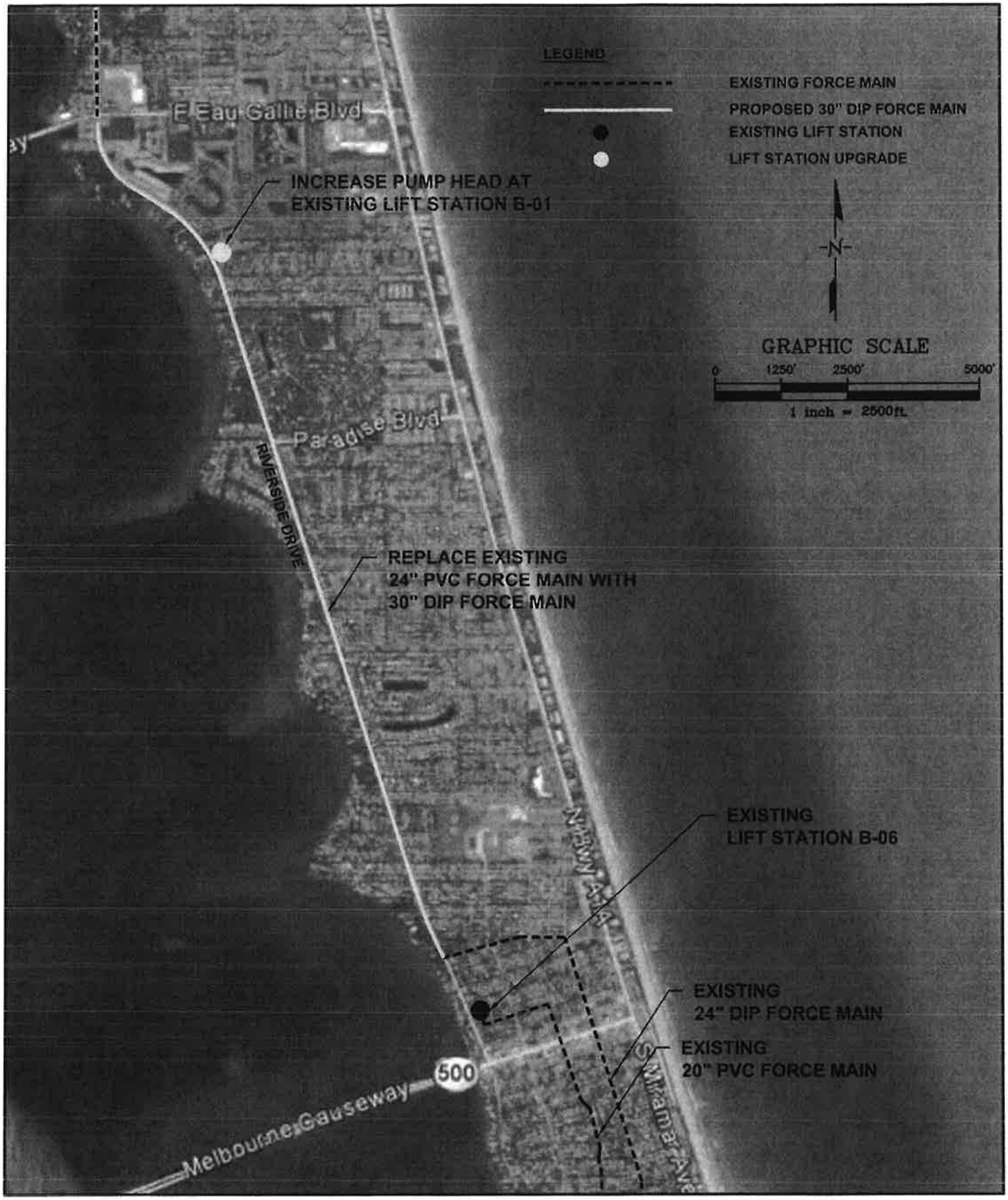
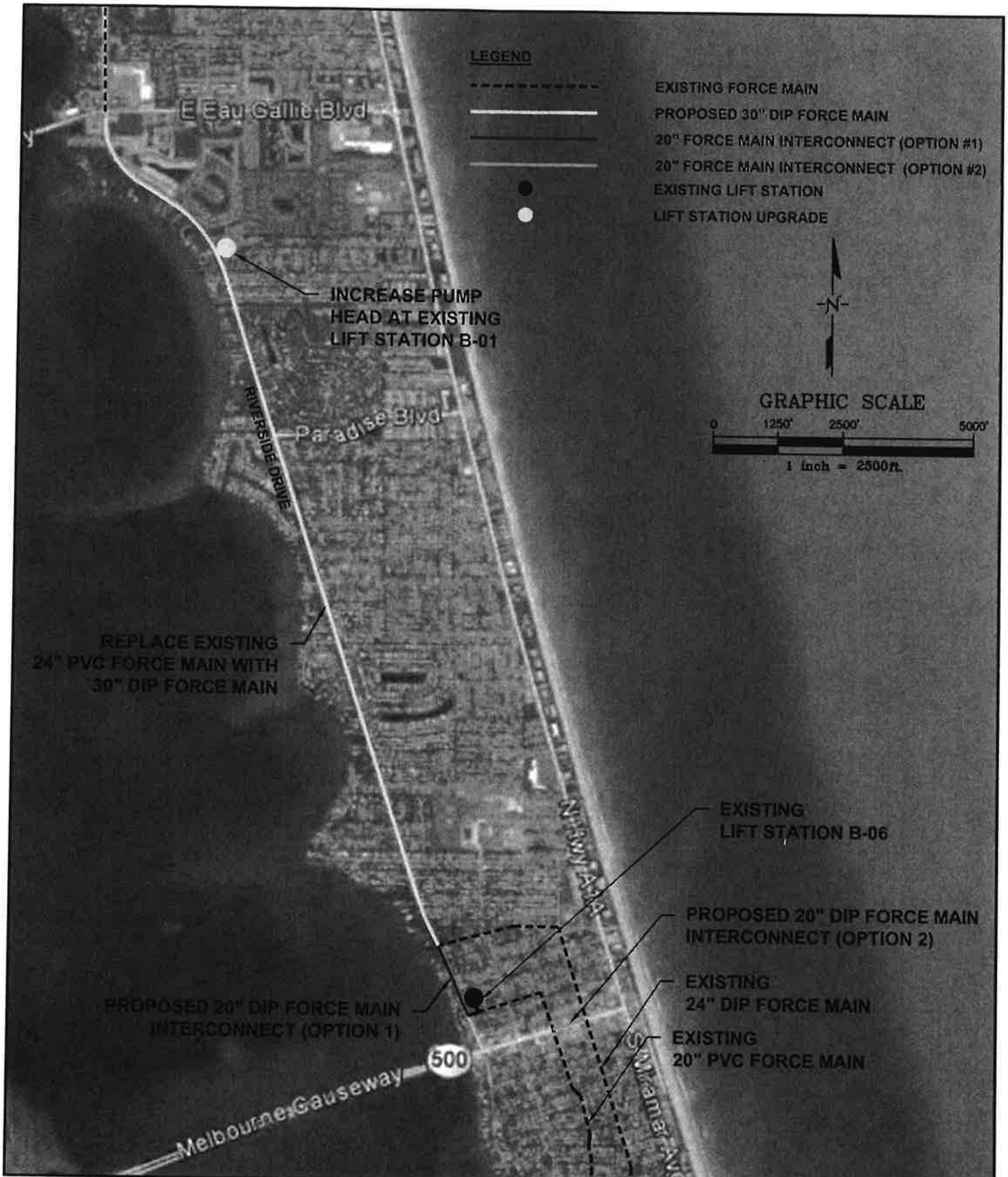
PROJECT TITLE SOUTH PATRICK RIVERSIDE FORCE MAIN IMPROVEMENTS		PROJECT NUMBER 10049527	REFERENCE SHEET -
SHEET TITLE **FIGURE-4** UPGRADE PUMPS AT B-01 PUMP STATION		PROJECT MANAGER J.CRIGLER	REFERENCE DOCUMENT -
		DATE 11/17	EXHIBIT NUMBER 4



PROJECT TITLE	PROJECT NUMBER	REFERENCE SHEET
SOUTH PATRICK RIVERSIDE FORCE MAIN IMPROVEMENTS	10049527	-
SHEET TITLE	PROJECT MANAGER	REFERENCE DOCUMENT
FIGURE-5 UPGRADE PUMPS AT B-01 PUMP STATION AND UPSIZE FORCE MAIN TO 30"	J.CRIGLER	-
	DATE	EXHIBIT NUMBER
	11/17	5

HDR



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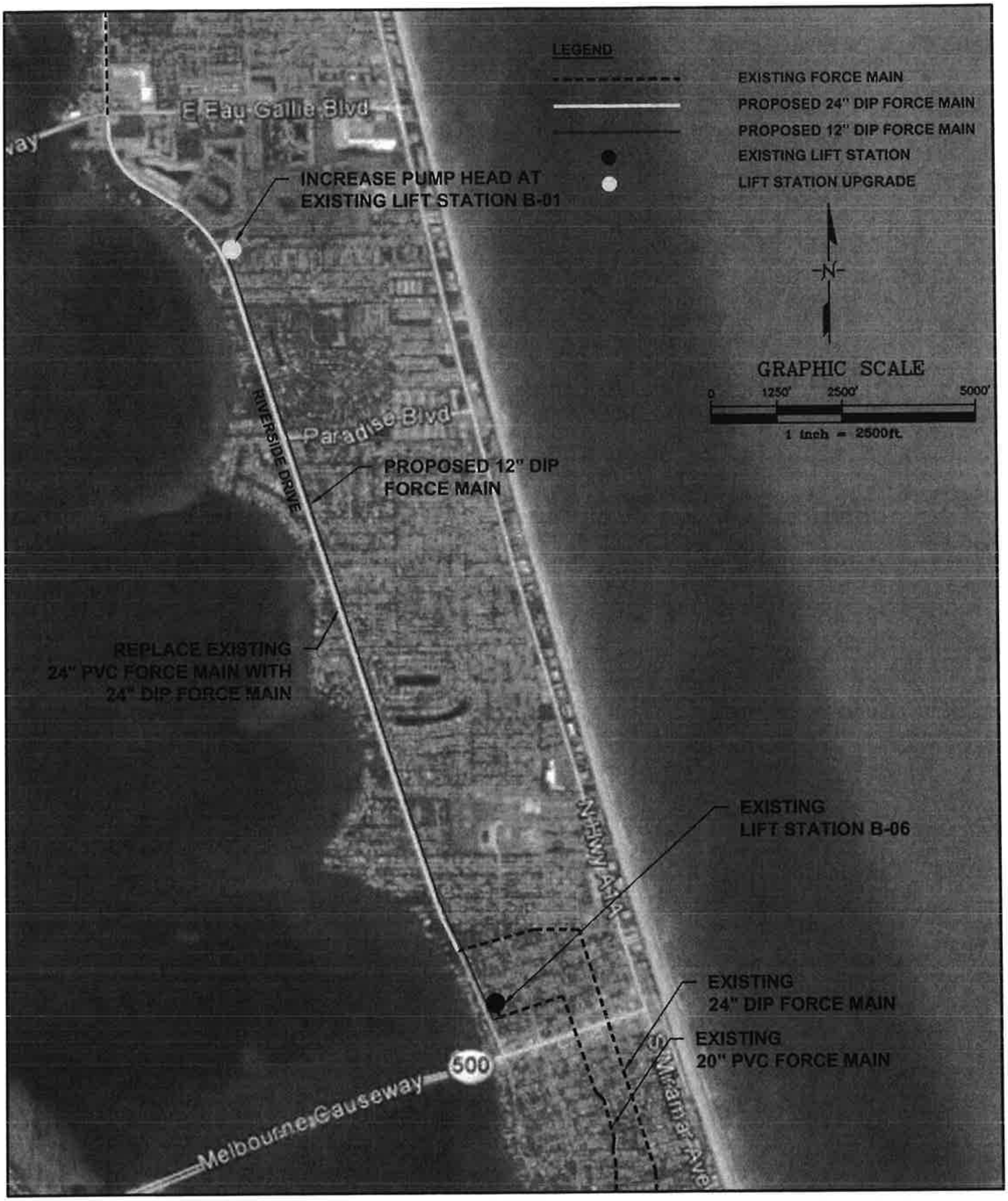
PROJECT TITLE SOUTH PATRICK RIVERSIDE FORCE MAIN IMPROVEMENTS	PROJECT NUMBER 10049527	REFERENCE SHEET -
SHEET TITLE **FIGURE-6** UPGRADE PUMPS AT B-01 PUMP STATION, UPSIZE FORCE MAIN TO 30", AND INSTALL 20" INTERCONNECT FORCE MAIN	PROJECT MANAGER J.CRIGLER	REFERENCE DOCUMENT -
	DATE 11/17	EXHIBIT NUMBER 6



HDR	PROJECT TITLE SOUTH PATRICK RIVERSIDE FORCE MAIN IMPROVEMENTS	PROJECT NUMBER 10049527	REFERENCE SHEET -
	SHEET TITLE **FIGURE-7** UPGRADE PUMPS AT B-01 PUMP STATION AND INSTALL 12" FORCE MAIN TO B-06	PROJECT MANAGER J.CRIGLER	REFERENCE DOCUMENT -
		DATE 11/17	EXHIBIT NUMBER 7

energy savings in terms of pumping operations; however, to accomplish this a new 12" force main 3 miles long would have to be built at a cost of approximately $1.5 million. This large capital expenditure would not justify the minor energy savings realized by the new 12" force main. The cost mentioned above does not include any upgrade costs at B-06 or B-07. This option was not modeled and is considered cost prohibitive because of the marginal benefits obtained at a substantial capital cost.

System Scenarios

Both high head and low head conditions (scenarios) were analyzed in order to determine the possible operational total dynamic head (TDH) range of the B01 pump station system. The TDH range was utilized to select efficient pumps to operate at the desired flow rate. Design flow rates of 800 gpm and 1,000 gpm were analyzed for the high head conditions. The design flow rate of 800 gpm was analyzed for the low head conditions, as this would generate a lower head than the 1,000 gpm alternative. Three system alternatives were analyzed:

1. Existing Condition,
2. Upsize of the 24-inch PVC force main sections to 30-inch,
3. Upsize of the 24-inch PVC force main sections to 30-inch with a 20-inch interconnect to B-06/B-07 force main system.

Existing Conditions

The existing condition scenarios represent the system as it is today with B-01 contributing flow changed to 800 gpm and 1,000 gpm for respective scenarios. In this scenario, the existing 24-inch PVC sections along N Riverside Dr are kept as is. For one sub-scenario, both B19 and B20 triplex stations were allowed to operate with two pumps running, each, and, thus, generated flow based on the head condition of the system. For comparison, another sub-scenario was run where the B-19 and B-20 pump stations were only permitted to generate flow equal to their design flow and no more, imitating variable frequency drive condition limit.

Future: 30-inch Force Main

Under this system scenario, the existing 24-inch PVC sections along N Riverside Dr are upsized to 30-inch ductile iron pipe (DIP). A total of 19,712 linear feet (LF) of 24-inch PVC was replaced with 30-inch DIP in this scenario. Both B19 and B20 triplex stations were allowed to operate with two pumps running, each, and, thus, generated flow based on the head condition of the system.

Future: 30-inch Force Main with 20-inch Interconnect

Under this system scenario, the existing 24-inch PVC sections along N Riverside Dr are upsized to 30-inch ductile iron pipe (DIP). A total of 19,712 linear feet (LF) of 24-inch PVC was replaced with 30-inch DIP in this scenario. Additionally, a 1,200 LF, 20-inch force main was added along N Riverside Dr from Oakland

Avenue to Second Avenue connecting the B-19/B-20/B-01 24-inch force main to the B-06/B07 20-inch force main. Both B19 and B20 triplex stations were allowed to operate with only one pump running, each, and, thus, generated flow based on the head condition of the system. Only one pump, each, was allowed to operate at B-19 and B-20 such that the stations would generate flow similar to their design condition. The lower head conditions at these stations resulting from the system improvements permitted the stations to run on one pump and still produce close to if not more than the design flow. For the high head scenarios, B-06 and B-07 pump stations were constrained to pump at their design flow rates of 1,940 gpm and 1,375 gpm, respectively. The inclusion of the 20-inch interconnect results in additional flow being conveyed through the B-06/B-7 force main system, the head condition at these pump stations would not allow the existing pumps to generate their design flow. If the interconnect is implemented in the County's system an in depth review of these stations will be needed.

An alternative system scenario was analyzed where the 20-inch interconnect alignment was moved from N Riverside Dr to 2nd Avenue. The 680 LF, 20-inch interconnect is further downstream from the alignment proposed along N Riverside Dr and thus the overall head condition as seen from the B-19/B-20/B-01 force main system pump stations are slightly higher, however, the difference is fairly negligible.

Results

The results of the EPAnet modeled scenarios described below are provided in Table 2 and Table 3 for high head and low head scenarios analyzed, respectively. As would be expected, replacing the 24-inch PVC force main sections in the B-19/B-20/B-01 force main system lowers the system head allowing the existing pumps at the stations to pump a greater flow. With the interconnect in place B-19/B-20/B-01 force main system head drops even lower, allowing for the shutdown of one pump at each of the major stations (B-19 and B-20) while allowing them to pump at or near their design flow rate. Under existing conditions where B-01 is designed to convey 800 gpm the B-01 proposed pumps would have to be able to pump at between 87 and 218 ft of TDH. Likewise, where the 24-inch PVC force main sections are replaced with 30-inch DIP the TDH range as seen at the B-01 pump station could range between 79 and 186 ft at a flow rate of 800 gpm. With the inclusion of the 20-inch interconnect to the B-06/B-07 force main system along N Riverside Dr, the system head at B-01 drops even further to a range of between 62 and 138 ft of TDH. Head conditions for the other major stations, B-19, B-20, B-06, and B-07 are also provided in Table 2 and Table 3, as well as, the flow conveyed from the B-19/B-20/B-01 force main system into the B-06/B-07 force main system via the 20-inch interconnect.

Future Surge Analysis

As the B-01 pump station is upgraded, it is recommended that the pressure monitoring and surge modeling be performed with the new pumps selected for B-01, and taking in to account the pipeline recommendations mentioned in this report. This will provide the most accurate representation of future surge conditions.

Brevard County Riverside Drive 24" FM Replacement
0% Conceptual Cost Estimate

ITEM Number	SUMMARY OF PAY ITEMS	QUANTITY	UNIT	AVERAGE UNIT PRICE	TOTAL AMOUNT
1	Mobilization	1	LS	$80,000.00	$80,000.00
2	Layout and Asbuilt	1	LS	$41,760.00	$41,760.00
3	Erosion Control	1	LS	$17,500.00	$17,500.00
4	Dewatering	1	LS	$100,000.00	$100,000.00
5	24" Ductile Iron FM	17700	LF	$230.00	$4,071,000.00
6	24" x 24" Tapping Sleeve & Valve	2	EA	$50,000.00	$100,000.00
7	24" Gate Valve	8	EA	$32,000.00	$256,000.00
8	2" Air Release Valve	4	EA	$11,300.00	$45,200.00
9	24" 45° DI Elbow	20	EA	$7,000.00	$140,000.00
10	24" 11.25° DI Elbow	8	EA	$6,500.00	$52,000.00
11	Canal/Ditch Crossing	2	EA	$80,000.00	$160,000.00
12	Lift Station Connections	4	EA	$45,000.00	$180,000.00
13	Demo and Remove Concrete Sidewalk	20000	SF	$1.25	$25,000.00
14	Sidewalk Replacement	20000	SF	$5.50	$110,000.00
15	Roadway Cut/Patch	1200	SY	$110.00	$132,000.00
16	Concrete Driveway	600	SY	$50.00	$30,000.00
17	Construct Curbing	800	LF	$40.00	$32,000.00
18	Striping	1	LS	$4,000.00	$4,000.00
19	Abandon 24" PVC FM	17700	LF	$6.00	$106,200.00
20	Remove and Replace Pipe Bedding	300	CY	$24.00	$7,200.00
21	Remove and Replace Pipe Backfill	300	CY	$26.00	$7,800.00
22	Seeding	3000	SY	$2.00	$6,000.00
23	Sodding	7000	SY	$4.00	$28,000.00
24	Landscaping	1	LS	$50,000.00	$50,000.00
25	Density Testing	1	LS	$27,200.00	$27,200.00
26	Pressure Testing	1	LS	$12,000.00	$12,000.00
27	Maintenance of Traffic	1	LS	$17,500.00	$17,500.00
	Sub-total				$5,838,360.00
	Contingency (20%)				$1,167,672.00
	TOTAL*				**$7,006,032.00**

Brevard County Riverside Drive 30" FM Replacement
0% Conceptual Cost Estimate

ITEM Number	SUMMARY OF PAY ITEMS	QUANTITY	UNIT	AVERAGE UNIT PRICE	TOTAL AMOUNT
1	Mobilization	1	LS	$40,000.00	$40,000.00
2	Layout and Asbuilt	1	LS	$41,760.00	$41,760.00
3	Erosion Control	1	LS	$12,500.00	$12,500.00
4	Dewatering	1	LS	$110,000.00	$110,000.00
5	30" Ductile Iron FM	17700	LF	$245.00	$4,336,500.00
6	30" x 24" Tapping Sleeve & Valve	2	EA	$56,000.00	$112,000.00
7	30" Gate Valve	8	EA	$42,000.00	$336,000.00
8	2" Air Release Valve	4	EA	$9,000.00	$36,000.00
9	30" 45° DI Elbow	20	EA	$8,500.00	$170,000.00
10	30" 11.25° DI Elbow	8	EA	$7,500.00	$60,000.00
11	Canal/Ditch Crossing	2	EA	$90,000.00	$180,000.00
12	Lift Station Connections	4	EA	$55,000.00	$220,000.00
13	Demo and Remove Concrete Sidewalk	20000	SF	$1.25	$25,000.00
14	Sidewalk Replacement	20000	SF	$5.50	$110,000.00
15	Roadway Cut/Patch	1200	SY	$110.00	$132,000.00
16	Concrete Driveway	600	SY	$50.00	$30,000.00
17	Construct Curbing	800	LF	$40.00	$32,000.00
18	Striping	1	LS	$4,000.00	$4,000.00
19	Abandon 24" PVC FM	17700	LF	$8.00	$141,600.00
20	Remove and Replace Pipe Bedding	300	CY	$24.00	$7,200.00
21	Remove and Replace Pipe Backfill	300	CY	$26.00	$7,800.00
22	Seeding	3000	SY	$2.00	$6,000.00
23	Sodding	7000	SY	$4.00	$28,000.00
24	Landscaping	1	LS	$50,000.00	$50,000.00
25	Density Testing	1	LS	$27,000.00	$27,000.00
26	Pressure Testing	1	LS	$12,000.00	$12,000.00
27	Maintenance of Traffic	1	LS	$17,500.00	$17,500.00
	Sub-total				$6,284,860.00
	Contingency (20%)				$1,256,972.00
	TOTAL*				**$7,541,832.00**

4.2 New Re-pump Station

As an alternative option to reducing the force main system pressure, consideration was given to replacing either Lift Station B-20 or Lift Station B-01 and construction a new re-pump station. In order to determine the viability of this option, a site analysis was done first to determine if adequate space was available to accommodate a larger pump station and an electrical building. Based on the initial site analysis the determination was made that the B-01 lift station site lacks the space needed to accommodate a new re-pump station. In addition, the B-01 lift station site is located in a visible residential area and the aesthetics of the new station would most likely cause issues. The existing B-20 lift station site was evaluated and determined to have the space required to consider the construction of a re-pump station for the system. In addition, the B-20 site is located off of the main road in an area that would not cause aesthetic concerns.

B-20 Re-Pump Station Consideration

Currently, the B-20 lift station discharges through a 20" force main and manifolds in to the existing 24" force main trunk line that run down South Patrick drive. Approximately 3.5 miles upstream of the B-20 lift station tie-in location is Lift Station B-19. If B-20 were converted into a re-pump station then Lift Station B-19 would pump directly into the new B-20 Lift Station. The new B-20 lift station would then re-pump all flow approximately 9 miles south to the South Beaches Regional WWTP. The potential benefit of having the flow from B-19 re-pumped from Lift Station B-19 would be the reduction in total head seen at both lift stations, reduction in overall energy consumption, and the reduction of overall system pressure. In order to determine the extent of these benefits, a system model is being developed. Once the modeling results are complete a determination can be made as to whether or not the benefits of this alternative is viable.

HDR	PROJECT TITLE SOUTH PATRICK RIVERSIDE FORCE MAIN IMPROVEMENTS	PROJECT NUMBER 10049527	REFERENCE SHEET -
	SHEET TITLE **FIGURE-8** CONVERT LIFT STATION B-20 TO RE-PUMP LIFT STATION	PROJECT MANAGER J.CRIGLER	REFERENCE DOCUMENT -
		DATE 11/17	EXHIBIT NUMBER 8

4.3 New WWTP at B-20 Site

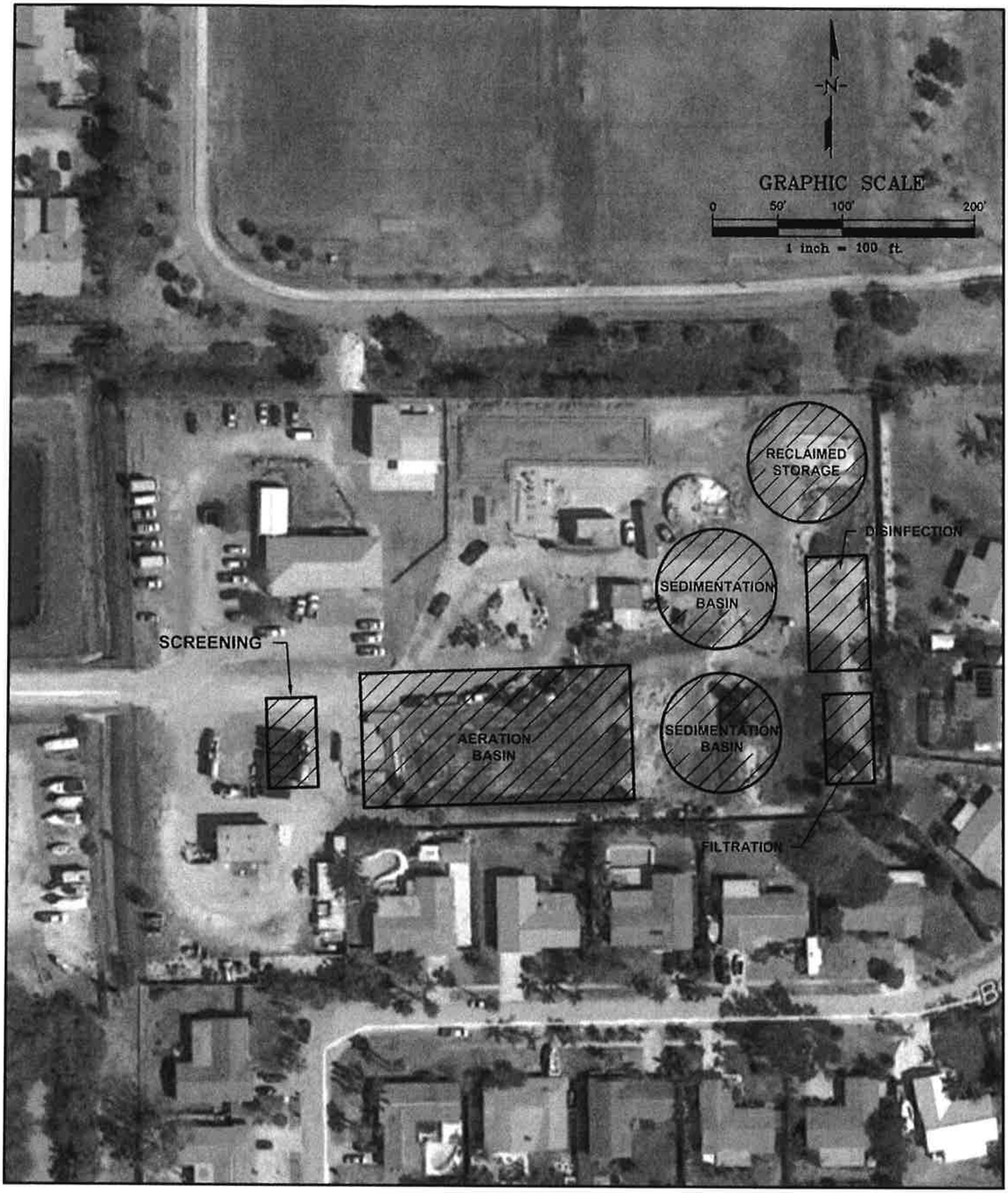
The USD has considered the option of constructing a new wastewater treatment plant in the South Beaches service area that could produce reclaimed water for the customers located on the island, as well as eliminate the need for a pipeline carrying raw wastewater. The concept is to develop the property currently owned by USD at the B-20 lift station site into a new wastewater treatment plant (WWTP) with the capability to produce reclaimed water as an effluent disposal method. The treated effluent would then be pumped into the existing wastewater pipelines from B-20 to the South Beaches WWTP. The reclaimed water could be utilized by customers along that route. Any reclaimed wastewater not delivered to customers would be sent to the South Beaches WWTP for disposal in the existing deep injection well(s). The following system improvements would be required for this alternative:

- Construct 12" FM from B-01 to B-06 FM (in existing 24" HDPE conduit). This would allow the raw wastewater from the B-01 basin to get to the South Beached WWTP
- Construct 6.0 MDG treatment plant at IHB site using similar design/layout from South Central WWTP. This will provide consistency with operators and plant personnel
- Divert flows from B-19 and B-20 to new plant. The flow for B-20 already comes to the proposed WWTP site, and all the flow from B-19 could be diverted with a short force main connection from S. Patrick Drive to the new plant influent
- Pump reclaimed water using the existing 24" Riverside FM to the South Beaches WWTP. While this pipeline currently has issues with main breaks, it would operate at a lower pressure than current operating pressure. In addition, spills from a reclaimed line into the Indian River Lagoon would be less harmful than a raw wastewater spill.
- Connect Riverside FM to existing injection well. Most reclaimed water is injected into a 2900 ft deep injection well at the treatment plant site
- Connect S-06 & S-07 pumping stations to the B-19 FM eliminating re-pumping, and eliminating need for upgrades to FM from S-06 to B-19
- Repurpose B-20 lift station to reclaimed water pump station. The influent

Currently Seven million gallons of wastewater is reclaimed every day at the South Beach plant and fewer than two million gallons are used for beneficial purposes. Additional capacity at South Beach plant provides opportunity for sewer service all the way to Sebastian Inlet (septic tank elimination). The following table lists some advantages/disadvantages of a new WWTP at this location.

Advantage	Disadvantages
Makes reclaimed water available from Satellite Beach to South Melbourne Beach	High Capital Cost
Lowers pressures on beachside force mains	Proximity to ocean
Adds treatment capacity to beachside system (4.5 MGD required to complete service area)	Vulnerable to Storms & Storm Surge
Reclaimed water trunk line and reclaimed pumping station already in place (Repurposed Riverside FM and B-20 pumping station)	Elevation concerns (flooding)
Creates "fail-safe" for Riverside force main break (reclaimed water – not sewage)	No pipelines to distribute the water to users

A preliminary layout of a new WWTP is shown in Figure 9. This layout is similar to the technology used at the S. Beaches plant, which will allow ease of operation for existing staff. Also, a conceptual cost estimate has been developed and is shown in Table 6.



PROJECT TITLE SOUTH PATRICK RIVERSIDE FORCE MAIN IMPROVEMENTS		PROJECT NUMBER 10049527	REFERENCE SHEET -
HDR	SHEET TITLE **FIGURE-9** B-20 LIFT STATION SITE WASTEWATER TREATMENT PLANT OPTION	PROJECT MANAGER J.CRIGLER	REFERENCE DOCUMENT -
		DATE 11/17	EXHIBIT NUMBER 9

New WWTP at B-20 Site
0% Conceptual Cost Estimate

ITEM Number	SUMMARY OF PAY ITEMS	QUANTITY	UNIT	AVERAGE UNIT PRICE	TOTAL AMOUNT
1	Design 6.0 MGD Plant	1	LS	$1,600,000.00	$1,600,000.00
2	6.0 MGD Plant	1	LS	$21,000,000.00	$21,000,000.00
3	12" FM from B-01 to B-06	1	LS	$1,800,000.00	$1,800,000.00
4	Piping at South Beaches Plant	1	LS	$350,000.00	$350,000.00
5	Piping Changes at B-20	1	LS	$150,000.00	$150,000.00
6	S-6 & S-17 Piping to B-19 FM	1	LS	$200,000.00	$200,000.00
7	B-20 conversion to Reclaimed Water PS	1	LS	$250,000.00	$250,000.00
	Sub-total				$25,350,000.00
	Contingency (15%)				$3,802,500.00
	TOTAL*				**$29,152,500.00**

4.4 Off-Line Wastewater Storage at B-20

The South Beaches collection system experiences a large amount of inflow/infiltration. The gravity collection system contains significant amounts of vitrified clay gravity pipe that has reached its design life. Significant storm events place a burden on the system, and the existing pump station and force main network becomes overburdened during large storm events. This can lead to SSO's. In addition to the system upgrades mentioned as options in this report, off-line storage of raw wastewater at the B-20 lift station site would provide some operational flexibility during periods of high flows and could mitigate SSO's daring these events. It would also provide additional flexibility when the station or force main systems requires being taken out of service for maintenance or repair. The B-20 lift station has insufficient wetwell storage, and the USD indicated that it overflows within 20 minutes of outage

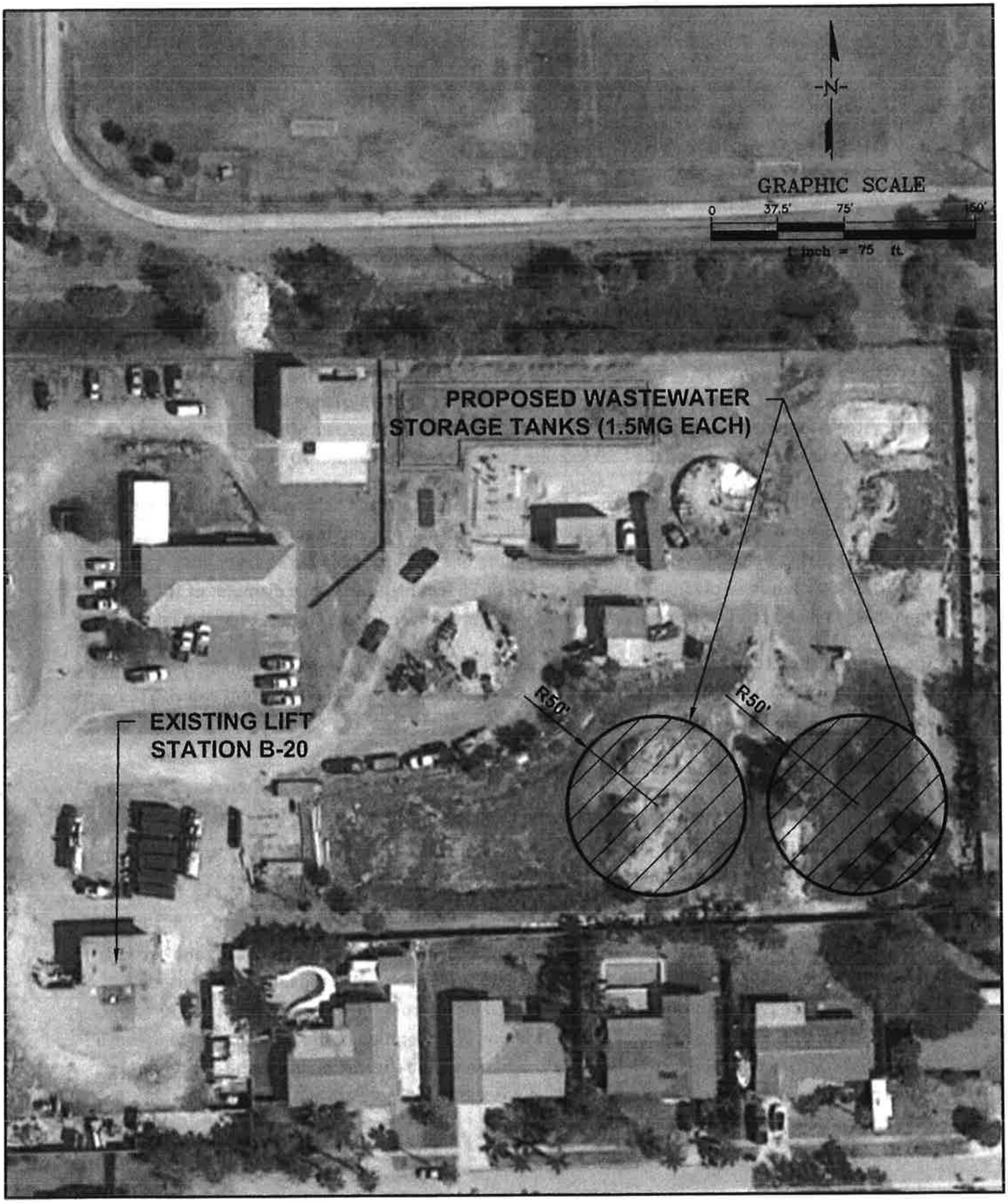
The B-20 lift station site has adequate room for construction of storage holding tanks. Based on the flows to B-19 and B-20 lift stations, two holding tanks of 1.5 million gallons each could be constructed on the B-20 site. This would provide approximately one to one and an half day of storage. The approximate dimension s of tanks of this size would be 25 feet high and with a diameter of 100 feet. The tanks would be pre=stressed concrete tanks which are proven for the storage of wastewater.

The operational protocol for these tanks would be that they are always emptied completely each time they are used. This ensures that storage capacity is available for the next rainfall event and also prevents consolidation of sludge in the base of the tank.

Such tanks should incorporate the following features:

- Facilities for re-suspension of solids as the tanks are drawn down,
- Tank mixer unit or air/water scour pumps;
- Facilities for cleaning of the tank floor on draw-down of the contents, so sludge can be automatically drawn off from the conical sump at the invert of the tank.

Raw wastewater storage will present some fundamental challenges, including aesthetics and securing support from the neighborhoods adjacent to the B-20 site. Even though the tanks will be used infrequently, these types of projects can be met with concern from the nearby residents. Also, the location of the site, while large enough for construction and operation of the tanks, is not ideal from a vulnerability perspective. The proximity to the ocean makes the site vulnerable to the effects of hurricanes and storm surge. A proposed site layout is shown in Figure 10, and a conceptual cost estimate has been developed and is shown in Table 7 .



HDR

PROJECT TITLE SOUTH PATRICK RIVERSIDE FORCE MAIN IMPROVEMENTS

SHEET TITLE
FIGURE-10
B-20 LIFT STATION SITE
WASTEWATER STORAGE TANKS

PROJECT NUMBER 10049527

PROJECT MANAGER J.CRIGLER

DATE 11/17

REFERENCE SHEET -

REFERENCE DOCUMENT -

EXHIBIT NUMBER 10

Wastewater Storage at B-20
0% Conceptual Estimate

ITEM Number	SUMMARY OF PAY ITEMS	QUANTITY	UNIT	AVERAGE UNIT PRICE	TOTAL AMOUNT
1	Earthwork	1	LS	$500,000.00	$500,000.00
2	1.5 Million Gallon Concrete Tank	2	EA	$1,750,000.00	$3,500,000.00
3	Site Piping	1	LS	$450,000.00	$450,000.00
4	Valves	1	LS	$250,000.00	$250,000.00
5	Site Work	1	LS	$175,000.00	$175,000.00
	Sub-total				$4,875,000.00
	Contingency (15%)				$731,250.00
	TOTAL*				**$5,606,250.00**

5.0 Recommendations

The proposed re-pump station is a viable alternative if Lift Station B-20 were at the end of its design life. This scenario will lower the head in the system somewhat, but does not negate the need to upgrade the force main on Riverside Drive. Any energy savings realized by lower head pumps at B-19 would be offset by the energy requirement to re-pump the wastewater at B-20.

Constructing a new WWTP at the B-20 site would provide beneficial reuse to customers at the beach, but the high capital cost of this option makes it impractical at this time. There are also concerns about locating another WWTP in an area vulnerable to storms and storm surge.

Raw wastewater storage tanks at the B-20 site would provide additional flexibility for operation of the system during significant rain events and for maintenance/repair, but there are significant challenges to the siting of these tanks relative to the surrounding neighborhoods and the vulnerability of the site location. Furthermore these tanks would not negate the need to upgrade the force main on Riverside Drive either.

Therefore, based on the analysis of the pipe break data and the hydraulic modeling results, HDR recommends at this time that the **USD replace the section of 24" force main on Riverside Drive from Banana River Drive south to the intersection of Oakland Ave and Riverside Drive, with 30" DI force main. In addition, based on the results of the surge analysis report pressure surge tanks should be installed at Lift Station B-19 and Lift Station B-20.** If additional funding becomes available, the USD should re-visit the options for off-line storage or a re-pump station, as these would improve operational flexibility.

Furthermore, if additional operational flexibility is desired, USD should consider constructing the interconnect between the two force main systems. This would require the upgrade of Lift Stations B-06 and B-07 but would improve the flow conditions in the B-19 system, while also providing operational flexibility for main breaks or force main maintenance/repair. The interconnect could be operated in either the normally closed or normally open condition, depending on the desired outcome. If intended to alleviate high pressures in the B-19 system, it could be operated in the normally open condition, whereas if it is intended to be used as an alternate transmission main in certain scenarios then the valve would be normally closed.

APPENDIX

Existing System Scenario: Major Components

Links: Pipes and Pumps

Pipes:

- B19_InfluenPipe: Gravity line flowing into B19. The length value of 1,000 LF does not affect calculations.
- 20" DIP: Force main just down stream of B19 and runs from B19 and south along Riverside Drive.
- 20" PVC: Force main section down stream of 20" DIP and runs south along Riverside Drive.
- B20_Inflow_16": Gravity line flowing into B20. The length value of 1,000 LF does not affect calculations.
- B20_18"FM: Force main section from B20 that ties into the 20"/24" at Riverside Drive.
- 24"_PVC_1: Force main section down stream of B20 tie in and runs south along Riverside Drive.
- 24"_DIP: Force main section down stream of 24"_PVC_1 and runs south along Riverside Drive.
- 24"_PVC_2: Force main section down stream of 24"_DIP and runs south along Riverside Drive.
- B1_18"GravitySewer: Gravity line flowing into B1. The length value of 1,000 LF does not affect calculations.
- 12"DIP_B1: Force main section from B1 that ties into the 20"/24" at Riverside Drive.
- 24"_PVC_3: Force main section down stream of 24"_PVC_2 and runs south along Riverside Drive.
- 24"_DIP_2: Force main section down stream of 24"_PVC_3 and runs south along Riverside Drive to the SBWRF.

Table 1 Force Main Attributes

Force Main Section	Minor Loss Coefficient (K)	Hazen-William Friction Coefficient (C)	Material	Diameter (inch)	Length (Ft), Model
20" DIP	12.00	100	DIP	20	15,491
20" PVC	1.75	110	PVC	20	2,981
B20_18"FM	0.00	100	DIP	18	1,000
24"_PVC_1	2.75	110	PVC	24	2,205
24"_DIP	2.75	100	DIP	24	5,144
24"_PVC_2	3.00	110	PVC	24	3,590
12"DIP_B1	0.00	100	DIP	12	150
24"_PVC_3	10.50	110	PVC	24	13,917
24"_DIP_2	19.50	100	DIP	24	25,730

Note: Influent gravity lines excluded from table, attributes are non-real world and do not affect model results.

Pumps:

- B1_P1: Lead pump for B1 pump station ABS AFP 1501310 mm, set open.
- B1_P2: Lag pump for B1 pump station, ABS AFP 1501310 mm, set closed as B1 is a duplex station.
- B19_P1: Lead pump for B19 pump station FLYGT C3231-63-430, set open.
- B19_P2: Lag pump for B19 pump station FLYGT C3231-63-430, set open as B19 is a triplex station.
- B19_P3: Lag-lag pump for B19 pump station FLYGT C3231-63-430, set open as B19 is a triplex station.
- B20_P1: Lead pump for B20 pump station FLYGT C3231-63-430, set open.
- B20_P2: Lag pump for B20 pump station FLYGT C3231-63-430, set open as B20 is a triplex station.
- B20_P3: Lag-lag pump for B20 pump station FLYGT C3231-63-430, set open as B20 is a triplex station

Wet Wells:

- B1_WetWell: B1 wet well, depth elevation and low (4.5 ft) and high (11.5 ft) water levels set based on asbuilts.

- B19_WetWell: B19 wet well, depth elevation and low (4.0 ft) and high (11.0 ft) water levels set based on asbuilts.

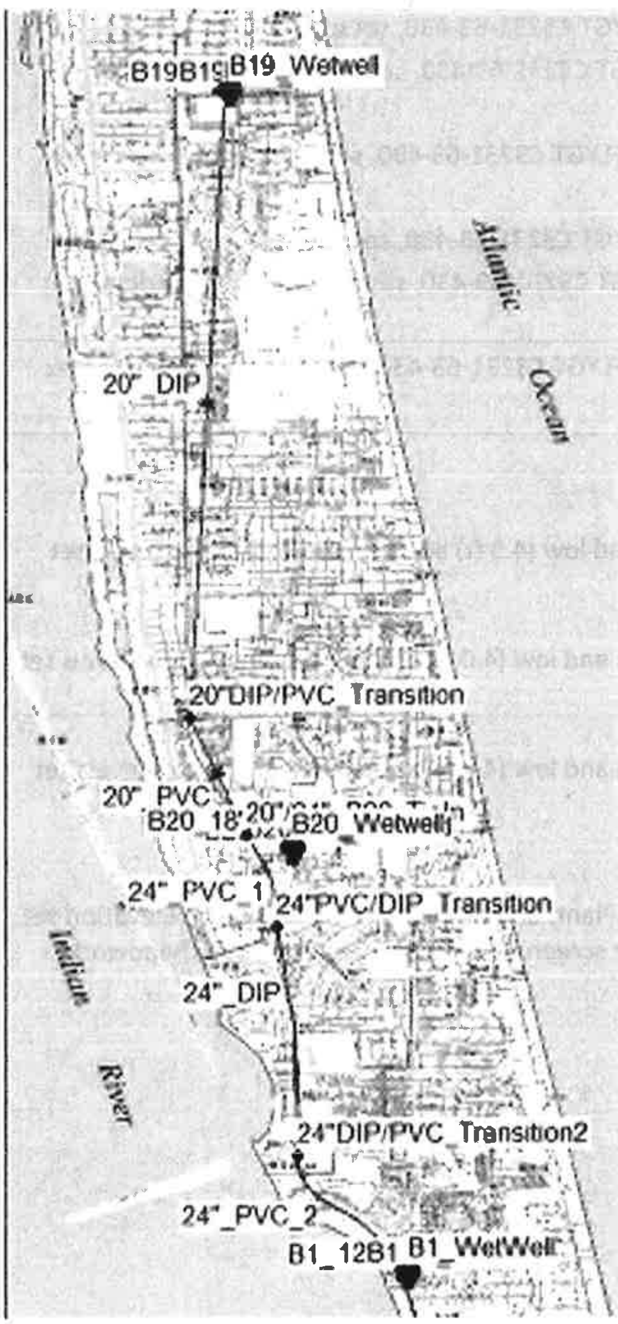
- B20_WetWell: B19 wet well, depth elevation and low (4.4 ft) and high (8.1 ft) water levels set based on asbuilts.

Reservoirs:

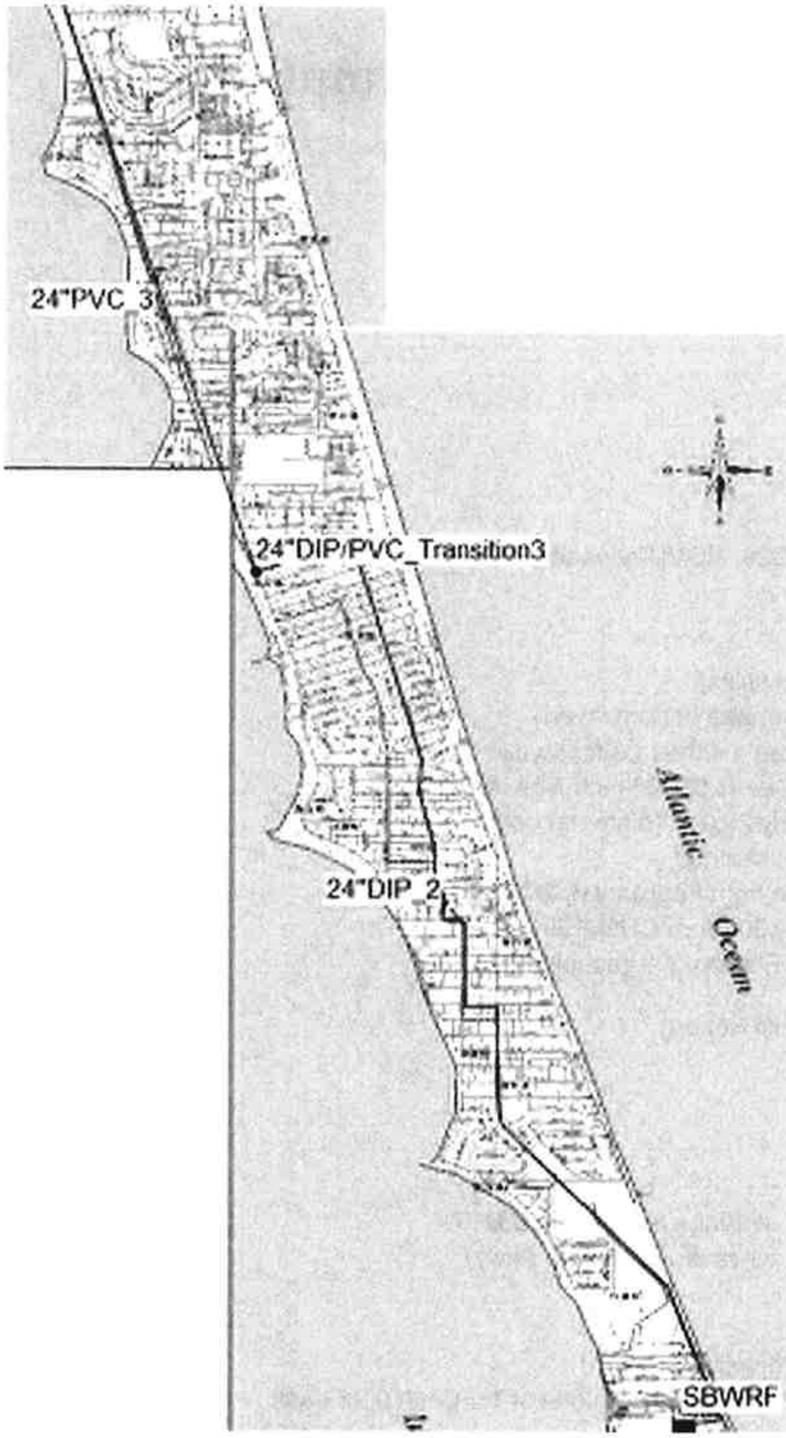
- SBWRF: South Beach Wastewater Treatment Plant, end of the force main system. Elevation set based on County provided information on bar screen high water elevation at the headworks.

EPAnet Network Map

North Section:



South Section:



Existing System: EPAnet Model Assumptions, Inputs and Notes

EPAnet Version 2.0, Build 2.00.12

Friction Equation: Hazen Williams

Flow: GPM

Energy Price: $0.06 / kw-hr

Assume elevations on asbuilts are NGVD29, NGVD29 used in model.

B19 Lift Station:

1. Wet well (information from asbuilts)
    a. Elevation = -15.2 (wet well bottom invert)
    b. Initial / Minimum Level = 4 ft, -11.2 ft elevation
    c. Maximum = 7.7 ft, Elev -7.5 ft 16-inch pipe influent invert
    d. dimensions: 12.5 ft by 29.0 ft (diameter equivalent = 21.5 ft)
2. Triplex with two running, one standby
3. Inflow = 2,000 gpm (estimate from February 6, 2010 Report)
4. Pumps = FLYGT C3231-63-430 (from CH2MHill Report)
5. Pipe Discharge Center Line Elev = 4.4 ft (asbuilts)

20" DIP&PVC = 18,473 LF (from CH2MHill Report)

20" DIP FM

1. Length: 15,491 LF
2. Top of Pipe Elevation =
    a. 2.0 ft (from asbuilts, starting elevation near B19)
    b. 0.0 ft (from asbuilts, elevation at Desoto Pkwy)
3. C = 100
4. Diameter = 20 inches
5. Minor Losses: (Coeff from Sanks)
    a. Assume 1 fitting per 1000 feet for a total of 16, Coeff 0.25 each
    b. Assume 1 isolation vale per 2000 feet for a total of 8, Coeff 1.00 each
    c. Total Minor Loss Coeff Assumption = 12

20" PVC FM

1. Length: 2,981 LF (based on Jeff C markups of PVC length on map)
2. Top of Pipe Elevation = 0 ft (from asbuilts, tie-in elevation with B20)

3. C = 110
4. Diameter = 20 inches
5. Minor Losses: (Coeff from Sanks)
    a. Assume 1 fitting per 1000 feet for a total of 3, Coeff 0.25 each
    b. Assume 1 isolation vale per 2000 feet for a total of 1, Coeff 1.00 each
    c. Total Minor Loss Coeff Assumption = 1.75

B20 Lift Station:

1. Wet well (from asbuilts)
    a. Elevation = -14.6 (wet well bottom invert)
    b. Initial / Minimum Level = 4.4 ft, -10.2 ft elevation
    c. Maximum = 8.1 ft, -6.5 ft 16-inch pipe influent invert
    d. dimensions: 12.5 ft by 29.0 ft (diameter equivalent = 21.5 ft)
2. Triplex with two running, one standby
3. Inflow = 3,500 gpm (estimate from CH2MHill Report)
4. Pumps = FLYGT C3231-63-430 (from CH2MHill Report)
5. 18" DIP from B20 to tie in with 24" FM, 1000 LF approximate, 5.0 ft elev, approximate

24" PVC FM

1. Length: 2,205 LF (based on Jeff C markups of PVC length on map)
2. Top of Pipe Elevation = -2.0 ft (from asbuilts, near Banana River Dr)
3. C = 110
4. Diameter = 24 inches
5. Minor Losses: (Coeff from Sanks)
    a. Assume 1 fitting per 1000 feet for a total of 3, Coeff 0.25 each
    b. Assume 1 isolation vale per 2000 feet for a total of 1, Coeff 1.00 each
    c. Total Minor Loss Coeff Assumption = 2.75

24" DIP FM (north segment)

1. Length: 5144 LF (based on Jeff C markups of DIP length on map)
2. Top of Pipe Elevation = 2.0 ft (from asbuilts, near bend in roadway)
3. C = 100
4. Diameter = 24 inches
5. Minor Losses: (Coeff from Sanks)
    a. Assume 1 fitting per 1000 feet for a total of 5, Coeff 0.25 each
    b. Assume 1 isolation vale per 2000 feet for a total of 1, Coeff 1.00 each
    c. Total Minor Loss Coeff Assumption = 2.75

24" PVC FM (from 24" DIP north section to B1 tie in)

1. Length: 3,590 LF (based on Jeff C markups of PVC length on map)
2. Top of Pipe Elevation = 2.0 ft (from asbuilts, near Coral Way West)
3. C = 110
4. Diameter = 24 inches
5. Minor Losses: (Coeff from Sanks)
    a. Assume 1 fitting per 1000 feet for a total of 4, Coeff 0.25 each
    b. Assume 1 isolation vale per 2000 feet for a total of 2, Coeff 1.00 each

    c. Total Minor Loss Coeff Assumption = 3.00

B1 Lift Station:

1. Wet well (information from asbuilts)
    a. Elevation = -15.40 (wet well bottom invert)
    b. Initial / Minimum Level = 4.5 ft, -10.9 ft elevation
    c. Maximum = 11.5 ft, Elev -3.9 ft 18-inch pipe influent invert
    d. dimensions: 10 ft diameter
2. Duplex with one running, one standby
3. Inflow = 600 gpm (estimate from CH2MHill Report); 800 gpm and 1,000 gpm (analyzed)
4. Pumps = ABS AFP 1501 310 mm (from CH2MHill Report)
5. Pipe Discharge Center Line Elev of 12-inch = 4.25 ft (asbuilts)

24" PVC FM (from B1 tie in to 24" DIP Transition)

1. Length: 13,917 LF (based on Jeff C markups of PVC length on map and CH2MHill Length)
2. Top of Pipe Elevation = 2.0 ft (from asbuilts, near Coral Way West)
3. C = 110
4. Diameter = 24 inches
5. Minor Losses: (Coeff from Sanks)
    a. Assume 1 fitting per 1000 feet for a total of 14, Coeff 0.25 each
    b. Assume 1 isolation vale per 2000 feet for a total of 7, Coeff 1.00 each
    c. Total Minor Loss Coeff Assumption = 10.50

24" DIP FM (southern segment to plant SBWRF)

1. Length: 25,730 LF (based on CH2MHill Length)
2. Top of Pipe Elevation = 2.0 ft (from PBSJ asbuilt at Oakland)
3. C = 100
4. Diameter = 24 inches
5. Minor Losses: (Coeff from Sanks)
    a. Assume 1 fitting per 1000 feet for a total of 26, Coeff 0.25 each
    b. Assume 1 isolation vale per 2000 feet for a total of 13, Coeff 1.00 each
    c. Total Minor Loss Coeff Assumption = 19.50

SBWRF: Elevation 33.814 ft (32.4 ft NAVD88)

To note, only pump stations noted above were incorporated into the model. "Smaller" stations were not included in the model or analysis.

C

Appendix C – South Beaches Surge Analysis Study

HydraTek

TECHNICAL MEMORANDUM

Date: 15 December 2017

Revision: Version 2 (Draft)

To: Jeff Crigler (HDR)

From: Johnathan D. Nault and Djordje Radulj (HydraTek)

Subject: Hydraulic Transient (Surge) Analysis
South Beaches Lift Stations and Forcemains, Brevard County

INTRODUCTION

HydraTek & Associates (HydraTek) has been retained by HDR Inc. (HDR) on behalf of Brevard County Utility Services (County) to conduct a hydraulic transient (surge) analysis of the South Beaches sewage forcemain system located in Brevard County, Florida. The objectives of the analysis are to (i) support an investigation into previous forcemain failures, including reviewing transient pressure monitoring field data, and (ii) support the design of system upgrades while recommending alternatives for reducing the hydraulic transient (surge) risk in the system.

