Draft Lee County Southeast Advanced Water Reclamation Facility Project Definition Report

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Lee County

Lee County Southeast Advanced Water Reclamation Facility
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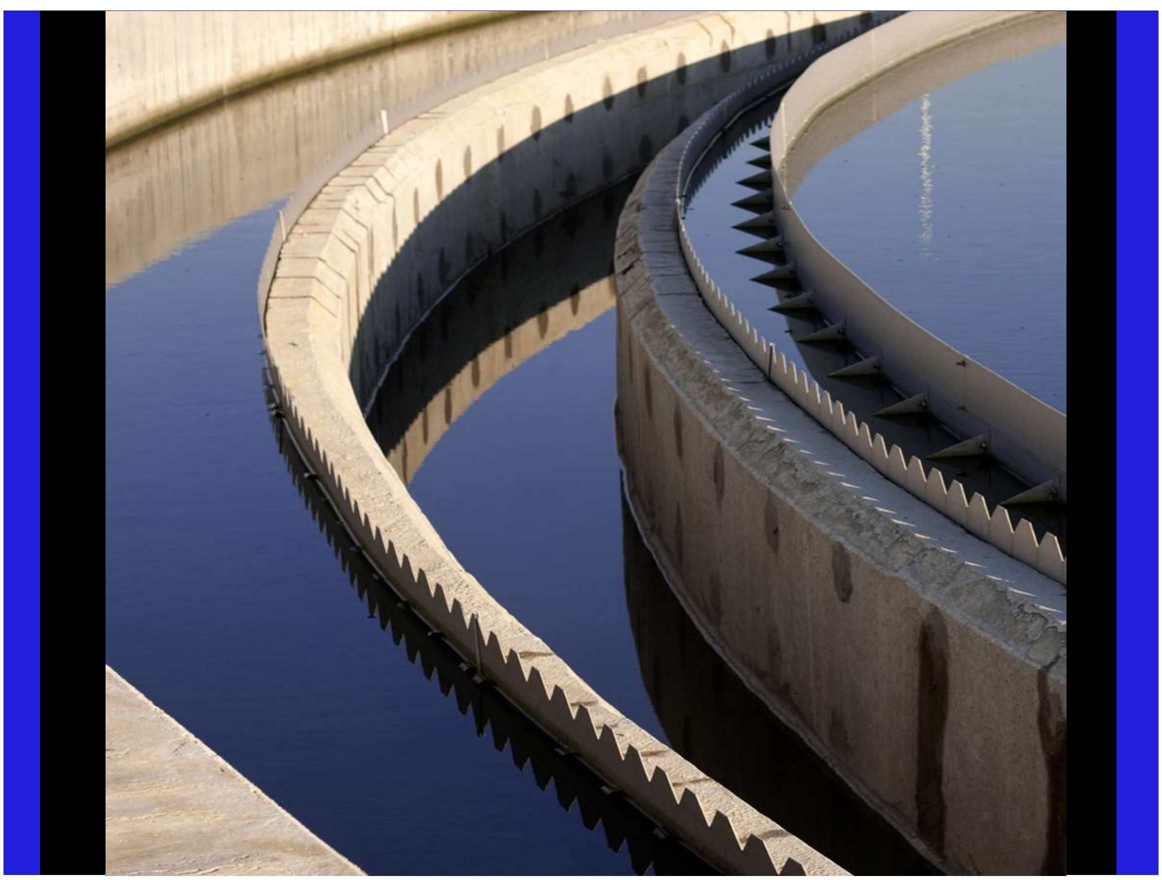

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Acronyms and Abbreviations

°C	degree(s) Celsius
AA	average annual
AAD	average annual day
AADF	annual average daily flow
AADL	annual average day load
ach	air change(s) per hour
AOR	actual oxygen requirement
AWT	advanced wastewater treatment
AWTF	advanced water treatment facility
BFP	belt filter press
bls	below land surface
BMAP	basin management action plan
BOD_5	5-day biochemical oxygen demand
$CBOD_5$	5-day carbonaceous biochemical oxygen demand
CCB	chlorine contact basin
Cl_2	chlorine
CMAR	construction management at-risk
CT	concentration time
DBP	disinfection byproducts
D/IPR	direct or indirect potable reuse
DIW	deep injection well
DO	dissolved oxygen
DPR	direct potable reuse
DR/GR	Density Reduction/Groundwater Resource
EA	each
EPA	U.S. Environmental Protection Agency
ERP	Environmental Resource Permit
FAC	*Florida Administrative Code*
FAS	Floridan Aquifer System

FDEP	Florida Department of Environmental Protection
FDOT	Florida Department of Transportation
FEMA	Federal Emergency Management Agency
FRP	fiberglass reinforced plastic
FS	Florida Statutes
g	gallon(s)
g/m^2	gallon(s) per square meter
gpd	gallon(s) per day
gph	gallon(s) per hour
gpm	gallon(s) per minute
gpm/m	gallon(s) per minute per meter
H_2S	hydrogen sulfide
HLD	high-level disinfection
HLR	hydraulic loading rate
HMI	human-machine interface
hp	horsepower
HRT	hydraulic residence time
LCU	Lee County Utilities
LRV	log reduction value
LS	lump sum
m^2	square meter(s)
M3M	maximum 3-month
M3MADF	maximum 3-month average day flow
M3MADL	maximum 3-month average day load
MADF	monthly average day flow
MD	maximum day
MDF	maximum daily flow
MDL	maximum day load
MF	microfiltration
mg	milligram(s)
MG	million gallon(s)

mg/L	milligram(s) per liter
mgd	million gallon(s) per day
mg-min/L	milligram-minute(s) per liter
mL	milliliter(s)
mL/g	milliliter(s) per gram
MLSS	mixed liquor suspended solid
MLVSS	mixed liquor volatile suspended solid
MM	maximum month
MMAD	maximum month average day
MMADF	maximum-month average daily flow
MMADL	maximum-month average day load
MW	maximum week
MWADF	maximum-week average daily flow
MWADL	maximum-week average day load
n/a	not applicable
NAVD88	North American Vertical Datum of 1988
NF	nanofiltration
NH_3-N	ammonia as nitrogen
NO_3-N	nitrate as nitrogen
NPDES	National Pollutant Discharge Elimination System
NTU	nephelometric turbidity unit(s)
O&M	operations and maintenance
ORP	oxidation reduction potential
Overlay Area	Environmental Enhancement and Preservation Communities Overlay
PD	positive displacement
PDR	project definition report
PH	peak hour
PHF	peak hour flow
ppmv	part(s) per million by volume
psi	pound(s) per square inch
psig	pound(s) per square inch gauge

RAS	return activated sludge
RO	reverse osmosis
Rule	Wetlands Application Rule
SCADA	supervisory control and data acquisition
SCFM	standard cubic foot (feet) per minute
SEAWRF	Southeast Advanced Water Reclamation Facility
SFWMD	South Florida Water Management District
SLR	solids loading rate
SOR	standard oxygen requirement
SRT	solids residence time
SST	stainless steel
S.U.	standard unit(s)
SWFIA	Southwest Florida International Airport
SWD	side water depth
TBD	to be determined
TDS	total dissolved solids
Ten State Standards	*Recommended Standards for Wastewater Facilities*
TKN	total Kjeldahl nitrogen
TN	total nitrogen
TOC	total organic carbon
TOWRF	Three Oaks Water Reclamation Facility
TOX	total organic halogen
TP	total phosphorus
TRC	total residual chlorine
TSS	total suspended solids
UFAS	Upper Floridan Aquifer System
UV	ultraviolet
UVAOP	ultraviolet advanced oxidation process
VFD	variable frequency drive
VSS	volatile suspended solids
W3	nonpotable plant service water

WAS	waste activated sludge
WoL	whole of life
WRF	water reclamation facility
WTP	water treatment plant

1 Introduction

The purpose of this Project Definition Report (PDR) is to document a preliminary engineering (conceptual level of design) for the proposed Southeast Advanced Water Reclamation Facility (SEAWRF) by gathering information to define the facility requirements, flow and load projections, and phasing; define equipment and process preferences; and evaluate alternatives for certain unit processes. The selected unit process alternatives and information presented will form the basis for detailed design.

The PDR presents three phases for project implementation: 6, 8, and 10 million gallon per day (mgd) annual average daily flow (AADF). The design documents will include Phase 1 only. Phase 2 and buildout will be a future design and construction contract.

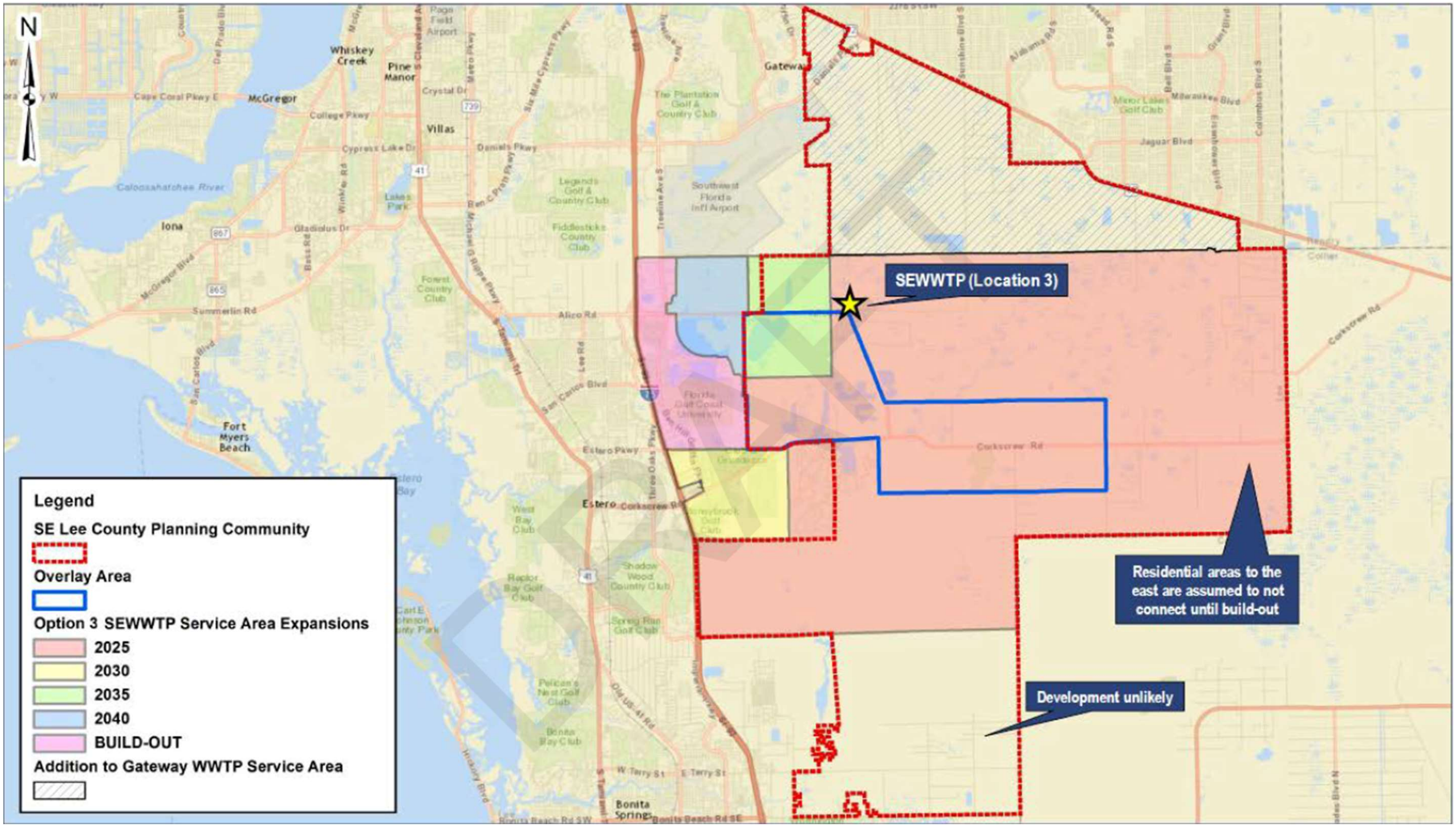
Process modeling was performed based on the defined flows and loads, and an overall process flow diagram was developed. The purpose of the model is to develop a mass balance for use in evaluation of liquid and solids process alternatives, size all major unit processes, and define process design criteria. Also, an overall control philosophy, including level of operator attention, preferences for local versus central control, and level of automation, is defined.

1.1 Project Background

Lee County Utilities (LCU) needs a new water reclamation facility (WRF) to serve projected wastewater flows within the Southeast Lee County Planning Community, including an area referred to as the Environmental Enhancement and Preservation Communities Overlay (Overlay Area). The proposed WRF will help serve existing and future flows in the southeast Lee County service area. These areas are currently served by the Three Oaks Water Reclamation Facility (TOWRF) or are currently not served (Figure 1-1). Wastewater flows from these areas, along with nanofiltration (NF) concentrate from the Pinewoods Water Treatment Plant, will be treated by the proposed SEAWRF.

LCU desires to achieve advanced wastewater treatment (AWT) standards for nutrient removal. The treatment process will consist of preliminary treatment, secondary biological nutrient removal activated sludge via oxidation ditches, secondary clarifiers, deep-bed filters, and high-level disinfection (HLD) using sodium hypochlorite. Solids produced from the liquid process will be digested aerobically and dewatered using belt filter presses or centrifuges. The dewatered cake will be either disposed of in a Class I landfill or transported to the County’s composting facility for further processing. At startup, a portion of the existing service areas from the TOWRF will be routed to the new SEAWRF. LCU plans to primarily send its treated reclaimed water that will meet AWT standards to restore and enhance nearby natural wetlands. The SEAWRF will include a deep injection well (DIW) as a backup disposal option.

Figure 1-1. Lee County Future Wastewater Service Area



The new SEAWRF will have a Phase 1 capacity of 6-mgd AADF, with the ability to expand to a buildout AADF of 10 mgd in 2-mgd increments. The major facilities anticipated for the SEAWRF through buildout are listed as follows:

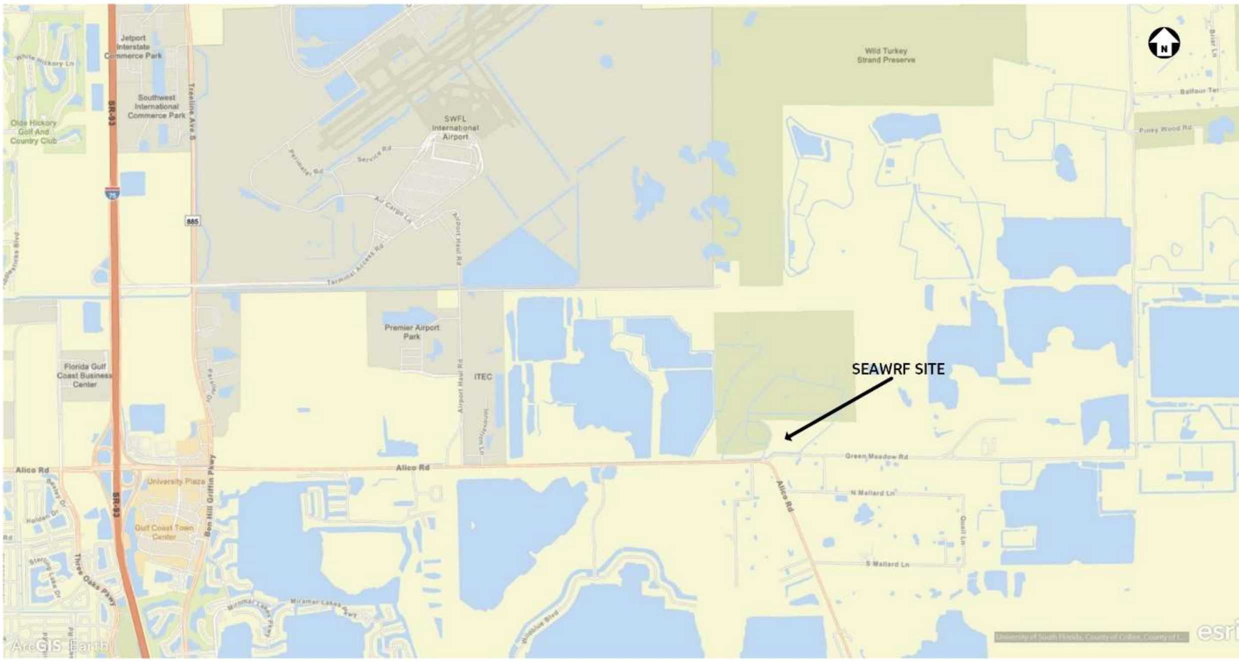
- Headworks (including screening and grit removal)
- Master influent pump station (if determined to be necessary)
- Odor control
- Process bioreactor splitter box
- Process bioreactors
- Secondary clarifier flow splitter
- Secondary clarifiers
- Return activated sludge (RAS) and waste activated sludge (WAS) pumping
- Deep-bed filters
- Disinfection
- Effluent transfer pumping
- Reclaimed water storage
- Reject water storage
- Reclaimed water and plant water distribution pumping
- DIW Pump Station
- DIW
- Chemical storage and feed systems (sodium hypochlorite, alum, supplemental carbon)
- Plant drain pumping
- Generators (standby power)
- Aerobic digesters
- Biosolids dewatering
- Administration Building
- Maintenance Building
- Electrical buildings

1.2 Location

The SEAWRF will be constructed on an undeveloped site that is currently used for agriculture. The site is located on the north side of Green Meadow Road and Alico Road (Figure 1-2). Access to the site will be from Green Meadow Road near the intersection with Alico Road.

The property is 112.2 acres in size and the project site is approximately 50.6 acres, with portions of the site containing 0.9 acre of exotic wetlands and approximately 1.49 acres of ditches. The site is outside the 100-year Federal Emergency Management Agency (FEMA) flood zone. The SEAWRF will be located on the northern and western portion of the property. The southern 250 feet of property will be reserved for a future right-of-way; this area is approximately 31.2 acres in size. The eastern 30.4 acres of wetlands on the property will remain undeveloped and exotic vegetation will be removed. The site configuration is not anticipated to affect the wetland areas except for those wetlands needed to widen the existing driveway.

Figure 1-2. Southeast Advanced Water Reclamation Facility Location Map



1.3 Lee County Standards

Wastewater treatment systems will be designed and constructed according to Lee County standards and preferences, including those in the data request dated May 31, 2022, provided by Lee County (Appendix A), and discussed at the project kickoff workshop held on June 29, 2022. In addition, the most current editions of the following publications will be used:

- Lee County Instrumentation Standards (provided by Lee County via email dated May 7, 2019, and included in Appendix A)
- *Recommended Standards for Wastewater Facilities* (Ten State Standards; Wastewater Committee of the Great Lakes 2014)
- Florida Department of Environmental Protection (FDEP) and other applicable federal, state, and local requirements

Any requirements for standards or specifications that are not adequately addressed in these documents will be evaluated by the Lee County and Jacobs project teams for specific use on this project.

1.4 Project Definition Report Organization

This PDR is organized into the following sections:

1. Introduction
2. Flows and Loads
3. Effluent Disposal and Water Quality Criteria
4. Site Plan and Environmental Effects
5. Process Design Summary
6. Treatment Facilities
7. Direct or Indirect Potable Reuse Alternatives Analysis
8. Project Delivery Approach
9. References

1.5 Phasing

The facility will be constructed in a 6-mgd AADF phase, followed by two 2-mgd AADF phases with a buildout capacity of 10-mgd AADF. The approximate timeline for required completion of construction for Phase 1, based on current growth projections, is 2028 (Table 1-1). The required construction completion dates for subsequent phases are to be determined in the future based on needs and demands. This PDR describes the facilities for Phase 1 and each subsequent phase. The design will include Phase 1 and have provisions for Phase 2 and buildout facilities. Phase 1 facilities are the basis for the project, with the Phase 2 and Phase 3 facilities to be constructed later.

Table 1-1. Projected Phase Timeline

Phase	AADF (mgd)	Year
1	6	2028
2	8	TBD
3	10	TBD

TBD = to be determined

1.6 Permitting

The SEAWRF parcel currently has a South Florida Water Management District (SFWMD) Environmental Resource Permit (ERP) to authorize operation of agricultural lands. Proposed modifications to the initial permit have been made on the parcel, but never constructed. Future development of the site will require modification of the existing SFWMD permit or obtaining a new FDEP ERP. Impacts to the existing ditches and swales may require permitting through the FDEP, SFWMD, and U.S. Army Corps of Engineers.

1.6.1 Florida Department of Environmental Protection

This PDR describes the wastewater treatment process and facility design. In accordance with Section 62-620.320 of the *Florida Administrative Code* (FAC), this report provides reasonable assurance that the proposed SEAWRF will provide treatment facilities for both wastewater and solids in accordance with good engineering practices, including hydraulic and organic loadings, process flow diagrams, and provisions for the reuse of wastewater; reliability and redundancy; and operation and maintenance (O&M) strategies.

The SEAWRF will require a domestic wastewater facility discharge permit for the direct/indirect potable reuse (D/IPR) alternative selected in Section 7 of this report and an underground injection control permit for the DIW. This report and the necessary application forms may be submitted to FDEP as part of the permit application process.

1.6.2 Lee County Development Ordinance

The proposed WRF project will need to meet most Lee County Development Code requirements. Some deviations of the code will be requested during the rezoning process for the project to change the zoning to Community Facilities Planned Unit Development. The eastern portion of the site is within the Density Reduction/Groundwater Resource (DR/GR) area future land use category. The County desires that a Comprehensive Plan Amendment be submitted to change the eastern portion of the site from DR/GR to Public Facilities. The project will also require a Development Order approval from Lee County. A preapplication meeting with Lee County Development Services will determine development requirements, exemptions, permit requirements, and other constraints. Landscaping will include 100% native plants and only consist of plants listed in the Lee County Port Authority Landscape List.

2 Flows and Loads

2.1 Introduction

Nine years of influent historical data (2014 through 2022) from the TOWRF were used to characterize the design flows and loads for the new SEAWRF. Influent historical data from the TOWRF were thought to be representative of the flows and loads to the new WRF because of the following:

- Most of the flow diversion to the new WRF will come from the Three Oaks service area.
- New development in the new WRF service area will be primarily residential, as is the case in the Three Oaks service area.

The flows and loads to be used for design of each phase are summarized in this section. More detail regarding the development of the flows and loads basis can be found in Appendix B.

2.2 Southeast Advanced Water Reclamation Facility Service Area

Due to capacity limitations at the TOWRF, a portion of that existing service area will be reallocated to the new SEAWRF. Figure 1-1 illustrates the new service area for the SEAWRF.

2.3 Influent Flow

Understanding flow variation is important for evaluating existing facilities and planning for new facilities. Flow variation is typically expressed in terms of the ratio to the AADF, commonly referred to as peaking factors. Peaking factors of interest include the maximum month (MM), maximum 3-month (M3M), maximum week (MW), maximum day (MD), and peak hour flow (PHF).

TOWRF historical flows and flow peaking factors are presented in Table 2-1 for the period from 2014 to 2022. Peak hour flow data was not available. A peaking factor of 3.0 relative to AADF is recommended for PHF based on review and consideration of several items, including the following:

- Ten State Standards guidelines (Wastewater Committee of the Great Lakes 2014), which suggest a PHF peak factor of about 2.3.
- Corkscrew Overlay Area Wastewater Master Planning Report (JEI 2016). This report was focused on modeling lift stations and forcemains and does not present an overall PHF peaking factor for a treatment plant. However, PHF peaking factors for master pump stations and forcemains discussed in the report ranged from approximately 2.6 to 3.1.
- Experience with other facilities in the area, including the Bonita Springs East WRF, which shows a design PHF peaking factor of 3.0. The TOWRF also has a design PHF peaking factor of 3.0, according to the Lee County 2011 *Capacity Analysis Report*.

In addition to the historical yearly data summarized in Table 2-1, the percentiles of the daily flow peaking factors relative to each year's average were computed to normalize the data set. The recommended peaking factors for the SEAWRF presented in Table 2-1 are based on engineering judgement considering the averages, range, and computed percentile peaking factors for the entire data set.

Table 2-1. Historical and Recommended Influent Flows and Peaking Factors

Year	Minimum (mgd)	AADF (mgd)	M3-MADF (mgd)	MMADF (mgd)	MWADF (mgd)	MDF (mgd)	M3M AA (mgd)	MM:AA (mgd)	MW:AA (mgd)	MD:AA (mgd)	PH:AA (mgd)
2014	1.96	2.70	3.10	3.16	3.30	3.60	1.15	1.17	1.22	1.33	
2015	2.37	2.90	3.20	3.32	3.40	4.20	1.10	1.14	1.17	1.45	

Year	Minimum (mgd)	AADF (mgd)	M3-MADF (mgd)	MMADF (mgd)	MWADF (mgd)	MDF (mgd)	M3M AA (mgd)	MM:AA (mgd)	MW:AA (mgd)	MD:AA (mgd)	PH:AA (mgd)
2016	2.80	3.20	3.50	3.53	3.70	4.10	1.09	1.10	1.16	1.28	
2017	2.63	3.10	3.30	3.57	4.80	5.90	1.06	1.15	1.55	1.90	
2018	2.91	3.30	3.60	3.84	4.00	4.70	1.09	1.16	1.21	1.42	
2019	3.21	3.79	4.04	4.29	4.48	5.09	1.07	1.13	1.18	1.34	
2020	3.10	3.64	4.18	4.29	4.52	4.72	1.15	1.18	1.24	1.30	
2021	3.26	3.84	4.07	4.16	4.32	5.00	1.06	1.08	1.12	1.30	
2022[a]	3.62	4.36	4.50	4.65	4.93	7.55	1.03	1.07	1.13	1.73	
Average							1.09	1.13	1.22	1.45	
Maximum							1.15	1.18	1.55	1.90	
Percentile[b]							1.06	1.14	1.21	1.34	--
Recommended SEAWRF Peaking Factors							1.1	1.2	1.3	1.9	3.0[c]

[a] Only 5 months of flow data were available for 2022.

[b] The 75.3th percentile corresponds to M3M:AA.

The 91.7th percentile corresponds to MM:AA.

The 98th percentile corresponds to MW:AA.

The 99.7th percentile corresponds to MD:AA.

[c] Peak hour flow data were not available. A peaking factor of 3.0 relative to AADF was assumed based on Ten State Standards guidelines and experience with other facilities in the area.

MADF = monthly average daily flow

MWADF = maximum-week average daily flow

PH = peak hour

Table 2-2 presents the recommended design flows for Phase 1, Phase 2, and the final buildout of the SEAWRF, based on the recommended flow peaking factors.

Table 2-2. Recommended Design Flows

Parameter	Phase 1 (mgd)	Phase 2 (mgd)	Buildout (mgd)
AADF	6	8	10
M3MADF	6.6	8.8	11.0
MMADF	7.2	9.6	12.0
MWADF	7.8	10.4	13.0
MDF	11.4	15.2	19.0
PHF	18.0	24.0	30.0
Startup Flow (AADF)	0.7	n/a	n/a
Minimum Hour Startup Flow	0.3	n/a	n/a

M3MADF = maximum 3-month average daily flow

MDF = maximum daily flow

MMADF = maximum-month average daily flow

MWADF = maximum-week average daily flow

n/a = not applicable

2.4 Influent Loads

TOWRF data of daily concentrations of influent 5-day carbonaceous biochemical oxygen demand ($CBOD_5$) and total suspended solids (TSS) were analyzed for the 9-year period from 2014 through 2022 to determine annual average influent design concentrations. Outliers, defined as values greater than three standard deviations from the mean, were not used when determining the historical annual average concentrations. However, very few outliers were identified. Monthly influent samples of total Kjeldahl nitrogen (TKN), ammonia, and phosphorus were also included in the 9 years of TOWRF data, and annual influent design concentrations were also calculated for these constituents. The historical annual average influent concentrations of various constituents are presented in Table 2-3. Overall, there does not appear to be a general trend up or down in the concentrations. Somewhat conservative average concentrations are recommended, as shown.

Currently, TOWRF receives NF concentrate from the Pinewoods Water Treatment Plant (WTP). On average, it accounted for 8% of the flow to TOWRF over the period from 2014 to 2022. This flow has the effect of diluting the influent concentrations of $CBOD_5$ and TSS by the same proportion. Beginning around 2030, the NF concentrate is projected to be redirected to the new SEAWRF. The recommended concentrations for the SEAWRF presented in Table 2-3 are based on engineering judgement considering the averages and range for the entire data set as well as being corrected for the dilution effect of the NF concentrate to the TOWRF influent concentrations.

Table 2-3. Historical and Recommended Influent Annual Average Concentrations

Year	$CBOD_5$ (mg/L)	True BOD_5[a] (mg/L)	TSS (mg/L)	TKN (mg/L)	NH_3-N (mg/L)	TP (mg/L)
2014	179	-	231	41.1	29.8	5.17
2015	178	-	232	40.1	31.1	5.16
2016	167	-	212	41.0	36.1	5.02
2017	164	-	204	46.9	43.2	5.15
2018	172	-	187	41.6	33.6	4.61
2019	157	-	189	42.1	31.2	5.07
2020	139	-	173	50.1	34.7	5.53
2021	141	-	175	48.8[b]	38.7[b]	6.02[b]
2022	183	-	188	58.5[b]	41.2[b]	6.10[b]
Average	164	-	199	45.6	35.5	5.31
Maximum	183	-	232	58.5	43.2	6.10
Minimum	139	-	173	40.1	29.8	4.61
Recommended SEAWRF Concentrations	200	240	240	50	41	5.8

[a] True BOD_5 is corrected for the effect of nitrification inhibitor added for the $CBOD_5$ test and represents the $CBOD_5$ divided by a factor of 0.84.

[b] The year 2021 only had five total monthly samples and 2022 had two total monthly samples, which may cause the average annual concentrations to appear to be higher.

BOD_5 = 5-day biochemical oxygen demand

mg/L = milligram(s) per liter

NH_3-N= ammonia as nitrogen

TP= total phosphorus

Historical influent pollutant loads in pounds per day from the 9-year period were analyzed for the TOWRF. Table 2-4 summarizes the historical $CBOD_5$ and TSS load peaking factors relative to the annual average load and the recommended peaking factors for the design of the SEAWRF. In addition to the TOWRF historical yearly peaking factors summarized in Table 2-4, the percentiles of the daily load peaking factors relative to each year's average were computed to normalize the data set.

Peaking factors for TKN, NH_3-N, and TP were not determined because these constituents are not sampled daily. Table 2-5 presents the recommended design peaking factors for TKN, NH_3-N, and TP, which are assumed to be consistent with the peaking factors for $CBOD_5$. Because these are load-based peaking factors and the concentrate does not contain $CBOD_5$ or TSS, it is not necessary to consider effects of concentration dilution by the NF concentrate flow.

Table 2-4. Historical Three Oaks Loading Peaking Factors and Recommended Peaking Factors

Year	$CBOD_5$				TSS			
	M3M:AA	MM:AA	MW:AA	MD:AA	M3M:AA	MM:AA	MW:AA	MD:AA
2014	1.43	1.48	1.56	1.94	1.27	1.31	1.47	1.97
2015	1.23	1.30	1.41	1.54	1.14	1.19	1.32	1.51
2016	1.16	1.24	1.39	1.66	1.18	1.25	1.38	1.71
2017	1.34	1.41	1.57	1.78	1.24	1.37	1.45	1.62
2018	1.29	1.44	1.54	1.83	1.23	1.31	1.39	1.81
2019	1.18	1.28	1.37	1.58	1.25	1.35	1.43	1.64
2020	1.27	1.32	1.39	2.11	1.33	1.41	1.52	1.86
2021	1.21	1.41	1.78	2.24	1.17	1.36	1.54	1.93
2022	1.08	1.12	1.26	1.58	1.06	1.09	1.26	1.76
Average	1.24	1.33	1.47	1.81	1.21	1.29	1.42	1.76
Maximum	1.43	1.48	1.78	2.24	1.33	1.41	1.54	1.97
Percentile	1.16	1.35	1.58	1.88	1.15	1.32	1.51	1.81
Recommended SEAWRF Peaking Factors	1.3	1.4	1.6	2.0	1.3	1.4	1.5	1.9

Table 2-5. Assumed Design Load Peaking Factors

Constituent	M3M:AA	MM:AA	MW:AA	MD:AA
TKN	1.3	1.4	1.6	2.0
NH_3-N	1.3	1.4	1.6	2.0
TP	1.3	1.4	1.6	2.0

Using the recommended flow peaking factors shown in Table 2-1, the annual average influent constituent concentrations presented in Table 2-3, and the recommended and assumed load peaking factors presented in Tables 2-4 and 2-5, the design loads and flow rates for the new SEAWRF were computed and are presented in Table 2-6. Design loads and flow rates are based on three phases for the new SEAWRF with AADFs of 6, 8, and 10 mgd. In addition, because Lee County had trouble maintaining sufficient dissolved oxygen (DO) in aeration basins in the past for diurnal daytime peak load during MM loading periods, it is recommended to apply an additional 15% safety factor for aeration on the selected maximum day loads (MDLs) presented in Table 2-6.

Table 2-6. Recommended Design Loadings

Parameter	Peaking Factor	AADF		
		6	8	10
Flow				
M3MADF (mgd)	1.1	6.6	8.8	11.0
MMADF (mgd)	1.2	7.2	9.6	12.0
MWADF (mgd)	1.3	7.8	10.4	13.0
MDF (mgd)	1.9	11.4	15.2	19.0
PHF (mgd)	3.0	18.0	24.0	30.0

Parameter	Peaking Factor	AADF		
		6	8	10
True BOD_5[a]				
AADL (pounds per day)	-	12,010	16,013	20,016
AAD (mg/L)	-	240	240	240
M3MADL (pounds per day)	1.3	15,612	20,817	26,021
MMADL (pounds per day)	1.4	16,813	22,418	28,022
MWADL (pounds per day)	1.6	19,215	25,621	32,026
MDL (pounds per day)	2.0	24,019	32,026	40,032
TSS				
AADL (pounds per day)	-	12,010	16,013	20,016
AAD (mg/L)	-	240	240	240
M3MADL (pounds per day)	1.3	15,612	20,817	26,021
MMADL (pounds per day)	1.4	16,813	22,418	28,022
MWADL (pounds per day)	1.5	18,014	24,020	30,024
MDL (pounds per day)	1.9	22,818	30424	38,030
TKN				
AADL (pounds per day)	-	2,502	3,336	4,170
AAD (mg/L)	-	50	50	50
M3MADL (pounds per day)	1.3	3,253	4,337	5,421
MMADL (pounds per day)	1.4	3,503	4,670	5,838
MWADL (pounds per day)	1.6	4,003	5,338	6,672
MDL (pounds per day)	2.0	5,004	6672	8,340
NH_3-N				
AADL (pounds per day)	-	2,052	2,735	3,419
AAD (mg/L)	-	41	41	41
M3MADL (pounds per day)	1.3	2,667	3,556	4,445
MMADL (pounds per day)	1.4	2,872	3,830	4,787
MWADL (pounds per day)	1.6	3,283	4,376	5,471
MDL (pounds per day)	2.0	4,103	5471	6,839
TP				
AADL (pounds per day)	-	290	387	484
AAD (mg/L)	-	5.8	5.8	5.8
M3MADL (pounds per day)	1.3	377	503	629
MMADL (pounds per day)	1.4	406	542	677
MWADL (pounds per day)	1.6	464	619	774
MDL (pounds per day)	2.0	580	774	967

[a] True BOD_5 is corrected for the effect of nitrification inhibitor added for the $CBOD_5$ test and represents the $CBOD_5$ divided by a factor of 0.84.

AAD = annual average day

AADL = annual average day load

M3MADL = maximum 3-month average day load

MMADL = maximum month average day load

MWADL = maximum week average day load

3 Effluent Disposal and Water Quality Criteria

The method of effluent and solids disposal determines the treatment requirements. The facility will be permitted under applicable Florida Statutes (FS) and FAC. The effluent from the SEAWRF will meet AWT standards and will primarily be used for beneficial reuse, which may include wetland recharge with an alternative discharge to a DIW as backup. The solids will be disposed in a Class I landfill or hauled to the County's composting facility for further treatment. This section summarizes the requirements for disposal that are used as the basis for the design.

3.1 Effluent Disposal

The SEAWRF will produce reclaimed water that meets AWT standards and is suitable for beneficial reuse, which could include restoring and enhancing nearby natural wetlands. A DIW will be provided onsite as a backup to discharge excess reclaimed water. Additionally, reject storage will be included to divert off-specification water for retreatment, and reclaimed water storge will be provided.

3.2 Water Quality Criteria

3.2.1 Advanced Wastewater Treatment Requirements

To maximize effluent reuse options and to provide enhanced protection to local water bodies, the SEAWRF will be designed to meet AWT standards. AWT is defined in FS Section 403.086 as the treated effluent containing not more than the following concentrations:

- $CBOD_5$ annual average will not exceed 5 mg/L.
- TSS annual average will not exceed 5 mg/L.
- Total nitrogen (TN) annual average will not exceed 3 mg/L.
- TP annual average will not exceed 1 mg/L.

In addition, HLD is required for AWT. Therefore, the proposed SEAWRF is anticipated to have limits as summarized in Table 3-1.

Table 3-1. Advanced Wastewater Treatment Limitations and Select Monitoring Requirements

Parameter	Unit	Discharge Limitations			Monitoring Requirements	
		Annual Average	Monthly/ Weekly Average	Single Sample	Monitoring Frequency	Sample Type
Flow	mgd	6.0	Report	n/a	Daily	Calculation
$CBOD_5$	mg/L	5.0	6.25/7.25	10.0	Weekly	24-hour composite
TSS	mg/L	5.0	6.25/7.25	10.0	Weekly	24-hour composite
TN	mg/L	3.0	3.75/4.50	6.0	Weekly	24-hour composite
NH_3-N	mg/L	n/a	2.0/n/a	Report	Weekly	24-hour composite
TP	mg/L	1.0	1.50/1.25	2.0	Weekly	24-hour composite
Fecal Coliform	75% of samples must measure below detection limits over a 30-day period			25 #/100 mL	Weekly	Grab
pH	S.U.	-	-	$6 \leq pH \leq 8.5$	Continuous	Meter
TRC	mg/L	-	-	≥ 1	Continuous	Meter
Turbidity	NTU	-	-	≤ 3	Continuous	Meter

≤ = less than or equal to
≥ = greater than or equal to
\# = number
mL = milliliter(s)
NTU = nephelometric turbidity unit
S.U. = standard unit
TRC = total residual chlorine

3.2.2 Deep Injection Well Disposal

A single Class I DIW will be provided onsite for disposal of excess treated wastewater to Class G-IV groundwater. The well will be 24 inches in diameter with a permitted peak capacity of 13,195 gallons per minute (gpm) (19 mgd) at a velocity of 10 feet per second. The well would be constructed to meet the requirements of Underground Injection Control as defined in FAC 62-528, including a required monitoring well. The anticipated requirements for DIW disposal are summarized in Table 3-2 and include HLD that is required for new DIWs in this area. In addition, space has been reserved onsite for a second future injection well.

Table 3-2. Underground Injection Limitations and Select Monitoring Requirements

Parameter	Unit	Discharge Limitations			Monitoring Requirements	
		Annual Average	Monthly/Weekly Average	Single Sample	Monitoring Frequency	Sample Type
Flow	mgd	10.0	Report	19 Peak	Continuous	Meter
$CBOD_5$	mg/L	20.0	30.0/45.0	60.0	5 Days/week	24-hour composite
TSS	mg/L	n/a	n/a	5.0	Daily	Grab
Fecal Coliform	75% of samples must measure below detection limits over a 30-day period			25 #/100 mL	Daily	Grab/Calculated
pH	(S.U.)	-	-	$6 \le pH \le 8.5$	Continuous	Meter
TRC	mg/L	-	-	≥ 1	Continuous	Meter
Turbidity	NTU	-	-	≤ 3	Continuous	Meter

3.3 Residuals Disposal

Biosolids from the SEAWRF will be dewatered and disposed of in a Class I landfill or transported to the County's composting facility for further processing. As such, treatment is not required to meet Class A or Class B standards for land application. Composting would meet Class AA standards; however, that process is not part of the scope of this project.

4 Site Plan and Environmental Effects

4.1 Site Plan

The new SEAWRF will be located on the north side of Alico Road, approximately 5 miles east of Interstate 75. The site address is 18990 Green Meadow Road, Fort Myers, Florida. The current parcel is an undeveloped open space with scattered trees; however, there are existing drainage ditches that will need to be filled for project construction.

The site plan, Figure 4-1, shows the proposed SEAWRF conceptual layout. The major facilities associated with the new water reclamation facility consist of treatment unit processes, storage tanks, pump stations, underground piping, aboveground piping, and Maintenance and Administration Buildings. Other SEAWRF improvements include a DIW and a monitoring well. Each of these site features are discussed in greater detail in other sections of this report. Additional site improvements include new roads and parking facilities, fencing, and a stormwater management system.

4.2 Entrance Road

As shown on the site plan, access to the SEAWRF will be via Green Meadow Road. The site development is set back 250 feet to account for the future right-of-way for the Alico Road extension. The SEAWRF will have a driveway from Green Meadow Road. The entrance will be sited on the east side of the site to provide as much space as possible from the western roadway intersection. The site entrance will have right turn in, left turn in, and right turn out traffic movements. An access driveway leading to the adjacent Wild Turkey Strand Conservation Area lands to the west will be provided within the plant site. It is anticipated that no offsite roadway improvements will be necessary for the project due to the limited number of roadway trips anticipated. Additionally, the future Alico Road extension is likely to be completed near the same time as the SEAWRF.

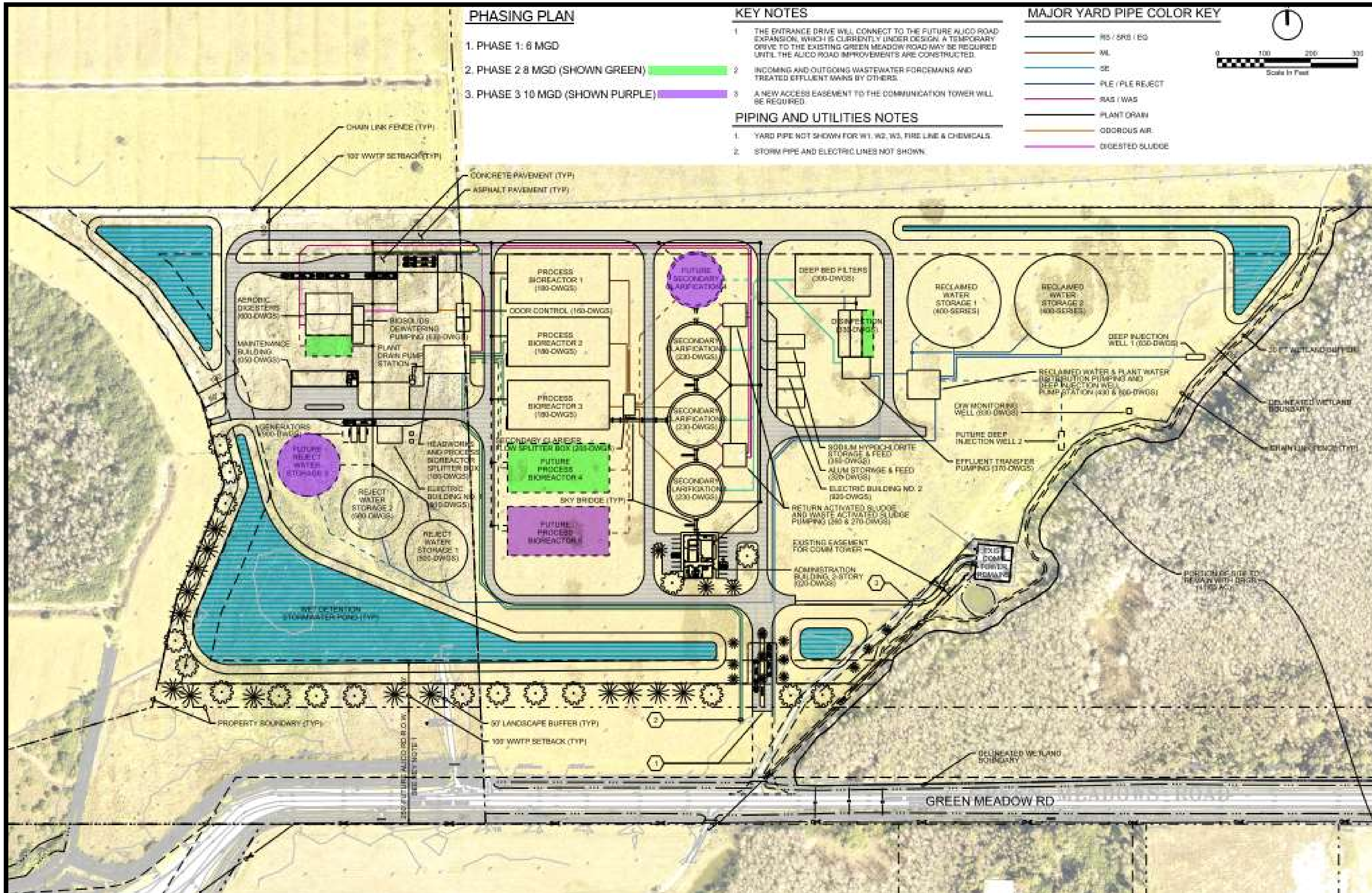
All internal roads will be composed of a minimum of 2 inches of asphalt, 12 inches of limerock base, and 12 inches of compacted subgrade. The bottom of the limerock base will be a minimum of approximately 2 feet above the seasonal high-water table. The primary internal roadways serving the facility will have a turning radius configured such that a WB-62 design (interstate semitrailer) vehicle can turn successfully. General access roadway radii will be sized for passenger vehicles and light-duty trucks. Roads within the SEAWRF will be a minimum of 20 feet wide.

4.3 Civil Site Design Criteria

4.3.1 Site Resiliency

Review of the FEMA flood maps indicates the site is within Flood Zone “X” and, therefore, not within the 100-year special flood hazard area. Because the site is designated as a No Special Flood Hazard area, a FEMA Flood Insurance Rate Map panel for the site was not produced. There are no defined floodways within the project limits, and the project will result in no floodplain encroachments.

Figure 4-1. Lee County SEAWRF Conceptual Site Plan



The design storm event for parking areas is the 5-year, 1-day storm event. The finished floor elevation will be 1 foot higher than the stages predicted from the 100-year, 3-day storm event. The minimum elevation of the perimeter berm will use the 25-year, 3-day design storm. The minimum finished floor elevation of the Administration Building and Maintenance Building will be approximately 30.0 North American Vertical Datum of 1988 (NAVD88) feet based upon the current stormwater management configuration. All critical electrical components and pumps should be at elevation 29.5 feet or higher. The minimum pavement elevation is estimated to be 27.6 feet. The 25-year, 3-day storm event elevation is estimated to be 27.6 feet. The perimeter berm for the site will be set near elevation 28.0 feet or higher. These elevations may change slightly depending on final paving and grading elevations and concurrence from the permitting agencies. As such, it is anticipated that approximately 4 to 6 feet of fill will be required to elevate the site to meet these requirements.

4.3.2 Stormwater Management

The stormwater management system will be designed in conformance with the standards of the Lee County Development Code and FDEP and SFWMD requirements. As shown on the conceptual site plan, wet detention stormwater ponds are proposed to handle water quality treatment and flood attenuation of runoff from the reclamation facility. The water quality treatment volume is 1.0 inch of runoff from the entire site, which is the state standard for projects in this watershed.

Runoff from the site will be collected through yard drains, swales, ditches, and catch basins, with conveyance via pipes or swales to the stormwater ponds. The overall site contains an interconnected stormwater pond system and will use a single control structure to discharge runoff to the western slough via a spreader swale or multiple structures to disperse the flow. The stormwater will then continue within the western slough, following existing drainage patterns. The site design will accommodate the future berm on the Wild Turkey Strand Conservation Area property to the west and north and will likely be constructed as part of this project if the County does not construct the adjacent berm before construction of the SEAWRF.

Protection against offsite erosion and sedimentation from construction activities will be in accordance with the National Pollutant Discharge Elimination System (NPDES) for large construction activities. A Stormwater Pollution Prevention Plan that outlines best management practices for the implementation and maintenance of erosion control measures will be prepared. At a minimum, silt fencing will be installed at the edge of construction and a construction access rock driveway will be built to prevent offsite discharge of sediment.

4.3.3 Vehicle Access and Parking

Employee and visitor parking will be provided at the site. Parking will include 16 standard parking spaces and 1 handicapped space next to the Administration Building. Typical parking spaces are 10 feet wide and 20 feet long. There is also enough open space for overflow parking areas close to the buildings for buses and cars, often associated with plant tours. Vehicle access and parking will need to be further refined for the SEAWRF once a better understanding of staffing and visitors' accommodation is defined. Additional parking spaces are provided at the Maintenance Building for operations vehicles.

4.3.4 Applicable Codes, Standards, and Design Criteria

The civil design will conform with the following codes and standards:

- Florida Building Code, 7th edition (2020)
- Lee County Development Code
- FDEP
- SFWMD
- Florida Department of Transportation (FDOT)
- NPDES

Jacobs will use the latest edition of each code unless a specific year is listed here.

4.4 Site Survey

A boundary and topographic survey was prepared in June 2019 by Johnson Engineering, Inc. Coordinates are projected onto the Florida State Plane Coordinate System, West Zone, North American Datum of 1983, 2011 adjustment. Elevations reference the NAVD88. The reference benchmark is FDOT Benchmark I75 81-A19. The site has an average grade elevation of 24.0 feet NAVD88, and the wet-season water table is at, or near, land surface. There are no wetland impacts proposed other than ditch impacts for a widened driveway.

Underground utility locates were not performed as part of the boundary and topographic survey; however, a review of available record drawings indicates existing utilities near the site. There is a 30-inch potable water main along Alico Road, directly between the roadway and the project site. The existing water main should have no significant impact on the project design.

The July 2019 survey will serve as the base file for the design. The date of the most recent fieldwork associated with the survey was May 8, 2019.

4.5 Exploratory Geotechnical Investigation

4.5.1 Preliminary Geotechnical Exploration

Preliminary geotechnical investigations and laboratory testing were performed by Ardaman and Associates, Inc. during January and February 2020. A total of 7 standard penetration test borings were performed, ranging from 30 to 95.5 feet deep, to evaluate foundation materials, along with 5 hand auger borings to 10 feet deep. Doublering infiltrometer tests were also performed adjacent to the hand auger borings to evaluate infiltration capacity for stormwater pond performance.

A geotechnical data report was provided by Ardaman and Associates, Inc. (2020) in (Appendix C). The report summarizes the findings of the exploration and laboratory testing. Based on site conditions and review of a piezometer installed to monitor groundwater, the groundwater level should be assumed to be near the existing ground surface. In general, materials encountered consisted of 7- to 14-foot thickness of sand (SP) to sand with silt (SP-SM) overlying 0- to 15-foot thickness of silty sand (SM) to clayey sand (SC), overlying alternating layers of very soft to hard limestone with varying amounts of weathering. Some limestone layers have been broken down to fat sandy clay with limestone fragments (CH) to silty sand (SM) with limestone fragments.

Reference Ardaman's report for general geologic site conditions as well as more detailed information regarding the individual soil borings and testing.

4.5.2 Additional Geotechnical Exploration

Based on the conditions encountered during the preliminary investigation, a second phase of exploration is required to determine if deep foundations, soil amendments, or other remediation such as preloading may be required for the structure foundations in areas not investigated in the initial geotechnical investigation. Additional subsurface data including soil borings or cone penetration testing are recommended, and structural loading and foundation sizes will be needed to be evaluated. A minimum of one exploration will be required per building with additional recommendations as required by codes such as American Concrete Institute 372 for prestressed concrete tanks.

4.6 Archeological Survey

A cultural resource reconnaissance survey and a Phase I archeological survey will be conducted of the project site, as necessary, to complete FDEP and SFWMD permitting. As necessary, the parcel will be surveyed to locate and assess any sites of archeological or historical significance. The cultural resource assessment will include a vehicular and pedestrian survey as well as judgmental shovel testing. The assessment will be conducted to fulfill historic resource requirements in response to the State of Florida historic preservation guidelines. The work and report will conform to the specifications described in Chapter IA46, FAC.

4.7 Public Accessibility and General Site Security

As shown on the conceptual site plan, continuous perimeter fencing for the new SEAWRF site will be installed. The proposed fencing along the south side of the property will be 8 feet high with barbed wire. The fencing on the north, west, and east sides of the site will be 10 feet high with 1 foot of three-string barbed wire on top projecting out from the site. The 11-foot-high fence will serve as a wildlife deterrent. An automated cantilever slide gate is proposed for the main entrance of the SEAWRF located off Green Meadow Road. The entrance gate will have a call box and camera that are monitored and controlled from the Administration Building. The gate will predominantly remain closed and will mostly be used by Lee County employees.

4.8 Environmental Effects

This section includes an assessment of environmental effects of the SEAWRF as required by FDEP guidelines for preliminary design reports, including odor and noise control, public accessibility, proximity to residential areas, flood protection, lighting, and aerosol drift. Environmental aspects related to wetlands, protected species, and mitigation are being addressed outside of this report.

4.8.1 Odor and Noise Control

The headworks will be covered, and the odorous air will be captured for treatment in a bio scrubber. Equipment with the greatest potential for generating odors and noise will be located away from the southern property line, where residential development exists on the south side of the 250-foot-wide Alico Road easement. The areas to the north and west are conservation areas that will not be developed. On the eastern side of the site, there is another parcel owned by the County providing a substantial buffer on the east, west, and north sides of the site. Nuisances such as odors and noise will be mitigated as part of the design. Sound attenuating enclosures will be provided with equipment, as necessary, to comply with local noise ordinances.

4.8.2 Public Access

The perimeter of the site will be fenced with security fencing to prevent unauthorized access. Vehicle access will be through a security gate with card reader access. Security cameras will be provided, and warning signs will be posted. Where wetlands are adjacent to the SEAWRF portion of the site, the fence will be located outside of the wetlands to avoid impacts and additional permitting requirements.

4.8.3 Proximity to Residential Areas

Residential development is located to the south of the site with a 250-foot roadway easement (Green Meadow Road) between the southern site boundary and the residential development. No other residential development is anticipated immediately adjacent to the site where the SEAWRF is proposed due to the Wild Turkey Strand Conservation Area to the north and west and a County-owned parcel to the east. The site is currently zoned as agricultural. The site will be rezoned to Community Facilities Planned Unit Development. A minimum 100-foot buffer is proposed between the property line and the new facility structures on the northern, eastern, and western sides of the site and a minimum of 100 feet from the 250-foot Alico Road easement on the south side of the site.

The closest residence is located on the south side of Green Meadow Road and is more than 500 feet away. The closest SEAWRF structures used for treatment of wastewater will be at least 100 feet away from the northern side of the Alico Road easement.

4.8.4 Flood Protection

Refer to Sections 4.3.1 and 4.3.2 for a discussion of flood protection and stormwater issues.

4.8.5 Site Lighting

To support operators, mechanics, and security at night and in dim light conditions, lighting will be placed throughout the facility in areas appropriate for light-emitting diode light fixtures. Site lighting will be designed to reduce offsite effects to adjacent properties and highways and will meet local ordinance requirements.

4.8.6 Aerosol Drift

The greatest potential for aerosol generation is from the process bioreactors (oxidation ditches). Aeration in the oxidation ditches will be accomplished with low-speed surface mechanical aerators. The area immediately around each aerator will be covered and will have extended walls on three sides to contain aerosols. The fourth side will have a beam that runs beneath the cover to limit the opening to just above the water level. The aerosol drift will be further controlled by locating this unit process near the center of the site. The setbacks on all sides of the site will further reduce the potential for aerosol drift. The aerobic digesters will be aerated with coarse bubble diffusers, which do not have the same potential for aerosol drift generation as surface aerators and are located on the northern portion of the site.

4.9 Utilities

4.9.1 Potable Water

Potable water to serve facility support buildings, such as the Maintenance Building and Administration Building, will be fed to the site from an existing 30-inch transmission main along Alico Road. A water main will be sized to meet fire flow conditions and, at a minimum, will be 12 inches in diameter to meet local code requirements for commercial development. Potable water lines will be sized appropriately to accommodate the number of employees the facility may have after future expansion.

4.9.2 Wastewater

Raw sewage will enter the site from the south through a forcemain that will be constructed along the Alico Road easement. In Phase 3, a master influent pump station near the headworks may be added. Space will be left for a future forcemain or piping along the same route, should it be necessary. Wastewater generated onsite will flow by gravity to a Plant Drain Pump Station, where it will be pumped to the headworks (downstream of the influent sample location) for treatment.

4.9.3 Reclaimed Water

Reclaimed water from the SEAWRF will be stored in dedicated storage tanks located on the eastern side of the site. The reclaimed water will be able to be pumped for offsite beneficial reuse such as one of the direct or indirect potable reuse (D/IPR) alternatives selected in Section 7 of this report, and a DIW will be constructed to serve as a backup disposal method. Alternatively, two DIWs may be used to dispose reclaimed water in lieu of offsite use and single DIW backup. Reclaimed water will be conveyed via the nonpotable plant service water (W3) pump station to provide service for onsite irrigation and other process water needs such as for cleaning screens, belt filter press (BFP) washwater, and hose bibbs.

5 Process Design Summary

5.1 Overall Process Concept

The treatment process will begin with preliminary treatment in a headworks structure consisting of fine screening and grit removal. Screened and degritted wastewater will flow by gravity to a suspended growth activated sludge treatment process consisting of a 5-Stage Bardenpho Process in an oxidation ditch configuration (process bioreactor) and circular secondary clarifiers. RAS will be conveyed from the clarifiers back to the process bioreactors directly through the process bioreactor splitter box or to the headworks and passing through the screens and grit removal processes. Clarified effluent will flow by gravity to a Filter Feed Pump Station, which will lift the flow to the deep bed filters. The filtered effluent will flow by gravity to be disinfected by sodium hypochlorite in chlorine contact basins (CCBs). The treated wastewater will then be pumped by the Effluent Transfer Pump Station to reclaimed water storage, or to reject water storage for retreatment if it does not meet treatment standards. Two dedicated reclaimed water storage tanks will be constructed on the east side of the site, close to the Reuse and DIW Pump Stations, to provide 1 day of AADF storage at buildout. Two reject water storage tanks will be constructed south of the headworks, with a third tank at buildout, to provide 1 day of AADF storage at all phases. The reclaimed water will be pumped to offsite beneficial reuse or to the DIW as backup by the Reuse and DIW Pump Stations.

WAS will be pumped from the clarifiers to aerobic digesters before final dewatering and disposal in trailers. The dewatered biosolids will be hauled offsite to either a landfill for disposal or to the County's compost facility for beneficial reuse. Filter backwash, filtrate from the dewatering process, and other process recycle streams will be pumped to the headworks by a Plant Drain Pump Station for treatment.

5.2 Reliability and Redundancy

The SEAWRF will be designed to meet a minimum of Class I reliability as defined by the U.S. Environmental Protection Agency (EPA) and referenced in FAC 62-610.300(1)(c). A summary of the reliability and redundancy requirements for vital components of the treatment facility are summarized in Table 5-1. The requirements are based on those in *Design Criteria for Mechanical, Electric, and Fluid System and Component Reliability* (EPA 1974). A vital component is defined as "a component whose operation or function is required to prevent a controlled diversion, is required to meet effluent limitation, or to protect other vital components from damage." Vital components include influent screening, grit removal, aeration, secondary clarification, RAS pumping, filtration, disinfection, and effluent pumping.

Class I reliability is required for facilities that treat for public-access reuse quality water (FAC 62-610.462). However, Class I reliability is not required if a permitted alternate treatment or discharge system exists that has sufficient capacity to handle any reclaimed water that does not meet public-access reuse standards. Although the DIW will satisfy the alternate discharge system requirements, Class I reliability is the desired standard for design and will be met. A minimum of two backup power generators (including one redundant) will be provided, which exceeds the Class I requirements.

Table 5-1. Reliability Criteria Summary for Class I Reliability

Unit Process or Operation	Reliability Criteria
Screens	Required; provide manually cleaned backup bar screen for mechanically cleaned screen
Grit removal	Required
Provisions for removal of settled solids	Required for all components, channels, pump wells, and piping before degritting
Unit operation bypass	Not applicable where two or more units are provided and operating unit can handle peak flow

Unit Process or Operation	Reliability Criteria
Pumps	Provide a backup pump that performs the same function for each set of pumps; with the largest pump out of service, remaining pumps must have the capacity to handle peak flow
Aeration basins	Minimum of two of equal volume
Aeration blowers	Multiple units; with largest unit out of service, the remaining units must be able to maintain design oxygen transfer; a backup unit may be uninstalled
Air diffusers	Multiple sections; with largest section out of service, oxygen transfer capability will not be measurably impaired
Final sedimentation basins (secondary clarifiers)	Multiple basins; with largest unit out of service, remaining units have capacity for at least 75% of design flow
Filters	Multiple units; provide minimum design capacity of at least 75% of design flow with one unit out of service
Disinfectant contact basins	Multiple basins; with largest unit out of service, the remaining units must have the capacity for at least 50% of design flow
Alternate methods of residuals disposal or treatment	Required for residuals treatment unit operations without installed backup capacity
Residuals holding tanks	Permissible as alternative to backup capability if adequate capacity for expected time of repair of component is provided; for continuous operations, excess capacity of downstream components will be provided for retained residuals as well as normal residual flow
Residuals pumps	Backup pump required but may be uninstalled; at least two pumps will be installed; with one pump out of service, remaining pumps will have capacity to handle peak flow
Aerobic digestion	Backup basin not required
Aerobic digester blowers	At least two units; permissible for less than design oxygen transfer with one unit out of service; backup unit may be uninstalled
Dewatering equipment	Multiple units with capacity to dewater design residuals flow with largest capacity unit out of service; backup may be uninstalled; additional equipment may not be required if installed equipment is operated less than 24 hours per day and normal operating hours can be extended on remaining units to make up capacity lost if unit is out of service
Power source	Provide two separate and independent power sources from either two separate utility substations or from one substation and one standby generator, with the backup power source sufficient to operate all vital components, including critical lighting and ventilation, during peak flow conditions; grit removal and sludge processing components are optional for backup power

Source: EPA 1974.

5.3 Biological Process Modeling

5.3.1 Basis of Process Modeling

Jacobs developed a treatment process model using the proprietary Pro2D2 software for sizing the process units in the first phase (6 mgd AADF) of the SEAWRF. This consisted of running steady-state process model simulations to confirm the selected treatment process unit sizes to produce the desired effluent quality at all design influent flows and mass load conditions. Section 2 of this report summarizes the design influent flows and mass loads for the SEAWRF that were used as the influent inputs into the process model. Other process model parameters were either kept at their default value or altered based on engineering judgement.

The main process units proposed for Phase 1 of the SEAWRF are summarized as follows:

- Three 2.2-million-gallon (MG) oxidation ditches, each having two surface aerators
- Three 110-foot diameter secondary clarifiers
- Four deep bed effluent filters with a total surface area of 3,000 square feet
- Two aerobic sludge storage tanks with a total volume of 1.2 MG

5.3.2 Key Process Modeling Results

The proposed activated sludge train treatment capacity was evaluated at a mixed liquor suspended solids (MLSS) of approximately 3,700 mg/L and a total system solids residence time (SRT) of 12 days at the maximum month average day (MMAD) influent loading conditions. The design aerobic SRT of 8 days has been proposed, which provides a significant nitrification safety factor. Table 5-2 summarizes the temperature and SRT inputs to the Pro2D2 process model, and Table 5-3 summarizes Pro2D2 model results. All the effluent water quality results meet AWT and HLD requirements for reclaimed water.

Table 5-2. Design Temperatures and Solids Residence Time

Parameter	Value
Minimum temperature (°C)	20
Maximum temperature (°C)	30
Overall SRT (days)	14.3
Aerobic SRT (days) (estimated)	8.58
Minimum aerobic SRT for nitrification at minimum temperature (days)	1.79
Nitrification SRT safety factor	4.79
Net yield (mg TSS/mg $CBOD_5$)	0.9
Sludge volume index (mL/g)	150

°C = degree(s) Celsius

mg = milligram(s)

mL/g = milliliter(s) per gram

Table 5-3. Process Modeling Summary for New Treatment Trains - Phase 1

Parameter	AAD[a]	MMAD[a]	MD[b]	Peak Hour
Total process reactor volume (MG)	6.6	6.6	6.6	6.6
Aerobic volume total (MG)	4.24	4.24	4.24	4.24
Aerobic volume per train (MG)	1.41	1.41	1.41	1.41
Preanoxic volume total (MG)	1.0	1.0	1.0	1.0
Preanoxic volume per train (MG)	0.33	0.33	0.33	0.33
Postanoxic volume total (MG)	0.70	0.70	0.70	0.70
Postanoxic volume per train (MG)	0.23	0.23	0.23	0.23
Anaerobic volume total (MG)	0.66	0.66	0.66	0.66
Anaerobic volume per train (MG)	0.22	0.22	0.22	0.22
Number of trains	3	3	3	3
Temperature (°C)	20	20	20	20
SWD (feet)	15	15	15	15
SRT (days)	12	12	12	12
MLSS (mg/L)	2,691	3,696	3,696	3,696

Parameter	AAD[a]	MMAD[a]	MD[b]	Peak Hour
MLVSS (mg/L)	1,766	2,462	2,462	2,462
Food/mass ratio (pounds $CBOD_5$/pounds MLVSS/day)	0.13	0.11	0.16	n/a
Net yield (pounds TSS/pound $CBOD_5$ removed)	0.93	0.92	0.98	n/a
HRT (hours)	23.3	20.1	19.8	n/a
Design oxygen concentration in aerated portion of basins (mg/L)	2.0	2.0	2.0	n/a
Design AOR (pounds per day) (20°C/30°C)	20,347/ 20,957	28,473/ 29,339	34,789/ 36,975	n/a
Design SOR (pounds per day) (20°C/30°C)	40,694/ 41,914	56,946/ 58,678	69,578/ 73,950	n/a
Aerator type	Low-speed surface aerators	Low-speed surface aerators	Low-speed surface aerators	Low-speed surface aerators
Standard aeration efficiency (pounds per horsepower-hour)	3.5	3.5	3.5	3.5
Field aeration efficiency (pounds per horsepower-hour)	2.3	2.3	2.3	2.3
Number of aerators per train	2	2	2	2
Aerator power (hp for each)	150	150	150	150
Number of secondary clarifiers	3	3	3	3
Secondary clarifier diameter (feet)	110	110	110	110
Secondary clarifier surface area (square feet for each)	9,503	9,503	9,503	9,503
Clarifier hydraulic loading rate (gpd per square foot) (all units in service)	239	281	428	660
Clarifier hydraulic loading rate (gpd per square foot) (one unit out of service)	358	421	642	990
RAS rate (range of 50% to 100% of MMADF)	100	100	100	n/a
Sludge volume index (mL/g)	150	150	150	150
Clarifier solids loading rate (pounds per day per square foot) (all units in service)	13	17	22	n/a
Clarifier solids loading rate (pounds per day per square foot) (one unit out of service)	19	26	33	n/a
Limiting solids loading rate (pounds per day per square foot)	43	43	43	43
WAS (pounds per day)	11,572	16,006	24,318	n/a
Filter hydraulic loading rate (gpm per square foot) (all units in service)	1.56	1.84	2.81	4.34
Filter hydraulic loading rate (gpm per square foot) (one unit out-of-service)	2.08	2.45	3.75	5.78
Filter solids loading rate (pounds per day per square foot) (all units in service, assuming maximum of 45 mg/L secondary effluent TSS)	0.84	0.99	1.52	2.34
Filter solids loading rate (pounds per day per square foot) (one unit out of service, assuming maximum of 45 mg/L secondary effluent TSS)	1.12	1.32	2.02	3.13

Parameter	AAD[a]	MMAD[a]	MD[b]	Peak Hour
Methanol addition to post-anoxic (gpd)	167	334	500	n/a
Effluent Values				
BOD_5 (mg/L)	1.4	1.5	1.8	n/a
TSS (mg/L)	2.5	2.4	2.4	n/a
TKN (mg/L)	1.2	1.3	1.7	n/a
NH_3-N (mg/L)	0.5	0.6	0.7	n/a
NO_3-N (mg/L)	1.8	1.5	1.9	n/a
TN (mg/L)	2.94	2.84	3.59	n/a
TP (mg/L)	0.2	0.2	0.1	n/a

[a] Alkalinity is set at 250 mg/L.
[b] Nitrification is alkalinity limited at MD conditions; alkalinity was increased from 250 mg/L to 400 mg/L.
AOR = actual oxygenation rate
gpd = gallon(s) per day
hp = horsepower
HRT = hydraulic residence time
MLVSS = mixed liquor volatile suspended solid
NO_3-N = nitrate as nitrogen
SOR = standard oxygen requirement
SWD = side water depth

Figure 5-1 presents the overall process flow diagram used for the SEAWRF mass balances. Tables 5-5 and 5-6 present the mass balances for both MMAD and AAD influent flows and loads at 20°C. The developed Pro2D2 model was used for the mass balance calculations.

5.4 Hydraulic Profile

Figure 5-2 presents the preliminary hydraulic profile based on the preliminary site plan to estimate yard piping distances between processes. The hydraulic profile presents two scenarios: Phase 1 startup flow and Phase 3 buildout PHF with one unit per process out of service. The hydraulic modeling program WinHydro was used to calculate the water surface elevations throughout the facility for the scenarios. Friction losses in piping, conduits, and open channels were obtained using Manning's equation, using a roughness coefficient of 0.013. The Phase 3 PHF with one unit out of service scenario establishes the maximum water level condition in the channels and basins. This scenario was also used to size the yard piping, weirs, and channels based on ensuring flow velocities and headlosses were not excessive. In addition, based on the maximum water level, the top of wall elevations were calculated to ensure that a minimum of 2 feet of freeboard is provided for all hydraulic structures. The startup flow scenario was used to ensure that pipe and channel flow velocities did not drop below 1 foot per second in all flows upstream of grit removal to prevent solids from settling. The hydraulic profile demonstrates that gravity flow will be provided from the headworks to the filter influent pump station and then from the filters to chlorine contact basins. The maximum water surface elevation at the headworks influent channel was calculated to be approximately 56.5 feet as compared to 37.5 feet at TOWRF and 53.3 feet at Gateway WRF in NAVD88.

The following assumptions were made for this analysis:

- A maximum Phase 3 PHF of 31.4 mgd is conveyed through the headworks (including plant drain recycle); RAS flows up to 12 mgd during wet weather events are diverted downstream of the headworks to the process bioreactor splitter box.

Figure 5-1. Lee County SEAWRF Overall Process Flow Diagram

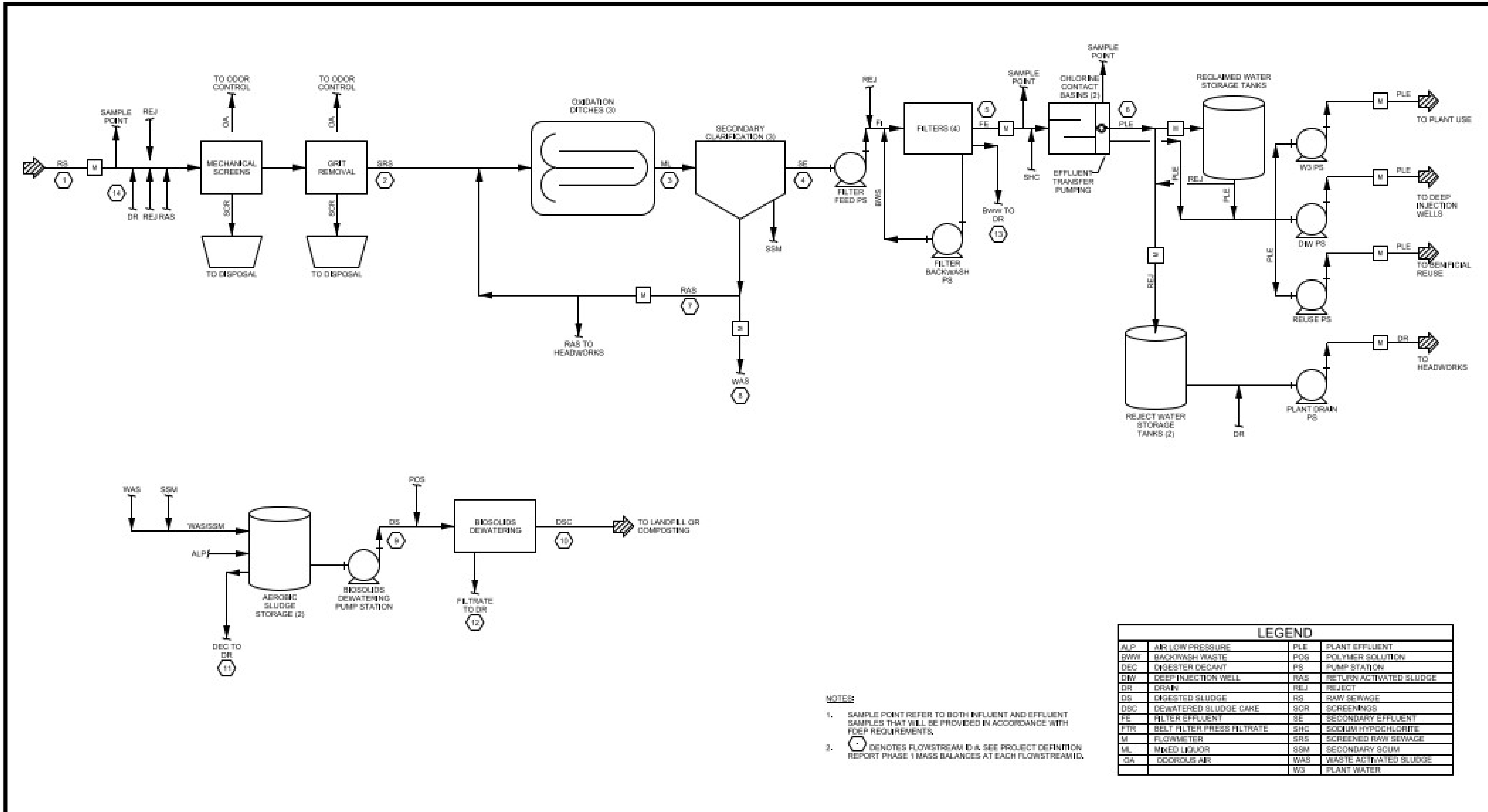
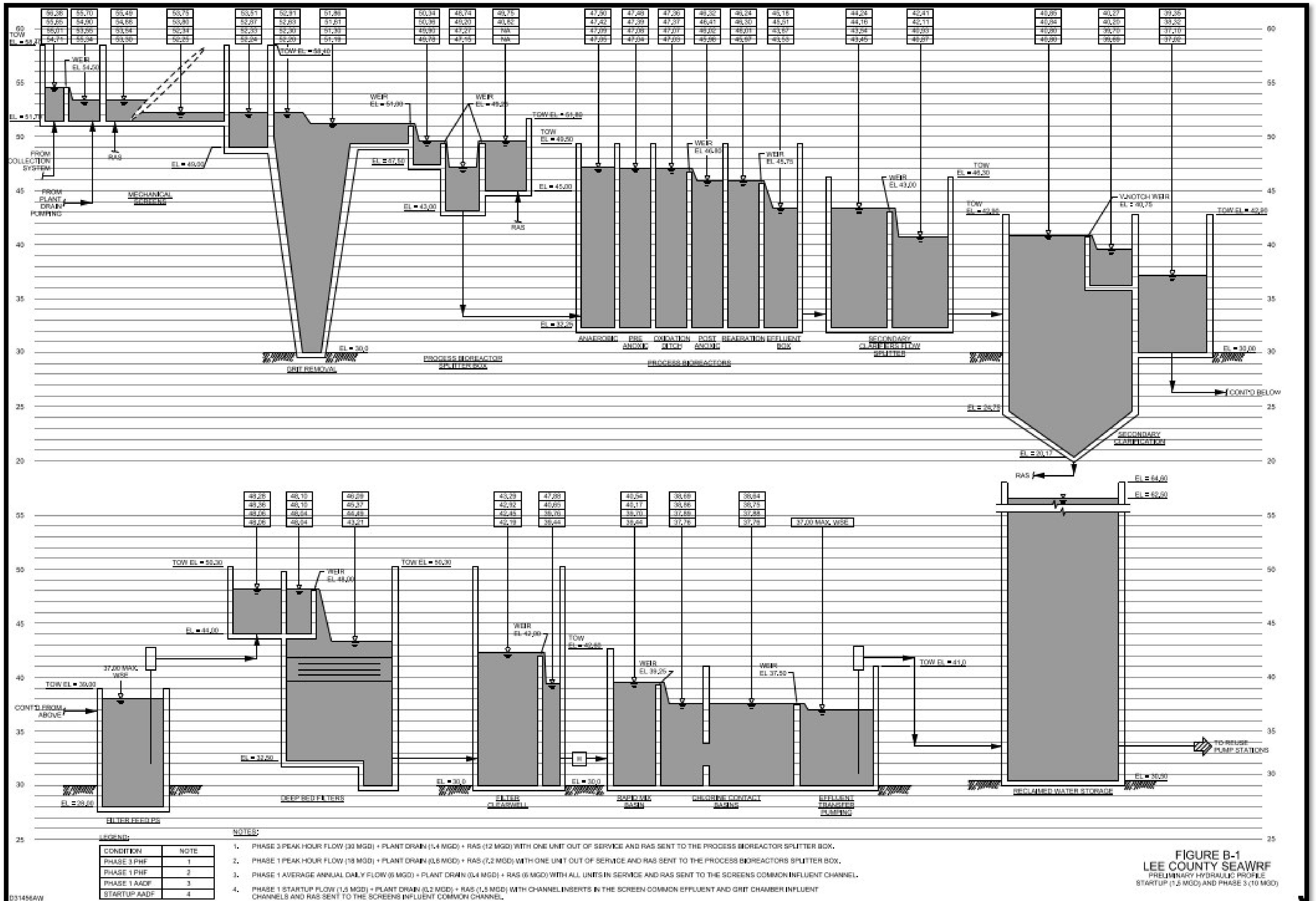


Figure 5-2. Lee County SEAWRF Preliminary Hydraulic Profile



- The Phase 1 startup flow is 1.5 mgd. The screens common effluent and grit chamber influent headworks channels will be initially constructed with inserts to reduce the widths of the channels during startup. This will be done to ensure the flow velocities in the channels stay above 1 foot per second during startup upstream of grit removal. The channel inserts will be easily removable so that the width of the channels can be increased once the wastewater influent flows increase.
- There will be proportional flow splitting to the process bioreactors, secondary clarifiers, deep bed filters, and chlorine contact basins.
- For the PHF with one unit out of service scenario, the following number of units are in service during Phase 3:
  - Three of four mechanical screens
  - One of two grit chambers
  - Four of five process bioreactors
  - Three of four secondary clarifiers
  - Five of six deep bed filters
  - Two of three chlorine contact basins
- The grade elevation will be 30.0 feet throughout the SEAWRF.
- Elevations were selected to reduce below grade structures because of the high groundwater table.
- Proposed major yard piping for the SEAWRF at the 10 mgd AADF Phase 3 Buildout includes the following:
  - Five 30-inch mixed liquor pipes from the process bioreactor splitter box at the headworks to each process bioreactor
  - Five 30-inch mixed liquor pipes from the effluent of the process bioreactors to the secondary clarifier splitter box
  - Four 30-inch mixed liquor pipes from the secondar clarifier splitter box to the four secondary clarifiers
  - Four 30-inch secondary effluent pipes from the four secondary clarifiers that combine into a 42-inch pipe to the Filter Feed Pump Station

5.5 Flow Metering and Sampling

Continuous flow metering and automatic sampling will be provided in accordance with FDEP regulations and to facilitate plant operations. The proposed flow metering locations are listed in Table 5-4. These flow readings will provide Lee County with information to meet regulatory requirements and maintain consistent operations. A bypass will be provided for all critical flowmeters that can be used with a strap-on meter if the primary flowmeter is out of service for maintenance. All flowmeters will be easily accessible and aboveground with considerations for removal of entrained air where necessary.

Table 5-4. Flow Metering Provisions

Flow Stream	Location	Measurement Method
Raw sewage	Headworks	Magnetic flowmeter
Plant drain – combined	Headworks	Magnetic flowmeter
RAS	RAS pump station discharge	Magnetic flowmeter
WAS	WAS pump discharge	Magnetic flowmeter
Dewatering	BFP feed line	Magnetic flowmeter
Process bioreactor feed	Yard piping	Magnetic flowmeter
Secondary effluent	Filter Feed Pump Station	Magnetic flowmeter
Filter effluent	CCB	Magnetic flowmeter

Flow Stream	Location	Measurement Method
Reject	Effluent Transfer Pump Station	Magnetic flowmeter
Reject return	Reject return pipe to Plant Drain Pump Station	Magnetic flowmeter
Plant effluent	Effluent Transfer Pump Station	Magnetic flowmeter
Plant effluent – beneficial reuse	Reuse Pump Station	Magnetic flowmeter
Plant effluent – DIW discharge	DIW aboveground piping	Magnetic flowmeter

Specific sampling requirements and locations will be determined during the permitting phase with FDEP. However, the following conditions will be incorporated into the design:

- A 24-hour flow proportioned composite sample of the raw influent wastewater will be collected prior to any recycle streams.
- There will be continuous oxidation reduction potential (ORP), DO, ammonia, and nitrate sampling in the process bioreactors.
- Continuous online sampling and analysis of the filter effluent TSS (or turbidity as a surrogate) and TP will be provided prior to disinfection.
- There will be continuous online monitoring of all parameters required for HLD, including pH and total residual chlorine. Off-specification effluent will automatically be pumped to reject water storage.
- Additional sampling required for vendor performance or chemical dosing will be coordinated during subsequent design phases.
- A 24-hour flow proportioned composite sample of the plant effluent will be collected.

5.6 Recycle Flows

In an oxidation ditch, the flow is circulated continuously around a circular racetrack-like open channel and typically designed to maintain a flow velocity of 1 foot per second. Flow control gates located at the influent of the internal recycle channels are used to recirculate a fraction of the flow back to the preanoxic basins. Because of the continual recirculation of flow, and the presence of the controlled gates in the oxidation ditch, there is no need for nitrified recycle pumping. Based on maintaining a flow velocity of 1 foot per second and selecting the channel water depth and width, the recirculation flows within the oxidation ditch, and between the oxidation ditch and preanoxic basins, are estimated.

Plant drain recycle flows will consist of filter backwash waste, decant from the aerobic digesters, filtrate from dewatering, and other miscellaneous basin and process drains. Plant drain flows will be pumped from a Plant Drain Pump Station to the headworks downstream of the influent sample location.

5.7 Overall Process Control Strategy

The process control strategy will include a combination of manual and automatic processes with monitoring available for all major equipment and processes at the control room. The process control strategy includes automatic controls and monitoring to limit the SEAWRF from producing off-specification effluent water. Some key aspects of the process and equipment strategy include the following:

- Automatic cleaning of the in-service mechanical screens based on level control; additional control features will be evaluated to maintain equal operation between the screens
- Automatic control of the aerator speed within the process bioreactors based on DO, ORP, nitrate, and ammonia measurement (part of oxidation ditch vendor package)

- Filter backwashing can be manually initiated by the operator or automated by a timer based on differential pressure within the filters media, but the filter backwash sequence will be automated; metering of backwash waste flow to the plant drain system will also be controlled automatically
- Automatic control of actuated valves at the Effluent Transfer Pump Station for automatic transfer of off-specification water to reject storage when any online instrumentation (for example, TSS or pH) indicates permit criteria are not being met
- Automatic level sensors in all tanks receiving pumped or gravity flow for storage (reclaimed water storage, reject water storage, aerobic digesters) and controls to alarm and automatically stop pumping or diversion to tanks that are at capacity
- Automatic level sensors in pump station wet wells to control pump operation
- Automatic control and metering of reject return flow to the Plant Drain Pump Station
- Automated control of pumping of reclaimed water to the D/IPR alternative selected in Section 7 or to the DIW

5.8 Future Considerations

Depending on the chosen method of primary reclaimed water use, the SEAWRF may or may not need to always meet AWT standards. When a facility is permitted to meet AWT standards, supplemental carbon and alum storage facilities are typically included to ensure the facility meets AWT standards for all conditions. Preliminary equipment sizing for these facilities is included in Section 6 of this report, but the need for these facilities to be constructed during Phase 1 will be discussed further with LCU once the D/IPR alternative is selected. In addition, it was noted during the process modeling of the MDL scenario that supplemental alkalinity was needed to fully nitrify. This, as well as whether supplemental alkalinity storage and feed systems have been required at LCU's other facilities, will also be discussed with LCU.

Table 5-5. Mass Balance for Annual Average Daily Flows and Loads at 20°C

Flow Stream			Flow	BOD_5		TSS		VSS		TKN		NH_3-N		NO_3-N		TN		TP	
#	ID	Description	mgd	mg/L	Pounds per day	mg/L	Pounds per day	mg/L	Pounds per day	mg/L	Pounds per day	mg/L	Pounds per day	mg/L	Pounds per day	mg/L	Pounds per day	mg/L	Pounds per day
1	RS	Raw sewage	6	240	12,010	240	12,010	192	9,608	50	2,502	41	2,052	0	0	50.0	2,502	5.8	290
2	SRS	Screened raw sewage	6.8	219	12,410	252	14,300	195	11,059	46	2,629	36	2,068	2	99	48.0	2,728	8.7	495
3	ML	Mixed liquor	13.6	775	88,029	2,686	304,974	1,762	200,019	127	14,459	0	56	2	201	129.1	14,659	108.0	12,261
4	SE	Secondary effluent	6.5	5	274	15	818	10	537	2	96	0	27	2	96	3.5	193	0.7	37
5	FE	Filtered effluent	6	1	70	2	123	2	80	1	59	0	24	2	88	2.9	147	0.2	9
6	PLE	Plant effluent	6	1	70	2	123	2	80	1	59	0	24	2	88	2.9	147	0.2	9
7	RAS	RAS	6.8	1,487	84,341	5,154	292,322	3,381	191,720	243	13,804	0	28	2	100	245.2	13,904	207.2	11,748
8	WAS	WAS	0.22	1,487	3,327	5,154	11,531	3,381	7,562	243	544	0	1	2	4	245.2	548	207.2	463
9	DS	Digested sludge	0.09	1,381	1,062	11,188	8,602	6,979	5,366	525	403	7	5	41	32	702.9	435	464.8	357
10	DSC	Dewatered sludge cake	0.005	20,990	956	170,000	7,742	106,049	4,829	7,873	359	7	0	41	2	7,914.2	360	6,298.6	287
11	DEC	Digester decant	0.12	62	91	500	734	312	458	30	44	7	10	41	61	71.6	105	72.2	106
12	FTR	BFP filtrate	0.09	147	106	1,189	860	742	537	62	45	7	5	41	30	103.5	75	97.5	71
13	BWW	Filter backwash waste	0.54	45	204	154	695	101	456	8	37	0	2	2	8	10.1	45	6.3	28
14	DR	Recycle combined discharge	0.80	60	400	342	2,290	217	1,451	19	127	2	16	15	99	33.7	226	30.6	205

VSS = volatile suspended solids

Table 5-6. Mass Balance for Maximum Month Average Day Flows and Loads at 20°C

Flow Stream			Flow	BOD5		TSS		VSS		TKN		NH3-N		NO3-N		TN		TP	
#	ID	Description	mgd	mg/L	Pounds per day	mg/L	Pounds per day	mg/L	Pounds per day	mg/L	Pounds per day	mg/L	Pounds per day	mg/L	Pounds per day	mg/L	Pounds per day	mg/L	Pounds per day
1	RS	Raw sewage	7.2	280	16,813	280	16,813	224	13,450	58	3,503	48	2,872	0	0	58.3	3,503	6.8	406
2	SRS	Screened raw sewage	7.9	264	17,322	298	19,564	232	15,215	56	3,682	44	2,917	2	112	57.8	3,794	10.0	657
3	ML	Mixed liquor	15.7	1,126	147,877	3,691	484,578	2,458	322,712	178	23,333	1	80	1	197	179.2	23,529	139.5	18,318
4	SE	Secondary effluent	7.6	5	339	15	951	10	633	2	123	1	38	1	95	3.4	218	0.7	42
5	FE	Filtered effluent	7.2	1	90	2	143	2	95	1	80	1	36	1	90	2.8	170	0.2	11
6	PLE	Plant effluent	7.2	1	90	2	143	2	95	1	80	1	36	1	90	2.8	170	0.2	11
7	RAS	RAS	7.9	2,173	143,167	7,123	469,298	4,744	312,536	342	22,522	1	40	1	99	343.3	22,620	269.2	17,734
8	WAS	WAS	0.27	2,173	4,883	7,123	16,006	4,744	10,660	342	768	1	1	1	3	343.3	772	269.2	605
9	DS	Digested sludge	0.09	2,062	1,623	15,406	12,125	9,723	7,652	743	585	20	16	49	39	964.3	623	606.2	477
10	DSC	Dewatered sludge cake	0.008	22,750	1,460	170,000	10,913	107,288	6,887	7,994	513	20	1	49	3	8,043.0	516	5,985.3	384
11	DEC	Digester decant	0.17	67	98	500	730	316	461	44	64	20	29	49	71	92.9	136	87.5	128
12	FTR	BFP filtrate	0.09	224	162	1,677	1,213	1,059	765	99	72	20	14	49	35	148.1	108	128.5	93
13	BWW	Filter backwash waste	0.40	74	249	239	808	159	538	13	43	1	2	1	5	14.2	48	9.1	31
14	DR	Recycle combined discharge	0.67	92	509	495	2,751	317	1,764	32	179	8	45	20	112	52.2	291	45.2	251

6 Treatment Facilities

A description of the major facilities associated with wastewater treatment, storage, and disposal for the new SEAWRF are discussed in the following sections. Design criteria for sizing the proposed facilities were developed from the guidelines recommended in the Water Environment Federation Manual of Practice No. 8 (WEF 2018), Ten State Standards (Wastewater Committee of the Great Lakes 2014), EPA design manual (1974), and engineering experience. Phasing of unit processes and equipment is described for planning purposes. The scope of project design is Phase 1 only. Preliminary layouts are developed to convey the design concepts. Final equipment design, selection, and layout are subject to change depending on a variety of factors that affect these types of capital projects as the design progresses.

6.1 Headworks

6.1.1 Description of Proposed Facility

The headworks will receive wastewater directly from lift stations within the collection system in Phase 1 through buildout. Therefore, as the new SEAWRF collection system is designed and constructed, LCU will need to ensure that lift station pumps sending wastewater to the SEAWRF will have sufficient discharge pressure to reach the headworks influent channel. This will prevent the need for a master lift station. The headworks facility will consist of a sewage influent box, a screen influent channel, screen channels, mechanically cleaned screens with screenings washer/compactor, grit removal basins, grit pumps, grit separators and dewatering equipment, and a screen bypass channel. A piped bypass from the influent channel will also be provided around the screens to the grit chamber effluent channel for maintenance purposes. Slide gates will be used for channel isolation. The top of the gates for the bypass channel will be set such that the bypass channel will provide a passive bypass if the duty screen(s) fail. The bypass channel will include a manually cleaned bar screen with 1-inch openings. The headworks channels, screens, grit basins, and splitter box will be covered for odor control.

Screenings will drop into a washer/compactor that will auger the screenings to a drop chute. Washed and compacted screenings will fall by gravity through the chute to a dumpster at grade. The dumpsters will be in an enclosed area with rollup doors below the headworks structure.

The screened raw sewage will then be introduced tangentially to a nonmechanical, stacked tray vortex type grit removal basin. Grit separated from the wastewater will drop by gravity to the bottom center cone. Settled grit will be pumped from the collection cone by recessed-impeller grit pumps to a grit washing/separating cyclone. Separated grit will then drop into a dewatering unit that will gently convey the grit to a chute, where it will drop by gravity to the screening dumpster. Drainage from the grit cyclone and classifier and washer/compactor will be returned directly to the headworks influent channel.

Degritted wastewater will flow over an effluent weir to an effluent channel. Screened and degritted wastewater will flow to a process bioreactor splitter box connected to the headworks, where it will be split to the downstream process bioreactors (oxidation ditches). RAS from the secondary clarifiers may be pumped upstream of the screens or to the splitter box through flow-splitting weirs depending on operator preference. Plant drain flows and reject return will also combine with the raw wastewater upstream of the screens, but downstream of the influent sampler and flowmeter.

6.1.2 Design Criteria

The headworks will be constructed as a single structure in Phase 1, accommodating buildout capacity. Channel widths will be narrowed for the first phases with temporary blocking or other inserts that may be easily removed to accommodate expansion flows to keep acceptable channel velocities. Design criteria are summarized in Table 6-1.

Table 6-1. Headworks - Design Criteria

Parameter	Value
Screen opening	6-millimeter perforated plate
Screen hydraulic capacity per screen	10.5 mgd PHF
Type of screen	Mechanically cleaned – continuous flow element
Manufacturers (screens and washer/compactors)	Kusters, Hydro-Dyne
Screen materials of construction	316 SST
Screen inlet channel velocity	1.5 (AADF) – 3.5 feet per second (PHF)
Screenings washer/compactor	30% dry solids
Screenings quantity	7 to 10 cubic feet per million gallons
Side gate materials of construction	316 SST
Channel covers	Aluminum (Hallsten)
Grit removal type	Stacked tray
Grit basin hydraulic capacity	31.4 PHF
Grit basin removal efficiency[a]	95% of grit > 100 micron
Grit pump capacity	Firm capacity with one unit out of service
Grit inlet channel velocity	2 to 3 feet per second at AADF
Grit pipe fluid velocity	Minimum 4 feet per second
Grit pipe material	Glass-lined ductile iron pipe
Grit cyclone/classifier type	Centrifugal
Grit quantity	5 to 10 cubic feet per million gallons

[a] Headcell will be limited to a 110-micron cut point for the Phase 3 PHF of 31.4 mgd.

> = greater than

SST = stainless steel

Preliminary equipment sizing and phasing are summarized in Table 6-2. The number of units described is the total installed, including previous phases. Motor sizes are estimated at this level of design.

Table 6-2. Headworks - Preliminary Equipment Sizing

Parameter	Phase 1	Phase 2	Buildout
Number of mechanical screens	2 duty + 1 standby	3 duty + 1 standby	3 duty + 1 standby
Capacity per screen	10.5 mgd	10.5 mgd	10.5 mgd
Installed screen capacity	31.5 mgd	42 mgd	42 mgd
Firm screen capacity	21 mgd	31.5 mgd	31.5 mgd
Screen motor	2 hp	2 hp	2 hp
Number of manual screens (in bypass channel)	1	1	1
Number of washer/ compactors	2 duty + 1 standby	3 duty + 1 standby	3 duty + 1 standby
Washer/compactor motor	5 hp	5 hp	5 hp
Number of grit basins	1 duty + 1 standby	1 duty + 1 standby	1 duty + 1 standby
Capacity per grit basin	18.9 mgd	25.1 mgd	31.4 mgd
Installed grit basin capacity	37.8 mgd	50.2 mgd	62.8 mgd
Firm grit capacity	18.9 mgd	25.1 mgd	31.4 mgd
Number of grit pumps	1 duty + 1 standby	1 duty + 1 standby	1 duty + 1 standby

Parameter	Phase 1	Phase 2	Buildout
Grit pump capacity (for each)	200 gpm	200 gpm	200 gpm
Estimated grit pump head	45 feet	45 feet	45 feet
Grit pump motor	7.5 hp	7.5 hp	7.5 hp
Number of grit cyclones	1 duty + 1 standby	1 duty + 1 standby	1 duty + 1 standby
Number of grit classifiers	1 duty + 1 standby	1 duty + 1 standby	1 duty + 1 standby

6.1.3 Process Control

The mechanical screens will be cleaned automatically with a cleaning cycle initiated by headloss across the screen or by timer. Washer/compactors will automatically turn on when a cleaning cycle is initiated and run for a preset amount of time after a cleaning cycle is complete. Grit pumps will be operated continuously or on a timer. Valves for directing RAS flow upstream of the screens, or directly to the process bioreactor splitter box, will be manually operated. Gates for isolating screen channels or directing flow to the bypass will be manually operated. The gate to the bypass channel will be set below the top of the channel wall such that it would passively flow into the bypass channel if the water level in the channel rises due to failure of the mechanical screens. High water level will alarm the supervisory control and data acquisition (SCADA) system.

6.2 Odor Control

6.2.1 Description of Proposed Facility

Odorous air will be captured from the headwork screens, channels, grit removal processes, process bioreactor splitter box, and enclosed dumpster room. The odor control system will consist of biotrickling filters that are located near the headworks. The potential need for second stage iron oxide media or carbon filter will be evaluated as the design progresses. Odorous air fans will capture and convey the odorous air to the treatment system. The drain from the biotrickling filters will be collected in a sump and pumped to the headworks influent channel with fiberglass sump pumps.

6.2.2 Design Criteria

Odor control facilities will be constructed for buildout in Phase 1. Because the headworks will be constructed for buildout in Phase 1, the volume for treatment does not vary as much over time. Ventilation air quantity was estimated for each source using one of the following criteria:

- Air exchange rates – for unoccupied, nonaerated areas, acceptable air exchange rates range between 6 and 12 air changes per hour (ach)
- Capture velocity – sufficient ventilation to maintain 200 to 300 feet per minute inrush velocity of air through open vents

Other design criteria for the odor control system are summarized in Table 6-3. Design inlet and peak hydrogen sulfide concentrations were estimated based on matching those used for design of odor control facilities at the TOWRF. These values will be refined as the design progresses.

Table 6-3. Odor Control - Design Criteria

Parameter	Value
H_2S removal efficiency	99%
Average inlet H_2S concentration	100 ppmv
Peak inlet H_2S concentration	150 ppmv
Media	Proprietary plastic

Parameter	Value
Makeup water source	Plant water
Recirculation rate	90 gpm
Ductwork air velocity	2,500 to 3,500 feet per minute
Ductwork location	Aboveground
Materials of construction – ductwork	FRP
Fan internal materials of construction	FRP
Redundancy	One standby fan; two biotrickling filters at 50% capacity each

FRP = fiberglass reinforced plastic

H_2S = hydrogen sulfide

ppmv = parts per million by volume

Preliminary equipment sizing and phasing are summarized in Table 6-4. The number of units described is the total installed, including previous phases. Ventilation rates are rough estimates at this level of design.

Table 6-4. Odor Control - Preliminary Equipment Sizing

Parameter	Phase 1	Phase 2	Buildout
Headworks ventilation rate	4,953 SCFM	4,953 SCFM	4,953 SCFM
Headworks ventilation rate basis	12 ach	12 ach	12 ach
Total odorous airflow	4,053 SCFM	4,053 SCFM	4,053 SCFM
Number of biotrickling filters	2 duty + 0 standby	2 duty + 0 standby	2 duty + 0 standby
Firm capacity of biotrickling filters	TBD	TBD	TBD
Empty bed contact time	TBD	TBD	TBD
Bed volume per unit	TBD	TBD	TBD
Bed diameter	TBD	TBD	TBD
Minimum bed depth	TBD	TBD	TBD
Number of odorous air fans	2 duty + 1 standby	2 duty + 1 standby	2 duty + 1 standby
Odorous air fan capacity (for each)	2,030 SCFM	2,030 SCFM	2,030 SCFM
Installed fan capacity	6,090 SCFM	6,090 SCFM	6,090 SCFM
Firm fan capacity	4,060 SCFM	4,060 SCFM	4,060 SCFM
Static pressure (inches water column)	15 inches	15 inches	15 inches
Fan motor	25 hp, variable speed	25 hp, variable speed	25 hp, variable speed
Number of recirculation pumps	2 duty + 1 standby	2 duty + 1 standby	2 duty + 1 standby
Recirculation pump size	TBD	TBD	TBD
Number of sump pumps	TBD	TBD	TBD
Sump pump size	TBD	TBD	TBD

SCFM = standard cubic foot (feet) per minute

6.2.3 Process Control

Odorous air will be continuously ventilated from the sources previously described using variable speed fans. Isolation dampers will be installed within ductwork at all connections to process equipment and covered areas to facilitate balancing airflow. Once balanced, overall control of system-wide airflow will be maintained by measurement of flow rates entering the fans and manual adjustment of fan speed. Isolation dampers will be located before and after each fan and on each biotrickling filter to allow operation of any combination of biotrickling filters and fans. Operation of the biotrickling filter will be automatic, run by vendor-supplied controls. Measurement of air flow rates within individual ducts will be by a portable air velocity measurement device such as a hot wire anemometer.

6.3 Process Bioreactor Splitter Box

6.3.1 Description of Proposed Facility

Screened and degritted wastewater will be split in a flow-splitting structure that is covered and integral to the headworks. Wastewater flowing over the weir of the grit structure will flow into a grit chamber effluent channel. Downward opening weir gates on the side of the channel will be used to split the grit chamber effluent flow to the process bioreactors inlet boxes. The weir gate may be raised for isolation. The bioreactor inlet boxes will have another set of downward opening weir gates that split RAS flow from the RAS channel on the other side of the inlet boxes.

6.3.2 Design Criteria

The splitter box will be constructed for buildout, with a weir to each bioreactor. Table 6-5 summarizes the design criteria for the process bioreactor splitter box.

Table 6-5. Process Bioreactor Splitter Box - Design Criteria

Parameter	Value
Number of grit chamber effluent flow split weirs	Five, one for each bioreactor
Number of RAS split weirs	Five, one for each bioreactor
Weir gate materials of construction	316 SST
Type of gates	Downward-opening weir gates

6.3.3 Process Control

Control of the flow split will be passive by the number of weirs that are open and the length of the weirs. Normally each weir gate to the process bioreactor will be lowered to the same elevation for an even flow split, but they may be adjusted if desired to split the flow unevenly. Process bioreactor weir gate operation will be manual. Each bioreactor may be isolated by raising the downward opening weir gates.

6.4 Process Bioreactors

6.4.1 Description of Proposed Facility

The three proposed process bioreactors (oxidation ditches) will be constructed of concrete. Each basin will have a volume of 2.15 MG and will be provided with two platform-mounted surface aerators that will be equipped with variable frequency drives (VFDs). The screened and degritted raw sewage and RAS flow to each basin will be split at the process bioreactor splitter box attached to the headworks structure. Three basins will be constructed in Phase 1. Each oxidation ditch will be in a folded over racetrack configuration. A clearance of 25 feet will be kept between the oxidation ditches for crane access to facilitate maintenance.

The process bioreactors will be in a 5-stage Bardenpho configuration that includes the following zones: anaerobic, preanoxic, aerated, postanoxic, and reaeration. Mixing will be provided in the oxidation ditch channels by maintaining a minimum fluid velocity of 1 foot per second by the pumping action of the low-speed surface aerators. Multiple 8-inch drains will be provided in strategic locations, with the floor sloped to the extent possible to allow each basin to be drained easily. The area around each aerator will be covered by a concrete platform with side skirt to reduce potential for splashing or aerosol drift. The aerators will be designed such that they can be pulled vertically out of the basin without taking the basin out of service. Simultaneous nitrification and denitrification will be possible within the aerated portion of the oxidation ditches by controlling the aeration input and, therefore, the DO concentration along the channel. Mixed liquor will leave each basin by flowing over downward-acting weir gates. The aerators will be controlled by a combination of VFDs and effluent weir adjustment. Mixers will also be included in the unaerated portions, the anaerobic and anoxic zones, of the process bioreactors. The reaeration zones process bioreactors will be aerated by diffused aeration and positive displacement blowers.

6.4.2 Design Criteria

The process bioreactors will be constructed in phases, with three basins in Phase 1 and two more added at buildout. Design criteria for the process bioreactors are summarized in Table 6-6.

Table 6-6. Process Bioreactor - Design Criteria

Parameter	Value
Design MMAD MLSS concentration	3,700 mg/L
Total SRT (MMAD)	12 days
Type of aerator	Low-speed, platform-mounted surface aerators with VFDs and bypass soft starts with built-in internal bypass
Mixing criteria	1 foot per second minimum channel velocity
Basin freeboard	2 feet
Type of tank	Cast-in-place
Oxidation ditch manufacturers	Ovivo, Westech
Mixer manufacturers	Rotamix, TBD
Minimum DO in aerated portions	2.0 mg/L
Aerator redundancy	Maintain ability to transfer oxygen for design conditions with one aerator out of service
Safety factor on MDL aeration	1.15

Preliminary equipment sizing and phasing is summarized in Table 6-7. The number of units described is the total installed including previous phases.

Table 6-7. Process Bioreactor - Preliminary Sizing

Parameter	Phase 1	Phase 2	Buildout
Number of bioreactors	3	4	5
Volume per tank	2.2 MG	2.2 MG	2.2 MG
Total volume	6.6 MG	8.8 MG	11.0 MG
Tank SWD	15 feet	15 feet	15 feet
Number of passes per train	4	4	4
Width per pass	25 feet	25 feet	25 feet
Approximate tank dimensions (length by width) each	205 feet by 105 feet	205 feet by 105 feet	205 feet by 105 feet
Number of aerators per tank	2	2	2

Parameter	Phase 1	Phase 2	Buildout
Aerator motor hp	150 hp	150 hp	150 hp
Total installed aerator power	900 hp	1,200 hp	1,500 hp
Number of PD blowers	4	5	6
Blower hp	30	30	30
Number of mixers in anaerobic and anoxic zones	18	24	30
Mixer hp	15	15	15

PD = positive displacement

6.4.3 Process Control

The speed of the surface aerators will be controlled automatically in a lead-lag approach based on DO measurements taken in the process bioreactor. One DO probe and a redundant spare will be installed downstream of the first surface aerator and just upstream of the effluent weir in each process bioreactor. A minimum speed will also be set to ensure that solids stay in suspension within the process bioreactor. DO set points will be adjustable to optimize simultaneous nitrification and denitrification and to save energy. Aerator speed may also be set manually by the operator. The effluent weir gate will be manually adjustable, providing an alternate means of controlling the aeration by adjusting the amount of aerator submergence (typically a seasonal adjustment). However, the primary means of control will be through aerator speed adjustment. An ORP probe will be provided at each DO measurement location and in both the preanoxic and anaerobic zones. Ammonia and nitrate will also be continuously monitored from near the discharge via the SCADA system. Redundant DO probes will be provided for maintenance purposes. The flow rate to each process bioreactor will be measured by a magnetic flowmeter on each pipe conveying mixed liquor from the process bioreactor splitter box to the basin. Redundant hydrostatic level transducers will be provided on each basin to measure the basin level and amount of impeller submergence. Power monitors will be provided in the VFD cabinet for each aerator to measure the power consumption of the VFD and motor assembly.