The South Beaches system features two separate forcemains – the larger east forcemain and the smaller west forcemain. The east system has roughly 69,000 ft of 20-inch to 24-inch diameter forcemain with a combination of PVC and ductile iron pipe, and it is supplied by a series of large and small pumping stations, the key ones of which are the larger Lift Stations B-19, B-20, and B-1. The west forcemain system comprises nearly 25,000 ft of 20-inch diameter PVC pipe and 24-inch diameter ductile iron pipe, and it too is supplied by a combination of large and small lift stations with Lift Stations B-6 and B-7 representing the primary supply. Both systems convey wastewater to the South Beaches Wastewater Treatment Plant (WWTP) located in Melbourne Beach at the south end of the County.

This work primarily concerns the east system that has, in recent years, experienced a number of forcemain (pipe) breaks. Most of these have been concentrated at the PVC sections of the forcemain, and previous work by others suggests that the breaks are predominantly caused by improperly installed pipe. Transient (surge) pressures resulting from pump start and stop operations are thought to be contributing to the failures. To better understand the likely source of the forcemain failures and assess whether transient pressures are related thereto, HDR was retained by the County to review previous studies of the system, conduct a transient pressure monitoring exercise (performed by Blacoh Industries; Blacoh), and analyze alternatives for improving the system.

Based on the previously completed review, HDR has proposed that approximately 17,800 ft of 24-inch diameter PVC forcemain along North Riverside Drive from Eau Gallie Boulevard to Oakland Avenue be replaced with 30-inch diameter ductile iron pipe and that Lift Station B-1 be upgraded in order to better operate with the larger pumps at Lift Stations B-19 and B-20. An interconnection between the east and west forcemain systems is also being considered along either Riverside Drive or along 2nd Avenue (though this has yet to be finalized). In facilitating the design of the proposed upgrades while supporting HDR's investigation, this report presents the results of a hydraulic transient (surge) analysis of the existing South Beaches forcemain system and proposed upgrades.

REFERENCE INFORMATION

Hydraulic steady-state and hydraulic transient (surge) models were developed to facilitate the hydraulic transient analysis using the following information provided by HDR:

Plan and Profile Drawings

- South Patrick and Indian Harbour Beach Force Main Interconnection record drawings showing original forcemain installation by Stottler Stagg & Associates, dated February 1986;
- South Patrick Drive Forcemain Relocation as-built drawings showing 20-inch and 24-inch PVC forcemain installation by Brevard County Water Resources Department, dated January 1997;
- South Beaches 24-inch HDPE Force Main Replacement record drawings showing 24-inch PVC and DI forcemain installation by PBS&J, dated June 2006; and
- South Beaches drawing index sketch, received from HDR on 03 June 2017.

Lift Station Drawings

- South Patrick Regional Pumping Station (Lift Station B-19) record drawings by Stottler Stagg & Associates, dated February 1986;
- Indian Harbour Beach Regional Pumping Station (Lift Station B-20) record drawings by Stottler Stagg & Associates, dated February 1986;
- South Beaches Lift Station B-01 Reconstruction and Connection to Regional Force Main final design drawings by Brevard County, dated 15 October 1992; and
- Lift Station B-1 60% Design Submission Drawings by HDR, dated November 2017.

Pump Performance Curves

- Flygt C3231-63-430 pump curves for Lift Stations B-19 and B-20, dated 02 March 2002;
- ABS AFP 1501 60 Hz pump curves for existing Lift Station B-01, dated 28 September 2007; and
- Xylem NP 3202 HT 3-466 pump curves for proposed Lift Station B-01, received from HDR on 16 November 2017.

Reports and Other Data

- South Beaches Regional Wastewater Treatment Facility Hydraulic Profile (sheet M-7) record drawing by Stottler Stagg & Associates, dated 20 July 1988;
- South Beaches Sewer Force Main Draft Surge Analysis report by PBS&J, dated October 2005;
- Expert Opinion of Scott C. Williams, PE, on a Construction Project Known as B-6 Lift Station and Force Main Modifications by CH2M Hill, dated 16 February 2010;
- Brevard County Utility Services Department AWWA C905 DR25 PVC Force Main Pipe Samples failure review report by Diamond Plastics, dated 18 August 2015;
- Pipe break map for the east forcemain system by Brevard County, dated 02 June 2016;
- Review of Previous Pipe Failure Data and Surge Reports Technical Memorandum by HDR, dated 07 April 2017;

- B01 Pump Station Analysis EPANET Model, Results, and Assumptions report by HDR, dated 15 June 2017;
- South Beaches Wastewater Treatment Plant service area schematic showing lift stations and typical flows, received from HDR on 31 October 2017;
- South Beaches System Map by Brevard County, received from HDR on 03 June 2017; and
- Transient pressure monitoring data at LSs B-19 and B-01 for the period of 26 September 2017 to 25 October 2017 by Blacoh.

SYSTEM DESCRIPTION AND ANALYSIS METHODOLOGY

The following sections describe the South Beaches forcemain system and how they are modeled. The system is described first, followed by a discussion of the model development. In developing the hydraulic transient model, some assumptions were made of the key factors that affect the response of the system: these are discussed where appropriate. Finally, the transient analysis scenarios are listed before presenting and discussing the results.

East and West Forcemain Systems

Figure 1 on the following page illustrates a schematic of the South Beaches system (excluding the smaller lift stations). The larger east forcemain spans Sea Park Boulevard at the north to the South Beaches WWTP at the south, and it is primarily supplied by three lift stations. Lift Station B-19 (the South Patrick Regional Pumping Station) is the north-most facility, and it has three dry-pit centrifugal pumps, each with a motor soft-starter. Lift Station B-20 (the Indian Harbour Beach Regional Pumping Station) is situated in the middle. Like Lift Station B-19, lift station B-20 features a similar configuration with three dry-pit pumps with motor soft-starters. Lastly for the east system, the somewhat smaller Lift Station B-1 is the south-most facility with two submersible wet-pit pumps. Following the proposed upgrades, Lift Station B-1 will still have two larger pumps and a similar arrangement. All pumps at all lift stations are constant speed pumps.

Figure 2 on the subsequent page shows the centerline profile for the east forcemain system, including the location of the proposed 30-inch forcemain upgrade from Eau Gallie Boulevard to Oakland Avenue. The west forcemain system spans 2nd Avenue at the north to the WWTP at the south, and it is supplied by Lift Stations B-6 and B-7, both of which feature similar configurations to Lift Station B-1 and each have two submersible wet-pit centrifugal pumps. All of the pumps at each lift station are equipped with a tilted disc check valve.

Operationally, the number of pumps on at each facility is controlled by the level in the facility's wet well. Table 1 lists the elevations and corresponding levels for each lift station based on the information provided by HDR. Each pump starts against a closed check valve, and the pump's flow develops as the system's static head is overcome. For the pumps at Lift Stations B-19 and B-20, the previous hydraulic transient analysis by PBS&J notes that the pump's motor soft-starters have a 30 s ramp duration for pump starts and stops. Motor soft-starters provide electrical control and, depending on the drive type, occasionally some albeit limited hydraulic control for pump starts and stops by limiting either the motor voltage or current. Though each station is designed to provide its nominal rated flow at firm capacity (that is, with all pumps on except for the largest pump), it is understood that, on occasion, a station may operate at full capacity with all pumps on (e.g., during periods with large inflow and infiltration from heavy rainfall, Lift Station B-19 may operate with three pumps on).

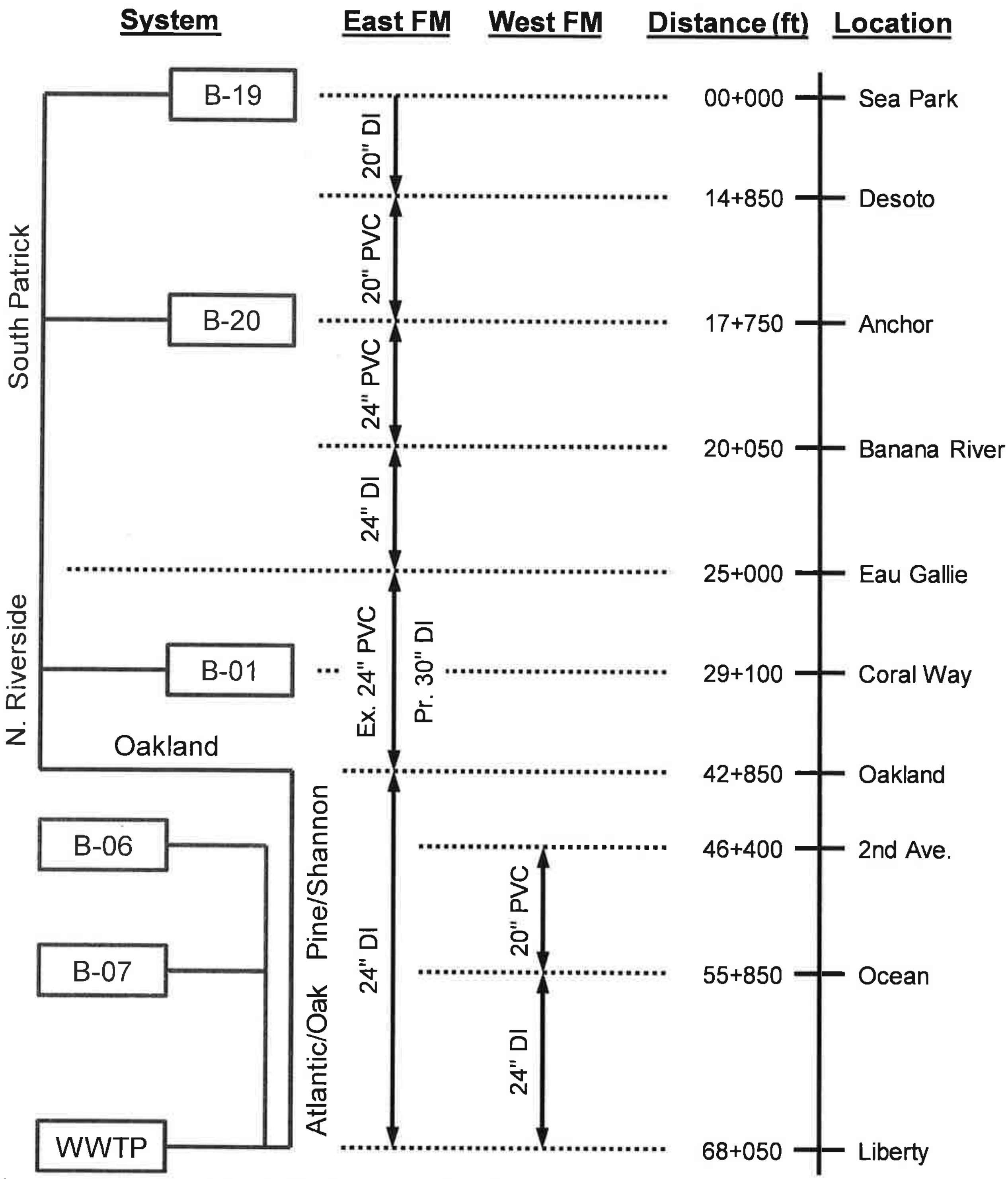


Figure 1: Schematic of South Beaches east and west forcemain systems

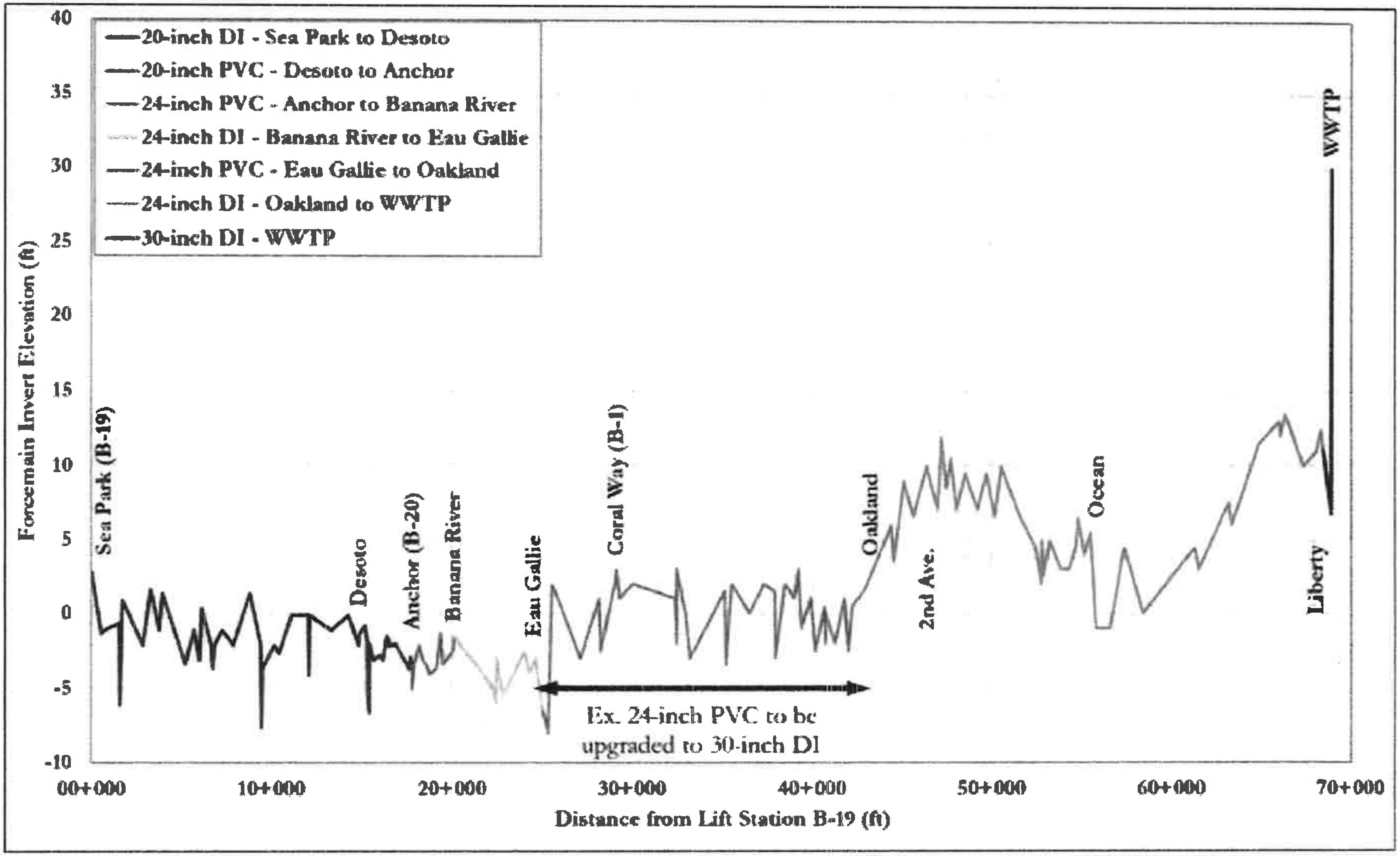


Figure 2: Centerline profile for the South Beaches existing east forcemain system

Table 1: Lift station wet well elevations and levels

Location	Elevation (ft)					Level (ft)				
	B-19	B-20	B-1	B-6*	B-7*	B-10	B-20	B-1	B-6*	B-7*
Wet well invert	-15.2	-14.6	-15.4	-15.4	-15.4	0.0	0.0	0.0	0.0	0.0
Pumps off	-9.2	-8.6	-12.4	-12.4	-12.4	6.0	6.0	3.0	3.0	3.0
Inlet pipe invert	-7.5	-6.5	-9.4	-9.4	-9.4	7.7	8.1	6.0	6.0	6.0
Lead pump	-6.2	-6.1	-6.4	-6.4	-6.4	9.0	8.5	9.0	9.0	9.0
Lag pump 1	-4.2	-5.1	-5.4	-5.4	-5.4	11.0	9.5	10.0	10.0	10.0
Lag pump 2	-3.2	-4.5	-	-	-	12.0	10.1	-	-	-
High alarm	-2.2	-4.1	-2.4	-2.4	-2.4	13.0	10.5	13.0	13.0	13.0
Top of wet well	7.7	7.1	8.5	8.5	8.5	22.9	21.7	23.9	23.9	23.9

* Elevations for B-6 and B-7 assumed to be as shown in HDR's EPANET model, and levels assumed as per those for B-1.

Hydraulic Model Development

The hydraulic transient (surge) analysis was performed using Bentley's HAMMER v10 software package. HAMMER is based on the proven method of characteristics solution of the governing water hammer equations, and it has been used for numerous projects globally for over a decade. To facilitate the numerical modeling analysis, a hydraulic model (shown in Figure 3) was developed using the information provided by HDR, namely the forcemain as-built drawings, pump performance curves, and previous analysis reports. Intermediate nodes were also added at local high points, local low points, and other key locations, such as transitions between pipe materials. Because the east forcemain system is the primary subject system (the west system is only concerned for scenarios with the east-west system interconnections), intermediate nodes at points of interest along the west forcemain were not modeled.

In developing the hydraulic transient model, some key assumptions were made to facilitate the analysis. The system pipes were modeled using their nominal diameters. The smaller lift stations (which are not shown in Figure 1) were omitted from the analysis, for they represent rather small flows compared to the larger Lift Stations B-19, B-20, and B-1 (and similarly B-6 and B-7 for the west system). A Hazen-Williams roughness (C-factor) of 110 was used for all existing piping due to their age, and a roughness factor of 140 was used for all new piping – the latter assumption is generally conservative for hydraulic transient analyses due to simulating larger initial flows and thus larger changes in hydraulic states. Further, the lift stations' levels were considered as either the low water level (LWL) and the high water level (HWL): the former corresponds to the "pumps off" level listed in Table 1, while the HWL values correspond to the higher lag pump start elevation (e.g., -3.2 ft for Lift Station B-19). Regarding the downstream boundary condition, the WWTP hydraulic profile schematic shows a submerged inlet condition for the forcemain. The WWTP is thus represented as a constant head reservoir with an elevation of 30.5 ft based on the LWL from the plant's hydraulic profile. Further assumptions were made in addition to those discussed above; these pertain to the pump properties, system wave speeds, and surge protection, which are discussed in the following sections.

Pump Properties

Pump properties such as size and speed are key parameters that directly affect the steady and transient hydraulics (flows and pressures) of a pumping system. Table 2 lists the properties of the pumps at each lift station. The combined rotational mass moment of inertia of a pump and its motor are the other key factor that affects the transient response of a pumping system. Under a power failure condition at a pump station, a pump with greater total inertia will take a longer time to completely rundown (i.e., push water through the system) than will a pump with a lower total inertia. The manufacturer values for pump inertia were not readily available for this analysis, so the radius-based (WR^2) pump inertia was estimated for each pump based on empirical relations - these values are also listed in Table 2. Because the total moment of inertia is occasionally a critical parameter to the hydraulic transient response of a system, a sensitivity analysis of these values is conducted.

Table 2: South Beaches forcemain system lift station pump properties

System	Facility	Pump ID	Status	Manufacturer and Model	Rated Flow* (GPM)	Rated Head* (ft)	Rated Power (hp)	Rated Speed (rpm)	Estimated Total Inertia ($lb\text{-}ft^2$)
East	B-19	1	Existing	Flygt C3231-63-430	3,643	147.0	185	1,780	75.8
		2	Existing	Flygt C3231-63-430	3,643	147.0	185	1,780	75.8
		3	Existing	Flygt C3231-63-430	3,643	147.0	185	1,780	75.8
	B-20	1	Existing	Flygt C3231-63-430	3,643	147.0	185	1,780	75.8
		2	Existing	Flygt C3231-63-430	3,643	147.0	185	1,780	75.8
		3	Existing	Flygt C3231-63-430	3,643	147.0	185	1,780	75.8
	B-1	1	Existing	ABS AFP 1501, 310 mm	1,698	108.0	70	1,785	20.4
		1	Proposed	Flygt NP 3202 HT 3~466	1,093	142.7	70	1,775	16.7
		2	Existing	ABS AFP 1501, 310 mm	1,698	108.0	70	1,785	20.4
		2	Proposed	Flygt NP 3202 HT 3~466	1,093	142.7	70	1,775	16.7
West	B-6	1	Existing	ABS AFP 1501, M430/4-44	1,600	90.6	60	1,775	15.6
		2	Existing	ABS AFP 1501, M430/4-44	1,600	90.6	60	1,775	15.6
	B-7	1	Existing	ABS AFP 1501 R25076-42.50	1,125	41.0	20	1,180	9.1
		2	Existing	ABS AFP 1501 R25076-42.50	1,125	41.0	20	1,180	9.1

* At the best efficiency point (BEP) on the pump's performance curve.

† Based on the empirical relations presented in Fluid Transients in Pipeline Systems, 2nd Ed. (Thorley, 2004).

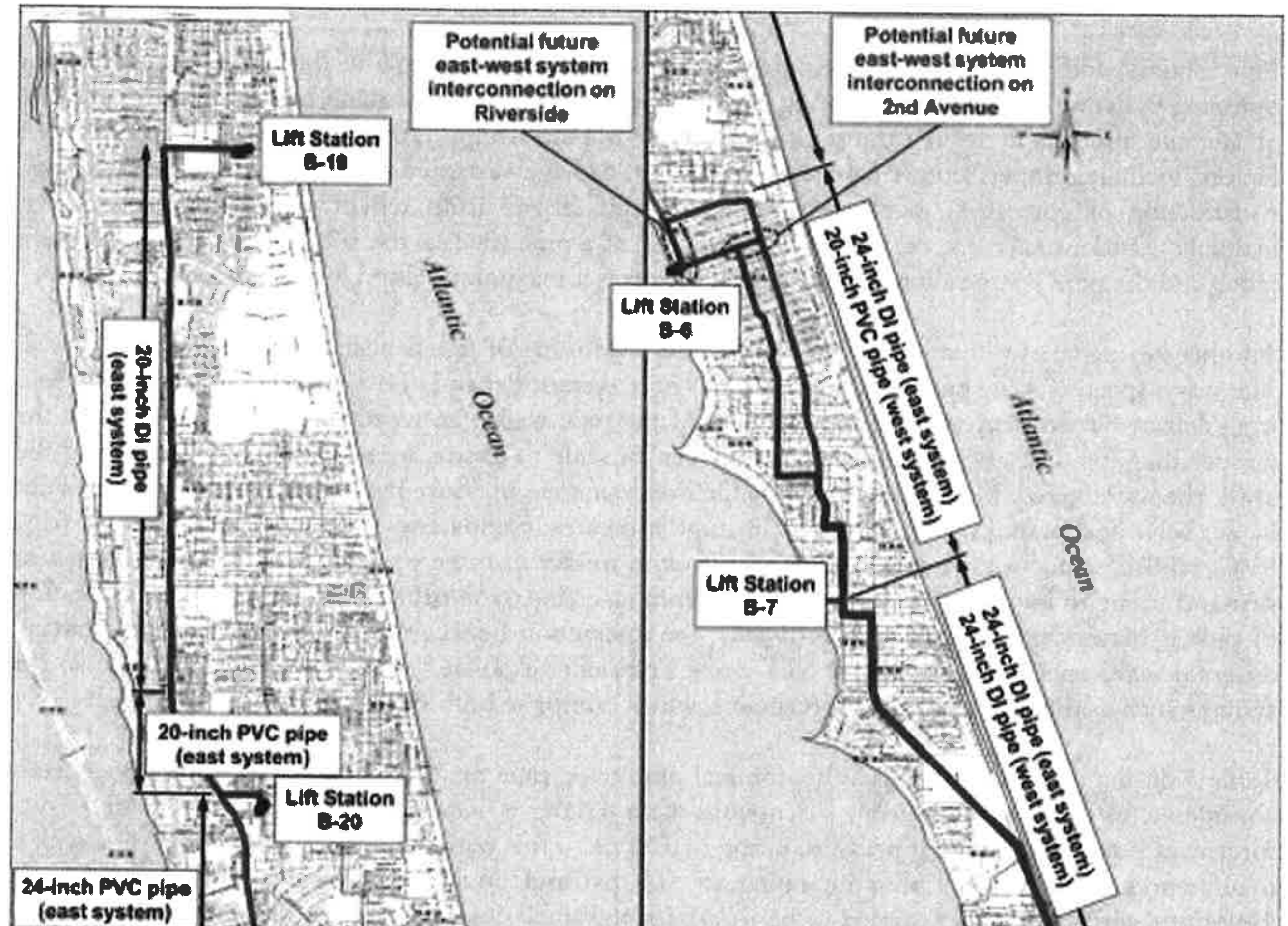


Figure 3: Hydraulic transient model of the South Beaches east and west forcemain systems

Pipe Class and Wave Speed

Pipe material and pipe class (i.e., pressure rating) play an important role in the transient response of pumping systems. The higher the rating of the pipe, the greater its working pressure rating, and the greater the upper limit against transient (i.e., surge) pressures. Pipe rating is affected by a number of factors, including imperfections from manufacturing, damage sustained during installation or use (e.g., construction or corrosion), depth of installation, and fatigue from repetitive pressure cycling. For hydraulic transient analyses, the total pressure rating of a pipe (that is, the sum of its working pressure rating and the pipe's surge allowance) is typically used as a maximum design limit or upper bound.

Another key parameter that affects the nature and magnitude of transient pressures is the wave speed. The wave speed (sonic speed through a liquid) of a system depends on both the nature of the fluid (e.g., density, air content) and on the elasticity of the pipe wall. The more elastic the material and the thinner the pipe wall, the more storage a pipe can provide to excess water being compressed and the lower the wave speed. Materials such as ductile iron, concrete pressure pipe, and steel typically produce higher wave speeds and more significant upsurge pressures; comparatively, more elastic materials (e.g., PVC, HDPE) tend to have lower wave speeds with milder upsurge pressures, yet they are often less stiff and prone to longer downsurge pressure durations. This trade-off means that neither classification of pipe is better than the other. Additionally, the interaction between different pipe materials having different wave speeds can also lead to a worse transient response – this is particularly the case for systems such as the South Beaches forcemains, which comprise both ductile iron and PVC pipe.

Table 3 on the following page lists the nominal pipe sizes, pipe materials and classes, and wave speeds considered in this analysis. From discussions with HDR, it is understood that the existing PVC forcemain pipe has a working pressure rating of 100 psi – this corresponds to DR41 PVC pipe with a total (working plus surge) pressure rating of 160 psi and an estimated wave speed of 850 ft/s. Therefore, all PVC pipe is assumed to be DR41 for this analysis. The pressure class of the ductile iron forcemain was not available at the time of this analysis. Accordingly, it is assumed to be Class 150, the lowest available for ductile iron pipe, which has a working pressure rating of 150 psi and a total pressure rating of 250 psi. The wave speed for ductile iron pipe typically ranges from 3,600 ft/s to 4,000 ft/s, so an average value of 3,800 ft/s is adopted here. A review of Blacoh's transient pressure monitoring data (which is discussed later) also suggests that the system wave speed varies with the system condition (number of pumps on, operating pressures and flows) – this is typical of forcemain systems, which often have varying amounts of entrapped and dissolved gases that affect the wave speed. Additionally, pipe wall thickness, the material Poisson ratio, joints, and the restraint conditions (as imposed by thrust and joint restraints, anchoring, and expansion joints) also affect the wave speed. For instance, a pipe that is sufficiently restrained at both ends will have a higher wave speed than one with anchoring at one end or none at all. The exact details of these properties are not known for the South Beaches system though, so no adjustments are made to consider these and the basic wave speed assumptions are utilized. Furthermore, sensitivity tests are performed for the assumed wave speed values to explore whether they are indeed a critical parameter.

Existing Surge Protection

Surge protection devices are intended to alleviate otherwise excessively high or low pressures in pressurized pipe systems. The as-built drawings for Lift Stations B-19 and B-20 show that each has a 6-inch surge control valve located on the discharge header, though the specific type and function of the valves is not known. In addition to the surge valves, the as-built drawings for the forcemains show air valves at a small number of high points from Lift Station B-19 to the WWTP. From discussions with HDR, the surge valves at Lift Stations B-19 and B-20 are presently isolated and thus not used, and the air valves along the forcemains are 2-inch nominal diameter air release valves – that is, the air valves are small outflow only air valves that do not provide vacuum protection. Altogether, the existing South Beaches forcemain system effectively does not feature dedicated surge protection.

Table 3: South Beaches forcemain pipe pressure ratings and wave speeds

System	Section	Nominal Diameter (in)	Pipe Material and Class	Working Pressure Rating (psi)	Total Pressure Rating (psi)	Wave Speed (ft/s)
East	Lift Station B-19 to Desoto	20	Ductile Iron, Class 150*	150*	250*	3,800
	Desoto to Anchor	20	PVC, DR41+	100	160	850
	Anchor to Banana River	24	PVC, DR41+	100	160	850
	Banana River to Eau Gallie	24	Ductile Iron, Class 150*	150*	150*	3,800
	Eau Gallie to Oakland (Existing)	24	PVC, DR41+	100	160	850
	Eau Gallie to Oakland (Proposed)	30	Ductile Iron, class to be determined	150*	250*	3,800
	Oakland to WWTP	24	Ductile Iron, Class 150*	150*	250*	3,800
West	Lift Station B-6 to 1st Avenue	20	PVC, DR41+	100	160	850
	1st Avenue to WWTP	24	Ductile Iron, Class 150*	150*	250*	3,800

* Because the class of ductile iron pipe is not known for this system, it is assumed to be class 150 with a working pressure rating of 150 psi and total (operating plus surge) pressure rating of 250 psi.

+From discussions with HDR, it is understood that the existing PVC pipe has a working pressure rating of 100 psi. This corresponds to DR41 PVC pipe, which is assumed for all PVC pipe in the system.

It is also worth briefly mentioning the previous hydraulic transient (surge) analysis performed by PBS&J in 2005. At the time of PBS&J's analysis, the status of the existing surge valves at Lift Stations B-19 and B-20 is not clear, and PBS&J's report does not list any other forms of existing surge protection in the system. Results from the analysis suggest that, without dedicated surge protection, negative pressures reaching full vacuum are problematic along the forcemain. To alleviate such conditions, PBS&J discusses two alternatives: installing combination air valves (CAVs) along the forcemain at critical locations, or installing surge tanks at the lift stations. From discussions with HDR, it is understood that neither of these recommendations has been implemented.

Transient Events

While most systems are often subjected to a variety of risks resulting from hydraulic transients, the key for the analysis and design against unsteady flow (surge, transient, or water hammer conditions) comes down to the selection of an appropriate design event or design events. In this system, the primary transient conditions arise from three events: operational changes (i.e., routine pump or valve operations), pipe failures (i.e., forcemain breaks), and power failures at the lift stations. The risks from routine pump or valve changes and other planned events (e.g., filling or draining) can be minimized and accommodated through proper operator training and by ensuring that these changes evolve over a moderate period of time. Pipe breaks are almost impossible to predict in any detail, are more difficult to control, and can result in negative pressure surges that yield subsequent pipe breaks. The key to minimizing the impact of such an event (in both water supply and wastewater systems) is providing adequate system storage, or in very large systems, pressurized vessels (i.e., surge tanks). Power failure events are often the most problematic. They can occur under almost any operating condition; as a result, the key transient protection challenge likely comes down to a power failure event at the source lift stations (i.e., Lift Stations B-19, B-20, B-1, B-6, and B-7). This design event is a suitable choice because power failures are, by their very nature, unpredictable. They are inevitable over the long life of pumping systems, and they are excellent surrogate design events for gauging a system's response to routine and non-routine pump trips, starts, and stops, and in some cases, pipe breaks.

When a power failure occurs, there is an immediate deceleration of the pump rotor and a decrease in the rate that fluid is forced into the associated pipe segment. This reduction of inflow creates a mass imbalance in the associated pipe, since the downstream end is not immediately informed of the loss of power. The net effect is a low-pressure wave that propagates through the pipe system until it is eventually reflected back at the downstream end. The reflected wave essentially sends a signal back to the pump and repeats the cycle, a process that eventually and gradually brings about a new equilibrium in the system. This is essentially a "conversation" between the ends of the system that takes place through a series of pressure wave propagations. Eventually, these pressure waves are dissipated by friction and other energy losses, and a new steady-state is established.

From a design perspective, there are three immediate challenges associated with power failure events. The first is the downsurge that immediately follows pump stoppage: this low-pressure wave can potentially collapse a "weak" or under-designed pipeline or allow an intrusion event to occur at susceptible points in the system. The second risk arises from the return surge that is generated from the reflected wave at the downstream end of a system. This reflected wave, referred to as a "return upsurge", actually increases pressures in the line, possibly to values that can considerably exceed the original steady-state or equilibrium levels experienced before the power failure. Unless properly controlled, the resulting transient pressures create either a risk of pipe failure or a need for much stronger pipes (i.e., thicker pipe walls, resulting in a higher pipe pressure rating and greater cost). Lastly, power failures can also create an upsurge event at the upstream (suction) line of the source pumping station. The effect of the upsurge at the suction side generally depends on the type of pumping station. In a "true" pumping station (such as Lift Station B-19), the upsurge is typically mitigated by the presence of a wet well and/or a short length of pipe.

Although power failures are likely the governing transient event, significant transient pressures can still arise from other source events. If not adequately controlled, the transient pressures resulting from routine pump starts and stops, which are more frequent than power failures, may be considerable; thus, it is worthwhile verifying that power failures are indeed the governing event. Under normal operating conditions, routine pump starts and stops at Lift Stations B-19 and B-20 are performed using the pumps' motor soft-starters. Based on information provided by HDR, pump start (and stop) operations involve starting (or stopping) a pump using its motor soft-starter with a 30 s ramp duration. Again, motor soft-starters can provide some (albeit limited) hydraulic control for pump start and stop operations depending on the drive type, so here routine starts and stops are represented using a linear torque ramp. The pumps at the remaining Lift Stations B-1, B6, and B-7 are simply started (and stopped) as an across-the-line operation with no hydraulic nor electrical control.

From the above, the following events are considered in the hydraulic transient analysis:

- Global power failure of all lift stations,
- Local power failure of select lift stations, and
- Routine pump starts and pump stops.

The pump start and stop operations at Lift Stations B-19 and B-20 are represented by a linear torque ramp having the same 30 s duration as the motor soft-starters. Soft-starters limit either current or voltage, thereby allowing power and thus torque to gradually adjust to the operating condition.

Analysis Scenarios

A total of 35 scenarios were simulated to explore the hydraulic transient (surge) response of the system under different conditions. They consider various combinations of the system state (existing, proposed upgrades, and potential east-west system interconnection), number of pumps initially on, wet well levels, transient event (global or local power failure, pump operations), and surge protection.

The first group of scenarios (Scenarios 1A through 5B; Table 4) concerns the existing east forcemain. These scenarios are intended to support the investigation into previous east forcemain failures, and they explore the system response to global power failure events under full capacity (Scenarios 1A and 1B), local power failures at Lift Stations B-19 (Scenario 2) and B-20 (Scenario 3), and routine pump operations at Lift Stations B-19 (Scenarios 4A and 5A) and B-20 (Scenarios 4B and 5B). Scenarios 2 through 5B consider HWLs at the lift stations, for this yields the largest initial flow and thus likely the greatest potential for extreme pressures.

Table 4: List of hydraulic transient analysis scenarios for existing conditions without surge protection

Scenario	Parent Scenario	No. Pumps					Well Levels	System	Event+
		B-19	B-20	B-1	B-6	B-7			
1A	-	3	3	2	-	-	HWL	Existing	Global Failure
1B	1A	3	3	2	-	-	LWL	Existing	Global Failure
2	-	3	0	0	-	-	HWL	Existing	Local Failure
3	-	0	3	0	-	-	HWL	Existing	Local Failure
4A	-	1*	0	0	-	-	HWL	Existing	B-19 Pump Start
4B	4A	0	1*	0	-	-	HWL	Existing	B-20 Pump Start
5A	-	1	0	0	-	-	HWL	Existing	B-19 Pump Stop
5B	5A	0	1	0	-	-	HWL	Existing	B-20 Pump Stop

* Pump start with pump initially off.

The second set of simulations (Scenarios 6A to 17B; Table 5 on the following page) explores the hydraulic transient response of the system with the proposed 30-inch ductile iron forcemain replacement, proposed Lift Station B-1 upgrades, and no surge protection. Scenarios 10 and 11 in particular consider interconnections between the east and west forcemain systems on North Riverside Drive and 2nd Avenue, respectively; the rational for these two scenarios is to test the potential hydraulic transient benefits (or detriments) of interconnecting the two systems. Scenarios 12 through 15 are sensitivity tests of key parameters, including wave speed, pump inertia, and vapor cavity formation. Additional simulations (Scenarios 16A to 17B) were performed for routine pump starts and stops at Lift Stations B-19 and B-20.

Table 5: List of hydraulic transient analysis scenarios for proposed conditions without surge protection

Scenario	Parent Scenario	No. of Pumps					Well Levels	System	Event+	Sensitivity
		B-19	B-20	B-1	B-6	B-7				
6A	-	3	3	2	-	-	HWL	Pr. Upgrades	GPF	-
6B	6A	2	2	1	-	-	HWL	Pr. Upgrades	GPF	-
7A	-	3	3	2	-	-	LWL	Pr. Upgrades	GPF	-
7B	7A	2	2	1	-	-	LWL	Pr. Upgrades	GPF	-
8A	-	3	0	0	-	-	LWL	Pr. Upgrades	GPF	-
8B	8A	2	0	0	-	-	LWL	Pr. Upgrades	GPF	-
9A	-	0	3	0	-	-	LWL	Pr. Upgrades	GPF	-
9B	9A	0	2	0	-	-	LWL	Pr. Upgrades	GPF	-
10	7A	3	3	2	2	2	LWL	Riverside Interconnection	GPF	-
11	7A	3	3	2	2	2	LWL	Oakland Interconnection	GPF	-
12	Worst GPF	0	3	0	-	-	LWL	Pr. Upgrades	GPF	+10% Wave Speed
13	Worst GPF	0	3	0	-	-	LWL	Pr. Upgrades	GPF	-10% Wave Speed
14	Worst GPF	0	3	0	-	-	LWL	Pr. Upgrades	GPF	50% Pump Inertia
15	Worst GPF	0	3	0	-	-	LWL	Pr. Upgrades	GPF	No Vapor Cavities
16A	-	1*	0	0	-	-	HWL	Pr. Upgrades	Pump Start	-
16B	-	0	1*	0	-	-	HWL	Pr. Upgrades	Pump Start	-
16C	-	0	0	1*	-	-	HWL	Pr. Upgrades	Pump Start	-
17A	-	1	0	0	-	-	HWL	Pr. Upgrades	Pump Stop	-
17B	-	0	1	0	-	-	HWL	Pr. Upgrades	Pump Stop	-
17C	-	0	0	1	-	-	HWL	Pr. Upgrades	Pump Start	-

* Pump start with pump initially off.
+GPF = global power failure, LPF = local power failure.

The aforementioned scenarios (Scenarios 1A through 17B) conservatively consider no dedicated surge protection. The remaining Scenarios 18A to 20 (Table 6 on the following page) explore the potential benefits of installing dedicated surge protection, namely CAVs at critical locations to alleviate the formation of vapor cavities (Scenarios 18A to 19B) and surge tanks (i.e., hydropneumatic air chambers or HACs) at the larger Lift Stations B-19 and B-20 (Scenarios 20A to 20D). The latter set of scenarios are mainly intended to demonstrate the potential benefits of installing HACs with bladders.

Table 6: List of hydraulic transient analysis scenarios for proposed conditions with surge protection

Scenario	Parent Scenario	System	Power Failure Event	Surge Protection
18A	Worst Base No. 1	Pr. Upgrades	Global	2-inch Standard CAVs†
18B	Worst Base No. 2	Pr. Upgrades	Global	2-inch Standard CAVs†
19A	Worst Base No. 1	Pr. Upgrades	Global	2-inch 3-Stage CAVs†
19B	Worst Base No. 2	Pr. Upgrades	Global	2-inch 3-Stage CAVs†
20A	Worst Base No. 1	Pr. Upgrades	Global	HAC+ at B-19
20B	Worst Base No. 1	Pr. Upgrades	Global	HAC+ at B-20
20C	Worst Base No. 1	Pr. Upgrades	Global	HAC+ at B-1
20D	Worst Base No. 1	Pr. Upgrades	Global	HACs at B-19 and B-20

+GPF = global power failure, HAC = hydropneumatic air chamber (pressurized surge tank).
† CAVs at select critical locations.

ANALYSIS RESULTS AND DISCUSSION

The following sections present and discuss the hydraulic transient (surge) analysis results. The initial discussion concerns the field monitoring data and simulation results for the existing east forcemain system, while subsequent sections discuss the results for the proposed upgrades, including alternatives for surge protection. Further, Appendix A contains a tabular summary of the modeling results, including initial lift station flows, the maximum predicted pressures, and the maximum predicted air/vapor cavity volumes; graphical results in the form of hydraulic grade line (HGL) profiles are also provided in Appendix B for the forcemain path from Lift Station B-19 to the WWTP. Lastly, Appendix C contains excerpt plots of the field monitoring data with Blacoh's full report provided in Appendix D.

Transient Pressure Monitoring Data

The transient pressure monitoring exercise was undertaken by Blacoh from 26 September 2017 to 25 October 2017. One high-frequency pressure logger was installed on the discharge side of Lift Station B-19 from 26 September 2017 to 25 October 2017, and another was installed on the discharge side of Lift Station B-1 from 26 September 2017 to 12 October 2017. Both installations were downstream of the pump check valves. Normal and transient pressures were recorded, the latter at up to 100 samples per second during transient (surge) events. Because the exercise was passive in nature (that is, no coordinated field tests were performed), the data presumably represents typical system conditions and routine operations. It is also worth noting that some transient pressure loggers suffer from time drift, which otherwise impacts the ability to correlate data between loggers and against other data. Consequently, the monitoring data cannot be directly compared against the modeling data due to uncertainties of the following: the precise pressure measurement elevations, boundary conditions (e.g., wet well and WWTP levels, active pumps), the precise operational changes (and nature thereof), and the system properties. Despite the aforementioned limitations, inferences and observations can be drawn from the field data for the existing east forcemain system. Sample monitoring results in the form of pressure histories are provided in Appendix C for 30 September 2017 (Figure 47 to Figure 50) and 05 October 2017 (Figure 51 to Figure 55), two dates where monitoring data are available at both locations. Table 7 below presents an overall summary of the pressure monitoring data.

Table 7: Summary of transient pressure monitoring data at Lift Stations B-19 and B-1

Lift Station	Monitoring Period		Pressure Summary (psi)				
	Start	End	Minimum	Average Low	Average	Average High	Maximum
B-19	26 Sep 2017	25 Oct 2017	-8	39	65	77	148
B-1	26 Sep 2017	12 Oct 2017	-10	27	44	54	130

From a review of the transient pressure monitoring data, the following general observations are made:

- Transient pressures at both monitoring locations regularly reach partial vacuum conditions. The minimum recorded pressure at Lift Station B-19 is -8 psi, and that at Lift Station B-1 is -10 psi. The minimum pressures are likely lower than -10 psi at forcemain high points, and these locations are therefore likely where full vacuum pressures and vapor cavity formation occurs. Given the low and even negative transient pressures, the system is at risk of experiencing vapor cavity formation and collapse, which can create significant transient pressures, extreme stresses on forcemain pipes, and exacerbate failures. The monitoring data in Figure 48 (Appendix C) shows what may even be the result of rapid air expulsion or vapor cavity collapse.

- From the transient pressure field monitoring data, the wave travel time from Lift Station B-19 to B-1 ranges from 30 s to 35 s. Given that the distance between these two stations is roughly 29,100 ft, this corresponds to an aggregate wave speed of 830 ft/s to 970 ft/s. This is less than that expected based on the estimated wave speeds for PVC and ductile iron (Table 3), though the field estimate is subject to the precise time synchronization of the two transient pressure loggers.
- Regarding pressures, the maximum pressures recorded at Lift Stations B-19 and B-1 are 148 psi and 130 psi, respectively. These are less than the assumed total (operating plus surge) pressure ratings for the ductile iron pipe (250 psi) and PVC pipe (160 psi). The peak pressure at Lift Station B-19 arose from what appears to be a single pump start at Lift Station B-19 following a period with all east system pumps off: Figure 56 and Figure 57 in Appendix C plot the pressure time series for this. The maximum pressure recorded at Lift Station B-1 arose from what is likely check valve slam following a pump stop at Lift Station B-1, and this is shown in Figure 55.
- In general, the system operation during the day time appears to have fewer 0-to-1 pump starts and fewer 1-to-0 pump stops. For example, Figure 47 and Figure 51 show frequent periods with all system pumping off over night, while Figure 49 and Figure 53 show fewer operations during the day time. This is likely due to more consistent wet well inflows during the day time.
- Following what is presumably full pump stoppage for the system (i.e., all pumps at the major lift stations are turned off), the monitoring data shows rapid and extreme pressure fluctuations at Lift Station B-19 (see Figure 48, Figure 50, and Figure 52 in Appendix C). The precise cause of this behavior cannot be ascertained from the data alone, though it is quite likely the result of either or a combination of (i) vapor cavity formation and collapse, (ii) rapid air accumulation, expulsion, or intake at leaky points in the system, or (iii) part of the forcemain draining.
- It is understood that the pumps at Lift Stations B-19 and B-20 are equipped with motor soft-starters that have ramp up and ramp down durations of 30 s. From the pressure monitoring data, the soft-starters at Lift Station B-19 do not appear to provide meaningful hydraulic control, particularly for pump stop operations. This may indicate that either (i) the soft-starters are not configured properly with respect to hydraulic control or (ii) the soft-starters simply provide no hydraulic control for pump start and stop operations.
- The field data for Lift Station B-1 shows check valve slam following routine pump stops at Lift Station B-1. This can be seen in Figure 54, and a closer view is provided in Figure 55. Check valve slam is caused by a reflected downsurge wave returning before a check valve has time to close, thereby slamming the valve closed. Check valve slam is detrimental to the local piping and valve itself, as it creates excessive pressures and wears the valve seating and seal, potentially leading to valve failure and an improperly seating (leaky) valve. The pumps at Lift Station B-1 are started and stopped with no hydraulic control (other than that provided by the check valves themselves). Consequently and following a pump stop at Lift Station B-1, the downsurge wave travels away from the facility along the relatively short 12-inch discharge line, reflects off of the existing 24-inch forcemain, and returns back to the pumps slamming the check valve closed.

Existing East Forcemain System

Table 9 in Appendix A summarizes the simulation results for the existing east forcemain system without the proposed upgrades nor any surge protection (Scenarios 1A through 5B). Scenarios 1A to 3 consider global and local power failures with the system initially operating at full capacity (that is, with all pumps on at an active facility under large inflow conditions). In each case, the system response is rather varied with a number of intermediate reflections, and these are caused by the use of both less elastic ductile iron and more elastic PVC pipe materials along the forcemain. The tabular results in Table 9 and graphical results in Appendix B also show partial to full vacuum conditions (negative pressures) along the forcemain from Lift Station B-19 to the WWTP – for example, the HGL envelope along the east forcemain for Scenario 1B (Figure 7 in Appendix B) shows negative pressures at nearly all locations. Negative pressures were simulated as persisting for up to roughly 30 s. Moreover, the full vacuum conditions are predicted to give rise to vapor cavity formation and collapse at both the lift stations and along the forcemain, as indicated by the non-zero vapor volumes.[1] Of Scenarios 1A to 3, Scenario 1B yields the most critical response with the largest predicted vapor cavities, and this is likely due to (i) the system operating at full capacity and with high initial velocities and (ii) the wet wells being at LWL, which yields a slightly lower initial HGL and thus less ability to accommodate downsurges. Overall, the long wave travel time for the PVC sections of the forcemain also prolong negative pressure durations, thus exacerbating concerns of vapor cavities.

The remaining Scenarios 4A through 5B explore the system response to routine single pump starts and stops at Lift Stations B-19 and B-20. The scenarios are based on an initial condition of no pumping at other locations, thereby yielding the largest flow change for the pump start and stop. The results (Figure 10 to Figure 13 in Appendix B) show rather high upsurge pressures (up to 95 psi at Lift Station B-19); for comparison, Figure 4 plots the results for Scenario 4A against the field monitoring data for what appears to be a routine pump start at Lift Station B-19. These data compare reasonably well for the initial upsurge; however, there are clear differences thereafter. Moreover, compared to the field monitoring data, the numerical modeling results predict lower maximum transient pressures. The difference between the model and field data is largely due to how pump start operations are actually performed. For this analysis, a 30 s linear torque ramp is used for pump starts, whereas the actual field conditions likely vary from this depending on the soft-starters' control and type (that is, whether the soft-starters limit voltage or current, and whether this translates to effective torque and thus hydraulic control). The differences are also affected by uncertainties of the boundary conditions (wet well levels, number of pumps on) and system properties. The results for Scenarios 5A and 5B show negative pressures and even full vacuum conditions following pump stop operations, which generally agrees with the low field monitoring pressures observed following full pump stoppage in the system.

Both the field monitoring data and numerical modeling results suggest that the typical pressures under steady and transient (surge) conditions are within the assumed total (operating plus surge) pressure ratings of the system's ductile iron and PVC pipes. There are, however, issues with check valve slam at Lift Station B-1, and Lift Stations B-19 and B-1 regularly experience negative pressures. Further, it is quite likely that negative pressures routinely occur throughout the entire forcemain, and this is likely due to a combination of the system having a mixture of pipe materials, a lack of dedicated surge protection, possible vapor cavity formation and line draining, and frequent pump start and stop operations with insufficient hydraulic control to minimize transient pressures. Regarding the latter, motor soft-starters alone generally provide inadequate (if any) hydraulic control, which is particularly important at the larger Lift Stations B-19 and B-20. Compared to motor soft-starters, hydraulic control valves and variable frequency drives (VFDs; also referred to as variable speed drives) tend to provide much better hydraulic control with greater capability for controlling transient pressures resulting from routine operations.

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[1] Indeed, vapor cavity formation is important, but it is important to note that the ability of numerical models to accurately simulate vapor cavity formation and collapse is difficult. Furthermore, the nature of the fluid (i.e., wastewater) will likely limit the duration of potential vapor cavities due to the presence of dissolved gases and the high number of nucleation sites. Accordingly, the actual system response is likely to differ from the simulation results, though negative pressures and vapor cavity formation are nonetheless problematic.

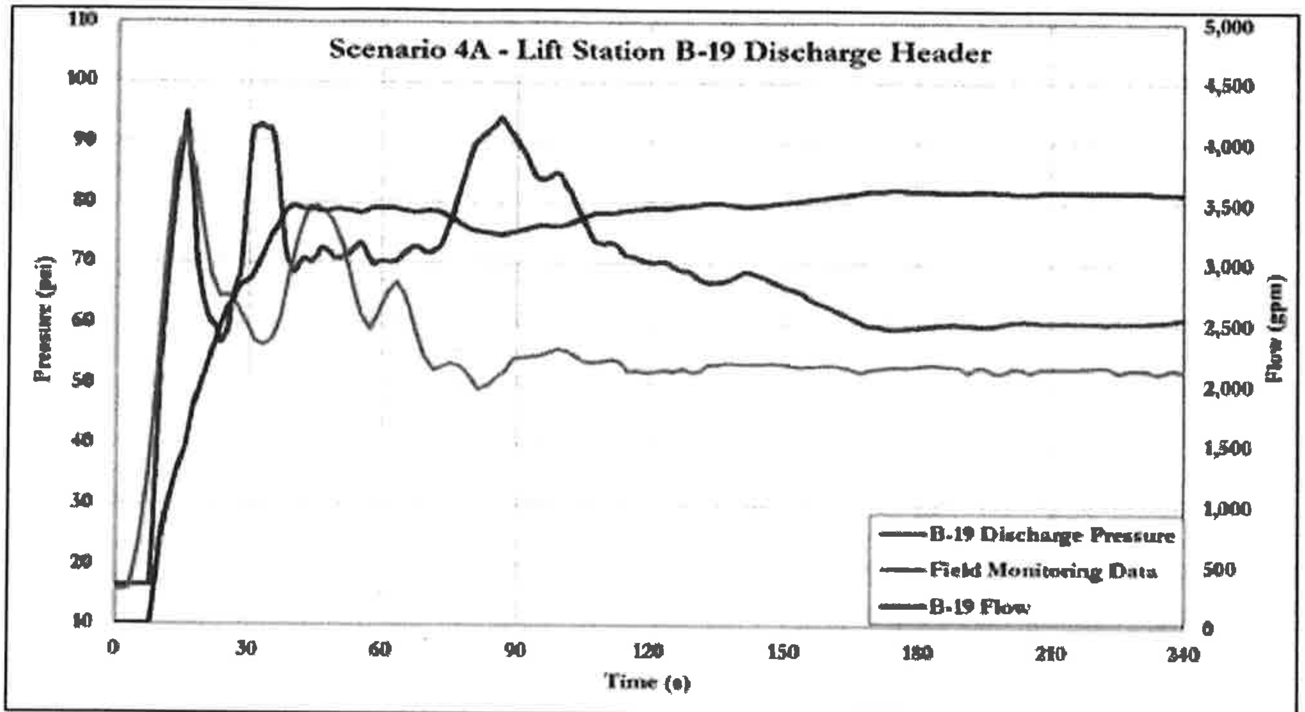


Figure 4: Comparison of simulation results for Scenario 4A (existing system, HWL, single pump start at Lift Station B-19) against Lift Station B-19 monitoring data on 30 September 2017 at 12:44 PM

Proposed Upgrades without Dedicated Surge Protection

Table 10 in Appendix A summarizes the simulation results for the east forcemain system with the proposed 30-inch ductile iron forcemain and upgrades to Lift Station B-1 (Scenarios 6A through 17B). As with the existing conditions, power failures are expected to create marginal upsurge pressures, yet negative pressures reaching full vacuum conditions are still predicted all along the forcemain from Lift Station B-19 to the WWTP. The negative pressures persist for up to roughly 10 s, which is less than that simulated for the existing conditions, likely due to the proposed 30-inch ductile iron forcemain having a higher wave speed than the existing PVC pipe. The simulation results show vapor cavities forming due to the negative pressures, and the subsequent vapor cavity collapse is predicted to create pressure spikes and high local stresses. Of the power failure scenarios for the potential system upgrades, Scenarios 7A and 9A are generally the worst: the former considers a system-wide power failure under full capacity pumping, while the latter scenario 9A considers a local power failure with only Lift Station B-20 on and at full capacity. Both scenarios result in larger vapor cavities forming compared to other scenarios. The total flow is high for Scenario 7A, so the resulting flow change for the forcemain is rather large, thus yielding extreme pressure drops. Scenario 9A features a high local flow at Lift Station B-20 due to no other system pumps being on, and this leads to an extreme local response that propagates to the rest of the system. Herein Scenarios 7A and 9A are considered as the base worst-case events due to having more severe responses, with both having overall greater vapor cavity volumes predicted along the forcemain compared to all other scenarios discussed thus far.

Scenarios 10 and 11, each based on Scenario 7A, test the system response with the proposed upgrades and potential future east-west forcemain interconnections on North Riverside Drive and along 2nd Avenue, respectively. The interconnections increase the downstream system capacity; consequently, the total upstream flow and thus initial upstream velocity are higher than for Scenario 7A. Though increased flows are beneficial for capacity, the results for Scenarios 10 (Figure 22) and 11 (Figure 23) suggest that the higher initial flows yield worse hydraulic transient responses with larger vapor cavities forming in the upstream segment of the forcemain.

Because a number of assumptions were made in developing the hydraulic transient model, sensitivity tests were performed to explore the effect of varying the wave speed (Scenarios 12 and 13), pump inertia (Scenario 14), and vapor cavity formation (Scenario 15). Scenario 9A was used as the base scenario for these due to having large predicted vapor cavity formation. In general, a comparison of the results for Scenarios 12 through 14 (Figure 23 to Figure 26 in Appendix B) against those of Scenario 9A shows that the wave speed and pump inertia indeed affect the results but not significantly. Regarding Scenario 15 (based on Scenario 9A without vapor cavity formation), Figure 27 in Appendix B shows a much lower minimum HGL envelope without vapor cavity formation, which suggests that vapor cavity formation and collapse is certainly possible for the system.

Lastly, Scenarios 16A through 17C test the system response to routine single pump start and stop operations with the proposed upgrades. Scenarios 16A, 16B, and 16C consider single pump starts at Lift Stations B-19, B-20, and B-1, respectively, while Scenarios 17A, 17B, and 17C consider single pump stops at the same respective facilities. From the results for the pump start scenarios (Figure 28 and Figure 33 in Appendix B), rather large upsurge pressures are predicted, with that at Lift Station B-19 reaching an estimated 120 psi and that at Lift Station B-20 reaching an estimated 110 psi. As with Scenarios 5A and 5B for the existing system, the results for the pump stop tests with the proposed upgrades (Scenarios 17A and 17B; Figure 31 and Figure 32 in Appendix C) show negative pressures following full pump stoppage in the system, with negative pressures reaching partial vacuum conditions along much of the east forcemain.

Proposed Upgrades with Surge Protection

From the simulation results for Scenarios 1A through 17B, the key hydraulic transients challenge in the South Beaches forcemain system (with or without the proposed upgrades) concerns negative pressures. Negative pressures reaching full vacuum are predicted throughout the system under a power failure condition. This can give rise to vapor cavity formation and collapse, which itself creates extreme local pressures and stresses that exacerbate pipe wear, ultimately and likely leading to pipe failures. The risk of negative pressures has also been confirmed via the parallel field pressure monitoring study. Two alternatives for controlling negative pressures that are considered as part of this analysis are (i) installing CAVs[2] at critical high points that will allow air to enter the forcemain under vacuum conditions to prevent local vapor cavity formation or (ii) introducing pressurized storage (e.g., surge tanks, aka HACs) that alleviates downsurges and thus negative pressures.

Scenarios 18A through 20C test the benefits of installing dedicated surge protection. The scenarios are based on the worst-case Scenarios 7A and 9A, which feature the largest predicted vapor cavity volumes. Scenarios 18A to 19B test the potential benefits of installing CAVs at forcemain high points located roughly 11,300 ft from Lift Station B-19 (near South Patrick Drive at Island Drive), 19,330 ft from Lift Station B-19 (near South Patrick Drive at Indrio Boulevard), and 47,210 ft from Lift Station B-19 (near North Shannon Avenue at Fifth Avenue) where previous scenarios show large vapor cavities forming. Scenarios 20A to 20C explore the system response with HACs (i.e., pressurized surge tanks) installed at Lift Stations B-19 and B-20. Table 8 lists the surge protection details.

[2] Combination air valves (CAVs) are a dual function air valve. Under pressurized conditions, they allow the controlled release of accumulated air through a small (and large) orifice, and under vacuum conditions, air enters the pipe through a larger orifice to alleviate negative pressures.