6.5 Secondary Clarifier Flow Splitter

6.5.1 Description of Proposed Facility

Mixed liquor from the process bioreactors will be split in a flow-splitting structure that is separate from the bioreactor structure and open on top. Mixed liquor will be piped from a collection box that combines the flow from the bioreactors to the splitter box. Downward opening weir gates will be used to split the flow to the secondary clarifiers. Plates will be bolted over the gate openings to the future fourth clarifier. Each weir gate may be raised up to isolate a clarifier. Flow will enter the splitter box from the bottom.

6.5.2 Design Criteria

The splitter box will be constructed for buildout with a weir gate to each planned secondary clarifier (four total). Design criteria for the secondary clarifier flow splitter are summarized in Table 6-8.

Table 6-8. Secondary Clarifier Flow Splitter - Design Criteria

Parameter	Value
Number of flow split weir gates	Four
Type of flow split	Downward-opening weir gates
Weir gate materials of construction	316 SST

6.5.3 Process Control

Control of the flow split will be passive by the number of weir gates that are open and the length of the weirs. Normally the weir gates will be set at the same elevation to achieve an even flow split, but they may be adjusted if desired to send more flow to certain clarifiers. Each clarifier may be isolated by raising the weir gate. Gate operation will be manual.

6.6 Secondary Clarification

6.6.1 Description of Proposed Facility

Mixed liquor from the process bioreactors will be settled in secondary clarifiers. The secondary clarifiers will be circular with center feed, energy dissipating inlet, flocculating feedwell, and Stamford baffles. Clarified water will flow over a perimeter weir with V-notches into an inboard launder. A walkway will be provided around the outside perimeter of the clarifier. Clarifier mechanisms will be center pier supported with full radius scum skimmers and two scum arms. Scum will flow into a small scum well that is integral to the scum trough. A double-disk scum pump will pump the scum to the solids handling processes. RAS and WAS will be pumped from the bottom of the clarifier by the RAS/WAS pump station. Weir covers will be provided to reduce algae growth. The ability to route secondary clarifier effluent directly to the Effluent Transfer Pump Station to be pumped to reject water storage will be provided through a piped connection.

6.6.2 Design Criteria

Clarifiers will be constructed by phase. Three clarifiers will be constructed for Phases 1 and 2 to satisfy reliability and redundancy requirements. A fourth clarifier will be constructed at buildout. Firm RAS pumping capacity will be provided with one RAS pump out of service. Design criteria for the secondary clarifiers are summarized in Table 6-9. A concrete pad with electric outlet and service water supply will be provided for locating a polymer tote and polymer feeder for temporary usage to feed to the clarifier influent, if necessary.

Table 6-9. Secondary Clarifiers - Design Criteria

Parameter	Value
Type	Circular, center-feed, peripheral withdrawal
Mechanism type	Spiral rake with full-radius scum skimmer
Sludge collection	RAS hopper (no sludge rings)
Mechanism manufacturer	Ovivo, Evoqua, Westech
Mechanism materials of construction	Painted steel
Launder	In-board, concrete
Launder covers	FRP (NEFCO)
Baffle type	Stamford
Sludge volume index	150 mL/g
RAS rate	50% to 100% of MMADF
Hydraulic loading rate	< 500 gpd per square foot at MMADF < 1,000 gpd per square foot at PHF
Solids loading rate	< 20 pounds per day per square foot at MMADF < 43 pounds per day per square foot at MDF
MLSS	4,300 mg/L at MMADF
Weirs	V-notch, FRP
Scum pump type	Double disk (Penn Valley)

Parameter	Value
Scum pump capacity	200 gpm
Scum pump discharge head	30 feet
Scum pump motor	3 hp, constant speed

< = less than

Preliminary equipment sizing and phasing are summarized in Table 6-10. The number of units described is the total installed, including previous phases.

Table 6-10. Secondary Clarifier - Preliminary Equipment Sizing

Parameter	Phase 1	Phase 2	Buildout
Number of clarifiers	3	3	4
Diameter	110 feet	110 feet	110 feet
Clarifier area (each)	9,503 square feet	9,503 square feet	9,503 square feet
Clarifier area (total)	28,509 square feet	28,509 square feet	38,012 square feet
SWD	16 feet	16 feet	16 feet
Drive motor	1 hp	1 hp	1 hp
RAS rate			
AADF (percent of AADF)	75	75	75
MMADF (percent of MMADF)	100	100	100
MDF (percent of MMADF)	100	100	100
PHF (percent of MMADF)	100	100	100
Hydraulic loading rate (gpd per square foot)			
All units in service			
AADF	239	318	298
MMADF	281	374	351
MDF	428	571	535
PHF	660	880	825
Solids loading rate (pounds per day per square foot)			
All units in service			
AADF	15	20	19
MMADF	20	27	25
MDF	25	34	32
Hydraulic loading rate (gpd per square foot)			
One unit out of service			
AADF	358	477	398
MMADF	421	561	468
MDF	642	856	714
PHF	990	1,320	1,100
75% of MDF	482	642	535
75% of PHF	742	990	825
Solids loading rate (pounds per day per square foot)			

Parameter	Phase 1	Phase 2	Buildout
One unit out of service			
AADF	22	30	25
MMADF	30	40	34
MDF	38	51	42
75% of MDF	32	43	36
Number of scum pumps	3	3	4

6.6.3 Process Control

The clarifier mechanism will be manually operated with a local on/off switch. Overtorque protection will be provided with a remote and local alarm on high torque. Flushing water will be automatically actuated by the passage of the scum skimmer. Scum pumps will be operated automatically based on activation by a limit switch trigger by the passage of the clarifier arm and an operator adjustable timer for pump run time to pump out the contents of the scum trough and piping. A remote switch for local, manual operation will be provided for operation of the scum pump when hosing out a scum box.

6.7 Return Activated Sludge/Waste Activated Sludge Pumping

6.7.1 Description of Proposed Facility

Mixed liquor from the process bioreactors that is settled in secondary clarifiers will be pumped back to the process bioreactors by the RAS/WAS pump station, either through the headworks screens or directly to the process bioreactor splitter box, at the operator's choice. A portion of the mixed liquor will be sent to the aerobic digesters for dewatering and disposal to maintain the target SRT in the bioreactor.

The Phase 1 RAS/WAS pump station will be between secondary clarifiers 1 and 2, with a second pump station added between clarifier 3 and future clarifier 4. The pump stations will be a slab-on-grade with an open-sided canopy cover. The slab will be sloped to gravity drains that flow to the Plant Drain Pump Station and will collect vent piping discharge from air relief valves, washdown, sample sink drain, and mechanical seal water. Each RAS header will have a sample port and collection sink. The RAS pump station will be valved such that a pump is dedicated to each clarifier with a flowmeter but will combine into a common discharge header. One RAS pipe per two clarifiers will be installed to return sludge back to the headworks or process bioreactor splitter structure. A similar approach will be used for the WAS piping.

6.7.2 Design Criteria

Five RAS pumps and three WAS pumps will be constructed in Phase 1 to satisfy reliability and redundancy requirements for Phase 1 and 2. In Phase 3 an additional one RAS pump and one WAS pump would be constructed to serve clarifier 4. Firm RAS and WAS pumping capacity will be provided with two RAS pumps and one WAS pump out of service. The pumps will be provided with variable-speed drives to adjust the flow rate. Design criteria for the RAS/WAS pump station are summarized in Table 6-11.

Table 6-11. RAS/WAS Pump Station - Design Criteria

Parameter	Value
RAS pump type	Chopper pump, dry pit
RAS pump manufacturer	Vaughan
RAS rate	50% to 100% of MMADF
RAS control	Manually set or automatically flow paced with influent flow
WAS pump type	Chopper pump, dry pit
WAS pump manufacturer	Vaughan

Parameter	Value
WAS rate	Waste in a maximum of 12 hours per day at MMAD condition Phase 1: 0.269 mgd (187 gpm at 24 hours per day, 7 days per week wasting) at MMAD and 100% RAS rate

Preliminary equipment sizing and phasing are summarized in Table 6-12. The number of units described is the total installed, including previous phases.

Table 6-12. RAS/WAS Pump Station - Preliminary Equipment Sizing

Parameter	Phase 1	Phase 2	Buildout
Number of clarifiers	3	3	4
Number of RAS pumps	3 duty + 2 standby	3 duty + 2 standby	4 duty + 2 standby
RAS pump capacity (to downstream of grit removal) (for each)	3 mgd (2,083 gpm)	3 mgd (2,083 gpm)	3 mgd (2,083 gpm)
RAS pump turndown (for each)	1.5 mgd (1,042 gpm)	1.5 mgd (1,042 gpm)	1.5 mgd (1,042 gpm)
RAS pump firm capacity	9 mgd	9 mgd	12 mgd
RAS pump discharge head (to downstream of grit removal)	23 feet	23 feet	23 feet
RAS pump motor	25 hp, variable speed	25 hp, variable speed	25 hp, variable speed
Number of WAS pumps	2 duty + 1 standby	2 duty + 1 standby	3 duty + 1 standby
WAS pump capacity (for each)	0.36 mgd (250 gpm)	0.36 mgd (250 gpm)	0.36 mgd (250 gpm)
WAS pump turndown (for each)	0.18 mgd (125 gpm)	0.18 mgd (125 gpm)	0.18 mgd (125 gpm)
WAS pump firm capacity	500 gpm	500 gpm	750 gpm
WAS pump discharge head	30 feet	30 feet	30 feet
WAS pump motor	3 hp, variable speed	3 hp, variable speed	3 hp, variable speed

6.7.3 Process Control

The RAS pumps will be provided with the option of manual control by operator-entered flow set point, or automatically controlled by flow pacing with the influent flow rate through the plant control system. The operator may choose to direct the RAS to upstream of the screens or directly to the process bioreactor splitter box via manually operated valves. WAS pumps will be controlled manually to an operator-entered flow rate or automatically to a total volume per day. The WAS pumps will be turned off automatically on high level in the aerobic digesters.

6.8 Deep Bed Filters

6.8.1 Description of Proposed Facility

Clarified water from the secondary clarifiers will flow to the Filter Feed Pump Station, which will lift the water to deep bed sand filters. The filter system will consist of filter cells, backwash supply storage tank (clearwell), backwash waste equalization tank (mudwell), air scour blowers, and backwash supply pumps. A canopy cover will be provided over the filters with a generous number of washdown hoses.

Flow entering each filter will pass through a dual inlet pipeline with isolation valves and into dual feed troughs to split the flow to each filter and distribute it along the length of the filter. After passing through the filter, a portion of the flow will be diverted into the clearwell for backwash supply, and the rest of the flow will proceed to disinfection. Backwash supply pumps will draw water from the clearwell for filter backwashing. The backwash water will be directed to a backwash waste tank (mudwell) that will include a spray nozzle system at the base of the tank to help direct settled solids to the tank discharge sump. The backwash water collected within the mudwell will be discharged to the Plant Drain Pump Station using a modulating valve to control flow to the pump station. Air scour blowers will provide air during a backwash cycle. The filter gallery will be at grade under an open-sided canopy roof cover. The southern face of the gallery will have a roll-up screen to block the sun. A means for bypassing the filters will be provided to divert flow from the inlet of the Filter Feed Pump Station directly to the Effluent Transfer Pump Station and be sent to the reject water storage tank for retreatment.

Filtered water will flow to disinfection. Turbidity will be continuously monitored in the clearwell effluent. If the turbidity exceeds a preset value, a valve on the discharge pipe from the Effluent Transfer Pump Station to reject water storage will open and the valve to reclaimed water storage will close, directing the flow to reject storage for future retreatment.

Filters will backwash automatically based on a timer or headloss but will require manual operator initiation. Filter backwash water will flow to the plant drain system after being equalized in the mudwell. A connection will be provided to the clearwell from the reclaimed water pump station to provide a backup means to fill the clearwell for backwashing, if necessary.

6.8.2 Design Criteria

Filters will be constructed by phase. Four filters will be constructed in Phase 1, a fifth filter will be added for Phase 2, and a sixth filter will be built in Phase 3 to satisfy reliability and redundancy requirements. Design criteria for the filters are summarized in Table 6-13. Phase 1 for the Filter Feed Pump Station will require installation of two jockey pumps and five larger sized pumps. Phase 2 will require removal and replacement of one of the jockey pumps with a larger sized pump. Phase 3 will replace the last jockey pump with a larger sized pump.

Table 6-13. Deep Bed Filter - Design Criteria

Parameter	Value
Type	Deep bed sand filters
Media	Sand and graded gravel
Media depth	6 feet
Secondary effluent TSS	15 mg/L average 45 mg/L maximum
Hydraulic loading rate	5.0 gpm per square foot at PHF
Solids loading rate	3.25 pounds per day per square foot maximum
Filter effluent TSS	5 mg/L maximum
Air scour rate	6 to 8 cubic feet per minute per square foot
Backwash rate	6 gpm per square foot
Total backwash water per backwash	150 to 200 gallons per square foot
Number of filter backwashes	Each filter, once per day
Filter underdrain system	Tetra or Leopold
Clearwell	Store volume for one backwash, minimum
Mudwell	Store backwash water for at least two backwashes and equalize for return to plant drain Slope to drain and include spray header system for cleaning
Filter feed pump type	Vertical turbine

Parameter	Value
Backwash supply pump type	Submersible
Air scour blower type	Positive displacement

Preliminary equipment sizing and phasing are summarized in Table 6-14. The number of units described is the total installed, including previous phases. Pump discharge head and motor size are estimated at this level of design.

Table 6-14. Filter System - Preliminary Equipment Sizing

Parameter	Phase 1	Phase 2	Buildout
Number of filters	4	5	6
Filter size (for each)	15 feet by 50 feet	15 feet by 50 feet	15 feet by 50 feet
Filter area (for each)	750 square feet	750 square feet	750 square feet
Total filter area	3,000 square feet	3,750 square feet	4,500 square feet
Hydraulic loading rate (gpm per square foot) All units in service			
AADF	1.56	1.66	1.73
MMADF	1.84	1.96	2.04
MDF	2.81	3.00	3.12
PHF	4.34	4.63	4.82
Hydraulic loading rate (gpm per square foot) One unit out of service			
AADF	2.08	2.08	2.08
MMADF	2.45	2.45	2.45
MDF	3.75	3.75	3.75
PHF	5.78	5.78	5.78
75% of PHF	4.34	4.34	4.34
Solids loading rate (pounds per day per square foot) All units in service			
MDF	1.52	1.62	1.69
PHF	2.34	2.50	2.60
Solids loading rate (pounds per day per square foot) One unit out of service			
PHF	3.13	3.13	3.13
75% of PHF	2.34	2.34	2.34
Number of filter feed pumps	7 (5 large + 2 jockey)	7 (6 large + 1 jockey)	7 (7 large + 0 jockey)
Filter feed pump capacity	5.21 mgd for each large pump 1 mgd for each jockey pump	5.21 mgd for each large pump 1 mgd for each jockey pump	5.21 mgd for each large pump
Filter feed pumping installed capacity	28.05 mgd	32.26 mgd	36.47 mgd

Parameter	Phase 1	Phase 2	Buildout
Filter feed pumping firm capacity	22.84 mgd	27.05 mgd	31.26 mgd
Filter feed pump discharge head	20 feet	20 feet	20 feet
Large filter feed pump motor	40 hp, variable speed	40 hp, variable speed	40 hp, variable speed
Jockey filter feed pump motor	7.5 hp, variable speed	7.5 hp, variable speed	n/a
Backwash waste volume	540,000 gpd	675,000 gpd	810,000 gpd
Number filter backwash supply pumps	2 (1 duty + 1 standby)	2 (1 duty + 1 standby)	2 (1 duty + 1 standby)
Filter backwash supply pump capacity (for each)	4,500 gpm	4,500 gpm	4,500 gpm
Filter backwash supply pump estimated head	23 feet	23 feet	23 feet
Filter backwash supply pump estimated motor	50 hp	50 hp	50 hp
Filter air scour blowers	TBD	TBD	TBD

6.8.3 Process Control

The filtration process will automatically alarm to the SCADA system on high headloss to alert the operator to manually initiate a filter backwash. Backwash may also be manually initiated by the operator or based on a timer setting. The filter backwash sequence will be automated but may also be sequenced manually from SCADA. The filter feed pumps will operate using variable-speed operation to maintain a wet well level set point. Flow from the mudwell will be returned at a controlled rate to equalize the flow back to the plant drain. Flowmeters with bypass will be provided on the filter feed flow, backwash pumps, air scour blowers, and backwash return flow.

6.9 Disinfection

6.9.1 Description of Proposed Facility

Filtered effluent will flow through a magnetic flowmeter for flow measurement ahead of the rapid mix and distribution channels for the CCBs. Sodium hypochlorite will be added to the filtered effluent within the rapid mix channel with an above grade feed point. A duty/standby rapid mix channel will be provided to include a mixer that will provide complete mixing for chlorine addition. Chlorinated effluent will then flow through a distribution channel that will provide even flow split to the CCBs, which will each consist of a serpentine three-pass configuration. The CCBs will be covered with a canopy to reduce loss of chlorine by ultraviolet (UV) exposure. Discharge from the rapid mix channel will flow over a rectangular effluent weir to the CCB distribution channel. Flow entering each CCB will pass through a sluice gate, which may be used for isolation. At the end of each CCB, chlorinated water will flow over a rectangular effluent weir to the CCB effluent channel and Effluent Transfer Pump Station. Drains will be provided for each pass of each CCB. Flow from the Effluent Transfer Pump Station that meets effluent requirements will be discharged to reclaimed water storage.

A total chlorine analyzer will be located within 15 feet of the beginning of each CCB to provide trim control of the sodium hypochlorite feed. A total and free chlorine analyzer will be located within the Effluent Transfer Pump Station wet well as the compliance monitoring point. Turbidity will be measured upstream of chorine addition at the clearwell.

If the filter effluent turbidity exceeds a preset value, TRC is below the minimum set point, or pH is outside the required range, the Effluent Transfer Pump Station will automatically divert flow to reject water storage by opening the valve to reject water storage and closing the valve to reclaimed water storage.

Once closed, this valve must be manually reopened by the operator after determination that effluent quality again meets the discharge standards.

6.9.2 Design Criteria

CCBs will be constructed by phase. Two basins will be constructed for Phase 1 to satisfy reliability and redundancy requirements. FAC 62-600.440 describes HLD requirements, including the required concentration x time (CT) for disinfection based on the influent fecal coliforms. A CT of 40 is selected based on Lee County experience at its other facilities, which is sufficient for filtered effluent with an expected range of 1,000 to 10,000 fecal coliform per 100 mL. Design criteria for disinfection are summarized in Table 6-15.

Table 6-15. Disinfection - Design Criteria

Parameter	Value
Type	Serpentine flow chlorine contact chamber
Number of passes per train	3
Minimum CT	40 mg-min/L
Filter effluent TSS	Less than 5 mg/L
Minimum TRC	1 mg/L
Minimum contact time at PHF	15 minutes
Minimum channel length to width ratio	40:1
Flow measurement	Magnetic flowmeter
Mixing	Mechanical mixer
Number of sodium hypochlorite addition points	One
Sodium hypochlorite addition location	Aboveground

mg-min/L = milligram-minute(s) per liter

Preliminary equipment sizing and phasing are summarized in Table 6-16. The basin structure would be constructed to add a third basin for Phase 3. The number of units described is the total installed, including previous phases.

Table 6-16. Disinfection - Preliminary Sizing

Parameter	Phase 1	Phase 2	Buildout
Number of contact basins	2	3	3
Number of passes, each basin	3	3	3
Length per pass	90 feet	90 feet	90 feet
Width (each pass)	6.50 feet	6.50 feet	6.50 feet
Depth	8 feet	8 feet	8 feet
Volume (each)	105,019 gallons	105,019 gallons	105,019 gallons
Total volume	210,038 gallons	315,057 gallons	315,057 gallons
Contact time at PHF	16.8 minutes	18.9 minutes	15.1 minutes
Contact time at PHF with one unit out of service	8.4 minutes	12.6 minutes	10.1 minutes
Contact time at 50% PHF with one unit out of service	16.8 minutes	25.2 minutes	20.2 minutes

6.9.3 Process Control

The addition of sodium hypochlorite addition will be flow paced for an operator selected dose, based on the flow measurement to the CCB. Dose will automatically be adjusted based on feedback from the TRC measurement located near the entrance of the CCBs. Because the control loop may introduce significant deadtime, feedback adjustment will be made using small step changes after a set time delay. The timer and step gain will be operator-adjustable from the SCADA system human-machine interface (HMI). In addition, this feedback control loop may be enabled or disabled while in automatic control. Decoupling the control loop may be warranted during startup or other abnormal process operations where the feedback loop cannot reliably track the system. Another scenario would be if the TRC analyzer encounters a fault and can no longer reliably provide feedback adjustment; decoupling the control would still allow for flow pacing control while the TRC analyzer is out of service. TRC will be continuously monitored at two locations in each CCB. The first location will be a point approximately 25% of the way down the CCB length for pacing, and the second location will be at the end of each CCB within the transfer pump station with an alarm if the TRC is less than 1 mg/L. The second TRC reading will be used for compliance. Turbidity at the influent to the CCB before sodium hypochlorite addition and pH at the effluent from the CCB will also be monitored and alarmed to the SCADA system. One compliance point TRC measurement will be provided for the combined CCB effluent.

6.10 Effluent Transfer Pumping

6.10.1 Description of Proposed Facility

Filtered and disinfected effluent will flow by gravity over a weir to the Effluent Transfer Pump Station, which will pump it to reclaimed water storage or reject water storage. The pump station will consist of vertical turbine pumps in a wet well connected to the CCB structure that will pump the treated effluent into a standpipe inside the storage tanks to reduce the range of head required by tank level variability. The discharge header will have two valves and two flowmeters, one on each pipe for effluent reject or for reclaimed water storage. The valves will be automatically actuated to send off-specification flow to reject water storage.

6.10.2 Design Criteria

Five larger pumps and one jockey pump, for startup flows, will be installed in Phase 1 to satisfy reliability and redundancy requirements. Phase 2 will require the installation of 1 additional larger pump to replace the jockey pump. Firm pumping capacity will be provided with one pump out of service. The pumps will be provided with variable-speed drives to adjust the flow rate. Design criteria for the Effluent Transfer Pump Station are summarized in Table 6-17.

Table 6-17. Effluent Transfer Pump Station - Design Criteria

Parameter	Value
Pump type	Vertical turbine
Configuration	Wet well
Firm capacity	PHF
Number of wet wells	1
Discharge destinations	Reject and reclaimed water storage tanks

Preliminary equipment sizing and phasing are summarized in Table 6-18. The number of units described is the total installed, including previous phases. Pump discharge head and motor size are estimated at this level of design.

Table 6-18. Effluent Transfer Pump Station - Preliminary Equipment Sizing

Parameter	Phase 1	Phase 2	Buildout
Number of pumps	3 duty + 1 standby	4 duty + 1 standby	4 duty + 1 standby
Pump capacity (for each)	7.5 mgd	7.5 mgd	7.5 mgd
Pump turndown (for each)	3.75 mgd	3.75 mgd	3.75 mgd
Firm capacity	22.5 mgd	30 mgd	30 mgd
Discharge head to reclaimed storage	40 feet	40 feet	40 feet
Discharge head to reject storage	43 feet	43 feet	43 feet
Pump motor	100 hp, variable speed	100 hp, variable speed	100 hp, variable speed
Number of jockey pumps	1	0	0
Capacity of jockey pump	3 mgd	n/a	n/a
Pump turndown	1.5 mgd	n/a	n/a
Discharge head	43 feet	n/a	n/a
Jockey pump motor	40 hp, variable speed	n/a	n/a

6.10.3 Process Control

The effluent transfer pumps will be operated automatically based on wet well level. Diversion to reject water storage will be automatic, based on continuous measurement of TRC, turbidity, and pH. Restoration of pumping to reclaimed water storage must be manually initiated by the operator once the cause of the off-specification water has been identified and remedied. Levels in the reject water storage and reclaimed water storage tanks will be monitored and alarmed.

6.11 Reclaimed Water Storage

6.11.1 Description of Proposed Facility

The Effluent Transfer Pump Station will pump effluent to reclaimed water storage. The pump station will pump the treated wastewater into a standpipe inside the future storage tanks to reduce the range of head required by tank level variability. In Phase 1, both reclaimed storage tanks will be constructed for storage of up to 10 MG, 1 day of AADF at buildout. One of the reclaimed tanks will also serve as a reject storage tank.

6.11.2 Design Criteria

Approximately 1 day of storage at AADF will be provided at the SEAWRF to maximize reuse of the treated effluent. Design criteria for the reclaimed water storage tanks are summarized in Table 6-19.

Table 6-19. Reclaimed Water Storage - Design Criteria

Parameter	Value
Storage (buildout)	1 day of AADF
Type (buildout)	Prestressed concrete tank with dome

Preliminary equipment sizing and phasing are summarized in Table 6-20. The number of units described is the total installed, including previous phases.

Table 6-20. Reclaimed Water Storage - Preliminary Equipment Sizing

Parameter	Phase 1	Phase 2	Buildout
Number of tanks	2	2	2
Volume per tank	5.0 MG	5.0 MG	5.0 MG
Total volume	10.0 MG	10.0 MG	10.0 MG
Tank diameter	164 feet	164 feet	164 feet
Tank depth	32 feet	32 feet	32 feet
Freeboard	1 foot	1 foot	1 foot
Total wall height	33 feet	33 feet	33 feet
Dome height	18.9 feet	18.9 feet	18.9 feet
Total height	51.9 feet	51.9 feet	51.9 feet

6.11.3 Process Control

The tank water level will be continuously measured by instrumentation with the level monitored on SCADA, including high-level and low-level alarms. A mechanical level measurement will be indicated on the side of the future storage tanks as backup. Flow will enter the tanks through a stand pipe and be drawn out through a pipe at the bottom of the tank. Passive overflows will be provided through vents at the top of the tank to protect the tank in case of overfilling. Isolation valves will be provided on each tank for maintenance purposes. The tanks will be interconnected by the effluent pipes with isolation valves such that all the tanks can rise and fall at the same level, if desired.

6.12 Reuse Pumping

6.12.1 Description of Proposed Facility

The Reuse Pump Station will pump stored reclaimed water to either the beneficial offsite D/IPR process selected in Section 7 or down an onsite Class I DIW. Although the beneficial reuse process selected will likely necessitate the higher pressure requirements, the DIW also needs to be considered. For the purposes of preliminary sizing, it is assumed that a distribution system for the D/IPR system would be the alternative with the farthest distance from the SEAWRF, approximately 8 miles, and the reuse flow is pumped throughout the day with a peak flow rate equal to the design MD flow rate to the treatment plant. The Reuse Pump Station will consist of horizontal split-case pumps connected to the reclaimed water storage tanks by a common header pipe.

6.12.2 Design Criteria

Design flow rates will vary by phase. Firm capacity will be provided with the largest pump out of service. The pumps will be provided with variable-speed drives to adjust the flow rate. Design criteria for the Reuse Pump Station are summarized in Table 6-21.

Table 6-21. Reuse Pump Station - Design Criteria

Parameter	Value
Pump type	Horizontal split case
Configuration	Slab-on-grade, common suction header pipe
Firm capacity	MDF
Services	Reclaimed water to D/IPR System, DIW, or both

Preliminary equipment sizing and phasing are summarized in Table 6-22. The number of units described is the total installed, including the previous phases. Pump discharge head and motor size are estimated at this level of design.

Table 6-22. Reuse Pump Station - Preliminary Sizing

Parameter	Phase 1	Phase 2	Buildout
Number of pumps	3 duty + 1 standby	4 duty + 1 standby	5 duty + 1 standby
Pump capacity (for each)	3.8 mgd	3.8 mgd	3.8 mgd
Pump turndown (for each)	1.9 mgd	1.9 mgd	1.9 mgd
Firm capacity	11.4 mgd	15.2 mgd	19 mgd
Required firm capacity	11.4 mgd	15.2 mgd	19 mgd
Discharge head	200 feet	200 feet	200 feet
Pump motor	200 hp, variable speed	200 hp	200 hp

6.12.3 Process Control

The Reuse Pump Station will be automatically controlled to maintain a selected flow rate to the reuse system and the remainder of the required effluent flow to the DIW via a separate DIW Pump Station. Flow will be measured by a flowmeter on the reuse effluent pipe. The pumps will be automatically stopped on low level in the reclaimed water storage tank.

6.13 Plant Water Distribution Pumping

6.13.1 Description of Proposed Facility

The plant water (W3) pump station will pump stored reclaimed water to the plant water distribution system.

The plant water distribution system requires a distribution pressure of approximately 70 pounds per square inch gauge (psig) for various plant uses such as hose bibbs, screens, and BFP washwater. Booster pumps will be provided for equipment where necessary.

The plant water pump station will consist of horizontal split-case pumps connected to the reclaimed water storage tanks by a common header pipe. Jockey pumps will be included to maintain plant water distribution system pressure at low-flow conditions.

6.13.2 Design Criteria

Design flow rates will vary by phase. Firm capacity will be provided with the largest pump out of service. The pumps will be provided with variable-speed drives to adjust the flow rate. Design criteria for the plant water pump station are summarized in Table 6-23. Estimates of plant water system demand for Phase 1 are summarized in Table 6-24. These estimates will be updated during detailed design. Demand at buildout would be similar to that of Phase 1 with the addition of unit processes.

Table 6-23. Plant Water Distribution Pump Station - Design Criteria

Parameter	Value
Pump type	Horizontal split case
Configuration	Slab-on-grade, common suction header pipe
Firm capacity	Peak usage
Services	Plant water system

Table 6-24. Plant Water Demand Estimate - Phase 1

Demand	Phase 1		
	Instantaneous	Usage Factor	Factored Used
BFP belt washwater	200 gpm	0.33	66 gpm
Hoses	75 gpm	0.5	37.5 gpm
Clarifier spray water	75 gpm	0.1	7.5 gpm
Polymer dilution water	20 gpm	0.33	6.6 gpm
Grit system	50 gpm	1	50 gpm
Screen system	60 gpm	0.5	30 gpm
Miscellaneous	50 gpm	1	50 gpm
Total	530 gpm	n/a	248 gpm
Minimum	75 gpm	n/a	n/a
Required firm	330 gpm	n/a	n/a

Preliminary equipment sizing and phasing are summarized in Table 6-25. The number of units described is the total installed, including the previous phases. Pump discharge head and motor size are estimated at this level of design.

Table 6-25. Plant Water Pump Station - Preliminary Sizing

Parameter	Phase 1	Phase 2	Buildout
Number of pumps	2	2	3
Pump capacity (for each)	330 gpm	330 gpm	330 gpm
Pump turndown (for each)	165 gpm	165 gpm	165 gpm
Discharge pressure	70 psig	70 psig	70 psig
Pump motor	20 hp, variable speed	20 hp, variable speed	20 hp, variable speed
Number of jockey pumps	1	1	0
Pump capacity	150 gpm	150 gpm	-
Pump turndown	75 gpm	75 gpm	-
Discharge pressure	70 psig	70 psig	-
Pump motor	10 hp, variable speed	10 hp, variable speed	-
Total firm capacity	480 gpm	480 gpm	660 gpm
Required firm capacity	330 gpm	440 gpm	550 gpm

6.13.3 Process Control

The plant water pump station will be automatically controlled to maintain system pressure to an operator adjustable set point. Flow will be measured by a flowmeter on the plant water system. As pressure decreases, pump speed will ramp up and additional pumps will be called to run, as necessary. As pressure increases and flow rate decreases, pump speed will be ramped down and turned off, as necessary. The pumps will be automatically stopped on low level in the reclaimed water storage tank. In the event that the reclaimed storage tank is at low level, provisions will be made for the W3 pumps to also draw directly from the Effluent Transfer Pump Station wet well.

6.14 Reject Water Storage

6.14.1 Description of Proposed Facility

Reject water storage will be provided in Phase 1 by two dedicated reject water storage tanks. A third reject water storage tank will be added at buildout. The reject water storage tank(s) will consist of open-topped, prestressed concrete tanks at grade. Manways will be provided for maintenance and inspection access at ground level. The bottom of the tanks will be flat, and a pipe will connect the tank to a Plant Drain Pump Station that will pump the water to the headworks.

The tanks will receive reject water from the Effluent Transfer Pump Station. The reject return will gravity flow to the Plant Drain Pump Station, where it is pumped to the headworks. The return flow rate will be controlled with a motorized plug valve and monitored with a flowmeter.

6.14.2 Design Criteria

A minimum of 1 day of storage at AADF will be provided. Design criteria for reject water storage are summarized in Table 6-26.

Table 6-26. Reject Water Storage - Design Criteria

Parameter	Value
Storage	Minimum of 1 day at AADF
Type	Prestressed concrete tank, open top

Preliminary equipment sizing and phasing are summarized in Table 6-27. The number of units described is the total installed, including previous phases.

Table 6-27. Reject Water Storage - Preliminary Equipment Sizing

Parameter	Phase 1	Phase 2	Buildout
Number of tanks	2	3	3
Volume per tank	3.34 MG	3.34 MG	3.34 MG
Total volume	6.7 MG	10.0 MG	10.0 MG
Required volume	6.0 MG	8.0 MG	10.0 MG
Tank diameter	135 feet	135 feet	135 feet
Tank depth	32 feet	32 feet	32 feet
Freeboard	1 foot	1 foot	1 foot
Total wall height	33 feet	33 feet	33 feet
Total height	33 feet	33 feet	33 feet
Reject return flow	3.0 mgd	4.0 mgd	5.0 mgd
Return flowmeter and control valve size	8 inches	8 inches	8 inches

6.14.3 Process Control

The tank water level will be continuously measured by instrumentation with the level monitored on SCADA, including high-level and low-level alarms. A mechanical level measurement will be indicated on the side of the tank as backup. Flow will enter the tank through a standpipe and be drawn out through a pipe at the bottom of the tank. Isolation valves will be provided on each tank for maintenance purposes. The tanks will be interconnected by the effluent pipes with isolation valves such that all the tanks can rise and fall at the same level, if desired. Flow will be returned for retreatment via the Plant Drain Pump

Station. The flow rate will be automatically controlled by a flow control valve and flowmeter to an operator-entered set point.

6.15 Sodium Hypochlorite Storage and Feed

6.15.1 Description of Proposed Facility

If necessary, 12.5% sodium hypochlorite solution will be used for disinfection and intermittent RAS chlorination for filamentous bacteria control. In addition, the sodium hypochlorite solution will be used for final disinfection at the chlorine contact basins. Sodium hypochlorite will be stored in tanks located in a containment area with a canopy cover. The containment area will be protected with a chemical resistant coating. All mounting hardware attached to the containment walls will be installed prior to application of coating. Site glasses will be provided on each tank as well as fill tubes that extend to the bottom of the tank.

Sodium hypochlorite feed pumps will be within the chemical containment area with controls accessible from outside the containment area. Each chemical feed line will have a Coriolis flowmeter for flow measurement and to indicate the pumps are in operation. Pump skids will be inside an enclosure with clear panels that can be taken off for access to prevent leaks from spraying on operators and an awning or cover to reduce exposure to UV. Pump skids will be turned sideways for ease of maintenance access. Controls for the pumps will be located outside of the skid. The suction pipe from the tank will be sloped back toward the tank and vented.

Containment will be provided for the truck fill connection. Safety showers and eye-wash stations will be provided near the truck fill connection and the feed pumps. Tubing within a carrier pipe will be provided to convey the sodium hypochlorite to each feed point. The carrier pipe will include long radius bends and a generous amount of hand holes to make maintenance easier.

6.15.2 Design Criteria

Firm capacity will be provided for the disinfection feed pumps. Storage will be provided for 21 days at MMADF to limit degradation during storage. Spare RAS chlorination feed pumps will not be provided. Design criteria for the sodium hypochlorite storage and feed facility are summarized in Table 6-28.

Table 6-28. Sodium Hypochlorite Storage and Feed - Design Criteria

Parameter	Value
Storage	21 days at MMADF (at 1.04 pound Cl_2/gallon)
Minimum number of storage tanks	2
Storage tank materials of construction	FRP
Storage tank freeboard	1 foot
Containment volume	Provide containment for largest tank volume
Containment freeboard	6 inches
Sodium hypochlorite solution	12.5% by weight (15% trade); 1.25 pounds Cl_2/gallon initial strength Assume 1.0 pound Cl_2/gallon degraded strength
Disinfection dose	8.0 mg/L maximum 6.0 mg/L average 4.0 mg/L minimum
RAS chlorination dose	2 to 10 pounds/1,000 pounds biomass/day
Truck delivery size	6,000 gallons
Minimum storage volume	Truck delivery quantity + 20%

Parameter	Value
Feed pump type	Diaphragm
Pump suction	Flooded

Cl_2 = chlorine

Feed pump and storage tank phasing and preliminary sizing are summarized in Table 6-29. The storage facility will be constructed for buildout in Phase 1.

Table 6-29. Sodium Hypochlorite Storage and Feed - Preliminary Equipment Sizing

Parameter	Phase 1	Phase 2	Buildout
Number of storage tanks	2	2	2
Storage tank volume (for each)	7,200 gallons	7,200 gallons	7,200 gallons
Storage tank dimensions	10 feet diameter by 12.5 feet SWD (13.5 feet side wall height; 15.5 feet total height with dome)	10 feet diameter by 12.5 feet SWD (13.5 feet side wall height; 15.5 feet total height with dome)	10 feet diameter by 12.5-feet SWD (13.5 feet side wall height; 15.5 feet total height with dome)
Number of feed pumps	3 duty + 1 standby	3 duty + 1 standby	3 duty + 1 standby
Maximum required feed rate	10.0 gph	13.3 gph	16.7 gph
Minimum required feed rate	3.0 gph	4.0 gph	5.0 gph
Minimum required feed rate at startup	0.75 gph	n/a	n/a
Feed pump capacity	17 gph	23 gph	28 gph
Firm capacity provided	51 gph	69 gph	84 gph
Minimum feed rate (speed adjustment only)	1.7 gph	2.3 gph	2.8 gph

gph = gallon(s) per hour

RAS chlorination feed pump phasing and preliminary sizing are summarized in Table 6-30.

Table 6-30. RAS Chlorination Pumps - Preliminary Equipment Sizing

Parameter	Phase 1	Phase 2	Buildout
Process bioreactor volume	6.46 MG	8.618 MG	10.77 MG
Basin MLSS	4,300 mg/L	4,300 mg/L	4,300 mg/L
Total biomass	231,669 pounds	308,772 pounds	386,234 pounds
Maximum required feed rate	96.5 gph	128.71 gph	160.9 gph
Minimum required feed rate	15.4 gph	20.6 gph	25.7 gph
Number of clarifiers	3	3	4
Number of feed points	2	2	2
Maximum required feed rate per feed point	48.3 gph	64.3. gph	80.5 gph
Minimum required feed rate per feed point	7.7 gph	10.3 gph	12.9 gph
Number of feed pumps	2 duty + 0 standby	2 duty + 0 standby	2 duty + 0 standby
Feed pump capacity	81 gph	81 gph	81 gph
Feed pump turndown	8 gph	8 gph	8 gph

6.15.3 Process Control

Sodium hypochlorite for disinfection will be paced to the flow rate measured at the magnetic flowmeter prior to the CCBs. The rate of chemical feed will be measured with a Coriolis type flowmeter. TRC will be continuously monitored at two locations in each CCB. The first location will be a point approximately 25% down the CCB length for pacing. The feed rate will be trimmed with a feedback loop from the TRC measured near the entrance of each CCB. The trim feedback would be made through small step changes after a time delay. Gain setting, time delay setting, and enabling and disabling the feedback control loop will be available for operator adjustment on the SCADA system HMI. Capability will also be provided to set the sodium hypochlorite feed rate manually. Storage tank level will be monitored and alarmed to the SCADA system. The RAS chlorination feed pump feed rate will be manually set and adjusted by the operator.

6.16 Aerobic Digesters

6.16.1 Description of Proposed Facility

WAS will be pumped to storage, where it will be held and aerated before dewatering. The storage will consist of rectangular, concrete, cast-in-place tanks with coarse bubble diffusers for aeration and mixing. A valve will be provided on each drop leg to balance the airflow and a union will be provided to allow each drop leg to be removed without taking the basin out of service. The tanks will be open topped and constructed to allow additional common wall tanks in future phases with passive overflow between tanks. Positive displacement blowers will provide the required air. For safety, air piping that is hot will be insulated in areas where it can be easily touched. Piping will be provided to decant the digested WAS to thicken the solids up to 2% concentration, if desired. The blowers will be provided with sound attenuating enclosures and be located under an open-sided canopy roof adjacent to the digesters. Digestion to meet Class B solids criteria for land application is not necessary because the solids will either be landfilled or transported to a composting facility for further processing.

6.16.2 Design Criteria

Design criteria for the aerobic digester facility are summarized in Table 6-31. In Phase 1, two tanks will be constructed to allow one tank to be taken out of service for maintenance. Normally, both tanks will be online and operated at the same level. A third tank will be added in Phase 2.

Table 6-31. Aerobic Digester Facility - Design Criteria

Parameter	Value
Storage (SRT)	12 days at MMADF (with decanting)
Minimum number of storage tanks	2
Storage tanks materials of construction	Concrete
Storage tank freeboard	3 feet
Maximum mixing air	30 SCFM/1,000 cubic feet
Safety factor on required air	1.1
Aeration blower type	Positive displacement
Blower manufacturer	Roots or Gardner Denver
Diffuser type	Coarse bubble (duck bill)
Diffuser manufacturer	Red Valve
DO concentration	1 to 2 mg/L
Thickening approach	Decanting

Aerobic digester tank and aeration blower phasing and preliminary sizing are summarized in Table 6-32. The number of units described is the total installed, including previous phases. Blower discharge head and motor size are estimated at this level of design.

Table 6-32. Aerobic Digester Facility - Preliminary Equipment Sizing

Parameter	Phase 1	Phase 2	Buildout
Number of digesters	2	3	3
WAS mass	16,140 pounds per day	21,085 pounds per day	26,750 pounds per day
WAS concentration	7,185 mg/L	7,048 mg/L	7,150 mg/L
Maximum thickened WAS concentration	17,800 mg/L	13,600 mg/L	17,200 mg/L
Thickened volume	108,722 gpd	185,895 gpd	186,479 gpd
Digester tank volume	0.6 MG	0.6 MG	0.6 MG
Total volume	1.2 MG	1.8 MG	1.8 MG
Tank dimensions (each)	99 feet long by 45 feet wide by 18 feet deep	99 feet long by 45 feet wide by 18 feet deep	99 feet long by 45 feet wide by 18 feet dep
Days of storage provided	11.0 days	9.7 days (12 days with digestion)	9.7 days (12 days with digestion)
Number of blowers	2 duty + 1 standby	3 duty + 1 standby	3 duty + 1 standby
Air required per tank (including safety factor)	2,647 SCFM	2,647 SCFM	2,647 SCFM
Total air required	4,813 SCFM	7,219 SCFM	7,219 SCFM
Blower capacity	2,647 SCFM	2,647 SCFM	2,647 SCFM
Total firm capacity	5,294 SCFM	7,941 SCFM	7,941 SCFM
Blower discharge head	9 psig	9 psig	9 psig
Blower motor	200 hp	200 hp	200 hp

6.16.3 Process Control

WAS will be pumped into the aerobic digesters by the WAS pumps. Levels in the tanks will be monitored and alarmed on high level to the SCADA system to automatically stop the WAS pumps. DO sensors will be provided to monitor concentrations. Blowers that supply air to the tanks will be manually controlled by the operator by setting the speed to maintain the target air flow set point. Air flow rate will be measured by flowmeters to each tank. Blowers will be piped such that each blower can be dedicated to a storage tank with a common spare. This will allow each tank to be operated at different levels with the air flow simply controlled by the blower speed. The air may be turned off periodically to allow the solids to settle and decant the liquid by opening valves on decant pipes that are located at several depths along the side of the tank. Decant sample pipes will be provided for each decant level that flow through a clear polyvinyl chloride section of pipe at grade so the operator may check the clarity of the decant.

6.17 Biosolids Dewatering Pumping

6.17.1 Description of Proposed Facility

BFP feed pumps will pump WAS solids stored in the aerobic digesters to the BFPs for dewatering. The pumps will be located on a concrete slab-on-grade with a canopy cover adjacent to the digesters. The feed pumps will be connected by a common suction header to all the aerobic digesters. Reclaimed water (W3) will be piped to the pump suction header with an automated valve to flush the sludge lines when a dewatering cycle is complete.

6.17.2 Design Criteria

Design criteria for the BFP feed pumps are summarized in Table 6-33.

Table 6-33. Biosolids Dewatering Pump Station - Design Criteria

Parameter	Value
Pump type	Progressing cavity
Pump manufacturer	Seepex
Capacity	1 feed pump per BFP 240 gpm for each
Feed solids concentration	5,000 to 20,000 mg/L

BFP feed pump phasing and preliminary sizing are summarized in Table 6-34. The number of units described is the total installed, including previous phases. Pump discharge head and motor size are estimated at this level of design.

Table 6-34. Biosolids Pump Station - Preliminary Equipment Sizing

Parameter	Phase 1	Phase 2	Buildout
Number of BFPs	2	3	4
Required flow rate	480 gpm	720 gpm	960 gpm
Number of total pumps	3	4	5
Pump capacity	240 gpm	240 gpm	240 gpm
Pump turndown	120 gpm	120 gpm	120 gpm
Discharge head	60 feet	60 feet	60 feet
Pump motor	20 hp	20 hp	20 hp

6.17.3 Process Control

BFP feed pumps will be controlled from the BFP control panel. Flow rates will be manually set by the operator and measured by a flowmeter on the feed pipe to each BFP. Each pump will be dedicated to a BFP. Overpressure and run-dry protection will be provided for each pump. A pressure indicator/transmitter will send the discharge pressure reading to the SCADA system, where high pressure and high-high pressure will be alarmed based on an operator-entered set point for high pressure and a hard-coded, high-high pressure that is set to match the pressure safety valve setting. The pressure safety valve on the pump discharge will relieve high pressure and the high or high-high pressure alarm setting will shut the pump off. A temperature indicator/transmitter will send the rotor/stator temperature reading to the SCADA system. A high temperature alarm will be generated by the SCADA system based on an operator-entered set point that will shut the pump off.

6.18 Biosolids Dewatering

6.18.1 Description of Proposed Facility

WAS will be dewatered using BFPs in a two-story dewatering building. The BFPs will be located on the second level, and the polymer storage and feed system and truck loading bays will be located on the first level. The polymer storage and feed system will be located in a concrete curbed containment area with grating over the top. Dewatered solids will drop into conveyors that will transport the dewatered cake and distribute it into truck trailers. The conveyors that distribute the solids into the trucks will have multiple slide gates to distribute the load. Two truck bays will be provided, each with a truck scale. Space will be provided to park two additional trailers outside of the building. Dewatered solids will be hauled offsite to landfill disposal or to a composting facility. The dewatering building will be open-sided with partial side walls to improve shade and reduce windblown rain while allowing natural ventilation without odor control. The BFPs will be three-belt type with the first belt providing a gravity thickening zone.

A booster pump will be provided for the BFP spray water system. A bridge crane will be provided over the BFP area for maintenance activities. A concrete pad with power, plant water supply, and drain to the plant drain system will be provided for a trailer-mounted centrifuge that will provide backup dewatering capacity if a BFP is out of service. This will include a 4-inch cam-lock connection to the suction side of the BFP feed pump station, electrical disconnect, and cable to provide power to the centrifuge. A cleaning station will be located onsite for hosing down trucks and trailers and capturing the wash-down water to the plant drain system.

6.18.2 Design Criteria

Design criteria for the biosolids dewatering system are summarized in Table 6-35.

Table 6-35. Biosolids Dewatering - Design Criteria

Parameter	Value
Dewatering equipment	BFP (three-belt system)
Manufacturer	Ashbrook
Belt width	2 meters
Belt washwater	60 gpm/m at 85 psi
Hydraulic loading capacity	100 gpm/m of belt width (dewatering)
Solids loading capacity	500 pounds per hour per meter of belt width
Feed solids concentration range	15,000 to 20,000 mg/L (17,000 mg/L average)
Hours of operation	10 hours per day, 5 days per week, MMAD condition (Buildout)
Cake solids	15% minimum
Conveyor type	Shaftless screw
Polymer feed	10 pounds per dry ton average; 15 pounds per dry ton maximum
Number of polymer feed systems	1 per BFP
Number of truck bays	2
Truck trailer size	31.5-foot length (23 ton)
Mobile centrifuge power requirements	TBD
Mobile centrifuge drain requirements	TBD
Mobile centrifuge plant water supply requirements	TBD

gpm/m = gallon(s) per minute per meter

psi = pound(s) per square inch

Dewatering equipment phasing and preliminary sizing are summarized in Table 6-36. The number of units described is the total installed, including previous phases. Pump discharge head and motor size are estimated at this level of design.

Table 6-36. Biosolids Dewatering - Preliminary Equipment Sizing

Parameter	Phase 1	Phase 2	Buildout
Number of BFPs	2	3	4
Feed rate range	120 to 240 gpm/BFP	120 to 240 gpm/BFP	120 to 240 gpm/BFP
WAS at MMAD condition	16,140 pounds per day	21,520 pounds per day	26,750 pounds per day
Average WAS feed concentration	17,000 mg/L	17,000 mg/L	17,000 mg/L

Parameter	Phase 1	Phase 2	Buildout
WAS volume	159,374 gpd	212,498 gpd	264,142 gpd
Effective WAS rate at 10 hours per day, 5 days per week operation	22,596 pounds per day 2,260 pounds per hour 266 gpm	30,128 pounds per day 3,013 pounds per hour 354 gpm	37,450 pounds per day 3,745 pounds per hour 440 gpm
Installed BFP capacity at 10 hours per day, 5 days per week operation	20,000 pounds per day 2,000 pounds per hour 320 gpm	30,000 pounds per day 3,000 pounds per hour 480 gpm	50,000 pounds per day 5,000 pounds per hour 640 gpm
Required hours of operation 5 days per week	5.5 (based on HLR) 11.3 (based on SLR)	4.9 (based on HLR) 10.0 (based on SLR)	4.6 (based on HLR) 9.4 (based on SLR)
Polymer feeder range (active pound basis) (for each)	1.8 to 9.0 pounds per hour	1.8 to 9.0 pounds per hour	1.8 to 9.0 pounds per hour
BFP drive motor	3 hp	3 hp	3 hp
BFP gravity belt motor	2 hp	2 hp	2 hp
BFP hydraulic belt tensioner	1 hp	1 hp	1 hp
BFP washwater booster pump	TBD	TBD	TBD

HLR = hydraulic loading rate

SLR = solids loading rate

6.18.3 Process Control

The dewatering system will be controlled by the BFP control panel. The system will control the BFP feed pumps, BFP, polymer feed system, conveyors, and washwater booster pump. Operators will enter the set point for feed pumping and polymer dose. Sludge cake conveyors will run automatically when the BFP is operating and for several minutes after shutdown to clear the contents. The conveyors will also have the capability to be manually operated. Slide gates on the distribution conveyors will be manually operated or may be operated based on an operator-entered cumulative time. Truck weights will be monitored by the truck weigh scales and displayed on the SCADA system with an alarm when an adjustable maximum weight target is reached to alert the operator.

6.19 Plant Drain Pumping

6.19.1 Description of Proposed Facility

Drainage from unit process operations will flow to the Plant Drain Pump Station located near the dewatering and aerobic digester facilities, which are the major source of plant drain flow. Basin drains from all unit processes will also flow into the Plant Drain Pump Station. Controlled flow from the reject storage tank will also be fed to the Plant Drain Pump Station. The pump station will consist of a wet well with submersible pumps. The pump station will discharge into the headworks upstream of the screens, but downstream of the influent sample location.

6.19.2 Design Criteria

Four pumps will be installed in Phase 1 and upgraded with each subsequent phase as plant drain flows increase. Firm pumping capacity will be provided with one pump out of service. During times when reject flow is being fed to the Plant Drain Pump Station, all four pumps will be in service. The pumps will be provided with variable speed drives to reduce fluctuation in flow rate. Design criteria for the Plant Drain Pump Station are summarized in Table 6-37.

Table 6-37. Plant Drain Pump Station - Design Criteria

Parameter	Value
Pump type	Submersible
Configuration	Wet well
Maximum number of starts per hour per pump	8
Number of wet wells	1
Maximum wet well water surface elevation	Below the lowest structure
Discharge destinations	Headworks

Preliminary equipment sizing and phasing are summarized in Table 6-38. The number of units described is the total installed, including previous phases. Pump discharge head and motor size are estimated at this level of design. Estimated plant drain flows are shown by phase, not including intermittent events, such as draining a basin. Installed pump capacity will be provided to allow draining of structures.