Table 8: List of surge protection simulated in Scenarios 18A to 20C

Scenario	Parent Scenario	Conditions	Surge Protection
18A	7A	Full Capacity, LWL, GPF+	2-inch standard CAVs at select locations*
18B	9A	Full Capacity at B-20, LWL, LPF+	2-inch standard CAVs at select locations*
19A	7A	Full Capacity, LWL, GPF+	2-inch three-stage CAVs at select locations*
19B	9A	Full Capacity at B-20, LWL, LPF+	2-inch three-stage CAVs at select locations*
20A	7A	Full Capacity, LWL, GPF+	4,000 gal HAC at B-19
20B	7A	Full Capacity, LWL, GPF+	10,000 gal HAC at B-20
20C	7A	Full Capacity, LWL, GPF+	4,000 gal HAC† at B-1
20D	7A	Full Capacity, LWL, GPF+	3,000 gal HAC† at B-19 6,000 gal HAC† at B-20

* CAVs were modeled at roughly 11,300 ft (near South Patrick Drive at Island Drive), 19,330 ft (near South Patrick Drive at Indrio Boulevard), and 47,210 ft (near North Shannon Avenue at Fifth Avenue) from Lift Station B-19.

+GPF = global power failure and LPF = local power failure.

† HAC = hydropneumatic air chamber with a bladder separating the liquid and gas phases. For Scenario 20A, the Lift Station B-19 HAC was modeled with a 5-inch connection, and for Scenario 20B the Lift Station B-20 HAC was modeled with an 8-inch connection. The Lift Stations B-19 and B-20 HACs were modeled with 4-inch and 6-inch connections, respectively, for Scenario 20C. The bladders were modeled with a pre-charge pressure of 10 psi, yielding initial air volumes of 22% and 24% for Lift Stations B-19 and B-20, respectively.

Table 11 in Appendix A summarizes the simulation results for Scenarios 18A through 20C. In general, the Scenarios 18A to 19B results (Figure 34 to Figure 37 in Appendix B) show that CAVs help alleviate the formation of some vapor cavities. However, negative pressures are still problematic along the entire forcemain, as the CAVs mostly provide localized benefits in their vicinity. Furthermore, reliance on multiple CAVs for surge protection requires all critical CAVs to be operational and in working order, and this can seldom be ensured for wastewater systems where solids in the fluid can damage CAVs.

Scenario 20A tests the benefits of installing a 4,000 gal HAC only at Lift Station B19. Similarly, Scenario 20B tests a 10,000 gal HAC only at Lift Station B-20. Tank sizes of 2,000 gal were initially tested (the results are not reported here) based on PBS&J's surge analysis report, and the current sizes were determined as the minimum necessary to keep the tanks from draining. The Scenario 20A results (Figure 38 and Figure 39 in Appendix B) show that an HAC at Lift Station B-19 helps reduce the formation of vapor cavities, though there are still some along the forcemain. Comparatively, the results for Scenario 20B with an HAC at Lift Station B-20 (Figure 40 and Figure 41 in Appendix B) show greater improvement over the parent Scenario 7A as well as Scenario 20A, with negative pressures eliminated along a majority of the east forcemain. To explore the system response with an HAC installed at Lift Station B-1, Scenario 20C tests a 4,000 gal HAC (the minimum size from Scenarios 20A and 20B). The results (Figure 42 and Figure 43 in Appendix B) show little change in the overall system response, indicating that a larger HAC is required and that Lift Station B-1 is likely not an optimal location due to being a smaller lift station and thus less severe source of transient pressures. Lastly, the results for Scenario 20D (Figure 44, Figure 45, and Figure 46 in Appendix B) suggest that installing an HAC at each of Lift Station B-19 (3,000 gal) and Lift Station B-20 (6,000 gal) nearly eliminates negative pressures altogether, with a few sections experiencing pressures down to -5 psi. These results indicate that HACs are the most effective alternative for controlling transient pressures in the system, and though not tested here, they would also benefit the system for routine pump starts and stops should no changes be made to the pump start-stop controls. The HAC sizes listed above are the minimum necessary to prevent draining, and larger sizes will provide an even greater benefit to the system in controlling transient pressures and likely even eliminating the occurrence of negative pressures.

CONCLUSIONS AND RECOMMENDATIONS

In response to previous failures in the South Beaches forcemain system, HDR was retained by Brevard County to review previous studies, conduct a transient pressure monitoring exercise, and propose alternatives for improving the system. In support of these, HydraTek performed a review of the field monitoring data, and a numerical hydraulic transient (surge) analysis was completed parallel thereto to analyze the existing system, proposed upgrades, and identify potential improvements. As with most flat forcemains, the key hydraulic transient challenge for the South Beaches system concerns negative pressures and frequent pump start-stop operations; moreover, the system response is complex and largely influenced by the mixed use of ductile iron and PVC pipes. Based on the review of the field monitoring data and numerical modeling analysis, the following concluding remarks and recommendations are provided to support HDR's work:

1. From a review of the transient pressure field monitoring data, the maximum pressures recorded at Lift Stations B-19 and B-1 are 148 psi and 130 psi, respectively. The average high pressures recorded at Lift Stations B-19 and B-20 are also 77 psi and 54 psi, respectively. At the time of this analysis, the working and total (operating plus surge) pressure ratings of the system's ductile iron and PVC pipes were not available; accordingly, the actual working and total pressure ratings of the ductile iron and PVC forcemain sections should be confirmed and compared against the aforementioned values.

2. Although the maximum recorded transient pressures are likely within the forcemain's upper pressure limit for the PVC and ductile iron pipe, it is quite possible that the combined effect of frequent pump start-stop events; a lack of suitable hydraulic control during such operations; and potential forcemain draining, vapor cavity collapse, or rapid air expulsion contributed to or exacerbated the previous failures.

3. The numerically predicted negative pressures should be avoided for the sake of maintaining joint seals – see ASTM D3139-98 (Standard Specification for Elastomeric Seals (Gaskets) for Joining Plastic Pipes), which recommends a minimum partial vacuum pressure of -10 psi; otherwise, repeated positive-negative pressure cycles can wear the seals, leading to leakage and ultimately joint failure.

4. In addition to the above, a fatigue loading analysis can be performed for the PVC section of the forcemain to determine whether repetitive pressure cycling from frequent pump start-stop operations is contributing to the forcemain failures. Such an analysis should consider a pressure range of -10 psi to 90 psi (the average recorded upsurge pressure) and a frequency of 30 complete cycles per system per day.

5. The field monitoring data for Lift Station B-1 shows pressure traces that are characteristic of check valve slam following pump stops. Check valve slam can create significant pressure spikes, and it causes high local stresses that can lead to or cause pipe and valve failure. The design of the proposed upgrades for Lift Station B-1 should consider check valve slam and the selection of better quality check valves. This may be accomplished by using valves with shorter closure strokes/angles, and either check valve accelerators (to close the valve faster before the arrival of a return wave) or dampeners (to cushion and slow the valve's closure).

6. It appears that the motor soft-starters at Lift Stations B-19 and B-20 provide less than adequate hydraulic control for pump starts (for example, as shown in Figure 57). There is also inadequate control for pump stops, and the field data shows that Lift Stations B-19, B-1, and likely B-20 routinely experience negative pressures reaching partial vacuum conditions following full pump stoppage for the system. The resulting peak pressures are also on the order of two to three times as high as the average operating pressures. Furthermore and as is typical of sewage forcemain systems, pump start and stop operations are rather frequent and likely contributing to cyclic fatigue of the piping and exacerbating failures. Consideration should thus be given to improving hydraulic control for pump starts and stops to better control transient pressures.

7. Two alternatives for better controlling pump operations are discharge control valves and VFDs. Discharge control valves provide direct hydraulic control by modulating the head and flow of individual pumps, whereas VFDs control the operating speed of a pump and thus its pump curve and operating point. Using either alternative, operations can be performed more slowly during pump starts and stops, thereby minimizing transient pressures and the shock to the system. Between control valves and VFDs, the former tend to be less costly and provide more direct control, though they are less common in wastewater applications due to the nature of the fluid. Comparatively, VFDs, while more expensive, can be used to operate the pumps at lower speeds during low inflow periods to minimize unnecessary pump on-off operations while improving operational efficiency. Another option is to stagger the pump start-stop wet well levels by a larger distance than is presently set to reduce the frequency of start-stop changes.

8. The data for Lift Station B-19 shows high frequency and large transient pressure fluctuations following full pump stoppage for the system (i.e., with no pumping from Lift Stations B19, B-20, and B-01). This is likely the result of either or a combination of: (i) sections of the forcemain draining, refilling, and re-pressurizing, (ii) rapid air intake, accumulation, or expulsion, or (iii) vapor cavity formation and collapse. The cause of these pressure fluctuations should be further investigated, starting with an investigation as to why the system pressures are low and even negative following complete stoppage. This may be tested by fully stopping all pumps for a period of at least 5 minutes (e.g., during low flow periods overnight), wet well levels permitting, and recording the system pressures at Lift Stations B-19, B-20, and B-1 to explore what the static system hydraulics are without pumping (and in comparison to an expected static HGL based on the WWTP outlet elevation of 30.5 ft.) A flow meter suited to wastewater applications can also be used to monitor potential backflow and forcemain draining. If not properly mitigated, this phenomenon may further contribute to failures by causing rapid and significant stress changes.

9. Further to the above, the original design and forcemain should be reviewed to inspect whether there are existing in-line check valves at any location, including at the WWTP. If not, consideration should be given to exploring the possibility of installing in-line check valves along the forcemain to prevent it from draining and depressurizing under conditions with no pumping.

10. The field monitoring data should be compared against the County's SCADA data for the South Beaches system to correlate operational changes to observed events. Furthermore, the system response and behavior can be further investigated with a coordinated transient pressure monitoring investigation, whereby more pressure loggers are installed and controlled, planned tests (such as pump emergency stops) are performed.

11. The proposed 30-inch ductile iron forcemain replacement and Lift Station B-1 upgrade will increase the system's capacity and thus maximum total flow. Despite this, the system response is expected to remain comparable to its current state, featuring negative pressures reaching full vacuum conditions along the forcemain and potentially with vapor cavity formation and collapse. Negative pressures can damage pipe seals, particularly for the PVC sections of the forcemain, and vapor cavity collapse creates high localized pressures and extreme stresses on the forcemain.

    Regarding the proposed 30-inch ductile iron forcemain replacement, the corresponding pipe class can be chosen according to the maximum expected operating pressure rather than transient pressure. For the Lift Station B-1 upgrades, check valve slam should be appropriately considered with design pressures based on the field monitoring data. It is also recommended that consideration be given to installing either discharge control valves or VFDs for new pump installations and modifications (e.g., at Lift Station B-1) to improve the hydraulic control of routine pump start and stop operations and thus minimize the transient pressures and shock to the system. All piping and appurtenances should also be designed to withstand full negative pressures reaching up to -14.7 psi, as well as have adequate bedding and structural support to accommodate frequent loading cycles.

12. Presently the South Beaches forcemain does not have any dedicated surge protection. The surge valves at each of Lift Stations B-19 and B-20 are currently isolated, and all air valves installed along the forcemain are air release (outflow only) valves. From the field data, numerical modeling results, and history of failures, the South Beaches system would benefit from the installation of dedicated surge protection. Two alternatives for this are (i) installing combination air valves (CAVs) along the forcemains at critical locations, namely local high points, or (ii) installing hydropneumatic air chambers (HACs; i.e., surge tanks) at Lift Stations B-19 and B-20.

    The analysis explored the potential benefits of installing CAVs at select locations on the east forcemain. The results suggest that CAVs help alleviate the formation and collapse of some vapor cavities, including the subsequent pressure shocks and high local pipe stresses that may otherwise exacerbate or cause failures. Though beneficial, CAVs are expected to only provide localized benefits to the system where they are installed, and negative pressures remain problematic along nearly all of the forcemain. Moreover, reliance on multiple CAVs for surge protection requires that all of the CAVs be maintained, operational, and reliable, and CAVs are typically less reliable than surge tanks and for wastewater applications. Thus, CAVs alone are insufficient unless combined with other forms of surge protection and better hydraulic control for pump start and stop operations.

    Numerical simulations were also performed to assess the benefits of installing HACs (i.e., pressurized surge tanks) at each of the larger Lift Stations B-19 and B-20. The model results suggest that suitably sized HACs (in the range of 4,000 gallons to 10,000 gallons per facility) can eliminate nearly all negative pressures along the forcemain. This is highly beneficial for avoiding vapor cavity formation, frequent and rapid pressure cycling, and wear on joint seals. Given the system's history of failures, this option should be given further consideration. The current study only explored a limited number of HAC combinations to demonstrate their potential benefits, with the sizes mainly determined to avoid tank draining. A more detailed analysis can be conducted to optimize the number of HACs, their sizes and locations, and performance – for instance, designing the tanks to not only avoid draining but also have a minimum allowable water volume/content under transient conditions (e.g., a minimum 10% of the overall tank volume) and/or a minimum water depth (i.e., to avoid vortex action at the outlet).

13. During and following commissioning of the proposed upgrades, transient pressure monitoring can be carried out to further identify and monitor the overall risk to the system. This would require the installation of high frequency transient pressure monitors similar to those used by Blacoh and ideally ones that can remotely transmit the data for ongoing review and confirmation.

14. Following a power failure event (or emergency pump stop or trip), Lift Stations B-19, B-20, and B-1 should stay idle for a period of at least 3 minutes in order to allow the transient pressure waves to dissipate. Therefore, the pumps should not be allowed to start (via utility or backup power) within this designated standby period unless the wet well level is at the high-high alarm. The mixed pipe material nature of this overall system further increases the risk of pressure wave superimposition and amplification.

APPENDIX A: TABULAR SUMMARY OF NUMERICAL MODELING RESULTS

Table 9: Hydraulic transient simulation results for existing conditions – Scenarios 1A to 5B

Scenario	Parent Scenario	Initial Lift Station Flows (gpm)					20-inch Forcemain			24/30-inch Force	
		B19	B20	B01	B06	B07	Min. Pressure (psi)	Max. Pressure (psi)	Max. Vapour Vol. (gal)	Min. Pressure (psi)	Max. Pressure (psi)
1A	-	2,289	3,359	962	-	-	**-14**	**108**	3	**-14**	**97**
1B	1A	2,241	3,380	889	-	-	**-14**	106	5	**-14**	95
2	-	4,529	0	0	-	-	**-14**	96	**14**	**-14**	58
3	-	0	5,597	0	-	-	**-14**	81	9	**-14**	79
4A	-	0	0	0	-	-	11	95	0	0	70
4B	-	0	0	0	-	-	11	88	0	0	82
5A	-	3,565	0	0	-	-	-10	68	0	-10	43
5B	-	0	4,143	0	-	-	-12	53	0	**-14**	52
Min.	-	-	-	-	-	-	-14	-	-	-14	-
Max.	-	4,529	5,597	962	-	-	-	108	14	-	97

Table 10: Hydraulic transient simulation results for proposed conditions without surge protection – Scenarios 6A tc

Scenario	Parent Scenario	Initial Lift Station Flows (gpm)					20-inch Forcemain			24/30-inch Forc	
		B19	B20	B01	B06	B07	Min. Pressure (psi)	Max. Pressure (psi)	Max. Vapour Vol. (gal)	Min. Pressure (psi)	Max. Pressure (psi)
6A	-	2,406	4,302	1,051	-	-	**-14**	107	6	**-14**	95
6B	6A	2,528	3,945	763	-	-	-14	100	8	-14	86
7A	-	2,355	4,318	973	-	-	**-14**	105	8	**-14**	93
7B	7A	2,477	3,939	717	-	-	-14	98	10	-14	84
8A	-	4,800	0	0	-	-	**-14**	92	25	**-14**	50
8B	8A	4,459	0	0	-	-	-14	82	20	-14	57
9A	-	0	6,758	0	-	-	**-14**	82	41	**-14**	80
9B	9A	0	6,137	0	-	-	-14	79	31	-14	70
10	7A	2,755	5,390	1,694	978	794	**-14**	103	34	**-14**	87
11	7A	2,635	5,060	1,469	1,480	844	-14	104	31	-14	89
12	9A	0	6,758	0	-	-	**-14**	82	40	**-14**	80
13	9A	0	6,758	0	-	-	-14	82	22	-14	80
14	9A	0	6,758	0	-	-	**-14**	82	**46**	**-14**	80
15	9A	0	6,758	0	-	-	**-55**	82	0	**-41**	80
16A	-	3,768*	0	0	-	-	11	120	0	0	92
16B	-	0	4,687*	0	-	-	11	**122**	0	0	**110**
16C	-	0	0		-	-	-11	62	0	-14	60
17A	-	3,768	0	0	-	-	-10	65	0	-10	38
17B	-	0	4,687	0	-	-	-11	49	0	-11	48
17C	-				-	-	-14	40	0	-12	32
Min.	-	-	-	-	-	-	-55	-	-	-41	-
Min.+	-	-	-	-	-	-	-14	-	-	-14	-
Max.	-	4,800	6,758	1,694	1,480	844	-	122	46	-	110

+Excluding the results for Scenario 15 with no vapor cavity formation.

Table 11: Hydraulic transient simulation results for proposed conditions with surge protection – Scenarios 18A to 2(

Scenario	Parent Scenario	Initial Lift Station Flows (gpm)					20-inch Forcemain			24/30-inch Force	
		B19	B20	B01	B06	B07	Min. Pressure (psi)	Max. Pressure (psi)	Max. Vapour Vol. (gal)	Min. Pressure (psi)	Max. Pressure (psi)
18A	7A	2,355	4,318	973	-	-	**-14**	160	2,627	-14	102
18B	9A	0	6,758	0	-	-	**-14**	**169**	**3,003**	-14	88
19A	7A	2,355	4,318	973	-	-	**-14**	128	2,298	-14	93
19B	9A	0	6,758	0	-	-	**-14**	114	2,612	-14	80
20A	7A	2,355	4,318	973	-	-	**-14**	105	8	-14	93
20B	7A	2,355	4,318	973	-	-	-13	105	0	-7	93
20C	7A	2,355	4,318	973	-	-	-14	105	19	-14	93
20D	7A	2,355	4,318	973	-	-	-5	105	0	-3	93
Min.	-	-	-	-	-	-	-14	-	-	-14	-
Max.	-	2,355	6,758	973	-	-	-	169	3,003	-	102

APPENDIX B: GRAPHICAL HYDRAULIC TRANSIENT ANALYSIS RESULTS

This appendix presents the numerical hydraulic transient analysis results for the South Beaches east forcemain system via transient pressure envelope figures. The transient pressure envelope figures are plotted as hydraulic head (pressure) versus distance along the forcemain profile path from Lift Station B-19 to the South Beaches Wastewater Treatment Plant (WWTP). The profile figures should be read from left to right. The left side corresponds to the location of Lift Station B-19, and the right side corresponds to the South Beaches WWTP (refer to Figure 5 on the following page). Lift Stations B-20 and B-1 are located at distances of approximately 17,750 ft and 29,100 ft from Lift Station B-19, respectively. The hydraulic head profiles are representative of that along the red-highlighted line in Figure 5 on the following page.

Unless otherwise noted, all of the values are presented in the units of feet (ft) and all figures can be deciphered using the following legend:

Red Line: Estimated vapor or air cavity volume in gallons

Light Blue Line: Steady-state (operating) hydraulic grade line (HGL)

Blue Lines: Maximum and minimum transient pressure envelope (HGLs)

Green Line: Pipe profile (elevation)

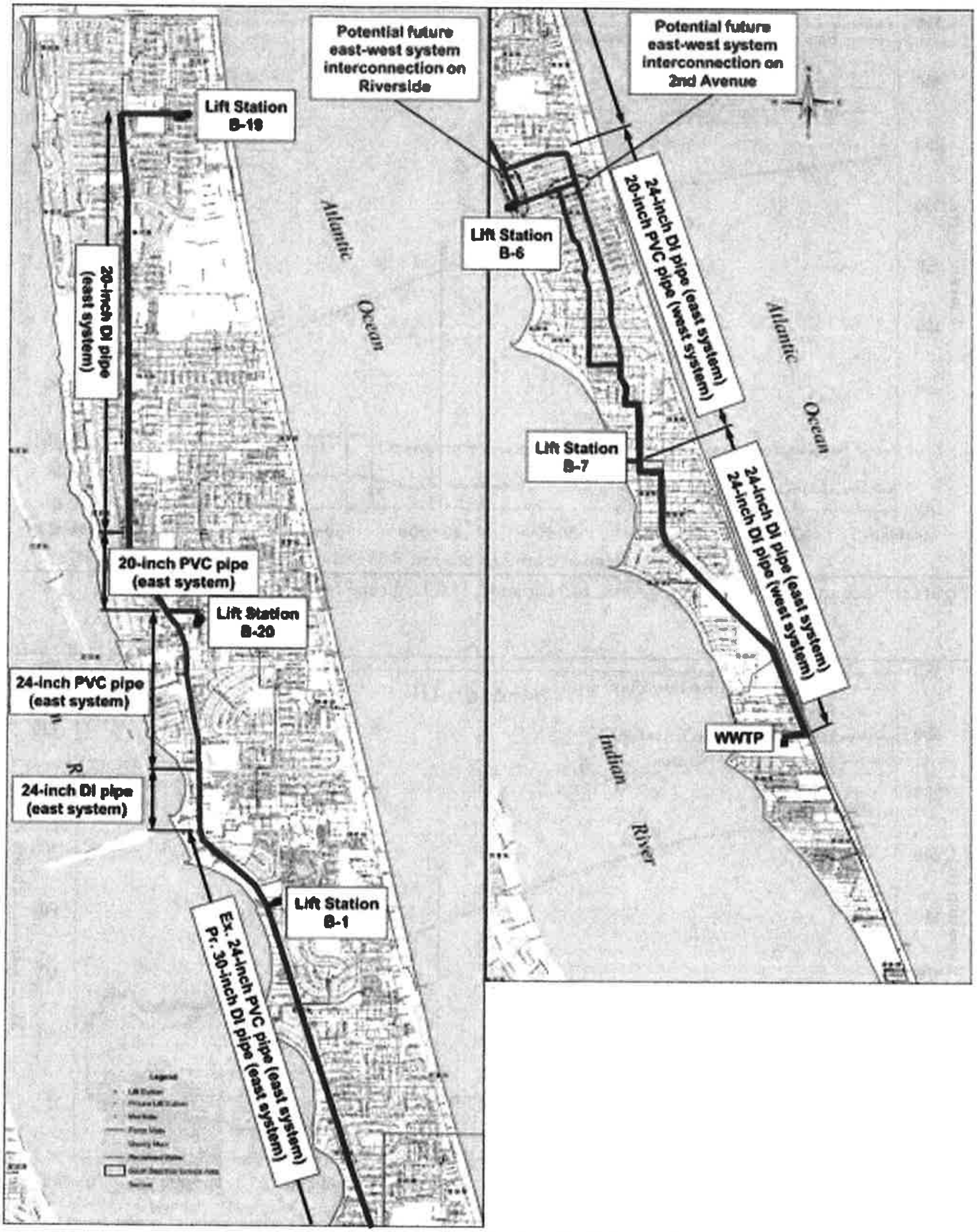


Figure 5: Profile path for analysis results, from Lift Station B-19 (top-left) to the South Beaches WWTP (bottom-right)

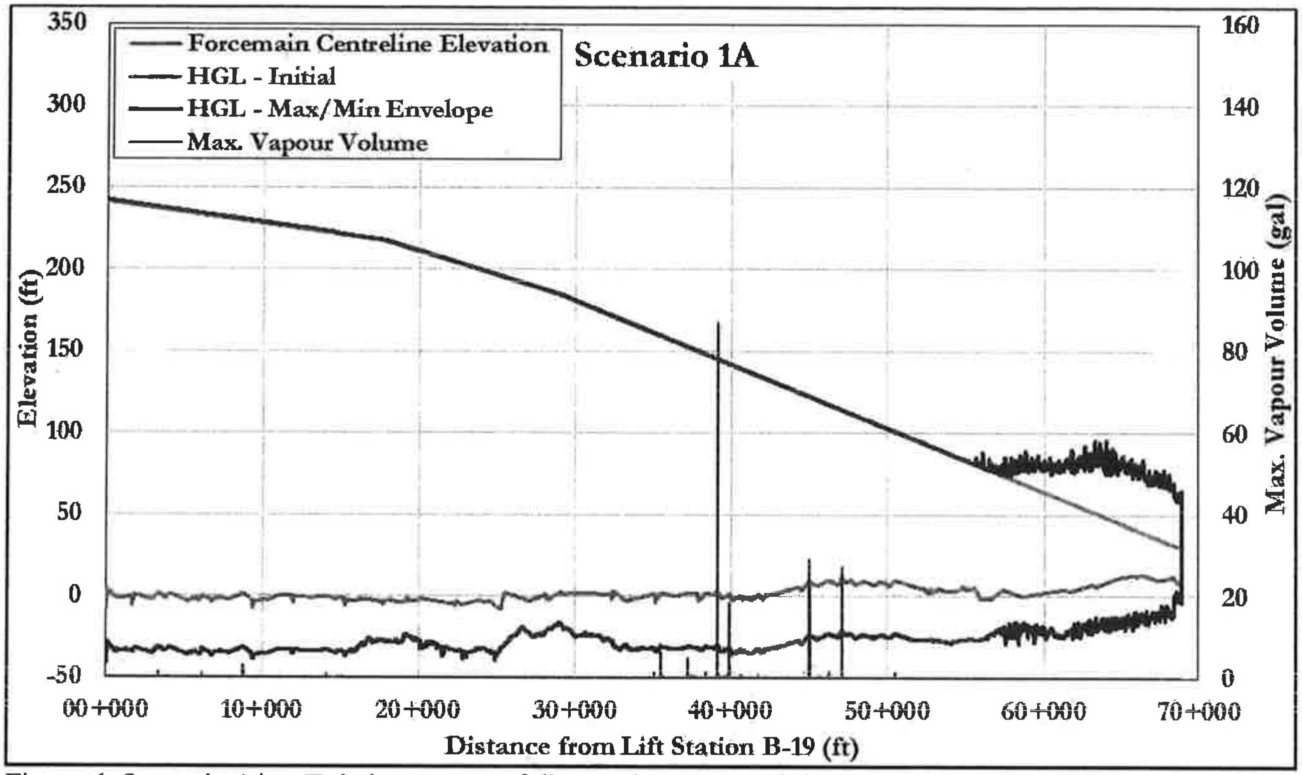


Figure 6: Scenario 1A – Existing system, full capacity, HWL, global power failure

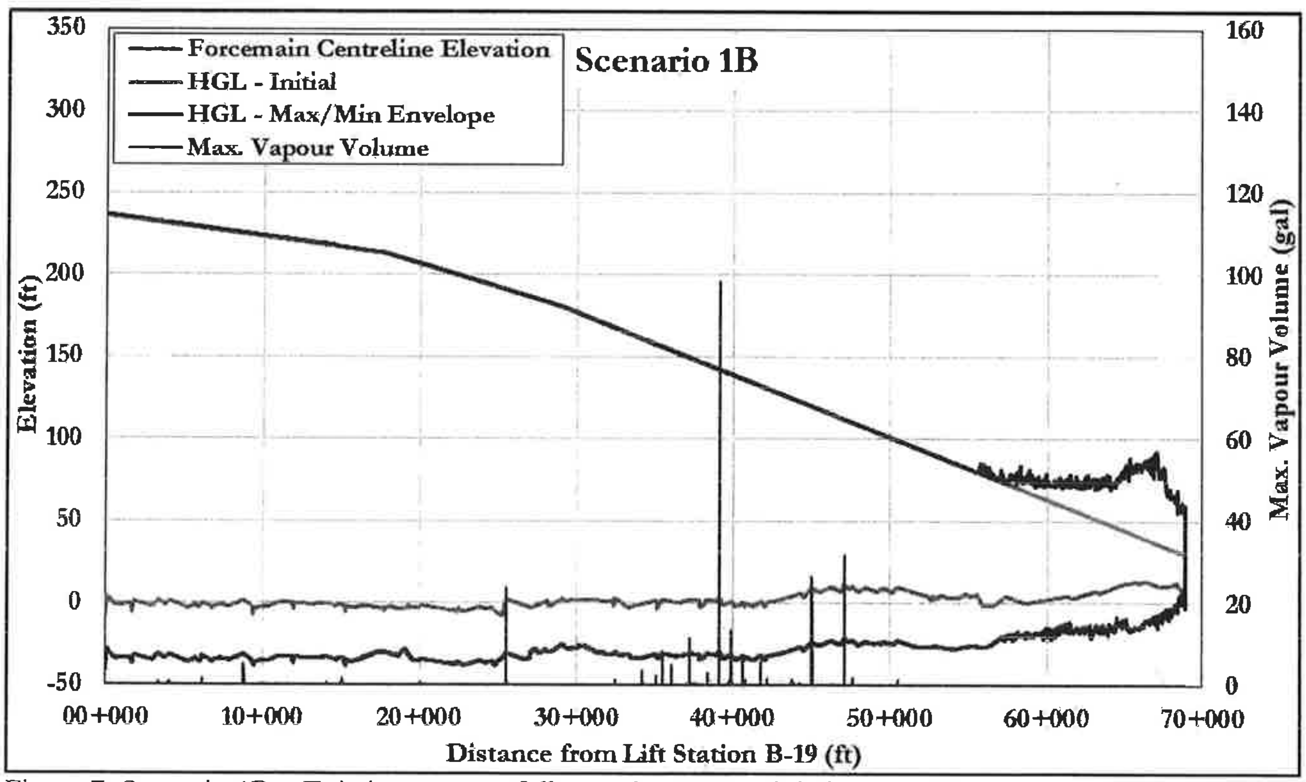


Figure 7: Scenario 1B – Existing system, full capacity, LWL, global power failure

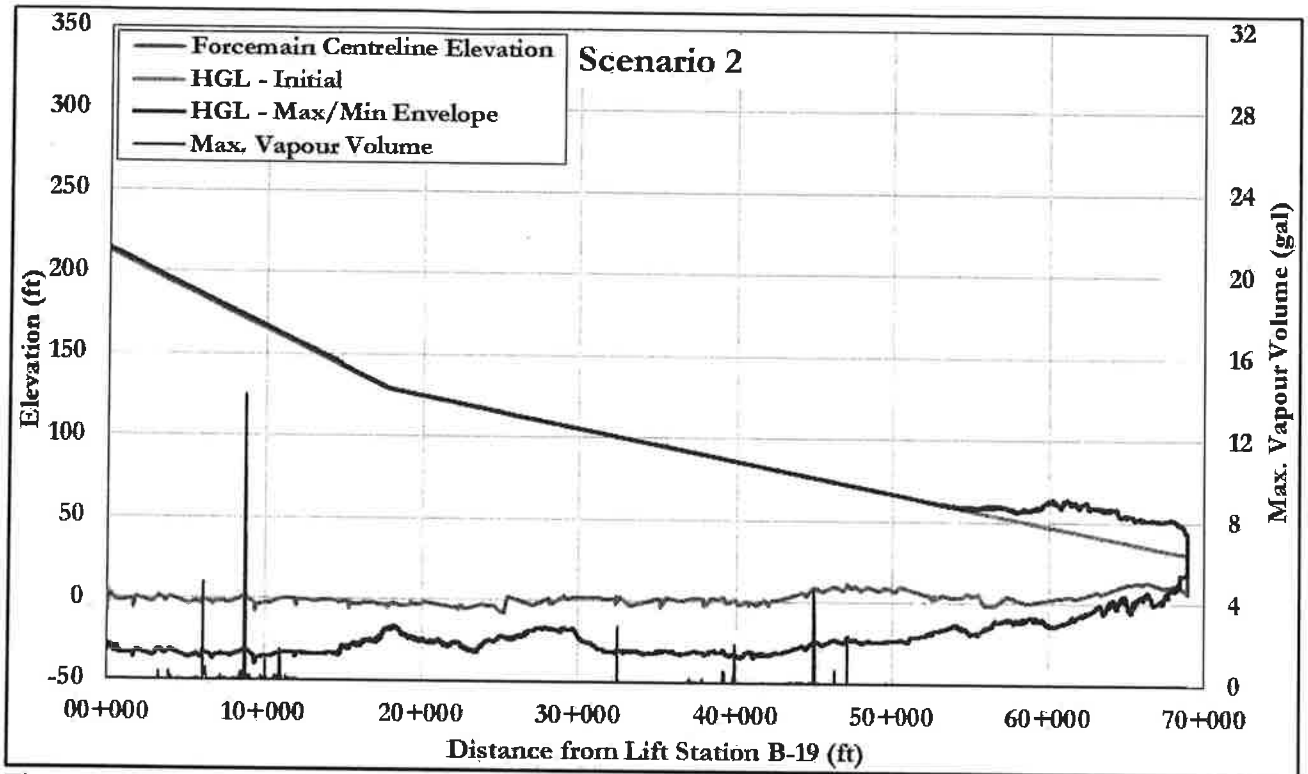


Figure 8: Scenario 2 – Existing system, only B-19 on at full capacity, HWL, local power failure

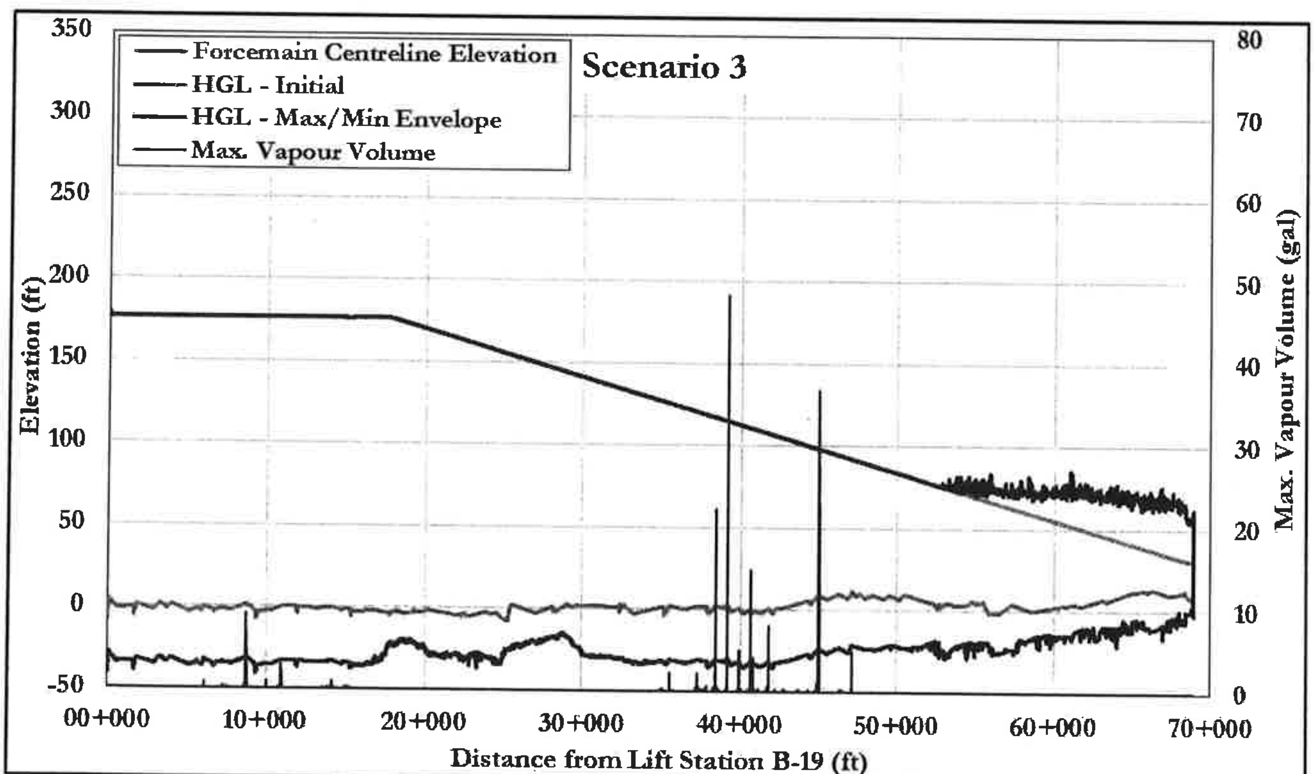


Figure 9: Scenario 3 – Existing system, only B-20 on at full capacity, HWL, local power failure

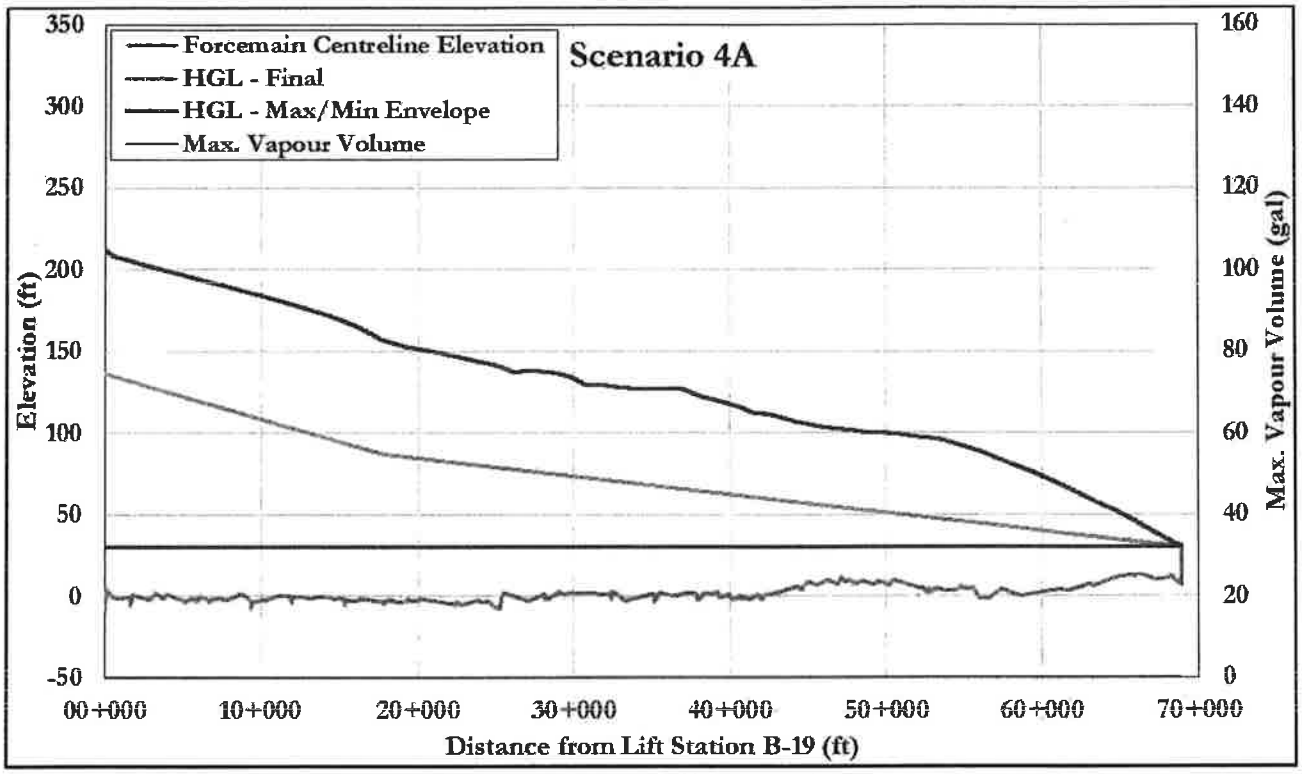


Figure 10: Scenario 4A – Existing system, all pumps off, HWL, single pump start at B-19

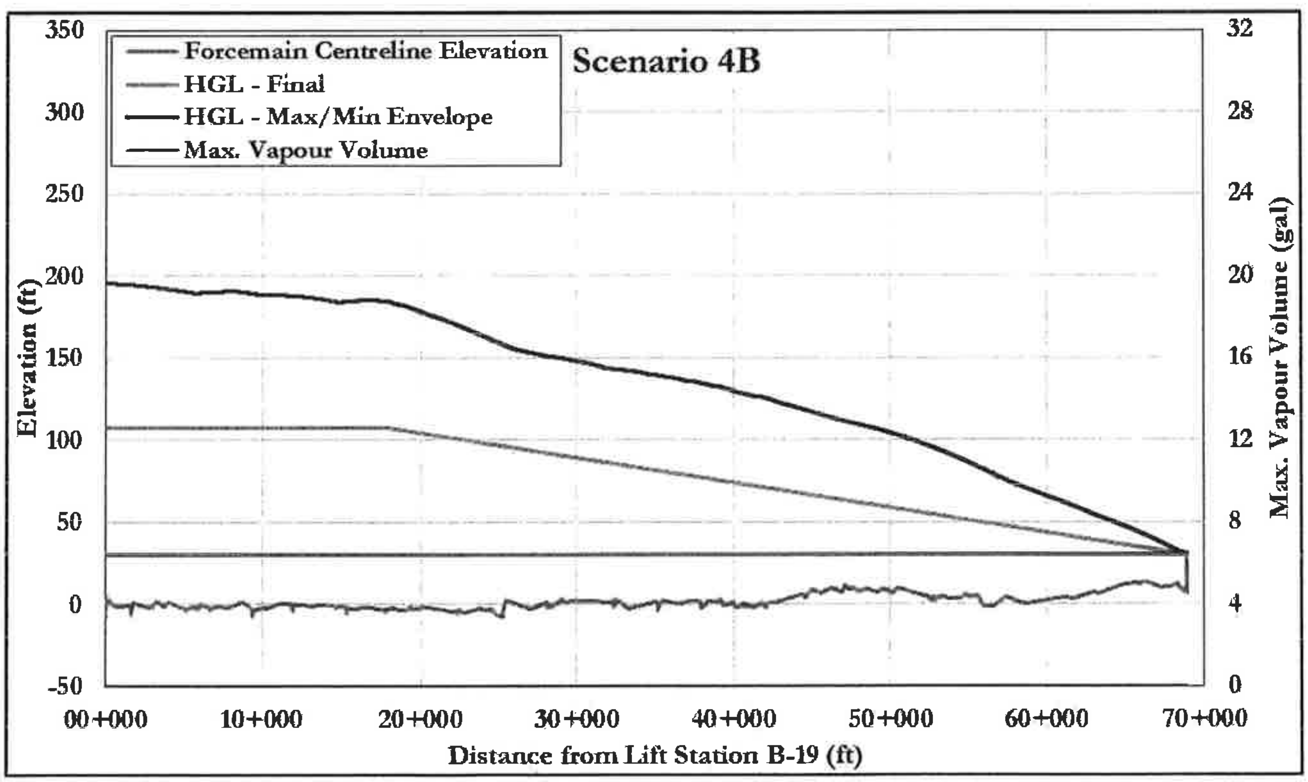


Figure 11: Scenario 4B – Existing system, all pumps off, HWL, single pump start at B-20

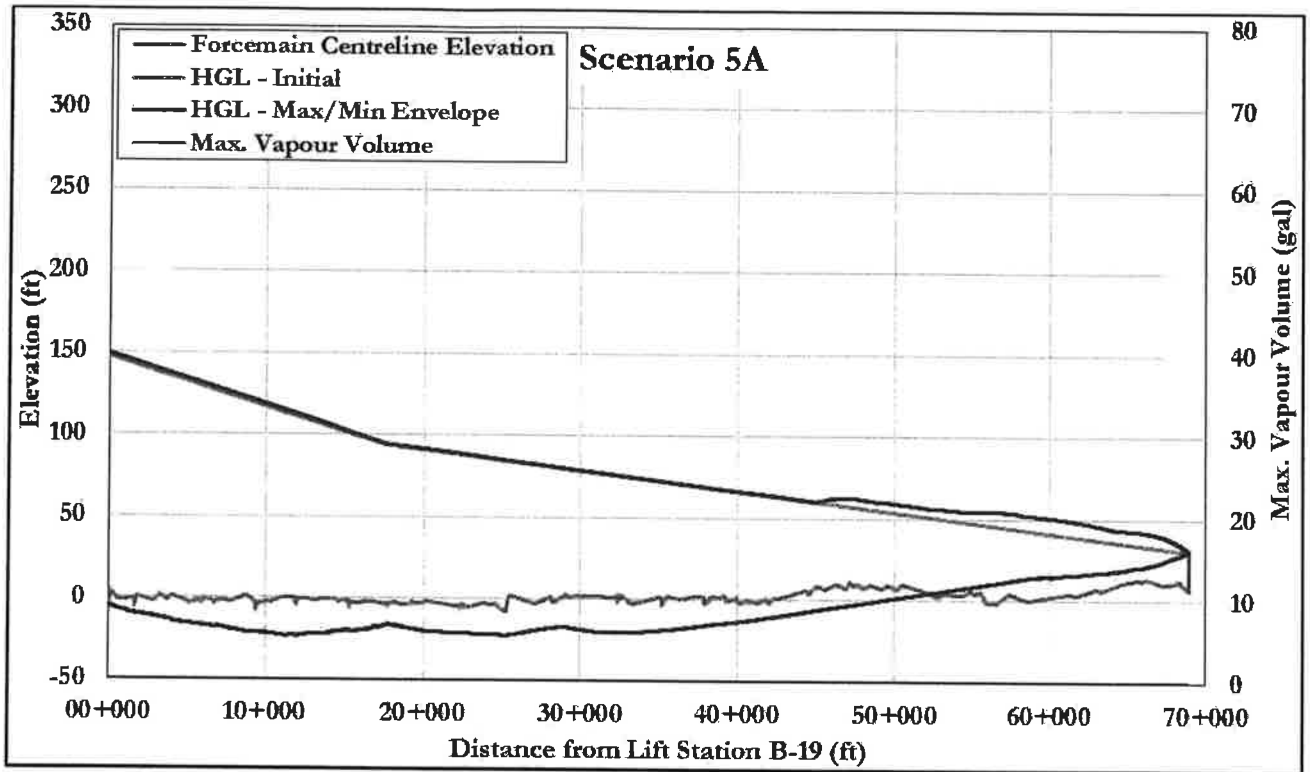


Figure 12: Scenario 5A – Existing system, HWL, single pump stop at B-19

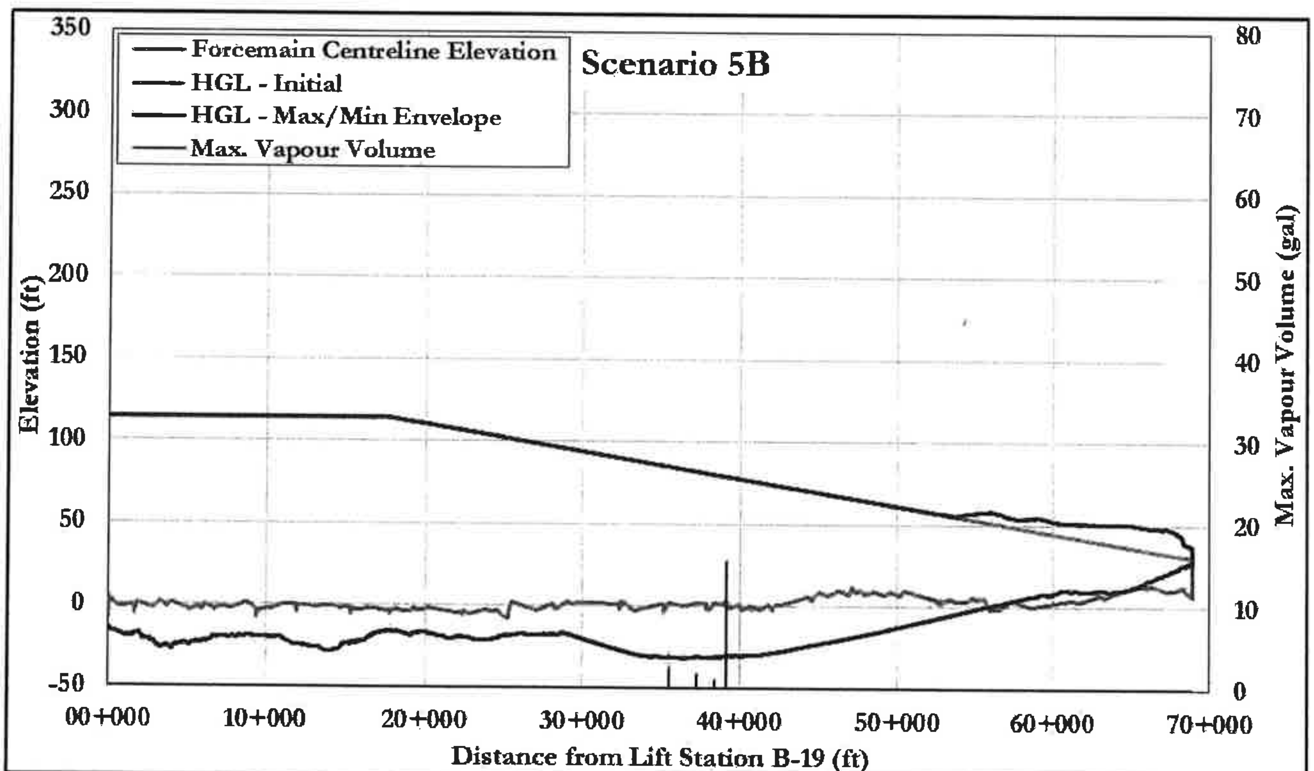


Figure 13: Scenario 5B – Existing system, HWL, single pump stop at B-19

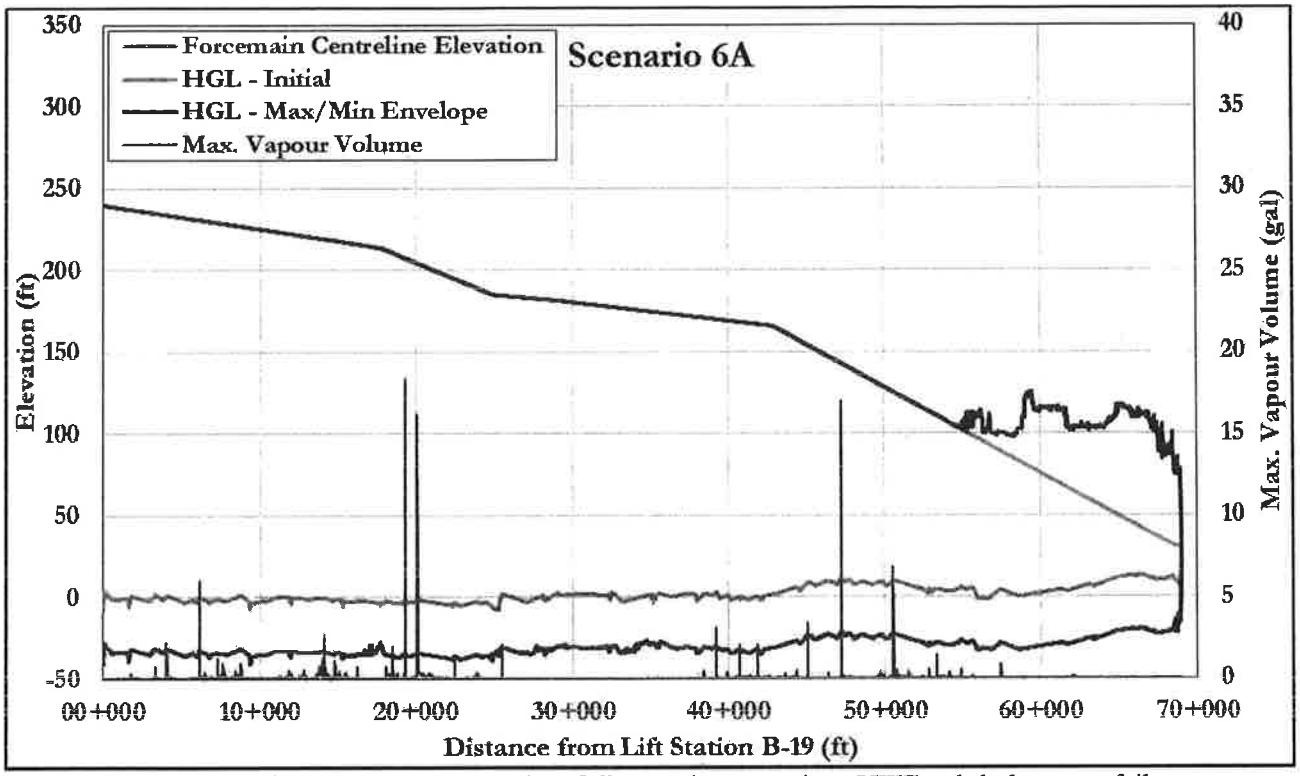


Figure 14: Scenario 6A – Proposed upgrades, full capacity pumping, HWL, global power failure

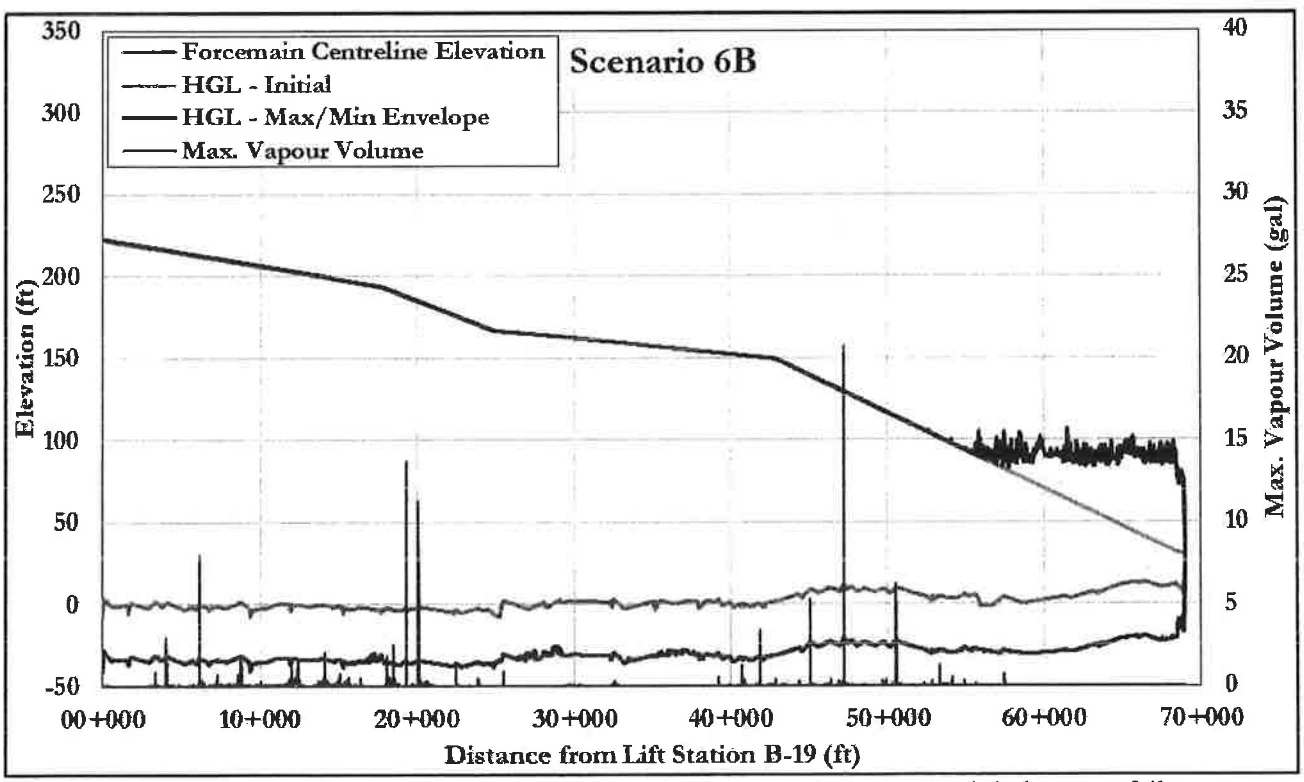


Figure 15: Scenario 6B – Proposed upgrades, firm capacity pumping, HWL, global power failure

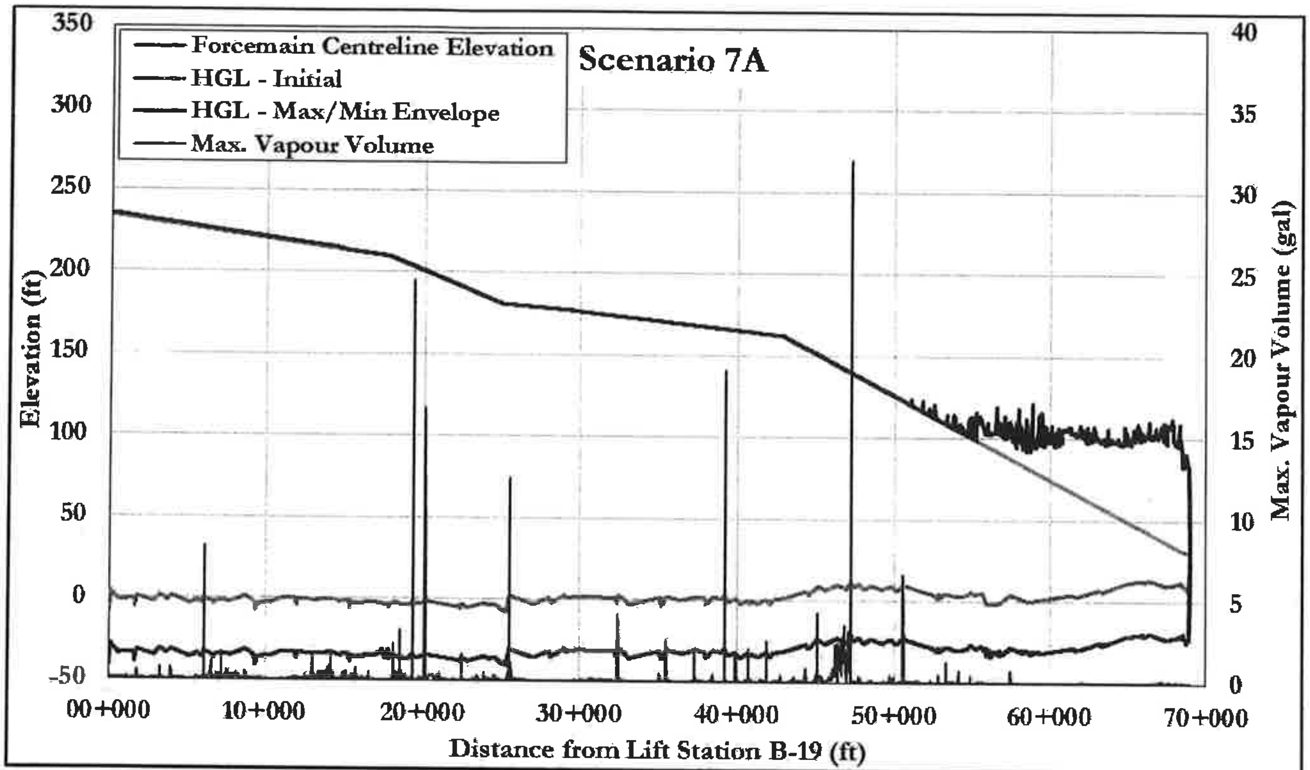


Figure 16: Scenario 7A – Proposed upgrades, full capacity pumping, LWL, global power failure

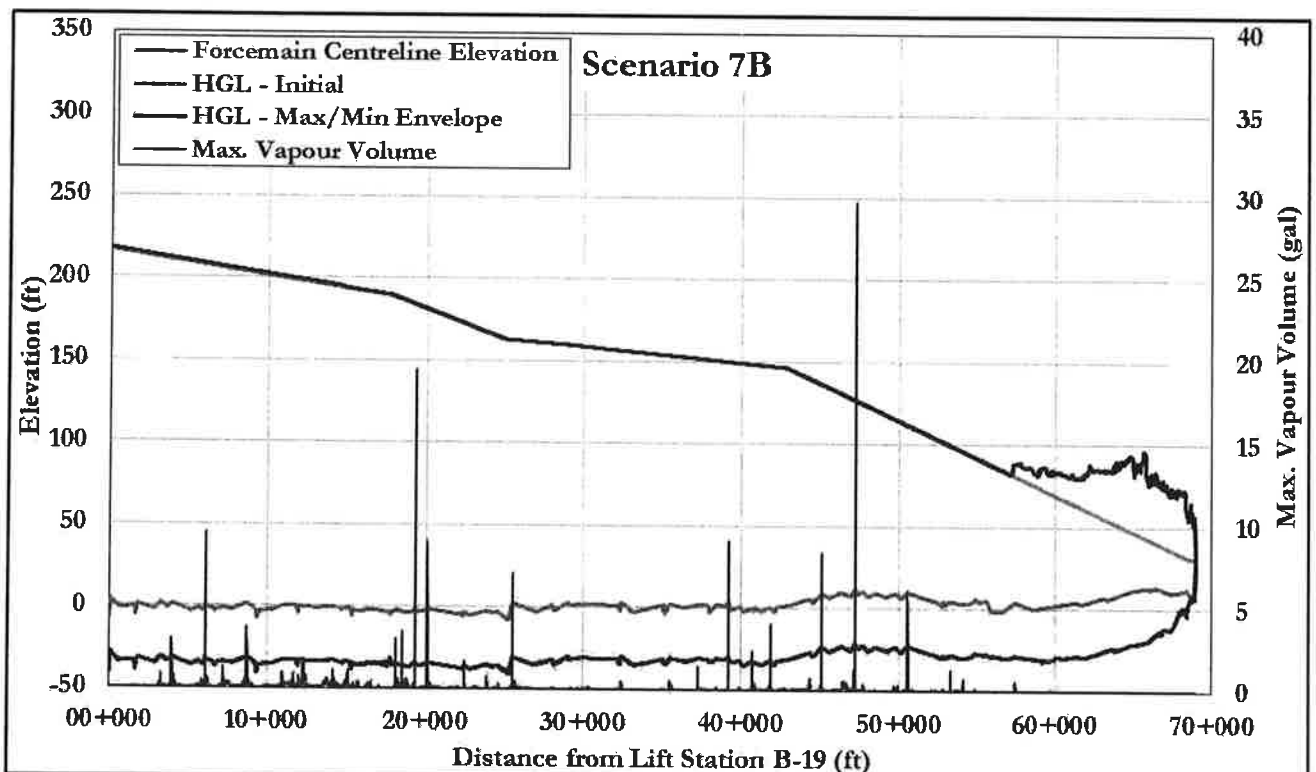


Figure 17: Scenario 7B – Proposed upgrades, firm capacity pumping, LWL, global power failure

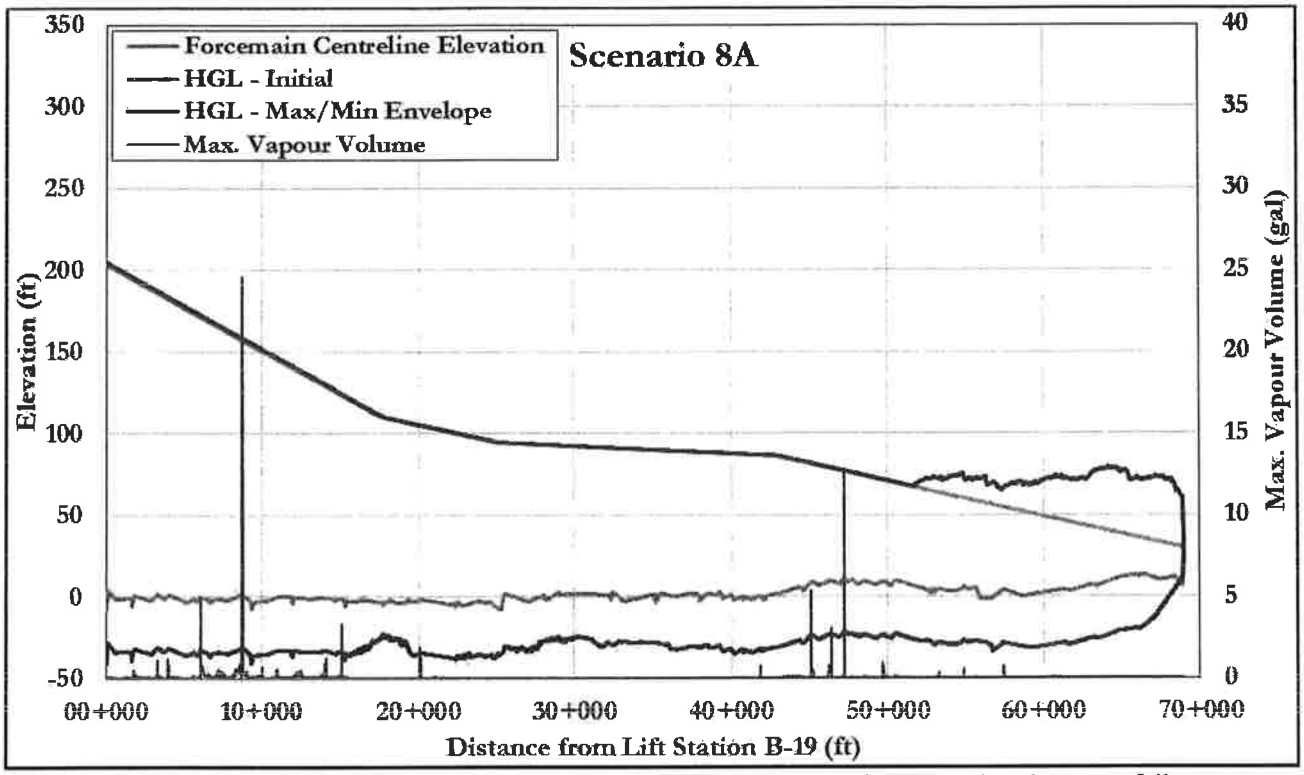


Figure 18: Scenario 8A – Proposed upgrades, full capacity at B-19 only, LWL, local power failure

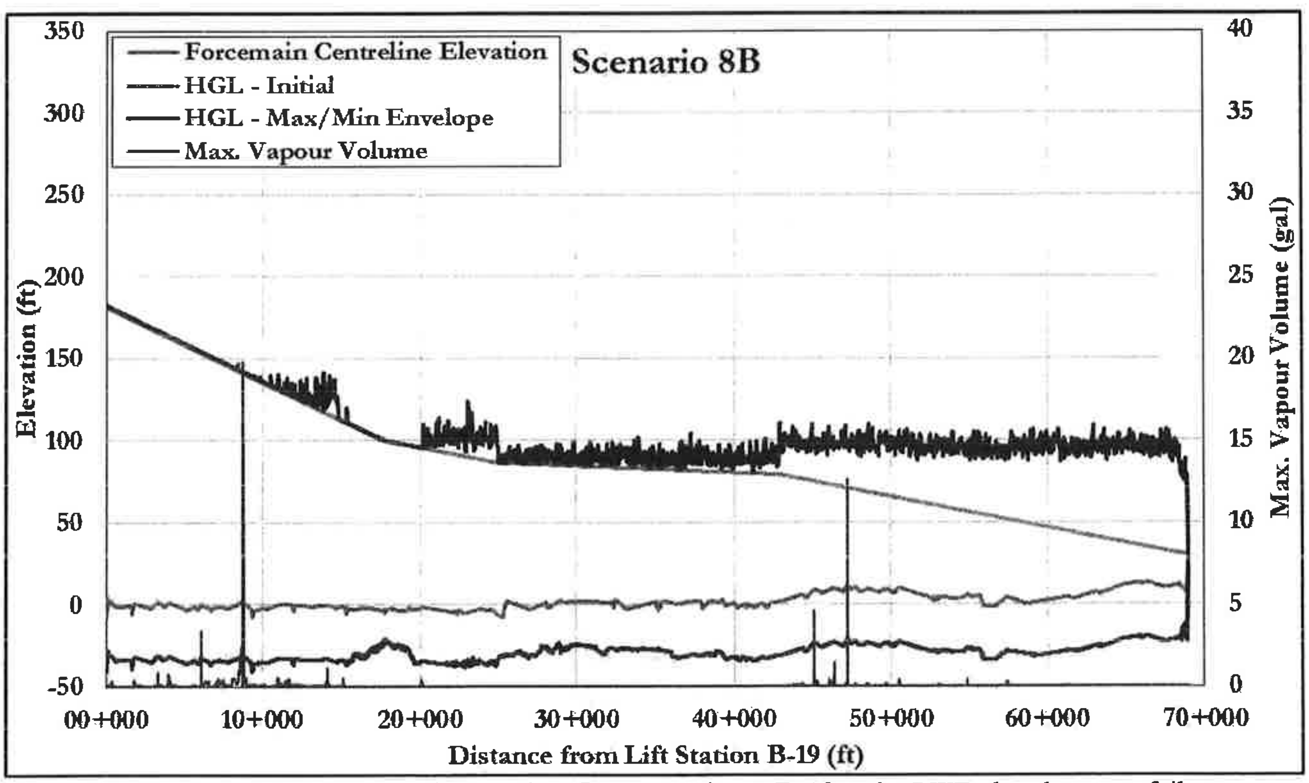


Figure 19: Scenario 8B – Proposed upgrades, firm capacity at B-19 only, LWL, local power failure

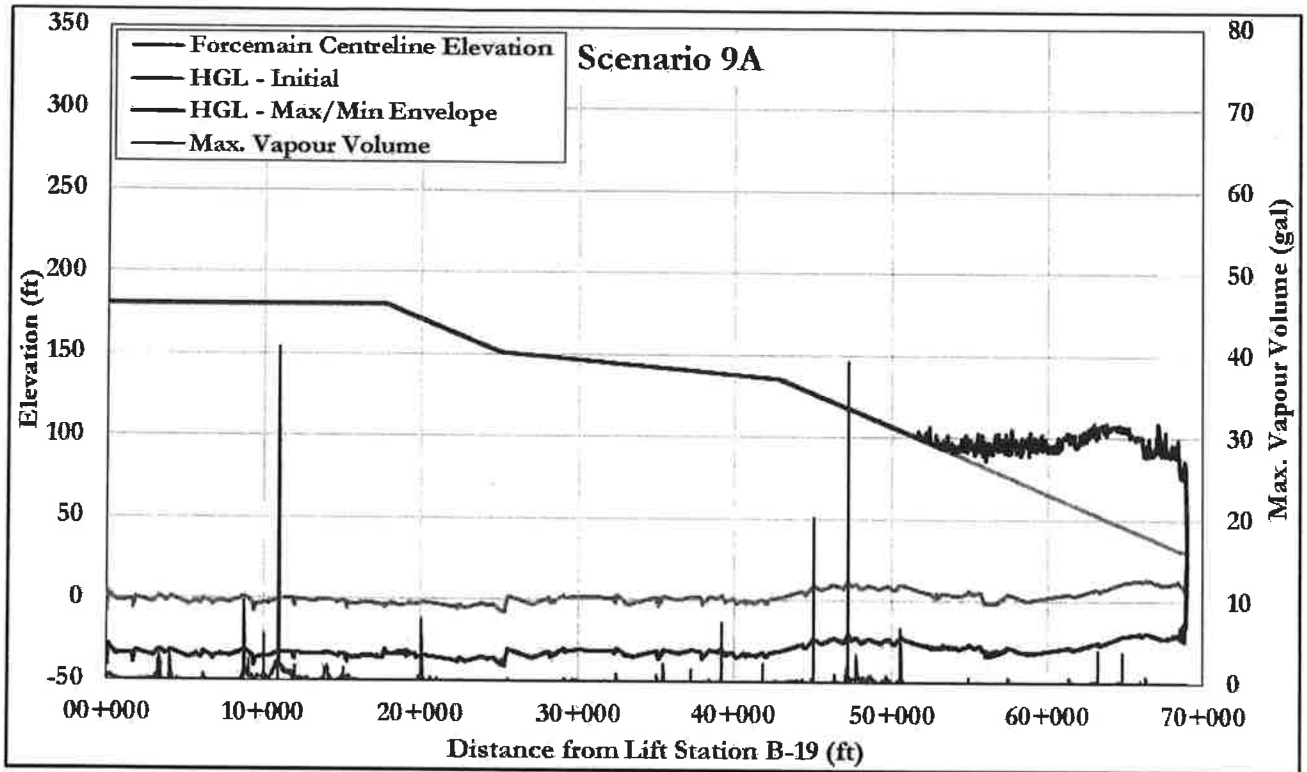


Figure 20: Scenario 9A – Proposed upgrades, full capacity at B-20 only, LWL, local power failure

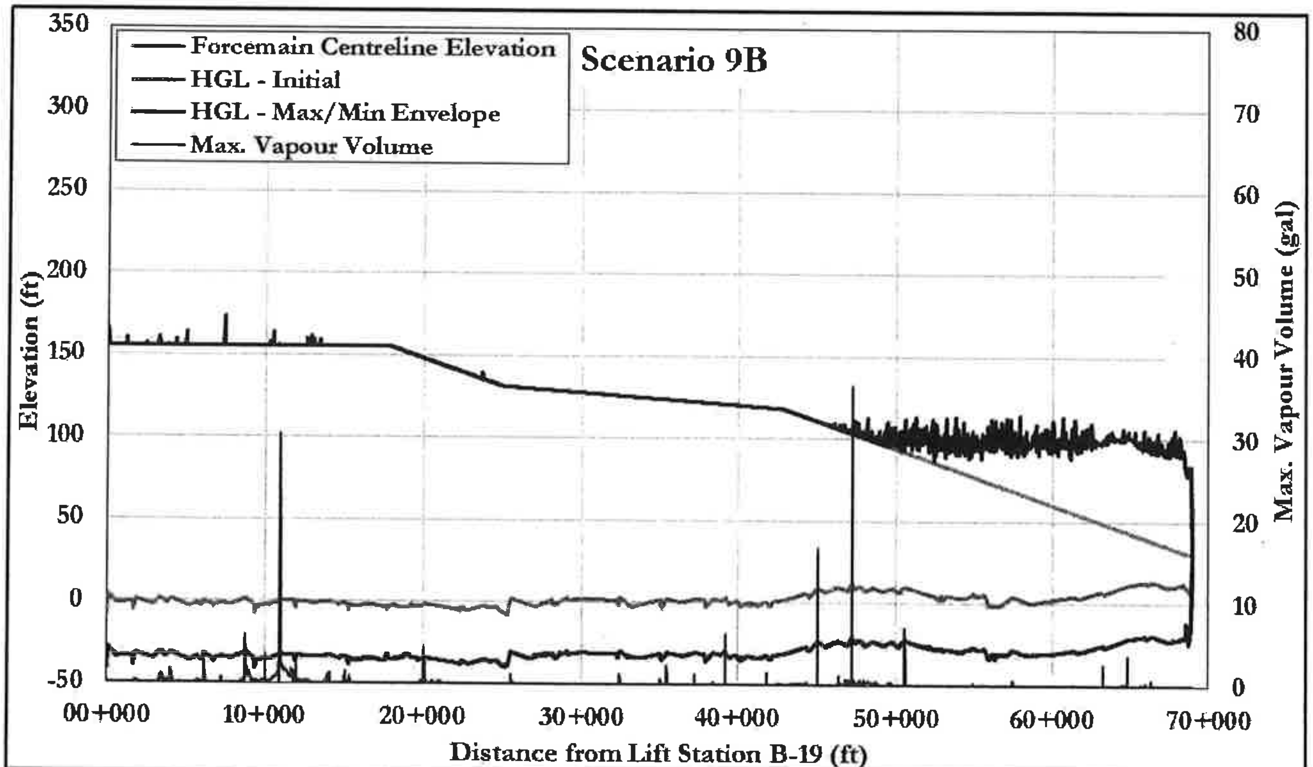


Figure 21: Scenario 9B – Proposed upgrades, firm capacity at B-20 only, LWL, local power failure

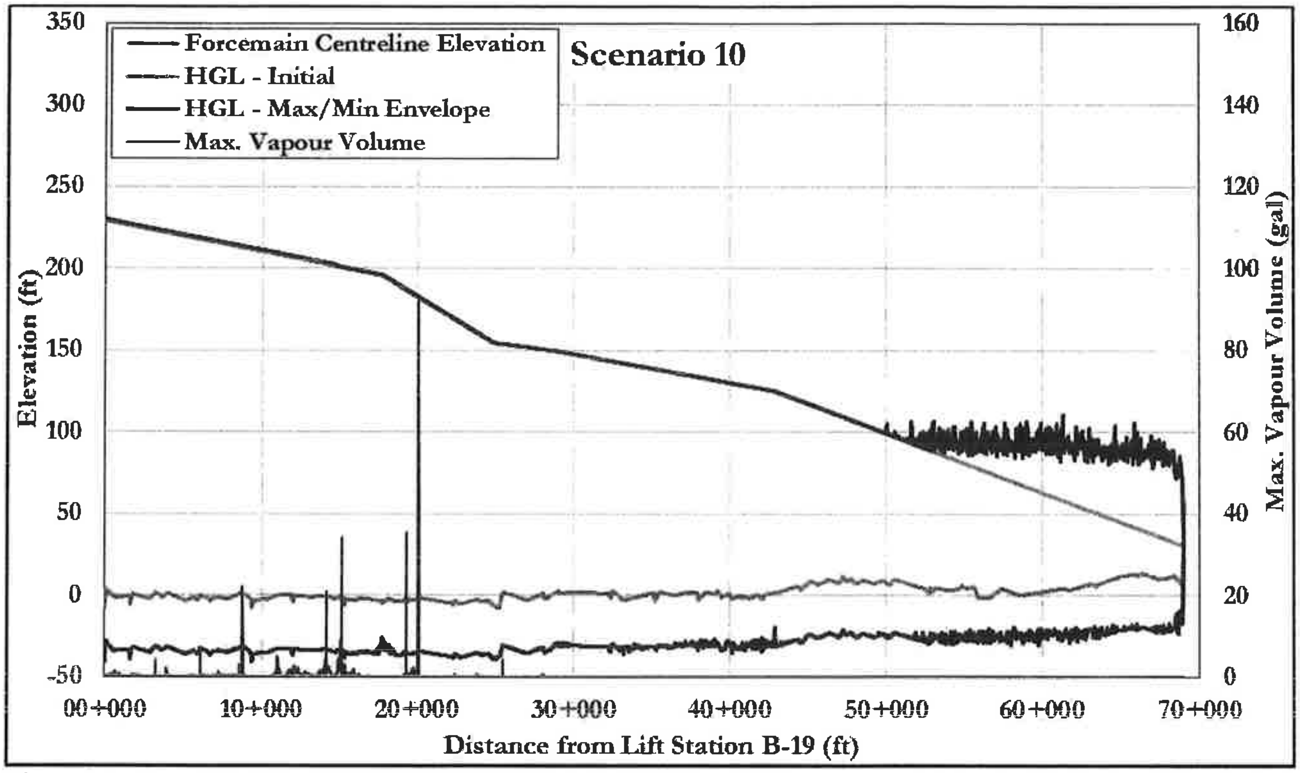


Figure 22: Scenario 10 – Proposed upgrades, full capacity, LWL, east-west forcemain interconnection at Riverside Drive, global power failure

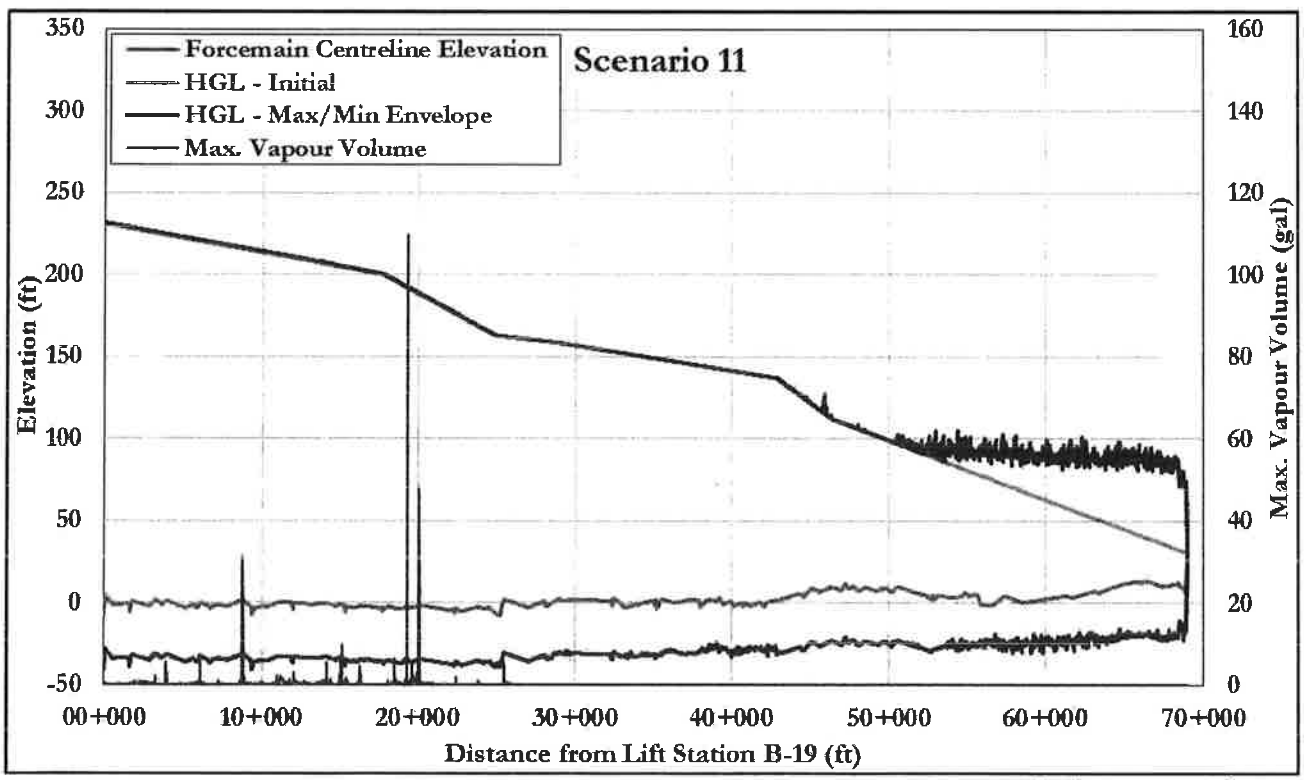


Figure 23: Scenario 11 – Proposed upgrades, full capacity, LWL, east-west forcemain interconnection at Oakland Avenue, global power failure

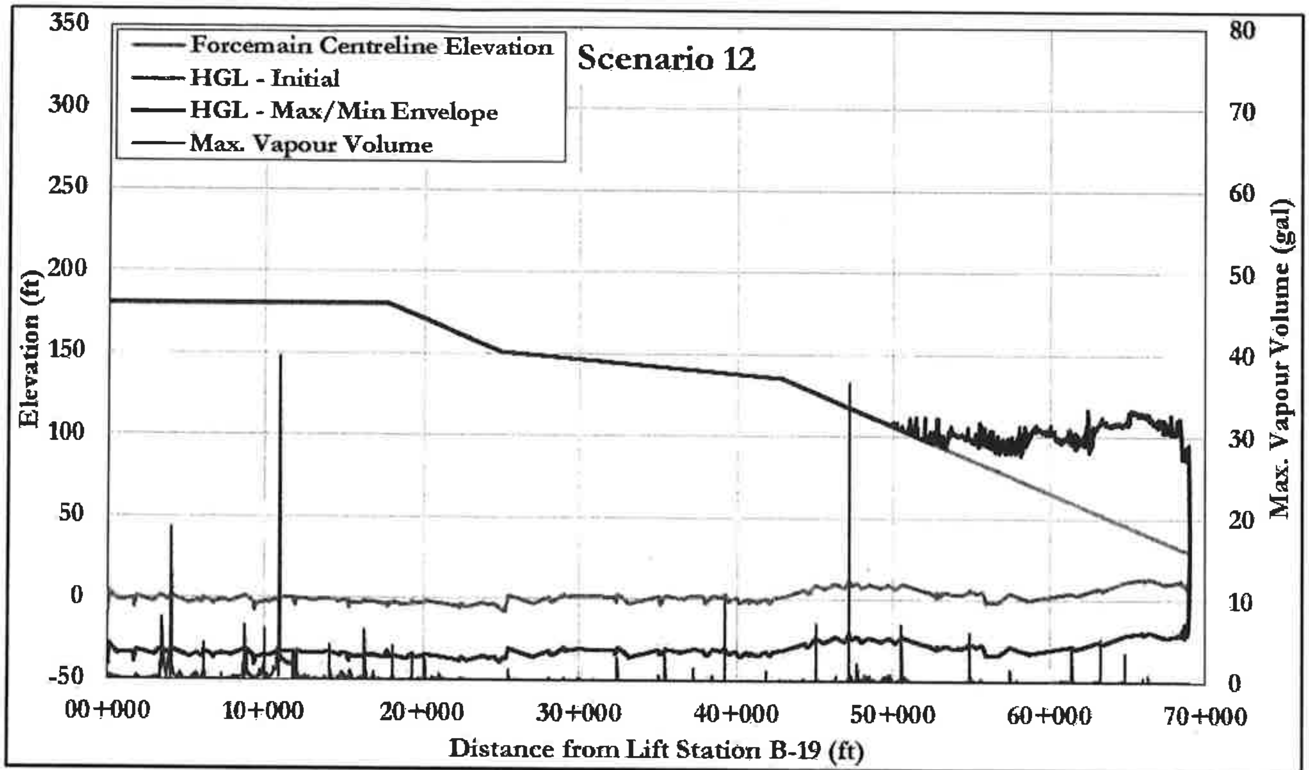


Figure 24: Scenario 12 – Scenario 9A (proposed upgrades, full capacity at B-20 only, LWL, global power failure) with +10% wave speed

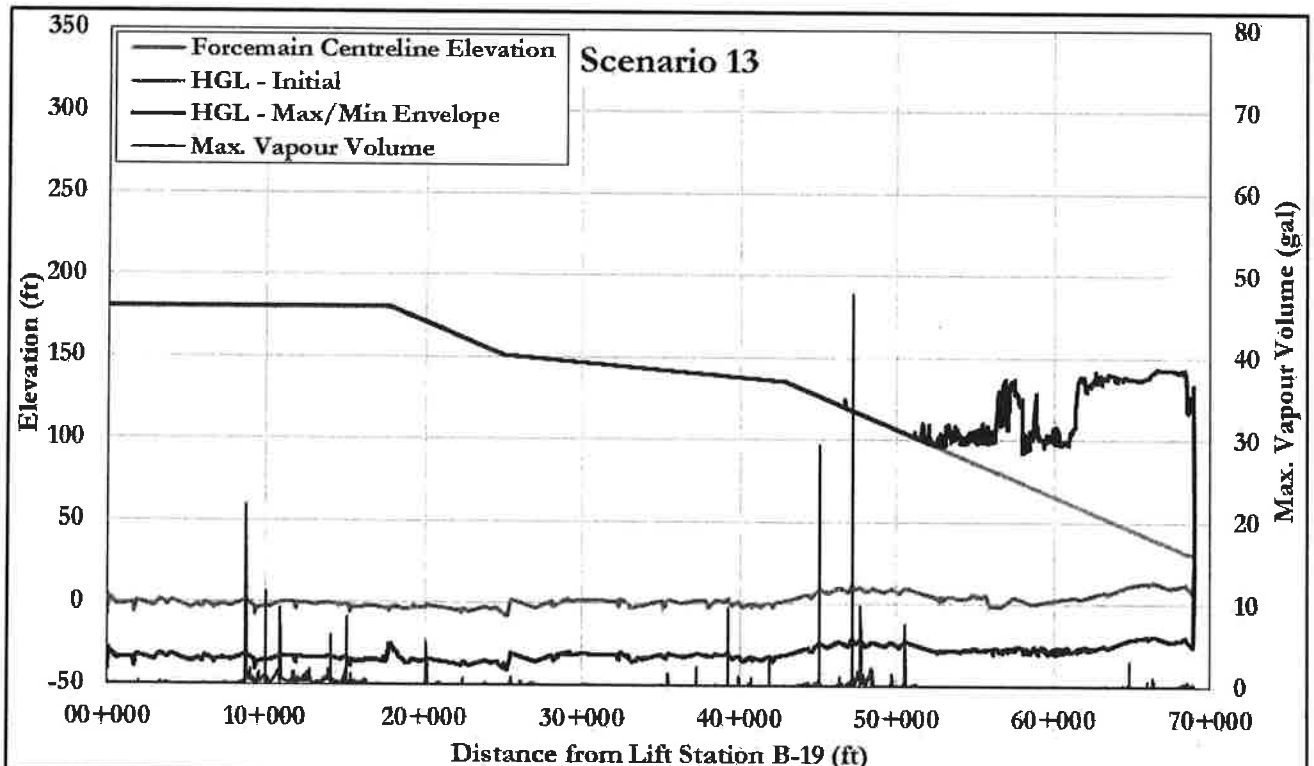


Figure 25: Scenario 13 – Scenario 9A (proposed upgrades, full capacity at B-20 only, LWL, global power failure) with -10% wave speed

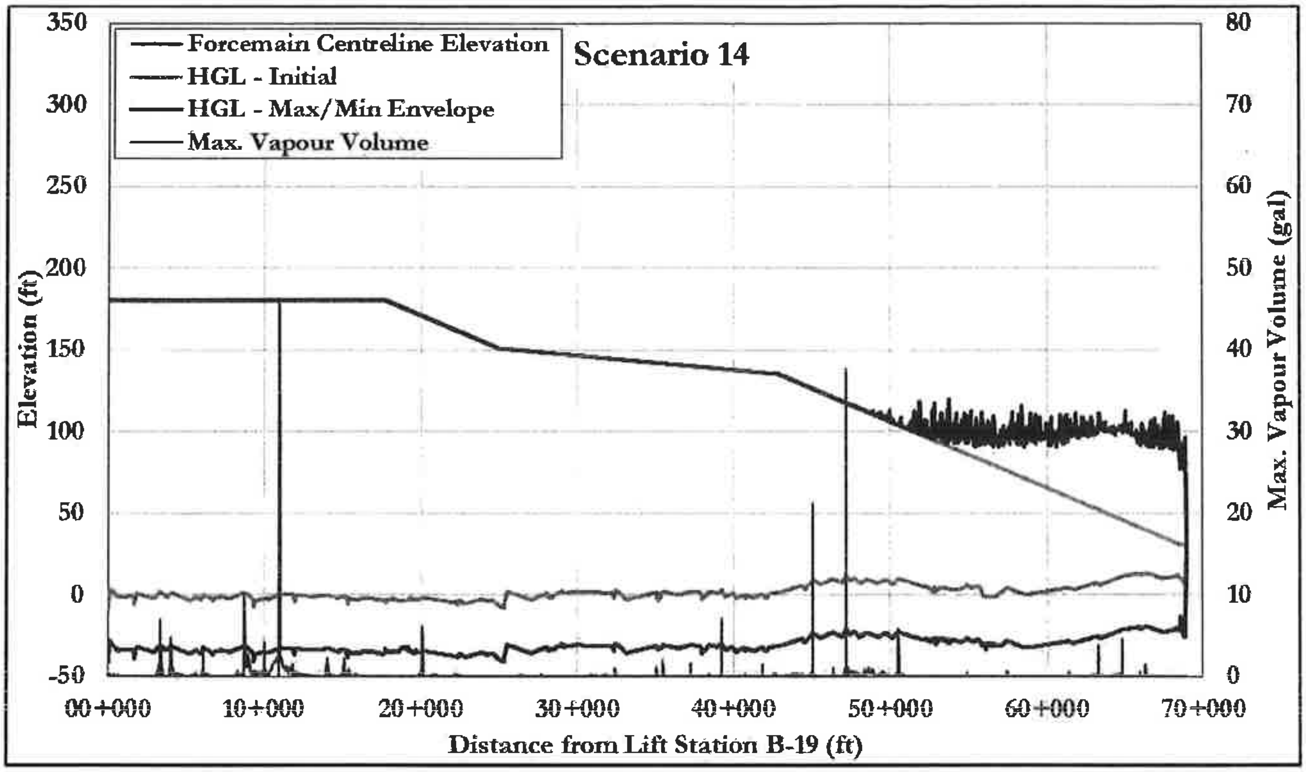


Figure 26: Scenario 14 – Scenario 9A (proposed upgrades, full capacity at B-20 only, LWL, global power failure) with 50% pump inertia

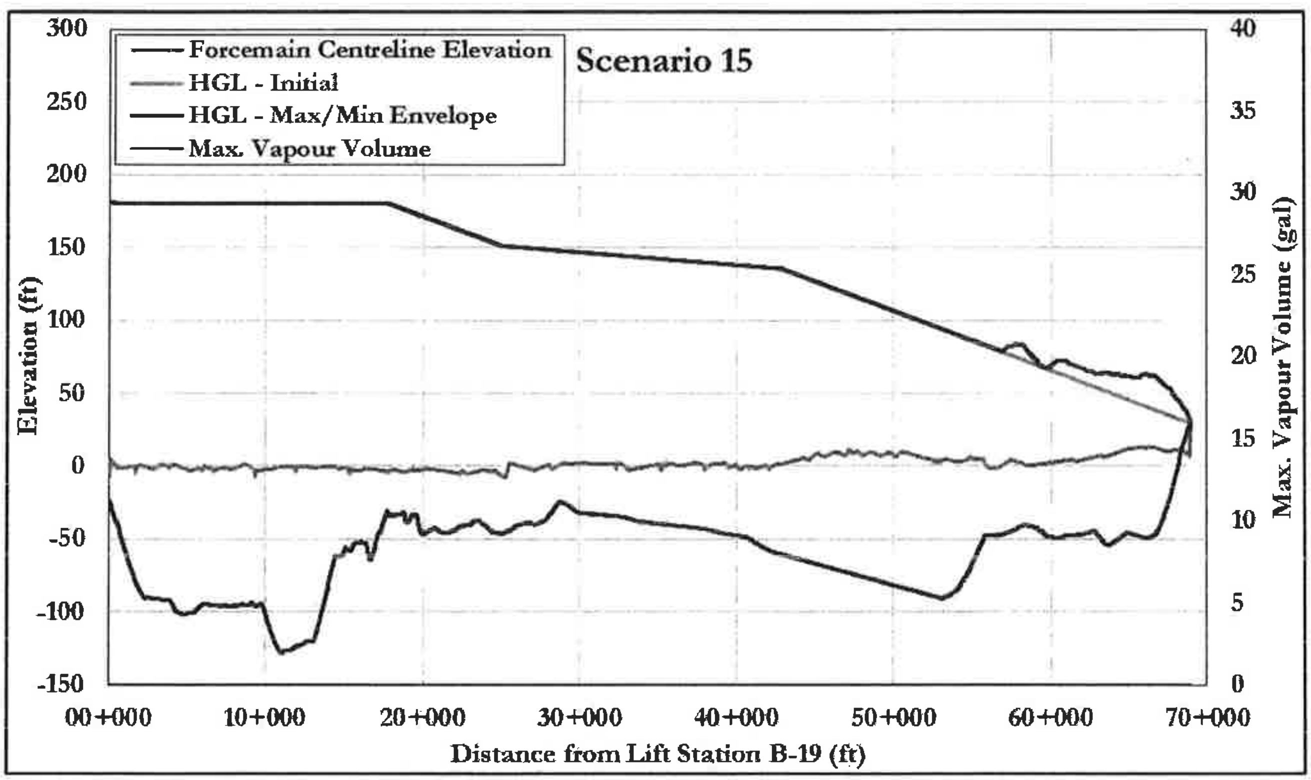


Figure 27: Scenario 15 – Scenario 9A (proposed upgrades, full capacity at B-20 only, LWL, global power failure) with no vapor cavities

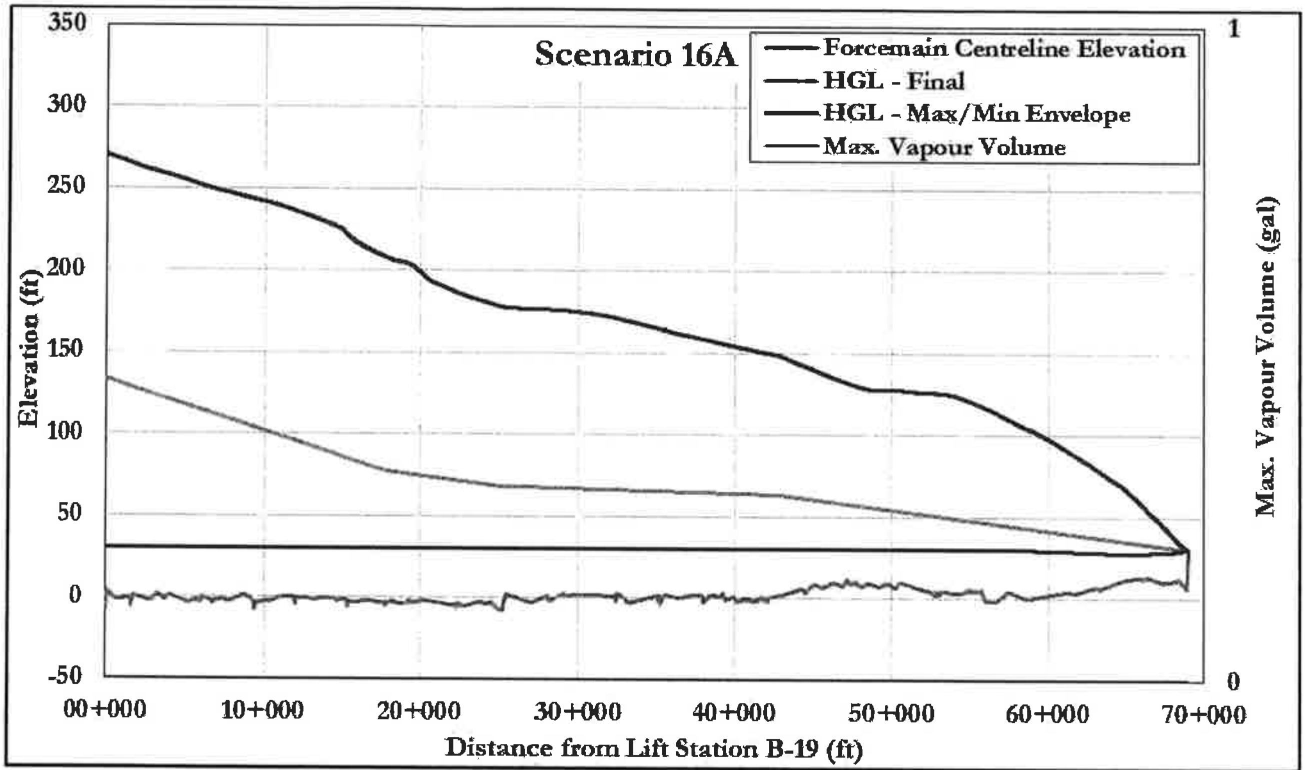


Figure 28: Scenario 16A – Proposed upgrades, all pumps off, HWL, single pump start at B-19

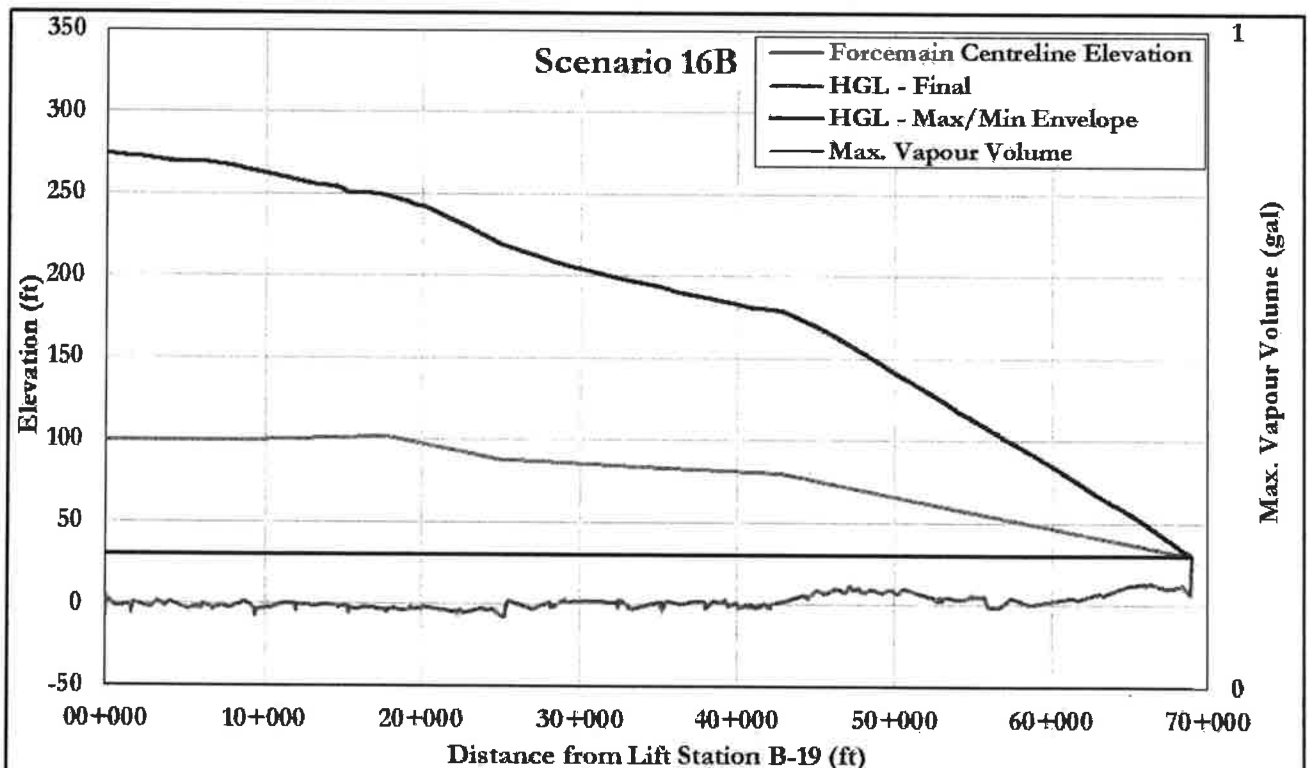


Figure 29: Scenario 16B – Proposed upgrades, all pumps off, HWL, single pump start at B-20

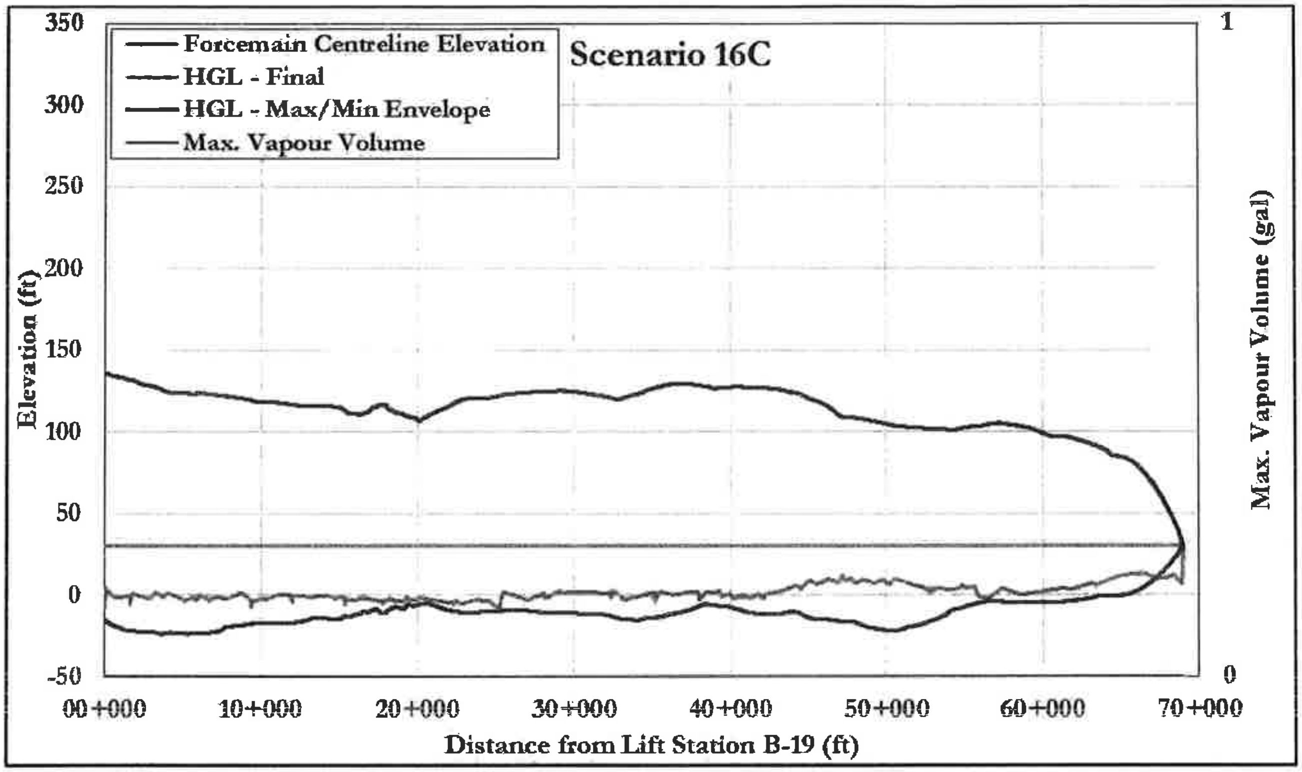


Figure 30: Scenario 16C – Proposed upgrades, all pumps off, HWL, single pump start at B-1

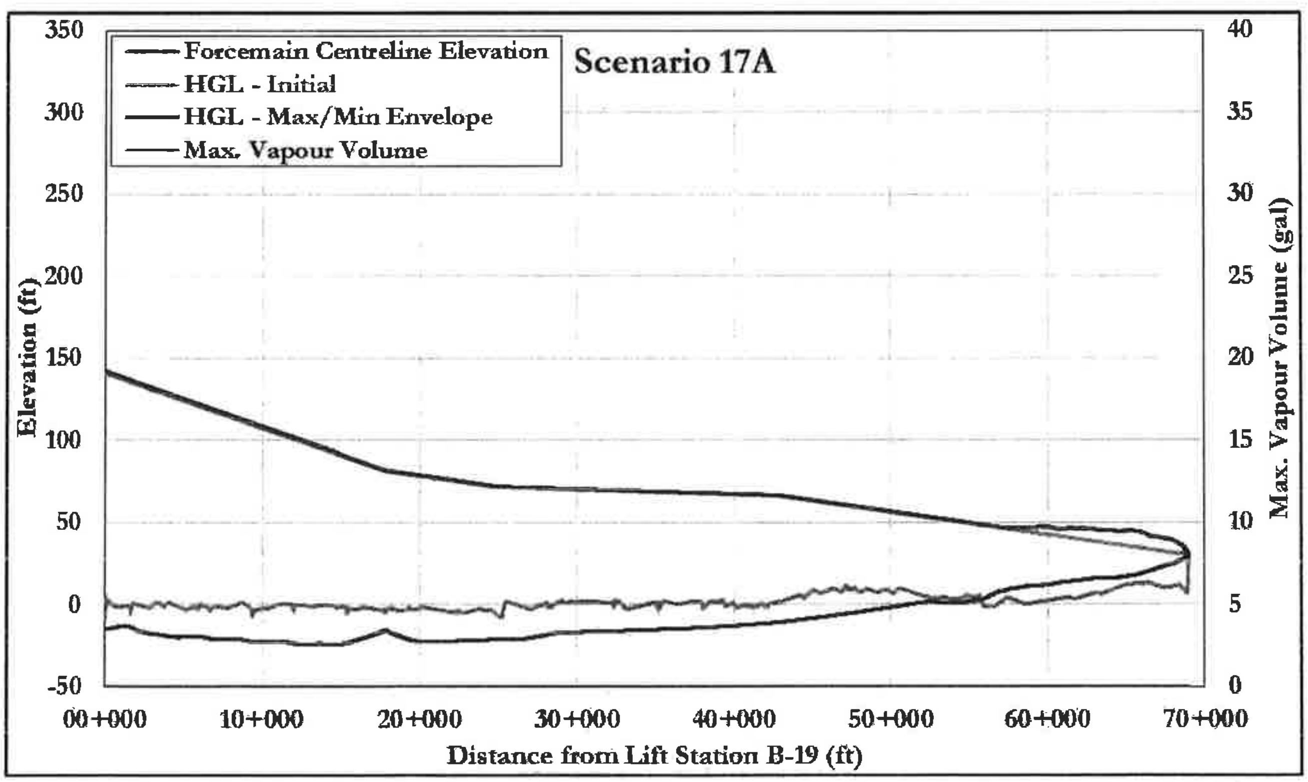


Figure 31: Scenario 17A – Proposed upgrades, one B-19 pump on, HWL, single pump stop at B-19

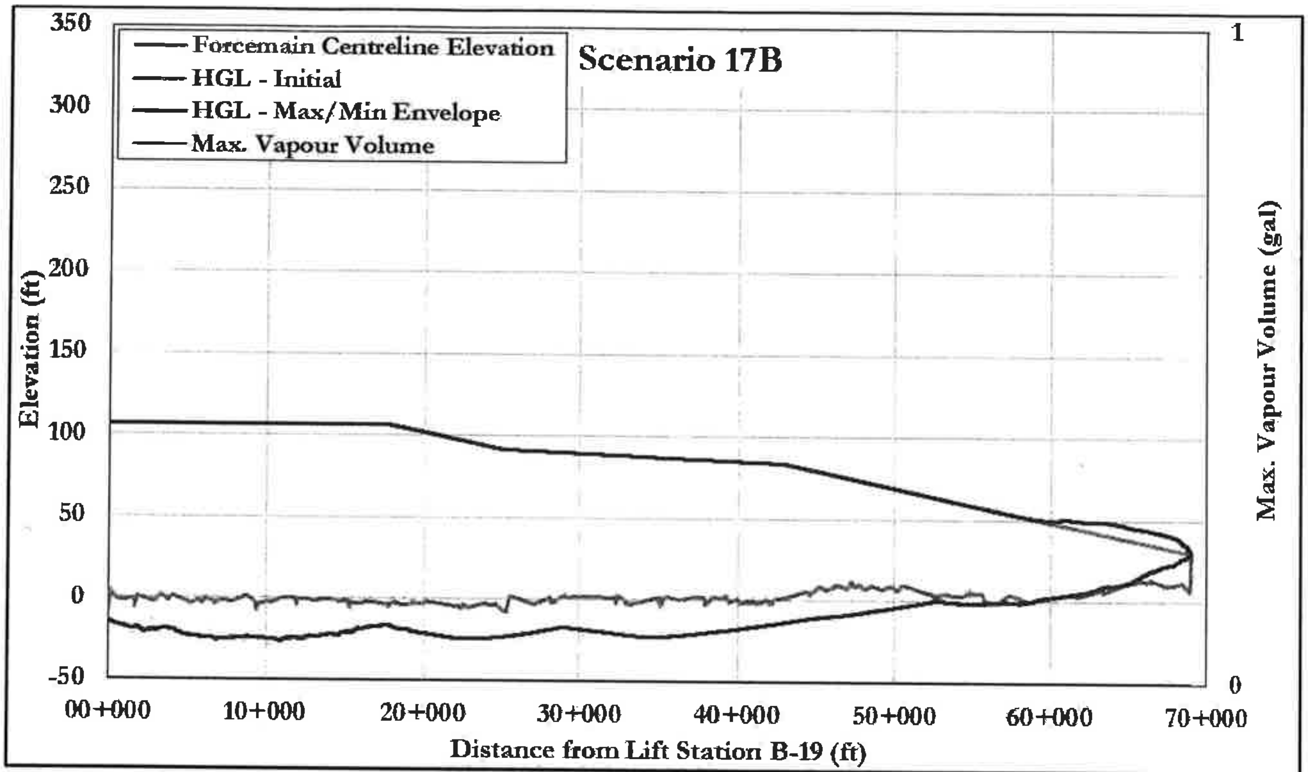


Figure 32: Scenario 17B – Proposed upgrades, one B-20 pump on, HWL, single pump stop at B-20

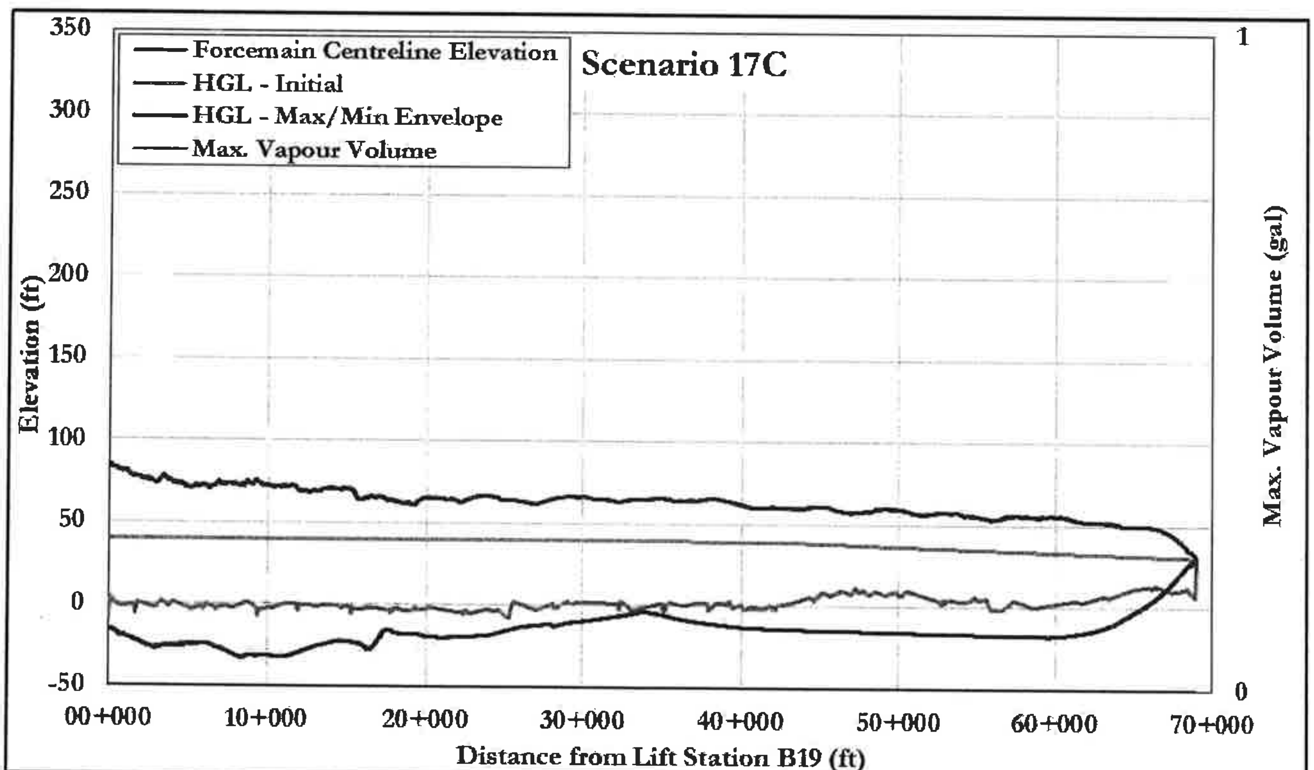


Figure 33: Scenario 17C – Proposed upgrades, one B-1 pump on, HWL, single pump stop at B-1

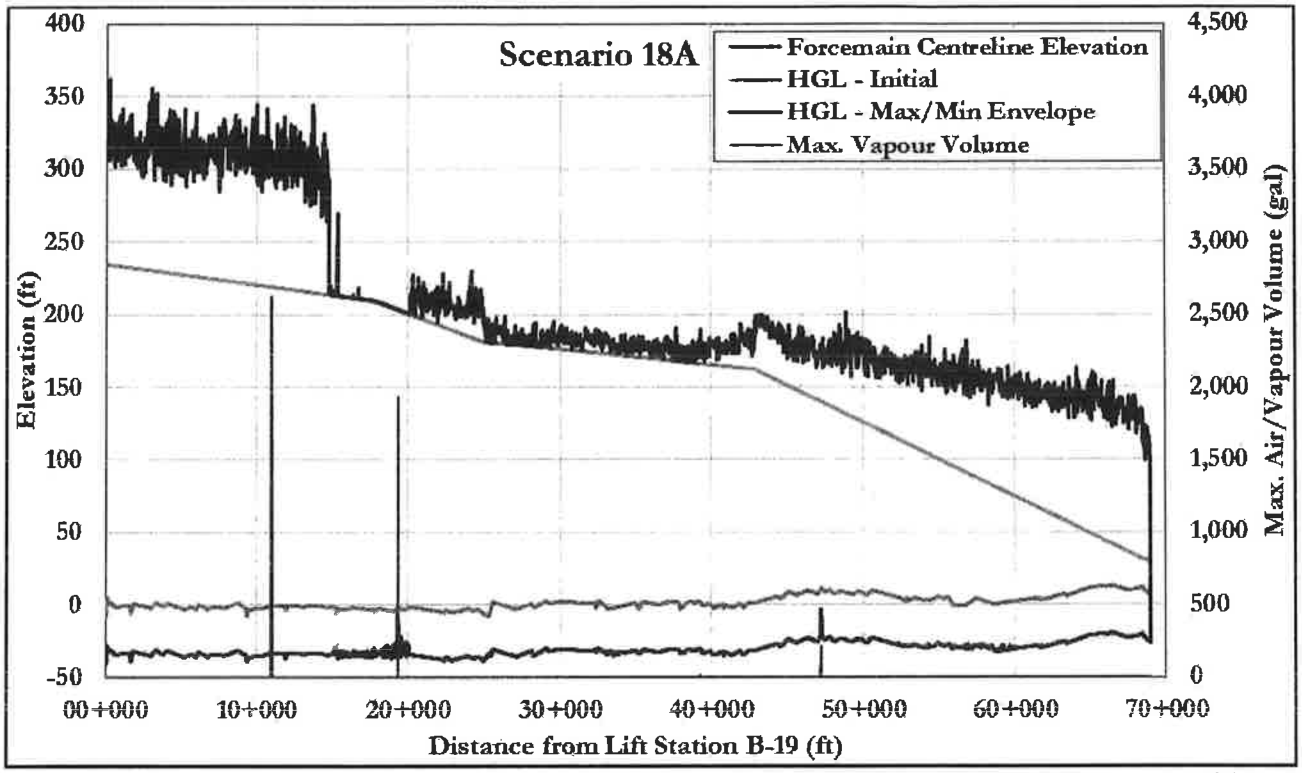


Figure 34: Scenario 18A – Scenario 7A (proposed upgrades, full capacity, LWL, global power failure) with 2-inch standard CAVs at select locations