Table 6-38. Plant Drain Pump Station - Preliminary Equipment Sizing

Parameter	Phase 1	Phase 2	Buildout
Filter backwash average flow	540,000 gpd	675,000 gpd	810,000 gpd
Filter backwash average flow	375 gpm	469 gpm	563 gpm
Filter backwash peaking factor	1.0	1.0	1.0
Filter backwash peak flow	375 gpm	469 gpm	563 gpm
Digester decant average flow	175,000 gpd	233,333 gpd	291,667 gpd
Digester decant average flow	122 gpm	162 gpm	203 gpm
Digester decant peak flow factor	3.0	3.0	3.0
Digester decant peak flow	365 gpm	486 gpm	608 gpm
BFP filtrate peak flow	440 gpm	660 gpm	880 gpm
Total peak plant drain flow	1,180 gpm	1,615 gpm	2,050 gpm
Safety factor	1.1	1.1	1.1
Total peak plant drain flow with safety factor	1,298 gpm	1,776 gpm	2,255 gpm
Number of pumps	4 duty + 1 standby	4 duty + 1 standby	5 duty + 1 standby
Pump capacity (for each)	874 gpm	874 gpm	874 gpm
Pump turndown (for each)	437 gpm	437 gpm	437 gpm
Firm capacity	2,622 gpm	3,496 gpm	4,370 gpm
Discharge head	50 feet	50 feet	50 feet
Pump motor	25 hp, variable speed	25 hp, variable speed	25 hp, variable speed

6.19.3 Process Control

The Plant Drain Pump Station will be operated automatically based on wet well level. The level in the wet well will be monitored and alarmed to the plant SCADA system on high level.

6.20 Deep Injection Well

6.20.1 Description of Proposed Facility

A 24-inch-diameter municipal DIW is planned for Phase 1 that will be used for disposal of effluent if the supply of reclaimed water exceeds demand. The well will be approximately 3,000 feet deep and inject treated effluent below the lowest usable source of drinking water. A gate valve will be provided immediately above the first flange on the final well casing for isolation of the well. The wellhead will include instrumentation, a tee at the top of the well casing above the gate valve, air vacuum release valve, flowmeter, and isolation valve. The well will be surrounded by a concrete pad that is curbed and sloped to a sump to capture drainage. A dual zone monitoring well will also be provided within 150 feet of the injection well to satisfy monitoring requirements.

A pressure sustaining valve will keep the pressure in the reclaimed water pump discharge header at 55 psi or higher. A motorized flow control valve will modulate the flow going to the DIW.

6.20.2 Design Criteria

The 24-inch-diameter DIW will have a theoretical flow rate of 19.0 mgd, equating to a maximum downhole velocity of 10 feet per second. This is the maximum permittable standard operating condition. Actual DIW capacity will be subject to testing at full flow rates and will vary with minor changes in wellhead injection pressure. The well will be designed with a cemented annulus instead of a fluid-filled annulus to minimize maintenance requirements. The required Phase 1 capacity is the plant MDF of 11.4 mgd. A surge analysis will be conducted, and surge protection will be provided at the DIW Pump Station. The well will be sufficient for buildout MDF, but a second well may be installed to accommodate redundancy or reliability.

6.20.3 Process Control

The flow control valve will modulate the flow to the DIW. The air vacuum release valve will operate automatically to exhaust air or break a siphon. A flowmeter will be provided to measure the flow rate and totalize the flow down the well, with the information transmitted to the SCADA system for remote monitoring.

The monitoring well pumps are manually controlled to purge the well for grab samples. The flow rates are monitored by totalizing flowmeters.

6.21 Deep Injection Well Pump Station

6.21.1 Description of Proposed Facility

A separate pump station will be dedicated to discharging to the DIW(s) because it will likely have a different discharge head requirement compared to the Reuse Pump Station. However, this will be further evaluated once a D/IPR alternative is selected prior to detailed design. The DIW Pump Station will serve as a backup disposal method when the required effluent flow exceeds the capacity of the selected beneficial reuse system. The required pressure to pump to the DIW is estimated to be a maximum of 90 psig. The required capacity is the MDF rate to the treatment plant, which varies by phase. A flowmeter will be provided on the wellhead for flow measurement. A single 24-inch DIW will be included in the first phase with a maximum capacity of 19 mgd of flow.

The pump station will consist of vertical turbine pumps in a "can" arrangement with connections to the transfer pump station wet well and the reclaimed water storage tanks. Normally the pumps will draw water from the reclaimed water storage tanks. A hydropneumatic-type surge tank will be provided to protect the pump station during sudden power loss. The size of the surge tank will be determined through detailed surge analysis as the design progresses.

6.21.2 Design Criteria

Design flow rates will vary by phase. Firm capacity will be provided with the largest pump out of service. The pumps will be provided with variable-speed drives to adjust the flow rate. Design criteria for the DIW Pump Station are summarized in Table 6-39.

Table 6-39. Deep Injection Well Pump Station - Design Criteria

Parameter	Value
Pump type	Vertical turbine
Configuration	"Can" arrangement
Firm capacity	MDF
Maximum discharge pressure	90 psig
Surge protection	Hydropneumatics tank

Preliminary equipment sizing and phasing are summarized in Table 6-40. The number of units described is the total installed, including the previous phases. Pump discharge head and motor size are estimated at this level of design.

Table 6-40. Deep Injection Well Pump Station - Preliminary Sizing

Parameter	Phase 1	Phase 2	Buildout
Number of pumps	2 duty + 1 standby	3 duty + 1 standby	4 duty + 1 standby
Pump capacity	5.7 mgd/each	5.7 mgd/each	5.7 mgd/each
Pump turndown	2.85 mgd/each	2.85 mgd/each	2.85 mgd/each
Firm capacity	11.4 mgd	17.1 mgd	22.8 mgd
Required firm capacity	11.4 mgd	15.2 mgd	19 mgd
Discharge head	208 feet	208 feet	208 feet
Pump motor	300 hp, variable speed	300 hp, variable speed	300 hp, variable speed

6.21.3 Process Control

The DIW Pump Station will be automatically controlled to an operator adjustable flow set point or to maintain level in the storage tank when pumping from the reclaimed water storage tanks, which is the normal operating mode. When pumping from the transfer pump station wet well, the pumps will be controlled automatically to maintain level in the wet well by adjusting the speed and starting/stopping on low flow. Pumping to the DIW when drawing from the transfer pump station wet well will not be permitted when in reject mode. Flow will be measured by a flowmeter on the DIW(s). The pumps will be automatically stopped on low level in the reclaimed water storage tank or in the transfer pump station wet well.

6.22 Alum Storage and Feed

Alum feed would consist of two storage tanks and feed pumps in a concrete containment area with an open-sided canopy cover. Alum would be fed to the secondary clarifier splitter box using the drop over the weir for mixing. The facility would be located between the secondary clarifiers and effluent transfer station, near the northern loop road. Storage will be provided for 30 days at MMADF. Design criteria for the alum storage and feed facility are summarized in Table 6-41.

Table 6-41. Alum Storage and Feed - Design Criteria

Parameter	Value
Storage	30 days at MMADF
Minimum number of storage tanks	2
Storage tanks materials of construction	FRP
Storage tank freeboard	1 foot
Containment volume	Provide containment for largest tank volume

Parameter	Value
Containment freeboard	6 inches
Alum solution	48%; 5.34 pounds alum/gallon
Alum dose	40.0 mg/L maximum 30.0 mg/L average 15.0 mg/L minimum
Truck delivery size	5,000 gallons
Minimum storage volume	Truck delivery quantity + 20%
Minimum storage volume per tank	6,000 gallons
Feed pump type	Diaphragm
Pump suction	Flooded

Feed pump and storage tank phasing and preliminary sizing are summarized in Table 6-42.

Table 6-42. Alum Storage and Feed - Preliminary Equipment Sizing

Parameter	Phase 1	Phase 2	Buildout
Required storage volume for average flow and dose	10,116 gallons	13,488 gallons	16,860 gallons
Number of storage tanks	2	2	2
Storage tank volume (for each)	8,500 gallons	8,500 gallons	8,500 gallons
Storage tank dimensions	11 feet diameter by 12 feet SWD (13 feet side wall height; 15 feet total height with dome)	11 feet diameter by 12 feet SWD (13 feet side wall height; 15 feet total height with dome)	11 feet diameter by 12 feet SWD (13 feet side wall height; 15 feet total height with dome)
Number of feed pumps	1 duty + 1 standby	1 duty + 1 standby	1 duty + 1 standby
Maximum required feed rate	29.7 gph	39.6 gph	49.4 gph
Minimum required feed rate	2.6 gph	3.5 gph	4.4 gph
Feed pump capacity each	29.7 gph	39.6 gph	49.4 gph
Firm capacity provided	29.7 gph	39.6 gph	50 gph
Minimum feed rate	3.0 gph	4.0 gph	5.0 gph

6.23 Supplemental Carbon Storage and Feed

Micro C glycerin is Lee County's preferred supplemental carbon source. Micro C would be flow paced and added at a rate of 0.77 gallons of Micro-C glycerin per 1 pound of nitrate-N removed. Nitrate measurement would be provided as feedback for controlling the rate of addition. Two tanks would be provided to store the Micro C glycerin. The tanks and feed pumps would be located in a concrete containment structure with an open-sided canopy roof cover. Other carbon sources may also be considered by Lee County. Design criteria for the supplemental carbon storage and feed facility are summarized in Table 6-43.

Table 6-43. Future Supplemental Carbon Storage and Feed - Design Criteria

Parameter	Value
Storage	30 days at MMADF
Minimum number of storage tanks	2
Storage tanks materials of construction	FRP
Storage tank freeboard	1 foot
Containment volume	Provide containment for largest tank volume
Containment freeboard	6 inches
Micro C solution	100%; 10.2 pounds methanol per gallon
Micro C dose	47.0 mg/L average
Truck delivery size	5,000 gallon
Minimum storage volume	Truck delivery quantity + 20%
Minimum storage volume per tank	6,000 gallons
Feed pump type	Diaphragm
Pump suction	Flooded

Feed pump and storage tank phasing and preliminary sizing are summarized in Table 6-44.

Table 6-44. Supplemental Carbon Storage and Feed - Preliminary Equipment Sizing

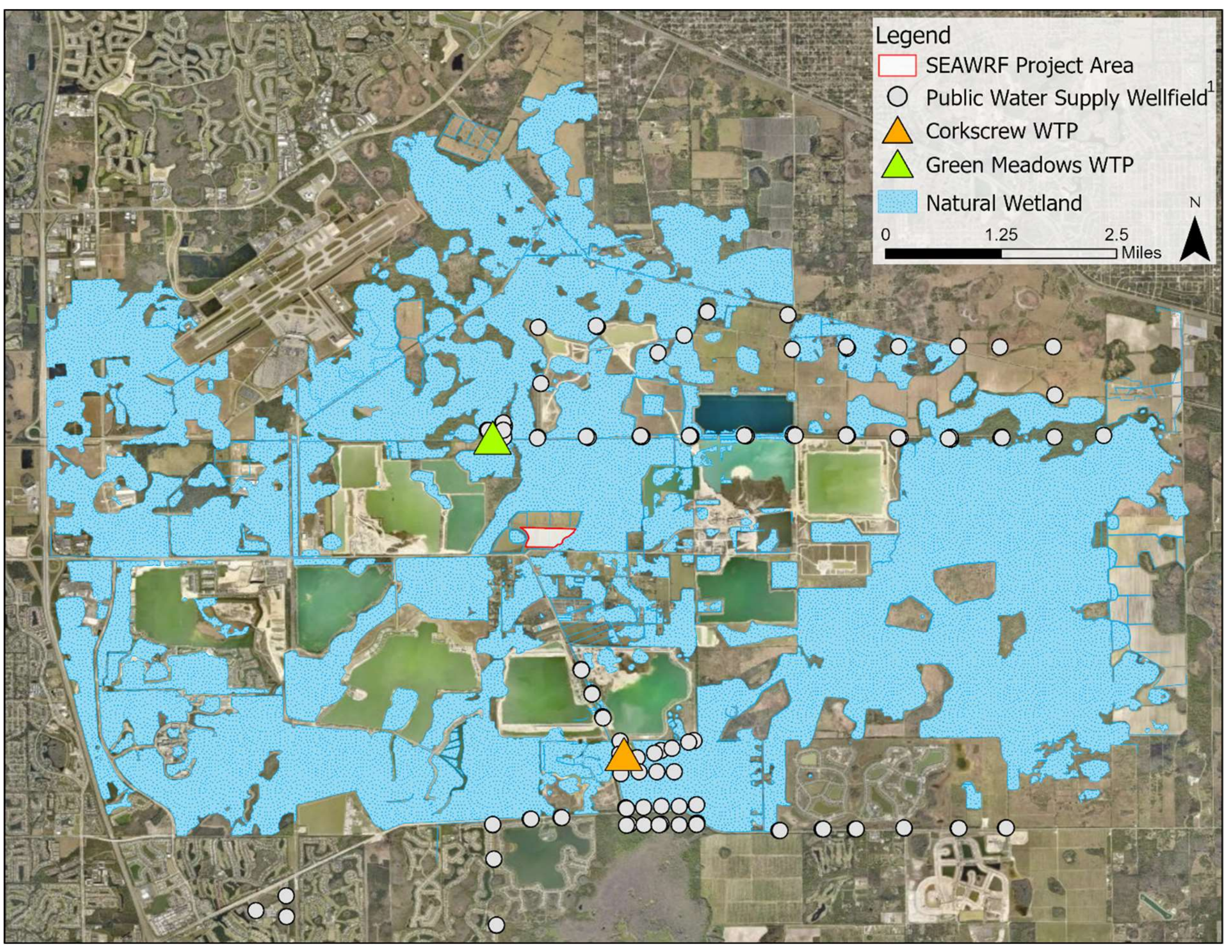
Parameter	Phase 1	Phase 2	Buildout
Required storage volume for average flow and dose	8,301 gallons	11,068 gallons	13,835 gallons
Number of storage tanks	2	2	3
Storage tank volume (for each)	7,000 gallons	7,000 gallons	7,000 gallons
Storage tank dimensions	10 feet diameter by 12 feet SWD (13 feet side wall height; 15 feet total height with dome)	10 feet diameter by 12 feet SWD (13 feet side wall height; 15 feet total height with dome)	10 feet diameter by 12 feet SWD (13 feet side wall height; 15 feet total height with dome)
Number of feed pumps	1 duty + 1 standby	1 duty + 1 standby	1 duty + 1 standby
Maximum required feed rate	23.3 gph	31.1 gph	38.8 gph
Minimum required feed rate	0.9 gph	1.2 gph	1.5 gph
Feed pump capacity each	40 gph	40 gph	40 gph
Firm capacity provided	40 gph	40 gph	40 gph
Minimum feed rate	4.0 gph	4.0 gph	4.0 gph

7 Direct or Indirect Potable Reuse Alternatives Analysis

Indirect potable reuse is defined as the planned discharge of reclaimed water to surface waters or groundwater to augment the supply of raw water available for drinking water and other uses. Indirect potable reuse is contrasted with direct potable reuse, which involves the discharge of reclaimed water directly into a drinking water treatment facility or into a drinking water distribution system.

The proposed SEAWRF site is in southeast Lee County in an area surrounded by extensive tracts of natural wetlands. It is also located between two of Lee County's largest water treatment facilities. The Green Meadow WTP and the wellfield that supplies it are located approximately 1 mile north of the proposed site for the SEAWRF, and the Corkscrew WTP and its wellfield are located approximately 2.5 miles south of the proposed SEAWRF site. Figure 7-1 presents the locations of the proposed site, the surrounding wetlands, and the water treatment facilities. The wellfields that supply both the Green Meadow WTP and the Corkscrew WTP consist of production well clusters completed into the surficial aquifer system, the Sandstone aquifer, and the Floridan Aquifer System (FAS). Consumptive use permits regulate the water produced from each aquifer to minimize local and regional impacts associated with drawdowns. Augmentation of surface waters connected to the surficial aquifer and Sandstone aquifer systems would improve the groundwater resources to either or both water treatment facilities in addition to the wetland benefits.

Figure 7-1. Location of Southeast Advanced Water Reclamation Facility, Wetlands, and Water Treatment Facilities



[1] The wellfield includes both constructed wells and wells that are permitted but not constructed.

This section includes an alternatives analysis of three strategies for D/IPR of the SEAWRF effluent to provide robust and sustainable water supplies for Lee County. The alternative strategies include the following:

- Alternative 1—Floridan aquifer recharge (indirect potable reuse)
  - Alternative 1A—Groundwater recharge via injection of SEAWRF effluent into the FAS to supplement water supply. The targeted recharge zone is in the UFAS within the Lower Hawthorn and Suwannee aquifers
  - Alternative 1B—Groundwater recharge via injection of SEAWRF effluent into the FAS to supplement water supply. The targeted recharge zone is the UFAS within the Ocala Limestone and Avon Park formations
- Alternative 2—Direct potable reuse via an advanced water treatment facility
- Alternative 3—Receiving wetlands application
  - Alternative 3A—Reuse application to natural receiving wetlands at the Southwest Florida International Airport (SWFIA) Mitigation Park site
  - Alternative 3B—Reuse application to natural receiving wetlands at the Northern Area site
  - Alternative 3C—Reuse application to natural receiving wetlands adjacent to SEAWRF

7.1 Alternative 1: Floridan Aquifer Recharge (Indirect Potable Reuse)

7.1.1 Alternative Description

Floridan aquifer recharge is performed through recharge wells. This alternative would be considered indirect potable reuse because it is the planned discharge of reclaimed water to an underground source of drinking water. Recharge into the FAS requires a subsurface recharge zone that demonstrates hydraulic characteristics (transmissivity) capable of accepting injection rates and volume. Water quality requirements depend on the ambient water quality of the proposed injection zone. Figures 7-2 and 7-3 present a hydrogeologic cross section of the site area. For this alternative, it was assumed that the recharge wells would be sited at SEAWRF.

7.1.1.1 Alternative 1A

Alternative 1A consists of constructing a well with a recharge zone in the Upper Floridan Aquifer System (UFAS), within the Lower Hawthorn and Suwannee Limestone formations, from 650 to 1,000 feet below land surface (bls). Typical recharge rates in this aquifer are on the order of 2 million gallons per day per well. The current rules allow for the injection of reclaimed water directly into aquifers with ambient total dissolved solid (TDS) concentrations in the receiving zone that are at or above 1,000 mg/L. In this zone TDS values range from 2,000 to 3,000 mg/L. This alternative would require a minimum of 5 recharge wells to achieve the maximum AADF capacity of 10 million gallons per day.

7.1.1.2 Alternative 1B

Alternative 1B consists of constructing a well with a recharge zone in the UFAS, within the Ocala Limestone and upper Avon Park formations, from 1,100 to 1,600 feet bls. Typical recharge rates in these aquifers are up to 1 million gallons per day per well. In this zone TDS values range from 3,000 to 10,000 mg/L. This alternative would likely require 10 recharge wells to achieve the maximum AADF capacity of 10 million gallons per day.

Figure 7-2. Plan View of Conceptual Hydrogeologic Cross Section

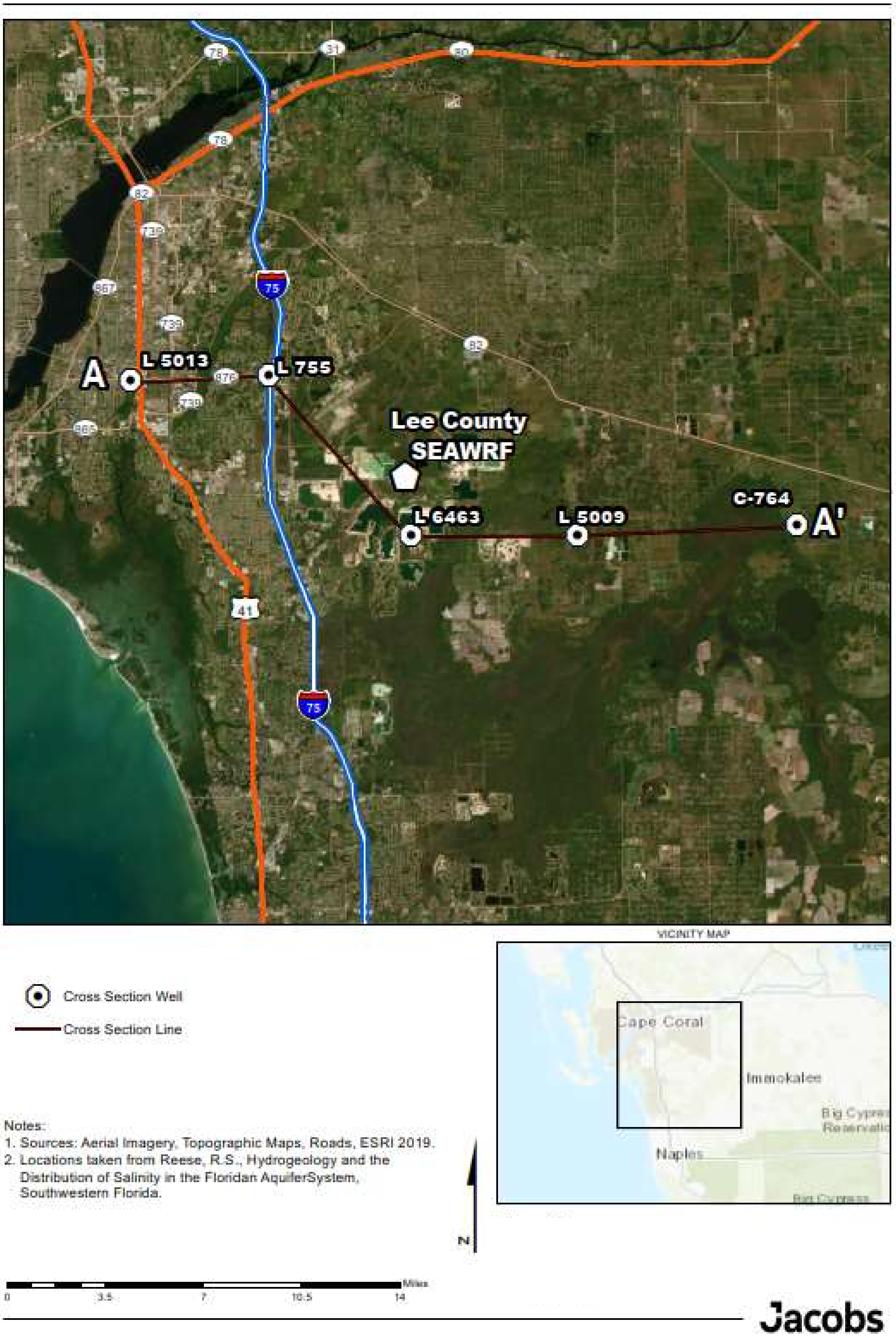
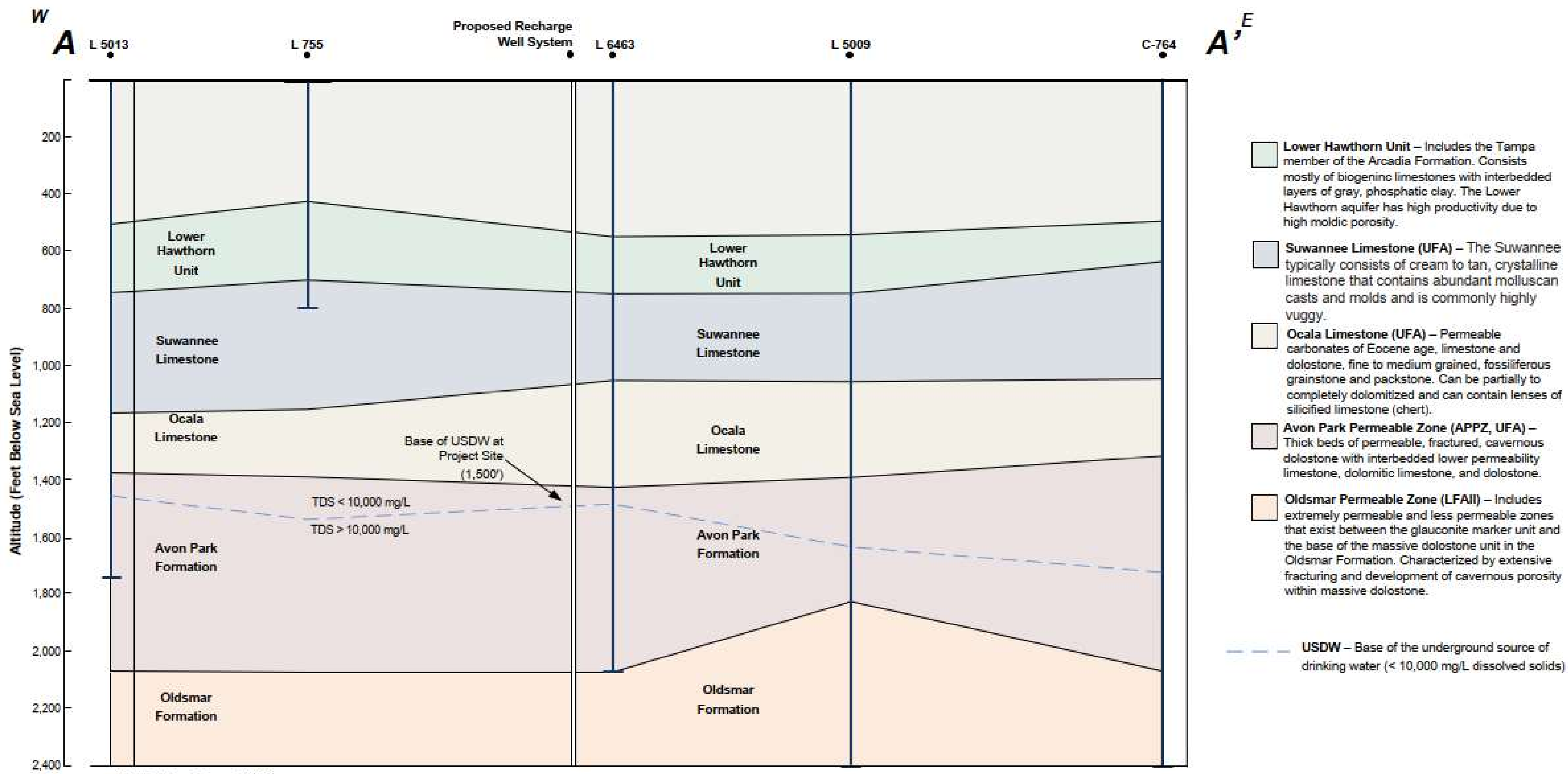


Figure 7-3. Conceptual Hydrogeologic Cross Section



Modified from Reese (2000).

7.1.1.3 Monitoring Well System for Any Alternative

Each recharge system will also require the construction of a comprehensive monitoring well system to monitor operations and ensure recharge operations follow the rules and regulations governing recharge wells, as well as individual recharge well permit stipulations. These wells monitor the storage zone and selected units immediately overlying the recharge zone and are monitored for water level and selected water quality parameters, which are used to evaluate recharge well operations and compliance. For Alternative 1A, three monitoring wells would be required, and for Alternative 1B, six monitoring wells would be required.

7.1.2 Regulatory and Treatment Requirements

For Alternatives 1A and 1B, the group of wells may be permitted as a system rather than as individual wells and permit renewal would occur every 5 years. The FDEP is the lead regulatory agency for recharge systems and appropriate regulations will depend on TDS levels in the recharge zone. If groundwater in the recharge zone contains less than 3,000 mg/L TDS in a Class F-1 or G-1 aquifer, the FDEP may require a feasibility study or an exploratory drilling program. Additionally, full treatment and disinfection requirements contained in subsection 62-610.563(3), FAC will be met, which include the following:

- Secondary treatment and high-level disinfection
- TSS < 5 mg/L before application of the disinfectant
- Total N < 10 mg/L
- Groundwater quality criteria
- Primary and secondary drinking water standards
- Total organic carbon (TOC) < 3.0 mg/L monthly average and no single sample will exceed 5.0 mg/L
- Total organic halogen (TOX) < 0.2 mg/L monthly average, and no single sample will exceed 0.3 mg/L

If groundwater in the recharge zone contains greater than 3,000 mg/L TDS, then it is designated as Class G-II aquifer and the following treatment requirements, found under subsection 62-610.563(2), FAC, are required:

- Secondary treatment and high-level disinfection
- TSS < 5.0 mg/L before application of the disinfectant
- Filtration for TSS control
- Total N < 10 mg/L
- Primary drinking water standards

These requirements are less stringent and pilot testing is not required if the zone of discharge does not extend into zones less than 3,000 mg/L TDS, based on the initial TDS characterization in the engineering report.

7.1.3 Cost Estimate

7.1.3.1 Capital Cost

A Class 5 planning-level engineer's opinion of probable capital cost was prepared for Alternatives 1A and 1B. Tables 7-1 and 7-2 present Class 5 cost estimates that have a predicted level of accuracy of -30% to +50%, consistent with the terminology and practices recommended for conceptual screening analysis by the Association of the Advancement of Cost Engineering. The accuracy of these planning-level cost estimates is intended to compare the capital cost for construction of each proposed alternative.

The capital cost estimates provided in Tables 7-1 and 7-2 are based on the conceptual design for Alternatives 1A and 1B as described previously and included the following assumptions:

- Recharge well and monitoring well costs were based on 2022 recharge and monitoring well bid prices for nearby utilities.
- Recharge well and monitoring well costs include a $100,000 fee to obtain permits for the Class 5 wells (all recharge wells included).
- Five recharge wells and 3 monitoring wells are assumed for Alternative 1A.
- Ten recharge wells and 6 monitoring wells are assumed for Alternative 1B.
- Reuse Pump Station has a maximum capacity of 19 mgd (the maximum potential flow rate).
- Lee County already owns the land, so no land purchase is required.
- Clearing and grubbing of land is assumed to not be required.
- Additional treatment will be required for the Alternative 1A option for primary and secondary drinking water standards and to meet the TOC and TOX requirements. This pretreatment was not included in the high-level cost estimate used for comparison purposes.

Table 7-1. Alternative 1A Class 5 Capital Cost Estimate

Alternative 1A	Quantity	Units	Unit Cost	Total Cost
LH and Suwannee LS Recharge Wells	5	EA	$2,739,218	$13,696,000
LH and Suwannee LS Monitoring Well	3	EA	$913,044	$2,739,000
Upper Monitoring Well	3	EA	$608,696	$1,826,000
Wellhead, Piping, and Electrical	5	EA	$913,044	$4,565,000
Reuse Pump Station	1	LS	$2,478,000	$2,478,000
Subtotal Project Cost				$25,304,000
Nonconstruction Project Costs at 20%				$5,061,000
Subtotal with Nonconstruction Project Costs				$30,365,000
Contractor Markups at 40%				$12,146,000
Subtotal with Contractor Markups				$42,511,000
Contingency of 20%				$8,502,000
Total Construction Cost				**$51,013,000**

EA = each

LS = lump sum

Based on the accuracy of this planning-level estimate, the range for the Class 5 capital cost estimate for Alternative 1A is $35,709,000 to $76,520,00.

Table 7-2. Alternative 1B Class V Capital Cost Estimate

Alternative 1B	Quantity	Units	Unit Cost	Total Cost
Avon Park Recharge Wells	10	EA	$3,652,174	$36,522,000
Avon Park Monitoring Wells	6	EA	$913,044	$5,478,000
Upper Monitoring Well	6	EA	$608,696	$3,652,000
Wellhead, Piping, and Electrical	10	EA	$913,044	$9,130,000
Reuse Pump Station	1	LS	$2,478,000	$2,478,000
Subtotal Project Cost				$57,260,000
Nonconstruction Project Costs at 20%				$11,452,000
Subtotal with Nonconstruction Project Costs				$68,712,000
Contractor Markups at 40%				$27,485,000
Subtotal with Contractor Markups				$96,197,000
Contingency of 20%				$19,239,000
Total Construction Cost				**$115,436,000**

Based on the accuracy of this planning-level estimate, the range for the Class 5 capital cost estimate for Alternative 1B is $80,805,000 to $173,154,000.

7.1.3.2 Operations and Maintenance and Total Present Worth Cost

A planning-level engineer's opinion of cost was estimated for O&M activities for Alternative 1A and 1B. Tables 7-3 and 7-4 provide the O&M cost on a 20-year net present worth cost basis. For net present worth analysis, a 3% discount rate was assumed over 20 years. Inflation was assumed to be 5%.

The O&M needs of the recharge well system are limited to water quality lab analysis, monitoring well pump replacement, Reuse Pump Station O&M (which includes all 6 pumps to be replaced once during the 20-year cycle and power costs). Power costs for the monitoring well pumps are assumed to be minimal, so this cost is accounted for within the 20% contingency provided in the O&M cost estimate.

Table 7-3. Alternative 1A Class 5 Operations and Maintenance Present Worth Estimate

Alternative 1A	Maintenance Frequency	Quantity	Units	Unit Cost	Annual O&M Cost (Year 1)
Monitoring Well System Sampling Lab Cost[a]	Annually	5	EA	$50,000	$250,000
Reuse Pump Station O&M[b]	Annually	1	LS	$750,000	$750,000
Acidization	Every 5 years	5	EA	$200,000	$1,000,000
Permitting	Every 5 years	1	LS	$100,000	$100,000
Monitoring Well Pump Replacement[c]	Once	3	EA	$16,000	$48,000
Present Worth of Annual Costs					$30,312,000
Contingency of O&M at 20%					$6,062,000
Total Present Worth of O&M					**$36,374,000**

[a] Monitoring well system sampling lab costs includes costs for monitoring of all five recharge wells

[b] Reuse Pump Station O&M is annualized cost assuming that each of the 6 pumps will be replaced once during the 20-year period.

[c] Three monitoring wells are required for the five recharge wells.

Table 7-4. Alternative 1B Class V Operations and Maintenance Cost Estimate

Alternative 1B	Maintenance Frequency	Quantity	Units	Unit Cost	Annual O&M Cost (Year 1)
Monitoring Well System Sampling Lab Cost[a]	Annually	10	EA	$50,000	$500,000
Reuse Pump Station O&M[b]	Annually	1	LS	$750,000	$750,000
Acidization	Every 5 years	10	EA	$200,000	$2,000,000
Permitting	Every 5 years	1	LS	$100,000	$100,000
Monitoring Well Pump Replacement[c]	Once	6	EA	$16,000	$96,000
Present Worth of Annual Costs					$41,643,000
Contingency of O&M at 20%					$8,329,000
Total Present Worth of O&M					**$49,972,000**

[a] Monitoring well system sampling lab costs includes costs for monitoring of all 10 recharge wells.

[b] Reuse Pump Station O&M is annualized cost assuming that each of the 6 pumps will be replaced once during the 20-year period.

[c] Six monitoring wells are required for the 10 recharge wells.

Table 7-5 provides a comparison of the capital cost and the O&M cost of Alternatives 1A and 1B on a 20-year net present worth cost basis. For net present worth analysis, a 3% discount rate was assumed over 20 years. Inflation was assumed to be 5%. As shown in Table 7-5, on a present worth basis, the total 20-year lifecycle cost of Alternative 1A is $87,387,000 and Alternative 1B is $165,408,000.

Table 7-5. Alternatives 1A and 1B Present Worth Cost Estimate

Present Worth Cost	Alternative 1A	Alternative 1B
Present Worth of O&M	$36,374,000	$49,972,000
Present Worth Capital Cost[a]	$51,013,000	$115,436,000
Total 20-Year Lifecycle Cost	**$87,387,000**	**$165,408,000**

[a] For the Class 5 estimate the range of cost could be within -30% to +50% of capital cost shown.

7.2 Alternative 2: Direct Potable Reuse

7.2.1 Alternative Description

Under this alternative, the feasibility of direct potable reuse (DPR), which involves the discharge of advanced treated reclaimed water directly into a drinking water treatment facility or into a drinking water distribution system, was evaluated. It should be noted that at the time of the writing of this report, the State of Florida does not have the legislation in place for direct potable reuse. However, it is anticipated that the new statues will be promulgated within the coming year. Some Florida communities have investigated and are also considering DPR. Utilities such as Altamonte Springs, Daytona Beach, Hillsborough County, and JEA have recently completed or are in the process of completing DPR pilot-scale or demonstration projects. For this alternative, the feasibility of constructing a 10-mgd capacity advanced water treatment facility (AWTF) to treat SEAWRF effluent to advanced treated water for direct potable reuse at the Green Meadow WTP was evaluated.

Under this alternative, it was assumed that the AWTF would be located onsite at the SEAWRF facility (Figure 7-4) and the DPR pipeline from the AWTF would be routed to the Green Meadow WTP located northwest of SEAWRF (Figure 7-5).

Figure 7-4. Alternative 2 Advanced Water Treatment Facility Potential Location at SEAWRF

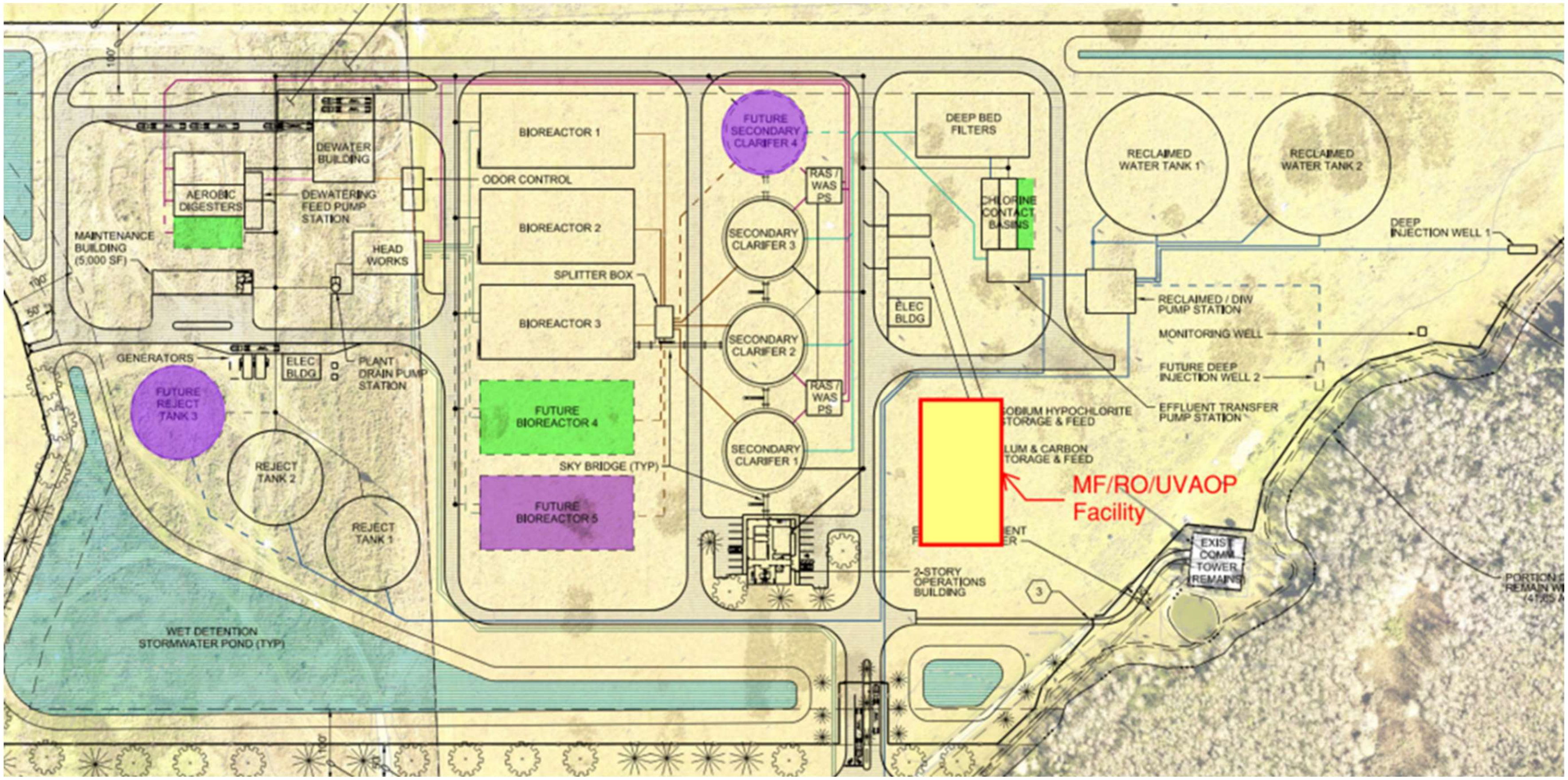
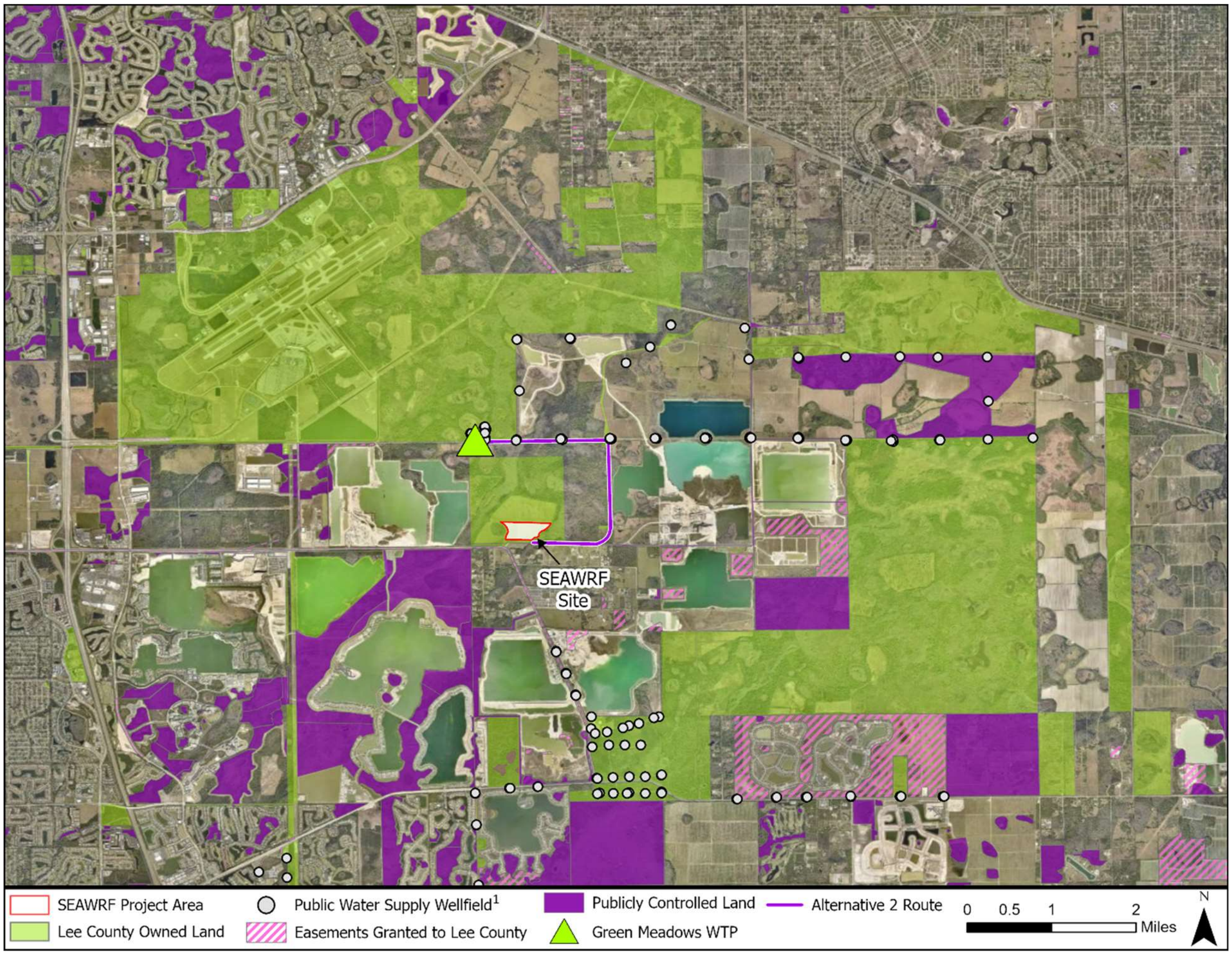


Figure 7-5. Location of Alternative 2 Direct Potable Reuse Pipeline Route from SEAWRF to Green Meadow WTP



[1] Wellfield includes both constructed wells and wells that are permitted but not constructed.

The direct potable reuse alternative evaluated was an AWTF with microfiltration, reverse osmosis, and ultraviolet advanced oxidation process (MF/RO/UVAOP) that ties into the SEAWRF just upstream of the chlorine contact basin (Figure 7-6). The AWTF's finished water would then be distributed to the intake of the Green Meadow WTP. Under this scenario, it is assumed that the RO concentrate is returned to SEAWRF and handled at the DIW. However, it should be noted that if the RO concentrate was disposed of at the SEAWRF DIW, the well would have to change to a Class I industrial well instead of the currently planned Class I municipal well. The DIW change to a Class I well would result in an approximate $4 million increase in cost.

Figure 7-6. Alternative 2 Direct Potable Reuse Process Flow Diagram

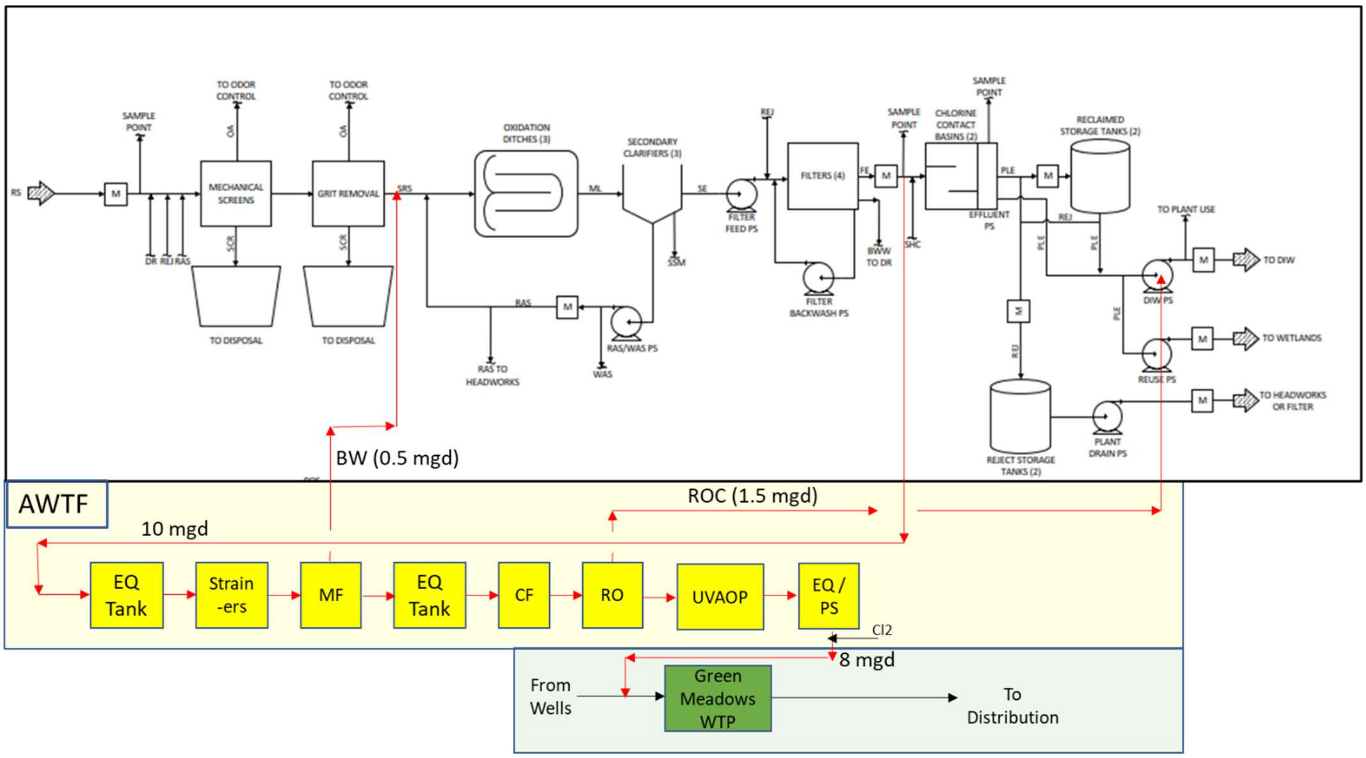


Table 7-6 displays the predicted treatment with Alternative 2's AWTF. As shown, the total log reduction values (LRVs) provided are anticipated to be above the total LRVs required under the existing draft rules. In the conceptual design of this system, the following assumptions were made:

- A site-specific pathogen removal study would be required at the SEAWRF to determine if any LRVs occur at the WRF, but for now it is assumed that zero LRVs occur at SEAWRF.
- No LRV pathogen credit would be provided by the SEAWRF CCBs because the water will be diverted to the AWTF prior to the CCBs to avoid disinfection byproducts (DBP) formation.

Table 7-6. Alternative 2 Direct Potable Reuse Treatment and Log Reduction Values Summary

Parameter	SEAWRF		AWTF				Total LRVs Provided	Total LRVs Required
	Secondary Treatment + Filtration[a]	CCB[b]	MF	RO	UVAOP	Cl_2[c]		
Enteric Viruses	0 to 2	0	0	1.5	6	6	13.5	12
Cryptosporidium	0 to 2	0	4	1.5	6	0	11.5	10
Giardia	0 to 2	0	4	1.5	6	0	11.5	10

[a] Site-specific pathogen removal study will be required; zero LRV was assumed.

[b] There is no pathogen credit because it is assumed water will be delivered prior to SEAWRF CCBs to avoid DBP formation.

[c] 6-log virus inactivation may require site-specific free chlorine CT study (note that 4-log virus removal achieved at CT of 4.8 mg/L-min at temperature 13.4°C per SWTF). CT is assumed to occur in pipeline from SEAWRF and Green Meadow WTP.

7.2.2 Regulatory and Treatment Requirements

7.2.2.1 Regulatory Requirements and Draft Florida Direct Potable Reuse Rules Review

At the time of the writing of this report, the State of Florida regulations do not address DPR. The basis for this analysis of the regulatory and treatment requirements was the language from the current draft rules from May 2021 for the following chapters:

- Chapter 62-550 FAC Drinking Water Standards, Monitoring, and Reporting
- Chapter 62-555 FAC Permitting, Construction, Operation, and Maintenance of Public Water Systems
- Chapter 62-610 FAC Reuse of Reclaimed Water and Land Application

Per the draft rules, advanced treated water is defined as "water produced from an advanced water treatment process for potable reuse applications." Advanced water treatment processes typically include RO and an oxidation process for emerging contaminants. An advanced water treatment facility is defined as "[t]he treatment facility where advanced treated water is produced. The specific combination of treatment technologies employed will depend on the quality of the source water, the type of potable reuse (i.e., indirect or direct potable reuse), and the existing treatment in place." For this alternative analysis, it was assumed that the AWTF would be located at the SEAWRF site upstream of the Green Meadow WTP. Direct potable reuse requires extensive testing and monitoring. The following is a summary of the requirements for a utility to implement direct potable reuse full scale per the draft rules:

- Must adopt an enhanced pretreatment program with enhanced source control
- Applicants will conduct a pilot study in accordance with Section 62-610.564
- Demonstrate the ability of the AWTF to provide a water source of the same quality or better than other sources used in the area
- Accumulate a minimum of 12 months of data using the final treatment design
- Applicants will conduct a full-scale test for at least 12 months with their final design after completion of the pilot test
- An emerging constituent monitoring protocol is required

The required studies must also identify and include the following evaluations:

- Evaluate how the system will treat the water to meet drinking water standards
- Identify any challenges they face in this treatment process
- Identify monitoring parameters to measure the performance of the system
- Identify critical control points to ensure the systems reliability and performance
- Evaluate the cost of the operation

7.2.2.2 Treatment Requirements per Draft Florida Direct Potable Reuse Rules

The AWTF must meet the following water quality performance requirements:

- All primary and secondary drinking water standards
- TOC ≤ 3mg/L
- TOX ≤ 0.2 mg/L
- 12 log – Viruses
- 10 log – *Cryptosporidium* Oocysts
- 10 log – *Giardia Lamblia*
- Excluding the domestic WWTP:
    - 8 log – Viruses
    - 5.5 log – *Cryptosporidium* Oocysts
    - 6 log – *Giardia Lamblia*

- Must also have multiple barriers required for resiliency, redundancy, and robustness (per new draft Rule Chapter 62-610.563)

7.2.3 Cost Estimate

7.2.3.1 Capital Cost

A Class 5 planning-level engineer's opinion of probable capital cost was prepared for the Alternative 2 10-mgd AWTF. Table 7-7 presents a Class 5 cost estimate that has a predicted level of accuracy of -30% to +50%, consistent with the terminology and practices recommended for conceptual screening analysis by the Association of the Advancement of Cost Engineering. The accuracy of these planning-level cost estimates is intended to help Lee County compare the capital cost for construction of each proposed alternative.

The capital cost estimates in Table 7-7 are based on the conceptual design for Alternative 2 as described previously and included the following assumptions:

- Lee County already owns the land, so no land purchase is required.
- No clearing and grubbing of land are required.
- Reuse Pump Station has a maximum capacity of 19 mgd (the maximum potential flow rate).
- A Class I DIW will be required to manage the RO concentrate. For this stage of planning, a high-level cost was assumed.

Table 7-7. Alternative 2 Class 5 Capital Cost Estimate

Alternative 2	Quantity	Units	Unit Cost	Total Cost
AWTF (MF/RO/UVAOP)	1	LS	$49,010,000	$49,010,000
Reuse Pump Station	1	LS	$2,478,000	$2,478,000
DIW	1	LS	$10,000,000	$10,000,000
30-inch Distribution System (3.1 mi)	1	LS	$5,376,000	$5,376,000
Subtotal Project Cost				$66,864,000
Nonconstruction Project Costs at 20%				$13,373,000
Subtotal with Nonconstruction Project Costs				$80,237,000
Contractor Markups at 40%[a]				$32,095,000
Subtotal with Contractor Markups				$112,332,000
Contingency of 20%				$22,466,000
Total Construction Cost				**$134,798,000**

[a] Contractor markups include overhead, profit, mobilization, bonds, and insurance.

Based on the accuracy of this estimate, the range for the Class 5 capital cost estimate for Alternative 2 is $94,359,000 to $202,197,000.

7.2.3.2 Operations and Maintenance and Total Present Worth Cost

A planning-level engineer's opinion of cost was estimated for O&M activities for Alternative 2. Table 7-8 provides the O&M cost on a 20-year net present worth cost basis. For net present worth analysis, a 3% discount rate was assumed over 20 years. Inflation was assumed to be 5%.

The O&M needs of the AWTF are limited to routine O&M, chemicals, power, labor required to run the AWTF, and Reuse Pump Station O&M (which includes all 6 pumps to be replaced once during the 20-year cycle and power costs). This estimate does not include the O&M needs that would be required for the Class I industrial DIW required to handle the RO concentrate from the AWTF.

Table 7-8. Alternative 2 Class 5 Operations and Maintenance Cost Estimate

Alternative 2	Maintenance Frequency	Annual O&M Cost (Year 1)
AWTF O&M	Annually	$3,420,000
Pump Station O&M[a]	Annually	$750,000
Present Worth of Annual Costs		$102,690,000
Contingency of O&M at 20%		$20,538,000
Total Present Worth of O&M		**$123,228,000**

[a] Reuse Pump Station O&M is annualized cost assuming that each of the 6 pumps will be replaced once during the 20-year period.

Table 7-9 provides a comparison the capital cost and the O&M cost of Alternative 2 on a 20-year net present worth cost basis. For net present worth analysis, a 3% discount rate was assumed over 20 years. Inflation was assumed to be 5%. As shown in Table 7-9, on a present worth basis, the total 20-year lifecycle cost of Alternative 2 is $258,026,000.

Table 7-9. Alternative 2 Class 5 Present Worth Cost Estimate

Present Worth Cost	Alternative 2
Present Worth of O&M	$123,228,000
Present Worth Capital Costs[a]	$134,798,000
Total 20-Year Lifecycle Cost	**$258,026,000**

[a] For the Class 5 estimate, the range of cost could be within -30% to +50% of capital cost shown.

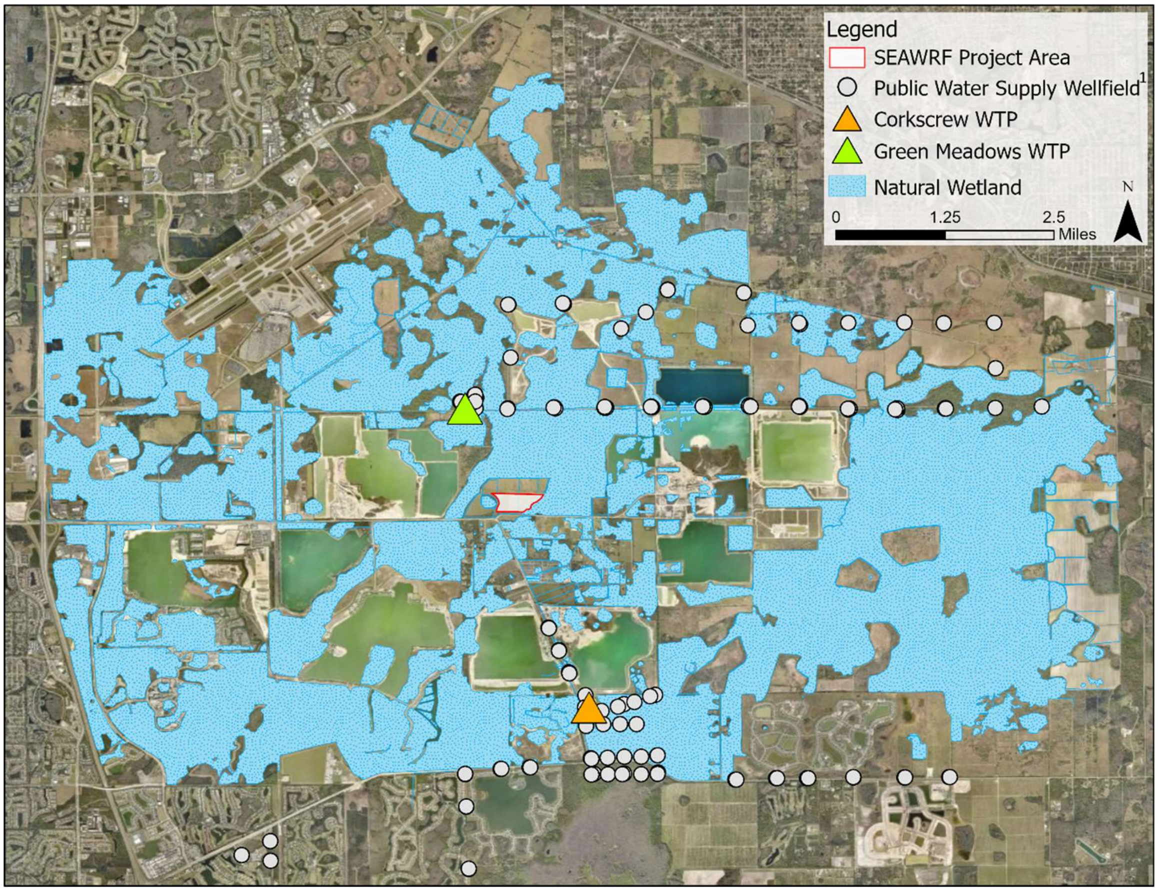
7.3 Alternative 3: Receiving Wetlands Application

7.3.1 Alternative Description

Chapter 62-610, FAC, Reuse of Reclaimed Water and Land Application classifies the creation, restoration, and enhancement of wetlands to be classified as "reuse." Therefore, the third alternative proposes to use existing natural wetlands to receive SEAWRF AWT reclaimed water as allowed under the Wetlands Application Rule (Rule), Chapter 62-611, FAC. The application of this alternative is straightforward and the application of reclaimed water to natural wetlands has been implemented across the State of Florida, including 19 operational facilities. Some of these operational receiving wetlands are adjacent to public supply wellfields for the purposes of offsetting water use impacts. Lee County has many existing large wetlands near SEAWRF that are hydraulically connected to the water supply of County owned and operated water treatment plants (Figure 7-7). Under the Rule, the discharge of the reclaimed water to natural wetlands would be defined as a "Receiving Wetland," which is defined as a natural wetland within the landward extent of waters of the state used to receive reclaimed water that contains not more, on an annual average basis, than the AWT standards with the following concentrations:

- Carbonaceous biological oxygen demand of 5 mg/L
- TSS of 5 mg/L
- TN (as N) of 3 mg/L
- TP (as P) of 1 mg/L

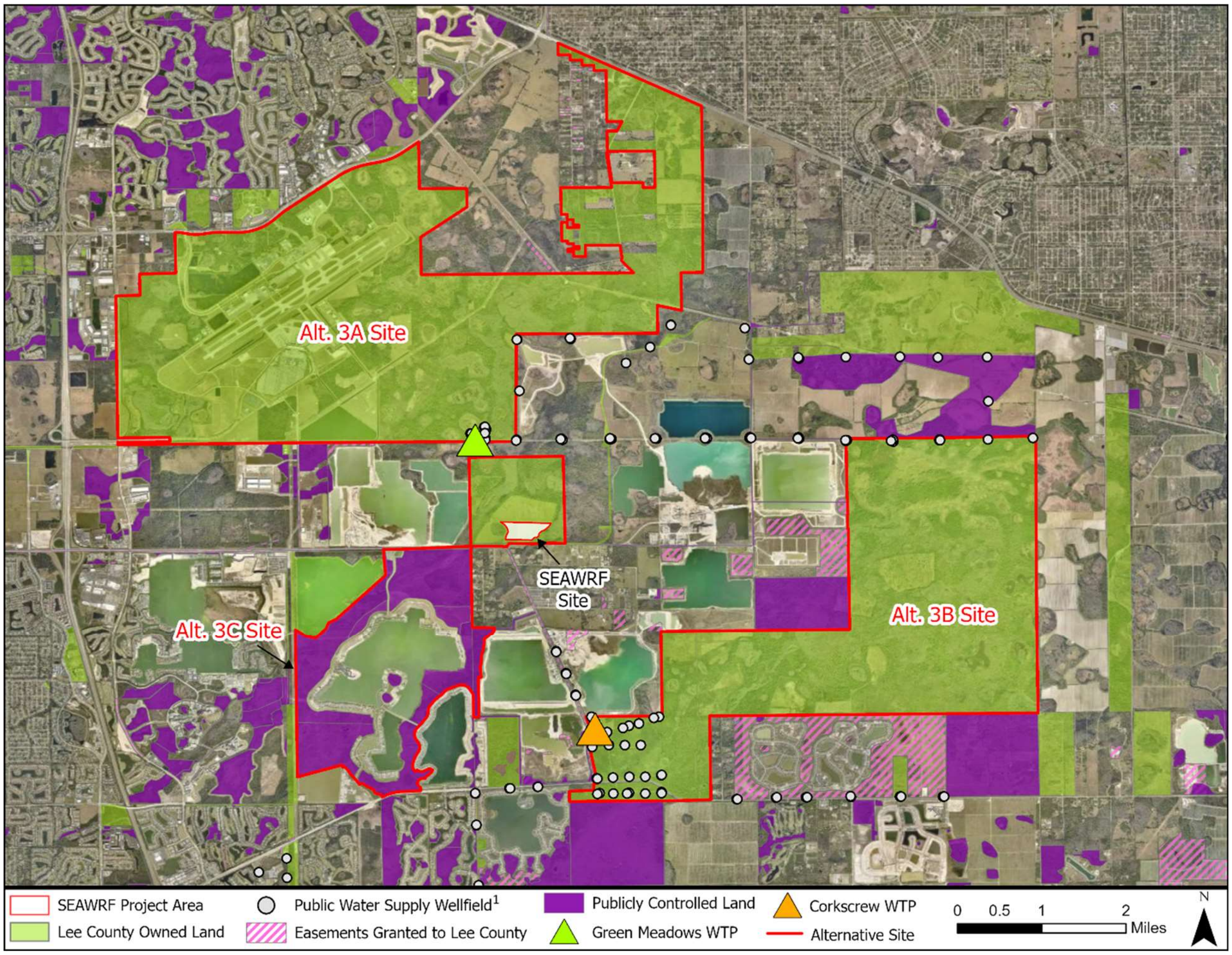
Figure 7-7. Natural Wetlands in Relation to Southeast Advanced Water Reclamation Facility Site



[1] Wellfield includes both constructed wells and wells that are permitted but not constructed.

The use of wetlands as a receiving wetland is allowed if the wetland is not within Class I or II waters and is not considered an "Herbaceous Wetland" as defined in the Rule. A wetland in which the herbaceous ground cover is greater than 30% of the uppermost stratum is considered an "Herbaceous Wetland." The natural wetlands near SEAWRF (Figure 7-7) are considered woody wetlands per the Rule because the wetland landcover consists of woody vegetation that is equal to or greater than 70% of the uppermost stratum. This alternative was evaluated for feasibility because natural wetlands for water reuse have a well-defined and straightforward permitting process and simple infrastructure of an inflow distribution system and monitoring stations. As shown on Figure 7-8, three individual sites near the SEAWRF site were evaluated for wetland application feasibility.

Figure 7-8. Wetland Application Alternative Sites in Relation to Southeast Advanced Water Reclamation Facility Site

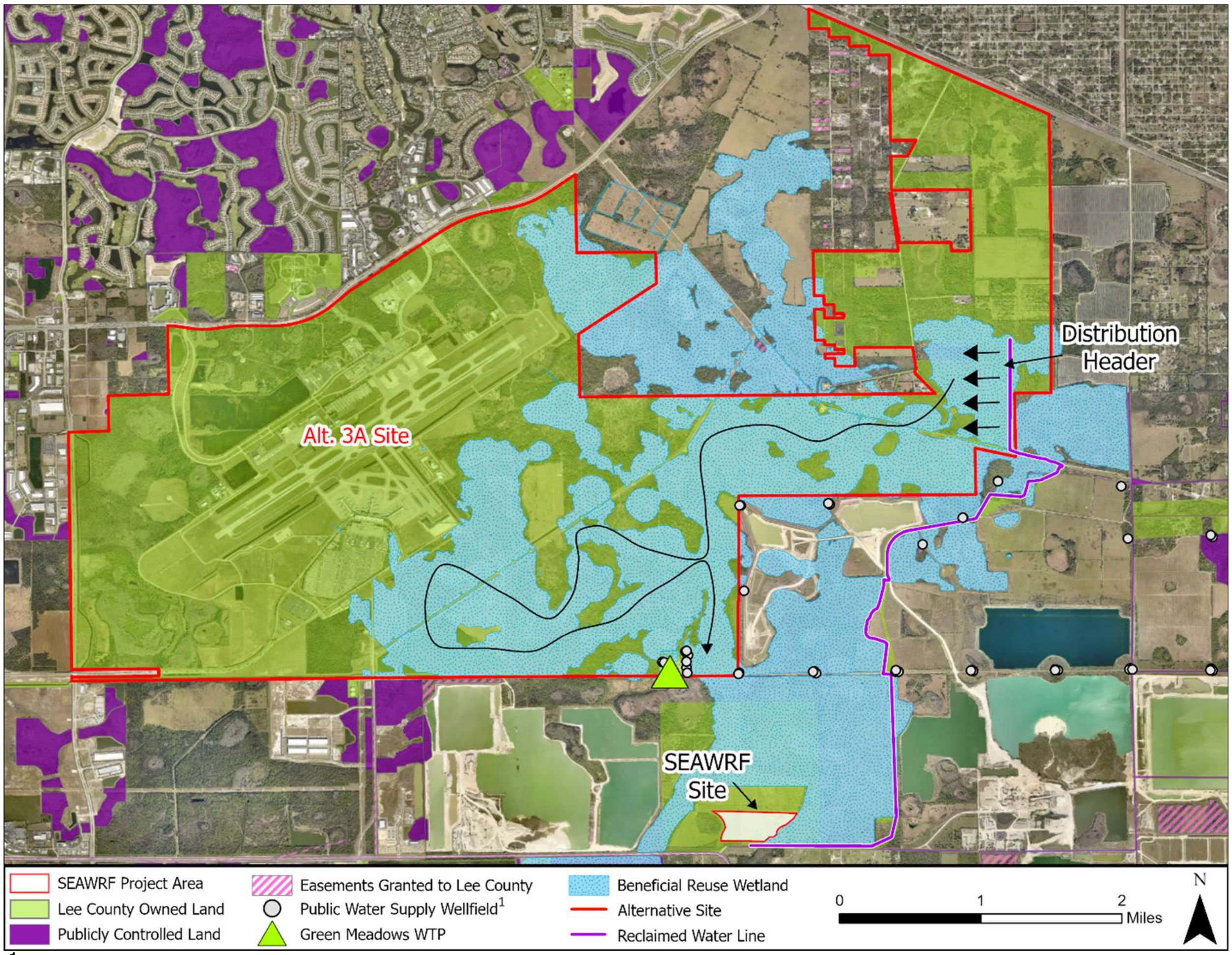


[1] Wellfield includes both constructed wells and wells that are permitted but not constructed.

7.3.1.1 Alternative 3A – Northern Area Site

Alternative 3A, the Northern Area wetlands site, contains 2,840 acres of natural wetlands, as shown on Figure 7-9. Discharge of reclaimed water to this site would require approximately 5.5 miles of pipeline, a portion of which would run along the new Alico Road corridor and another portion of which would require an easement. This alternative has approximately 1 mile available for distribution and would require additional permitting to construct the distribution system within the wetland. This option provides easy access to the required monitoring stations for the baseline and operational monitoring programs that the Rule requires. In addition, this alternative would augment the local water resources and help to offset groundwater withdrawals from the Green Meadow WTP wellfield.

Figure 7-9. Alternative 3A Northern Area Receiving Wetland Application Site and Potential Conveyance and Distribution System



[1] Wellfield includes both constructed wells and wells that are permitted but not constructed.

Table 7-10 compares the Rule design parameter requirements with Alternative 3A's design parameters. Assuming a receiving wetland inflow of 10 mgd, this alternative easily meets the minimum requirements as shown in Table 7-10. However, following a November 2022 coordination meeting with the Lee County Port Authority , it was determined that the Northern Area wetland area will likely decrease due to the Port Authority's future land use plans. Long-term wetland area availability should be considered for this alternative should it be carried forward for further evaluation.

Table 7-10. Comparison of Rule Design Parameter Requirements with Alternative 3A Design Parameters

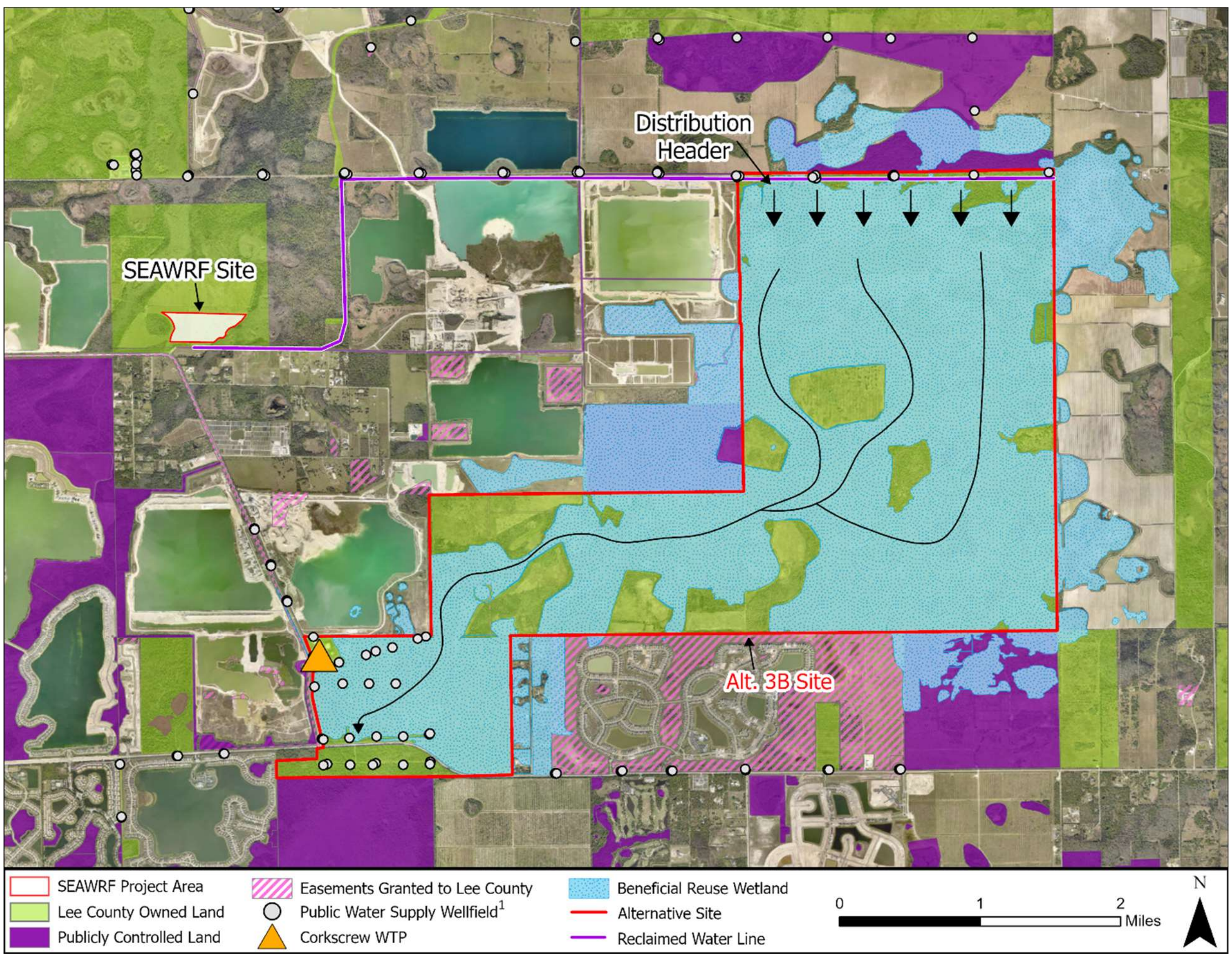
Design Parameter	Rule Requirement	Alternate 3A Design
HRT (days)	≥ 14	92
HLR (inches per week)	≤ 2	0.9
TN Load (g/m^2 per year)	≤ 25	3.6
TP Load (g/m^2 per year)	≤ 3	1.2

g/m^2 = gallon(s) per square meter

7.3.1.2 Alternative 3B – Southwest Florida International Airport Mitigation Park

The second potential receiving wetland application site is at the Southwest Florida International Airport (SWFIA) Mitigation Park, which contains 6,910 acres of natural wetlands, as shown on Figure 7-10. Discharge of reclaimed water to this site would require approximately 6.0 miles of pipeline, a portion of which would run along the new Alico Road corridor. This alternative has approximately 2 miles available for a wetland distribution and the opportunity to construct the distribution system within uplands. This would significantly simplify the permitting process and construction requirements. This option has easy access to the required monitoring stations for the baseline and operational monitoring program, as the Rule requires. This alternative also provides some benefit associated with enhancing the water resources of local wetlands and offsetting withdrawals from the Corkscrew WTP wellfield. This alternative may also offset some of the groundwater withdrawal of Green Meadow WTP wells. This alternative has several benefits because this site is a mitigation park. For example, the wetland mitigation park's baseline conditions have been well documented with existing monitoring systems already in place, and opportunities to share management responsibilities with the Lee County Port Authority are possible.

Figure 7-10. Alternative 3B Southwest Florida International Airport Mitigation Park Receiving Wetland Application Site and Potential Conveyance and Distribution System



[1] Wellfield includes both constructed wells and wells that are permitted but not constructed.

Table 7-11 compares the Rule design parameter requirements with Alternative 3B's design parameters. Assuming a receiving wetland inflow of 10 mgd, this alternative easily meets the minimum requirements as shown in Table 7-11.

Table 7-11. Comparison of Rule Design Parameter Requirements with Alternative 3B Design Parameters

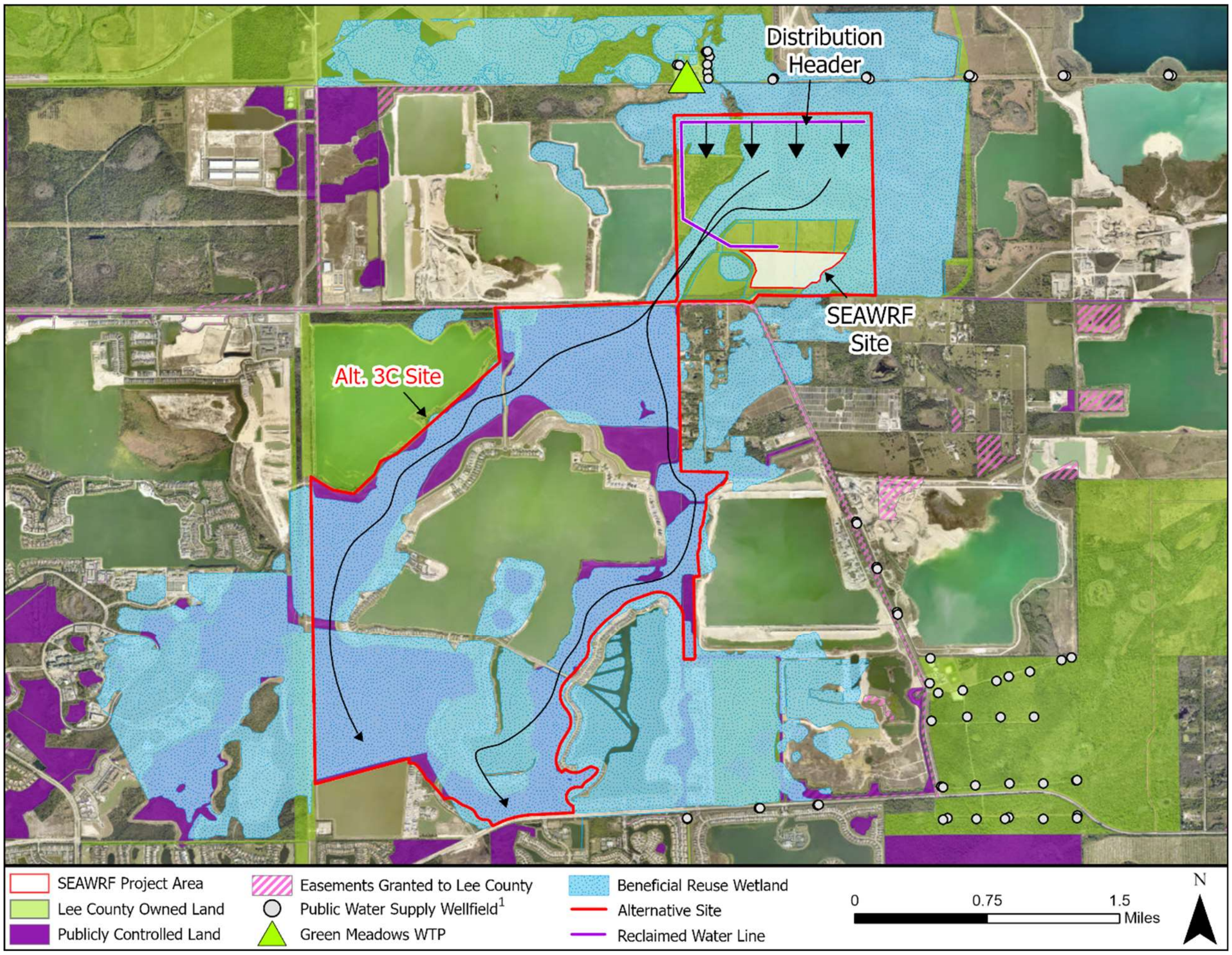
Design Parameter	Rule Requirement	Alternate 3B Design
HRT (days)	≥ 14	225
HLR (inches per week)	≤ 2	0.4
TN Load (g/m^2 per year)	≤ 25	1.5
TP Load (g/m^2 per year)	≤ 3	0.5

This application site is preferred over the Northern Area site because the Lee County Port Authority has plans to expand its footprint within the Northern Area and the County does not have existing easements along the entire proposed conveyance route. Although this site was originally within the Imperial River Basin Management Action Plan, following a meeting with the South District of the FDEP, one of the FDEP representatives noted that the Basin Management Action Plan (BMAP) will not prohibit the project. This BMAP is scheduled to be updated in 2025 and is anticipated to have similar wastewater requirements to that of the Caloosahatchee BMAP.

7.3.1.3 Alternative 3C – Wetland Application Adjacent to SEAWRF

Alternative 3C is the final receiving wetland application site considered and is located just north of SEAWRF on County-owned land and on publicly controlled area southwest of SEAWRF. This site contains 2,040 acres of natural wetlands (Figure 7-11). Discharge of reclaimed water to this site would require approximately 2 miles of pipeline. This alternative has approximately 1 mile available for distribution and would require additional permitting to construct the distribution system within the wetland. Although a portion of this alternative's application area is owned by Lee County and located on the SEAWRF property, the publicly controlled land area, to the southwest, would require agreements with the landowners. In addition, this alternative would have significant permitting issues because the pipeline route would affect natural wetlands. Finally, it is likely that the applied reclaimed water could flow offsite to privately owned wetlands, and this would complicate the permitting of this project. Due to its location, this alternative does not provide a benefit to the water supply because the wells are not located within Alternative 3C's application area.

Figure 7-11. Alternative 3C Area Adjacent to SEAWRF Receiving Wetland Application Site and Potential Conveyance and Distribution



[1] Wellfield includes both constructed wells and wells that are permitted but not constructed.

Table 7-12 compares the Rule design parameter requirements with Alternative 3C's design parameters. Assuming a receiving wetland inflow of 10 mgd, this alternative meets the minimum requirements as shown in Table 7-12.

Table 7-12. Comparison of Rule Design Parameter Requirements with Alternative 3c Design Parameters

Design Parameter	Rule Requirement	Alternate 3C Design
HRT (days)	≥14	67
HLR (inches per week)	≤2	1.3
TN Load (g/m²/year)	≤25	5
TP Load (g/m²/year)	≤3	1.7

7.3.2 Regulatory and Treatment Requirements

The regulatory pathway for the receiving wetlands application alternative is well defined under the Rule. An NPDES permit will be required and must be obtained before project implementation.

For Alternatives 3A and 3C, which require construction within the wetland, a State 404 permit issued by the FDEP may be required. All alternatives would require an ERP permit, and this would be permitted through FDEP or SFWMD. It was recommended by the FDEP that a joint FDEP/SFWMD preapplication meeting should occur to finalize jurisdiction.

A baseline monitoring study and report will be required as part of the permit application, and once the system is in operation, quarterly monitoring events are required. Figure 7-12 presents the monitoring program requirements for both the baseline monitoring program and the operational monitoring program required for a receiving wetland per the Rule. For the monitoring, a minimum of three permanent stations would be developed and must be located at the following sites:

- Point of discharge to the wetland
- Approximate geographical midpoint
- Point of discharge from the wetland

Under this proposed solution, no additional treatment requirements would be required to discharge the AWT reclaimed water to a natural receiving wetland. Discharge from the receiving wetland would be monitored by the operational monitoring program to ensure that on an annual average discharge from the receiving wetland does not contain more than 3 mg/L TN and 0.2 mg/L TP. To ensure the wetland achieves the discharge goals, the Rule provides several requirements to apply reclaimed water to a receiving wetland. Per the Rule, the discharge of reclaimed water to the receiving wetland should accomplish the following:

- Minimize channelized flow and maximize sheet flow
- Minimize the loss of dissolution of sediments due to erosion
- Not cause adverse effects on endangered or threatened species
- Have a hydraulic loading that minimizes the alteration of the natural hydroperiod (less than or equal to 2 inches per week). This would require a land area of approximately 129 acres per one mgd
- Have 24-hour storage of reclaimed water for off-spec water
- Have a minimum hydraulic detention time in the wetland of greater than 14 days
- Have an influent nutrient load of less than or equal to 25 grams (g) TN/m^2/year or 3 g TP/m^2/year

For further development of this alternative, comprehensive modeling would be required to understand impacts to the natural wetland hydroperiod. This hydrologic modeling would be required to ensure that the natural wetland water budgets are not altered in a way that would negatively affect the natural wetland vegetation communities.

Figure 7-12. Chapter 62-611.700 *Florida Administrative Code* Table 3 Monitoring Program for a Receiving Wetland

TABLE 3
MONITORING PROGRAM FOR A RECEIVING WETLAND

PARAMETER	Baseline Monitoring[1] Program			Operational Monitoring[2] Program		
	Surface[3]	Sediment[3]	Biota[4]	Surface[3]	Sediment[3]	Biota[4]
Temperature	O(DD)			Q(DD)		
Dissolved Oxygen	O(DD)			Q(DD)		
pH	O			Q		
Conductivity	O			Q		
Cl_2 (TRC)						
Color						
$CBOD_5$	O			Q		
TSS	O			Q		
TP (as P)	O			Q		
OP (as P)						
TKN (as N)	O			Q		
NH^3 (as N)	O			Q		
NO_3 - NO_2 (as N)	O			Q		
SO_4 (as S)	O			Q		
$S^=$ (as S)		O			A	
Fecal Coliforms	O			Q		
Chl a	O			Q		
Non-metallic priority pollutants						
Metals (Hg, Pb, Cd, Cu, Zn, Fe, Ni, Ag)						
Stage[5]	C			C		
Benthic Macroinvertebrates						
Woody Vegetation			O			A
Herbaceous Vegetation-line intercept method			O			Q
Fish			O			Q
Mosquitoes						
Threatened and Endangered Plant and Animal Species List			O			A
Plant Tissue Analysis[6] (Hg, Pb, Cd, Cr, Cu, Zn, Fe, Ni, Ag, TKN, TP)						
Plant Tissue Analysis[7] (TP, TKN, Fe, Zn)						

M = monthly
DO = 48 hour dawn-dusk, max of four hour intervals
O = once during baseline monitoring period
C = continuous

A = annually
SA = semi-annually
Q = quarterly

[1]Domestic wastewater treatment plant monitoring shall be conducted pursuant to Chapter 62-601, FAC.
[2]A minimum of two stations shall be sampled in the wetland; 1) at the proposed point of discharge to, and 2) at the point of discharge from the wetland. Additional stations may be required to determine compliance with section 62-611, FAC.
[3]A minimum of three permanent stations shall be established as follows; 1) in the immediate vicinity of the point of discharge to the wetland, 2) in the approximate geographical middle of the wetland, and 3) in the vicinity of the point(s) of discharge from the wetland. Additional stations may be required to determine compliance with section 62-611, FAC.
[4]Monthly monitoring required if wetland used for phosphorus treatment.
[5]Stage used to determine flow and only required at the point(s) of discharge from the wetland.

7.3.3 Cost Estimate

7.3.3.1 Capital Cost

A Class 5 planning-level engineer's opinion of probable capital cost and O&M cost was prepared for Alternative 3A, Alternative 3B, and Alternative 3C. Tables 7-13 through 7-15 present Class 5 cost estimates that have a predicted level of accuracy of -30% to +50%, consistent with the terminology and practices recommended for conceptual screening analysis by the Association of the Advancement of Cost Engineering. The accuracy of these planning-level cost estimates is intended to help Lee County compare the capital cost for construction of each proposed alternative.

The capital cost estimates provided in Tables 7-13, 7-14, and 7-15 are based on the conceptual design for Alternatives 3A, 3B, and 3C as described previously and included the following assumptions:

- Alternative 3A assumes 6.5 miles of 30-inch distribution pipeline, Alternative 3B assumes 8 miles of 30-inch distribution pipeline, and Alternative 3C assumes 2 miles of 30-inch distribution pipeline.
- Installation of distribution system assumes ductile iron pipe and 4 feet of cover.
- Reuse Pump Station has a maximum capacity of 19 mgd (the maximum potential flow rate).
- Lee County already owns the land, so no land purchase is required.
- No clearing and grubbing of land are required.

Table 7-13. Alternative 3A Class 5 Capital Cost Estimate

Alternative 3A	Quantity	Units	Unit Cost	Total Cost
Reuse Pump Station	1	LS	$2,478,000	$2,478,000
30-inch Distribution System (6.5 miles)	1	LS	$11,127,000	$11,127,000
Subtotal Project Cost				$13,605,000
Nonconstruction Project Costs at 20%				$2,721,000
Subtotal with Additional Project Costs				$16,326,000
Contractor Markups at 40%[a]				$6,530,000
Subtotal with Contractor Markups[a]				$22,856,000
Contingency of 20%[a]				$4,571,000
Total Construction Cost				$27,427,000

[a] Contractor markups include overhead, profit, mobilization, bonds and insurance.

Based on the accuracy of this estimate, the range for the Class 5 cost estimate for Alternative 3A is $19,199,000 to $41,141,000.

Table 7-14. Alternative 3B Class 5 Capital Cost Estimate

Alternative 3B	Quantity	Units	Unit Cost	Total Cost
Reuse Pump Station	1	LS	$2,478,000	$2,478,000
30-inch Distribution System (8 miles)	1	LS	$13,874,000	$13,874,000
Subtotal Project Cost				$16,352,000
Nonconstruction Project Costs at 20%				$3,270,000
Subtotal with Nonconstruction Project Costs				$19,622,000
Contractor Markups at 40%[a]				$7,849,000
Subtotal with Contractor Markups				$27,471,000

Alternative 3B	Quantity	Units	Unit Cost	Total Cost
Contingency of 20%				$5,494,000
Total Construction Cost				$32,965,000

[a] Contractor markups include overhead, profit, mobilization, bonds, and insurance.

Based on the accuracy of this estimate, the range for the Class 5 cost estimate for Alternative 3B is $23,076,000 to $49,448,000.

Table 7-15. Alternative 3C Class 5 Capital Cost Estimate

Alternative 3C	Quantity	Units	Unit Cost	Total Cost
Reuse Pump Station	1	LS	$2,478,000	$2,478,000
30-inch Distribution System (2 miles)	1	LS	$3,989,000	$3,989,000
Subtotal Project Cost				$6,467,000
Nonconstruction Project Costs at 20%[a]				$1,293,000
Subtotal with Additional Project Costs				$7,760,000
Contractor Markups at 40%				$3,104,000
Subtotal with Contractor Markups[b]				$10,864,000
Contingency of 20%				$2,173,000
Total Construction Cost				$13,037,000

Based on the accuracy of this estimate, the range for the Class 5 cost estimate for Alternative 3C is $9,126,000 to $19,556,000.

7.3.3.2 Operations and Maintenance and Total Present Worth Cost

A planning-level engineer's opinion of cost was developed for O&M activities for Alternatives 3A, 3B, and 3C. For this cost comparison estimate, it is assumed that the O&M activities would be approximately the same for Alternatives 3A, 3B, and 3C. Although Alternative 3C's distribution pipeline is a shorter run and because of this, the power costs for Alternative 3C are likely lower than for Alternatives 3A and 3B, for this high-level conceptual cost comparison, the difference in pump station power consumption was not accounted for in the O&M costs. Table 7-16 also provides the O&M cost on a 20-year net present worth cost basis. For net present worth analysis, a 3% discount rate was assumed over 20 years. Inflation was assumed to be 5%. The total present worth cost of both alternatives was calculated by adding the capital cost to the present worth cost of O&M.

The O&M needs of the wetland application well system is limited to operational monitoring, wetland water quality lab analysis, and Reuse Pump Station O&M (which includes all six pumps to be replaced once during the 20-year cycle and power costs).

Table 7-16. Alternative 3A, 3B, and 3C Class 5 Operations and Maintenance Cost Estimate

Alternatives 3A, 3B and 3C	Maintenance Frequency	Annual O&M Cost (Year 1)
Operational Monitoring	Annually	$50,000
Maintenance[a]	Annually	$25,000
NPDES Permit Renewal[b]	Every 5 years	$50,000
Pump Station O&M[c]	Annually	$750,000
Present Worth of Annual Costs		$20,567,000

Alternatives 3A, 3B and 3C	Maintenance Frequency	Annual O&M Cost (Year 1)
Contingency of O&M at 20%		$4,113,000
Total Present Worth of O&M		**$24,680,000**

[a] General maintenance for miscellaneous items such as distribution cleanout or valve exercising and repair.

[b] Includes cost for effluent toxicity testing.

[c] Pump station O&M is annualized cost assuming that each of the 6 pumps will be replaced once during the 20-year period.

Table 7-17 provides a comparison the Capital cost and the O&M cost of Alternatives 3A, 3B, and 3C on a 20-year present worth cost basis. As shown in Table 7-17, on a present worth basis the total 20-year lifecycle cost for Alternative 3A is $52,107,000, Alternative 3B is $57,645,000 and Alternative 3C is $37,669,000.

Table 7-17. Alternatives 3A, 3B, and 3C Class 5 Present Worth Cost Estimate

Present Worth Cost	Alternative 3A	Alternative 3B	Alternative 3C
Present Worth of O&M	$24,680,000	$24,680,000	$24,680,000
Present Worth Capital Costs[a]	$27,427,000	$32,965,000	$13,037,000
Total 20-Year Lifecycle Cost	**$52,107,000**	**$57,645,000**	**$37,669,000**

[a] For the Class 5 estimate the range of cost could be within -30% to +50% of capital cost shown.

7.4 Recommendations

Three alternatives for reuse of the SEAWRF AWT effluent were evaluated for feasibility, regulatory and treatment requirements:

- Alternative 1: Indirect potable reuse via Floridan aquifer recharge using recharge wells
- Alternative 2: Direct potable reuse via the construction of an AWTF at SEAWRF
- Alternative 3: Receiving wetlands application through the beneficial reuse of effluent at a natural forested wetland site

Overall, the Alternative 3 receiving wetlands application alternative was deemed to be the most cost-effective for the County and the most beneficial solution for the County's water resources and the local environment. This alternative was highly supported by the regulatory agencies due to its multiple benefits such as water reuse, ecosystem hydration and enhancement, increased ecosystem productivity, water supply augmentation, and protection of public health. In addition, as shown in Table 7-18 this alternative is significantly lower in cost as compared to the Alternative 1A and 1B Recharge Wells and Alternative 2 Direct Potable Reuse. Although this alternative would require quarterly monitoring, the monitoring costs are significantly lower when compared to the operational costs of Alternatives 1A, 1B, and 2.

Table 7-18. Summary of Present Worth Cost for Alternatives

Present Worth Cost	Alternative 1A	Alternative 1B	Alternative **2**	Alternative 3A	Alternative 3B	Alternative 3C
Present Worth of O&M	$36,374,000	$49,972,000	$102,690,000	$24,680,000	$24,680,000	$24,680,000
Present Worth Capital Cost[a]	$51,013,000	$115,436,000	$134,798,000	$27,427,000	$32,965,000	$13,037,000
Total 20-Year Lifecycle Cost	**$87,387,000**	**$165,408,000**	**$236,966,000**	**$52,107,000**	**$57,645,000**	**$37,669,000**

[a] For the Class 5 estimate the range of cost could be within -30% to +50% of capital cost shown.

Between Alternatives 3A, 3B, and 3C it was determined that Alternative 3B was the more feasible solution because Alternative 3A and 3C have several site and permitting constraints. After a meeting with the Lee County Port Authority in November 2022, it was determined that the Port had development plans for the Alternative 3A Northern Area site location that may make this alternative infeasible. In addition, an easement would be required to allow for the needed pipeline crossings. Implementation of Alternative 3C is contingent on several landowners and would have an impact on wetlands during construction, complicating the permitting process. Alternative 3B would also provide more benefits than Alternatives 3A and 3C based on its location and opportunities to enhance local water resources. Alternative 3B would provide benefits associated with augmenting water resources of not only the Corkscrew WTP wellfield area, but also parts of the Green Meadow WTP wellfield area. In addition, because Alternative 3B is already a Mitigation Park, the wetland's baseline conditions have been well-documented with existing monitoring already in place and opportunities to share some management responsibilities with the mitigation park are possible.

8 Project Delivery Approach

The Phase 1 project will be delivered using the construction management at-risk (CMAR) approach. This approach allows the contractor to bring construction insight to the project as early as practical in the design process. The method maintains two separate contracts with the owner but encourages collaboration between the engineer and contractor during design to reduce risk. Once selected by Lee County, the CMAR provider will coordinate with the design engineer and provided information needed to develop cost estimates for the project. The CMAR provider will provide constructability review and input to the design engineer for design approaches to reduce construction cost. It is anticipated that a CMAR provider will be under contract with Lee County as the project completes the 30% plans.

9 References

Lee County. 2019. *Lee County Instrumentation Standards*.

Lee County. 2011. *Capacity Analysis Report*.

Johnson Engineering, Inc. (JEI). 2016. *Corkscrew Overlay Area Wastewater Master Planning Report*.

U.S. Environmental Protection Agency (EPA). 1974. *Design Criteria for Mechanical, Electric, and Fluid System and Component Reliability*.

Wastewater Committee of the Great Lakes. 2014. *Recommended Standards for Wastewater Facilities* (Ten State Standards).

Water Environment Federation (WEF). 2018. *Water Environment Federation Manual of Practice No. 8*.

Appendix A. Lee County Instrumentation Standards

LEE COUNTY UTILITIES INSTRUMENTATION STANDARDS

PROCESS VARIABLE	MEASUREMENT	SERVICE	MANUFACTURER	MODEL	PART NUMBER	VENDOR	NOTES
Analytical	Chlorine Residual	Wastewater aplications, w/Reagents	Emerson	TCL(total chlorine system)	TCL-11-280-32/mount bracket#28020-00	Emerson Rosemount Inc	Total Chlorine, Either the SFC or the MFC depending on application. Cable #23747-02 10'
Analytical	Chlorine Residual Analyzer Transmitter	Water Aplications, w/Reagentless	Evoqua	Depolox 3 +	W3T166418	Water Treatment & Controls Co.	Specify whether Free or Total Chlorine Probe
Analytical	Chlorine Residual Analyzer Free Chlorine Probe	Water Aplications, w/Reagentless	Evoqua	Depolox 3 +	W3T164492	Water Treatment & Controls Co.	Free Chlorine
Analytical	Chlorine Residual Analyzer Total Chlorine Proble	Water Aplications, w/Reagentless	Evoqua	Depolox 3 +	W3T171787	Water Treatment & Controls Co.	Total Chlorine,
Analytical	Conductivity Transmitter	Wall, panel, or pipe mount	Yokogawa	EXAxt 450	SC450G-A-A**	Classic Controls	** installation specific option.
Analytical	Conductivity Probe	Low conductivity	Yokogawa		SC4A-S-**PR-NN**-010-15-T1Q	Classic Controls	PR is for retractable fixed length.
Analytical	Conductivity Probe	Low conductivity	Yokogawa		SC4A-S-**AD-09**-010-20-T1/Q	Classic Controls	AD is for adapter mounting, 09 is the short one with angular cable entry for Pinewoods WTP.
Analytical	Dissolved Oxigen Transmitter	Wastewater	Hach	SC200	LXV404.99.00552	Hach	2 Channel inputs, 2 mA outputs, digital is for plug and play with the probes.
Analytical	Dissolved Oxigen Probe	Wastewater	Hach	LDO Probe	9020000	Hach	
Analytical	Iron	Water	ABB	Aztec	AB AW63353000010	AWW, Inc.	Also order the reagent kit AB AWRK6330619US, 2 month supply
Analytical	NTU	Water and Wastewater	Swan	Turbiwell	A-25.411.700.1	Swan	
Analytical	NTU	Water and Wastewater	H.F. Scientific	MicroTOL 2	20053	H.F. Scientific	Also order preliminary calibration kit full range PN: 29957
Analytical	pH Transmitter	Wall, panel, or pipe mount	Yokogawa	EXAxt 450	PH450G-A-A**	Classic Controls	
Analytical	pH Probe	All	Yokogawa		FU20-VP-T1-NPT w/WU10-V-S-xx	Classic Controls	** /U option for pipe and wall mount, /PM for panel mount, xx is the length of cable in meters
Analytical	Fluoride	Water	Thermo Scientific	Orion 2100	Orion 2109XP	Classic Controls	
Analytical	ORP Transmitter	can do both ORP and PH	Yokogawa		PH450G-A-A**	Classic Controls	** installation specific option
Analytical	ORP Probe	can do both ORP and PH	Yokogawa		FU20-VP-T1-NPT w/WU10-V-S-xx	Classic Controls	
Analytical	Gas Detection	NH3, H2S, LEL Gas Detection	Sensidyne	Sensalert ASI (Hart)	S22-3HTH-AA	Gilson Engineering	
Analytical	Ammonia Gas 0-100 ppm	Gas Dectection	Sensidyne		823-0201-21	Gilson Engineering	
Analytical	LEL (Methane) Gas 0-100%	Gas Dectection	Sensidyne		823-0211-51	Gilson Engineering	
Analytical	H2S Gas 0-50ppm	Gas Dectection	Sensidyne		823-0206-22	Gilson Engineering	
Flow	Coriolis	Chemicals systems	Micro Motion	F-Series Coriolis	Depends on size & chemical	Emerson Process Control	Never use Yokogawa RotaMASS, Drew Sherry 813-478-9164 sale rep.
Flow	Flowmeter Strap	Water, wastewater	Endress Hauser	Prosonic 93W		Trinova	3
Flow	Flowmeter Strap	Water, wastewater	Flexim	F721 transmitter	FLUXUS F721 with Hart	Classic Controls	Also need to size Transducer
Flow	Magnetic	Water, wastewater	Yokogawa	Admag AXG(size)		Classic Controls	1, 3 (1" to 20")
Flow	Magnetic	Water, wastewater	Yokogawa	Admag AXW(size)		Classic Controls	Over 20"
Flow	Switch						No preference other than don't use FCI.
Level	Clarifier Level	Sludge blanket level	Hach	SC1000	Sonatax probe	Hach	
Level	Diesel level	Generator fuel tanks	Ohmart Vega	VegaFlex 81	FX81.FELTHGHXANKX-xx	Classic Controls	xx is insertion length in inches Make sure to get Blue-Tooth Displays
Level	Non contacting - radar w/Blue Tooth Diplays new	Dry Chemical bulk tanks	Ohmart Vega	VegaPlus PS69	PS69.IXTTDAHXKNKXX	Classic Controls	Make sure to get Blue-Tooth Displays
Level	Non contacting - radar w/Blue Tooth Diplays new	Chemical day tanks	Ohmart Vega	VegaPlus 61	PS61.UDANPHANKX	Classic Controls	w/local display built in for chemical day tanks, Make sure to get Blue-Tooth Displays
Level	Non contacting - radar w/o display	Chemical bulk tanks	Ohmart Vega	VegaPlus 61	PS61.UXANPHKNXX	Classic Controls	w/remote display located in fill panel for bulk chemical tanks, Make sure to get Blue-Tooth Displays
Level	Non contacting - radar remote display	Chemical bulk tanks	Ohmart Vega	VegaDis 61	DIS81.FEIANNKAX	Classic Controls	This display works in conjunction with a transmitter
Level	Non contacting - radar w/o display	Lift stations w/mixer	Ohmart Vega	VegaPlus WL61	PSWL61.XXBXHDKAX	Classic Controls	
Level	Hydrostatic pressure	Lift stations, other tanks	Contegra	SLX 130		Mader	
Level	Hydrostatic pressure	Wells: ASR, DIW, PW	IN-SITU	Level TROLL 500	Level TROLL 500	IN-SITU	
Level	Switch						No preference other than don't use point conductivity.
Local Display	Panel Mount Display	for display	Yokogawa	UTAdvanced	UM33A-000-10/LP	Classic Controls	
Local Control	PID Controller	for PID control	Yokogawa	UTAdvanced	UT55A-000-10-00/LP	Classic Controls	
Pressure	Differential	Venturi, hydrostatic level	Yokogawa	EJA Series	EJA110E-JHS4G-922EB/FF1/D1	Classic Controls	3, 4
Pressure	Gauge	Line (pipe) pressure	Yokogawa	EJA Series	EJA530E-JBS4N-022EL/FF1/D1	Classic Controls	3, 4
Pressure	Swtich						No preference but all the MOCs must be 316SS or better.

General Notes:

A. Always select Hart capabilities if available.

Double Checked w/Manufacturer

Notes:

1. Typically provided with integral display
2. Typically provide with remote display, meter body located in chemical room in process line, remote display located outside of chemical room.
3. Select the option that provides SS or aluminum housing. Do not want the epoxy coated housing.
4. If necessary provide w/chemical seal, MOC selection dependent on process.

Lee County SEAWRF Data Request Update

Date:	May 31, 2022
Project name:	Lee County SEAWRF
Project no:	D31456AW
Attention:	LCU
Client:	LCU
Prepared by:	Claes Westring
Reviewed by:	Randy Boe, Kerstin Kenty
Revision no:	1

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1.1 Background

In 2019, the preliminary design was developed for the Lee County Utilities (LCU) Southeast Water Reclamation Facility (SEWRF). As part of the preliminary design, five years of historical influent data (2014 to 2018) from the Three Oaks Wastewater Treatment Plant (WWTP) were used to characterize the design flows and loads for the SEWRF. Influent historical data from the Three Oaks WWTP are considered representative of the flows and loads for the SEWRF because:

- The majority of the flow to the SEWRF will be diverted from the Three Oaks service area.
- New development in the SEWRF service area will be primarily residential, similar to the Three Oaks service area.

In addition, LCU preferred manufacturers, equipment types and materials were established in 2019. Since a few years has passed since this information was received, the purpose of the TM is to request recent flows and loads data and confirm if any of the LCU preferences have changed.

1.2 Data Request

Jacobs previously analyzed flows and loads data from Three Oaks WWTP through 2018. For this data request, Jacobs requests updated influent flows and loads data from 2019-present. In addition, Three Oaks WWTP receives nanofiltration (NF) concentrate from the Pinewoods Water Treatment Plant (WTP). Beginning around 2030, the NF concentrate is projected to be redirected to the new SEWRF. Therefore, Jacobs requests the following Three Oaks daily influent parameters (or as much data that is available for each parameter) from 2019-present:

Three Oaks Influent Parameters:

- Flow
- CBOD
- TSS
- TKN
- NH3
- Total Phosphorus

Pinewoods WTP NF Concentrate:

- Daily Flow
- TKN
- TDS
- pH
- Chloride
- Sulfate
- Sodium
- Calcium
- Potassium
- Magnesium
- Iron,
- Carbonate
- Bi-Carbonate

1.3 Preferred Manufacturers Confirmation

When this project began in 2019, LCU provided a list of preferences (Attachment A). If this list has been updated, Jacobs requests an updated list. As a summary, the following preferred manufacturers were provided in 2019:

Headworks:

- Perforated (6 mm) bar screens; Kuster screens shall not be used
- Hallstan Channel Covers are preferred
- Coating system shall be EuroFlex, as installed by Preferred painting.
- EUTek grit removal systems have worked well for LCU, if the sizing is appropriate

Oxidation Ditches:

- Preferred Manufacturers Ovivo and Westech

Secondary Clarifiers:

- Ovivo and/or Evoqua center feed clarifiers have worked well in the past
- Westech is another preferred manufacturer
- NEFCO weir covers are preferred
- Penn Valley Double disk scum pumps preferred

RAS Pumps:

- All RAS and WAS pumps shall be Vaughan Chopper type

Chemical Systems:

- Lee County has a sole source with ProMinent chemical dosing pumps

Electrical:

- Cummins has a sole source with Lee County for generators and switch gear/transfer switches

BIosolids Disposal:

- Seepex progressing cavity pumps are preferred for belt filter press feed pumps
- Ashbrook belt filter presses are preferred

Attachment A
2019 LCU Water Reclamation Facility Preferences

LEE COUNTY UTILITIES
WATER RECLAMATION FACILITY PREFERENCES
2/15/2019

GENERAL:
- No steel tanks shall be allowed; concrete tanks are preferred.
- WRF gravity collection, plant lift station shall be pumped back into the WRF upstream of the bar screen.
- If the site allows, layout of process units to follow the process flow.
- ~~Lighting~~ protection on all structures.