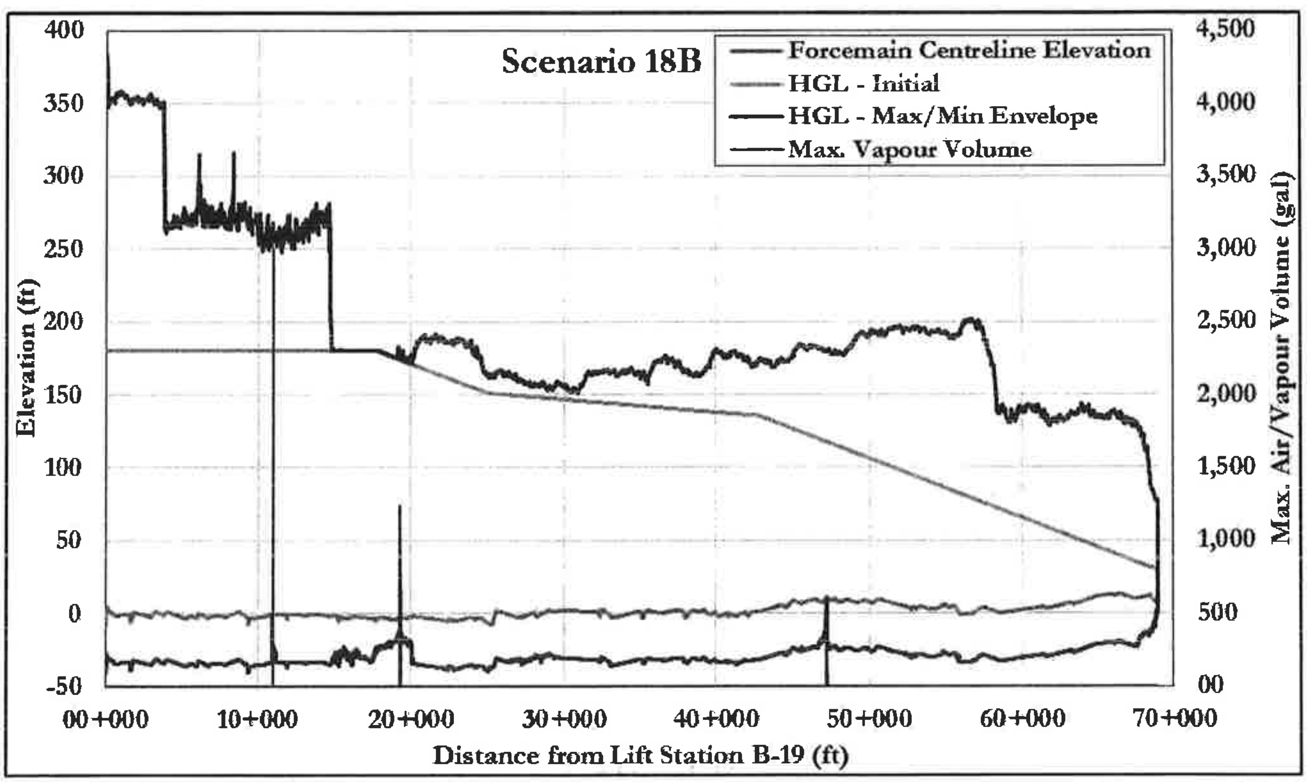


Figure 35: Scenario 18B – Scenario 9A (Proposed upgrades, full capacity at B-20 only, LWL, global power failure) with 2-inch standard CAVs at select locations

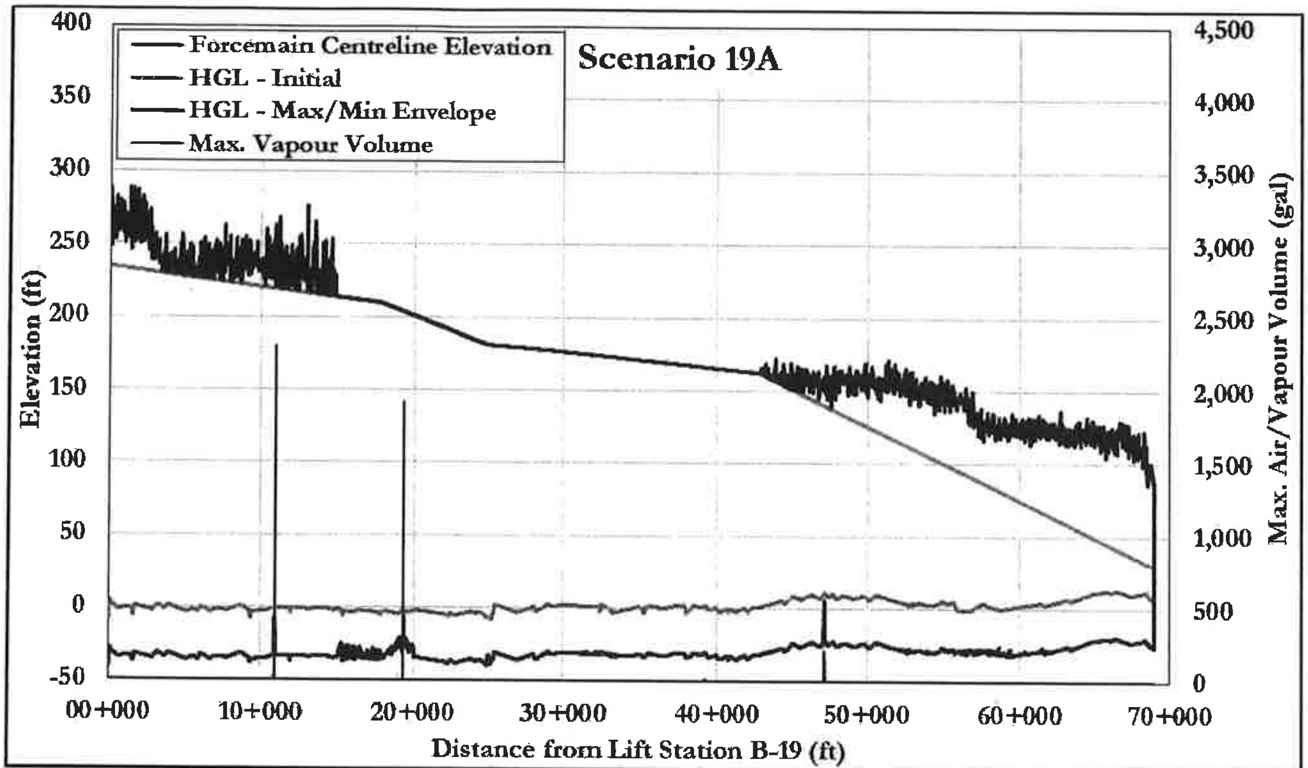


Figure 36: Scenario 19A – Scenario 7A (proposed upgrades, full capacity, LWL, global power failure) with 2-inch three-stage CAVs at select locations

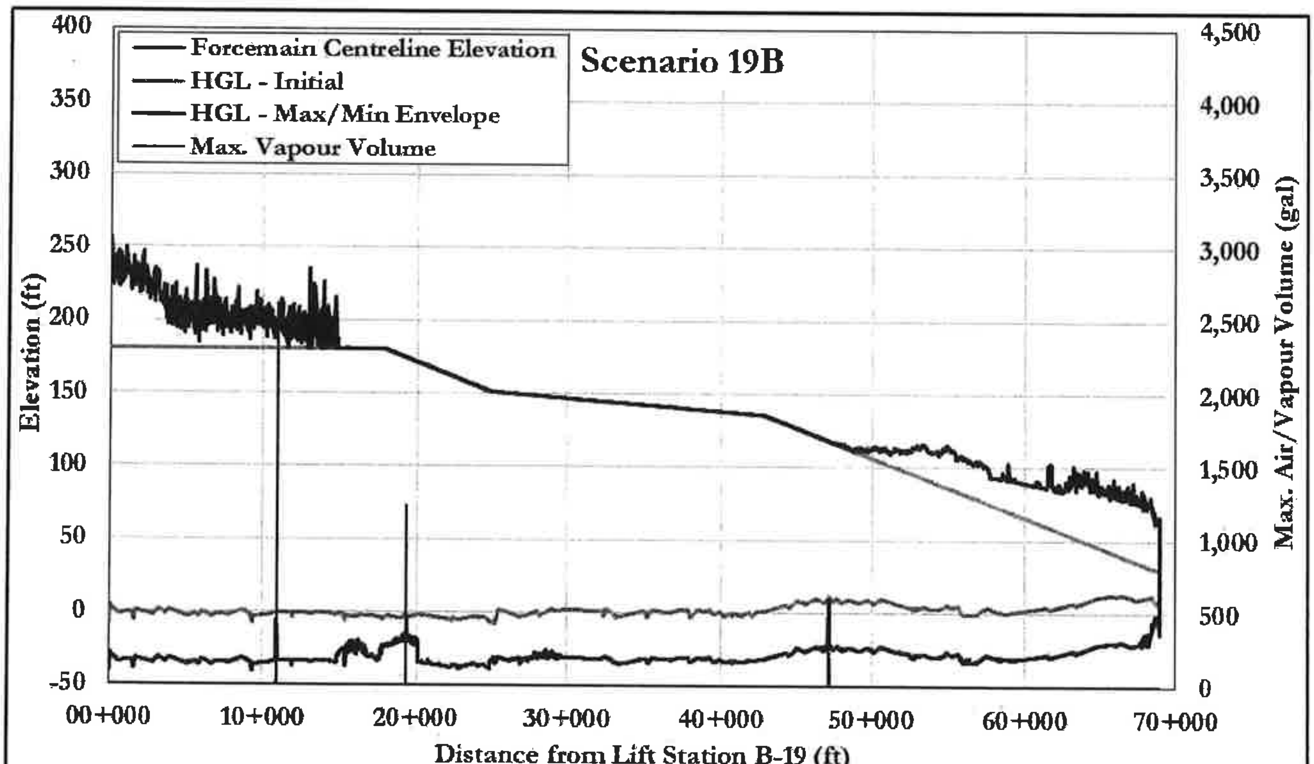


Figure 37: Scenario 19B – Scenario 9A (Proposed upgrades, full capacity at B-20 only, LWL, global power failure) with 2-inch three-stage CAVs at select locations

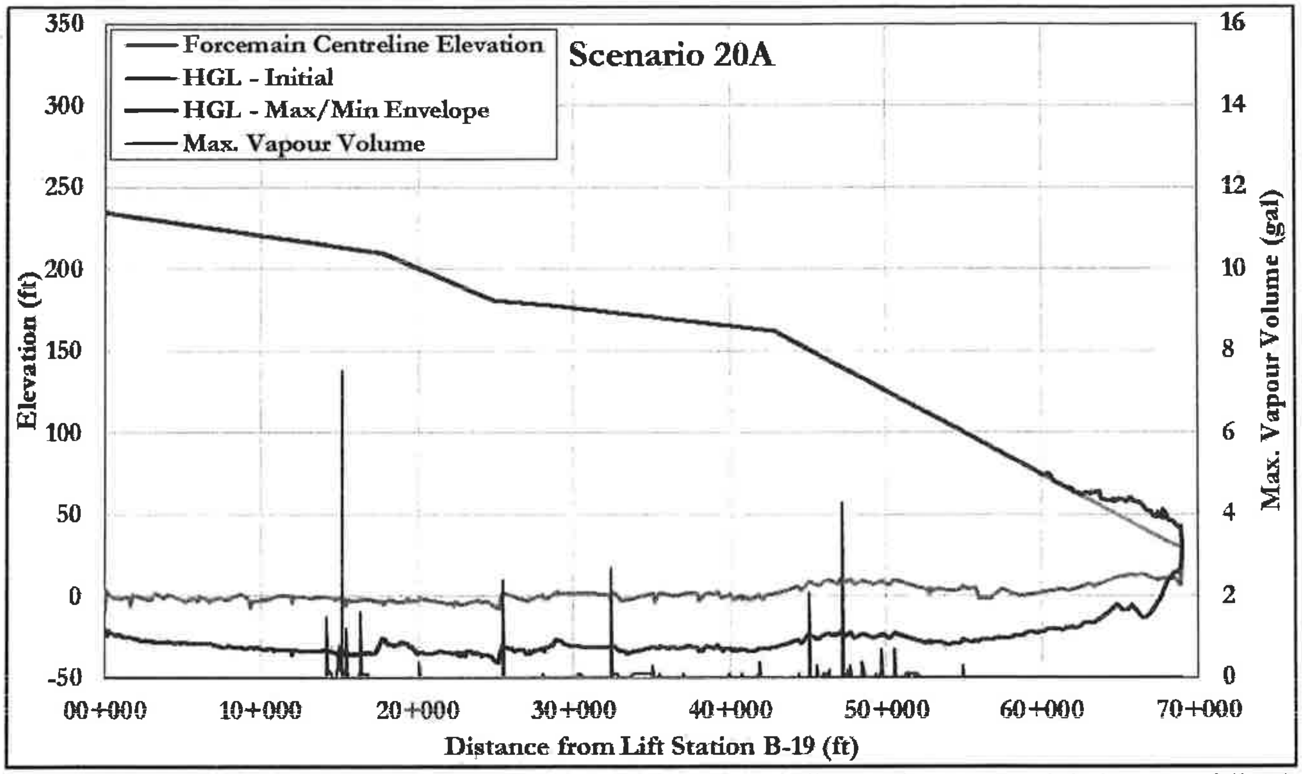


Figure 38: Scenario 20A – Scenario 7A (proposed upgrades, full capacity, LWL, global power failure) with a 4,000 gal HAC at Lift Station B-19

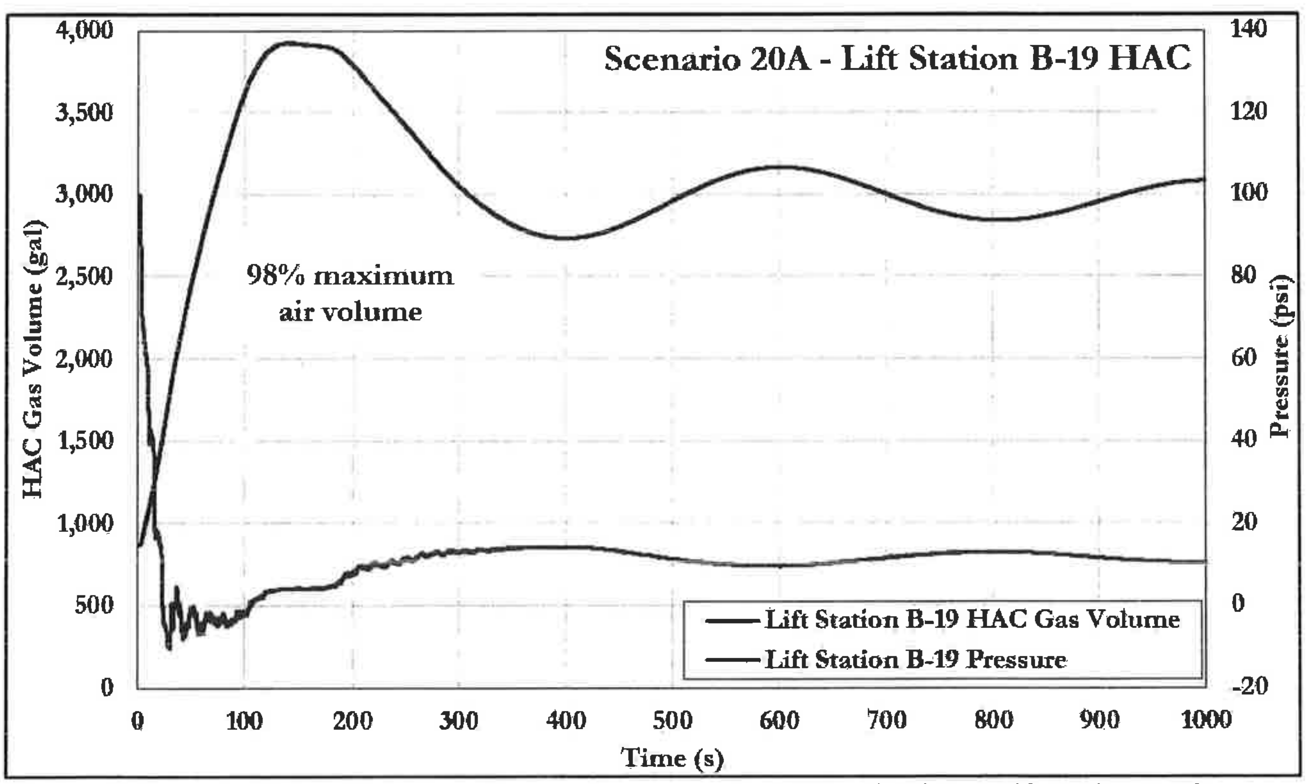


Figure 39: Scenario 20A – HAC gas volume and pressure for 4,000 gal HAC at Lift Station B-19

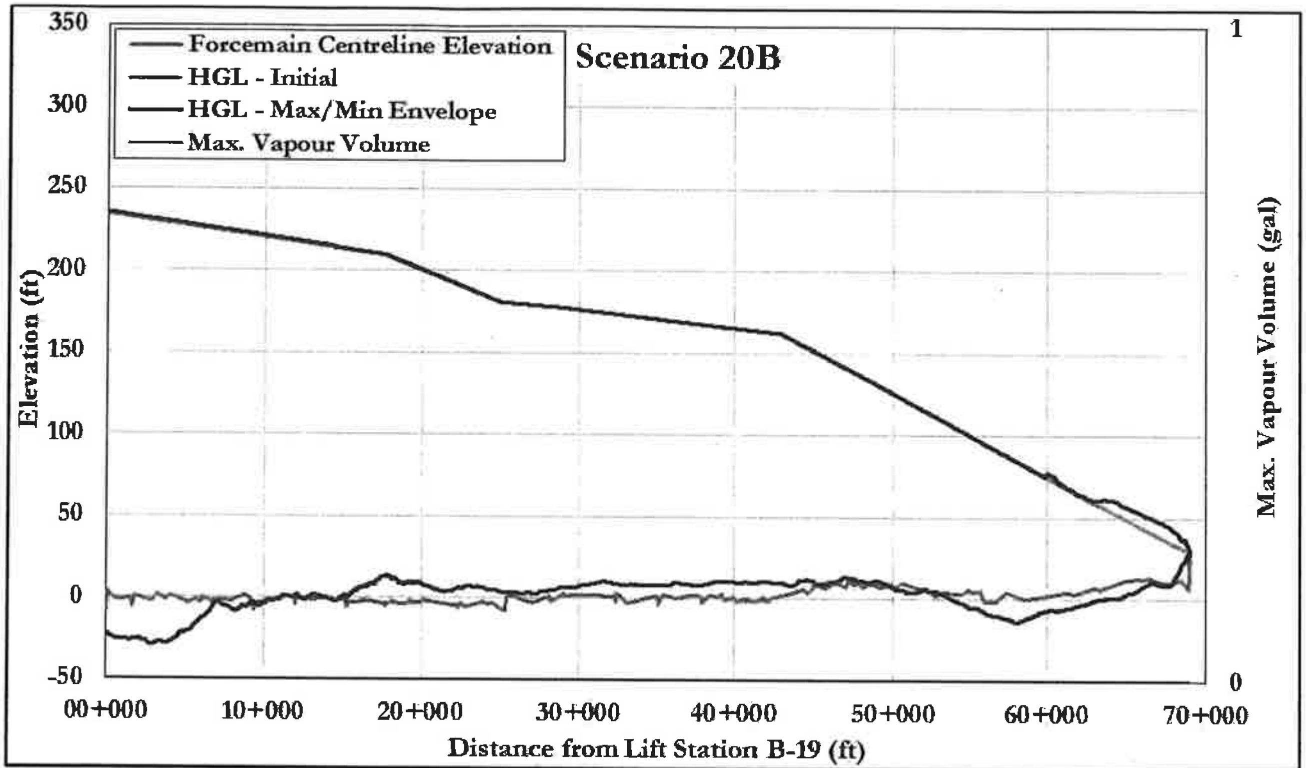


Figure 40: Scenario 20B – Scenario 7A (proposed upgrades, full capacity, LWL, global power failure) with a 10,000 gal HAC at Lift Station B-20

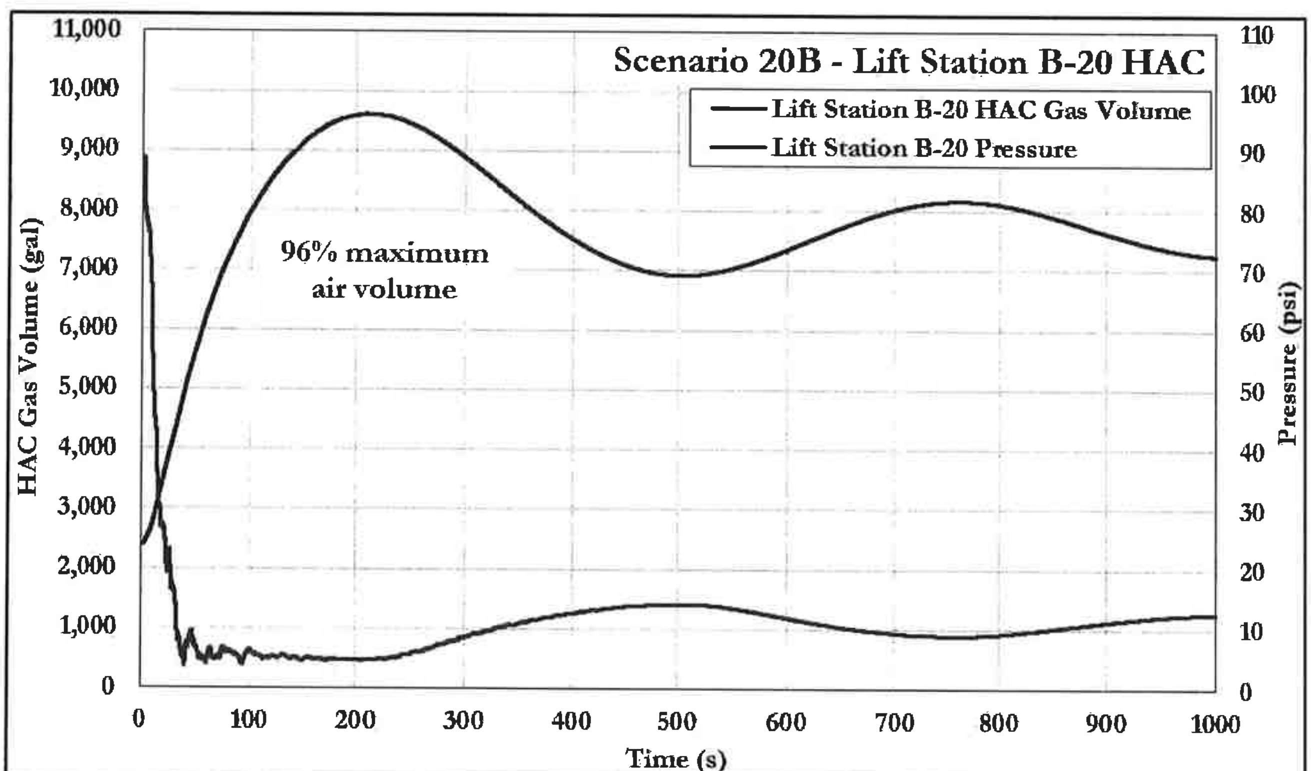


Figure 41: Scenario 20B – HAC gas volume and pressure for 10,000 gal HAC at Lift Station B-20

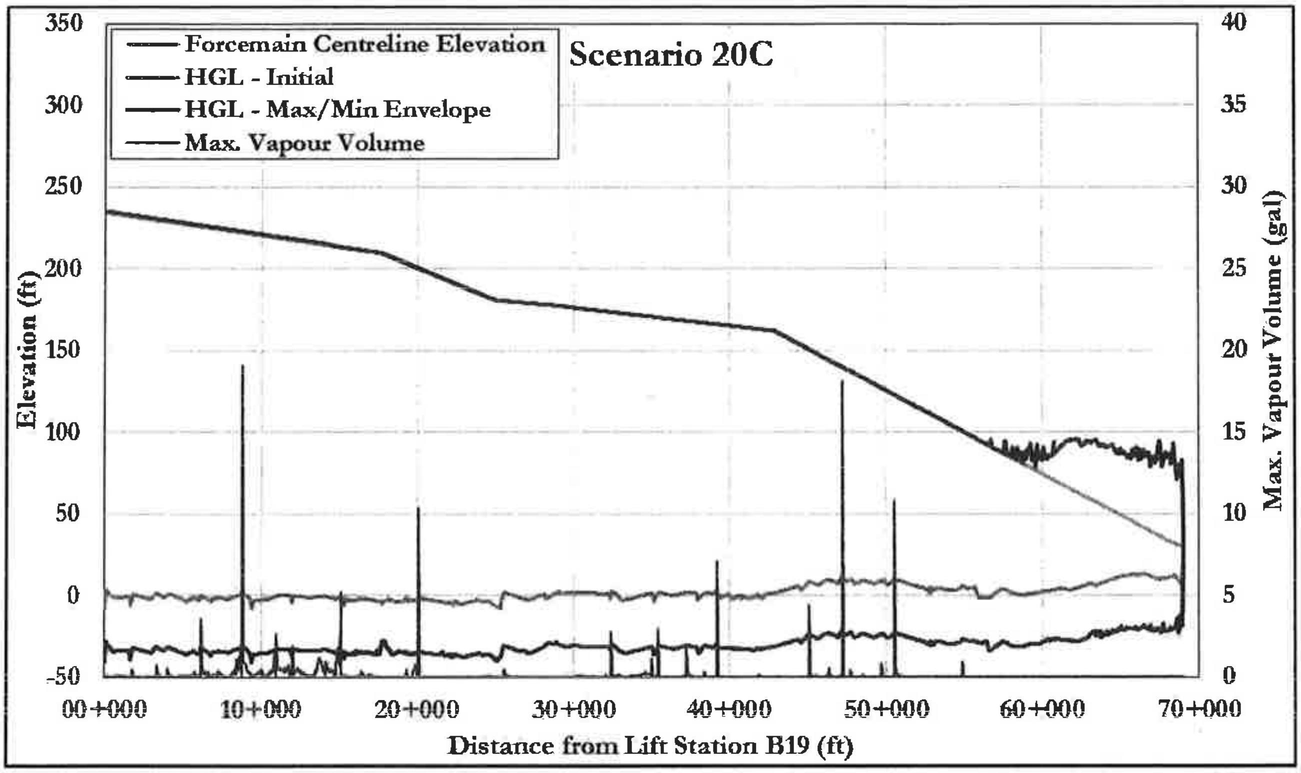


Figure 42: Scenario 20C – Scenario 7A (proposed upgrades, full capacity, LWL, global power failure) with a 4,000 gal HAC at Lift Station B-1

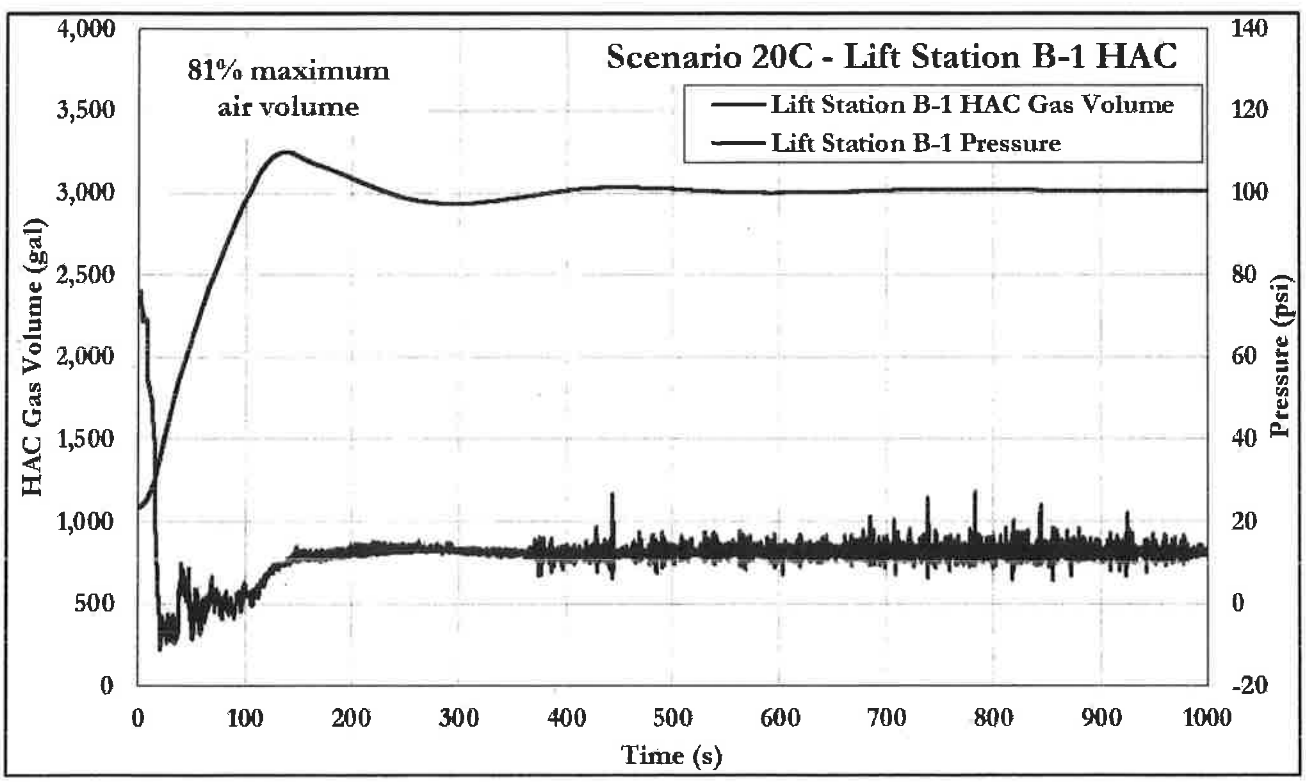


Figure 43: Scenario 20C – HAC gas volume and pressure for 4,000 gal HAC at Lift Station B-1

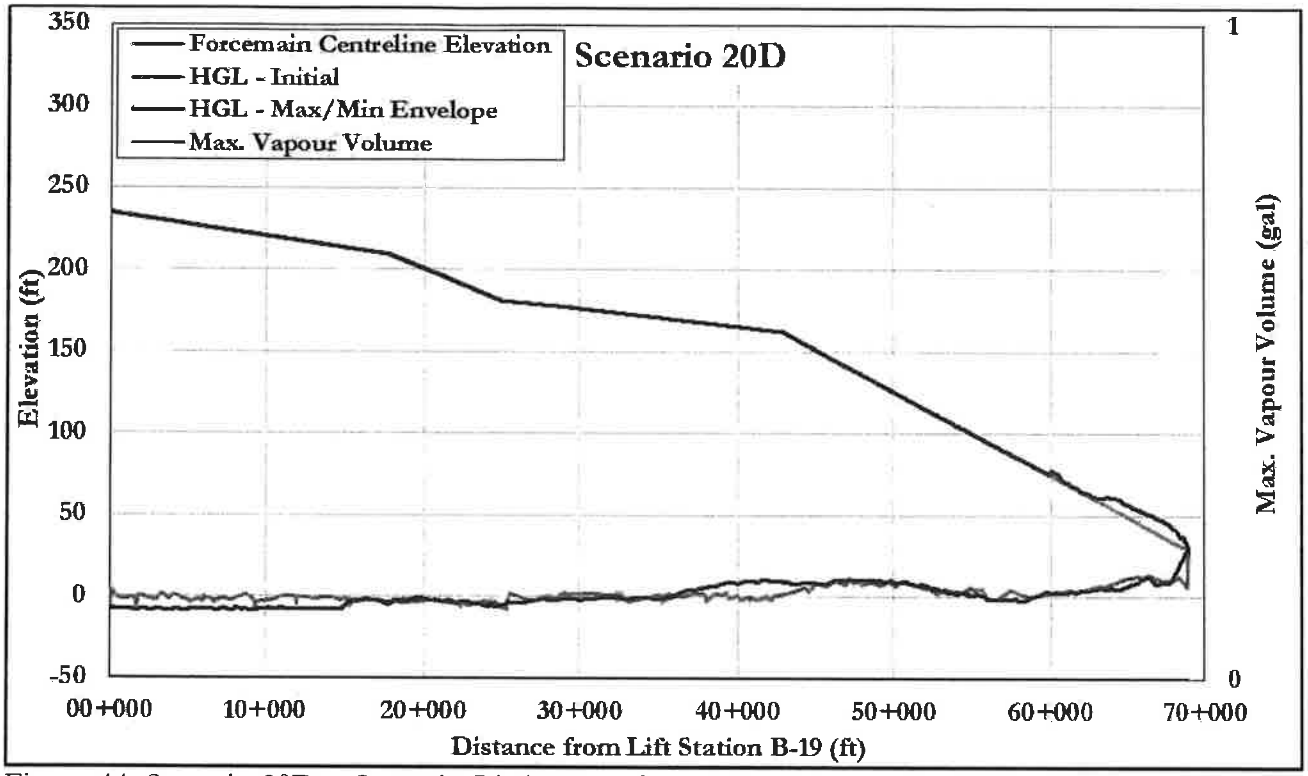


Figure 44: Scenario 20D – Scenario 7A (proposed upgrades, full capacity, LWL, global power failure) with a 3,000 gal HAC at Lift Station B-19 and a 6,000 gal HAC at Lift Station B-20

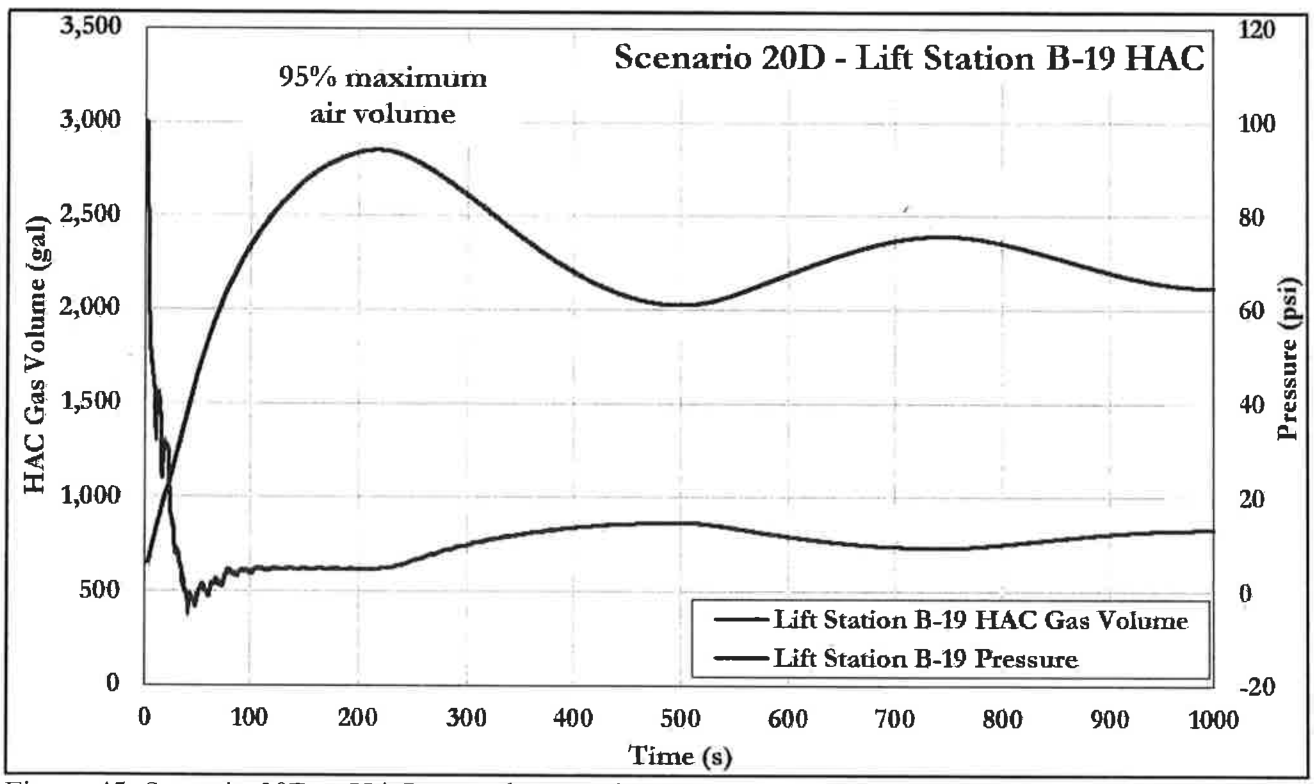


Figure 45: Scenario 20D – HAC gas volume and pressure for 3,000 gal HAC at Lift Station B-19 (with a 6,000 gal HAC at Lift Station B-20)

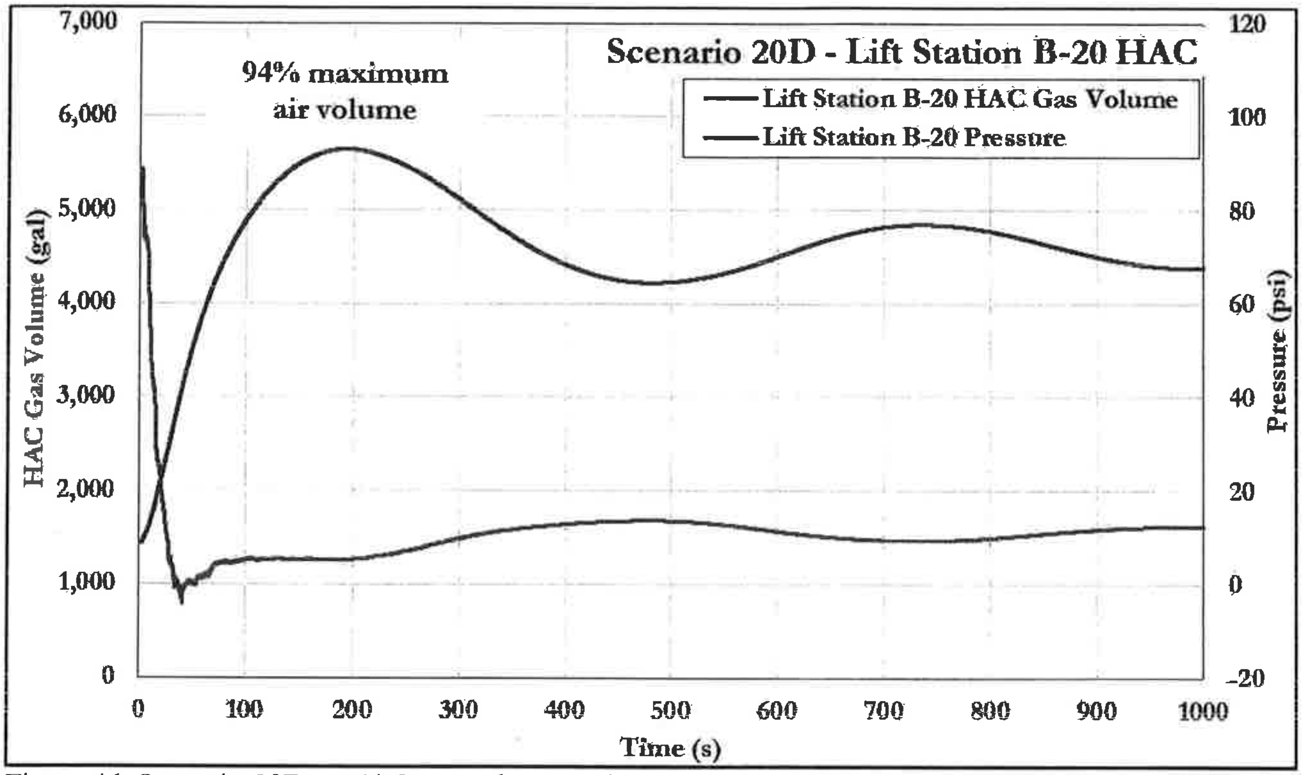


Figure 46: Scenario 20D – HAC gas volume and pressure for 6,000 gal HAC at Lift Station B-20 (with a 3,000 gal HAC at Lift Station B-19)

APPENDIX C: SAMPLE PLOTS OF TRANSIENT PRESSURE MONITORING DATA AT LIFT STATIONS B-19 AND B-1

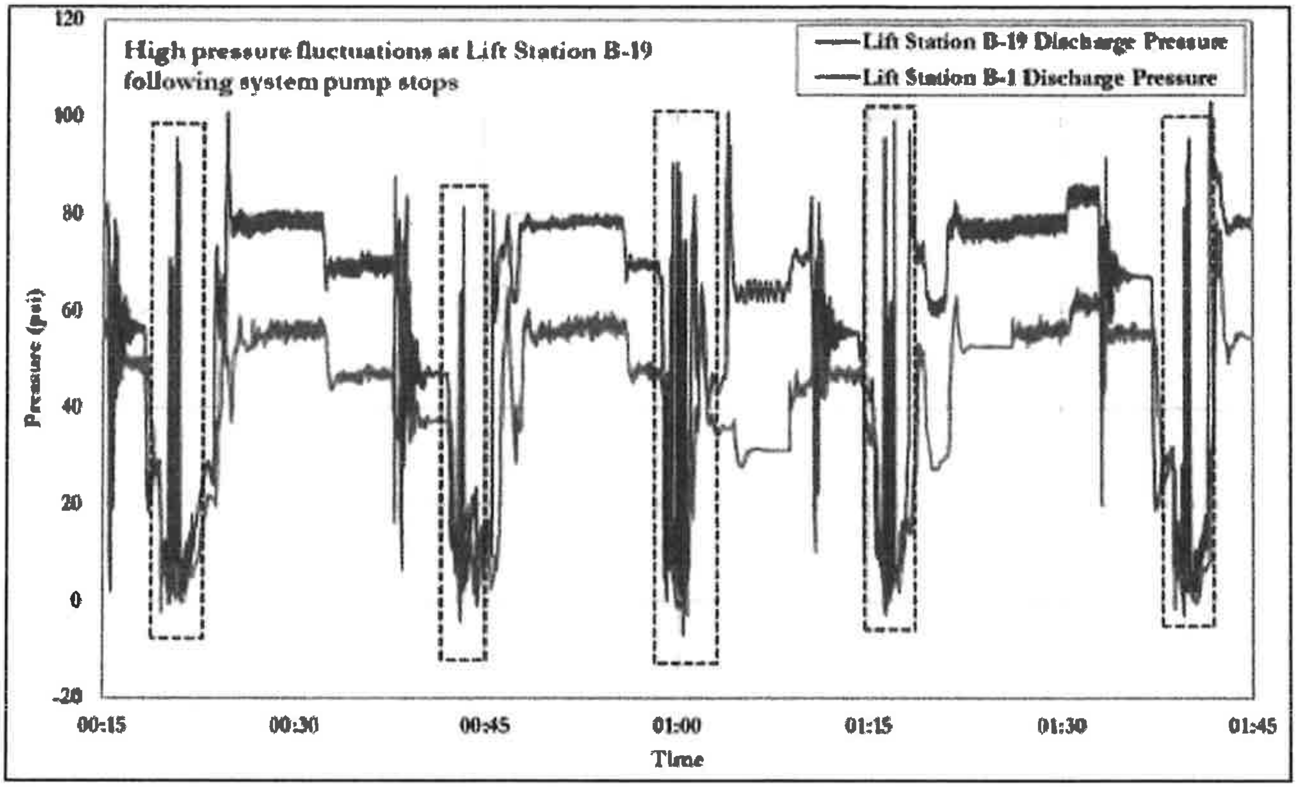


Figure 47: Transient pressure monitoring data from 12:15 AM to 1:45 AM on 30 September 2017 showing high pressure fluctuations at Lift Station B-19 following pump stop operations

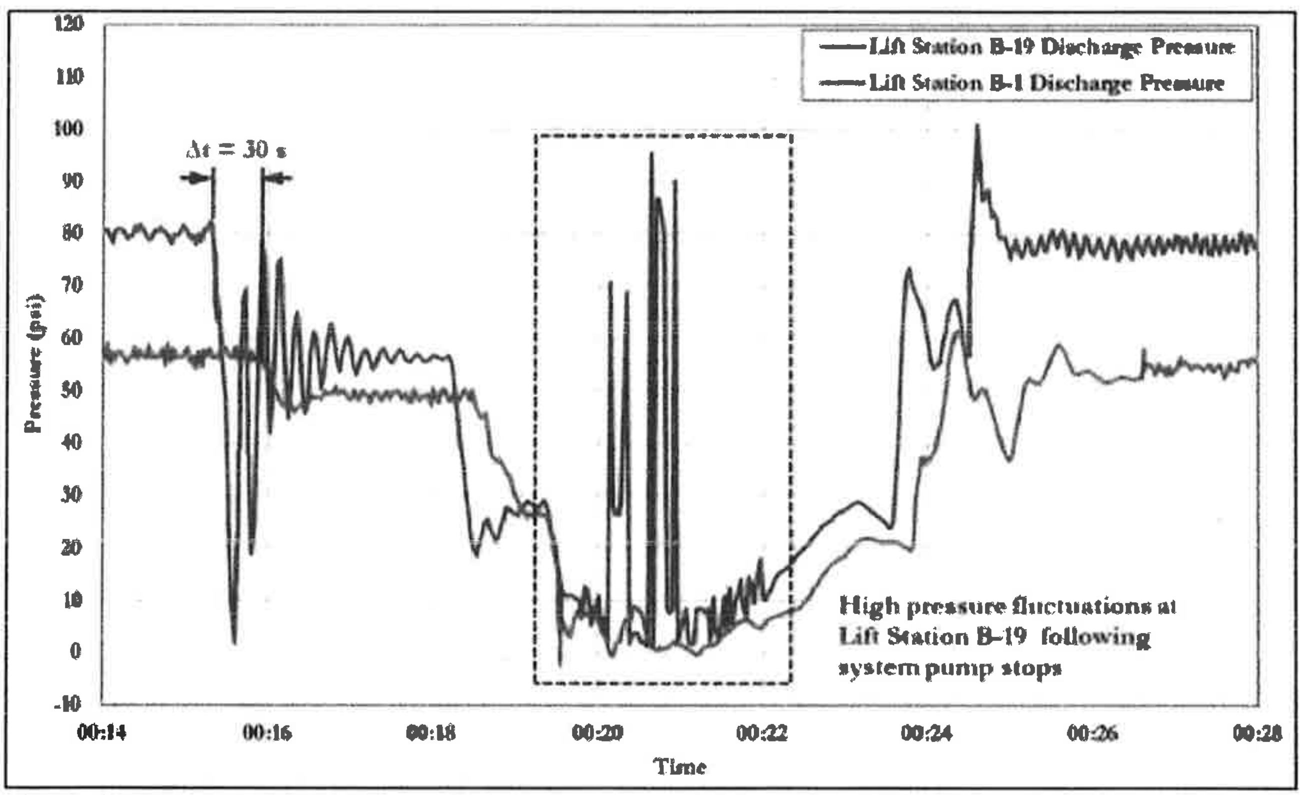


Figure 48: Transient pressure monitoring data from 12:14 AM to 12:28 AM on 30 September 2017 showing high pressure fluctuations at Lift Station B-19 following pump stop operations

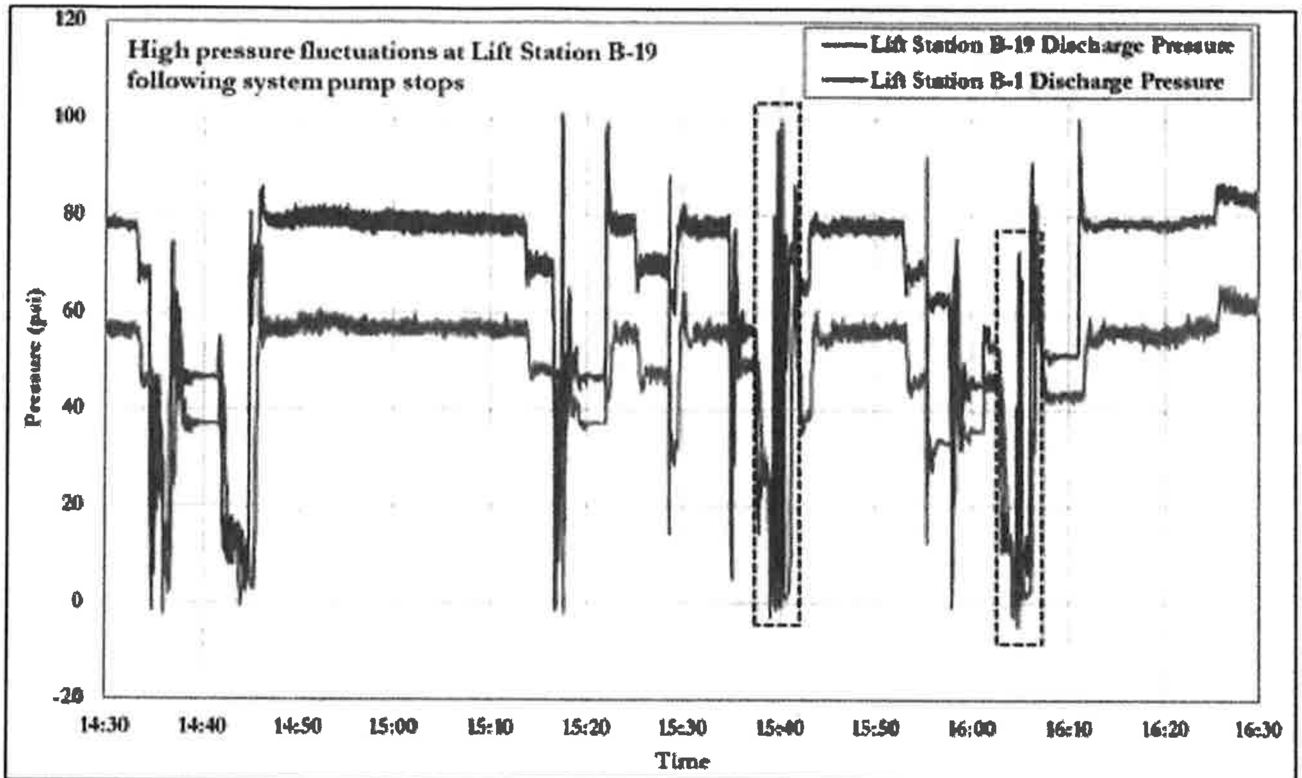


Figure 49: Transient pressure monitoring data from 2:30 PM to 4:30 PM on 30 September 2017 showing high pressure fluctuations at Lift Station B-19 following pump stop operations

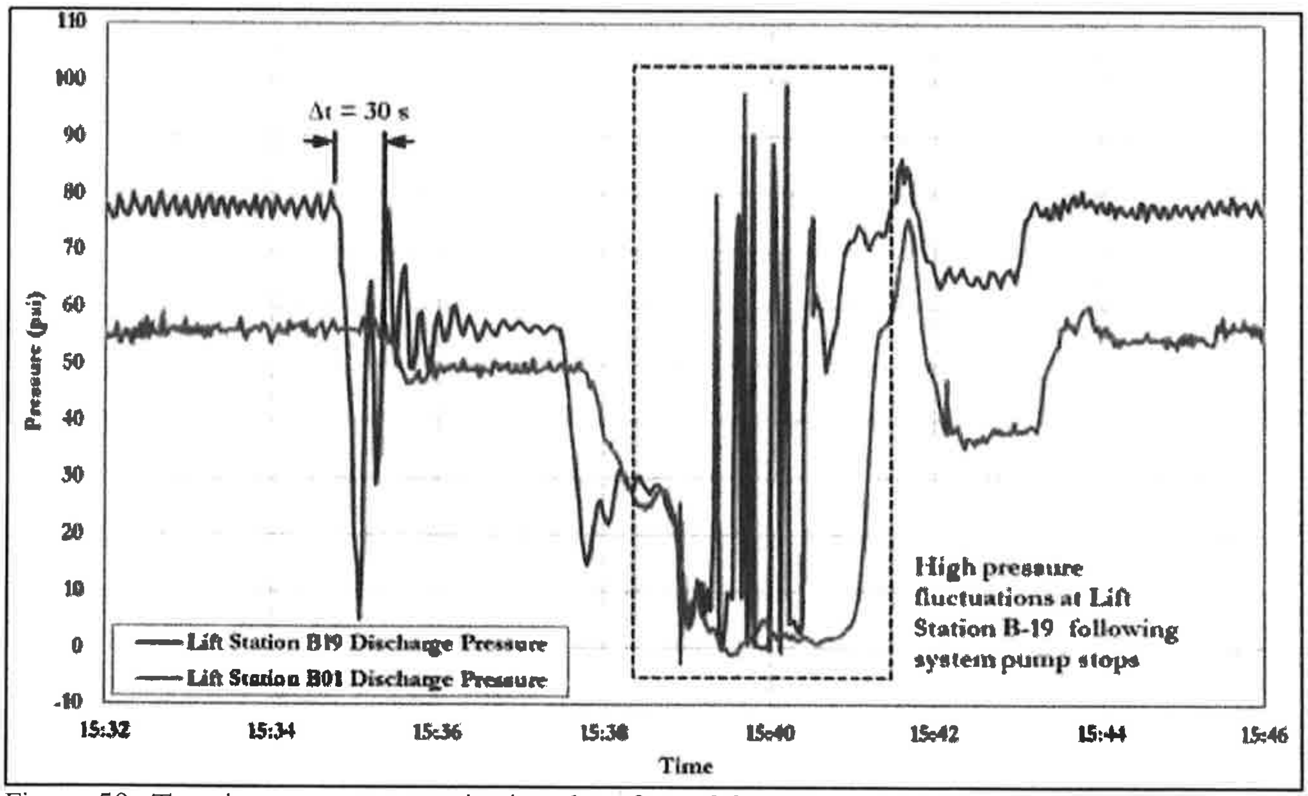


Figure 50: Transient pressure monitoring data from 3:32 PM to 3:46 PM on 30 September 2017 showing high pressure fluctuations at Lift Station B-19 following pump stop operations

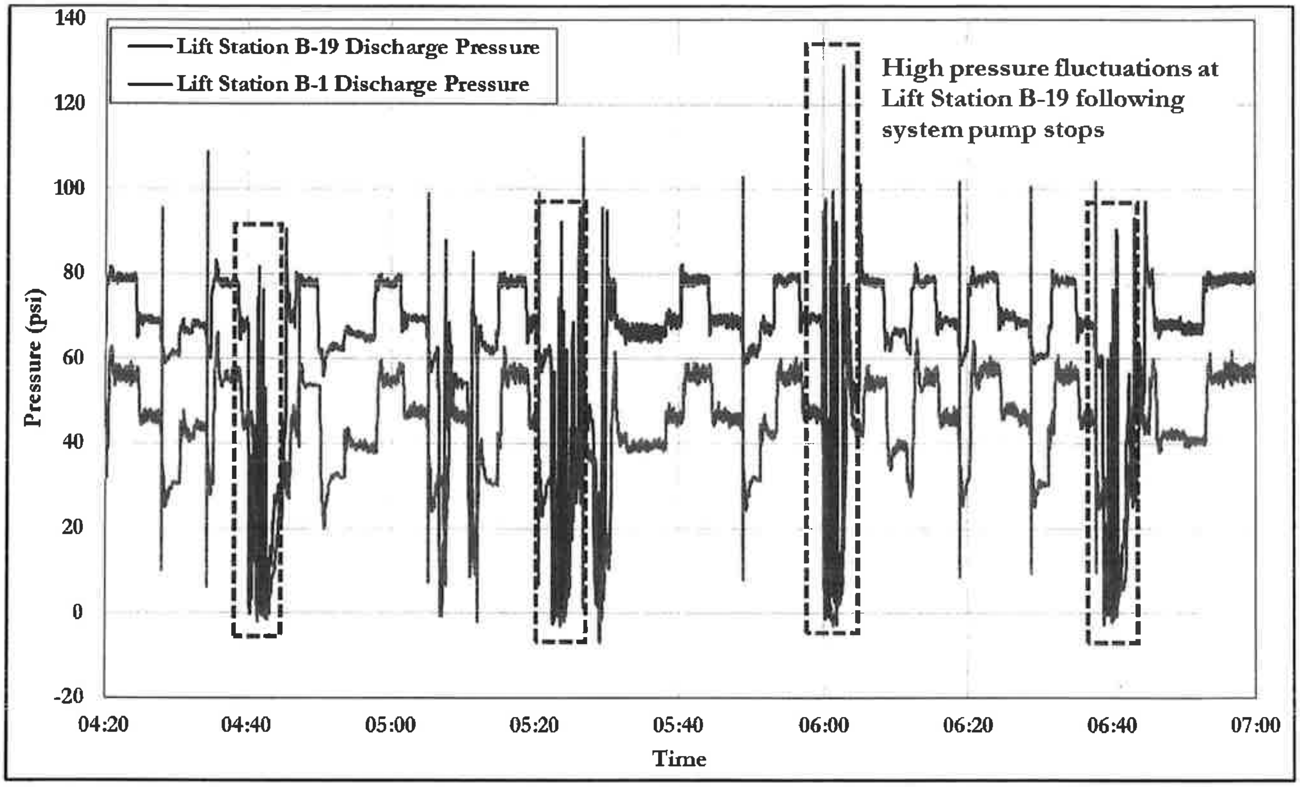


Figure 51: Transient pressure monitoring data from 4:20 AM to 7:00 AM on 05 October 2017 showing high pressure fluctuations at Lift Station B-19 following pump stop operations