EQUALIZATION TANKS:
- EQ Tank are necessary as they reduce:
  - Chemical costs
  - Operating costs
  - Increases the capacity of the WRF

  For example without an EQ Tank, both Chlorine Contact Chambers (CCC) may need to be run to meet detentions times at peak flow. With an EQ Tank the flow into the plant can remain constant while the excess peak flows are captured in the tank.
- There shall be one flow meter metering the influent flow to the EQ Tank. See “Critical flow meter note” located in the Instrumentation Section.
- Consultant to provide recommendation on injection of Bioxide or other odor control chemical at the station feeding the EQ Tank or at the EQ Tank to assist with odors and grease.
- Diffused air shall not be used in the EQ Tank, submersible mixers are preferred.
- Consultant to provide recommendation for influent flow directly into EQ Tank, or first Headworks then EQ Tank, both options shall allow for bypass.
- EQ Tank shall not have a cover.
- EQ Tank shall have an internal coating system.
- EQ Tank shall have manways at ground level to allow access for routine maintenance and cleaning.

HEADWORKS:

Debris and girt removal at the Headworks is critical in order to extend the life of equipment, pumps, and valves. It also preserves the treatment volume.
- There shall be one flow meter on the influent flow to the Headworks (other than that provided for the EQ Tank). Metering influent flow into the Headworks is critical for chemical flow pacing and operations. All flow should go through one flow meter. See “Critical flow meter note” located in the Instrumentation Section. Note, Headworks influent flow meter shall not measure RAS flow. RAS flow will come from RAS flow meter at the RAS pump station.
- There shall be a permanent piped bypass, at minimum, around the one half of the Headworks, allowing at minimum the other half of the headworks to stay in service. This configuration will allow for equipment and structure maintenance.
- Headworks concrete coating system shall be monolithic, spark tested with a written report, and carry a 10 year warranty. Warranty will allow for yearly inspections and repair to coating system. Weir plate hardware shall be installed over the coating system and all hardware

penetrating the system shall be covered up by the coating system after installation prior to startup.

- All weir gates, hardware, etc shall be 316SS, aluminum and FRP are not allowed.
- Screening:
  - At minimum there shall be two automated (self-cleaning) bar screens and one manual. Passive overflow shall exist from the automated bar screens to the manual bar screen.
  - We would like the consultant to investigate the following two options for the automated bar screens:
    - Self-cleaning static bar screens
    - Perforated screen (6mm) bar screens; Kuester shall not be used. Perforated creens with excessive moving parts and gearboxes shall be minimized.
  - Headworks shall be one structure. It shall have three influent channels for bar screens. Each screen shall be sized for the maximum build out and RAS flow.
- Headworks shall have channel covers with access doors to mitigate odor. Hallstan Covers are preferred, they are being used at the Fiesta WRF.
- Coating system shall be the EuroFlex, as installed by Prefferred Painting. It shall come with a 10 year warranty.
- Compactor or wash press shall be provided to remove water from debris prior to dumping.
- Grit Removal:
  - EUTek grit removal systems have worked well for LCU, **<u>if</u>** the sizing is appropriate, possible build banks of these to accommodate actual flows and not future flows. Head cell shall have an internal built in spray down header. The same coating system as the headworks shall be used.
  - Grit pumps shall always have flooded suction.
  - Pinch valves shall not be used.
- RAS should be introduced into Headworks influent flow, see RAS Section.

TREATMENT:

- Oxidation Ditches shall be used, they are LCU's standard treatment unit.
- Multiple Oxidation Ditches can be constructed with common walls.
- Oxidation Ditch design shall take into consideration serviceability, and will allow for continued operation of other ditches while one ditch is off line for cleaning, maintenance, or repair.
- Rotors (brushes) are preferred as opposed to vertical aerators. Rotors are lower horsepower and create less spray and are more hygienic.
- All rotors shall have associated variable frequency drives (VFDs) and bypass soft starts w/built in internal bypasses. Rotor speed shall be adjustable. Bypass soft starts will allow for operation of the rotor while the VFD is down for maintenance or repair. VFDs reduce energy consumption/costs while improving dissolved oxygen (DO) control.
- LCU would like the rotor width to be no greater than 15ft, however depending on cost escalations, it may entertain wider rotors.
- Oxidation Ditch water level control shall be adjusted by discharge weir height.
- Oxidation Ditches shall have sloped floors with many appropriately sized drains to allow for gravity flow. Multiple drains necessary to allow for flow with maximum. Oversize or add more drains such that flow will still meet design if 25% of the drains are blocked for whatever reason.
- Rotors shall have covers, the covers and mounts at Gateway WRF are a good example of what is desired.

- There shall be pedestrian access to both sides of the rotor.
- Drains located in the rotor mount pits shall flow directly to the plant lift station, and NOT back into the ditch.
- Passive overflow shall exit between all tanks ditches, to avoid overflows onto the ground.
- Each Oxidation Ditch shall be provided with it's own flow meter.
- There shall be continuous online monitoring of DO and ammonia. Redundancy shall be built into the instrumentation to allow for continuous online measurement while redundant instrumentation is cleaned, maintained, or repaired.
- Control of aeration shall be via the online DO and rotor VFDs.

CLARIFICATION:

- Only center feed clarifiers shall be allowed. Ovivo and/or Evoqua center feed clarifiers have worked well in the past. They have proven to lower power consumption.
- Clarifiers shall NOT have draft tubes.
- Larger clarifiers are preferred over many smaller clarifiers. LCU is suggesting the following arrangement for clarification:
    - At 2MGD design capacity, 2 clarifiers
    - At 4MGD design capacity, 3 clarifiers
    - At 6MGD design capacity, 4clarifiers
- Continous walkway shall be provided around clarifier.
- Weir covers shall be included to prevent algae growth, Nefco are preferred. Weir covers shall be easily accessible and serviceable from a continuous walk way around the clarifier.
- Stamdford baffles shall be used.
- RAS flow shall be controlled based off of the flow entering the Oxidation Ditches.
- Hydraulic modeling shall be done that will allow any Oxidation Ditch, or any two Oxidation Ditches to feed any one Clarifier. Splitter boxes shall be designed in to accommodate this request.
- At a minimum for any two Clarifiers there shall be three RAS pumps and one RAS flow meter.
- Waste pumps shall not be oversized.
- **Noteworthy**, each Clarifier shall have bypass piping to a Reject Tank.

RETURN ACTIVATED SLUDGE (RAS):

- RAS introduced into Headworks influent flow shall be modeled to optimize odor reduction and grease control.
- The flow of RAS shall come from RAS flow meter located at each RAS pump station. See "Critical flow meter note" located in the Instrumentation Section.
- There shall be RAS flow with metering directly to Oxidation Ditches.
- There shall be good control of the RAS flow through appropriately sized RAS pumps. LCU has had issues in the past with oversized RAS pumps. The pumps where sized for build out, and did not have the appropriate turndown ratio to work at WRF startup flows.
- All RAS pumps shall have their own VFD.
- All RAS pumps shall be Vaughan Chopper type.
- RAS should be injected prior to the bar screen. It may need to be screened.
- RAS pump station shall have sample port and collection sink.

- RAS pump station shall have collection drain system that gravity flows to the plant lift station. The following shall be directed to the collection drain system, vent piping from air relief valves, wash down, sample sink drain, and mechanical seal water.

FILTRATION:

- Deep bed gravity filters shall be used.
- At minimum there shall be 3 filters. Each filter shall be an exact reproduction of the others.
- Filter gallery shall have an open sided, roof structure. The southern face of the filter gallery roof shall have a roll up/down screen to block sun. Screen shall be automated and run on a timer and wind sensor.
- Wet well shall have vertical turbine pumps.
- Screw lifts are not allowed.
- **Noteworthy**, filter backwash water source shall be other than Clearwell to prevent dips in effluent flow for better chlorine control. Open to suggestions from the Consultant on what water we could use.
- Mudwell shall have a sloped floor with a built in spray down header system.
- LCU would like to investigate a means of back washing filters without robbing water from the chlorine contact chamber and there by spiking chlorine residual. Possibly a filter feed tank could hold water for back washing.

DISINFECTION:

- Liquid disinfectant shall be sodium hypochlorite solution; gas disinfectant is not allowed.
- At minimum there must be two disinfectant storage tanks.
- Tank construction shall be FRP.
- Storage tank capacity shall be such to allow for off load of one full tanker truck, no half loads; while maintaining the optimal strength of the sodium hypochlorite solution.
- Consultant to investigate the use of the CCC transfer pump station to move reject water to a Reject Tank, while bypassing the CCC.

CHEMICAL SYSTEMS:

- A paved road shall be required for delivery of any chemicals.
- Bulk chemical tanks and day tanks shall be under a covered structure.
- All chemical tanks shall be located inside of concrete containment. Multiple tanks holding the same chemical can share containment.
- Chemical concrete containment coating system shall be monolithic and chemically compatible to the chemical tank located within it.
- All mounting hardware, affixed to the chemical containment interior walls or floor, shall be installed prior to application of coating. Mounting hardware will be coated at the same time as containment. Consultant shall provide a detail capturing this information.
- All chemical tanks shall come with a site glass. Site glasses shall be provided with valves where they connected to the tank.
- Lee County has a sole source with ProMinent chemical dosing pumps.
- Eye wash stations with flow alarms shall be provided near chemical dosing pump skids and tanks.

EFFLUENT DISPOSAL:

- A reuse storage tank(s) shall be provided.
- Reuse tanks shall be covered.
- Some means for storing reject water shall be provided.
- An automated reject system tied into pH, CL Residual, turbidity shall be provided.
- One Deep Injection Well (DIW) shall be provided.
- Locating a future second DIW shall be part of the design. Second DIW shall not be constructed.

DIGESTERS:

- Digesters shall be aerobic.
- Digesters shall have centrifugal pumps for pumping down.
- Blowers shall be centrifugal with diffused air.
- Digesters shall be approximately sized.

BIOSOLIDS DISPOSAL:

- Belt presses shall be provided as opposed to centrifuge. Belt presses have lower power consumption, and allow for in house repairs, while providing similar percent solids numbers as other WRFs.
- Trailer loading system shall allow for the loading of two trailers at a time.
- Screw conveyors preferred over conveyor belts.
- Screw conveyor trailer loading manifold to have multiple shoots to allow for a more distributed trailer loading.
- Weight scales shall be provided to verify the weight of sludge hauling trucks.
- Electrical, reuse water, drain connections shall be provided to allow for hook up of County's portable centrifuge system
- A cleaning station shall be provided for the trucks and trailers.

ODOR CONTROL:

- Biological treatment helps cut down on costly carbon replacement.
- Possible first stage bio-filter, second stage iron oxide media or carbon polishing.
- Would prefer the use of reuse water to feed bio-filter.
- If the first stage is biotrickling there shall be a high H_2S concentration alarms to notify Operations. High H_2S may kill the bacteria in the bio-filter.

ELECTRIC:

- Control room floor shall be over the 100 year flood plane.
- Electrical room floor shall be over the 1000 year flood plane.
- Switchgear shall be ARC rated.
- Switchgear shall be main-tie-main.
- MCC's shall be main-tie-main.
- Cummins has a sole source with Lee County for generators and switchgear/transfer switches.
- Redundant generators shall be provided.

INSTRUMENTATION:

- All flow meters shall be easily accessible. No flow meter shall be installed in an in ground vault.

- Sludge blanket monitors shall not be provided.
- Instruments shall come with sun covers to increase longevity.
- Instrument housings shall be more corrosion resistant. Stainless steel enclosure preferred over epoxy coated aluminum.
- All level readings shall be redundant.
- Low level floats necessary to lock pumps out on low level (with override); even if there are redundant analog level transmitters. Shutdown logic shall be based off of 2 out of 3.
- Critical flow meter note: flow meter shall have a piped bypass to allow for removal and maintenance of the flow meter. The bypass line around the flow meter shall have the proper upstream and downstream straight runs to accommodate a temporary ultrasonic strap-on flow meter in case the permanent meter needs to be serviced, maintained, or replaced.
- There shall be continuous online monitoring instrumentation for Oxidation Ditch DO and ammonia.
- Use technical specification from GM WTP project as starting place for standard instrumentation makes and models.

CONTROL SYSTEM:

- Consultant to investigate the possibility of designing technology that would allow the Lead Operator to have access to WRF SCADA system at home, for monitoring purposes only, no control. The use of the County's Citrix system could be a possible solution.
- Consultant to investigate the possibility of designing technology that would allow the Operations staff to access Citect SCADA alarms on a mobile device while making rounds at the WRF.
- SCADA shall alarm on loss of a remote I/O (RIO) or programmable logic controller (PLC) and the associated unit operation.
- I&C design to be such that control panels (CPs) are dedicated to an individual process and not multiple processes, except for the main plant PLC.
- The main plant PLC shall have redundant processors.
- Trend screen shall be capable of storing Operator notes documenting rational for process upset that is permanently retrievable in the future.
- All control system networking shall be done over fiber optics.
- SCADA (Citect) Primary and Secondary servers shall be located in different locations to avoid a single point of failure.
- Model the chlorine flow pacing loop description from that at Fiesta Village WRF.
- Chemical pacing with trim shall be provided.

EMERGENCY GENERATOR:

- Paralleling generators for redundancy. Note generator power transfer will be open transition, generators will parallel to each other.
- Each generator shall be sized to carry the entire WRF load. Generator will not be sized to carry any load from Solid Waste portion of the site.

ADMINISTRATION BUILDING:

- Shall be provided as a two story building.
- Control room shall be located on the second story of the building.
- Aluminum walkways shall be provided from the second story to the WRF unit operations.
- Lab facilities shall be regular, the size of Three Oaks is appropriate.

- Size of admin building to match the square footage of GM WTP.

MAINTENANCE BUILDING:

- Shall be provided, maybe part of the Administration Building or separate.

Appendix B. Development of Design Flows and Loads for the New WRF

Technical Memorandum: Development of Design Flows and Loads for the New WRF

Date:	June 24, 2022
Project name:	Lee County Water Reclamation Facility (WRF)
Project no:	D31456AW
Attention:	Lee County Utilities
Client:	Lee County Utilities
Prepared by:	Claes Westring
Reviewed by:	Randy Boe
Revision no:	2
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1. Development of Wastewater Design Flows and Loads

In 2019, the initial preliminary design was developed for the Lee County Utilities (LCU) Southeast Advanced Water Reclamation Facility (SEAWRF). As part of the preliminary design, five years of historical influent data (2014 to 2018) from the Three Oaks Wastewater Treatment Plant (WWTP) were used to characterize the design flows and loads for the SEAWRF.

To update influent design flows and loads to recent historical data for the development of an updated preliminary design of the SEAWRF, nine years of historical influent data (2014-2022) from the Three Oaks Water Reclamation Facility (WRF) was utilized to characterize the design flows and loads for the new SEAWRF. Influent historical data from the Three Oaks WRF was thought to be representative of the flows and loads to the new WRF because of the following:

- The majority of the flow diversion to the new WRF will come from the Three Oaks service area
- New development in the new WRF service area will be primarily residential as is the case in the Three Oaks Service area.

1.1 Design Flow Definitions

The following definitions are used in this report to establish and express design flows:

Annual Average Daily Flow (AADF) – The total volume of wastewater flowing into a wastewater facility during any consecutive 365 days divided by 365 and expressed in units of million gallons per day (mgd).

Monthly Average Daily Flow (MADF) – The total volume of wastewater flowing into a wastewater treatment facility during 30 calendar days divided by 30 days and expressed in units of mgd.

3 Month Average Daily Flow (3MADF) – The total volume of wastewater flowing into a wastewater treatment facility during 90 calendar days divided by 90 days and expressed in units of mgd.

Weekly Average Daily Flow (WADF) – The total volume of wastewater flowing into a wastewater treatment facility during 7 calendar days divided by 7 days and expressed in units of mgd.

Maximum-Month Average Daily Flow (MMADF) – The highest MADF.

Maximum 3-Month Average Daily Flow (M3MADF) – The highest 3MADF.

Maximum-Week Average Daily Flow (MWADF) – The highest WADF.

Maximum Day Flow (MDF) – The highest flow volume in millions of gallons during any consecutive 24-hour period.

Peak Hour Flow (PHF) – The highest flow rate in millions of gallons during any consecutive 1-hour period.

1.2 Wastewater Design Flows

Understanding flow variation is important for evaluating existing facilities and planning for new facilities. Flow variation is typically expressed in terms of the ratio to the AADF, commonly referred to as peaking factors. Peaking factors of interest include the maximum month, maximum 3-month, maximum week, maximum day, and peak hour.
Three Oaks WRF historical flows and flow peaking factors are presented in Table 1 for the period from 2014 to 2022. Peak hour flow data was not available. A peaking factor of 3.0 relative to AADF is recommended for PHF based on review and consideration of several items including:

- Ten State Standards guidelines (Wastewater Committee of the Great Lakes 2014), which suggest a PHF peaking factor of approximately 2.3.

- *Corkscrew Overlay Area Wastewater Master Planning Report* (JEI 2016). This report was focused on modeling lift stations and forcemains and does not present an overall PHF peaking factor for treatment plant. However, PHF peaking factors for master pump stations and forcemains discussed in the report ranged from approximately 2.6 to 3.1.

- Experience with other facilities in the area, including the Bonita Springs East WRF, which has a design PHF peaking factor of 3.0. The Three Oaks WWTP also has a design PHF peaking factor of 3.0 according to the Capacity Analysis Report (Lee County 2011).

In addition to the historical yearly data summarized in Table 1, the percentiles of the daily flow peaking factors relative to each year's average were computed to normalize the data set. The updated recommended peaking factors for the SEAWRF presented in Table 1 are based on engineering judgement considering the averages, range, and computed percentile peaking factors for the entire data set. The recommended flow peaking factors have not changed due to the latest 2019 to 2022 flow data.

Table 1. Historical and Recommended Influent Flows and Peaking Factors

Year	Min (mgd)	AADF (mgd)	M3-MADF (mgd)	MMADF (mgd)	MWADF (mgd)	MDF (mgd)	M3M :AA	MM:AA	MW:AA	MD:AA	PH:AA
2014	1.96	2.70	3.10	3.16	3.30	3.60	1.15	1.17	1.22	1.33	
2015	2.37	2.90	3.20	3.32	3.40	4.20	1.10	1.14	1.17	1.45	
2016	2.80	3.20	3.50	3.53	3.70	4.10	1.09	1.10	1.16	1.28	
2017	2.63	3.10	3.30	3.57	4.80	5.90	1.06	1.15	1.55	1.90	
2018	2.91	3.30	3.60	3.84	4.00	4.70	1.09	1.16	1.21	1.42	
2019	3.21	3.79	4.04	4.29	4.48	5.09	1.07	1.13	1.18	1.34	
2020	3.10	3.64	4.18	4.29	4.52	4.72	1.15	1.18	1.24	1.30	
2021	3.26	3.84	4.07	4.16	4.32	5.00	1.06	1.08	1.12	1.30	
2022[b]	3.62	4.36	4.50	4.65	4.93	7.55	1.03	1.07	1.13	1.73	
Average							1.09	1.13	1.22	1.45	
Maximum							1.15	1.18	1.55	1.90	
92nd – 99.7th Percentile							1.06	1.14	1.21	1.34	--
Recommended SEAWRF Peaking Factors							1.1	1.2	1.3	1.9	3.0[a]
Previously Recommended in 2020							1.1	1.2	1.3	1.9	3.0

[a] Peak hour flow data was not available. A peaking factor of 3.0 relative to AADF was assumed based on Ten State Standards guidelines and experience with other facilities in the area.

[b]Only 5 months of flow data was available for 2022
M3M:AA = ratio of maximum 3-month average daily flow to annual average daily flow
MD:AA = ratio of maximum day flow to annual average daily flow
MDF = maximum daily flow
MM:AA = ratio of maximum-month average daily flow to annual average daily flow
MW:AA = ratio of maximum-week average daily flow to annual average daily flow

Table 2 presents the recommended design flows for each of the two phases of the SEAWRF, based on the recommended flow peaking factors. The two SEAWRF phases are based on AADFs of 6 and 10 mgd.

Table 2. Recommended Design Flows

Parameter	Phase 1	Phase 2
AADF (mgd)	6	10
M3MADF (mgd)	6.6	11.0
MMADF (mgd)	7.2	12.0
MWADF	7.8	13.0
MDF	11.4	19.0
PHF	18.0	30.0
Startup Flow (AADF)	0.7	n/a
Minimum Hour Startup Flow	0.3	n/a

M3MADF = Maximum 3-month average daily flow
MMADF = Maximum-month average daily flow
MWADF = Maximum-week average daily flow
n/a = not applicable

1.3 Wastewater Design Loads

1.3.1 Average Constituent Concentrations

Three Oaks WWTP data of daily concentrations of influent 5-day Carbonaceous Biochemical Oxygen Demand ($CBOD_5$) and Total Suspended Solids (TSS) were analyzed for the 9-year period from 2014 through 2022 to determine annual average influent design concentrations. Outliers, defined as values greater than three standard deviations from the mean, were not used when determining the historical annual average concentrations. However, very few outliers were identified. Monthly influent samples of Total Kjeldahl Nitrogen (TKN), Ammonia, and Phosphorus were also included in the 9 years of Three Oaks WRF data and annual influent design concentrations were also calculated for these constituents. The historical annual average influent concentrations of various constituents are presented in Table 3. Overall, there does not appear to be a general trend up or down in the concentrations. Somewhat conservative average concentrations are recommended as shown.

Currently, Three Oaks WWTP receives nanofiltration (NF) concentrate from the Pinewoods Water Treatment Plant (WTP). On average, it accounted for 8 percent of the flow to Three Oaks WWTP over the period from 2014 to 2022. This flow has the effect of diluting the influent concentrations of CBOD5 and TSS by the same proportion. Beginning around 2030, the NF concentrate is projected to be redirected to the new SEWRF. The recommended concentrations for the SEAWRF presented in Table 3 are based on engineering judgement considering the averages and range for the entire data set as well as being corrected for the dilution effect of the NF concentrate to the Three Oaks WWTP influent concentrations. Recommended influent concentrations were not changed from concentration previously recommended in 2020.

Table 3. Historical and Recommended Influent Annual Average Concentrations

Year	$CBOD_5$ (mg/L)	True BOD_5[a] (mg/L)	TSS (mg/L)	TKN (mg/L)	NH3-N (mg/L)	TP (mg/L)
2014	179	-	231	41.1	29.8	5.17
2015	178	-	232	40.1	31.1	5.16
2016	167	-	212	41.0	36.1	5.02
2017	164	-	204	46.9	43.2	5.15
2018	172	-	187	41.6	33.6	4.61
2019	157	-	189	42.1	31.2	5.07
2020	139	-	173	50.1	34.7	5.53
2021	141	-	175	48.8[b]	38.7[b]	6.02[b]
2022	183	-	188	58.5[b]	41.2[b]	6.10[b]
Average	164	-	199	45.6	35.5	5.31
Maximum	183	-	232	58.5	43.2	6.10
Minimum	139	-	173	40.1	29.8	4.61
Recommended SEAWRF Concentrations	200	240	240	50	41	5.8
Previously Recommended in 2020	200	240	260	50	41	5.8

[a]True BOD_5 is corrected for the effect of nitrification inhibitor added for the $CBOD_5$ test and represents the $CBOD_5$ divided by a factor of 0.84
[b]2021 only had 5 total monthly samples and 2022 had 2 total monthly samples which may cause the average annual concentrations to appear to be higher.
$CBOD_5$ = 5-day Carbonaceous Biochemical Oxygen Demand
TSS = Total Suspended Solids
TKN= Total Kjeldahl Nitrogen
NH_3-N= Ammonia as nitrogen
TP= Total Phosphorus

1.3.2 Influent Loads and Load Peaking Factors

Historical influent pollutant loads in pounds per day (ppd) from the 9-year period were analyzed for the Three Oaks WWTP. Table 4 summarizes the historical $CBOD_5$ and TSS load peaking factors relative to the annual average load and the recommended peaking factors for the design of the SEAWRF. In addition to the Three Oaks WWTP historical yearly peaking factors summarized in Table 4, the percentiles of the daily load peaking factors relative to each year's average were computed to normalize the data set. Only the recommended max day peaking factor for $CBOD_5$ and the max month peaking factor for TSS increased from what was previously recommended in 2020.

Peaking factors for TKN, NH3-N, and TP were not determined because these constituents are not sampled daily. Table 5 presents the recommended design peaking factors for TKN, NH3-N, and TP, which are assumed to be consistent with the peaking factors for $CBOD_5$. Because these are load-based peaking factors and the concentrate does not contain $CBOD_5$ or TSS, it is not necessary to consider effects of concentration dilution by the NF concentrate flow.

Table 4. Historical Three Oaks Loading Peaking Factors and Recommended Peaking Factors

Year	CBOD$_5$				TSS			
	M3M:AA	MM:AA	MW:AA	MD:AA	M3M:AA	MM:AA	MW:AA	MD:AA
2014	1.43	1.48	1.56	1.94	1.27	1.31	1.47	1.97
2015	1.23	1.30	1.41	1.54	1.14	1.19	1.32	1.51
2016	1.16	1.24	1.39	1.66	1.18	1.25	1.38	1.71
2017	1.34	1.41	1.57	1.78	1.24	1.37	1.45	1.62
2018	1.29	1.44	1.54	1.83	1.23	1.31	1.39	1.81
2019	1.18	1.28	1.37	1.58	1.25	1.35	1.43	1.64
2020	1.27	1.32	1.39	2.11	1.33	1.41	1.52	1.86
2021	1.21	1.41	1.78	2.24	1.17	1.36	1.54	1.93
2022	1.08	1.12	1.26	1.58	1.06	1.09	1.26	1.76
Average	1.24	1.33	1.47	1.81	1.21	1.29	1.42	1.76
Maximum	1.43	1.48	1.78	2.24	1.33	1.41	1.54	1.97
Percentile	1.16	1.35	1.58	1.88	1.15	1.32	1.51	1.81
Recommended SEAWRF Peaking Factors	1.3	1.4	1.6	2.0	1.3	1.4	1.5	1.9
Previously Recommended in 2020	1.3	1.4	1.6	1.9	1.3	1.3	1.5	1.9

AA = Average Annual
CBOD$_5$ = 5-day Carbonaceous Biochemical Oxygen Demand
M3M = Maximum 3-month
MM = Maximum Month
MW = Maximum Week
MD = Maximum Day
TSS = Total Suspended Solids

Table 5. Assumed Design Load Peaking Factors

Constituent	M3M:AA	MM:AA	MW:AA	MD:AA
TKN	1.3	1.4	1.6	2.0
NH_3-N	1.3	1.4	1.6	2.0
TP	1.3	1.4	1.6	2.0

AA = Average Annual
M3M = Maximum 3-month
MM = Maximum Month
MW = Maximum Week
MD = Maximum Day
TKN= Total Kjeldahl Nitrogen
NH_3-N= Ammonia as nitrogen
TP= Total Phosphorus

1.4 Summary of Design Influent Flows and Loads

Using the recommended flow peaking factors shown in Table 1, the annual average influent constituent concentrations presented in Table 3, and the recommended and assumed load peaking factors presented in Table 4 and 5, the design loads and flow rates for the new SEAWRF were computed and are presented in

Table 6. Design loads and flow rates are based on two phases for the new SEAWRF with AADFs of 6 and 10 mgd. In addition, because Lee County has experienced difficulty maintaining sufficient dissolved oxygen (DO) in aeration basins in the past for diurnal daytime peak load during MM loading periods, it is recommended to apply an additional 15 percent safety factor for aeration on the selected maximum daily loads (MDL) presented in Table 6.

Table 6. Recommended Design Loadings

Parameter	Peaking Factor	AADF (mgd)	
		6	10
Flow			
M3MADF (mgd)	1.1	6.6	11.0
MMADF (mgd)	1.2	7.2	12.0
MWADF	1.3	7.8	13.0
MDF	1.9	11.4	19.0
PHF	3.0	18.0	30.0
True BOD_5[a]			
AADL (lbs/day)	-	12,010	20,016
AAD (mg/L)	-	240	240
M3MADL (lbs/day)	1.3	15,612	26,021
MMADL (lbs/day)	1.4	16,813	28,022
MWADL (lbs/day)	1.6	19,215	32,026
MDL (lbs/day)	2.0	24,020	40,032
TSS			
AADL (lbs/day)	-	12,010	20,016
AAD (mg/L)	-	240	240
M3MADL (lbs/day)	1.3	15,612	26,021
MMADL (lbs/day)	1.4	16,813	28,022
MWADL (lbs/day)	1.5	18,014	30,024
MDL (lbs/day)	1.9	22,818	38,030
TKN			
AADL (lbs/day)	-	2,502	4,170
AAD (mg/L)	-	50	50
M3MADL (lbs/day)	1.3	3,253	5,421
MMADL (lbs/day)	1.4	3,503	5,838

MWADL (lbs/day)	1.6	4,003	6,672
MDL (lbs/day)	2.0	5,004	8,340
NH3-N			
AADL (lbs/day)	-	2,052	3,419
AAD (mg/L)	-	41	41
M3MADL (lbs/day)	1.3	2,667	4,445
MMADL (lbs/day)	1.4	2,872	4,787
MWADL (lbs/day)	1.6	3,283	5,471
MDL (lbs/day)	2.0	4,104	6,838
TP			
AADL (lbs/day)	-	290	484
AAD (mg/L)	-	5.8	5.8
M3MADL (lbs/day)	1.3	377	629
MMADL (lbs/day)	1.4	406	677
MWADL (lbs/day)	1.6	464	774
MDL (lbs/day)	2.0	580	968

[a]True BOD_5 is corrected for the effect of nitrification inhibitor added for the $CBOD_5$ test and represents the $CBOD_5$ divided by a factor of 0.84
AA = Average Annual
BOD_5 = 5-day Biochemical Oxygen Demand
M3M = Maximum 3-month
MM = Maximum Month
MW = Maximum Week
MD = Maximum Day
TSS = Total Suspended Solids

Appendix C. Geotechnical Report

Subsurface Soil Exploration
Proposed Lee County Water Treatment and Transfer Facilities Project
Fort Myers, Lee County, Florida

Ardaman & Associates, Inc.

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Ardaman & Associates, Inc.

Geotechnical, Environmental and
Materials Consultants

21 February 2020
File No. 20-33-4505

Johnson Engineering, Inc.
2122 Johnson Street
Fort Myers, Florida 33901

Attention: Mr. Erik L. Howard, P.E., P.S.M.

Subject: Subsurface Soil Exploration
Proposed Lee County Water Treatment and
Transfer Facilities Project
Fort Myers, Lee County, Florida

Dear Mr. Howard:

As requested and authorized, we have completed a subsurface soil exploration for the subject project. The purposes of performing this exploration were to explore soil stratigraphy and groundwater levels at locations designated by Jacobs Engineering Group (Jacobs). This data report documents our findings.

SITE LOCATION AND SITE DESCRIPTION

The site for the proposed Lee County Water Treatment and Transfer Facilities is located northeast of the intersection of Alico Road and Green Meadows Road in Fort Myers, Lee County, Florida (Section 4, Township 46 South, Range 26 East). The general site location is shown superimposed on a Google Earth Pro aerial photograph presented on Figure 1.

The site currently consists of two parcels. The Lee County Property Appraiser identifies the parcels by STRAP Nos. 04-46-26-00-00001.0010 and 04-46-26-00-00001.1010. The parcels currently consist of gently sloping, structurally undeveloped grass covered pastureland. The ground surface elevation across the site is approximately +24 to +25 feet (NAVD88) according to Lee County's Web GIS.

PROPOSED CONSTRUCTION

It is our understanding that the proposed development includes new solid waste transfer facilities and wastewater treatment plant.

REVIEW OF SOIL SURVEY MAPS

Based on the 1984 Soil Survey for Lee County, Florida, as prepared by the U.S. Department of Agriculture Soil Conservation Service, the site is located in an area mapped as the “Pineda fine sand”, “Oldsmar sand”, and “Felda fine sand” soil series.

The “Pineda fine sand” soil series consists of nearly level sandy and loamy soils on sloughs. The internal drainage of the “Pineda fine sand” is poor and the soil permeability is rapid in the surface and subsurface layers and the upper, sandy part of the subsoil and slow or very slow in the lower, loamy part of the subsoil. According to the Soil Survey, the seasonal high water table for the “Pineda fine sand” soil series is typically within 10 inches of the natural ground surface.

The “Oldsmar sand” soil series consists of nearly level sandy and loamy soils on low, broad flatwoods areas. The internal drainage of the “Oldsmar sand” is poor and the soil permeability is rapid in the surface and subsurface layers, moderate in the upper part of the subsoil, and slow or very slow in the lower part of the subsoil. According to the Soil Survey, the seasonal high water table for the “Oldsmar sand” soil series is typically within 10 inches of the natural ground surface.

The “Felda fine sand” soil series consists of nearly level sandy soils on broad, nearly level sloughs. The internal drainage of the “Felda fine sand” is poor and the soil permeability is rapid in the surface and subsurface layers, moderate or moderately rapid in the subsoil, and rapid in the substratum. According to the Soil Survey, the seasonal high water table for the “Felda fine sand” soil series is typically within 10 inches of the natural ground surface.

FIELD EXPLORATION PROGRAM

SPT and Auger Borings

The field exploration program included performing ten Standard Penetration Test (SPT) borings and five auger borings at locations designated by Jacobs. The SPT borings were advanced to depths ranging from 30 to 95 feet below the ground surface using the methodology outlined in ASTM D-1586. A summary of this field procedure is included in Appendix I. Split-spoon soil samples recovered during performance of the borings were visually classified in the field and representative portions of the samples were transported to our laboratory in sealed sample jars.

The auger borings were drilled using a truck-mounted, 4-inch diameter, continuous flight auger to a depth of 10 feet below the ground surface. A summary of this field procedure is included in Appendix I. Representative soil samples were recovered from the auger borings and transported to our laboratory for further analysis.

The groundwater level at each of the boring locations was measured during and/or upon completion of drilling. The borings were grouted with cement-bentonite slurry upon completion.

Double-Ring Infiltrometer Tests

Five double-ring infiltration (DRI) tests were conducted at locations designated by Jacobs. The double-ring infiltration tests were conducted in general accordance with ASTM D-3385 procedure, "Infiltration Rate of Soils in Field Using Double-Ring Infiltrometer".

Prior to running the test, an excavation was made to a depth of 6 or 12 inches below the ground surface at the test location(s). The double-ring infiltration tests consisted of driving two open cylinders, one inside the other, into the ground at the test locations. Both rings were seated 6 inches below the bottom of the excavation. The rings were partially filled with water until a constant water level was achieved. A measurement of time versus water volumes added to the inner ring to maintain a constant water level was then recorded, the water level in the outer ring was maintained at a constant level during the duration of the test. The results of the Double-Ring Infiltrometer (DRI) tests are presented in the Double-Ring Infiltrometer Tests (DRI) results section of this report and presented in Appendix II.

Undisturbed Samples

Two relatively undisturbed tube samples of clayey soil encountered in Boring SPT-W3 were obtained to be held for potential laboratory classification and consolidation testing. The samples were retrieved using 3-inch diameter, thin-walled Shelby tubes. The samples were sealed in Shelby tubes and transferred to our laboratory for holding. The undisturbed samples from the Shelby tubes are presented adjacent to the sample recovery depth on the attached boring logs.

In addition, one wash boring was performed adjacent to Boring SPT-W5 and an attempt was made to obtain a relatively undisturbed tube sample of soil around 28 feet below the ground surface. The sample was attempted to be retrieved using a 3-inch diameter, thin-walled Shelby tube. Recovery of an undisturbed sample from the Shelby tube was unsuccessful. The tube appeared to be pushed through silty fine sand with some limestone fragments, damaging the Shelby tube. This disturbed sample was partially recovered and transported to our laboratory in a sealed sample jar.

Test Locations

The approximate locations of the borings, piezometer, and DRI tests are schematically illustrated on a site aerial photograph and site plan shown on Figures 2 and 3, respectively. These locations were determined in the field by Global Positioning System (GPS) utilizing hand-held GPS equipment and coordinates obtained from Google Earth Pro. Boring locations should be considered accurate only to the degree implied by the method of locating used. The DRI tests were conducted adjacent to Borings HA-W1 through HA-W5.

LABORATORY PROGRAM

Representative soil samples obtained during our field sampling operation were packaged and transferred to our laboratory for further visual examination and classification. The soil samples were visually classified in general accordance with the Unified Soil Classification System (ASTM D-2488). The resulting soil descriptions are shown on the attached soil boring logs.

In addition, we conducted 25 natural moisture content tests (ASTM D-2216), 5 grain size analyses (ASTM D-6913), 20 percent fines analyses (ASTM D-1140), 4 Atterberg limits tests (ASTM D-4318), and 3 corrosion series (FM 5-550, FM 5-551, FM 5-552, and FM 5-553) on selected soil samples obtained from the borings. Corrosion series testing was also performed from a water sample taken from Piezometer, PZ-W3. The results of these tests are presented adjacent to the sample depth on the attached boring logs and presented in Appendix III. The results of the corrosion series tests are presented in the Corrosion Series Test Results section of this report.

Bulk samples of soil were obtained adjacent to Borings SPT-W1 and SPT-W3. These samples were obtained from depths of ½ to 1½ feet below the ground surface. Modified Proctor (ASTM D-1557) and grain size analyses (ASTM D-6913) tests were performed on the recovered bulk samples. The results are presented in Appendix IV.

GENERAL SUBSURFACE CONDITIONS

General Soil Profile

The results of the field exploration and laboratory programs are graphically summarized on the attached soil boring logs. The stratification of the boring logs represents our interpretation of the field boring logs and the results of laboratory examinations of the recovered samples. The stratification lines represent the approximate boundary between soil types. The actual transitions may be more gradual than implied.

The results of the borings indicate the following general soil profile:

Depth Below Ground Surface (feet)		Description
From	To	
0	2 – 3	Loose to medium dense fine sand (SP) and slightly silty fine sand (SP-SM).
2 – 3	13 – 14	Medium dense fine sand (SP), slightly silty fine sand (SP-SM), silty fine sand (SM), and clayey fine sand (SC).
13 – 14	25	Medium dense to dense slightly silty fine sand (SP-SM), silty fine sand (SM), and clayey fine sand (SC) or soft weathered to hard limestone.
25	50	Very loose to medium dense silty fine sand with some gravel (SM) or soft weathered to hard limestone.
50	91	Very loose to medium dense clayey fine sand or firm to stiff sandy clay to clay with sand (CL/CH).
91	95	Hard limestone.

The above soil profile is outlined in general terms only. Please refer to the attached boring logs for soil profile details.

Groundwater Level

The groundwater level was measured in the boreholes during and/or upon completion of drilling. As shown on the attached boring logs, groundwater was encountered at depths that ranged from 1½ to 3½ feet below the existing ground surface on the dates indicated. Fluctuations in groundwater levels should be anticipated throughout the year primarily due to seasonal variations in rainfall and other factors that may vary from the time the borings were conducted.

In addition, one 2-inch diameter groundwater piezometer, designated PZ-W3, was installed on February 4, 2020, adjacent to Boring SPT-W3 using a drill rig and hollow stem auger. The piezometer is approximately 10 feet deep and included 5 feet of 0.010-inch factory slotted well screen and 8 feet of solid riser (3-foot stick up above ground). A groundwater level reading was taken from PZ-W3 on February 11, 2020. Groundwater was encountered at a depth of approximately 2 feet below the existing ground surface. We note that Piezometer PZ-W3 was installed permanently for long-term groundwater field monitoring as requested by Jacobs. Refer to the attached Monitor Well Installation Log for a schematic of Piezometer PZ-W3 presented in Appendix V.

Double-Ring Infiltrometer Test (DRI) Results

The results of the Double-Ring Infiltration (DRI) tests are presented in the following table.

Test No.	Measured Infiltration Rate (in/hr)
DRI-W1	0.5
DRI-W2	2.2
DRI-W3	2.0
DRI-W4	5.4
DRI-W5	3.2

Corrosion Series Test Results

The results of the Corrosion Series tests are presented in the following table.

Boring	Sample Nos.	Depth (ft)	pH	Resistivity (ohm-cm)	Chlorides (ppm)	Sulfates (ppm)
SPT-W1	4 – 5	4½ – 7½	8.5	7,030	15	33
SPT-W6	3	3 – 4½	8.4	8,010	15	18
SPT-T1	3 – 5	3 – 7½	8.2	5,182	30	27
PZ-W3 (water)	--	--	7.3	2,527	30	93

CLOSURE

The information submitted herein is based on the data obtained from the soil borings presented on the attached boring logs. This data report does not reflect any variations which may occur adjacent to or between the borings. The nature and extent of the variations between the borings may not become evident until during construction.

This data report has been prepared for the exclusive use of Johnson Engineering, Inc. in accordance with generally accepted geotechnical engineering practices for the purpose of the subject project. No other warranty, expressed or implied, is made.

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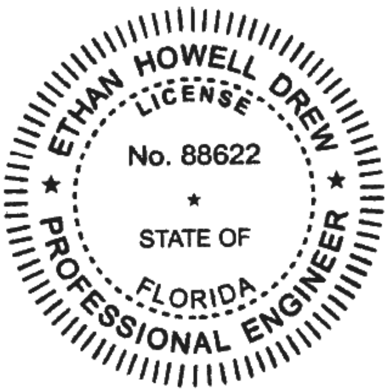
We are pleased to be of assistance to you on this phase of the project. When we may be of further service to you or should you have any questions, please contact us.

Very truly yours,
ARDAMAN & ASSOCIATES, INC.



This document has been digitally signed and sealed by

on the date adjacent to the seal.

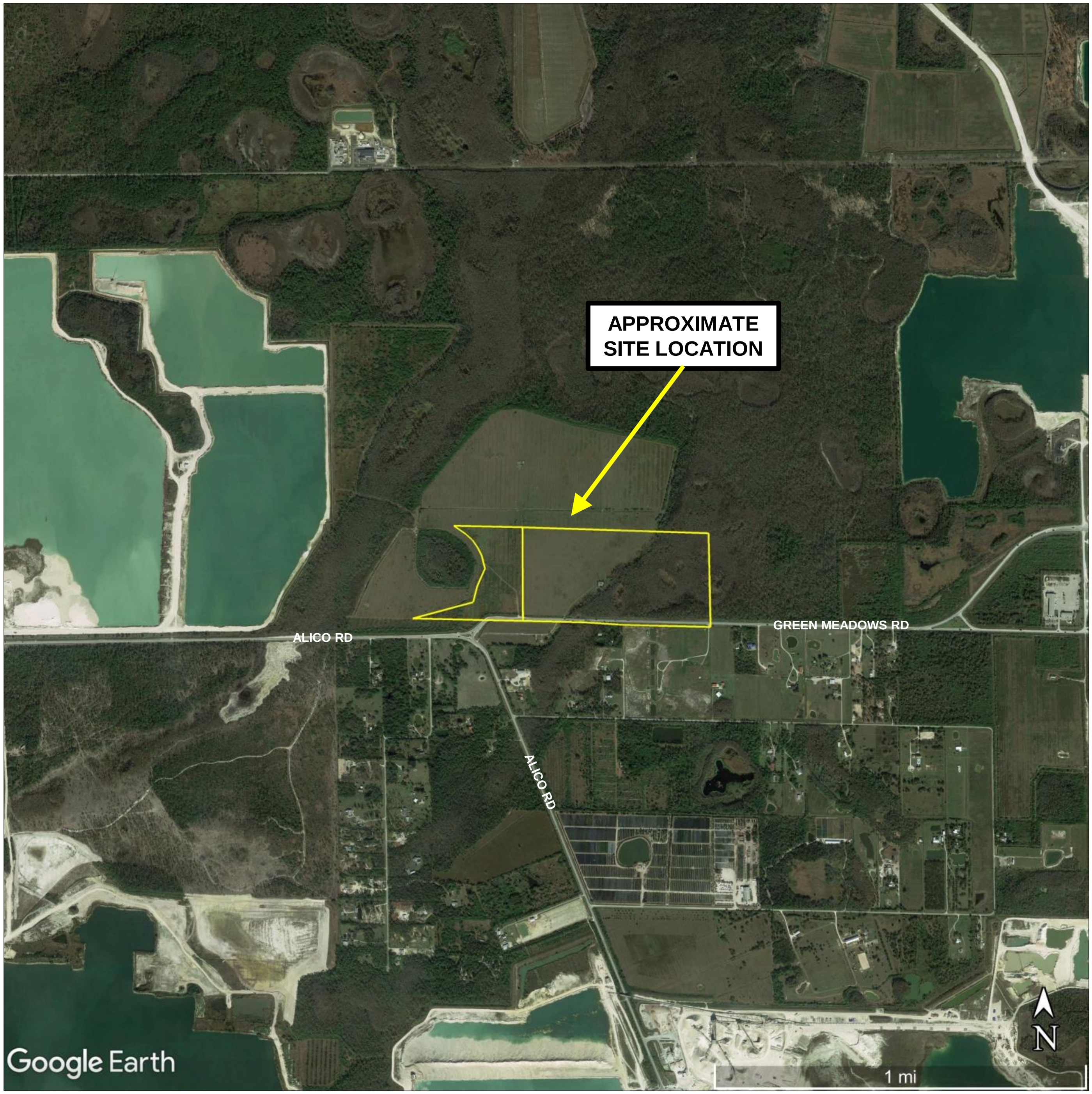
Printed copies of this document are not considered signed and sealed and the signature must be verified on any electronic copies.

Ethan H. Drew, P.E. No. 88622
Project Engineer

Ivan F. Sokolic, P.E.
Senior Engineer/Branch Manager

EHD:IFS

20-33-4505_Report_02-20-2020.docx(Geo 2020)



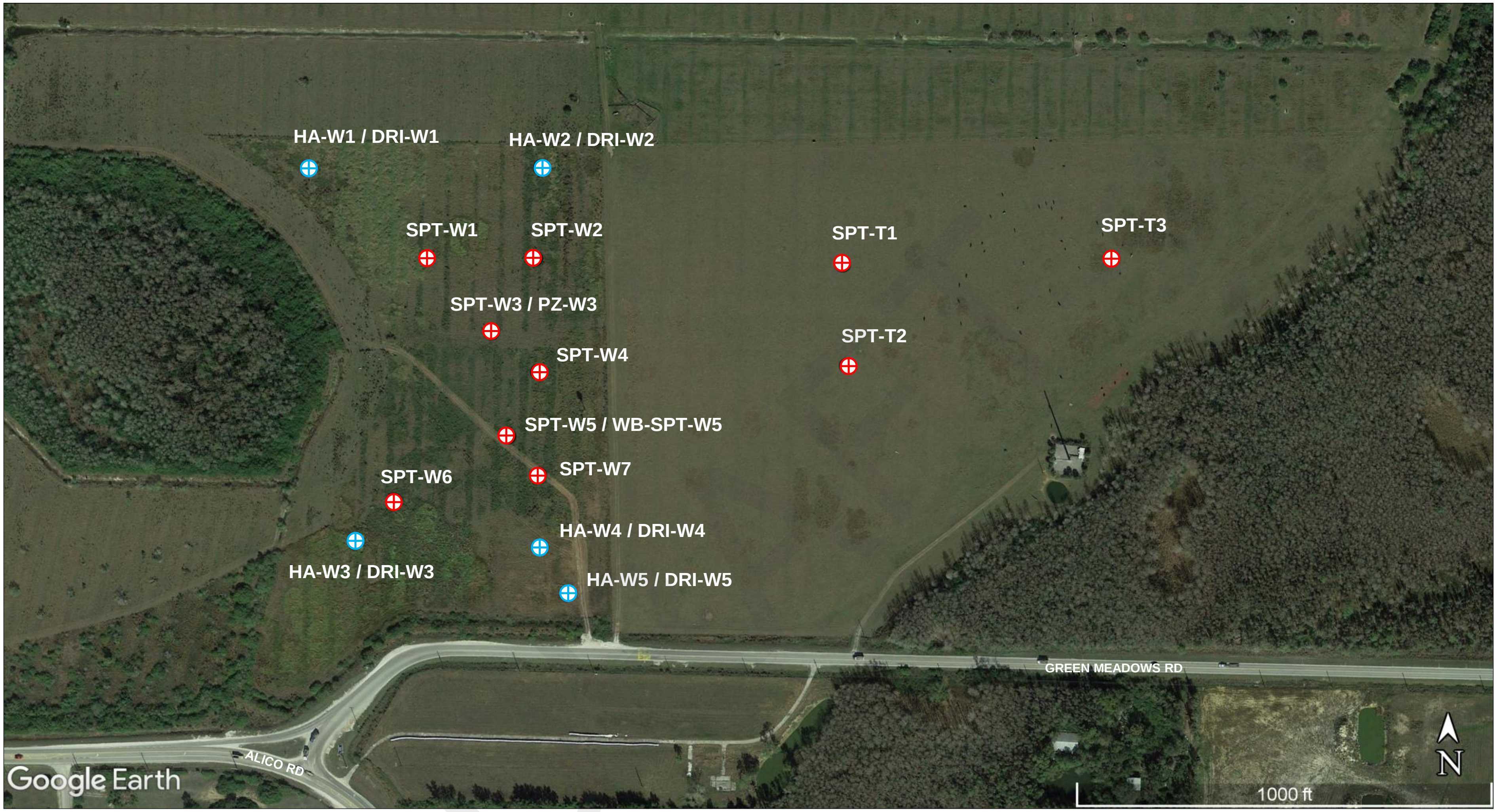
SITE LOCATION MAP

SOURCE: GOOGLE EARTH PRO

Ardaman & Associates, Inc.
Geotechnical, Environmental, and Materials Consultants

PROPOSED LEE COUNTY WATER TREATMENT AND TRANSFER FACILITIES PROJECT FORT MYERS, LEE COUNTY, FLORIDA

DRAWN BY: **ED**	CHECKED BY:	DATE: **12 FEB 2020**
FILE NO. **20-33-4505**	APPROVED BY:	FIGURE: **1**



Legend

- SPT = Standard Penetration Test Boring Location
- HA = Auger Boring Location
- PZ = Piezometer Location
- DRI = Double Ring Infiltrometer Test Location
- WB = Wash Boring

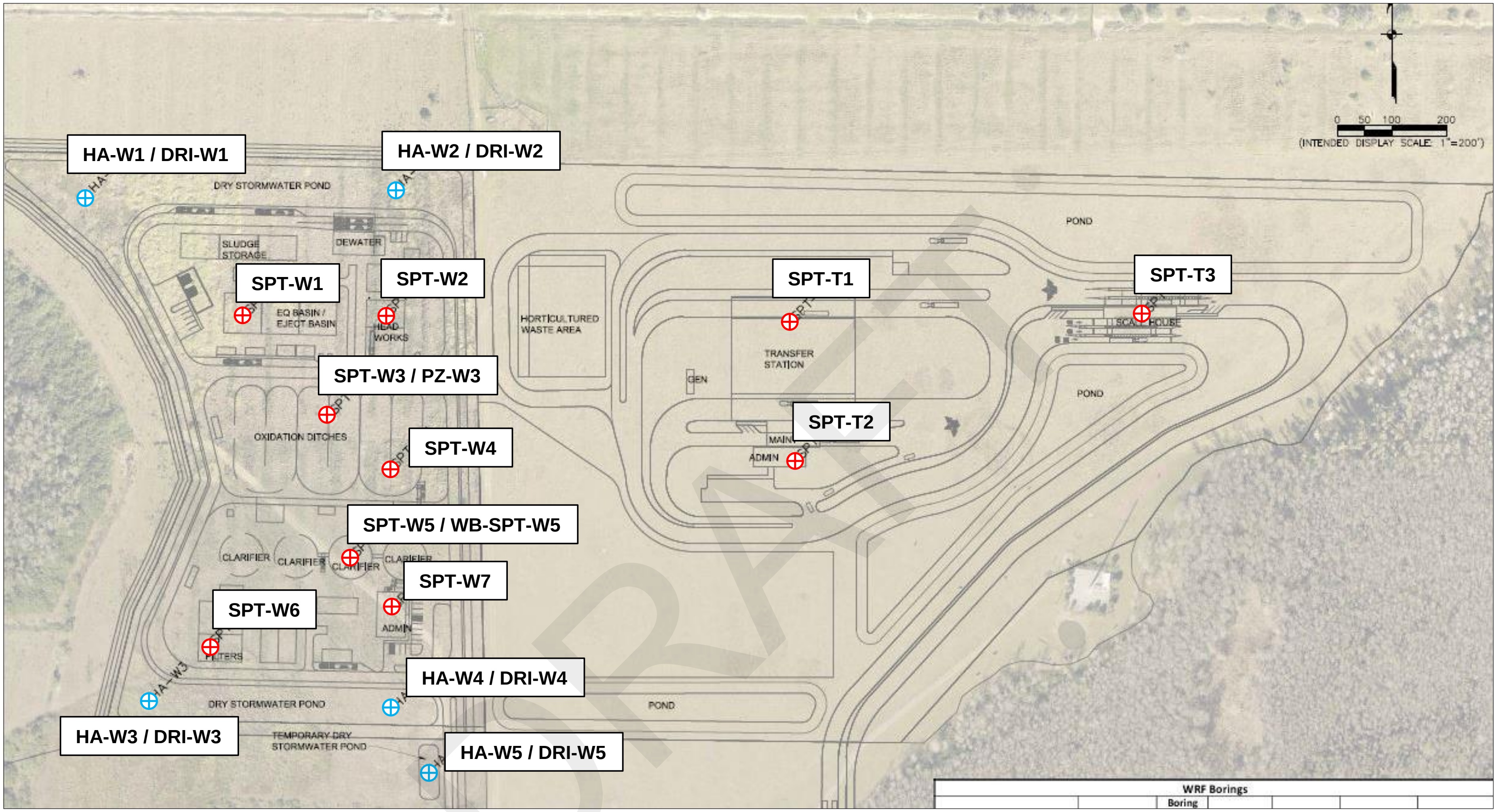
BORING LOCATION PLAN

SOURCE: GOOGLE EARTH PRO

Ardaman & Associates, Inc.
Geotechnical, Environmental, and Materials Consultants

PROPOSED LEE COUNTY WATER TREATMENT AND TRANSFER FACILITIES PROJECT FORT MYERS, LEE COUNTY, FLORIDA

DRAWN BY: ED	CHECKED BY:	DATE: 13 FEB 2020
FILE NO. 20-33-4505	APPROVED BY:	FIGURE: 2



Legend

SPT = Standard Penetration Test Boring Location

HA = Auger Boring Location

PZ = Piezometer Location

DRI = Double Ring Infiltrometer Test Location

WB = Wash Boring

N

SITE PLAN

SOURCE: SHEET NUMBER 01 TITLED SOUTHEAST WATER RECLAMATION FACILITY AND TRANSFER STATION LEE COUNTY, FL BY JOHNSON ENGINEERING DATED JANUARY 2020

Ardaman & Associates, Inc.
Geotechnical, Environmental, and Materials Consultants

PROPOSED LEE COUNTY WATER TREATMENT AND TRANSFER FACILITIES PROJECT FORT MYERS, LEE COUNTY, FLORIDA

DRAWN BY: **ED**	CHECKED BY:	DATE: **13 FEB 2020**
FILE NO. **20-33-4505**	APPROVED BY:	FIGURE: **3**

BORING LOCATION: SEE BORING LOCATION PLAN
LATITUDE: N 26° 29' 49.0" **LONGITUDE:** W 81° 43' 06.6"
DATE DRILLED: JAN 29 2020 **START:** **FINISH:**
GROUND SURFACE ELEVATION: **TIME:**
WATER TABLE DEPTH (ft): 3.0 **DATE:** JAN 29 2020

CLIENT: JOHNSON ENGINEERING, INC.
PROJECT: LEE COUNTY WATER TREATMENT AND TRANSFER FACILITIES PROJECT
LOCATION: FORT MYERS, LEE COUNTY, FLORIDA
DRILL CREW: LOCKLEY / CENTENO **LOGGED BY:** EHD

DRILL MAKE & MODEL: MOBILE B-57 W/ AUTO **BIT:** 3-7/8" DIA. DRAG **DRILLING RODS:** NW
DRILLING METHOD: ROTARY WASH WITH DRILLING FLUID **WEATHER CONDITIONS:** SUNNY

DEPTH, FT.	BLOWS	SPT N-VALUE	SAMPLE NO.	GRAPHIC LOG	USCS	SOIL DESCRIPTION	REMARKS	% WATER CONTENT	PERCENT FINES	% ORGANIC CONTENT	LIQUID LIMIT	PLAST. INDEX
0	1- 2- 4	6	1		SP-SM	Poorly Graded Sand with Silt - Brown to orange brown slightly silty fine sand.	20 feet of HW casing installed to stabilize borehole.					
	3- 4- 5	9	2									
	3- 2- 2	4	3									
5	4- 6- 7	13	4		SC	Clayey Sand - Grayish brown clayey fine sand.						
	5- 5- 6	11	5									
	7- 8- 10	18	6		SP	Poorly Graded Sand - Light gray fine sand.						
10	6- 7- 8	15	7									
						Soft Weathered Limestone.						
15	4- 9- 8	17	8									
						Hard Limestone.						
20	50/0"- -	50/0"	9									
25	10- 50/3"-	50/3"	10									
						Soft Weathered Limestone.						
30	10- 7- 10	17	11									
						TERMINATED AT 30.5 FEET						
35												

Ardaman & Associates, Inc.
Geotechnical, Environmental and Materials Consultants

REVIEWED BY: ETHAN H. DREW, P.E. **FILE NO:** 20-33-4505 **BORING NO.:** SPT-W1

BORING LOCATION: SEE BORING LOCATION PLAN
LATITUDE: N 26° 29' 49.0" **LONGITUDE:** W 81° 43' 03.7"
DATE DRILLED: 27 JAN 2020 **START:** **FINISH:**
GROUND SURFACE ELEVATION: **TIME:**
WATER TABLE DEPTH (ft): 3.5 **DATE:** 27 JAN 2020

CLIENT: JOHNSON ENGINEERING, INC.
PROJECT: LEE COUNTY WATER TREATMENT AND TRANSFER FACILITIES PROJECT
LOCATION: FORT MYERS, LEE COUNTY, FLORIDA
DRILL CREW: LOCKLEY / CENTENO **LOGGED BY:** EHD

DRILL MAKE & MODEL: MOBILE B-57 W/ AUTO **BIT:** 3-7/8" DIA. DRAG **DRILLING RODS:** NW
DRILLING METHOD: ROTARY WASH WITH DRILLING FLUID **WEATHER CONDITIONS:** SUNNY

DEPTH, FT.	BLOWS	SPT N-VALUE	SAMPLE NO.	GRAPHIC LOG	USCS	SOIL DESCRIPTION	REMARKS	% WATER CONTENT	PERCENT FINES	% ORGANIC CONTENT	LIQUID LIMIT	PLAST. INDEX
0	1- 2- 4	6	1		SP	Poorly Graded Sand - Grayish brown to light brown fine sand.	25 feet of HW casing installed to stabilize borehole.					
	4- 5- 8	13	2									
	4- 6- 10	16	3		SP-SM	Poorly Graded Sand with Silt - Brown slightly silty fine sand.		28	7			
5	6- 9- 12	21	4		SP	Poorly Graded Sand - Light gray fine sand.						
	8- 7- 8	15	5									
	7- 8- 10	18	6		SM	Silty Sand - Brown silty fine sand.		18	15			
10	9- 8- 8	16	7		SP	Poorly Graded Sand - Light brown fine sand.						
					SP-SM	Poorly Graded Sand with Silt - Gray slightly silty fine sand.						
15	3- 6- 7	13	8					24	7		NP	NP
						Hard Limestone.						
20	50/4"- -	50/4"	9									
25	10- 19- 50/4"	50/4"	10									
						Soft Weathered Limestone.	Loss of drilling fluid circulation at 28 feet.					
30	7- 5- 14	19	11									
35	12- 12- 16	28	12									

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REVIEWED BY: ETHAN H. DREW, P.E. **FILE NO:** 20-33-4505 **BORING NO.:** SPT-W2

DEPTH, FT.	BLOWS	SPT N-VALUE	SAMPLE NO.	GRAPHIC LOG	USCS	SOIL DESCRIPTION	REMARKS	% WATER CONTENT	PERCENT FINES	% ORGANIC CONTENT	LIQUID LIMIT	PLAST. INDEX
40	14- 10- 29	39	13									
					SM	Silty Sand with Gravel - Gray silty fine sand, some gravel (limestone fragments).						
45	6- 5- 2	7	14									
50	4- 4- 6	10	15									
						TERMINATED AT 50.5 FEET						
55												
60												
65												
70												
75												

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Geotechnical, Environmental and Materials Consultants

REVIEWED BY: ETHAN H. DREW, P.E. FILE NO: 20-33-4505 BORING NO.: SPT-W2

BORING LOCATION: SEE BORING LOCATION PLAN
LATITUDE: N 26° 29' 47.1" **LONGITUDE:** W 81° 43' 04.9"
DATE DRILLED: 24 JAN 2020 **START:** **FINISH:**
GROUND SURFACE ELEVATION: **TIME:**
WATER TABLE DEPTH (ft): 2.5 **DATE:** 24 JAN 2020

CLIENT: JOHNSON ENGINEERING, INC.
PROJECT: LEE COUNTY WATER TREATMENT AND TRANSFER FACILITIES PROJECT
LOCATION: FORT MYERS, LEE COUNTY, FLORIDA
DRILL CREW: LOCKLEY / CENTENO **LOGGED BY:** EHD

DRILL MAKE & MODEL: MOBILE B-57 W/ AUTO **BIT:** 3-7/8" DIA. DRAG **DRILLING RODS:** NW
DRILLING METHOD: ROTARY WASH WITH DRILLING FLUID **WEATHER CONDITIONS:** SUNNY

DEPTH, FT.	BLOWS	SPT N-VALUE	SAMPLE NO.	GRAPHIC LOG	USCS	SOIL DESCRIPTION	REMARKS	% WATER CONTENT	PERCENT FINES	% ORGANIC CONTENT	LIQUID LIMIT	PLAST. INDEX
0	2- 3- 3	6	1		SP-SM	Poorly Graded Sand with Silt - Grayish brown slightly silty fine sand.	40 feet of HW casing installed to stabilize borehole.					
	4- 5- 3	8	2		SP	Poorly Graded Sand - Light brown to light gray fine sand.						
	2- 3- 6	9	3									
5	5- 8- 10	18	4									
	5- 6- 7	13	5									
	8- 10- 10	20	6									
10	7- 6- 10	16	7		SC	Clayey Sand - Gray clayey fine sand.		18	18			
					SM	Silty Sand - Gray silty fine sand.						
15	5- 6- 6	12	8									
						Hard Limestone.						
20	50/3"- -	50/3"	9									
					SM	Silty Sand - Gray silty fine sand, trace gravel (limestone fragments).						
25	3- 17- 20	37	10									
					SM	Silty Sand with Gravel - Gray silty fine sand, some gravel (limestone fragments).						
30	1- 2- 1	3	11									
35	20- 16- 12	28	12				Partial loss of drilling fluid circulation at 34 feet.					

DEPTH, FT.	BLOWS	SPT N-VALUE	SAMPLE NO.	GRAPHIC LOG	USCS	SOIL DESCRIPTION	REMARKS	% WATER CONTENT	PERCENT FINES	% ORGANIC CONTENT	LIQUID LIMIT	PLAST. INDEX
						Hard Limestone.						
40	50/1"- -	50/1"	13									
					SM	Silty Sand with Gravel - Gray silty fine sand, some gravel (limestone fragments).						
45	12- 5- 4	9	14									
					SM	Silty Sand - Gray very silty fine sand, trace gravel (limestone fragments).						
50	1- 2- 3	5	15					31	37			
					SC	Clayey Sand - Light brown very clayey fine sand.						
55	1- 0- 1	1	16					44	45			
					CL	Sandy Lean Clay - Light brown sandy clay.						
							PP = 0.5 tsf					
60	0- 2- 2	4	17					47	58		30	7
					SC	Clayey Sand - Greenish gray clayey fine sand.						
							Shelby tube recovered from 64 to 66 feet.					
65			U1									
	1- 0- 5	5	18									
70	0- 0- 4	4	19									
					CH	Fat Clay with Sand - Gray clay, trace to some sand.						
							PP = 0.75 tsf					
75	0- 0- 4	4	20					133	84			
					CH	Sandy Fat Clay - Greenish gray sandy clay.						

DEPTH, FT.	BLOWS	SPT N-VALUE	SAMPLE NO.	GRAPHIC LOG	USCS	SOIL DESCRIPTION	REMARKS	% WATER CONTENT	PERCENT FINES	% ORGANIC CONTENT	LIQUID LIMIT	PLAST. INDEX
							Shelby tube recovered from 79 to 81 feet.					
80			U2									
	0- 6- 5	11	21				PP = 0.5 tsf	66	81		71	45
85	0- 0- 5	5	22				PP = 0.5 tsf	53	54			
					SC	Clayey Sand - Greenish gray clayey fine sand.						
90	5- 4- 5	9	23									
						Hard Limestone.						
95	31- 32- 28	60	24									
						TERMINATED AT 95.5 FEET						
100												
105												
110												
115												
120												

BORING LOCATION: SEE BORING LOCATION PLAN
LATITUDE: N 26° 29' 46.2" **LONGITUDE:** W 81° 43' 03.6"
DATE DRILLED: 29 JAN 2020 **START:** **FINISH:**
GROUND SURFACE ELEVATION: **TIME:**
WATER TABLE DEPTH (ft): 2.25 **DATE:** 29 JAN 2020

CLIENT: JOHNSON ENGINEERING, INC.
PROJECT: LEE COUNTY WATER TREATMENT AND TRANSFER FACILITIES PROJECT
LOCATION: FORT MYERS, LEE COUNTY, FLORIDA
DRILL CREW: LOCKLEY / CENTENO **LOGGED BY:** EHD

DRILL MAKE & MODEL: MOBILE B-57 W/ AUTO **BIT:** 3-7/8" DIA. DRAG **DRILLING RODS:** NW
DRILLING METHOD: ROTARY WASH WITH DRILLING FLUID **WEATHER CONDITIONS:** SUNNY

DEPTH, FT.	BLOWS	SPT N-VALUE	SAMPLE NO.	GRAPHIC LOG	USCS	SOIL DESCRIPTION	REMARKS	% WATER CONTENT	PERCENT FINES	% ORGANIC CONTENT	LIQUID LIMIT	PLAST. INDEX
0	1- 2- 4	6	1		SP-SM	Poorly Graded Sand with Silt - Grayish brown slightly silty fine sand.	25 feet of HW casing installed to stabilize borehole.					
	2- 4- 4	8	2		SP	Poorly Graded Sand - Brown fine sand.						
	3- 4- 4	8	3									
5	5- 8- 11	19	4		SP-SM	Poorly Graded Sand with Silt - Brown slightly silty fine sand.						
	8- 10- 10	20	5									
	7- 10- 11	21	6									
10	5- 6- 9	15	7		SP	Poorly Graded Sand - Light brown fine sand.						
15	6- 12- 7	19	8			Soft Weathered Limestone.						
20	56- 10- 10	20	9				Loss of drilling fluid circulation between 17.5 and 20 feet.					
						Hard Limestone.						
25	27- 49- 50/2"	50/2"	10				Potential void between 26 and 27 feet.					
						Soft Weathered Limestone.	Potential void between 28 and 29 feet.					
30	26- 23- 11	34	11									
						TERMINATED AT 30.5 FEET						
35												

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Geotechnical, Environmental and Materials Consultants

REVIEWED BY: ETHAN H. DREW, P.E. **FILE NO:** 20-33-4505 **BORING NO.:** SPT-W4

BORING LOCATION: SEE BORING LOCATION PLAN
LATITUDE: N 26° 29' 44.6" **LONGITUDE:** W 81° 43' 04.4"
DATE DRILLED: 28 JAN 2020 **START:** **FINISH:**
GROUND SURFACE ELEVATION: **TIME:**
WATER TABLE DEPTH (ft): 3.5 **DATE:** 28 JAN 2020

CLIENT: JOHNSON ENGINEERING, INC.
PROJECT: LEE COUNTY WATER TREATMENT AND TRANSFER FACILITIES PROJECT
LOCATION: FORT MYERS, LEE COUNTY, FLORIDA
DRILL CREW: LOCKLEY / CENTENO **LOGGED BY:** EHD

DRILL MAKE & MODEL: MOBILE B-57 W/ AUTO **BIT:** 3-7/8" DIA. DRAG **DRILLING RODS:** NW
DRILLING METHOD: ROTARY WASH WITH DRILLING FLUID **WEATHER CONDITIONS:** SUNNY

DEPTH, FT.	BLOWS	SPT N-VALUE	SAMPLE NO.	GRAPHIC LOG	USCS	SOIL DESCRIPTION	REMARKS	% WATER CONTENT	PERCENT FINES	% ORGANIC CONTENT	LIQUID LIMIT	PLAST. INDEX
0	1- 3- 5	8	1		SP-SM	Poorly Graded Sand with Silt - Grayish brown slightly silty fine sand.	25 feet of HW casing installed to stabilize borehole.					
	3- 4- 4	8	2		SP	Poorly Graded Sand - Light gray fine sand.						
	5- 5- 6	11	3		SP-SM	Poorly Graded Sand with Silt - Brown slightly silty fine sand.						
5	6- 6- 7	13	4									
	4- 5- 8	13	5		SP	Poorly Graded Sand - Brown fine sand.		21	3			
	5- 5- 7	12	6									
10	4- 6- 8	14	7		SP-SM	Poorly Graded Sand with Silt - Brown slightly silty fine sand.						
					SC	Clayey Sand - Grayish brown clayey fine sand.						
15	7- 8- 9	17	8					16	18			
20	1- 50/1"-	50/1"	9			Hard Limestone.						
							Loss of drilling fluid circulation at 21.5 feet.					
25	48- 21- 37	58	10									
					SM	Silty Sand with Gravel - Gray silty fine sand, some gravel (limestone fragments).	Loss of drilling fluid circulation at 27 feet.					
30	4- 1- 0	1	11									
35	3- 1- 3	4	12									

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Geotechnical, Environmental and Materials Consultants

REVIEWED BY: ETHAN H. DREW, P.E. **FILE NO:** 20-33-4505 **BORING NO.:** SPT-W5

DEPTH, FT.	BLOWS	SPT N-VALUE	SAMPLE NO.	GRAPHIC LOG	USCS	SOIL DESCRIPTION	REMARKS	% WATER CONTENT	PERCENT FINES	% ORGANIC CONTENT	LIQUID LIMIT	PLAST. INDEX
						Hard Limestone.						
40	50/0"- -	50/0"	13									
					SM	Silty Sand with Gravel - Gray silty fine sand, some gravel (limestone fragments).						
45	8- 2- 3	5	14									
50	3- 4- 2	6	15									
						TERMINATED AT 50.5 FEET						
55												
60												
65												
70												
75												

BORING LOCATION: SEE BORING LOCATION PLAN
LATITUDE: N 26° 29' 44.6" **LONGITUDE:** W 81° 43' 04.4"
DATE DRILLED: 31 JAN 2020 **START:** **FINISH:**
GROUND SURFACE ELEVATION: **TIME:**
WATER TABLE DEPTH (ft): GNM **DATE:** 31 JAN 2020

CLIENT: JOHNSON ENGINEERING, INC.
PROJECT: LEE COUNTY WATER TREATMENT AND TRANSFER FACILITIES PROJECT
LOCATION: FORT MYERS, LEE COUNTY, FLORIDA
DRILL CREW: LOCKLEY / CENTENO **LOGGED BY:** EHD

DRILL MAKE & MODEL: MOBILE B-57 W/ AUTO **BIT:** 3-7/8" DIA. DRAG **DRILLING RODS:** NW
DRILLING METHOD: ROTARY WASH WITH DRILLING FLUID **WEATHER CONDITIONS:** SUNNY

DEPTH, FT.	BLOWS	SPT N-VALUE	SAMPLE NO.	GRAPHIC LOG	USCS	SOIL DESCRIPTION	REMARKS	% WATER CONTENT	PERCENT FINES	% ORGANIC CONTENT	LIQUID LIMIT	PLAST. INDEX
0						Wash Boring to 28 feet to attempt to obtain shelby tube (undisturbed sample).	25 feet of HW casing installed to stabilize borehole.					
5												
10												
15												
20												
25												
			1		SM	Silty Sand with Gravel - Gray silty fine sand, some gravel (limestone fragments).	Attempt to recover shelby tube from 28 - 30 feet. Disturbed recovery.					
30						TERMINATED AT 30 FEET						
35												

Ardaman & Associates, Inc.
Geotechnical, Environmental and Materials Consultants

REVIEWED BY: ETHAN H. DREW, P.E. **FILE NO:** 20-33-4505 **BORING NO.:** WB-SPT-W5

BORING LOCATION: SEE BORING LOCATION PLAN
LATITUDE: N 26° 29' 43.0" **LONGITUDE:** W 81° 43' 07.4"
DATE DRILLED: 29 JAN 2020 **START:** **FINISH:**
GROUND SURFACE ELEVATION: **TIME:**
WATER TABLE DEPTH (ft): 3.0 **DATE:** 29 JAN 2020

CLIENT: JOHNSON ENGINEERING, INC.
PROJECT: LEE COUNTY WATER TREATMENT AND TRANSFER FACILITIES PROJECT
LOCATION: FORT MYERS, LEE COUNTY, FLORIDA
DRILL CREW: LOCKLEY / CENTENO **LOGGED BY:** EHD

DRILL MAKE & MODEL: MOBILE B-57 W/ AUTO **BIT:** 3-7/8" DIA. DRAG **DRILLING RODS:** NW
DRILLING METHOD: ROTARY WASH WITH DRILLING FLUID **WEATHER CONDITIONS:** SUNNY

DEPTH, FT.	BLOWS	SPT N-VALUE	SAMPLE NO.	GRAPHIC LOG	USCS	SOIL DESCRIPTION	REMARKS	% WATER CONTENT	PERCENT FINES	% ORGANIC CONTENT	LIQUID LIMIT	PLAST. INDEX
0	1- 2- 2	4	1		SP-SM	Poorly Graded Sand with Silt - Grayish brown slightly silty fine sand.	20 feet of HW casing installed to stabilize borehole.					
	3- 4- 5	9	2		SP	Poorly Graded Sand - Brown fine sand.						
	3- 5- 5	10	3		SC	Clayey Sand - Brown clayey fine sand.						
5	4- 4- 5	9	4		SP-SM	Poorly Graded Sand with Silt - Brown slightly silty fine sand.						
	5- 6- 7	13	5									
	6- 8- 7	15	6		SP	Poorly Graded Sand - Light gray fine sand.						
10	7- 7- 8	15	7									
					SC	Clayey Sand - Gray clayey fine sand.						
15	6- 7- 9	16	8									
						Hard Limestone.						
20	17- 50/1"-	50/1"	9				Loss of drilling fluid circulation at 20 feet.					
25	50/0"- -	50/0"	10									
						Soft Weathered Limestone.						
30	12- 12- 10	22	11									
						TERMINATED AT 30.5 FEET						
35												

Ardaman & Associates, Inc.
Geotechnical, Environmental and Materials Consultants

REVIEWED BY: ETHAN H. DREW, P.E. **FILE NO:** 20-33-4505 **BORING NO.:** SPT-W6

BORING LOCATION: SEE BORING LOCATION PLAN
LATITUDE: N 26° 29' 43.6" **LONGITUDE:** W 81° 43' 03.6"
DATE DRILLED: 28 JAN 2020 **START:** **FINISH:**
GROUND SURFACE ELEVATION: **TIME:**
WATER TABLE DEPTH (ft): 2.75 **DATE:** 28 JAN 2020

CLIENT: JOHNSON ENGINEERING, INC.
PROJECT: LEE COUNTY WATER TREATMENT AND TRANSFER FACILITIES PROJECT
LOCATION: FORT MYERS, LEE COUNTY, FLORIDA
DRILL CREW: LOCKLEY / CENTENO **LOGGED BY:** EHD

DRILL MAKE & MODEL: MOBILE B-57 W/ AUTO **BIT:** 3-7/8" DIA. DRAG **DRILLING RODS:** NW
DRILLING METHOD: ROTARY WASH WITH DRILLING FLUID **WEATHER CONDITIONS:** SUNNY

DEPTH, FT.	BLOWS	SPT N-VALUE	SAMPLE NO.	GRAPHIC LOG	USCS	SOIL DESCRIPTION	REMARKS	% WATER CONTENT	PERCENT FINES	% ORGANIC CONTENT	LIQUID LIMIT	PLAST. INDEX
0	2- 3- 3	6	1		SP-SM	Poorly Graded Sand with Silt - Grayish brown slightly silty fine sand.	25 feet of HW casing installed to stabilize borehole.					
	4- 6- 8	14	2		SP	Poorly Graded Sand - Light gray to brown fine sand.						
	6- 6- 5	11	3									
5	5- 4- 4	8	4									
	2- 2- 2	4	5		SP-SM	Poorly Graded Sand with Silt - Brown slightly silty fine sand.						
	2- 4- 6	10	6		SM	Silty Sand - Brown silty fine sand.		18	13			
10	6- 8- 10	18	7									
					SC	Clayey Sand - Grayish brown clayey fine sand.						
15	6- 8- 10	18	8									
20	W- O- H	WOH	9				Loss of drilling fluid circulation at 18 feet.	26	20			
						Hard Limestone.						
25	50/1"- -	50/1"	10									
						Soft Weathered Limestone.	Loss of drilling fluid circulation between 28 and 29 feet (potential void).					
30	11- 20- 14	34	11									
35	20- 12- 13	25	12									

Ardaman & Associates, Inc.
Geotechnical, Environmental and Materials Consultants

REVIEWED BY: ETHAN H. DREW, P.E. **FILE NO:** 20-33-4505 **BORING NO.:** SPT-W7

DEPTH, FT.	BLOWS	SPT N-VALUE	SAMPLE NO.	GRAPHIC LOG	USCS	SOIL DESCRIPTION	REMARKS	% WATER CONTENT	PERCENT FINES	% ORGANIC CONTENT	LIQUID LIMIT	PLAST. INDEX
						Hard Limestone.						
40	26- 16- 50/3"	50/3"	13									
						Soft Weathered Limestone.						
45	8- 13- 10	23	14									
					SM	Silty Sand with Gravel - Gray silty fine sand, some gravel (limestone fragments).						
50	4- 4- 4	8	15									
						TERMINATED AT 50.5 FEET						
55												
60												
65												
70												
75												

DRAFT

BORING LOCATION: SEE BORING LOCATION PLAN
LATITUDE: N 26° 29' 48.8" **LONGITUDE:** W 81° 42' 55.6"
DATE DRILLED: 30 JAN 2020 **START:** **FINISH:**
GROUND SURFACE ELEVATION: **TIME:**
WATER TABLE DEPTH (ft): 2.8 **DATE:** 30 JAN 2020

CLIENT: JOHNSON ENGINEERING, INC.
PROJECT: LEE COUNTY WATER TREATMENT AND TRANSFER FACILITIES PROJECT
LOCATION: FORT MYERS, LEE COUNTY, FLORIDA
DRILL CREW: LOCKLEY / CENTENO **LOGGED BY:** EHD

DRILL MAKE & MODEL: MOBILE B-57 W/ AUTO **BIT:** 3-7/8" DIA. DRAG **DRILLING RODS:** NW
DRILLING METHOD: ROTARY WASH WITH DRILLING FLUID **WEATHER CONDITIONS:** SUNNY

DEPTH, FT.	BLOWS	SPT N-VALUE	SAMPLE NO.	GRAPHIC LOG	USCS	SOIL DESCRIPTION	REMARKS	% WATER CONTENT	PERCENT FINES	% ORGANIC CONTENT	LIQUID LIMIT	PLAST. INDEX
0	1- 2- 2	4	1		SP-SM	Poorly Graded Sand with Silt - Brown slightly silty fine sand.	20 feet of HW casing installed to stabilize borehole.					
	3- 3- 4	7	2									
	4- 7- 7	14	3		SC	Clayey Sand - Gray clayey fine sand.						
5	6- 9- 10	19	4									
	9- 10- 12	22	5									
	10- 11- 10	21	6		SP-SM	Poorly Graded Sand with Silt - Brown slightly silty fine sand.						
10	11- 12- 13	25	7									
15	5- 6- 6	12	8									
20	50/0"- -	50/0"	9			Hard Limestone.	Loss of drilling fluid circulation at 19 feet.					
25	50/0"- -	50/0"	10									
30	14- 50/1"-	50/1"	11									
						TERMINATED AT 30.1 FEET						
35												

Ardaman & Associates, Inc.
Geotechnical, Environmental and Materials Consultants

REVIEWED BY: ETHAN H. DREW, P.E. **FILE NO:** 20-33-4505 **BORING NO.:** SPT-T1

BORING LOCATION: SEE BORING LOCATION PLAN
LATITUDE: N 26° 29' 46.3" **LONGITUDE:** W 81° 42' 55.4"
DATE DRILLED: 30 JAN 2020 **START:** **FINISH:**
GROUND SURFACE ELEVATION: **TIME:**
WATER TABLE DEPTH (ft): 3.0 **DATE:** 30 JAN 2020

CLIENT: JOHNSON ENGINEERING, INC.
PROJECT: LEE COUNTY WATER TREATMENT AND TRANSFER FACILITIES PROJECT
LOCATION: FORT MYERS, LEE COUNTY, FLORIDA
DRILL CREW: LOCKLEY / CENTENO **LOGGED BY:** EHD

DRILL MAKE & MODEL: MOBILE B-57 W/ AUTO **BIT:** 3-7/8" DIA. DRAG **DRILLING RODS:** NW
DRILLING METHOD: ROTARY WASH WITH DRILLING FLUID **WEATHER CONDITIONS:** SUNNY

DEPTH, FT.	BLOWS	SPT N-VALUE	SAMPLE NO.	GRAPHIC LOG	USCS	SOIL DESCRIPTION	REMARKS	% WATER CONTENT	PERCENT FINES	% ORGANIC CONTENT	LIQUID LIMIT	PLAST. INDEX
0	1- 2- 2	4	1		SP-SM	Poorly Graded Sand with Silt - Brown slightly silty fine sand.	20 feet of HW casing installed to stabilize borehole.					
	4- 6- 3	9	2		SP	Poorly Graded Sand - Light brown fine sand.						
	4- 4- 7	11	3		SM	Silty Sand - Brown silty fine sand.		19	16		NP	NP
5	7- 8- 11	19	4									
	7- 10- 11	21	5		SP	Poorly Graded Sand - Light brown to light gray fine sand.						
	8- 8- 9	17	6									
10	6- 7- 7	14	7		SC	Clayey Sand - Gray clayey fine sand.						
15	6- 6- 7	13	8									
						Hard Limestone.						
20	16- 50/0"-	50/0"	9									
25	21- 50/1"-	50/1"	10									
						Soft Weathered Limestone.						
30	8- 5- 8	13	11									
						Hard Limestone.						
35	50/1"- -	50/1"	12									

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Geotechnical, Environmental and Materials Consultants

REVIEWED BY: ETHAN H. DREW, P.E. **FILE NO:** 20-33-4505 **BORING NO.:** SPT-T2

DEPTH, FT.	BLOWS	SPT N-VALUE	SAMPLE NO.	GRAPHIC LOG	USCS	SOIL DESCRIPTION	REMARKS	% WATER CONTENT	PERCENT FINES	% ORGANIC CONTENT	LIQUID LIMIT	PLAST. INDEX
40	22- 23- 26	49	13									
						Soft Weathered Limestone.						
45	21- 10- 9	19	14									
50	3- 5- 7	12	15									
						TERMINATED AT 50.5 FEET						
55												
60												
65												
70												
75												

BORING LOCATION: SEE BORING LOCATION PLAN
LATITUDE: N 26° 29' 48.9" **LONGITUDE:** W 81° 42' 48.4"
DATE DRILLED: 30 JAN 2020 **START:** **FINISH:**
GROUND SURFACE ELEVATION: **TIME:**
WATER TABLE DEPTH (ft): 3.0 **DATE:** 30 JAN 2020

CLIENT: JOHNSON ENGINEERING, INC.
PROJECT: LEE COUNTY WATER TREATMENT AND TRANSFER FACILITIES PROJECT
LOCATION: FORT MYERS, LEE COUNTY, FLORIDA
DRILL CREW: LOCKLEY / CENTENO **LOGGED BY:** EHD

DRILL MAKE & MODEL: MOBILE B-57 W/ AUTO **BIT:** 3-7/8" DIA. DRAG **DRILLING RODS:** NW
DRILLING METHOD: ROTARY WASH WITH DRILLING FLUID **WEATHER CONDITIONS:** SUNNY

DEPTH, FT.	BLOWS	SPT N-VALUE	SAMPLE NO.	GRAPHIC LOG	USCS	SOIL DESCRIPTION	REMARKS	% WATER CONTENT	PERCENT FINES	% ORGANIC CONTENT	LIQUID LIMIT	PLAST. INDEX
0	1- 2- 4	6	1		SP-SM	Poorly Graded Sand with Silt - Brown slightly silty fine sand.	20 feet of HW casing installed to stabilize borehole.					
	4- 3- 3	6	2		SP	Poorly Graded Sand - Brown fine sand.						
	2- 1- 3	4	3		SC	Clayey Sand - Gray to brown clayey fine sand.						
5	5- 6- 10	16	4									
	10- 10- 10	20	5									
	7- 9- 10	19	6		SP-SC	Poorly Graded Sand with Clay - Brown slightly clayey fine sand.		20	9			
10	9- 7- 7	14	7		SC	Clayey Sand - Gray clayey fine sand.						
15	5- 6- 5	11	8									
						Hard Limestone.						
20	50/0"- -	50/0"	9									
25	50/0"- -	50/0"	10									
						Soft Weathered Limestone.						
30	48- 26- 10	36	11									
						TERMINATED AT 30.5 FEET						
35												

DRAFT

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Geotechnical, Environmental and Materials Consultants

PAGE 1 **OF** 1
REVIEWED BY: ETHAN H. DREW, P.E. **FILE NO:** 20-33-4505 **BORING NO.:** SPT-T3

APPENDIX I

Soil Boring, Sampling and Testing Methods and
Project Soil Description Procedure - Unified

SOIL BORING, SAMPLING AND TESTING METHODS

STANDARD PENETRATION TEST

The Standard Penetration Test (SPT) is a widely accepted method of in-situ testing of foundation soils (ASTM D-1586). A 2-foot (0.6 m) long, 2-inch (50 mm) O.D. split-barrel sampler attached to the end of a string of drilling rods is driven 18 inches (0.45 m) into the ground by successive blows of a 140-pound (63.5 Kg) hammer freely dropping 30 inches (0.76 m). The number of blows needed for each 6 inches (0.15 m) of penetration is recorded. The sum of the blows required for penetration of the second and third 6-inch (0.15 m) increments penetration constitutes the test result or N-value. After the test, the sampler is extracted from the ground and opened to allow visual description of the retained soil sample. The N-value has been empirically correlated with various soil properties allowing a conservative estimate of the behavior of soils under load. The following tables relate N-values to a qualitative description of soil density and, for cohesive soils, an approximate unconfined compressive strength (Qu):

Cohesionless Soils:

N-Value Safety Hammer	N-Value Auto Hammer	Description	Relative Density
< 4	< 3	Very loose	0 - 15%
4 - 10	3 - 8	Loose	15 - 35%
10 - 30	8 - 24	Medium dense	35 - 65%
30 - 50	24 - 40	Dense	65 - 85%
> 50	> 40	Very dense	85 - 100%

Cohesive Soils:

N-Value Safety Hammer	N-Value Auto Hammer	Description	Unconfined Compressive Strength, Qu
< 2	< 1	Very soft	< 0.25 tsf (25 kPa)
2 - 4	1 - 3	Soft	0.25 - 0.50 tsf (25 - 50 kPa)
4 - 8	3 - 6	Firm	0.50 - 1.0 tsf (50 - 100 kPa)
8 - 15	6 - 12	Stiff	1.0 - 2.0 tsf (100 - 200 kPa)
15 - 30	12 - 24	Very stiff	2.0 - 4.0 tsf (200 - 400 kPa)
> 30	> 24	Hard	> 4.0 tsf (400 kPa)

The tests are usually performed at 5-foot (1.5 m) intervals. However, more frequent or continuous testing is done by our firm through depths where a more accurate definition of the soils is required. The test holes are advanced to the test elevations by rotary drilling with a cutting bit, using circulating fluid to remove the cuttings and hold the fine grains in suspension. The circulating fluid, which is bentonitic drilling mud, is also used to keep the hole open below the water table by maintaining an excess hydrostatic pressure inside the hole. In some soil deposits, particularly highly pervious ones, flush-coupled casing must be driven to just above the testing depth to keep the hole open and/or prevent the loss of circulating fluid. After completion of a test boring, the hole is kept open until a steady state groundwater level is recorded. The hole is then sealed by backfilling with neat cement.

Representative split-spoon samples from each sampling interval and from different strata are brought to our laboratory in air-tight jars for classification and testing, if necessary. Afterwards, the samples are discarded unless prior arrangements have been made.

POWER AUGER BORINGS

Auger borings are used when a relatively large, continuous sampling of soil strata close to the ground surface is desired. A 4-inch (100 mm) diameter, continuous flight, helical auger with a cutting head at its end is screwed into the ground in 5-foot (1.5 m) sections. It is powered by the rotary drill rig. The sample is recovered by withdrawing the auger out of the ground without rotating it. The soil sample so obtained, is described and representative samples put in bags or jars and returned to the laboratory for classification and testing, if necessary.

HAND AUGER BORINGS

Hand auger borings are used, if soil conditions are favorable, when the soil strata are to be determined within a shallow (approximately 5-foot [1.5 m]) depth or when access is not available to power drilling equipment. A 3-inch (75 mm) diameter hand bucket auger with a cutting head is simultaneously turned and pressed into the ground. The bucket auger is retrieved at approximately 6-inch (0.15 m) intervals and its contents emptied for inspection. Sometimes post-hole diggers are used, especially in the upper 3 feet (1 m) or so. The soil sample obtained is described and representative samples put in bags or jars and transported to the laboratory for classification and testing, if necessary.

UNDISTURBED SAMPLING

Undisturbed sampling implies the recovery of soil samples in a state as close to their natural condition as possible. Complete preservation of in-situ conditions cannot be realized; however, with careful handling and proper sampling techniques, disturbance during sampling can be minimized for most geotechnical engineering purposes. Testing of undisturbed samples gives a more accurate estimate of in-situ behavior than is possible with disturbed samples.

Normally, we obtain undisturbed samples by pushing a 2.875-inch (73 mm) I.D., thin wall seamless steel tube 24 inches (0.6 m) into the soil with a single stroke of a hydraulic ram. The sampler, which is a Shelby tube, is 30 (0.8 m) inches long. After the sampler is retrieved, the ends are sealed in the field and it is transported to our laboratory for visual description and testing, as needed. Undisturbed sampling is noted on the boring logs as thus "U-".

LABORATORY TEST METHODS

Soil samples returned to our laboratory are looked at again by a geotechnical engineer or geotechnician to obtain more accurate descriptions of the soil strata. Laboratory testing is performed on selected samples as deemed necessary to aid in soil classification and to help define engineering properties of the soils. The test results are presented on the soil boring logs at the depths at which the respective sample was recovered, except that grain-size distributions or selected other test results may be presented on separate tables, figures or plates as discussed in this report, the results of which will be located in an Appendix. The soil descriptions shown on the logs are based upon visual-manual procedures in accordance with local practice. Soil classification is in general accordance with the Unified Soil Classification System (ASTM D-2487) and is also based on visual-manual procedures. Following is a list of abbreviations that may appear in the Remarks column on the boring logs indicating additional laboratory testing was performed, the results of which will usually be located in an Appendix.

DD: Unit Weight/Classification of Undisturbed "Shelby Tube" samples

PP: Pocket Penetrometer reading on cohesive samples in tons per sq. ft. (tsf)

k: Hydraulic Conductivity

Qu: Unconfined Compression Strength; ASTM D-2166

UU: Unconsolidated-Undrained Triaxial Test; ASTM D 2850

Consol: One-Dimensional Consolidation test performed on subsample from undisturbed sample; ASTM D-2435

THE PROJECT SOIL DESCRIPTION PROCEDURE FOR SOUTHWEST FLORIDA[1]
For use with the ASTM D 2487 Unified Soil Classification System
CLASSIFICATION OF SOILS FOR ENGINEERING PURPOSES

BOULDERS (>12" [300 mm]) and COBBLES (3" [75 mm] TO 12" [300 mm]):

GRAVEL:

Coarse Gravel: 3/4" (19 mm) to 3" (75 mm)
Fine Gravel: No. 4 (4.75 mm) Sieve to 3/4" (19 mm)

Descriptive adjectives:

0 – 5%	---	no mention of gravel in description
5 – 15%	---	trace
15 – 29%	---	some
30 – 49%	---	gravelly (shell, limerock, cemented sands)

SANDS

COARSE SAND: No. 10 (2 mm) Sieve to No. 4 (4.75 mm) Sieve
MEDIUM SAND: No. 40 (425 μm) Sieve to No. 10 (2 mm) Sieve
FINE SAND: No. 200 (75 μm) Sieve to No. 40 (425 μm) Sieve

Descriptive adjectives:

0 – 5%	---	no mention of sand in description
5 – 15%	---	trace
15 – 29%	---	some
30 – 49%	---	sandy

SILT/CLAY: < #200 (75 μm) sieve

SILTY OR SILT: PI < 4
SILTY CLAYEY OR SILTY CLAY: 4 ≤ PI ≤ 7
CLAYEY OR CLAY: PI > 7

Descriptive adjectives:

0 – 5%	--- clean (no mention of silt or clay in description)
5 – 12% to 15%	--- slightly
16 – 35%	--- clayey, silty, or silty clayey
36 – 49%	--- very

ORGANIC SOILS

Organic Content	Descriptive adjectives	Classification
0 – 2.5%	no mention of organics in description	See above
2.6 – 5%	slightly organic	See above
5 – 20%	organic	Add "with organic fines" to group name

THE PROJECT SOIL DESCRIPTION PROCEDURE FOR SOUTHWEST FLORIDA[1]
For use with the ASTM D 2487 Unified Soil Classification System
CLASSIFICATION OF SOILS FOR ENGINEERING PURPOSES

HIGHLY ORGANIC SOILS AND MATTER

Organic Content	Description	Classification
20-75%	highly organic sand or muck	Peat (PT)
	sandy peat	Peat (PT)
>75%	amorphous or fibrous peat	Peat (PT)

STRATIFICATION AND STRUCTURE

Descriptive Term with interbedded	Thickness
seam:	less than 1/2-inch (13 mm) thick
layer:	1/2 to 12-inches (13 to 300 mm) thick
stratum:	more than 12-inches (300 mm) thick
pocket:	small, erratic deposit, usually less than 1-foot
occasional:	one or less per foot of thickness
frequent:	more than one per foot of thickness
calcareous:	containing calcium carbonate (reaction to diluted HCL)
hardpan:	spodic horizon usually medium dense
marl:	mixture of carbonate clays, silts, shells and sands.

ROCK CLASSIFICATION

Description

Hard Limestone or Caprock – N-values >50 bpf

Soft Weathered Limestone – N values <50 bpf

---

(1) This soil description procedure was developed specifically for projects in southwest Florida because it is believed that the terminology will be better understood as a result of local practice. It is not intended to supplant other visual-manual classification procedures for description and identification of soils such as ASTM D 2488. BY: G.A. DREW, P.E. (1995) (Revised 2016).

UNIFIED SOIL CLASSIFICATION SYSTEM (ASTM D2487)

Criteria for Assigning Group Symbols and Group Names Using Laboratory Tests [A]				Soil Classification: Group Symbol	Soil Classification: Group Name [B]
Coarse Grained Soils: More than 50% retained on No. 200 sieve	**Gravels:** More than 50% of coarse fraction retained on No. 4 sieve	**Clean Gravels:** Less than 5% fines [C]	$Cu \geq 4$ and $1 \leq Cc \leq 3$ [E]	GW	Well-graded gravel [F]
			$Cu < 4$ and/or $1 > Cc > 3$ [E]	GP	Poorly graded gravel [F]
		Gravels with Fines: More than 12% fines [C]	Fines classify as ML or MH	GM	Silty gravel [F,G,H]
			Fines classify as CL or CH	GC	Clayey gravel [F,G,H]
	Sands: 50% or more of coarse fraction passes No. 4 sieve	**Clean Sands:** Less than 5% fines [D]	$Cu \geq 6$ and $1 \leq Cc \leq 3$ [E]	SW	Well-graded sand [I]
			$Cu < 6$ and/or $1 > Cc > 3$ [E]	SP	Poorly graded sand [I]
		Sands with Fines: More than 12% fines [D]	Fines classify as ML or MH	SM	Silty sand [G,H,I]
			Fines classify as CL or CH	SC	Clayey sand [G,H,I]
Fine-Grained Soils: 50% or more passes the No. 200 sieve	**Silts and Clays:** Liquid limit less than 50	**Inorganic:**	PI > 7 and plots on or above "A" line [J]	CL	Lean clay [K,L,M]
			PI < 4 or plots below "A" line [J]	ML	Silt [K,L,M]
		Organic:	$\frac{\text{Liquid limit - oven dried}}{\text{Liquid limit - not dried}} < 0.75$	OL	Organic clay [K,L,M,N]; Organic silt [K,L,M,O]
	Silts and Clays: Liquid limit 50 or more	**Inorganic:**	PI plots on or above "A" line	CH	Fat clay [K,L,M]
			PI plots below "A" line	MH	Elastic Silt [K,L,M]
		Organic:	$\frac{\text{Liquid limit - oven dried}}{\text{Liquid limit - not dried}} < 0.75$	OH	Organic clay [K,L,M,P]; Organic silt [K,L,M,Q]
Highly organic soils:	Primarily organic matter, dark in color, and organic odor			PT	Peat

[A] Based on the material passing the 3-in. (75-mm) sieve

[B] If field sample contained cobbles or boulders, or both, add "with cobbles or boulders, or both" to group name.

[C] Gravels with 5 to 12% fines require dual symbols: GW-GM well-graded gravel with silt, GW-GC well-graded gravel with clay, GP-GM poorly graded gravel with silt, GP-GC poorly graded gravel with clay.

[D] Sands with 5 to 12% fines require dual symbols: SW-SM well-graded sand with silt, SW-SC well-graded sand with clay, SP-SM poorly graded sand with silt, SP-SC poorly graded sand with clay

[E] $Cu = D_{60}/D_{10} \quad Cc = \frac{(D_{30})^2}{D_{10} \times D_{60}}$

[F] If soil contains $\geq$ 15% sand, add "with sand" to group name.

[G] If fines classify as CL-ML, use dual symbol GC-GM, or SC-SM.

[H] If fines are organic, add "with organic fines" to group name.

[I] If soil contains $\geq$ 15% gravel, add "with gravel" to group name.

[J] If Atterberg limits plot in shaded area, soil is a CL-ML, silty clay.

[K] If soil contains 15 to 29% plus No. 200, add "with sand" or "with gravel," whichever is predominant.

[L] If soil contains $\geq$ 30% plus No. 200 predominantly sand, add "sandy" to group name.

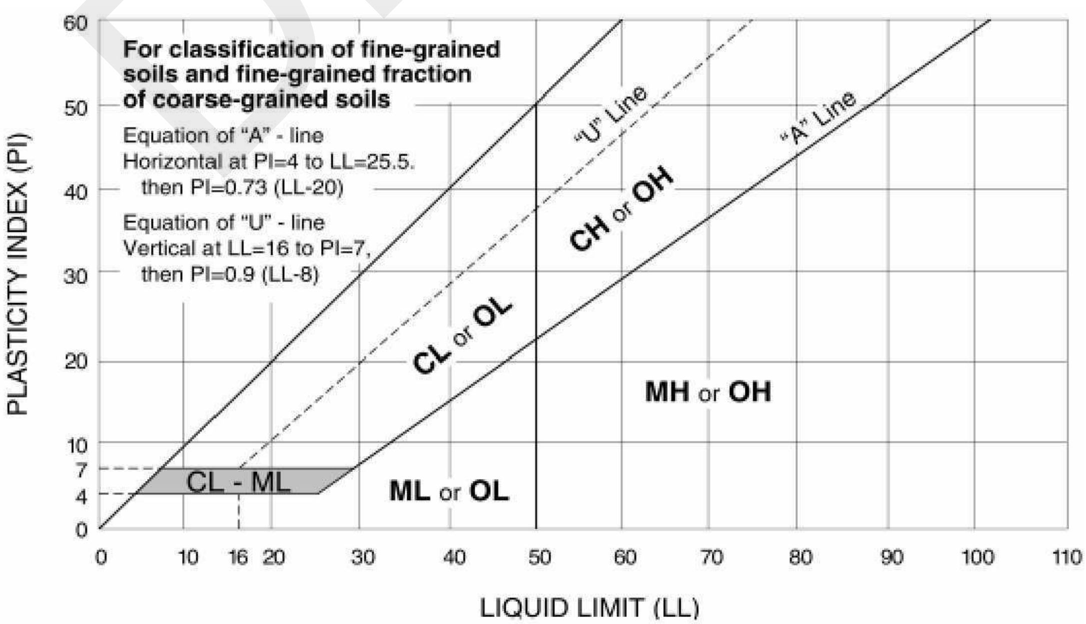
[M] If soil contains $\geq$ 30% plus No. 200, predominantly gravel, add "gravelly" to group name.

[N] PI $\geq$ 4 and plots on or above "A" line.

[O] PI < 4 or plots below "A" line.

[P] PI plots on or above "A" line.

[Q] PI plots below "A" line.



APPENDIX II

Double-Ring Infiltrometer (DRI) Test Results

ARDAMAN & ASSOCIATES, INC.

Geotechnical, Environmental and
Materials Consultants

DOUBLE-RING INFILTRATION TEST RESULTS

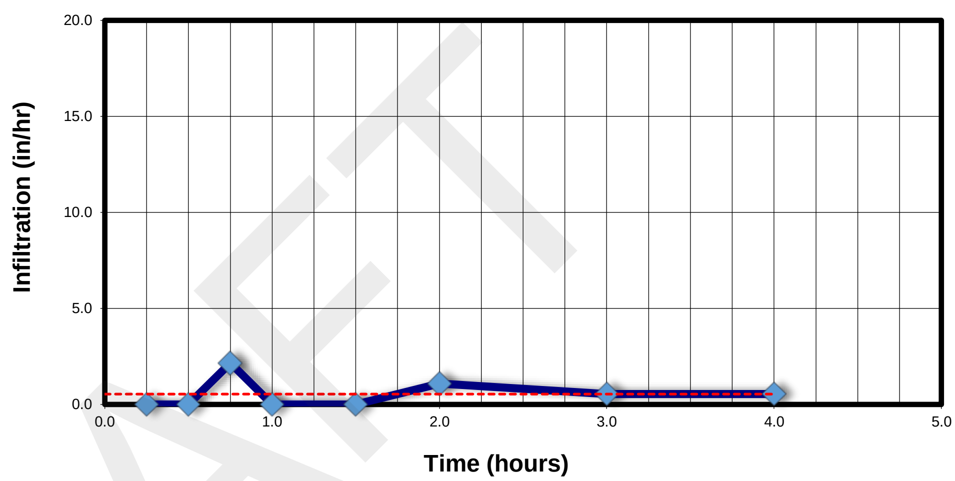
(ASTM STANDARD D-3385)

Project Name:	**Lee County WRF and TF1**	**Test Date:**	**2/3/2020**
Project Location:	**Fort Myers, Lee County, FL**	**Test Location:**	**DRI-W1**
Project Number:	**20-33-4505**	**Test Depth:**	**12" below existing ground surface**
Outer Ring Diameter (in):	**24**	**Duration (hours):**	**4**
Inner Ring Diameter (in):	**12**	**Test Head (inches):**	**4**

INFILTRATION RATE: 0.5 inches per hour

Time Increment (minutes)	Infiltration per Time Period (inches)
15	0.00
15	0.00
15	0.54
15	0.00
30	0.00
30	0.54
60	0.54
60	0.54

INFILTRATION RATE



SUBSURFACE SOIL DATA

Depth (ft) From	-	To	BORING DATA
0.0		1.0	Brown slightly silty fine sand (SP-SM)
1.0		3.0	Orange brown slightly silty fine sand (SP-SM)
3.0		4.0	Orange brown silty fine sand (SM)
4.0		5.0	Grayish brown slightly silty fine sand (SP-SM)

Groundwater level encountered at a depth of 3 feet below the existing ground surface at time of test.

TEST PROCEDURES:

The double-ring infiltration test was performed in general accordance with procedures outlined in the ASTM Standard D-3385. Two 18-inch high concentric rings were placed on a prepared test surface at a given depth and driven into the ground 4 to 6-inches. The inner ring used in the test had an inside diameter of approximately 12-inches, while the outer ring had an inside diameter of approximately 24-inches. The test was performed by filling both rings with water to a heigth of 12 inches. A head of 3 to 6-inches is then maintained in both rings, and the amount of water required to maintain the head in the inner ring was recorded.

ARDAMAN & ASSOCIATES, INC.