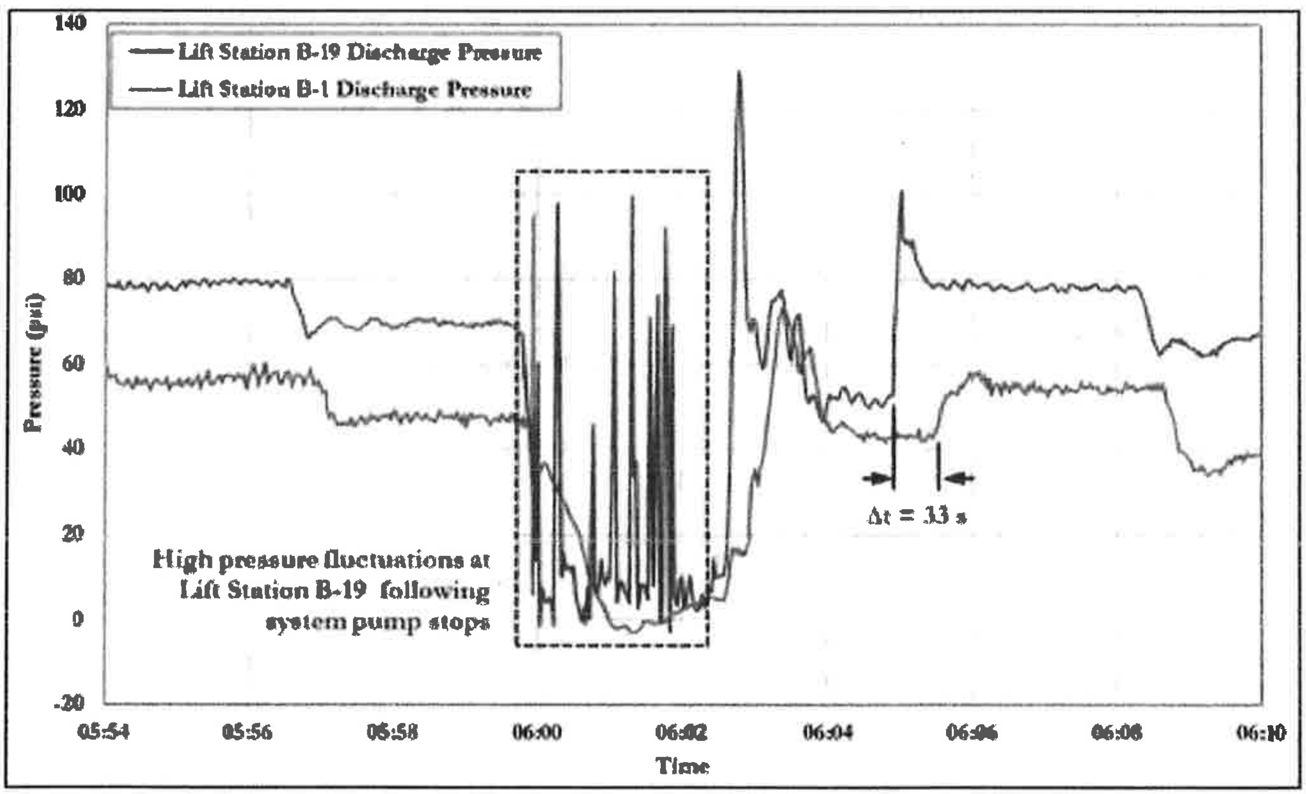


Figure 52: Transient pressure monitoring data from 5:54 AM to 6:10 AM on 05 October 2017 showing high pressure fluctuations at Lift Station B-19 following pump stop operations

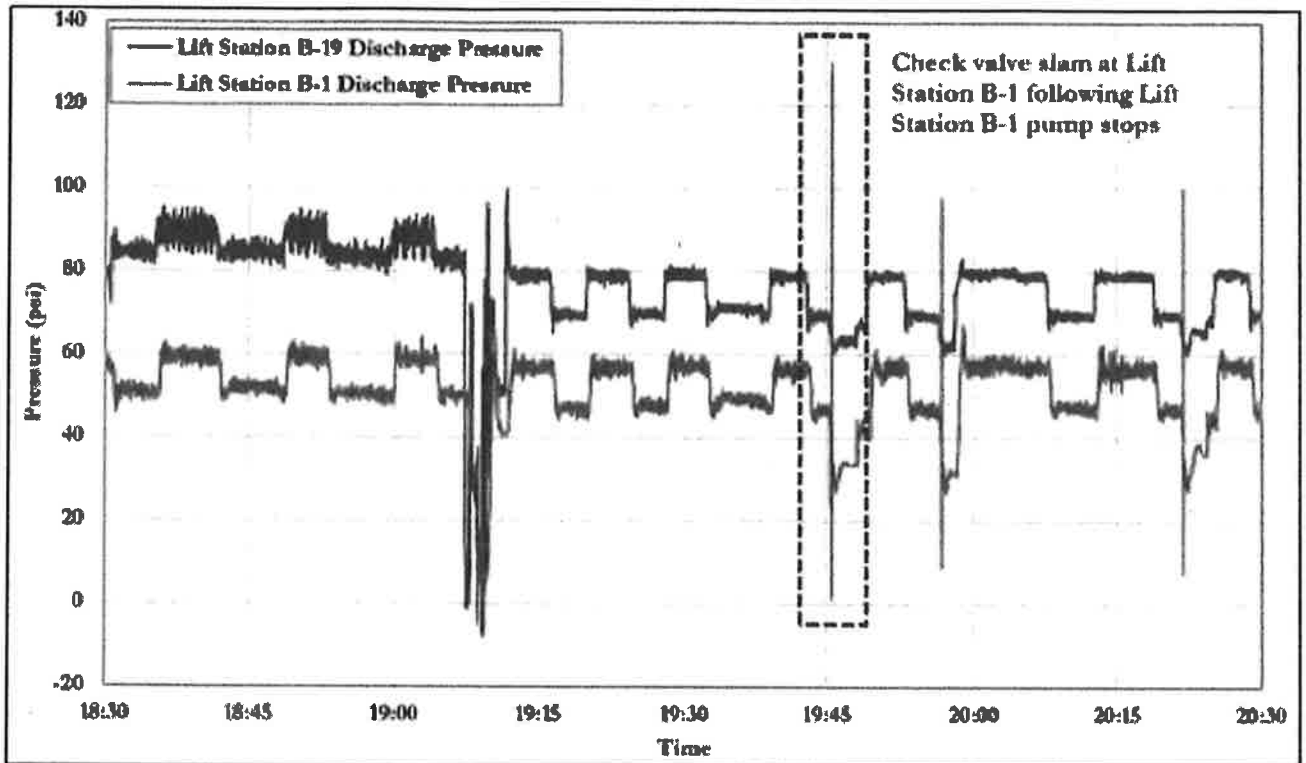


Figure 53: Transient pressure monitoring data from 6:30 PM to 8:30 PM on 05 October 2017 showing normal pump on-off operations and check valve slam at Lift Station B-1

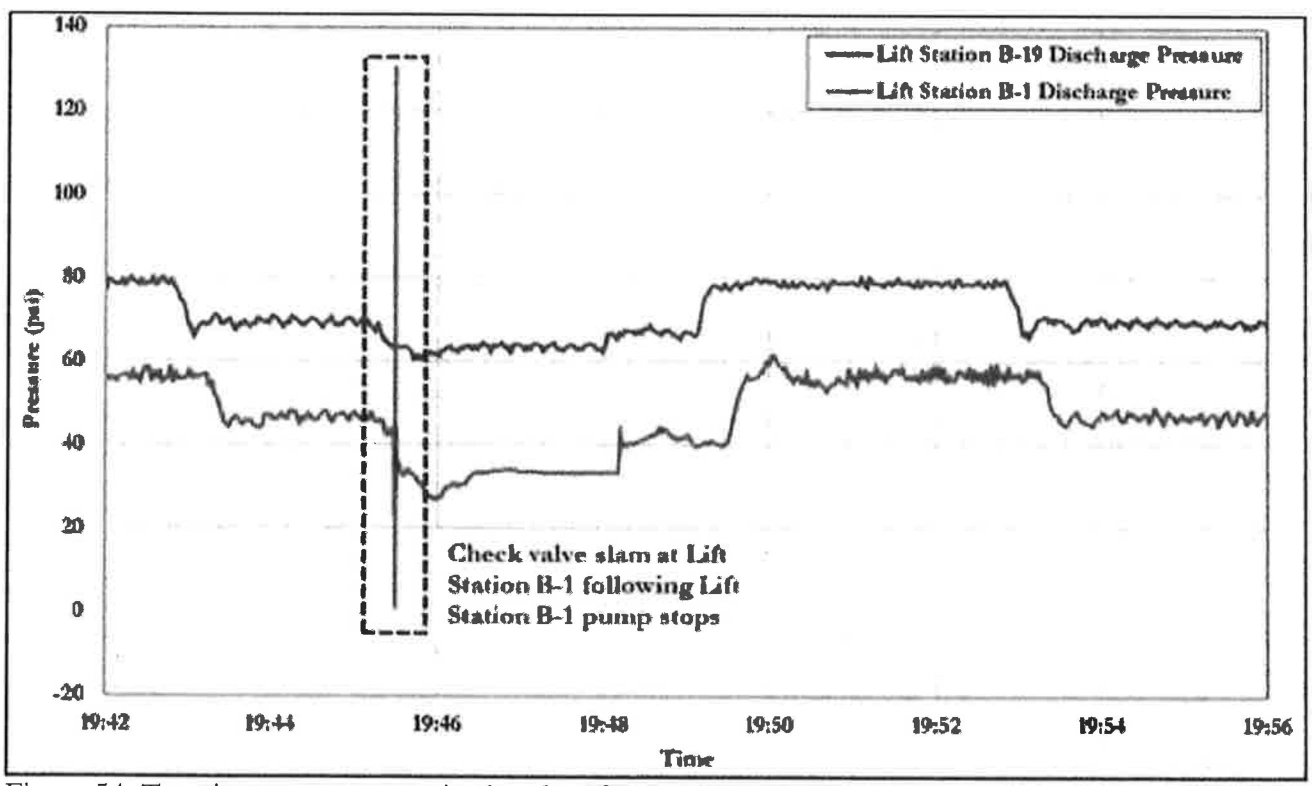


Figure 54: Transient pressure monitoring data from 7:42 PM to 7:56 PM on 05 October 2017 showing normal pump on-off operations and check valve slam at Lift Station B-1

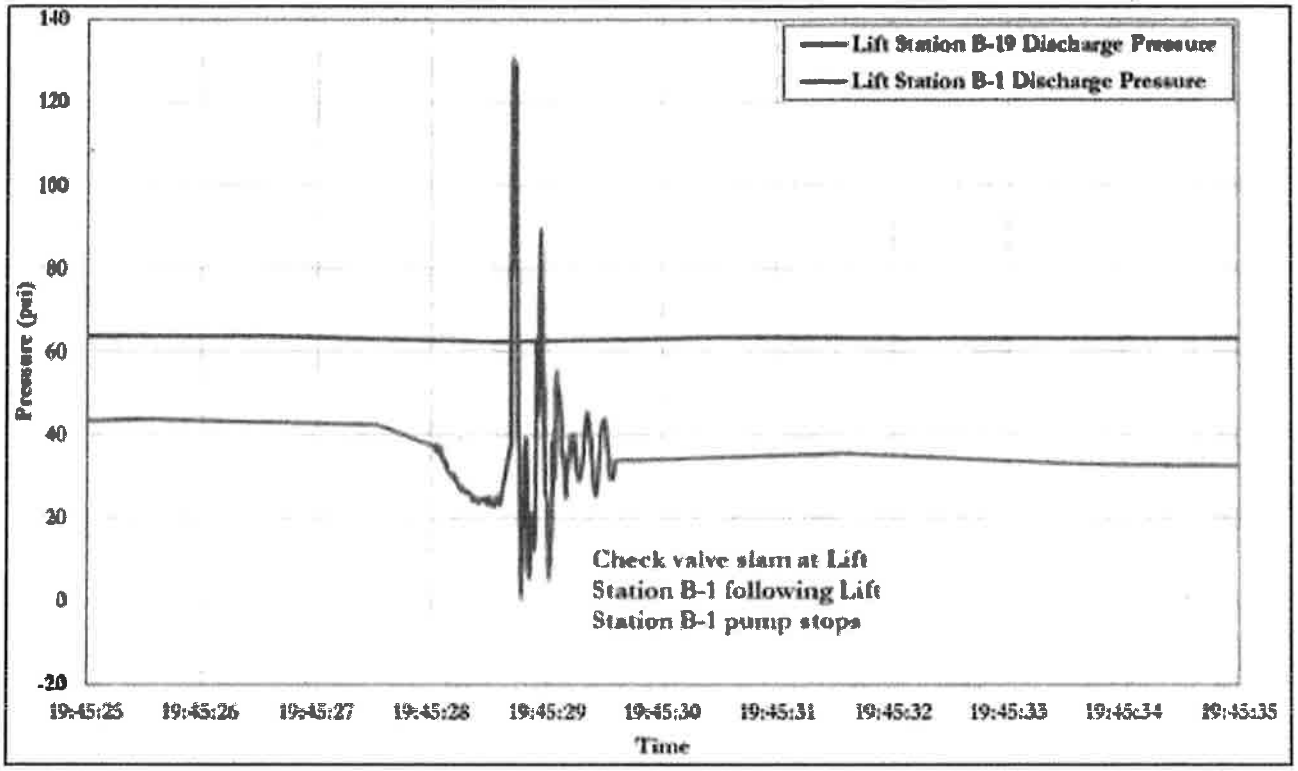


Figure 55: Transient pressure monitoring data at 7:45 PM on 05 October 2017 showing check valve slam at Lift Station B-1

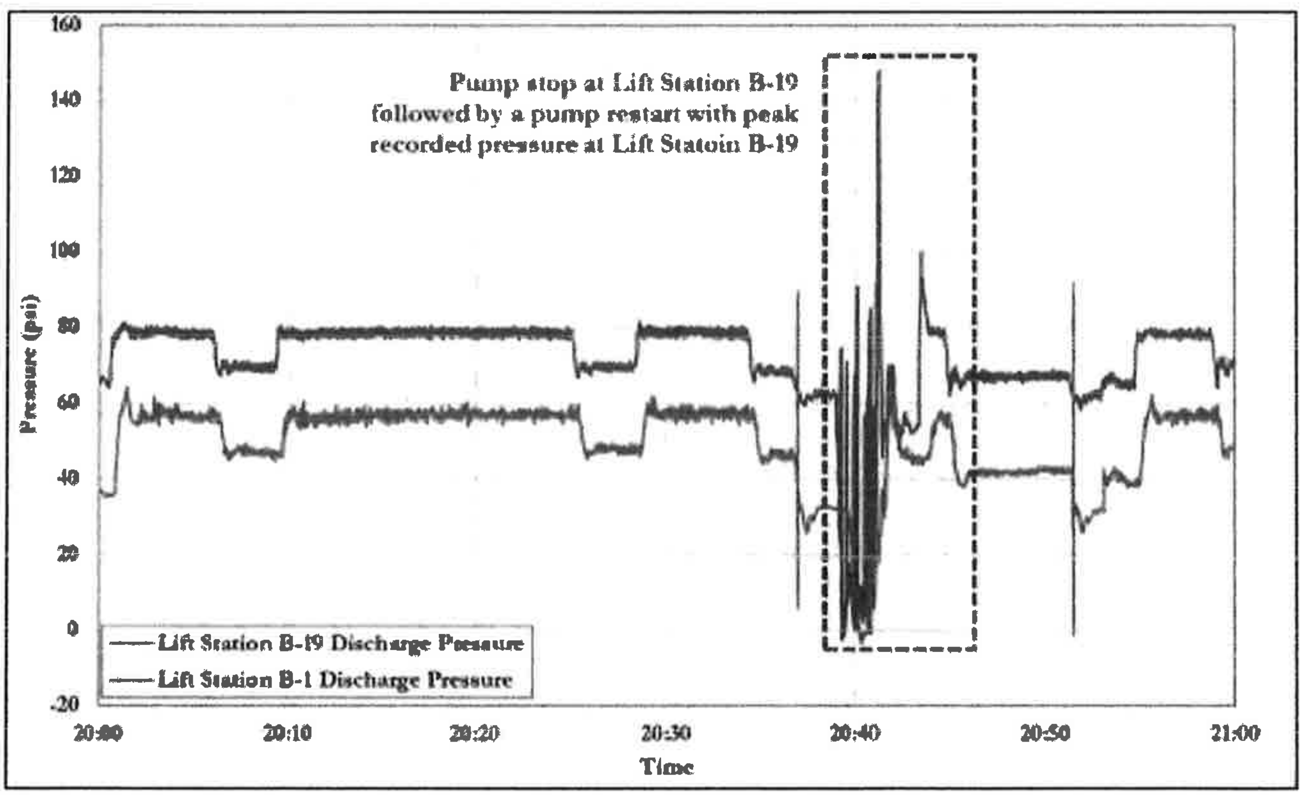


Figure 56: Transient pressure monitoring data from 8:00 PM to 9:00 PM on 08 October 2017 showing peak pressure recorded at Lift Station B-19

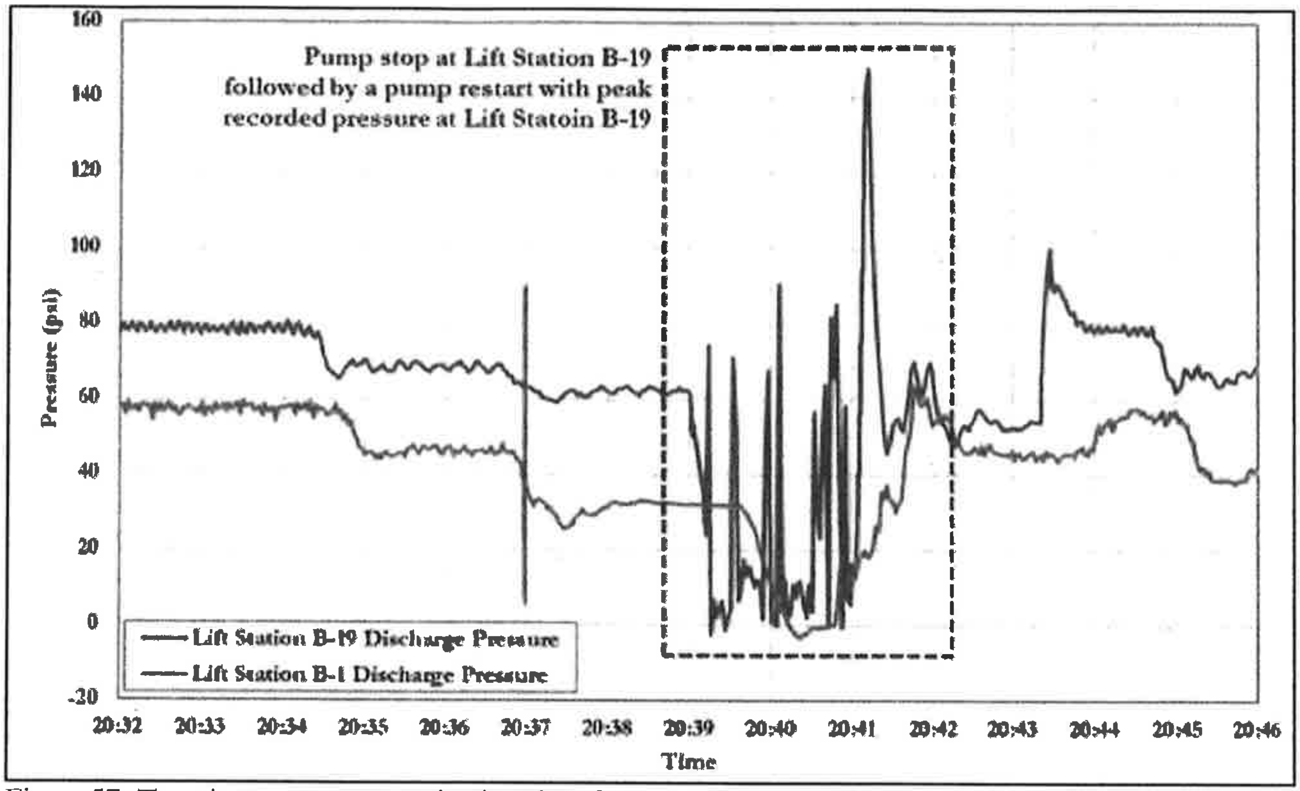


Figure 57: Transient pressure monitoring data from 8:32 PM to 8:36 PM on 08 October 2017 showing peak pressure recorded at Lift Station B-19

APPENDIX D: BLACOH COMPLETE TRANSIENT PRESSURE DATA PLOTS

HDR – B19

September 26, 2017 – October 25, 2017

BLACOH™ SURGE CONTROL

HDR – B19
November 8, 2017

Figure 1: September 26, 2017 – October 25, 2017

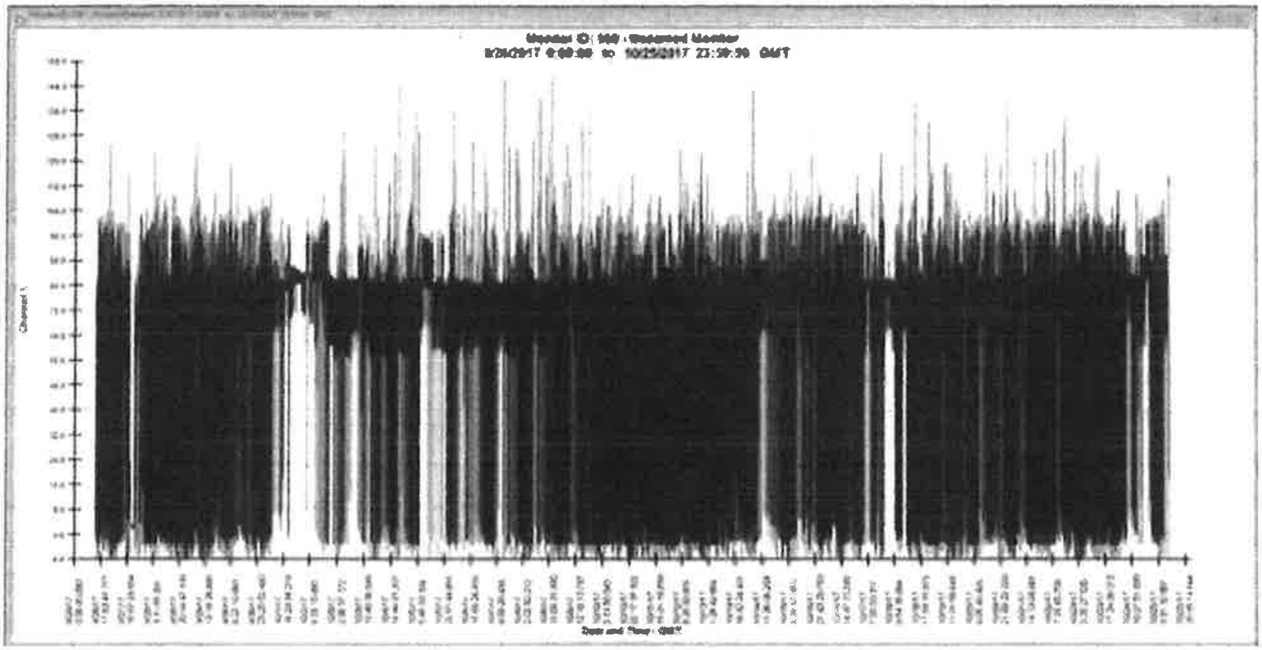


Figure 2: Highest Recorded Data Point (October 8, 2017)

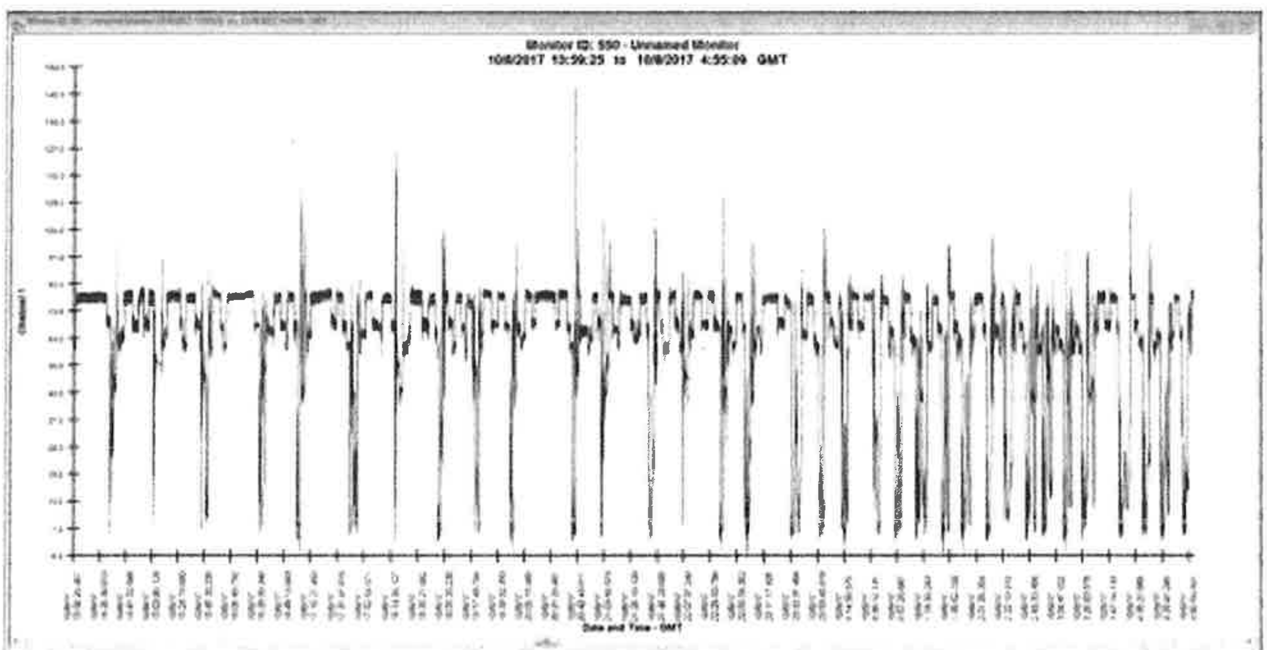


Figure 3: Highest Recorded Data Point (Zoom Level A)

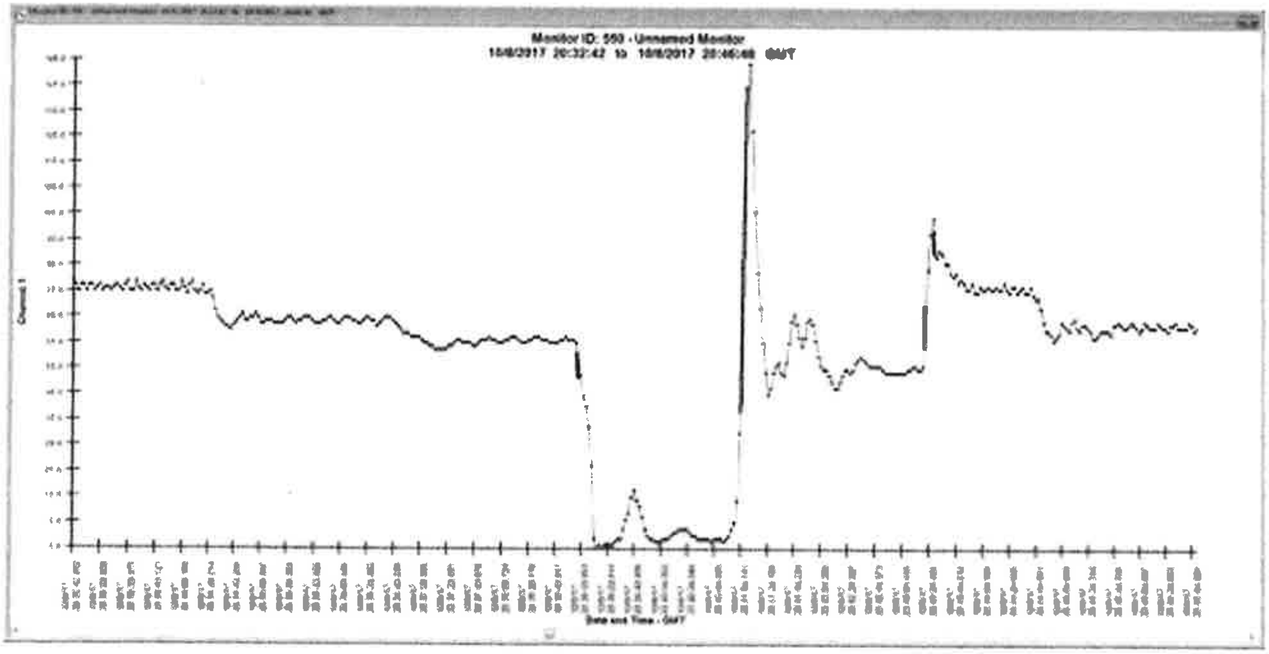


Figure 4: Highest Recorded Data Point (Zoom Level B)

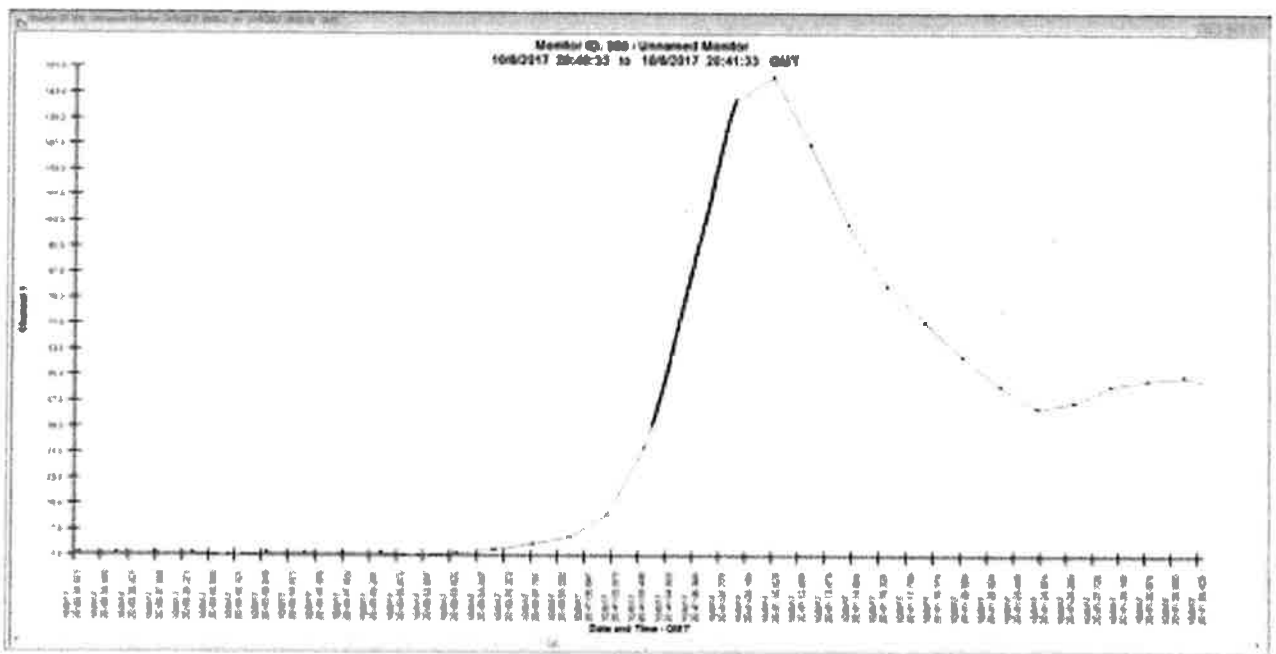


Figure 5: Lowest Recorded Data Point *(One instance)*

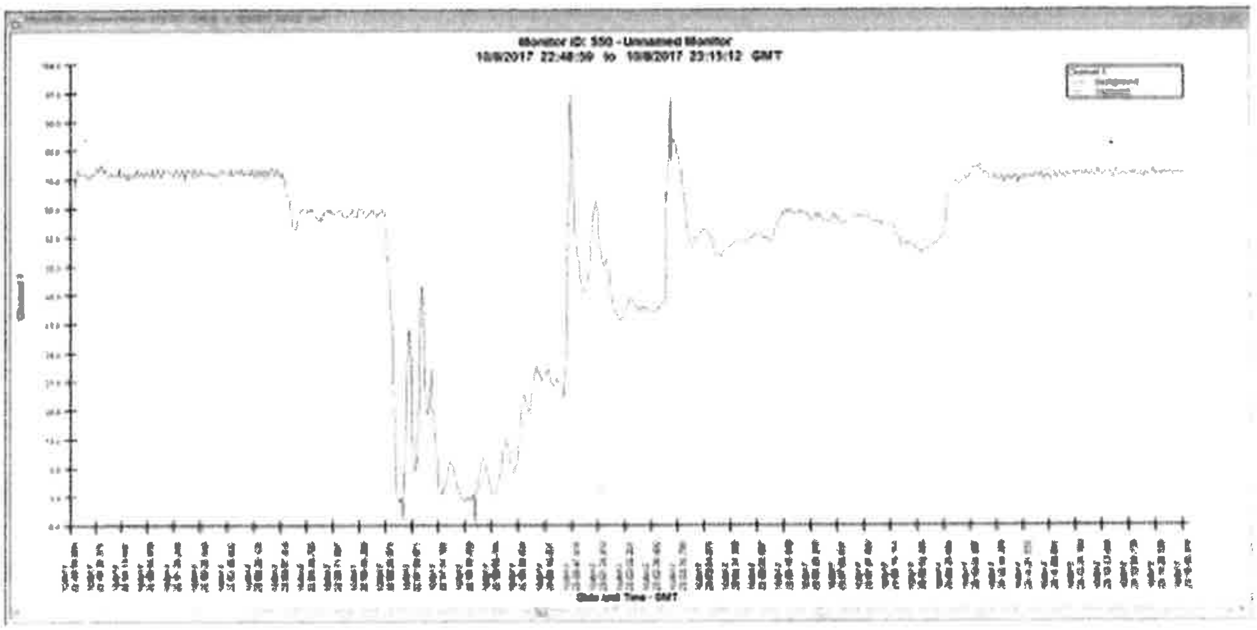


Figure 6: Lowest Recorded Data Point (Zoom Level A)

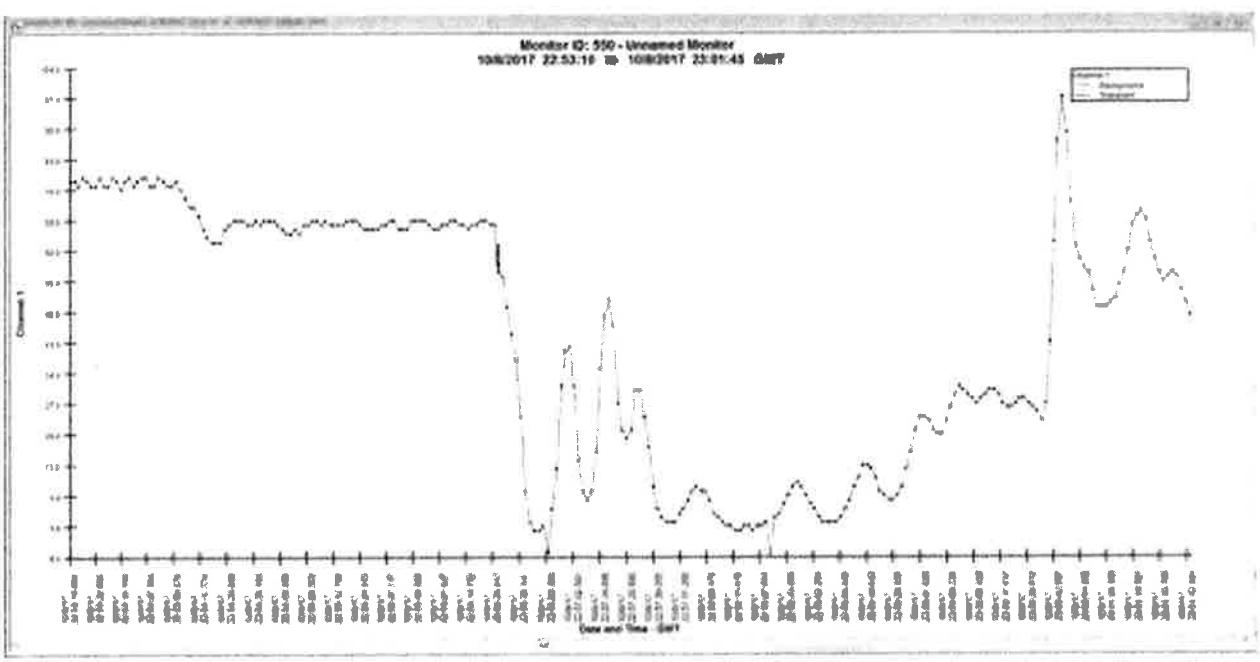


Figure 7: Lowest Recorded Data Point (Zoom Level B)

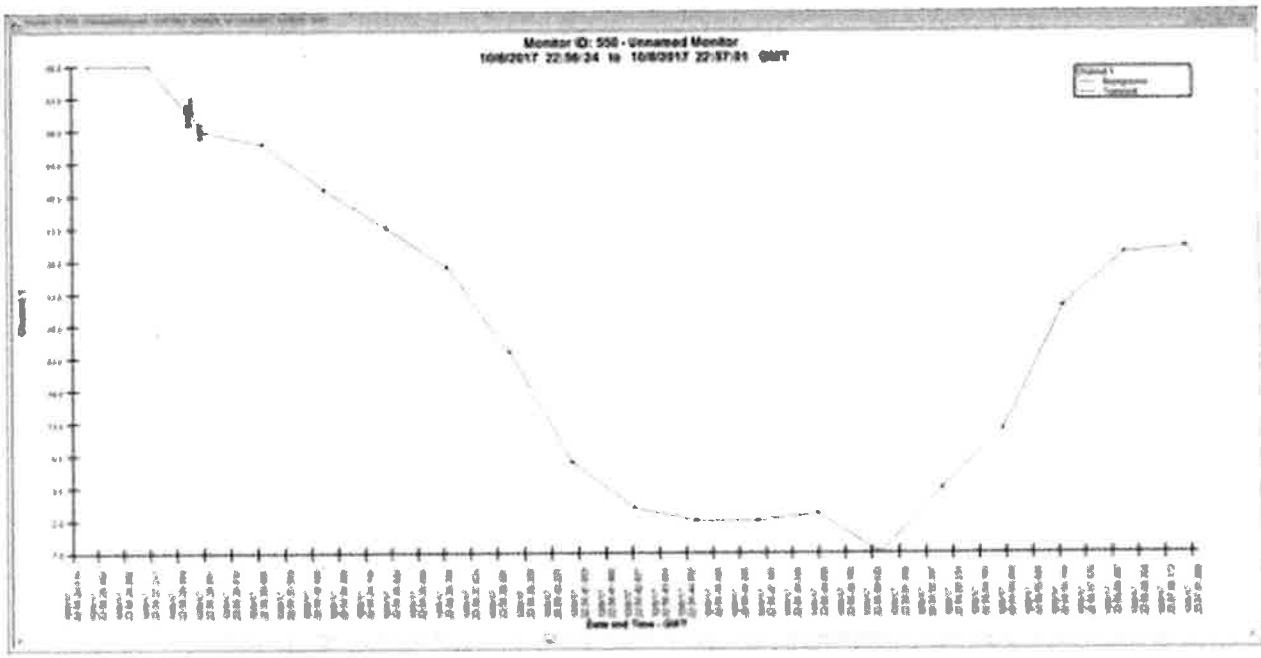


Figure 8: September 26, 2017

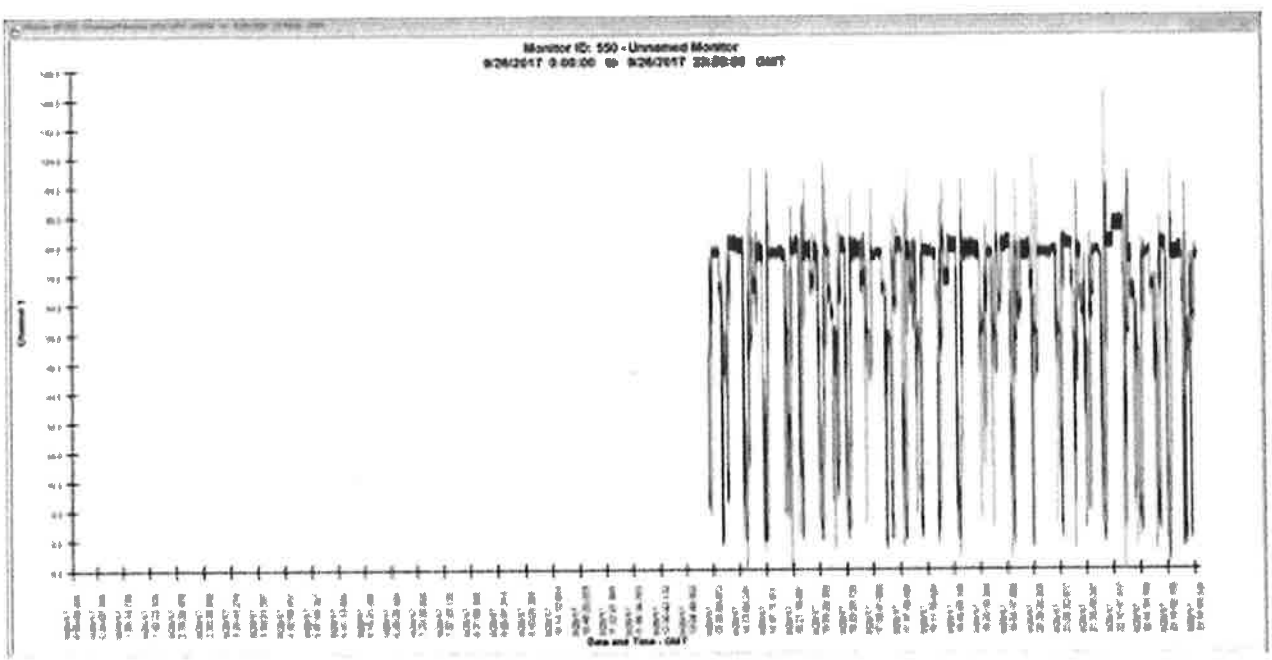


Figure 9: September 27, 2017

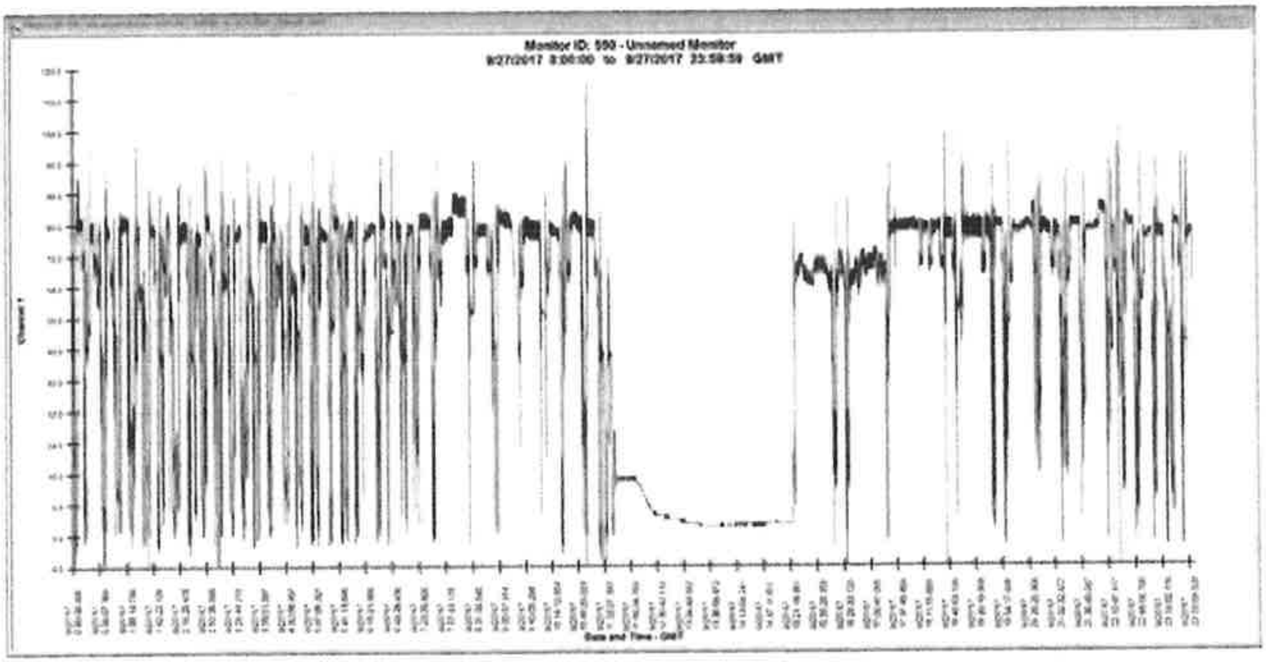


Figure 10: September 28, 2017

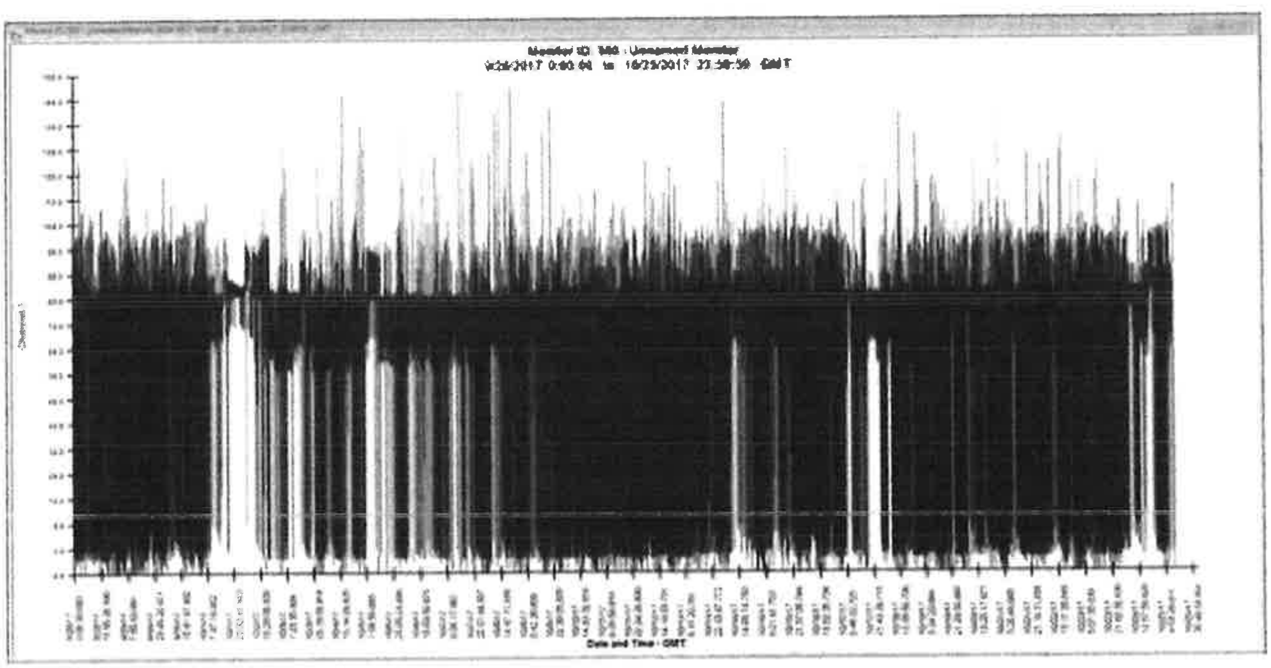


Figure 11: September 29, 2017

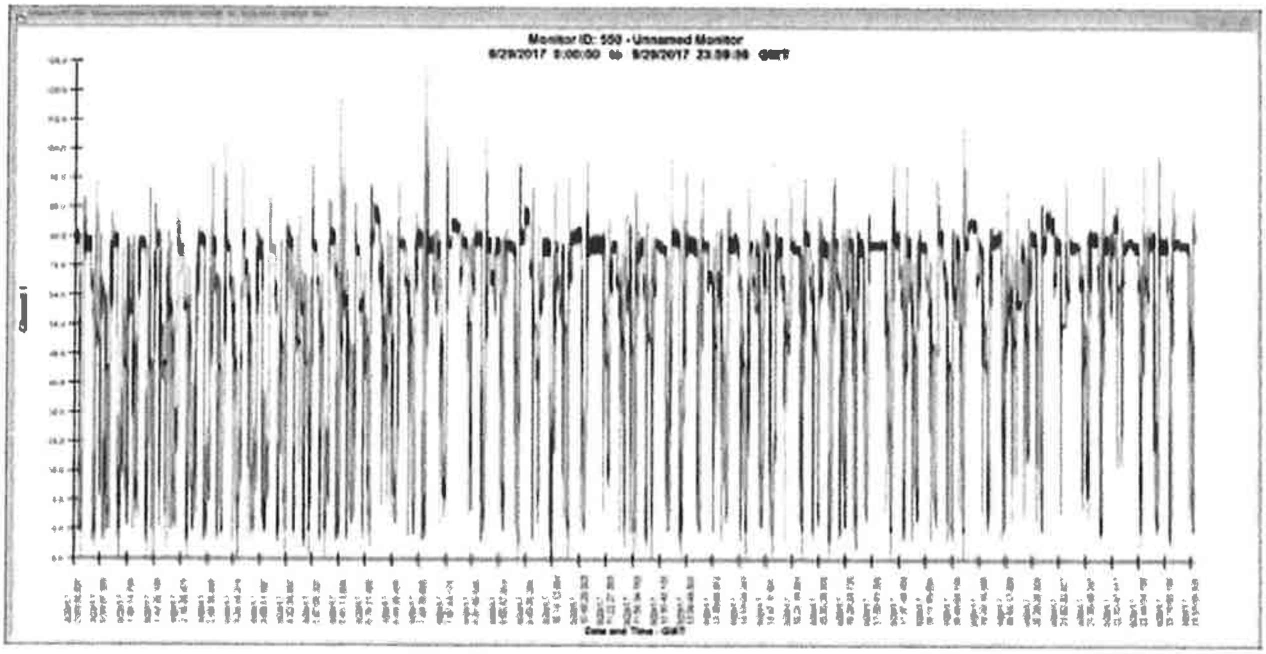


Figure 12: September 30, 2017

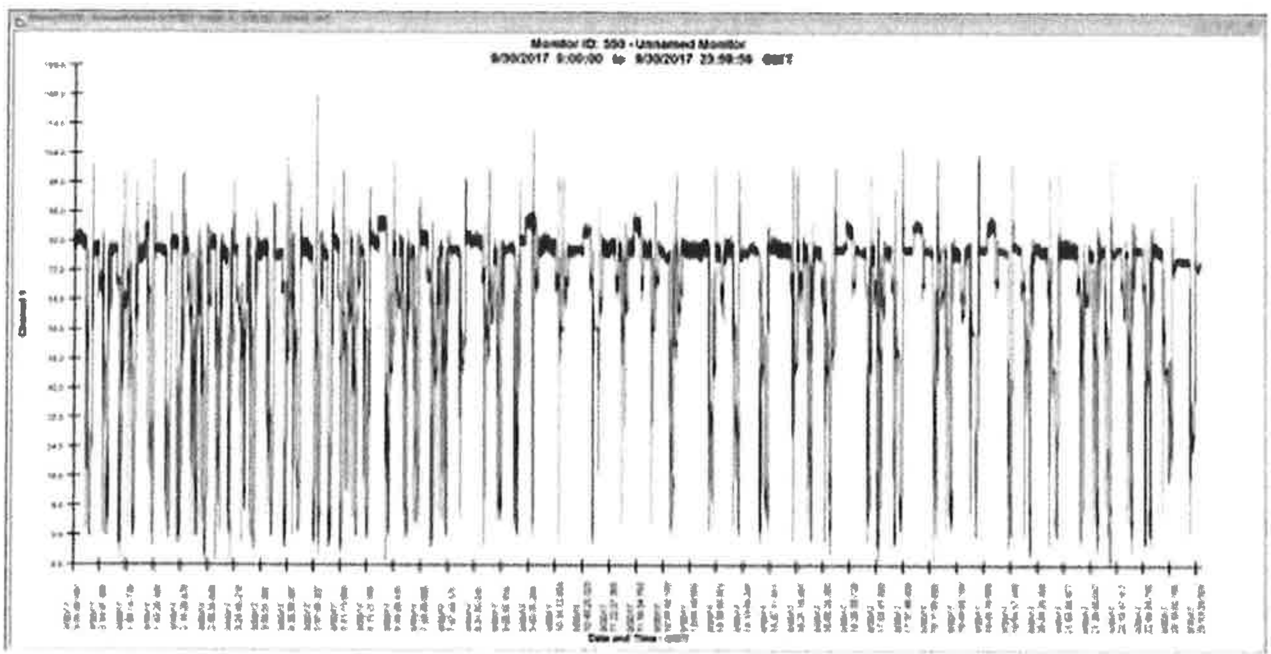


Figure 13: October 1, 2017

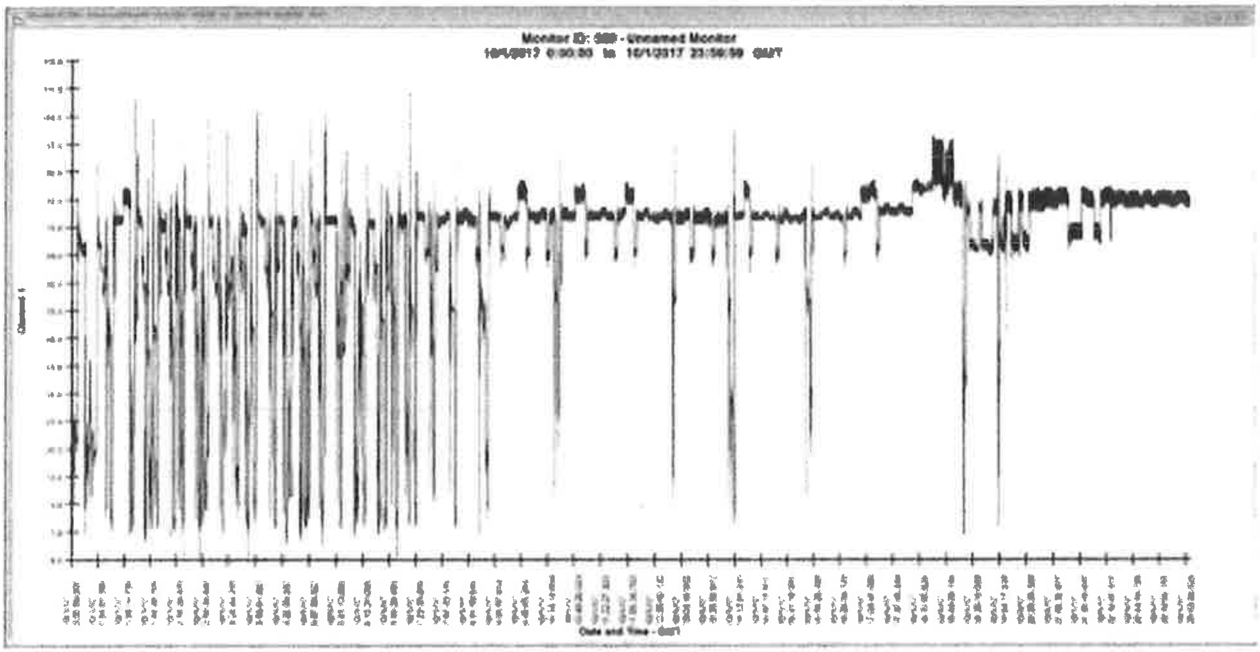


Figure 14: October 2, 2017

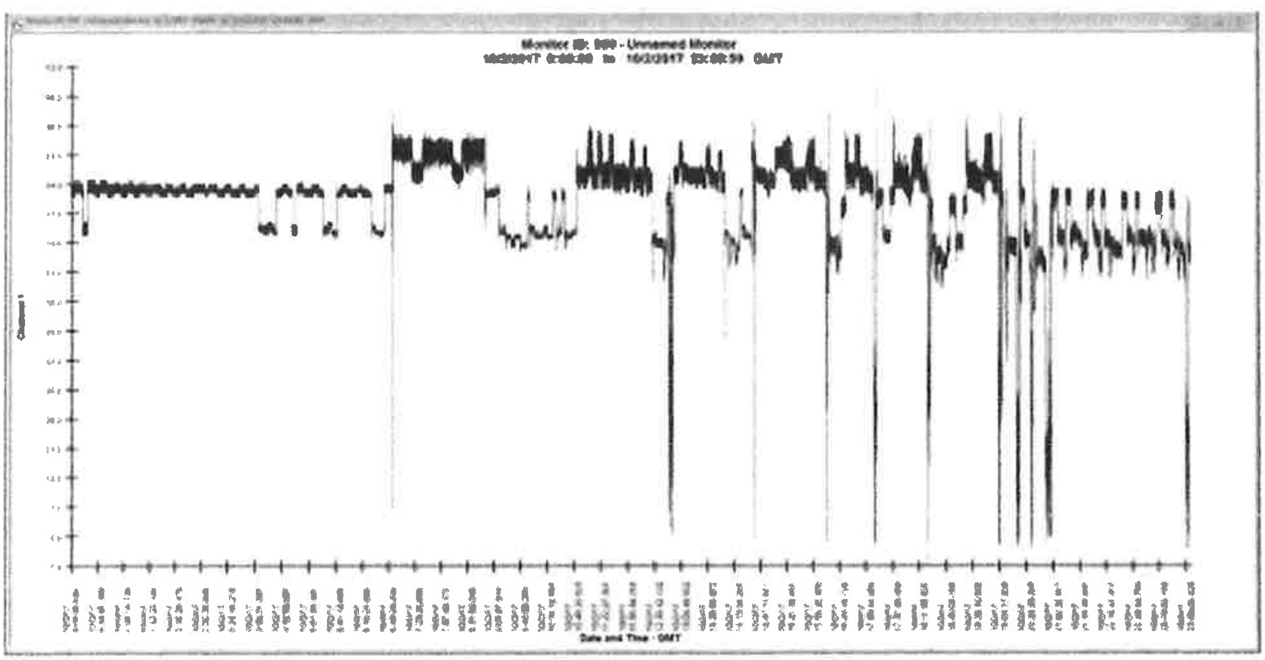


Figure 15: October 3, 2017

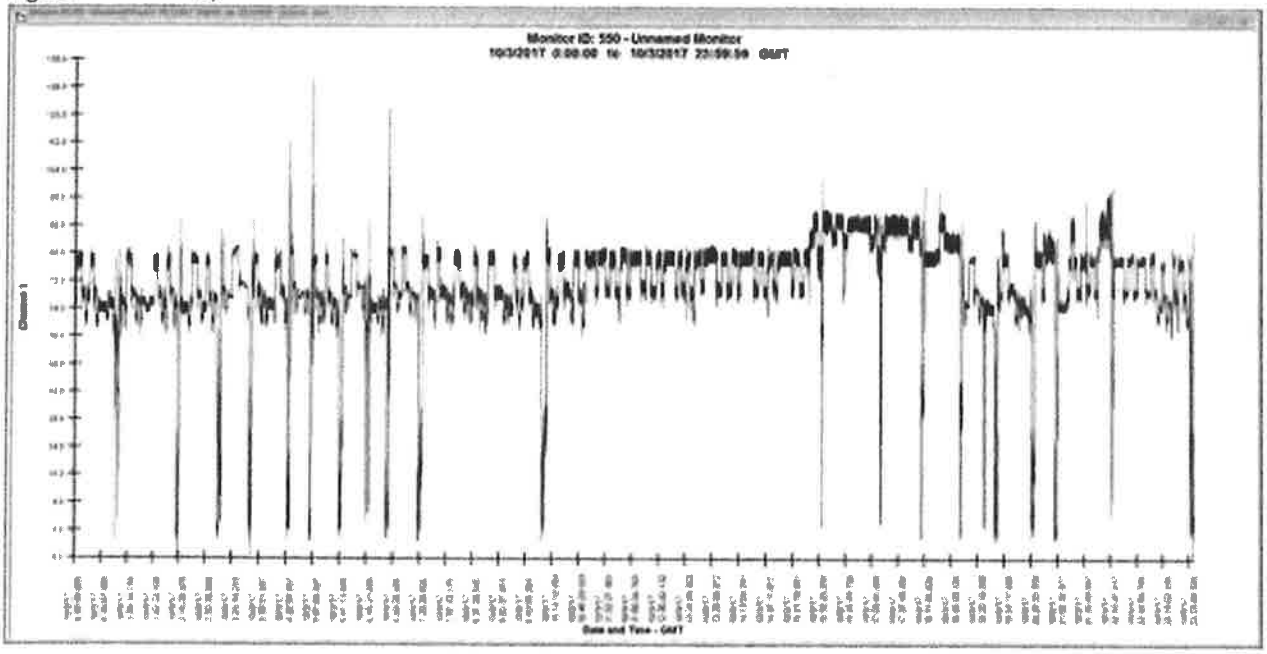


Figure 16: October 4, 2017

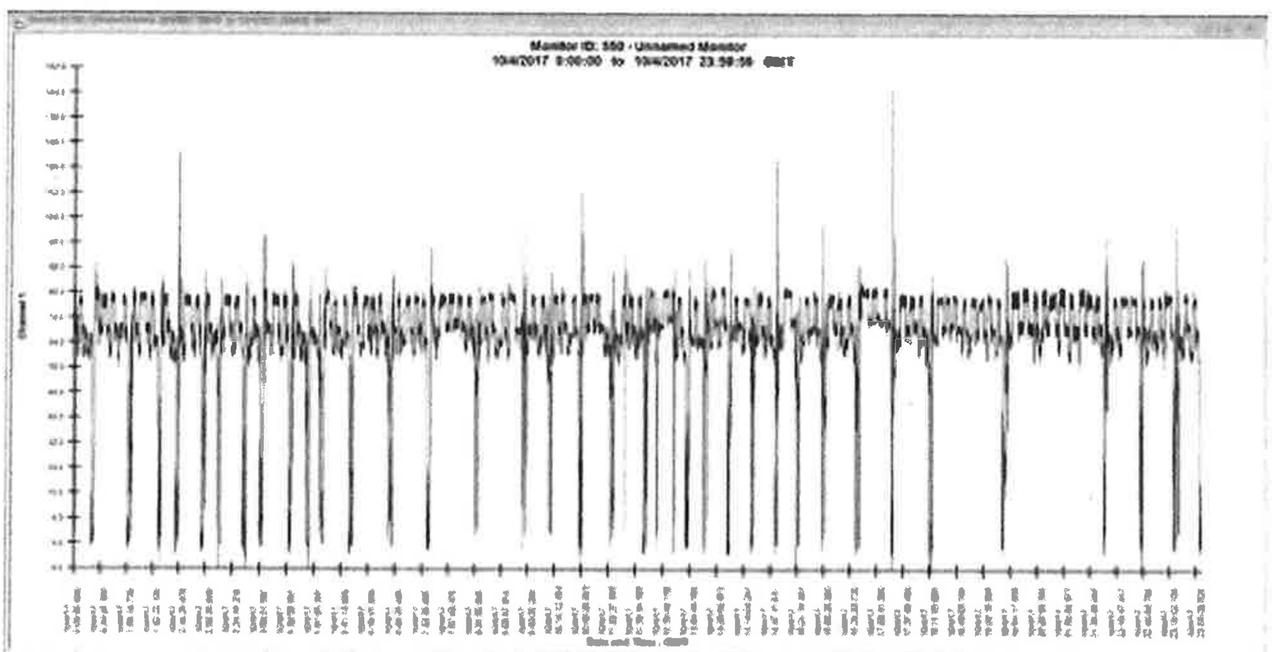


Figure 17: October 5, 2017

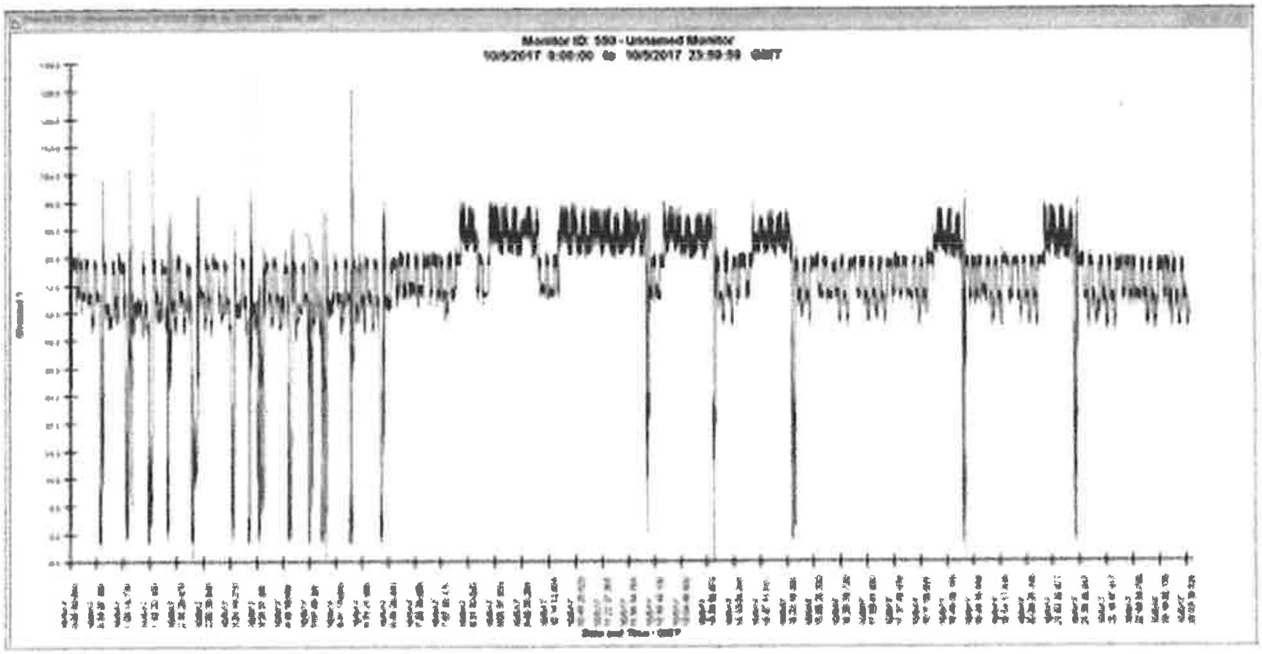


Figure 18: October 6, 2017

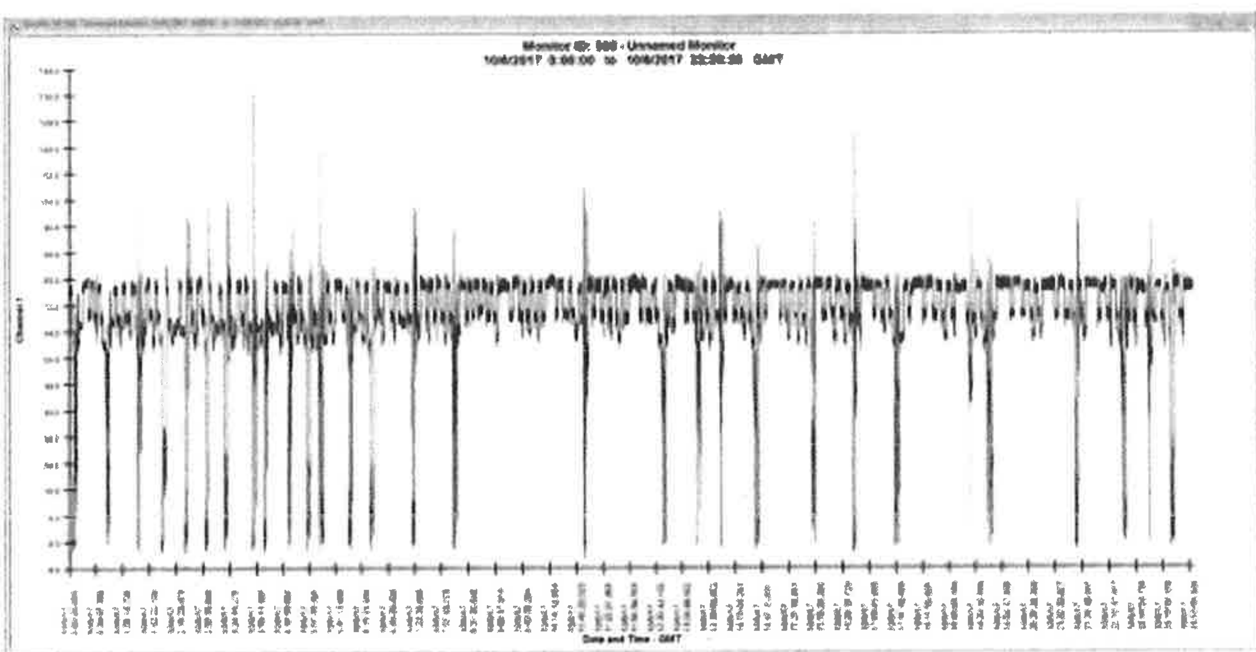


Figure 19: October 7, 2017

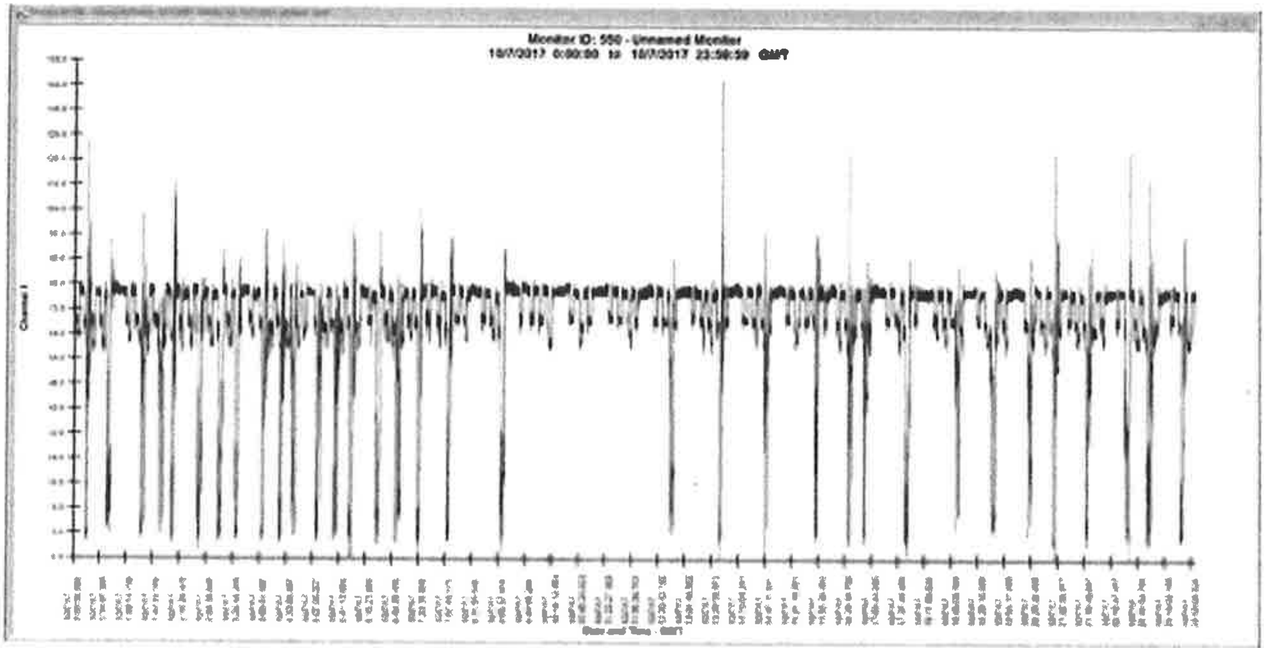


Figure 20: October 8, 2017

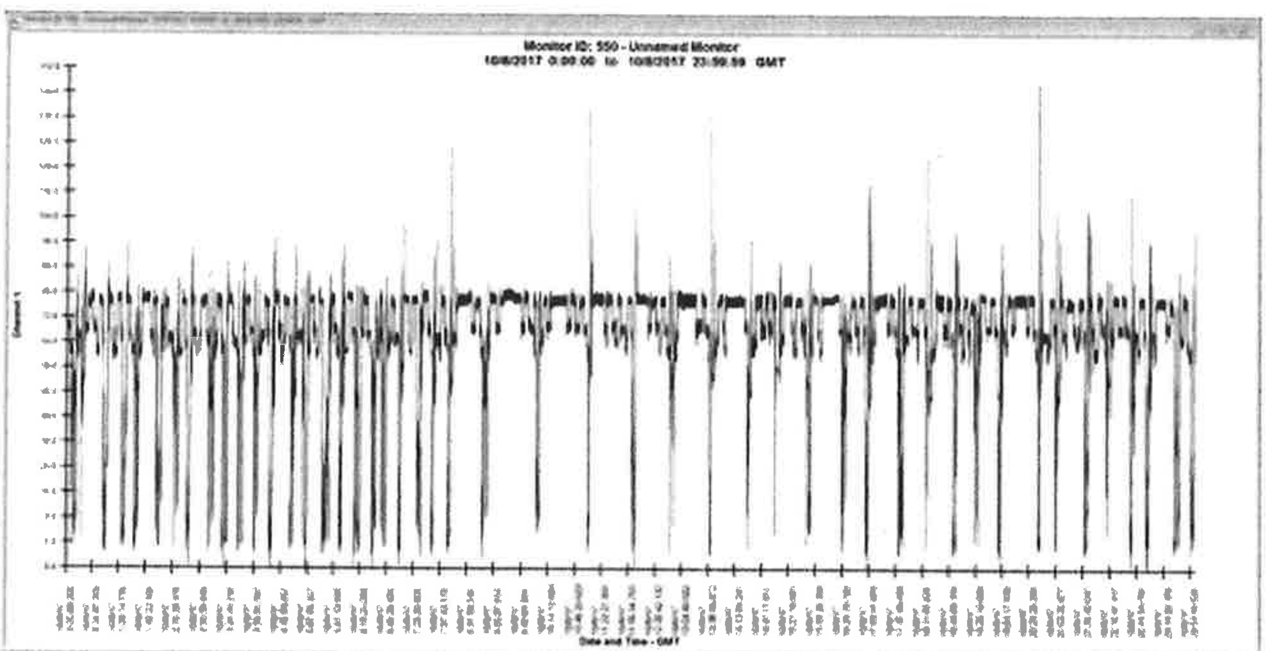


Figure 21: October 9, 2017

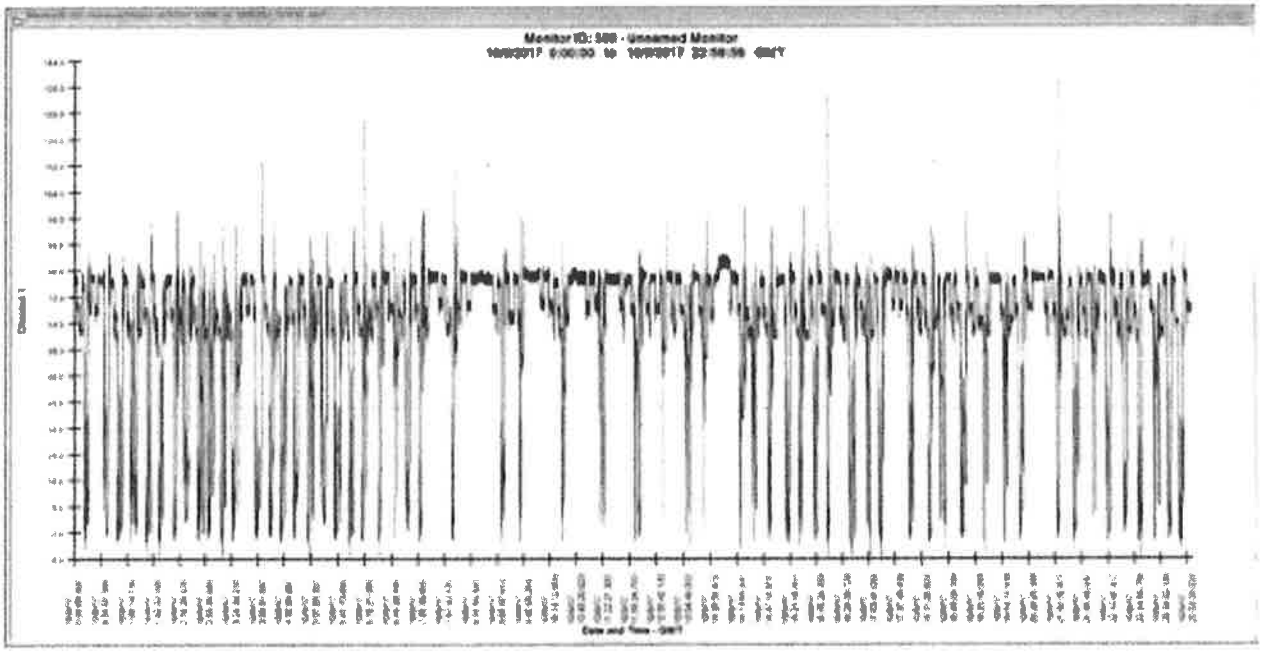


Figure 22: October 10, 2017

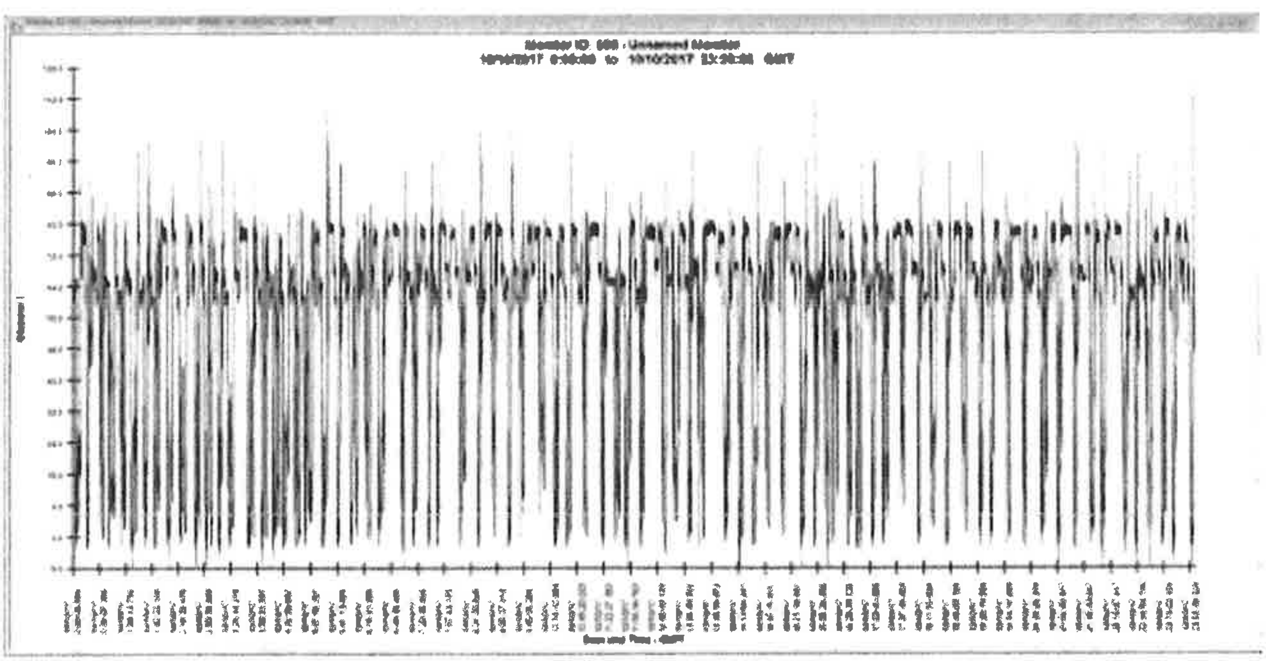


Figure 23: October 11, 2017

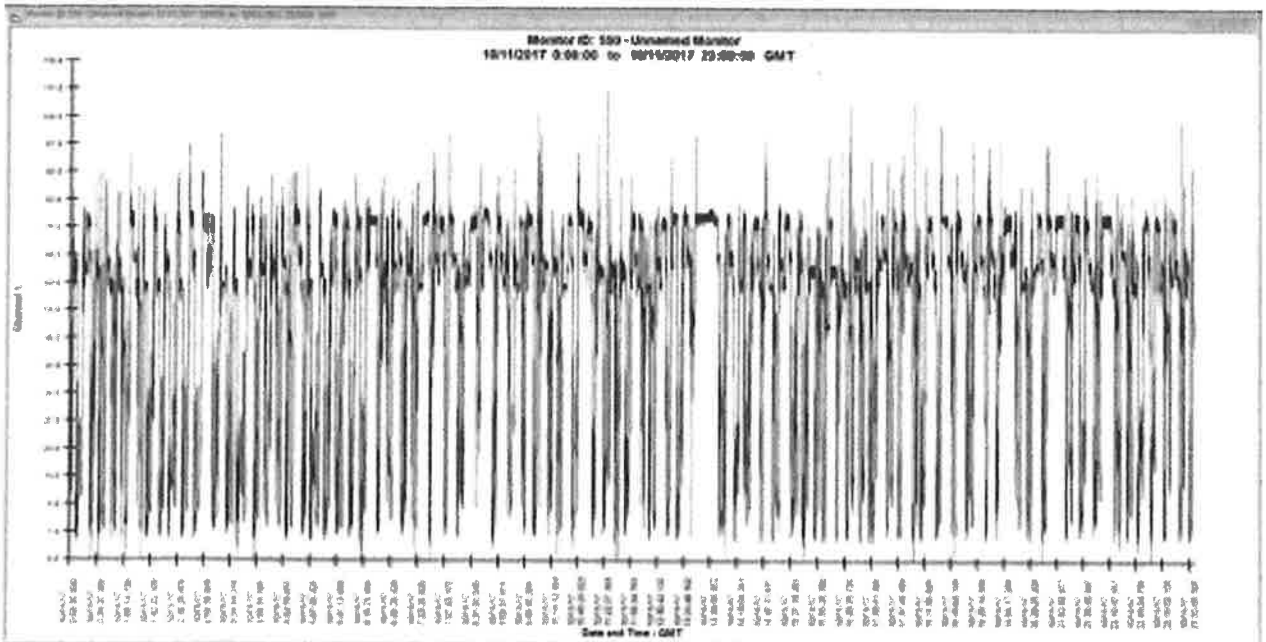


Figure 24: October 12, 2017

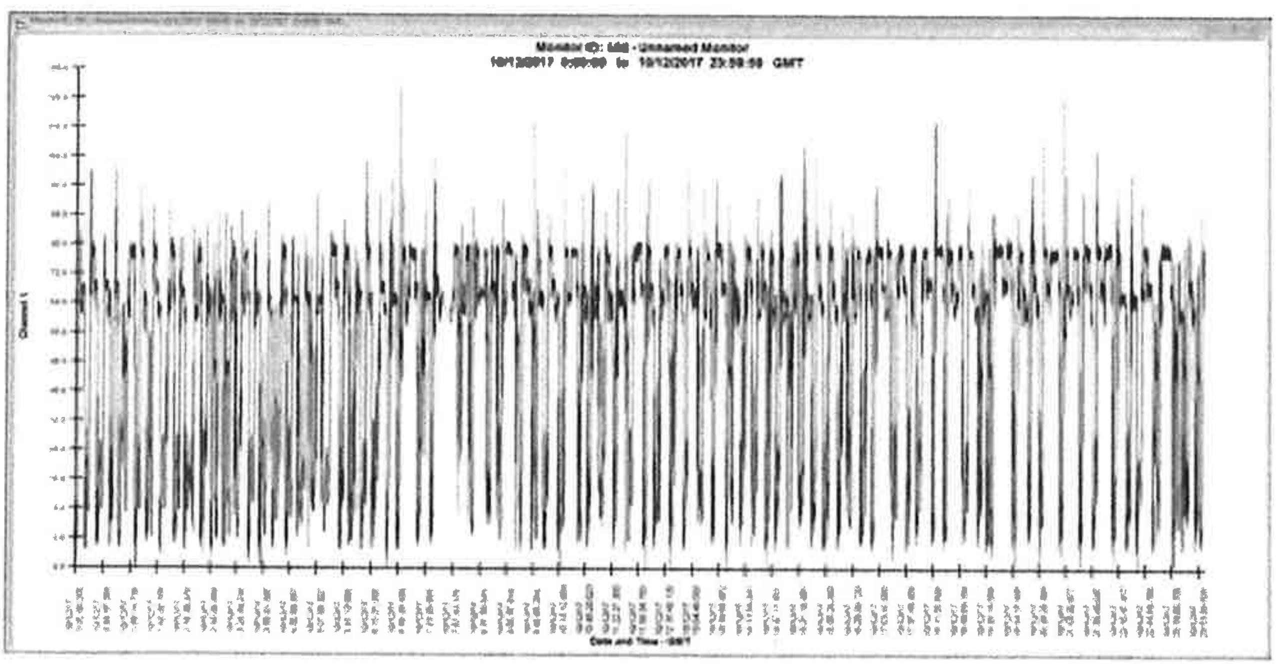


Figure 25: October 13, 2017

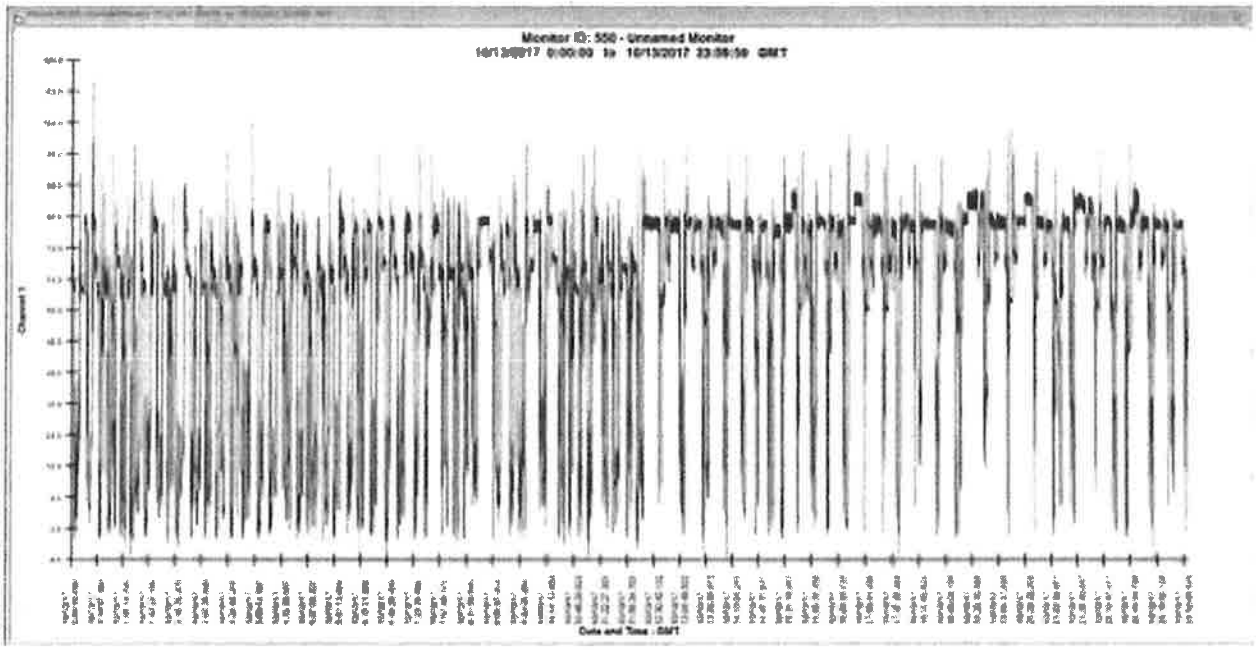


Figure 26: October 14, 2017

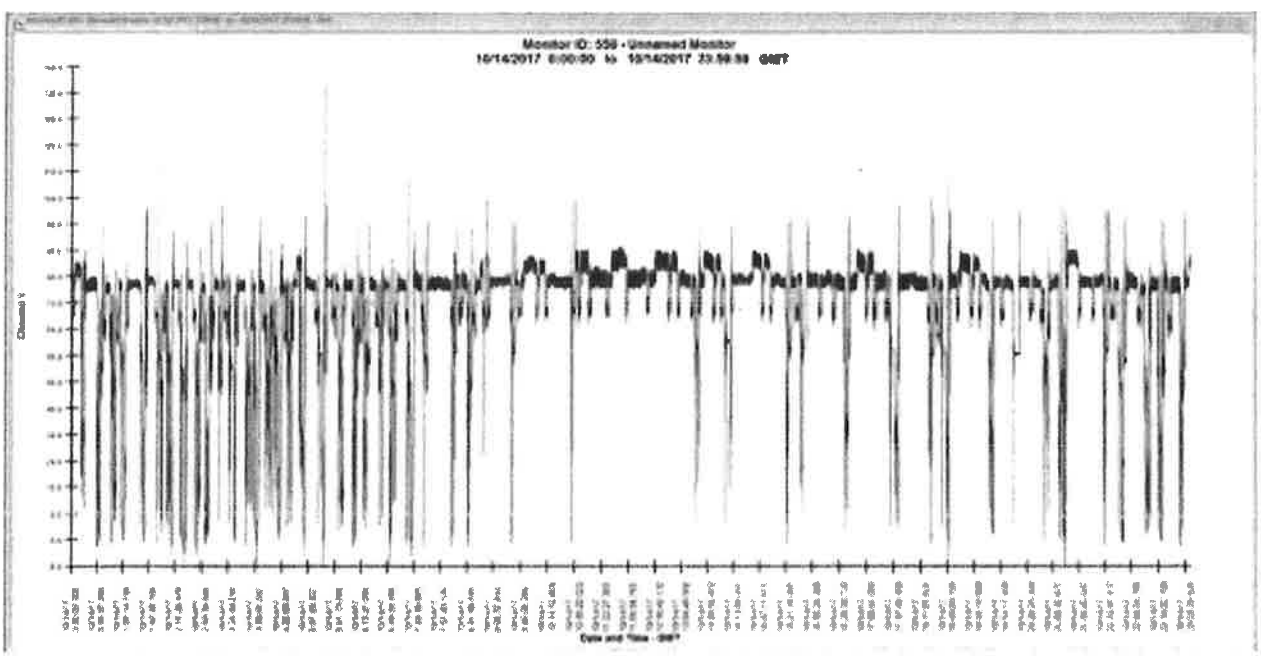


Figure 27: October 15, 2017

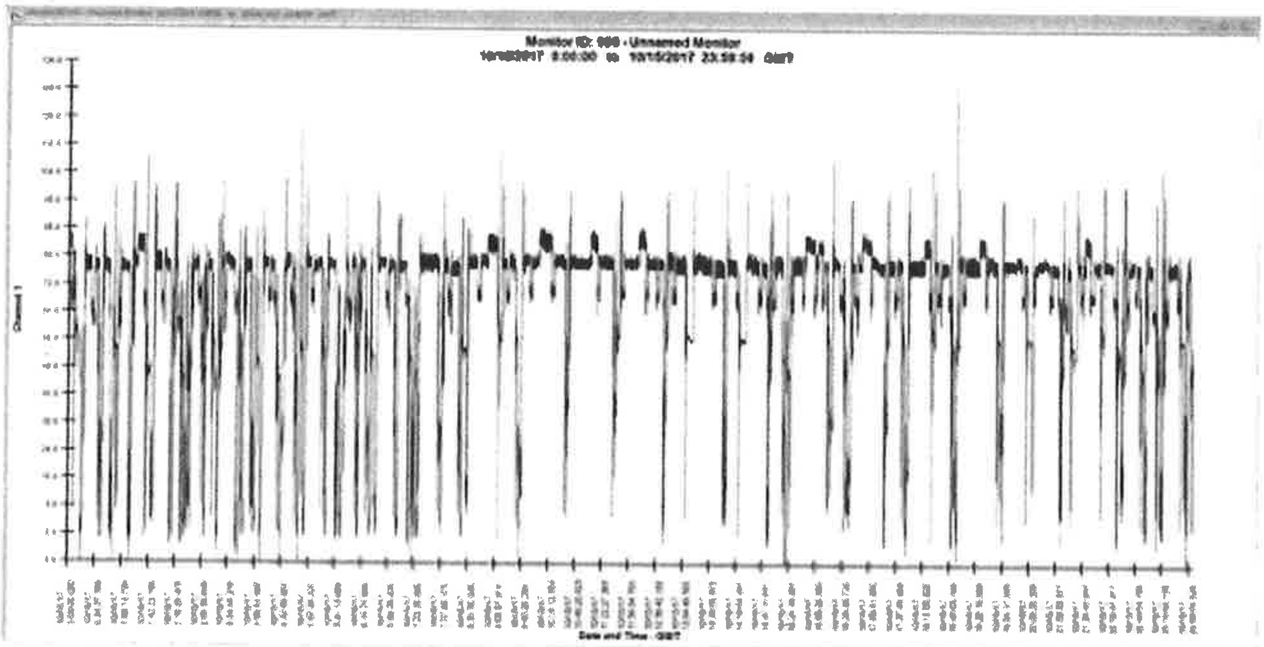


Figure 28: October 16, 2017

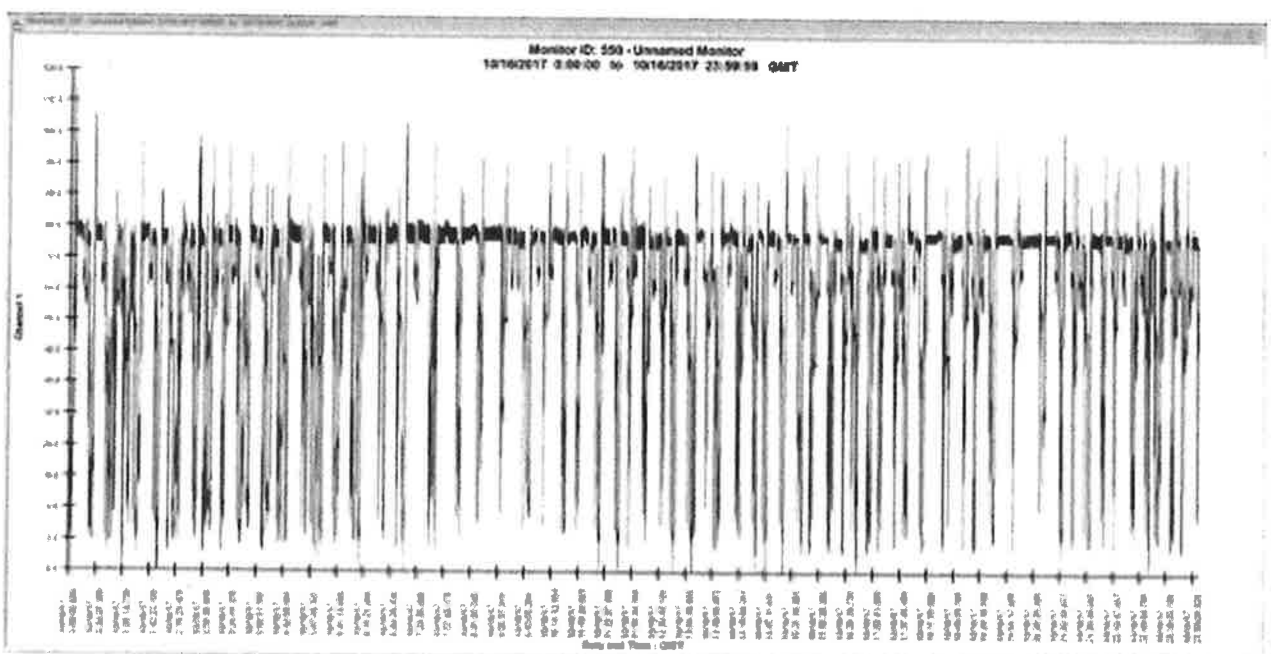


Figure 29: October 17, 2017

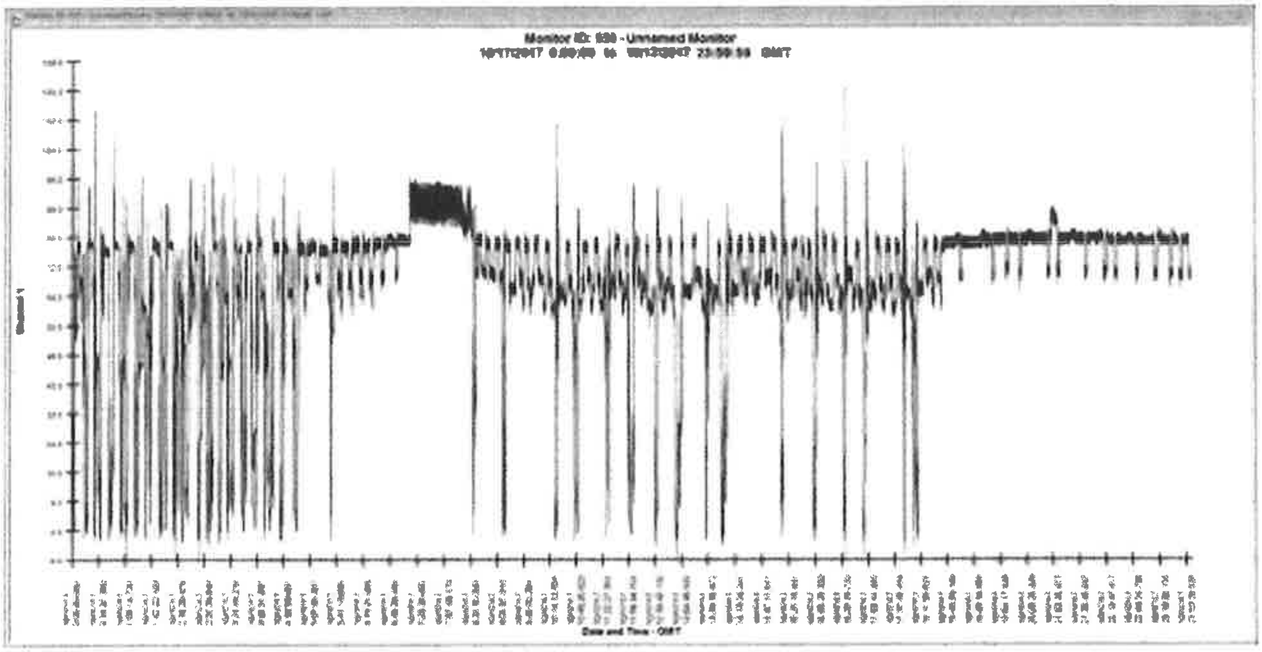


Figure 30: October 18, 2017

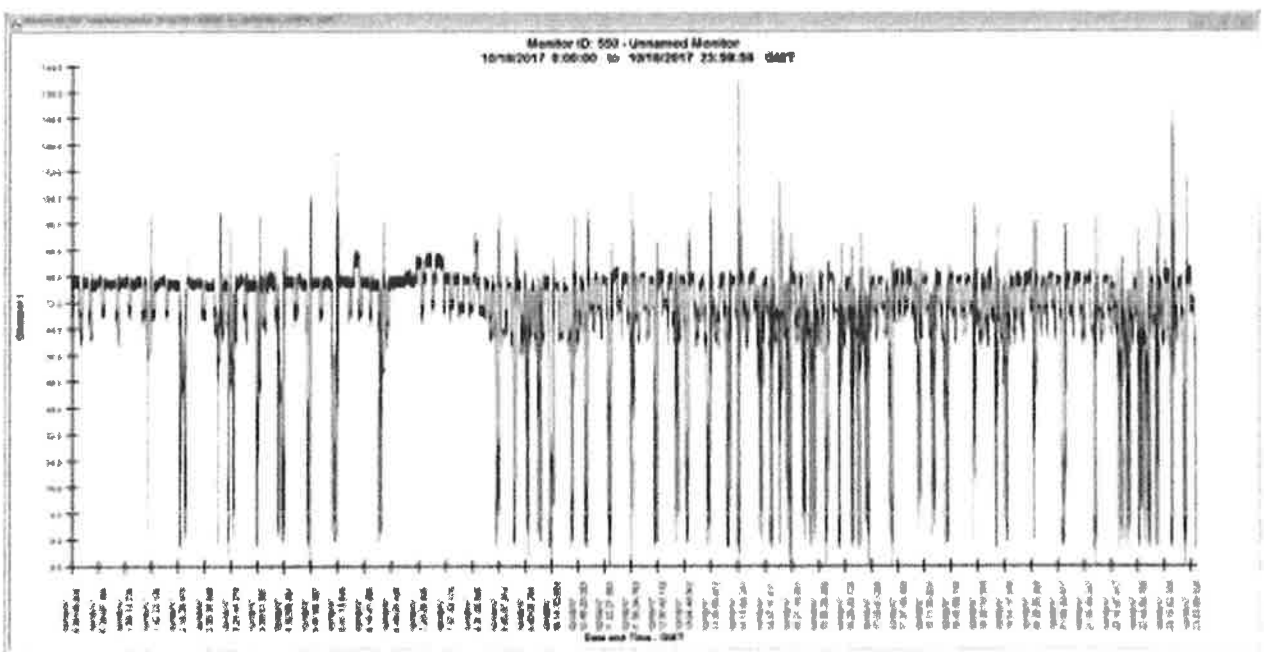


Figure 31: October 19, 2017

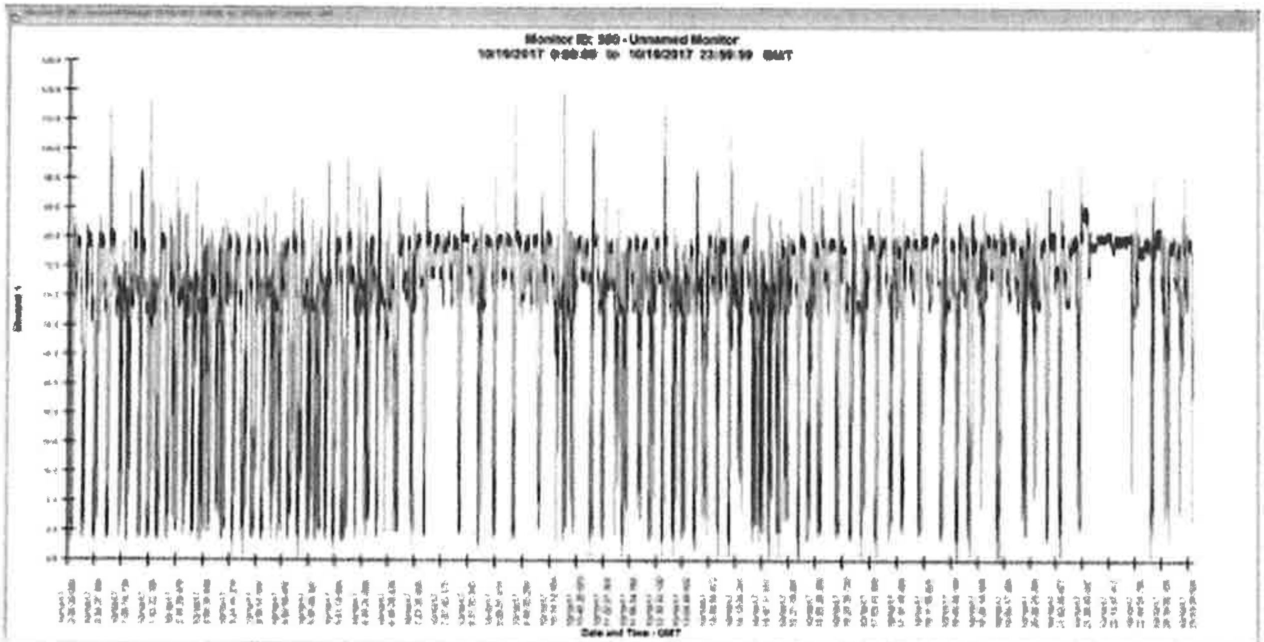


Figure 32: October 20, 2017

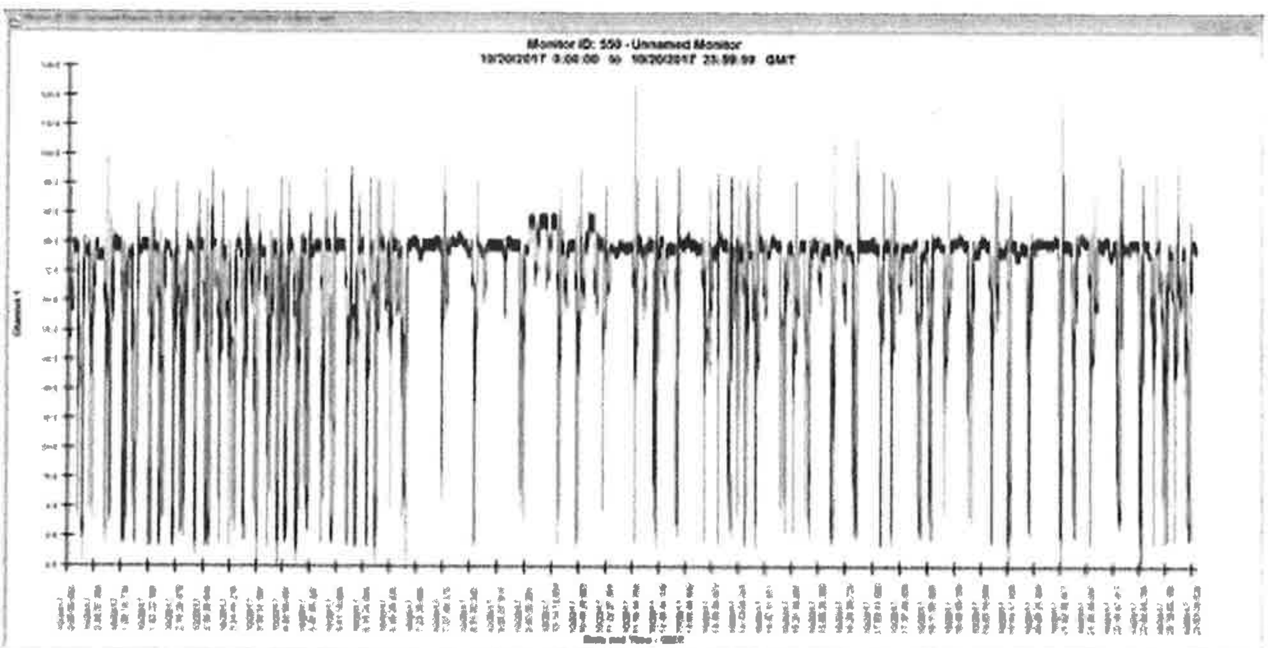


Figure 33: October 21, 2017

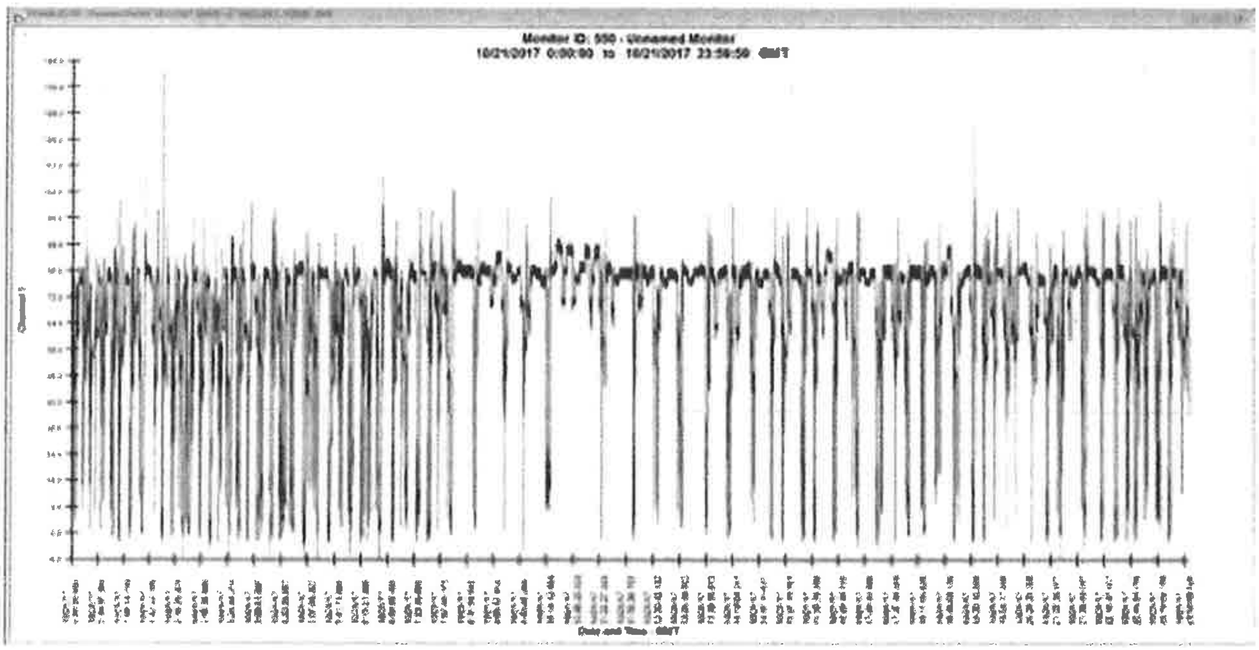


Figure 34: October 22, 2017

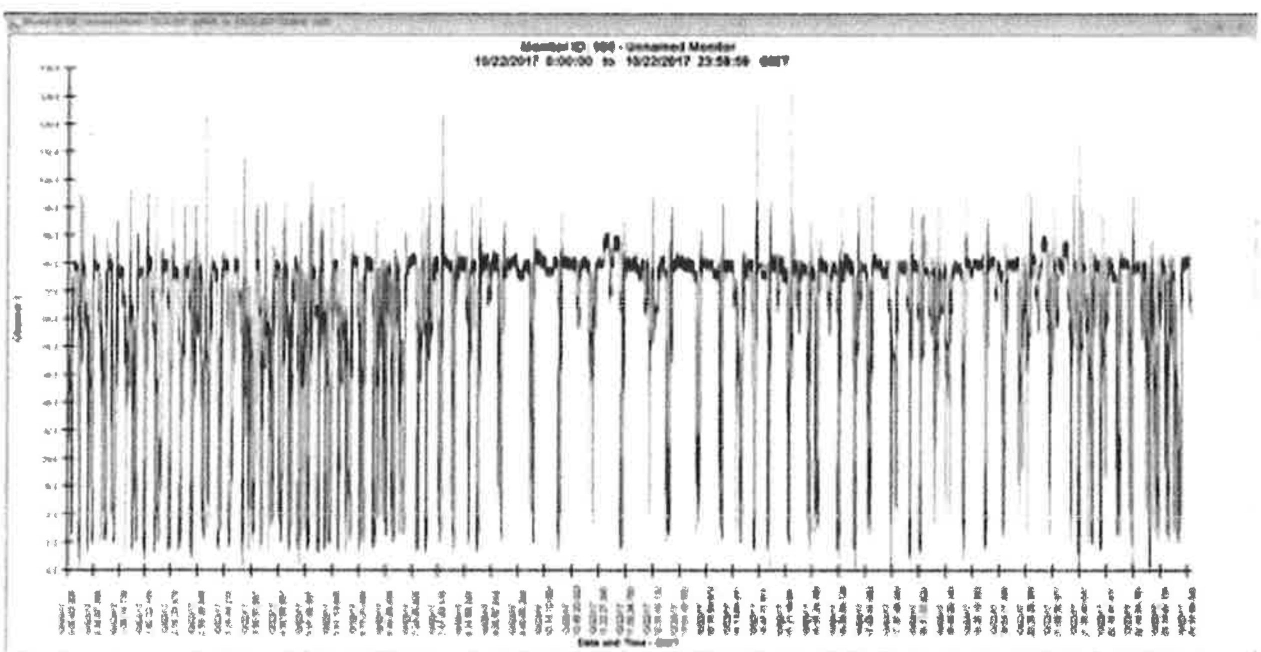


Figure 35: October 23, 2017

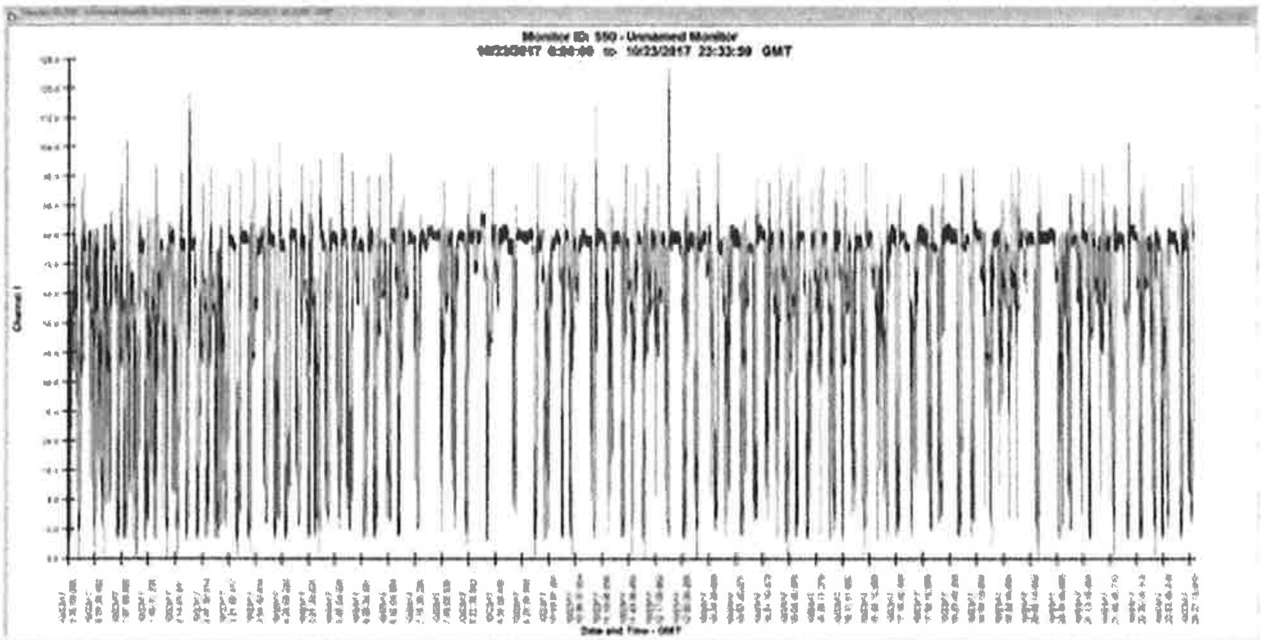


Figure 36: October 24, 2017

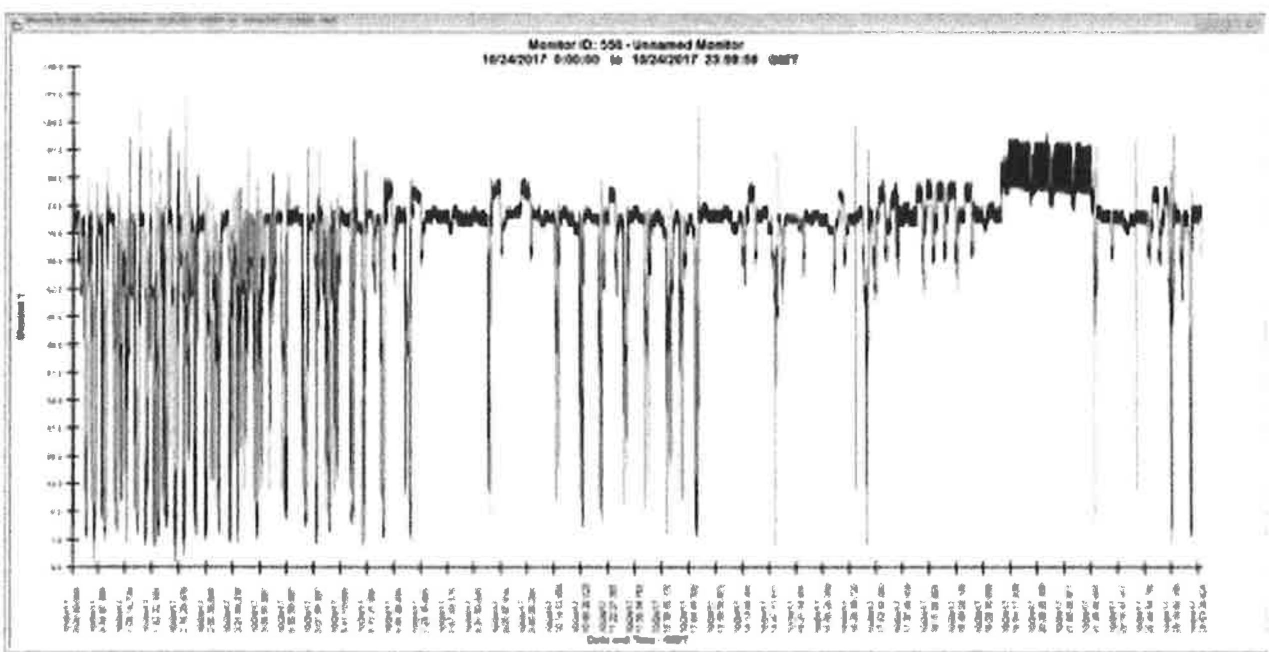
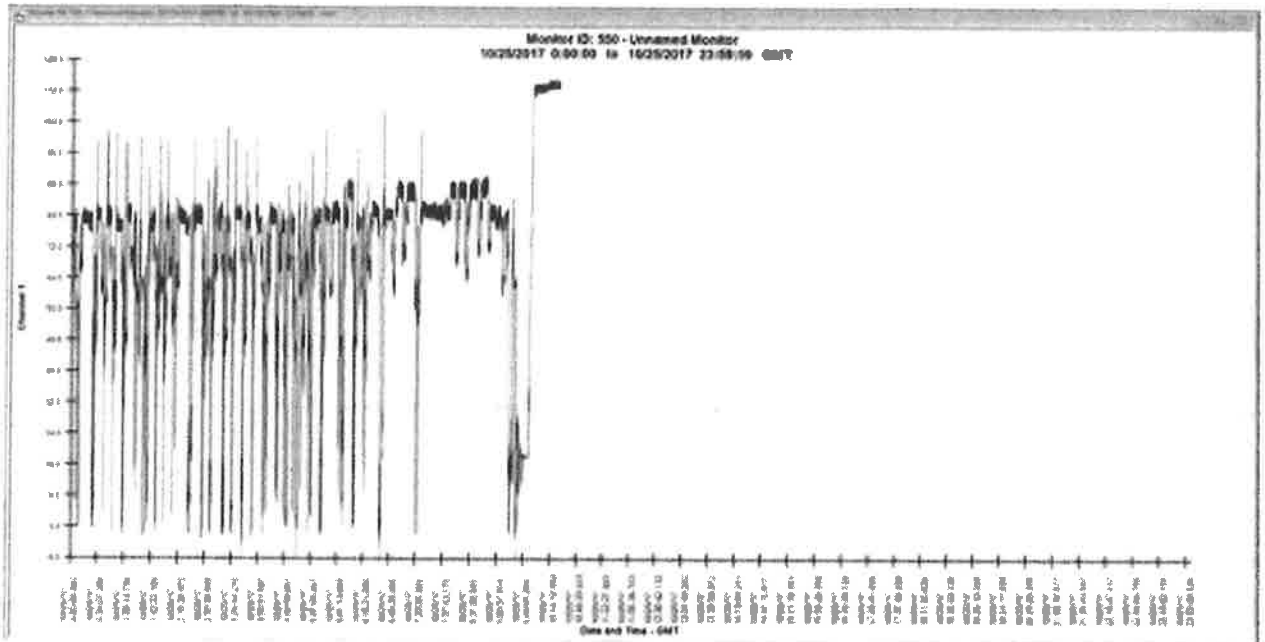


Figure 37: October 25, 2017



D

Appendix D – USDA NRCS Soil Survey

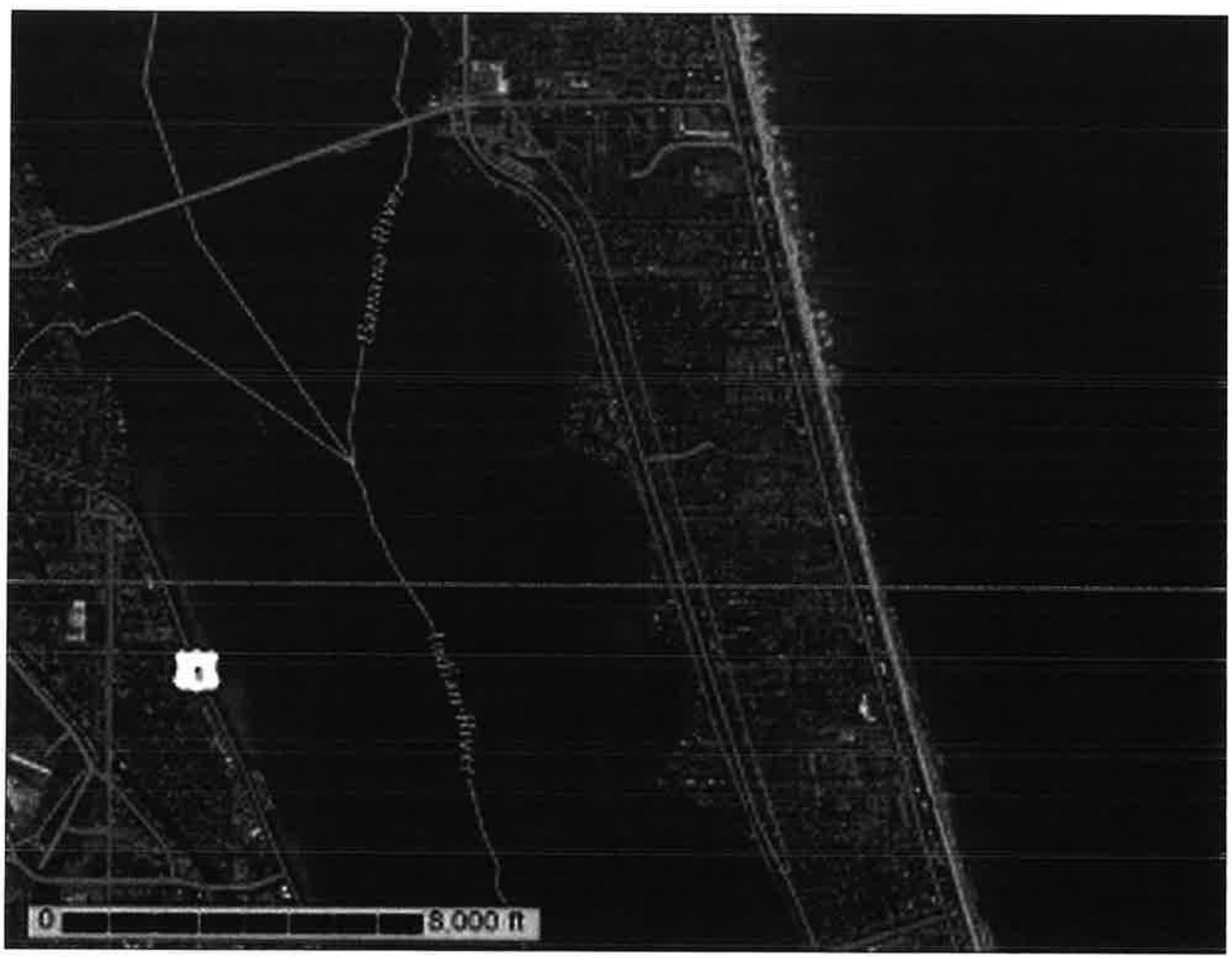
USDA United States Department of Agriculture

NRCS Natural Resources Conservation Service

A product of the National Cooperative Soil Survey, a joint effort of the United States Department of Agriculture and other Federal agencies, State agencies including the Agricultural Experiment Stations, and local participants

Custom Soil Resource Report for Brevard County, Florida

Riverside Drive Force Main



Preface

Soil surveys contain information that affects land use planning in survey areas. They highlight soil limitations that affect various land uses and provide information about the properties of the soils in the survey areas. Soil surveys are designed for many different users, including farmers, ranchers, foresters, agronomists, urban planners, community officials, engineers, developers, builders, and home buyers. Also, conservationists, teachers, students, and specialists in recreation, waste disposal, and pollution control can use the surveys to help them understand, protect, or enhance the environment.

Various land use regulations of Federal, State, and local governments may impose special restrictions on land use or land treatment. Soil surveys identify soil properties that are used in making various land use or land treatment decisions. The information is intended to help the land users identify and reduce the effects of soil limitations on various land uses. The landowner or user is responsible for identifying and complying with existing laws and regulations.

Although soil survey information can be used for general farm, local, and wider area planning, onsite investigation is needed to supplement this information in some cases. Examples include soil quality assessments (http://www.nrcs.usda.gov/wps/portal/nrcs/main/soils/health/) and certain conservation and engineering applications. For more detailed information, contact your local USDA Service Center (https://offices.sc.egov.usda.gov/locator/app?agency=nrcs) or your NRCS State Soil Scientist (http://www.nrcs.usda.gov/wps/portal/nrcs/detail/soils/contactus/?cid=nrcs142p2_053951).

Great differences in soil properties can occur within short distances. Some soils are seasonally wet or subject to flooding. Some are too unstable to be used as a foundation for buildings or roads. Clayey or wet soils are poorly suited to use as septic tank absorption fields. A high water table makes a soil poorly suited to basements or underground installations.

The National Cooperative Soil Survey is a joint effort of the United States Department of Agriculture and other Federal agencies, State agencies including the Agricultural Experiment Stations, and local agencies. The Natural Resources Conservation Service (NRCS) has leadership for the Federal part of the National Cooperative Soil Survey.

Information about soils is updated periodically. Updated information is available through the NRCS Web Soil Survey, the site for official soil survey information.

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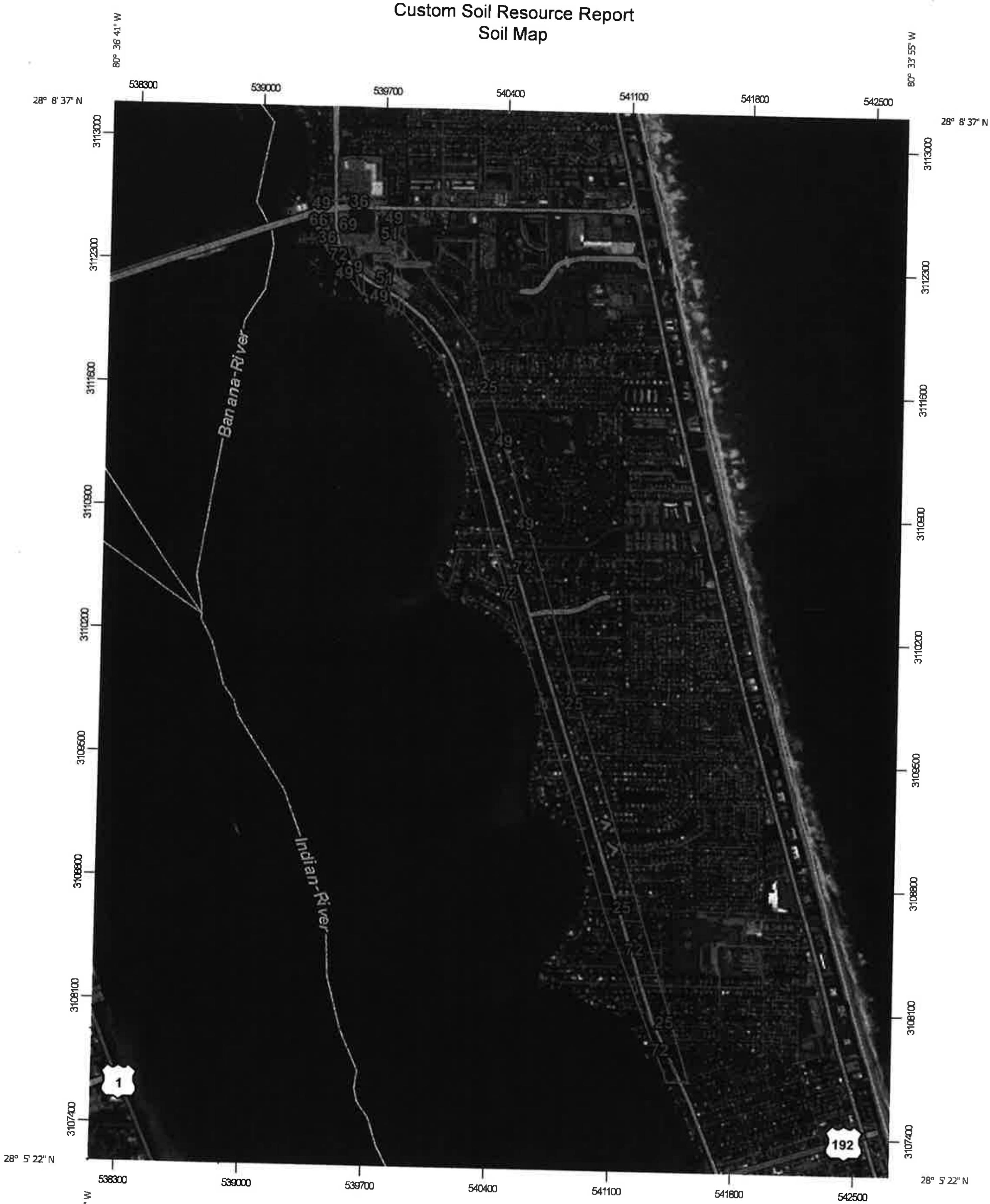
Contents

Soil Map

The soil map section includes the soil map for the defined area of interest, a list of soil map units on the map and extent of each map unit, and cartographic symbols displayed on the map. Also presented are various metadata about data used to produce the map, and a description of each soil map unit.

Custom Soil Resource Report
Soil Map



Map Scale: 1:29,300 if printed on A portrait (8.5" x 11") sheet.

N

0 400 800 1600 2400 Meters

0 1000 2000 4000 6000 Feet

Map projection: Web Mercator Corner coordinates: WGS84 Edge tics: UTM Zone 17N WGS84

MAP LEGEND

Area of Interest (AOI)
- Area of Interest (AOI)

Soils
- Soil Map Unit Polygons
- Soil Map Unit Lines
- Soil Map Unit Points

Special Point Features
- Blowout
- Borrow Pit
- Clay Spot
- Closed Depression
- Gravel Pit
- Gravelly Spot
- Landfill
- Lava Flow
- Marsh or swamp
- Mine or Quarry
- Miscellaneous Water
- Perennial Water
- Rock Outcrop
- Saline Spot
- Sandy Spot
- Severely Eroded Spot
- Sinkhole
- Slide or Slip
- Sodic Spot
- Spoil Area
- Stony Spot
- Very Stony Spot
- Wet Spot
- Other
- Special Line Features

Water Features
- Streams and Canals

Transportation
- Rails
- Interstate Highways
- US Routes
- Major Roads
- Local Roads

Background
- Aerial Photography

MAP INFORMATION

The soil surveys that comprise your AOI were mapped at 1:24,000.

Please rely on the bar scale on each map sheet for map measurements.

Source of Map: Natural Resources Conservation Service
Web Soil Survey URL:
Coordinate System: Web Mercator (EPSG:3857)

Maps from the Web Soil Survey are based on the Web Mercator projection, which preserves direction and shape but distorts distance and area. A projection that preserves area, such as the Albers equal-area conic projection, should be used if more accurate calculations of distance or area are required.

This product is generated from the USDA-NRCS certified data as of the version date(s) listed below.

Soil Survey Area: Brevard County, Florida
Survey Area Data: Version 16, Sep 26, 2017

Soil map units are labeled (as space allows) for map scales 1:50,000 or larger.

Date(s) aerial images were photographed: Mar 12, 2011—Mar 16, 2014

The orthophoto or other base map on which the soil lines were compiled and digitized probably differs from the background imagery displayed on these maps. As a result, some minor shifting of map unit boundaries may be evident.

Map Unit Legend

Map Unit Symbol	Map Unit Name	Acres in AOI	Percent of AOI
9	Canaveral-Anclote complex, gently undulating	2.3	0.8%
25	Canaveral-Palm Beach-Urban land complex	27.1	9.4%
36	Myakka sand, 0 to 2 percent slopes	1.6	0.6%
49	Pomello sand	18.2	6.3%
51	Pompano sand, 0 to 2 percent slopes	9.0	3.1%
66	Bessie muck, tidal	0.7	0.3%
69	Urban land	19.4	6.8%
72	Welaka sand	206.5	71.9%
100	Waters of the Atlantic Ocean	2.4	0.8%
Totals for Area of Interest		**287.3**	**100.0%**

Map Unit Descriptions

The map units delineated on the detailed soil maps in a soil survey represent the soils or miscellaneous areas in the survey area. The map unit descriptions, along with the maps, can be used to determine the composition and properties of a unit.

A map unit delineation on a soil map represents an area dominated by one or more major kinds of soil or miscellaneous areas. A map unit is identified and named according to the taxonomic classification of the dominant soils. Within a taxonomic class there are precisely defined limits for the properties of the soils. On the landscape, however, the soils are natural phenomena, and they have the characteristic variability of all natural phenomena. Thus, the range of some observed properties may extend beyond the limits defined for a taxonomic class. Areas of soils of a single taxonomic class rarely, if ever, can be mapped without including areas of other taxonomic classes. Consequently, every map unit is made up of the soils or miscellaneous areas for which it is named and some minor components that belong to taxonomic classes other than those of the major soils.

Most minor soils have properties similar to those of the dominant soil or soils in the map unit, and thus they do not affect use and management. These are called noncontrasting, or similar, components. They may or may not be mentioned in a particular map unit description. Other minor components, however, have properties and behavioral characteristics divergent enough to affect use or to require different management. These are called contrasting, or dissimilar, components. They generally are in small areas and could not be mapped separately because of the scale used. Some small areas of strongly contrasting soils or miscellaneous areas are identified by a special symbol on the maps. If included in the database for a given area, the contrasting minor components are identified in the map unit descriptions along with some characteristics of each. A few areas of minor components may not have been observed, and consequently they are not mentioned in the descriptions, especially where the pattern was so complex that it was impractical to make enough observations to identify all the soils and miscellaneous areas on the landscape.

The presence of minor components in a map unit in no way diminishes the usefulness or accuracy of the data. The objective of mapping is not to delineate pure taxonomic classes but rather to separate the landscape into landforms or landform segments that have similar use and management requirements. The delineation of such segments on the map provides sufficient information for the development of resource plans. If intensive use of small areas is planned, however, onsite investigation is needed to define and locate the soils and miscellaneous areas.

An identifying symbol precedes the map unit name in the map unit descriptions. Each description includes general facts about the unit and gives important soil properties and qualities.

Soils that have profiles that are almost alike make up a *soil series*. Except for differences in texture of the surface layer, all the soils of a series have major horizons that are similar in composition, thickness, and arrangement.

Soils of one series can differ in texture of the surface layer, slope, stoniness, salinity, degree of erosion, and other characteristics that affect their use. On the basis of such differences, a soil series is divided into *soil phases*. Most of the areas shown on the detailed soil maps are phases of soil series. The name of a soil phase commonly indicates a feature that affects use or management. For example, Alpha silt loam, 0 to 2 percent slopes, is a phase of the Alpha series.

Some map units are made up of two or more major soils or miscellaneous areas. These map units are complexes, associations, or undifferentiated groups.

A *complex* consists of two or more soils or miscellaneous areas in such an intricate pattern or in such small areas that they cannot be shown separately on the maps. The pattern and proportion of the soils or miscellaneous areas are somewhat similar in all areas. Alpha-Beta complex, 0 to 6 percent slopes, is an example.

An *association* is made up of two or more geographically associated soils or miscellaneous areas that are shown as one unit on the maps. Because of present or anticipated uses of the map units in the survey area, it was not considered practical or necessary to map the soils or miscellaneous areas separately. The pattern and relative proportion of the soils or miscellaneous areas are somewhat similar. Alpha-Beta association, 0 to 2 percent slopes, is an example.

An *undifferentiated group* is made up of two or more soils or miscellaneous areas that could be mapped individually but are mapped as one unit because similar interpretations can be made for use and management. The pattern and proportion of the soils or miscellaneous areas in a mapped area are not uniform. An area can be made up of only one of the major soils or miscellaneous areas, or it can be made up of all of them. Alpha and Beta soils, 0 to 2 percent slopes, is an example.

Some surveys include *miscellaneous areas*. Such areas have little or no soil material and support little or no vegetation. Rock outcrop is an example.

Brevard County, Florida

9—Canaveral-Anclote complex, gently undulating

Map Unit Setting

National map unit symbol: 1lg2n
Elevation: 10 to 60 feet
Mean annual precipitation: 49 to 57 inches
Mean annual air temperature: 68 to 75 degrees F
Frost-free period: 350 to 365 days
Farmland classification: Not prime farmland

Map Unit Composition

Canaveral and similar soils: 60 percent
Anclote and similar soils: 30 percent
Minor components: 10 percent
Estimates are based on observations, descriptions, and transects of the mapunit.

Description of Canaveral

Setting

Landform: Ridges on marine terraces, dunes on marine terraces
Landform position (three-dimensional): Interfluve
Down-slope shape: Convex
Across-slope shape: Linear
Parent material: Sandy marine deposits

Typical profile

A - 0 to 6 inches: sand
C - 6 to 12 inches: sand
C - 12 to 80 inches: coarse sand

Properties and qualities

Slope: 0 to 5 percent
Depth to restrictive feature: More than 80 inches
Natural drainage class: Somewhat poorly drained
Runoff class: Low
Capacity of the most limiting layer to transmit water (Ksat): Very high (19.98 to 50.02 in/hr)
Depth to water table: About 12 to 36 inches
Frequency of flooding: None
Frequency of ponding: None
Calcium carbonate, maximum in profile: 15 percent
Salinity, maximum in profile: Nonsaline to very slightly saline (0.0 to 2.0 mmhos/cm)
Sodium adsorption ratio, maximum in profile: 6.0
Available water storage in profile: Very low (about 1.4 inches)

Interpretive groups

Land capability classification (irrigated): None specified
Land capability classification (nonirrigated): 6s
Hydrologic Soil Group: A/D
Other vegetative classification: Forage suitability group not assigned (G156BC999FL)
Hydric soil rating: No

Description of Anclote

Setting

Landform: Flats on marine terraces
Landform position (three-dimensional): Talf
Down-slope shape: Linear
Across-slope shape: Linear
Parent material: Sandy marine deposits

Typical profile

A - 0 to 19 inches: sand
Cg - 19 to 72 inches: sand

Properties and qualities

Slope: 0 to 2 percent
Depth to restrictive feature: More than 80 inches
Natural drainage class: Very poorly drained
Runoff class: Very high
Capacity of the most limiting layer to transmit water (Ksat): High to very high (5.95 to 19.98 in/hr)
Depth to water table: About 0 to 6 inches
Frequency of flooding: None
Frequency of ponding: None
Salinity, maximum in profile: Nonsaline to very slightly saline (0.0 to 2.0 mmhos/cm)
Sodium adsorption ratio, maximum in profile: 4.0
Available water storage in profile: Low (about 5.3 inches)

Interpretive groups

Land capability classification (irrigated): None specified
Land capability classification (nonirrigated): 3w
Hydrologic Soil Group: A/D
Other vegetative classification: Sandy soils on flats of mesic or hydric lowlands (G156BC141FL)
Hydric soil rating: Yes

Minor Components

Pomello

Percent of map unit: 5 percent
Landform: Rises on marine terraces, flats on marine terraces
Landform position (three-dimensional): Interfluve
Down-slope shape: Convex
Across-slope shape: Linear
Other vegetative classification: Sand Pine Scrub (R155XY001FL), Sandy soils on rises and knolls of mesic uplands (G156BC131FL)
Hydric soil rating: No

Palm beach

Percent of map unit: 5 percent
Landform: Dunes on marine terraces
Landform position (three-dimensional): Interfluve
Down-slope shape: Convex
Across-slope shape: Linear
Other vegetative classification: Sandy soils on ridges and dunes of xeric uplands (G156BC111FL)

Hydric soil rating: No

25—Canaveral-Palm Beach-Urban land complex

Map Unit Setting

National map unit symbol: 1lg35
Elevation: 10 to 20 feet
Mean annual precipitation: 49 to 57 inches
Mean annual air temperature: 68 to 75 degrees F
Frost-free period: 350 to 365 days
Farmland classification: Not prime farmland

Map Unit Composition

Canaveral and similar soils: 31 percent
Palm beach and similar soils: 30 percent
Urban land: 29 percent
Minor components: 10 percent
Estimates are based on observations, descriptions, and transects of the mapunit.

Description of Canaveral

Setting

Landform: Flats on marine terraces, ridges on marine terraces
Landform position (three-dimensional): Interfluve
Down-slope shape: Convex
Across-slope shape: Linear
Parent material: Sandy marine deposits

Typical profile

A - 0 to 6 inches: sand
C - 6 to 12 inches: sand
C - 12 to 80 inches: coarse sand

Properties and qualities

Slope: 0 to 2 percent
Depth to restrictive feature: More than 80 inches
Natural drainage class: Somewhat poorly drained
Runoff class: Low
Capacity of the most limiting layer to transmit water (Ksat): Very high (19.98 to 50.02 in/hr)
Depth to water table: About 12 to 36 inches
Frequency of flooding: None
Frequency of ponding: None
Calcium carbonate, maximum in profile: 15 percent
Salinity, maximum in profile: Nonsaline to very slightly saline (0.0 to 2.0 mmhos/cm)
Sodium adsorption ratio, maximum in profile: 6.0
Available water storage in profile: Very low (about 1.4 inches)

Interpretive groups

Land capability classification (irrigated): None specified
Land capability classification (nonirrigated): 6s
Hydrologic Soil Group: A/D
Other vegetative classification: Forage suitability group not assigned (G156BC999FL)
Hydric soil rating: No

Description of Palm Beach

Setting

Landform: Flats on marine terraces
Landform position (three-dimensional): Talf
Down-slope shape: Convex
Across-slope shape: Linear
Parent material: Shells and sandy marine deposits

Typical profile

A - 0 to 3 inches: sand
C - 3 to 80 inches: sand

Properties and qualities

Slope: 0 to 2 percent
Depth to restrictive feature: More than 80 inches
Natural drainage class: Excessively drained
Runoff class: Negligible
Capacity of the most limiting layer to transmit water (Ksat): Very high (19.98 to 50.02 in/hr)
Depth to water table: More than 80 inches
Frequency of flooding: None
Frequency of ponding: None
Calcium carbonate, maximum in profile: 30 percent
Salinity, maximum in profile: Nonsaline to very slightly saline (0.0 to 2.0 mmhos/cm)
Sodium adsorption ratio, maximum in profile: 4.0
Available water storage in profile: Very low (about 1.8 inches)

Interpretive groups

Land capability classification (irrigated): None specified
Land capability classification (nonirrigated): 7s
Hydrologic Soil Group: A
Other vegetative classification: Forage suitability group not assigned (G156BC999FL)
Hydric soil rating: No

Description of Urban Land

Setting

Landform: Marine terraces
Landform position (three-dimensional): Interfluve, talf
Down-slope shape: Linear
Across-slope shape: Linear
Parent material: No parent material

Interpretive groups

Land capability classification (irrigated): None specified

Other vegetative classification: Forage suitability group not assigned (G156BC999FL)
Hydric soil rating: Unranked

Minor Components

Paola

Percent of map unit: 4 percent
Landform: Flats on marine terraces, rises on marine terraces
Landform position (three-dimensional): Interfluve
Down-slope shape: Convex
Across-slope shape: Linear
Other vegetative classification: Sand Pine Scrub (R155XY001FL), Forage suitability group not assigned (G156BC999FL)
Hydric soil rating: No

Pomello

Percent of map unit: 3 percent
Landform: Flats on marine terraces, rises on marine terraces
Landform position (three-dimensional): Interfluve
Down-slope shape: Convex
Across-slope shape: Linear
Other vegetative classification: Sand Pine Scrub (R155XY001FL), Forage suitability group not assigned (G156BC999FL)
Hydric soil rating: No

Welaka

Percent of map unit: 3 percent
Landform: Rises on marine terraces
Landform position (three-dimensional): Interfluve
Down-slope shape: Convex
Across-slope shape: Linear
Other vegetative classification: Sand Pine Scrub (R155XY001FL), Forage suitability group not assigned (G156BC999FL)
Hydric soil rating: No

36—Myakka sand, 0 to 2 percent slopes

Map Unit Setting

National map unit symbol: 2twt9
Elevation: 10 to 130 feet
Mean annual precipitation: 43 to 62 inches
Mean annual air temperature: 64 to 77 degrees F
Frost-free period: 280 to 365 days
Farmland classification: Farmland of unique importance

Map Unit Composition

Myakka and similar soils: 85 percent

Minor components: 15 percent
Estimates are based on observations, descriptions, and transects of the mapunit.

Description of Myakka

Setting

Landform: Flatwoods on marine terraces
Landform position (three-dimensional): Tread, talf
Down-slope shape: Convex
Across-slope shape: Linear
Parent material: Sandy marine deposits

Typical profile

A - 0 to 6 inches: sand
E - 6 to 20 inches: sand
Bh - 20 to 36 inches: sand
C - 36 to 80 inches: sand

Properties and qualities

Slope: 0 to 2 percent
Depth to restrictive feature: More than 80 inches
Natural drainage class: Poorly drained
Runoff class: High
Capacity of the most limiting layer to transmit water (Ksat): Moderately high to high (0.57 to 5.95 in/hr)
Depth to water table: About 6 to 18 inches
Frequency of flooding: None
Frequency of ponding: None
Salinity, maximum in profile: Nonsaline to very slightly saline (0.0 to 2.0 mmhos/cm)
Sodium adsorption ratio, maximum in profile: 4.0
Available water storage in profile: Low (about 5.0 inches)

Interpretive groups

Land capability classification (irrigated): None specified
Land capability classification (nonirrigated): 4w
Hydrologic Soil Group: A/D
Other vegetative classification: South Florida Flatwoods (R155XY003FL), Sandy soils on flats of mesic or hydric lowlands (G155XB141FL)
Hydric soil rating: No

Minor Components

Basinger

Percent of map unit: 5 percent
Landform: Drainageways on marine terraces
Landform position (three-dimensional): Dip
Down-slope shape: Convex, linear
Across-slope shape: Linear, concave
Other vegetative classification: Sandy soils on flats of mesic or hydric lowlands (G155XB141FL)
Hydric soil rating: Yes

Oldsmar

Percent of map unit: 5 percent
Landform: Flatwoods on marine terraces
Landform position (three-dimensional): Talf

Down-slope shape: Convex
Across-slope shape: Linear
Other vegetative classification: South Florida Flatwoods (R155XY003FL), Sandy soils on flats of mesic or hydric lowlands (G155XB141FL)
Hydric soil rating: No

Valkaria

Percent of map unit: 5 percent
Landform: Drainageways on marine terraces
Landform position (three-dimensional): Dip
Down-slope shape: Convex, linear
Across-slope shape: Linear, concave
Other vegetative classification: Slough (R155XY011FL), Sandy soils on flats of mesic or hydric lowlands (G155XB141FL)
Hydric soil rating: Yes

49—Pomello sand

Map Unit Setting

National map unit symbol: 1lg3y
Mean annual precipitation: 49 to 57 inches
Mean annual air temperature: 68 to 75 degrees F
Frost-free period: 350 to 365 days
Farmland classification: Not prime farmland

Map Unit Composition

Pomello and similar soils: 90 percent
Minor components: 10 percent
Estimates are based on observations, descriptions, and transects of the mapunit.

Description of Pomello

Setting

Landform: Rises on marine terraces, flats on marine terraces
Landform position (three-dimensional): Interfluve
Down-slope shape: Convex
Across-slope shape: Linear
Parent material: Sandy marine deposits

Typical profile

A - 0 to 3 inches: sand
E - 3 to 50 inches: sand
Bh - 50 to 62 inches: sand
Cg - 62 to 80 inches: sand

Properties and qualities

Slope: 0 to 2 percent
Depth to restrictive feature: More than 80 inches
Natural drainage class: Moderately well drained
Runoff class: Negligible

Capacity of the most limiting layer to transmit water (Ksat): High (1.98 to 5.95 in/hr)
Depth to water table: About 24 to 42 inches
Frequency of flooding: None
Frequency of ponding: None
Salinity, maximum in profile: Nonsaline to very slightly saline (0.0 to 2.0 mmhos/cm)
Sodium adsorption ratio, maximum in profile: 4.0
Available water storage in profile: Low (about 3.4 inches)

Interpretive groups

Land capability classification (irrigated): None specified
Land capability classification (nonirrigated): 6s
Hydrologic Soil Group: A
Other vegetative classification: Sand Pine Scrub (R155XY001FL), Sandy soils on rises and knolls of mesic uplands (G156BC131FL)
Hydric soil rating: No

Minor Components

Myakka

Percent of map unit: 5 percent
Landform: Flats on marine terraces
Landform position (three-dimensional): Talf
Down-slope shape: Convex
Across-slope shape: Linear
Other vegetative classification: South Florida Flatwoods (R155XY003FL), Sandy soils on flats of mesic or hydric lowlands (G156BC141FL)
Hydric soil rating: No

Immokalee

Percent of map unit: 5 percent
Landform: Flats on marine terraces
Landform position (three-dimensional): Talf
Down-slope shape: Convex
Across-slope shape: Linear
Other vegetative classification: South Florida Flatwoods (R155XY003FL), Sandy soils on flats of mesic or hydric lowlands (G156BC141FL)
Hydric soil rating: No

51—Pompano sand, 0 to 2 percent slopes

Map Unit Setting

National map unit symbol: 2tzw4
Elevation: 0 to 40 feet
Mean annual precipitation: 44 to 58 inches
Mean annual air temperature: 68 to 77 degrees F
Frost-free period: 350 to 365 days
Farmland classification: Not prime farmland

Map Unit Composition

Pompano and similar soils: 82 percent
Minor components: 18 percent
Estimates are based on observations, descriptions, and transects of the mapunit.

Description of Pompano

Setting

Landform: Drainageways on flatwoods on marine terraces
Landform position (three-dimensional): Tread, dip, talf
Down-slope shape: Linear
Across-slope shape: Linear, concave
Parent material: Sandy marine deposits

Typical profile

A - 0 to 6 inches: sand
C - 6 to 80 inches: sand

Properties and qualities

Slope: 0 to 2 percent
Depth to restrictive feature: More than 80 inches
Natural drainage class: Poorly drained
Runoff class: Very high
Capacity of the most limiting layer to transmit water (Ksat): High to very high (6.00 to 20.00 in/hr)
Depth to water table: About 0 to 12 inches
Frequency of flooding: None
Frequency of ponding: None
Salinity, maximum in profile: Nonsaline to very slightly saline (0.0 to 2.0 mmhos/cm)
Sodium adsorption ratio, maximum in profile: 4.0
Available water storage in profile: Very low (about 2.4 inches)

Interpretive groups

Land capability classification (irrigated): None specified
Land capability classification (nonirrigated): 4w
Hydrologic Soil Group: A/D
Other vegetative classification: Slough (R155XY011FL), Sandy soils on flats of mesic or hydric lowlands (G155XB141FL)
Hydric soil rating: Yes

Minor Components

Myakka

Percent of map unit: 8 percent
Landform: Drainageways on flatwoods on marine terraces
Landform position (three-dimensional): Tread, dip, talf
Down-slope shape: Linear
Across-slope shape: Concave, linear
Other vegetative classification: South Florida Flatwoods (R155XY003FL), Sandy soils on flats of mesic or hydric lowlands (G155XB141FL)
Hydric soil rating: No

Holopaw

Percent of map unit: 4 percent
Landform: — error in exists on —

Landform position (three-dimensional): Tread, dip, talf
Down-slope shape: Convex, concave, linear
Across-slope shape: Linear, concave
Other vegetative classification: Slough (R155XY011FL), Sandy soils on flats of mesic or hydric lowlands (G155XB141FL)
Hydric soil rating: No

Hallandale

Percent of map unit: 4 percent
Landform: Flatwoods on marine terraces
Landform position (three-dimensional): Tread, talf
Down-slope shape: Linear
Across-slope shape: Linear
Other vegetative classification: South Florida Flatwoods (R155XY003FL), Sandy soils on flats of mesic or hydric lowlands (G155XB141FL)
Hydric soil rating: Yes

Samsula

Percent of map unit: 2 percent
Landform: Depressions on marine terraces
Landform position (three-dimensional): Tread, dip
Down-slope shape: Concave
Across-slope shape: Concave
Other vegetative classification: Freshwater Marshes and Ponds (R155XY010FL), Organic soils in depressions and on flood plains (G155XB645FL)
Hydric soil rating: Yes

66—Bessie muck, tidal

Map Unit Setting

National map unit symbol: 1lg4g
Mean annual precipitation: 49 to 57 inches
Mean annual air temperature: 68 to 75 degrees F
Frost-free period: 350 to 365 days
Farmland classification: Not prime farmland

Map Unit Composition

Bessie, tidal, and similar soils: 100 percent
Estimates are based on observations, descriptions, and transects of the mapunit.

Description of Bessie, Tidal

Setting

Landform: Tidal marshes on marine terraces
Landform position (three-dimensional): Dip
Down-slope shape: Linear
Across-slope shape: Concave
Parent material: Organic material over clayey and sandy marine deposits

Typical profile

Oa - 0 to 18 inches: muck

2Cg - 18 to 44 inches: sandy clay
3Cg - 44 to 80 inches: loamy fine sand

Properties and qualities

Slope: 0 to 1 percent
Depth to restrictive feature: More than 80 inches
Natural drainage class: Very poorly drained
Runoff class: Very high
Capacity of the most limiting layer to transmit water (Ksat): Moderately low to moderately high (0.06 to 0.20 in/hr)
Depth to water table: About 0 inches
Frequency of flooding: Very frequent
Frequency of ponding: Frequent
Calcium carbonate, maximum in profile: 20 percent
Salinity, maximum in profile: Strongly saline (16.0 to 32.0 mmhos/cm)
Sodium adsorption ratio, maximum in profile: 60.0
Available water storage in profile: High (about 11.1 inches)

Interpretive groups

Land capability classification (irrigated): None specified
Land capability classification (nonirrigated): 8
Hydrologic Soil Group: C/D
Other vegetative classification: Forage suitability group not assigned (G156BC999FL)
Hydric soil rating: Yes

69—Urban land

Map Unit Composition

Urban land: 100 percent
Estimates are based on observations, descriptions, and transects of the mapunit.

Description of Urban Land

Setting

Landform: Marine terraces
Landform position (three-dimensional): Interfluve, talf
Down-slope shape: Linear
Across-slope shape: Linear
Parent material: No parent material

Interpretive groups

Land capability classification (irrigated): None specified
Other vegetative classification: Forage suitability group not assigned (G156BC999FL)
Hydric soil rating: Unranked

72—Welaka sand

Map Unit Setting

National map unit symbol: 1lg4n
Mean annual precipitation: 49 to 57 inches
Mean annual air temperature: 68 to 75 degrees F
Frost-free period: 350 to 365 days
Farmland classification: Not prime farmland

Map Unit Composition

Welaka and similar soils: 90 percent
Minor components: 10 percent
Estimates are based on observations, descriptions, and transects of the mapunit.

Description of Welaka

Setting

Landform: Rises on marine terraces
Landform position (three-dimensional): Interfluve
Down-slope shape: Convex
Across-slope shape: Linear
Parent material: Sandy marine deposits

Typical profile

A - 0 to 3 inches: sand
E - 3 to 18 inches: sand
Bw - 18 to 55 inches: sand
2C - 55 to 80 inches: gravelly fine sand

Properties and qualities

Slope: 0 to 2 percent
Depth to restrictive feature: More than 80 inches
Natural drainage class: Well drained
Runoff class: Negligible
Capacity of the most limiting layer to transmit water (Ksat): Very high (19.98 to 50.02 in/hr)
Depth to water table: More than 80 inches
Frequency of flooding: None
Frequency of ponding: None
Calcium carbonate, maximum in profile: 5 percent
Salinity, maximum in profile: Nonsaline to very slightly saline (0.0 to 2.0 mmhos/cm)
Sodium adsorption ratio, maximum in profile: 4.0
Available water storage in profile: Very low (about 2.0 inches)

Interpretive groups

Land capability classification (irrigated): None specified
Land capability classification (nonirrigated): 6s
Hydrologic Soil Group: A
Other vegetative classification: Sand Pine Scrub (R155XY001FL), Sandy soils on ridges and dunes of xeric uplands (G156BC111FL)
Hydric soil rating: No

Minor Components

Pomello

Percent of map unit: 5 percent
Landform: Flats on marine terraces, rises on marine terraces
Landform position (three-dimensional): Interfluve
Down-slope shape: Convex
Across-slope shape: Linear
Other vegetative classification: Sand Pine Scrub (R155XY001FL), Sandy soils on rises and knolls of mesic uplands (G156BC131FL)
Hydric soil rating: No

Paola

Percent of map unit: 5 percent
Landform: Flats on marine terraces, rises on marine terraces
Landform position (three-dimensional): Interfluve
Down-slope shape: Convex
Across-slope shape: Linear
Other vegetative classification: Sand Pine Scrub (R155XY001FL), Sandy soils on ridges and dunes of xeric uplands (G156BC111FL)
Hydric soil rating: No

100—Waters of the Atlantic Ocean

Map Unit Composition

Waters of the atlantic ocean: 100 percent
Estimates are based on observations, descriptions, and transects of the mapunit.

Description of Waters Of The Atlantic Ocean

Interpretive groups

Land capability classification (irrigated): None specified
Other vegetative classification: Forage suitability group not assigned (G156BC999FL)
Hydric soil rating: Unranked

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Table 2: System Analysis (High Head Scenarios)

Scenario	Pump Station B01		Pump Station B19			Pump Station B20			Pump Station B06		Pump Station B07		20" Interconnect
	Flow (gpm)	TDH (ft)	Flow (gpm)	TDH (ft)	No. Pumps On	Flow (gpm)	TDH (ft)	No. Pumps On	Flow (gpm)	TDH (ft)	Flow (gpm)	TDH (ft)	Flow (gpm)
Exist. 24" PVC, No Interconnect (1)	800	191	1915	241	2	3029	224	2	1940	79	1375	66	NA
Exist. 24" PVC, No Interconnect (2)	1000	195	1873	243	2	2945	226	2	1940	79	1375	66	NA
Exist. 24" PVC, No Interconnect (3)	800	218	2000	276	VFD	3500	259	VFD	1940	79	1375	66	NA
Exist. 24" PVC, No Interconnect (4)	1000	228	2000	286	VFD	3500	269	VFD	1940	79	1375	66	NA
30" Upsize, No Interconnect (1)	800	186	2190	238	2	3377	215	2	1940	79	1375	66	NA
30" Upsize, No Interconnect (2)	1000	189	2102	238	2	3354	218	2	1940	79	1375	66	NA
30" Upsize, 20" Interconnect N River Side Dr (1)	800	138	2309	193	1	3212	167	1	1940	119	1375	83	1480
30" Upsize, 20" Interconnect N River Side Dr (2)	1000	141	2328	194	1	3093	168	1	1940	120	1375	83	1526
30" Upsize, 20" Interconnect Second Ave (1)	800	143	2290	196	1	3078	170	1	1940	112	1375	81	1317
30" Upsize, 20" Interconnect Second Ave (2)	1000	145	2269	196	1	2981	171	1	1940	113	1375	81	1353

Notes, relevant notes apply to Low Head Condition Table as well:

1. The scenarios above noted as "Exist. 24" PVC" include the existing 24-inch PVC force main within the B01/B19/B20 system.
2. The Non-Interconnected system is based on condition where there is no connection between B01, B19 and B20 force main system and the B06 and B07 force main system.
3. In scenarios presented above noted as "30" Upsize", the existing 24-inch PVC force main of the B19, B20 and B01 force main system has been replaced with the 30-inch ductile iron force main.
4. Two interconnect systems were analyzed. One scenario includes a 1,200 feet long, 20-inch force main which connects the B01/B19/B20 system's new 30-inch force main on N River Dr at Oakland Ave to the existing 20-inch force main at the B06 pump station at 2nd Ave and N River Dr. The second interconnect system analyzed includes a 680 feet long, 20-inch force main which connects the B01/B19/B20 system's existing 24-inch force main on N Shannon Ave to the existing 20-inch force main downstream of B06 at 2nd Ave and N Shannon Ave.
5. Model was set up such that B06 pump station contributes 1,940 gpm regardless of head condition. Likewise, B07 pump station was set to contribute 1,375 gpm regardless of head condition. There were no asbuilts for the B06 and B07 pump stations, modeling the stations operating at design flow provides for conservative head conditions. B19 and B20 pump stations were allowed to operate on their curves; both the asbuilts and pump curves were acquired and incorporated into the model.
6. When the systems are interconnected and the other large stations (B01/B19/B20) are operating, head conditions at B06 and B07 pump stations are such that they would not pump their set operational conditions of 1,940 gpm and 1,375 gpm, respectively. This can be verified by comparing the head condition noted in the table above for B06 and B07 and comparing them to their pump curves.
7. Static head for B01 is calculated to the low water level (as noted in the asbuilt) which is 4.5 ft above the bottom elevation -15.46. Static head for B19 is calculated to the low water level (as noted in the asbuilt) which is 4.0 ft above the bottom elevation -15.20. Static head for B20 is calculated to the low water level (as noted in the asbuilt) which is 4.4 ft above the bottom elevation -14.60.
8. The total dynamic head (TDH) noted above for B06 and B07 are the low water levels which, for both, was approximated as that of B01 (4.5 feet above the bottom elevation of -15.46) since asbuilt information was not available for these stations.
9. The flow (gpm) value noted in the "20" Interconnect" column in the above table, if positive, notes flow from the B01/B19/B20 force main system into the B06/B07 force main system. If the flow value is negative, the flow is in the opposite direction.
10. Highest head condition check where stations B19 and B20 are set to pump their desired operating flow of 2,000 gpm and 3,500 gpm, respectively, would not occur under existing conditions due to the limited pump/head capacity of the existing pumps at B19 and B20 which have a shut off head of 275 ft. The scenario values are only shown for comparison purposes.

Table 3: System Analysis (Low Head Scenarios)

Scenario	Pump Station B01		Pump Station B19		20" Interconnect
	Flow (gpm)	TDH (ft)	Flow (gpm)	TDH (ft)	Flow (gpm)
Exist. 24" PVC, No Interconnect	800	87	2000	115	NA
30" Upsize, No Interconnect	800	79	2000	105	NA
30" Upsize, 20" Interconnect N River Side Dr	800	62	2000	88	1232

Notes:

1. Low Head Scenarios, only stations contributing flow to the force main system is B01 and B19.
2. B19 was restricted to 2,000 gpm of contributing flow in the above low head scenarios, imitating VFD setting.
3. See also relevant notes for Table 2.

Table 2: System Analysis (High Head Scenarios)

Scenario	Pump Station B01		Pump Station B19			Pump Station B20			Pump Station B06		Pump Station B07		20" Interconnect
	Flow (gpm)	TDH (ft)	Flow (gpm)	TDH (ft)	No. Pumps On	Flow (gpm)	TDH (ft)	No. Pumps On	Flow (gpm)	TDH (ft)	Flow (gpm)	TDH (ft)	Flow (gpm)
Exist. 24" PVC, No Interconnect (1)	800	191	1915	241	2	3029	224	2	1940	79	1375	66	NA
Exist. 24" PVC, No Interconnect (2)	1000	195	1873	243	2	2945	226	2	1940	79	1375	66	NA
Exist. 24" PVC, No Interconnect (3)	800	218	2000	276	VFD	3500	259	VFD	1940	79	1375	66	NA
Exist. 24" PVC, No Interconnect (4)	1000	228	2000	286	VFD	3500	269	VFD	1940	79	1375	66	NA
30" Upsize, No Interconnect (1)	800	186	2190	238	2	3377	215	2	1940	79	1375	66	NA
30" Upsize, No Interconnect (2)	1000	189	2102	238	2	3354	218	2	1940	79	1375	66	NA
30" Upsize, 20" Interconnect N River Side Dr (1)	800	138	2309	193	1	3212	167	1	1940	119	1375	83	1480
30" Upsize, 20" Interconnect N River Side Dr (2)	1000	141	2328	194	1	3093	168	1	1940	120	1375	83	1526
30" Upsize, 20" Interconnect Second Ave (1)	800	143	2290	196	1	3078	170	1	1940	112	1375	81	1317
30" Upsize, 20" Interconnect Second Ave (2)	1000	145	2269	196	1	2981	171	1	1940	113	1375	81	1353

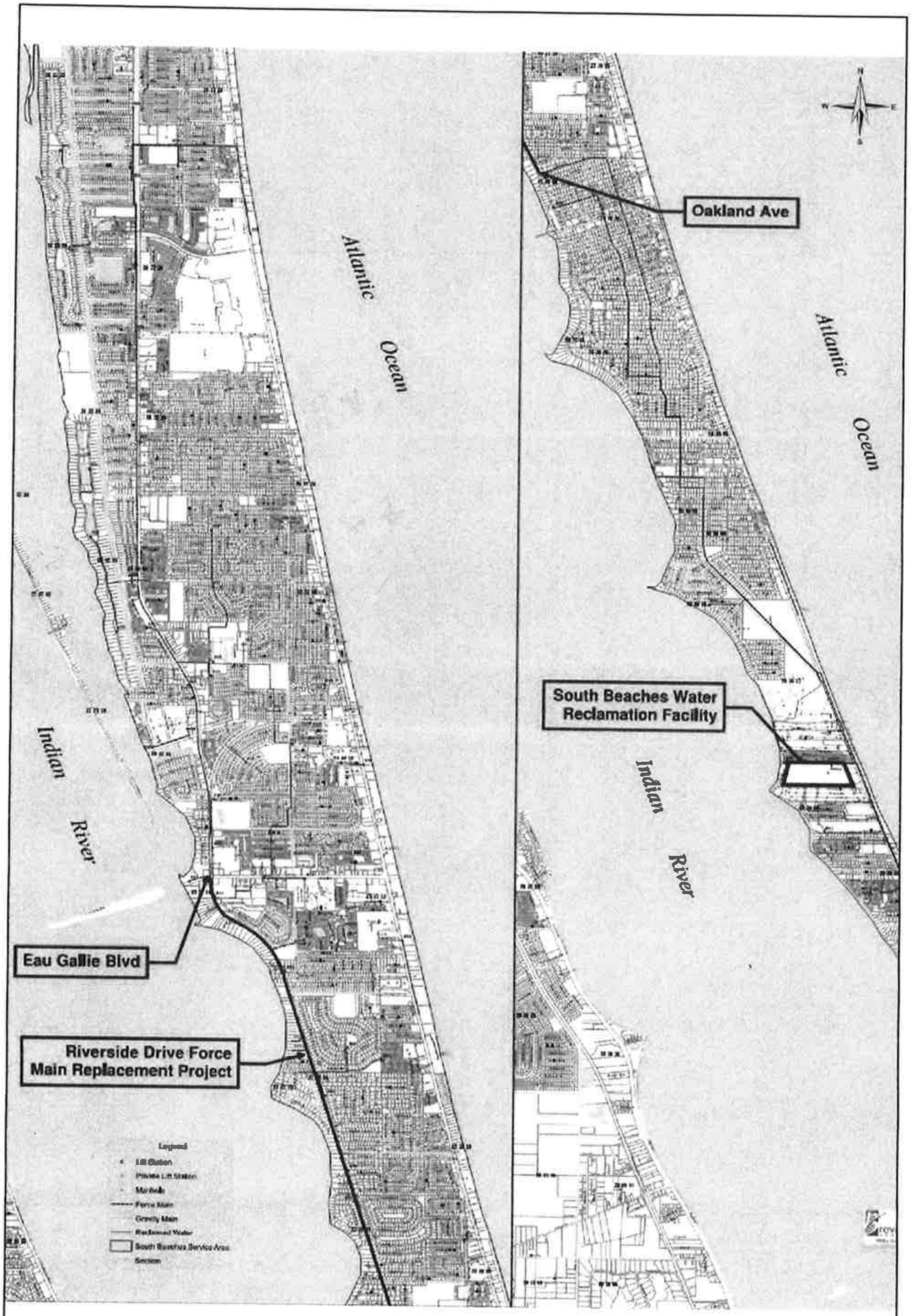
Notes, relevant notes apply to Low Head Condition Table as well:

1. The scenarios above noted as "Exist. 24" PVC" include the existing 24-inch PVC force main within the B01/B19/B20 system.
2. The Non-Interconnected system is based on condition where there is no connection between B01, B19 and B20 force main system and the B06 and B07 force main system.
3. In scenarios presented above noted as "30" Upsize", the existing 24-inch PVC force main of the B19, B20 and B01 force main system has been replaced with the 30-inch ductile iron force main.
4. Two interconnect systems were analyzed. One scenario includes a 1,200 feet long, 20-inch force main which connects the B01/B19/B20 system's new 30-inch force main on N River Dr at Oakland Ave to the existing 20-inch force main at the B06 pump station at 2nd Ave and N River Dr. The second interconnect system analyzed includes a 680 feet long, 20-inch force main which connects the B01/B19/B20 system's existing 24-inch force main on N Shannon Ave to the existing 20-inch force main downstream of B06 at 2nd Ave and N Shannon Ave.
5. Model was set up such that B06 pump station contributes 1,940 gpm regardless of head condition. Likewise, B07 pump station was set to contribute 1,375 gpm regardless of head condition. There were no asbuilts for the B06 and B07 pump stations, modeling the stations operating at design flow provides for conservative head conditions. B19 and B20 pump stations were allowed to operate on their curves; both the asbuilts and pump curves were acquired and incorporated into the model.
6. When the systems are interconnected and the other large stations (B01/B19/B20) are operating, head conditions at B06 and B07 pump stations are such that they would not pump their set operational conditions of 1,940 gpm and 1,375 gpm, respectively. This can be verified by comparing the head condition noted in the table above for B06 and B07 and comparing them to their pump curves.
7. Static head for B01 is calculated to the low water level (as noted in the asbuilt) which is 4.5 ft above the bottom elevation -15.46. Static head for B19 is calculated to the low water level (as noted in the asbuilt) which is 4.0 ft above the bottom elevation -15.20. Static head for B20 is calculated to the low water level (as noted in the asbuilt) which is 4.4 ft above the bottom elevation -14.60.
8. The total dynamic head (TDH) noted above for B06 and B07 are the low water levels which, for both, was approximated as that of B01 (4.5 feet above the bottom elevation of -15.46) since asbuilt information was not available for these stations.
9. The flow (gpm) value noted in the "20" Interconnect" column in the above table, if positive, notes flow from the B01/B19/B20 force main system into the B06/B07 force main system. If the flow value is negative, the flow is in the opposite direction.
10. Highest head condition check where stations B19 and B20 are set to pump their desired operating flow of 2,000 gpm and 3,500 gpm, respectively, would not occur under existing conditions due to the limited pump/head capacity of the existing pumps at B19 and B20 which have a shut off head of 275 ft. The scenario values are only shown for comparison purposes.

Table 3: System Analysis (Low Head Scenarios)

Scenario	Pump Station B01		Pump Station B19		20" Interconnect
	Flow (gpm)	TDH (ft)	Flow (gpm)	TDH (ft)	Flow (gpm)
Exist. 24" PVC, No Interconnect	800	87	2000	115	NA
30" Upsize, No Interconnect	800	79	2000	105	NA
30" Upsize, 20" Interconnect N River Side Dr	800	62	2000	88	1232

Notes:
1. Low Head Scenarios, only stations contributing flow to the force main system is B01 and B19.
2. B19 was restricted to 2,000 gpm of contributing flow in the above low head scenarios, imitating VFD setting.
3. See also relevant notes for Table 2.



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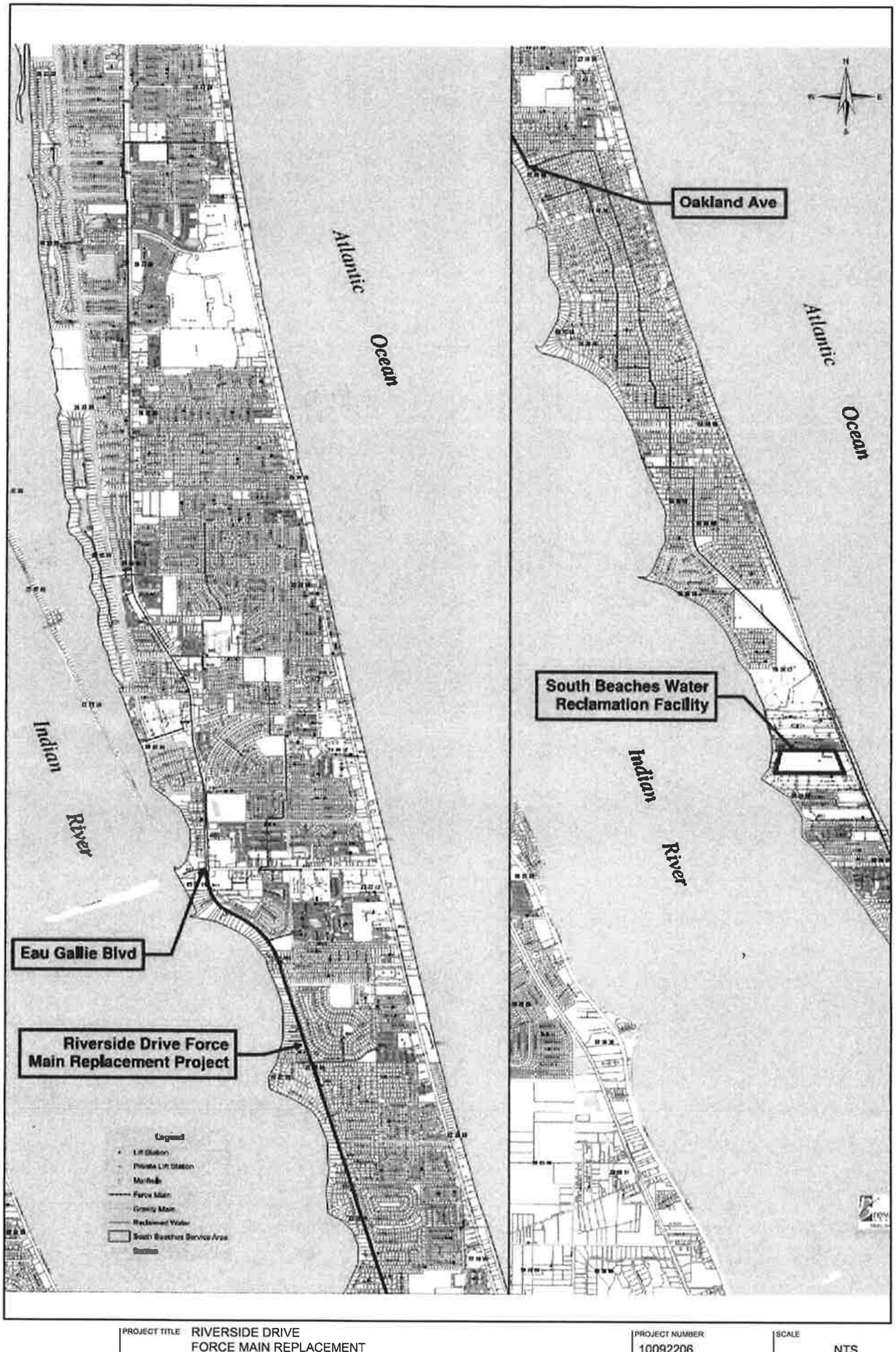
PROJECT TITLE	PROJECT NUMBER	SCALE
RIVERSIDE DRIVE FORCE MAIN REPLACEMENT	10092206	NTS
SHEET TITLE	PROJECT MANAGER	REFERENCE DOCUMENT
FIGURE:1-2 SOUTH BEACHES EXISTING SERVICES MAP	H.HARDY	-
	DATE	EXHIBIT NUMBER



PROJECT TITLE	RIVERSIDE DRIVE FORCE MAIN REPLACEMENT	PROJECT NUMBER 10092206	SCALE NTS
SHEET TITLE	FIGURE:1-2 SOUTH BEACHES EXISTING SERVICES MAP	PROJECT MANAGER H.HARDY	REFERENCE DOCUMENT -
		DATE	EXHIBIT NUMBER

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