Geotechnical, Environmental and
Materials Consultants

DOUBLE-RING INFILTRATION TEST RESULTS

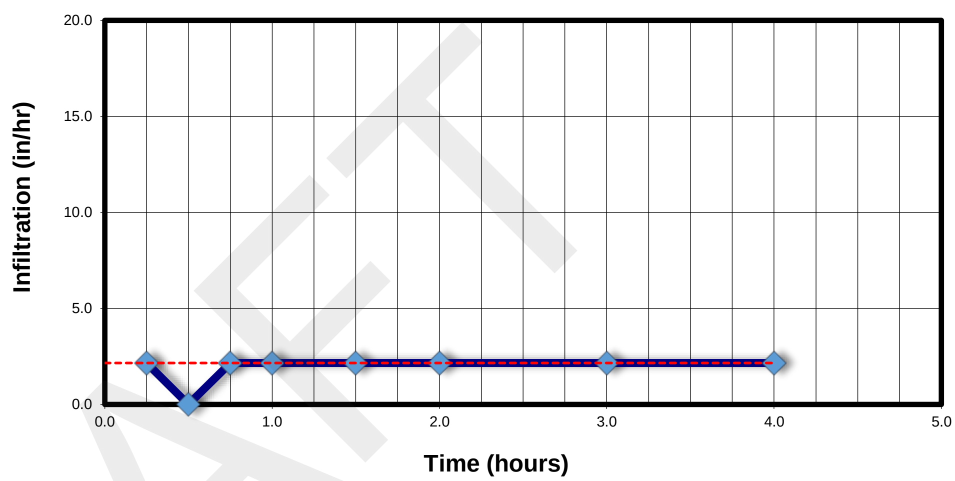
(ASTM STANDARD D-3385)

Project Name:	**Lee County WRF and TF1**	**Test Date:**	**2/3/2020**
Project Location:	**Fort Myers, Lee County, FL**	**Test Location:**	**DRI-W2**
Project Number:	**20-33-4505**	**Test Depth:**	**6" below existing ground surface**
Outer Ring Diameter (in):	**24**	**Duration (hours):**	**4**
Inner Ring Diameter (in):	**12**	**Test Head (inches):**	**4**

INFILTRATION RATE: 2.2 inches per hour

Time Increment (minutes)	Infiltration per Time Period (inches)
15	0.54
15	0.00
15	0.54
15	0.54
30	1.08
30	1.08
60	2.16
60	2.16

INFILTRATION RATE



SUBSURFACE SOIL DATA

Depth (ft) From	-	To	BORING DATA
0.0		2.0	Brown slightly silty fine sand (SP-SM)
2.0		3.0	Brown silty fine sand (SM)
3.0		4.0	Brown clayey fine sand (SC)
4.0		5.0	Brown fine sand (SP)

Groundwater level encountered at a depth of 2.5 feet below the existing ground surface at time of test.

TEST PROCEDURES:

The double-ring infiltration test was performed in general accordance with procedures outlined in the ASTM Standard D-3385. Two 18-inch high concentric rings were placed on a prepared test surface at a given depth and driven into the ground 4 to 6-inches. The inner ring used in the test had an inside diameter of approximately 12-inches, while the outer ring had an inside diameter of approximately 24-inches. The test was performed by filling both rings with water to a heigth of 12 inches. A head of 3 to 6-inches is then maintained in both rings, and the amount of water required to maintain the head in the inner ring was recorded.

ARDAMAN & ASSOCIATES, INC.

Geotechnical, Environmental and
Materials Consultants

DOUBLE-RING INFILTRATION TEST RESULTS

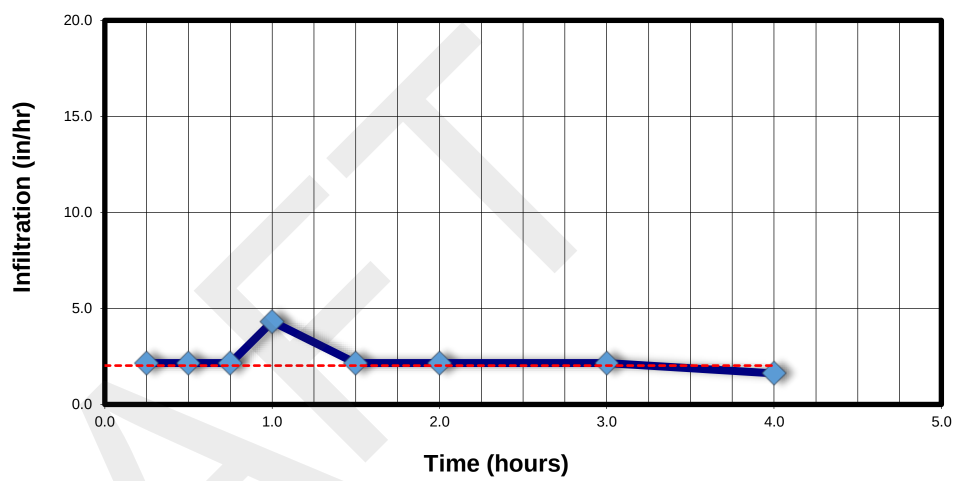
(ASTM STANDARD D-3385)

Project Name:	**Lee County WRF and TF1**	**Test Date:**	**2/4/2020**
Project Location:	**Fort Myers, Lee County, FL**	**Test Location:**	**DRI-W3**
Project Number:	**20-33-4505**	**Test Depth:**	**12" below existing ground surface**
Outer Ring Diameter (in):	**24**	**Duration (hours):**	**4**
Inner Ring Diameter (in):	**12**	**Test Head (inches):**	**4**

INFILTRATION RATE: 2.0 inches per hour

Time Increment (minutes)	Infiltration per Time Period (inches)
15	0.54
15	0.54
15	0.54
15	1.08
30	1.08
30	1.08
60	2.16
60	1.62

INFILTRATION RATE



SUBSURFACE SOIL DATA

Depth (ft) From	-	To	BORING DATA
0.0		2.0	Grayish brown slightly silty fine sand (SP-SM)
2.0		4.0	Brown slightly silty fine sand (SP-SM)
4.0		5.0	Light brown fine sand (SP)

Groundwater level encountered at a depth of 2 feet below the existing ground surface at time of test.

TEST PROCEDURES:

The double-ring infiltration test was performed in general accordance with procedures outlined in the ASTM Standard D-3385. Two 18-inch high concentric rings were placed on a prepared test surface at a given depth and driven into the ground 4 to 6-inches. The inner ring used in the test had an inside diameter of approximately 12-inches, while the outer ring had an inside diameter of approximately 24-inches. The test was performed by filling both rings with water to a heigth of 12 inches. A head of 3 to 6-inches is then maintained in both rings, and the amount of water required to maintain the head in the inner ring was recorded.

ARDAMAN & ASSOCIATES, INC.

Geotechnical, Environmental and
Materials Consultants

DOUBLE-RING INFILTRATION TEST RESULTS

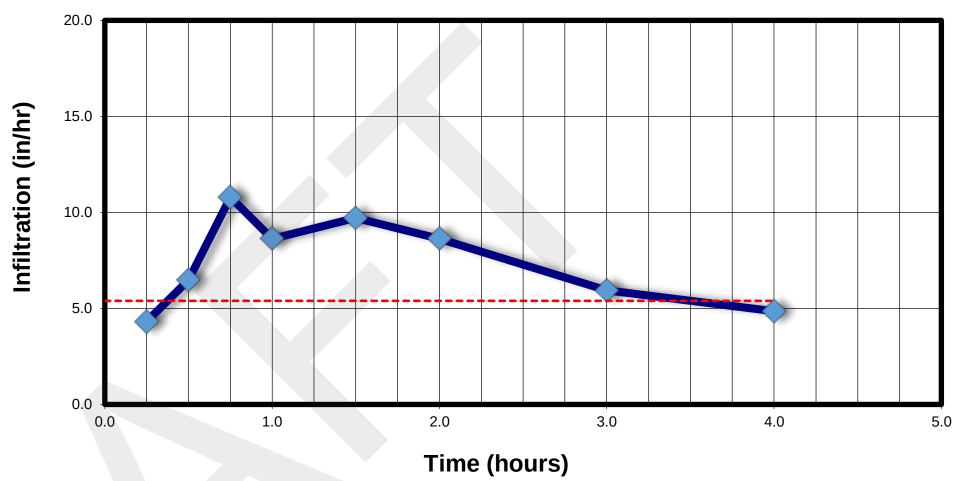
(ASTM STANDARD D-3385)

Project Name:	**Lee County WRF and TF1**	**Test Date:**	**2/11/2020**
Project Location:	**Fort Myers, Lee County, FL**	**Test Location:**	**DRI-W4**
Project Number:	**20-33-4505**	**Test Depth:**	**6" below existing ground surface**
Outer Ring Diameter (in):	**24**	**Duration (hours):**	**4**
Inner Ring Diameter (in):	**12**	**Test Head (inches):**	**4**

INFILTRATION RATE: 5.4 inches per hour

Time Increment (minutes)	Infiltration per Time Period (inches)
15	1.08
15	1.62
15	2.70
15	2.16
30	4.86
30	4.32
60	5.94
60	4.86

INFILTRATION RATE



SUBSURFACE SOIL DATA

Depth (ft) From	-	To	BORING DATA
0.0		2.0	Light brown slightly silty fine sand (SP-SM)
2.0		3.0	Brown slightly silty fine sand (SP-SM)
3.0		4.0	Light brown clayey fine sand (SC)
4.0		5.0	Light brown silty fine sand (SM)

Groundwater level encountered at a depth of 1.5 feet below the existing ground surface at time of test.

TEST PROCEDURES:

The double-ring infiltration test was performed in general accordance with procedures outlined in the ASTM Standard D-3385. Two 18-inch high concentric rings were placed on a prepared test surface at a given depth and driven into the ground 4 to 6-inches. The inner ring used in the test had an inside diameter of approximately 12-inches, while the outer ring had an inside diameter of approximately 24-inches. The test was performed by filling both rings with water to a heigth of 12 inches. A head of 3 to 6-inches is then maintained in both rings, and the amount of water required to maintain the head in the inner ring was recorded.

ARDAMAN & ASSOCIATES, INC.

Geotechnical, Environmental and
Materials Consultants

DOUBLE-RING INFILTRATION TEST RESULTS

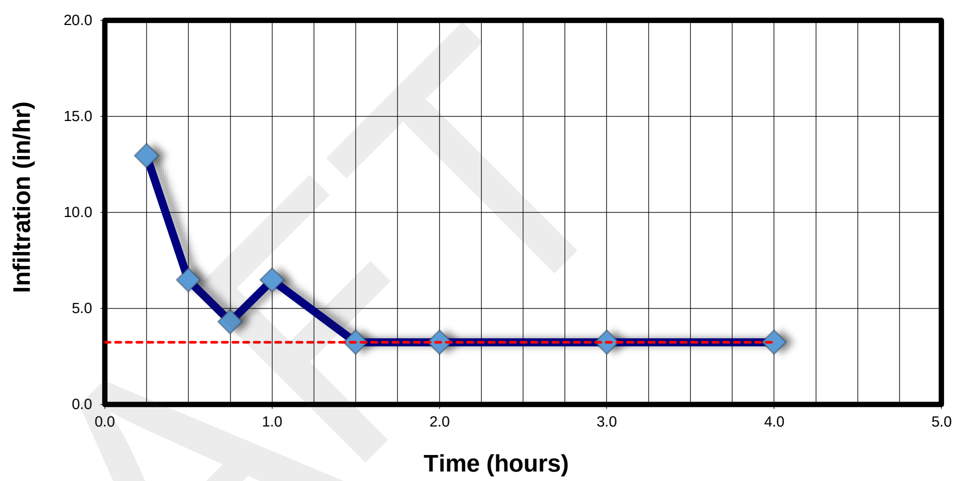
(ASTM STANDARD D-3385)

Project Name:	**Lee County WRF and TF1**	**Test Date:**	**2/11/2020**
Project Location:	**Fort Myers, Lee County, FL**	**Test Location:**	**DRI-W5**
Project Number:	**20-33-4505**	**Test Depth:**	**6" below existing ground surface**
Outer Ring Diameter (in):	**24**	**Duration (hours):**	**4**
Inner Ring Diameter (in):	**12**	**Test Head (inches):**	**4**

INFILTRATION RATE: 3.2 inches per hour

Time Increment (minutes)	Infiltration per Time Period (inches)
15	3.24
15	1.62
15	1.08
15	1.62
30	1.62
30	1.62
60	3.24
60	3.24

INFILTRATION RATE



SUBSURFACE SOIL DATA

Depth (ft) From	- To	BORING DATA
0.0	1.0	Grayish brown slightly silty fine sand (SP-SM)
1.0	2.0	Gray fine sand (SP)
2.0	3.0	Grayish brown slightly silty fine sand (SP-SM)
3.0	5.0	Brown slightly silty fine sand (SP-SM)

Groundwater level encountered at a depth of 2 feet below the existing ground surface at time of test.

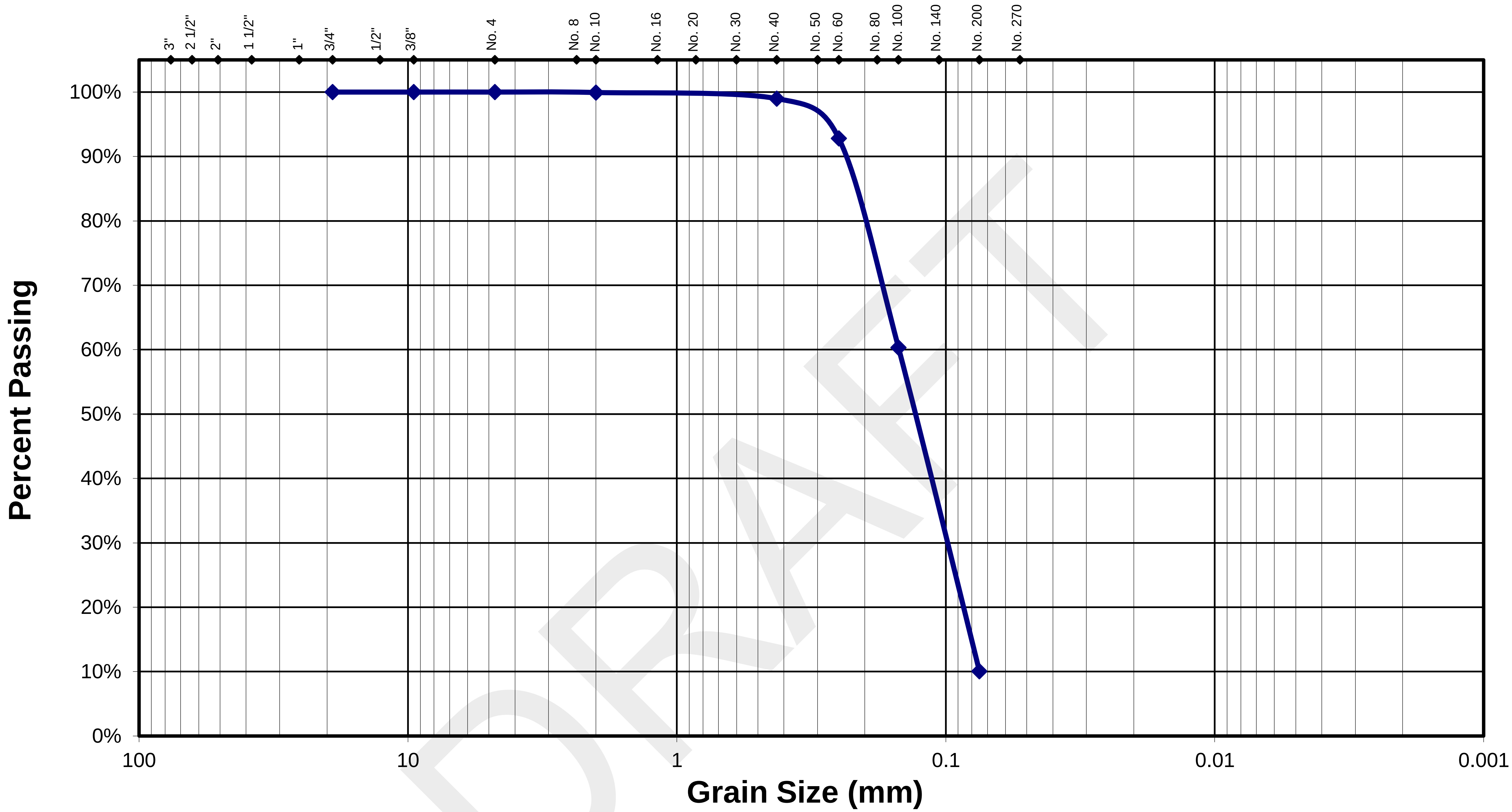
TEST PROCEDURES:

The double-ring infiltration test was performed in general accordance with procedures outlined in the ASTM Standard D-3385. Two 18-inch high concentric rings were placed on a prepared test surface at a given depth and driven into the ground 4 to 6-inches. The inner ring used in the test had an inside diameter of approximately 12-inches, while the outer ring had an inside diameter of approximately 24-inches. The test was performed by filling both rings with water to a heigth of 12 inches. A head of 3 to 6-inches is then maintained in both rings, and the amount of water required to maintain the head in the inner ring was recorded.

APPENDIX III

Grain-Size Distribution Curves

GRAIN SIZE DISTRIBUTION CURVE



GRAVEL	SAND		SILT	CLAY
	Coarse to Medium	Fine		

Project Name: Lee County WRF and TF1

Project Location: Lee County, Florida

Client Name: Johnson Engineering

A&A File Number: 20-33-4505

Sample No.: 2 **Sample Location:** HA-W1

Sample Description: SP-SM - Orange brown slightly silty fine sand

Percent Passing No. 200 Sieve = 10.1% **LL:** Not Tested **PL:** Not Tested **PI:** Not Tested

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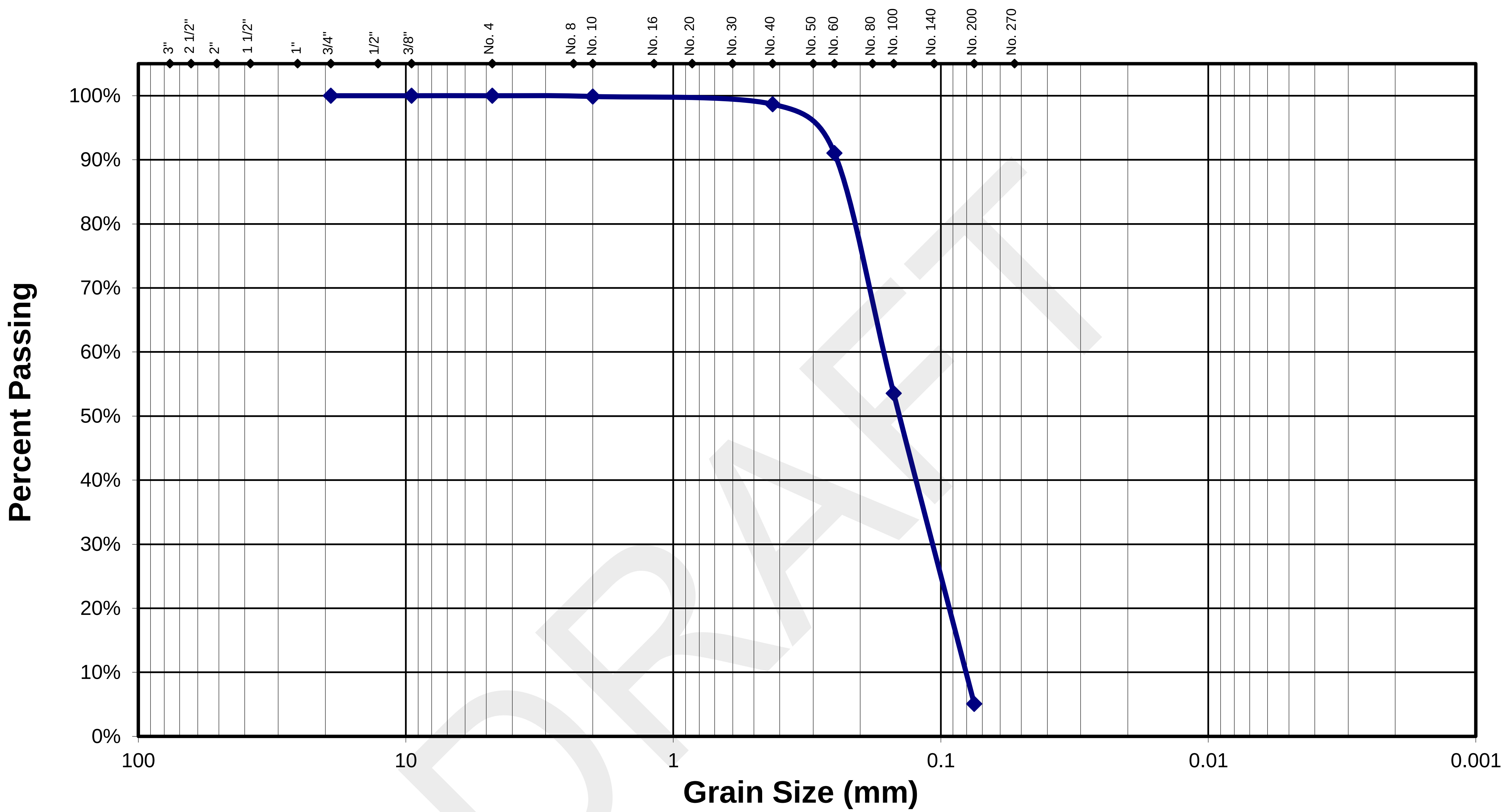
GRAIN SIZE DISTRIBUTION CURVE



GRAVEL	SAND		SILT	CLAY
	Coarse to Medium	Fine		

Project Name: Lee County WRF and TF1
Project Location: Lee County, Florida
Client Name: Johnson Engineering

A&A File Number: 20-33-4505

Sample No.: 1 **Sample Location:** HA-W2
Sample Description: SP-SM - Brown slightly silty fine sand
Percent Passing No. 200 Sieve = 5.1% **LL:** Not Tested **PL:** Not Tested **PI:** Not Tested

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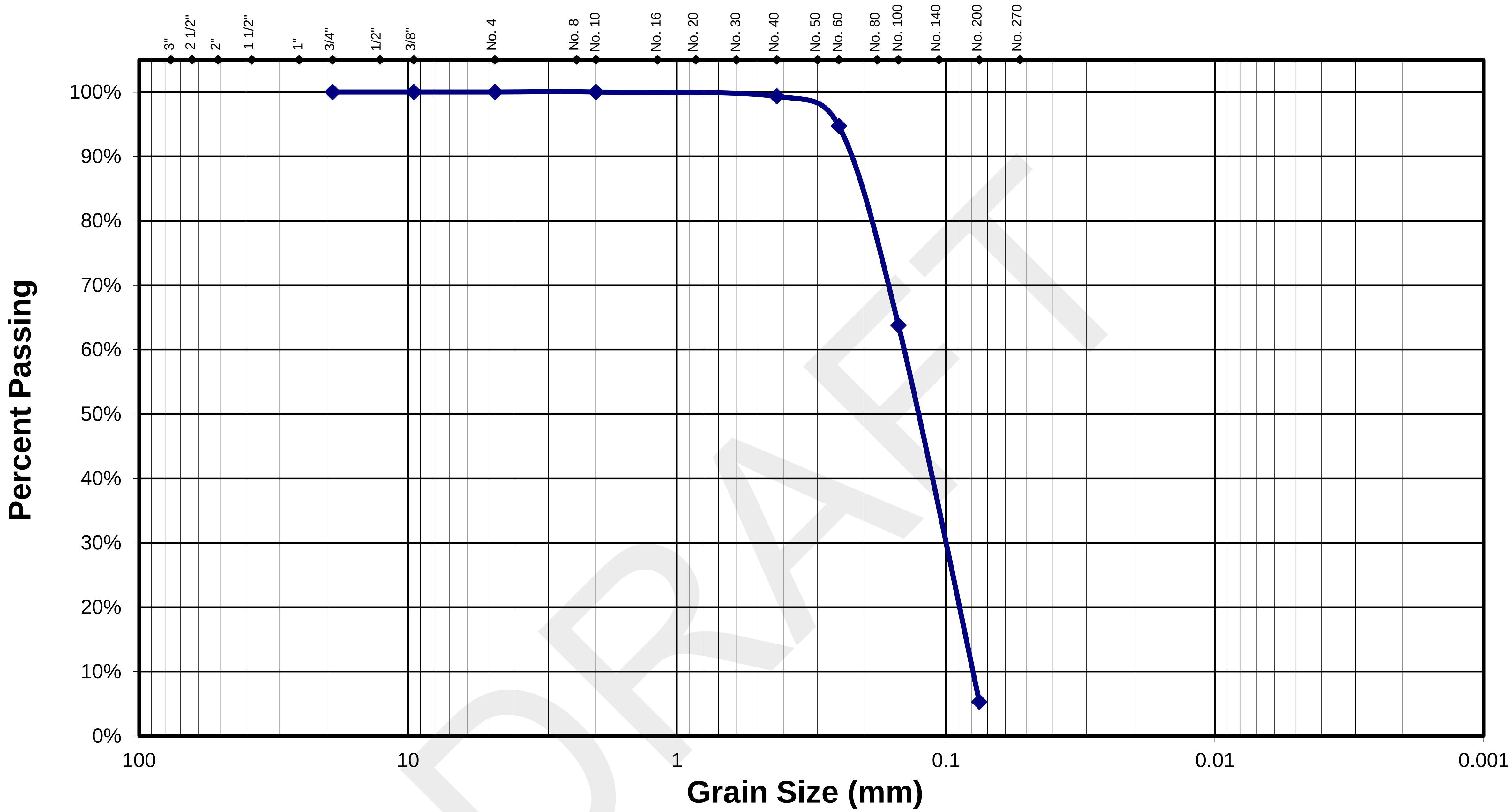
GRAIN SIZE DISTRIBUTION CURVE



GRAVEL	SAND		SILT	CLAY
	Coarse to Medium	Fine		

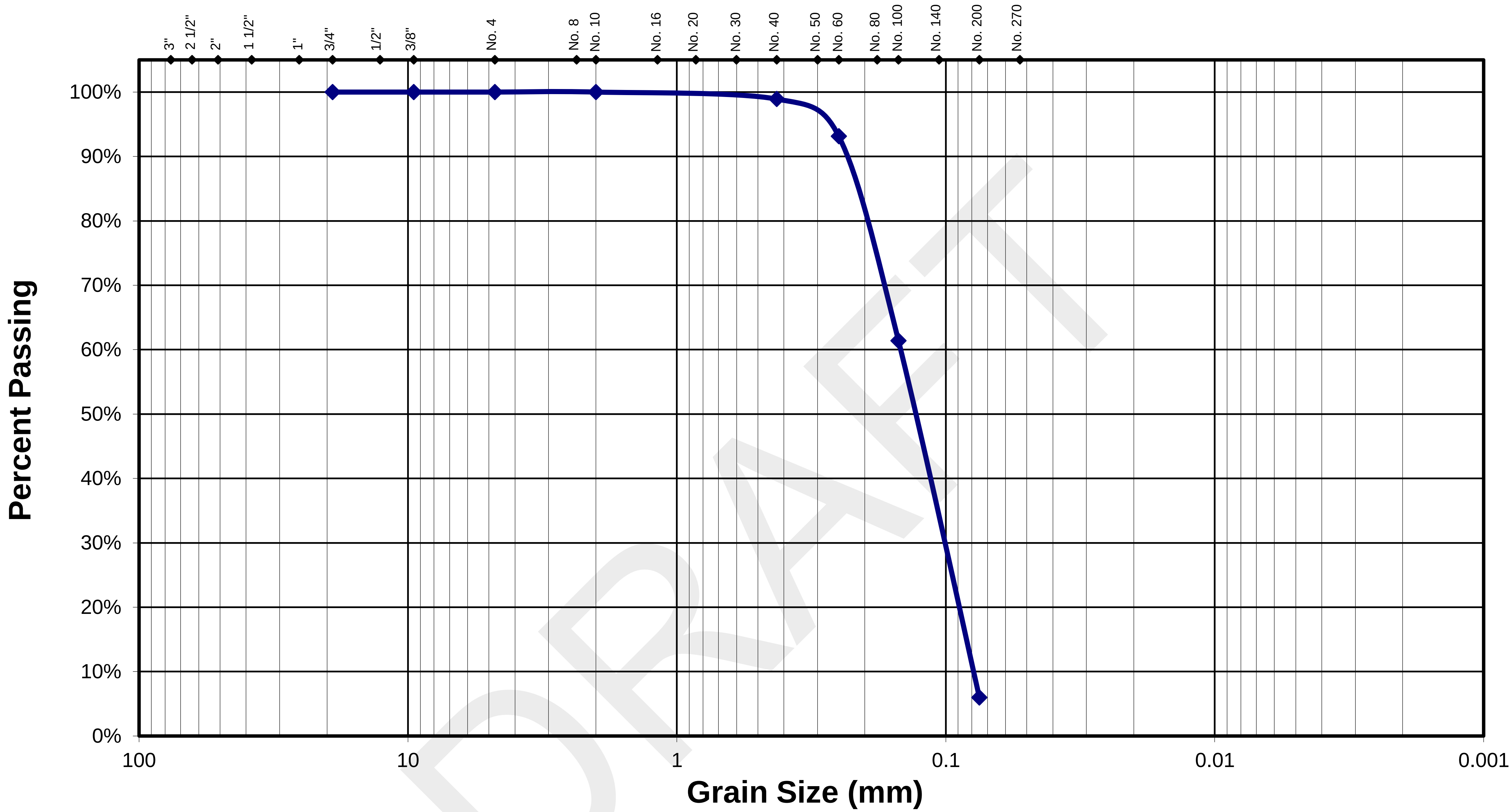
Project Name: Lee County WRF and TF1
Project Location: Lee County, Florida
Client Name: Johnson Engineering

A&A File Number: 20-33-4505

Sample No.: 2 **Sample Location:** HA-W3
Sample Description: SP-SM - Grayish brown slightly silty fine sand
Percent Passing No. 200 Sieve = 5.3% **LL:** Not Tested **PL:** Not Tested **PI:** Not Tested

ARDAMAN & ASSOCIATES, INC.
Geotechnical, Environmental and Materials Consultants

GRAIN SIZE DISTRIBUTION CURVE



GRAVEL	SAND		SILT	CLAY
	Coarse to Medium	Fine		

Project Name: Lee County WRF and TF1

Project Location: Lee County, Florida

Client Name: Johnson Engineering

A&A File Number: 20-33-4505

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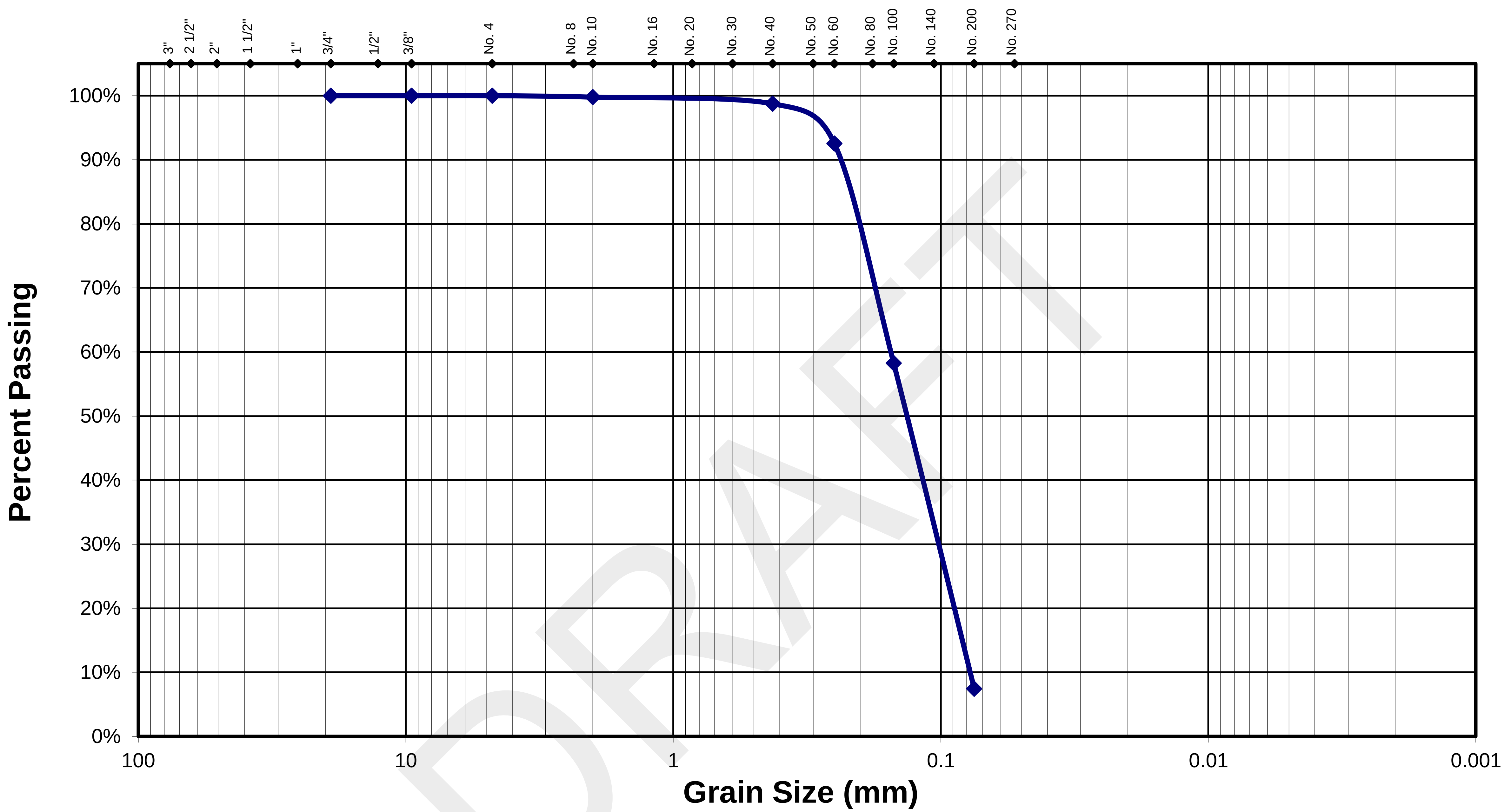
Geotechnical, Environmental and Materials Consultants

Sample No.: 1 **Sample Location:** HA-W4

Sample Description: SP-SM - Light brown slightly silty fine sand

Percent Passing No. 200 Sieve = 6.0% **LL:** Not Tested **PL:** Not Tested **PI:** Not Tested

GRAIN SIZE DISTRIBUTION CURVE



GRAVEL	SAND		SILT	CLAY
	Coarse to Medium	Fine		

Project Name: Lee County WRF and TF1

Project Location: Lee County, Florida

Client Name: Johnson Engineering

A&A File Number: 20-33-4505

Sample No.: 1 **Sample Location:** HA-W5

Sample Description: SP-SM - Grayish brown slightly silty fine sand

Percent Passing No. 200 Sieve = 7.4% **LL:** Not Tested **PL:** Not Tested **PI:** Not Tested

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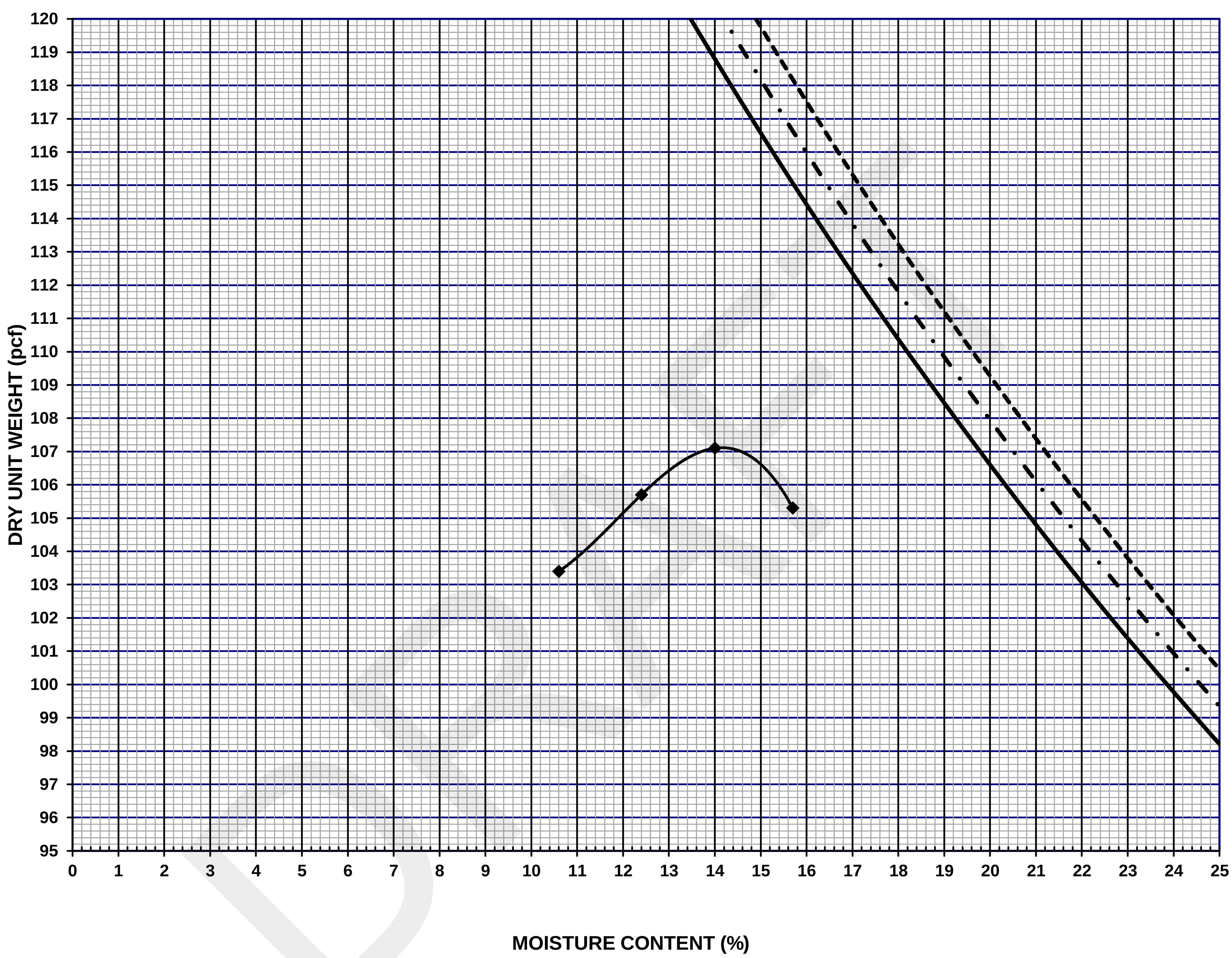
APPENDIX IV

Modified Proctor with Grain-Size Distribution Curve Results

Ardaman & Associates, Inc.
Ardaman & Associates, Inc.
9970 Bavaria Road
Fort Myers, Florida 33913
Phone 239-768-6600
FAX 239-768-0409

REPORT OF MOISTURE-DENSITY RELATIONSHIP

Project Name:	Lee County WRF and TF1	**Date Sampled:**	2/4/20
Project Location:	Lee County, Florida	**Sampled By:**	ZS
File Number:	20-33-4505	**Date Tested:**	2/6/20
Client Name:	Johnson Engineering	**Tested By:**	GW



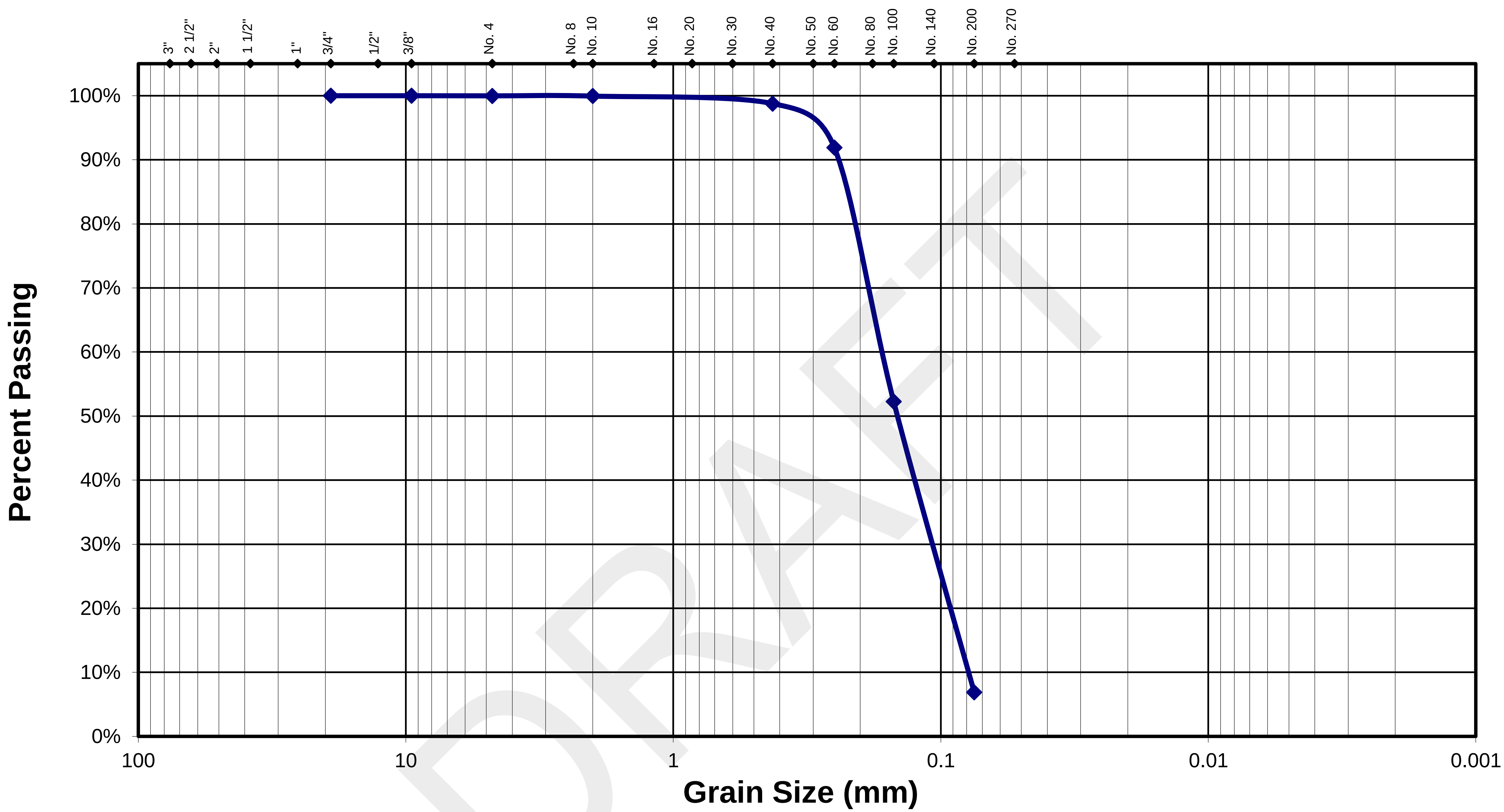
Curves of 100% Saturation for Specific Gravity Equal to:
2.60
2.65
2.70

TEST RESULTS		
Maximum Dry Density (pcf)	Optimum Moisture Content (%)	Fines Passing #200 Sieve (%)
107.1	**14.0**	**6.9**

SAMPLE NUMBER:	MP-W1
TEST METHOD:	ASTM D-1557
SAMPLE DESCRIPTION:	Brown to orange-brown slightly silty fine sand (SP-SM)
SAMPLE LOCATION:	SPT-W1

Ethan H. Drew, P.E.
Florida License No. 88622

GRAIN SIZE DISTRIBUTION CURVE



GRAVEL	SAND		SILT	CLAY
	Coarse to Medium	Fine		

Project Name: Lee County WRF and TF1
Project Location: Lee County, Florida
Client Name: Johnson Engineering

A&A File Number: 20-33-4505

Sample No.: MP-W1 **Sample Location:** SPT-W1
Sample Description: SP-SM - Brown to orange-brown slightly silty fine sand
Percent Passing No. 200 Sieve = 6.9% **LL:** Not Tested **PL:** Not Tested **PI:** Not Tested

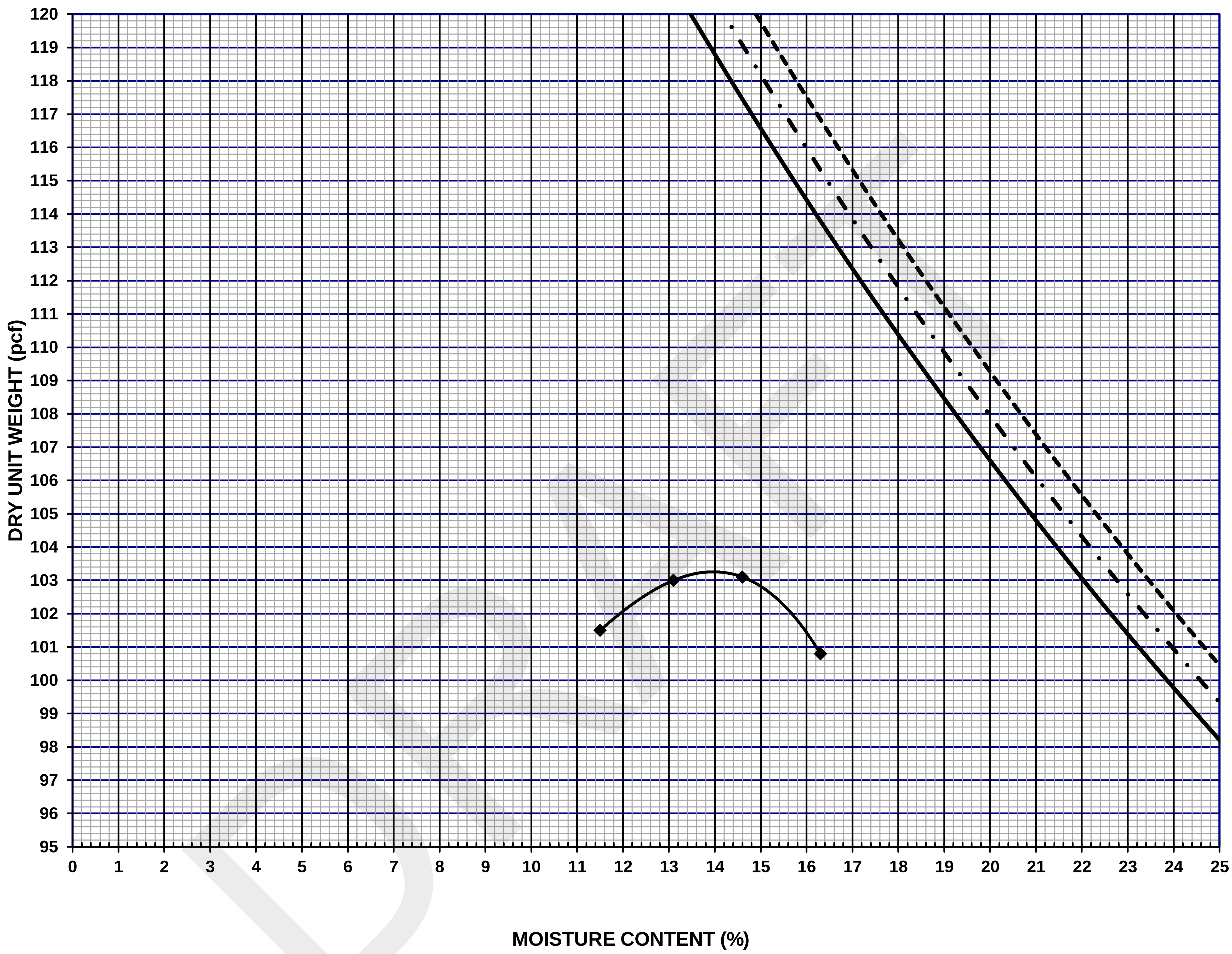
ARDAMAN & ASSOCIATES, INC.
Geotechnical, Environmental and Materials Consultants

Ardaman & Associates, Inc.
Florida Certificate of Authorization No. 00005950

Ardaman & Associates, Inc.
9970 Bavaria Road
Fort Myers, Florida 33913
Phone 239-768-6600
FAX 239-768-0409

REPORT OF MOISTURE-DENSITY RELATIONSHIP

Project Name:	Lee County WRF and TF1	Date Sampled:	2/4/20
Project Location:	Lee County, Florida	Sampled By:	ZS
File Number:	20-33-4505	Date Tested:	2/6/20
Client Name:	Johnson Engineering	Tested By:	GW

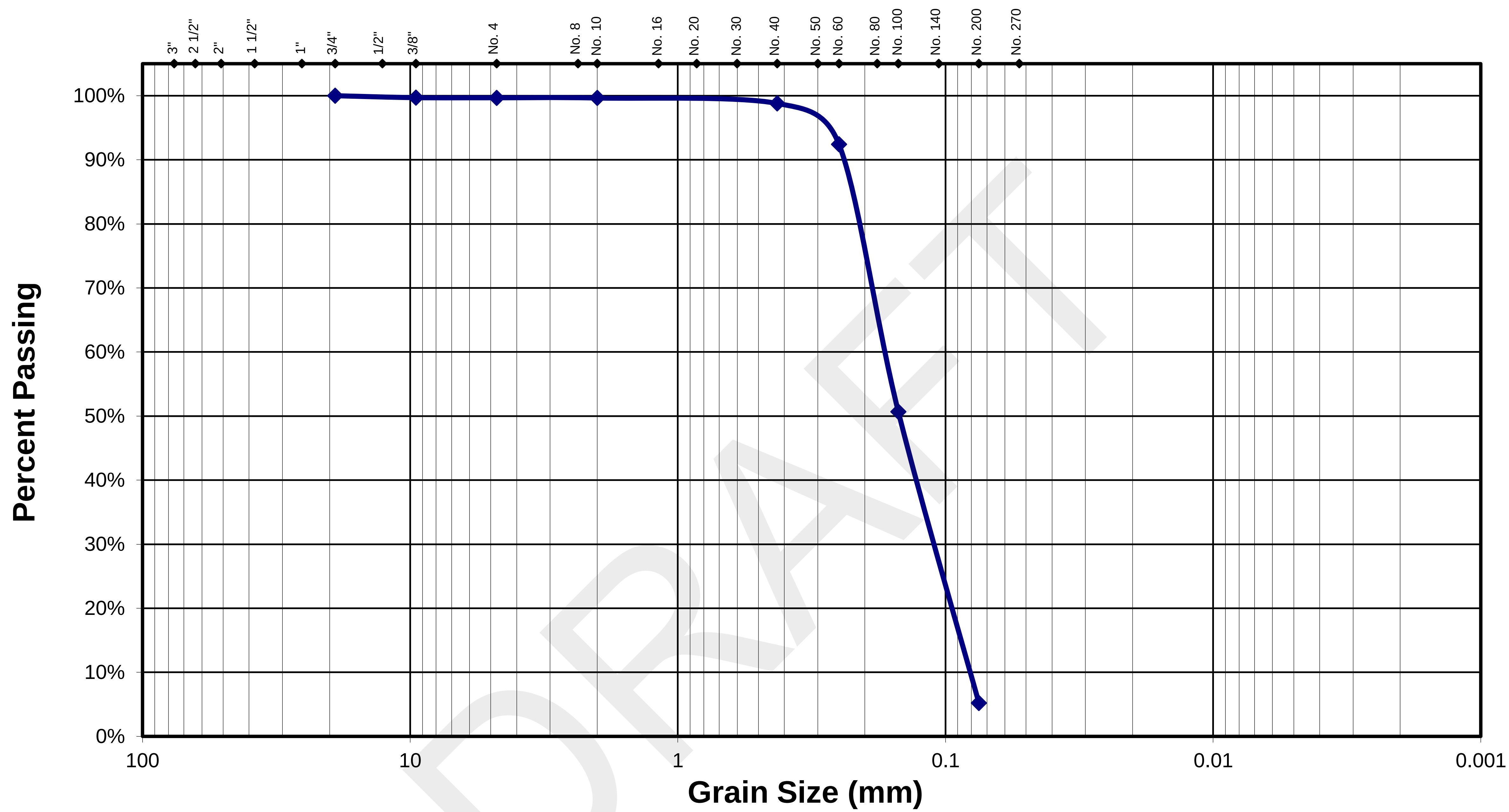


TEST RESULTS		
Maximum Dry Density (pcf)	Optimum Moisture Content (%)	Fines Passing #200 Sieve (%)
103.1	14.6	5.2

SAMPLE NUMBER:	MP-W3
TEST METHOD:	ASTM D-1557
SAMPLE DESCRIPTION:	Gray slightly silty fine sand (SP-SM)
SAMPLE LOCATION:	SPT-W3

Ethan H. Drew, P.E.
Florida License No. 88622

GRAIN SIZE DISTRIBUTION CURVE



GRAVEL	SAND		SILT	CLAY
	Coarse to Medium	Fine		

Project Name: Lee County WRF and TF1
Project Location: Lee County, Florida
Client Name: Johnson Engineering

A&A File Number: 20-33-4505

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Sample No.: MP-W3 **Sample Location:** SPT-W3
Sample Description: SP-SM - Gray slightly silty fine sand
Percent Passing No. 200 Sieve = 5.2% **LL:** Not Tested **PL:** Not Tested **PI:** Not Tested

APPENDIX V

Monitor Well Installation Log – PZ-W3

Ardaman & Associates, Inc.
Geotechnical, Environmental and Materials Consultants

MONITOR WELL INSTALLATION LOG

File No.: 20-33-4505	Client: Johnson Engineering, Inc.	Project: Lee County WRF and TF1	Permit Number:
Installed by:		Date finished: 2/4/2020	WELL No.: PZ-W3
Rob Lockley	Julian Centeno		

Remarks: