



April 24, 2020

Dear Ms. Rebekah Page,

For my team's senior design project, we worked with both First Utility District (FUD) in West Knoxville and E/One to design a low-pressure sewer system for the Choto Peninsula. The existing wastewater management on the peninsula consists almost exclusively of septic systems. These systems inhibit development on the peninsula due to the negative environmental impacts of high densities of septic tanks. In addition, FUD expects the peninsula to develop rapidly in the near future. To alleviate environmental issues and account for expected growth, the team will design sewer connections to all existing roadway infrastructure on the peninsula at pipe sizes that can account for increased development over a 20-year time span. The pipe network will primarily use E/One's low pressure pipe technology. The primary advantage of low-pressure networks is that they can cheaply move over hilly topography that is present on the peninsula. Conventional gravity systems would be cost-prohibitive and require several additional lift stations in this application. Using low-pressure required the design of one new lift station which was followed by a short section of gravity sewer. Finally, a hydraulic and structural redesign of the main lift station on the peninsula, the Choto Road Station, was performed to account for the additional demand generated over a 20-year period. The existing force main and infrastructure leaving the Choto road station was found to be sufficient for projected wastewater flow and was unaltered.

Work on the team was distributed among its five members; my responsibilities included both the estimation of future flows in the design year and the site design of the existing Choto and new Channel Point pump stations. For the estimation of future wastewater flows, I used GIS zoning data from both Knoxville Geographic Information Systems and the City of Farragut's Planning Department to generate maps of the existing and projected zoning for the project area. The existing zones were used to identify the location of business and existing development, such as dense neighborhoods, where additional development was not expected in the next

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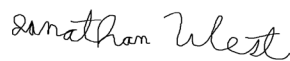
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20 years. The projected zoning map was used to estimate the density of future developments in the areas not identified as “no development expected” from the existing zone map. This data was used to generate the expected number of new dwelling units in the project area. Literature values of wastewater demand per capita and household size were used to estimate the design peak wastewater from new developments. This value was combined with the existing peak to generate the total peak wastewater demand for the Peninsula.

For the site designs, I selected a location for the new wet wells which included selecting the location of the new Channel Point pump station. The criteria used for this selection include geometric constraints, head requirements, and presence of existing structures. After the selection of location, I designed the connections to the new wet well including the influent pipe and manholes. AutoCAD was used to create estimations for the excavation and fill required for each pump stations including the necessary excavation to install the new wet wells. I performed turning radius checks to verify that maintenance trucks could operate in the station and drive sufficiently close to the new wet wells. In addition, I also designed a simple Portland cement concrete slab for the stations and a stormwater drain for the Channel Point Station. This project has increased my confidence in working on large-scale engineering projects, especially the initial stages where the design path forward is not yet clear. I think that this experience will help me greatly once I begin working since I have a better understanding of the steps of an engineering project and how to coordinate with an engineering team.

Thank you,



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Enclosed:
Choto Peninsula Low-Pressure Sewer Design Report and Appendix

Choto Peninsula Low-Pressure Sewer Design

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Senior Design Course Experience
CE399S & CE400
Fall 2019 – Spring 2020



Acknowledgements

Tennessee Valley Wastewater Solutions would like to express our gratitude by acknowledging all parties that contributed to the development of this project. This project was made possible by contributions of the following people:

Mark McKinney - First Utility District

Jill Davis - Environmental One Corp

Josh Cravins – Wascon Inc.

Christina Heise - First Utility District

Dr. Jennifer Retherford – Senior Lecturer at the University of Tennessee, Knoxville CEE Department

Disclaimer

The following report and engineering plans were developed by students as part of a senior design project for the University of Tennessee. All analysis and design in the report and engineering plans were performed in accordance with applicable standards but should not be taken as construction drawings on account of the students not being supervised directly by a licensed engineer. The University of Tennessee and the individuals involved in this project assume no liability for services, construction, or designs attached in this report. **All work must be reviewed and independently confirmed in entirety before implementation of any recommendation contained within the student project documentation.** The documents of this project should not be considered for construction.

Team Vision and Community Awareness Statement

All proposals included in this report have been developed with an awareness of the needs for constituents and users of infrastructure. Land developers, landowners, and the First Utility District of Knoxville were considered as the primary users of the proposed design plan. The mission of Tennessee Valley Wastewater Solutions (TVWA) is to meet the National Academy of Engineering's (NAE) "Restore and Improve Urban Infrastructure" Challenge by expanding sustainable, low-pressure wastewater systems. We also believe that developing our engineering leadership capabilities is an essential prerequisite to enacting these principles gained from the NAE challenges to improve wastewater system infrastructure both within and beyond Tennessee Valley Wastewater Solutions. TVWA strives to provide designs for safe and reliable infrastructure that will encourage future land development and increase the quality of existing developments.



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1.0 Introduction

The engineering design and documentation associated with the development of a low-pressure sewer system on the Choto Peninsula in Knoxville, Tennessee will be discussed in this report. The First Utility District of Knoxville (FUD) currently provides gravity and low-pressure sewer services to properties on the northern portion of the peninsula. Small existing residential properties on the southern portion of the peninsula are not serviced by sewer systems and primarily rely on septic tanks. FUD expects the peninsula to become fully developed within the next fifteen years. The construction of sewer systems to service existing properties and upgrading existing infrastructure to accommodate increased flows associated with future developments will encourage developers to incorporate sewer into new developments. Due to the highly variable topography of the peninsula, Environmental One Corporation's (E/One) low pressure sewer systems will be the primary sewer systems utilized in areas where gravity sewers are not feasible.

1.1 Team Information

The team that conducted the work on this project consisted of five members shown in Figure 1.1. The team members are currently completing their Civil and Environmental Engineering undergraduate studies at the University of Tennessee, Knoxville with a tentative graduation date of May 2020. Table 1.1 provides contact information and roles for each team member. Table 1.2 provides the information and roles for each of the clients and mentors.



Fig. 1.1. TVWS Team, From Left to Right: Ryan Livesey, Tyler Galyon, Jonathan West, Alexander Gothard, Joey Bresson

Table 1.1. Team Information

Team Member	Roles	Contact Information
Jonathan West	Team Leader/Civil Site Development for Pump Station Redesign	jwest59@vols.utk.edu (865) 919-4396
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Table 1.2. Client and Mentor Information

Mentor or Client	Organization	Role
Josh Cravins	Wascon Inc.	Sewer component distribution and pricing.
Jill Davis	Environmental One Corp	Mentor for low pressure sewer design.
Christina Heise	First Utility District	Client and mentor for pump station design.
Mark McKinney	First Utility District	Client and mentor for pump station design.
Dr. Jennifer Retherford	University of Tennessee - Knoxville	CE 400 course professor

1.2 Project Overview

Future developments on the Choto Peninsula in Knoxville, Tennessee will require existing infrastructure to be evaluated for future sewage demand capacity. The peninsula currently utilizes three pump stations that deliver water from the basin to the FUD water resources recovery facility (WRRF). As land is developed on the southern portion of the peninsula, sewage demand will increase, and existing infrastructure must accommodate the increased flows. Existing properties

will also incorporate low-pressure sewers in place of existing septic tanks to encourage land development in the surrounding areas.

1.3 Existing Site Conditions

The existing site consists of gravity sewer and low-pressure systems in the Northern sections of the peninsula. The northern portion of the peninsula is zoned for high-density residential, while the southern portion of the peninsula has small areas of low-density residential and undeveloped agricultural. These low-density residential areas consist of about 1 house per acre serviced by septic tanks. The Holder Branch pump station currently services the southern portion of the peninsula and is currently oversized to accommodate future flows. All pump stations on the peninsula currently feed into the Choto Road pump station which will need to be evaluated for future flow capacity. Water is discharged from the Choto Road pump station through a force main that follows Northshore Drive and discharges into the WRRF. A topographic map of the peninsula is provided in Figure 1.2.



Figure 1.2. Topographic Map of Choto Peninsula

1.4 Technical Scope of Work

FUD has contracted TVWS to develop a sanitary sewer system master plan for the Choto Peninsula in Knoxville, Tennessee. Existing infrastructure will be evaluated for flow demands based on population increases caused by future developments on the southern portion of the peninsula. A low-pressure sewer system will service existing unsewered properties in low-density residential zones, and future properties in undeveloped agricultural zones. The Choto Road pump station and force main will require an analysis of future flow capacity, and necessary upgrades will be determined.

2.0 Water Demand Estimate

A water demand estimate was performed for the Choto Basin for future population estimates to determine appropriate system upgrades. Current and projected zoning information was utilized to estimate the number of dwelling units in the project area. Water demand was estimated by projecting future flows for existing unsewered properties, in addition to future developments. The total projected water demand was calculated by summing the existing water demand with future estimates.

A project boundary was established based on the projected basin boundary expansions found in First Utility District's "Choto Pump Station Condition Report" (Figure 2.1) (2019). The boundary labeled "Choto" represents the principle area that flows directly into the Choto pump station. This area was not considered for estimating demand growth because it was either largely developed, or not expected to be developed based on the guidance of FUD. The Harvey pump station basin and most of the Holder Branch pump station basin were also excluded from growth projections due to existing development. Some agricultural area inside the Holder Branch basin was included. Two potential expansions to the Choto pump basin have been identified by FUD and are outlined in orange. These are the primary areas of expected demand growth.

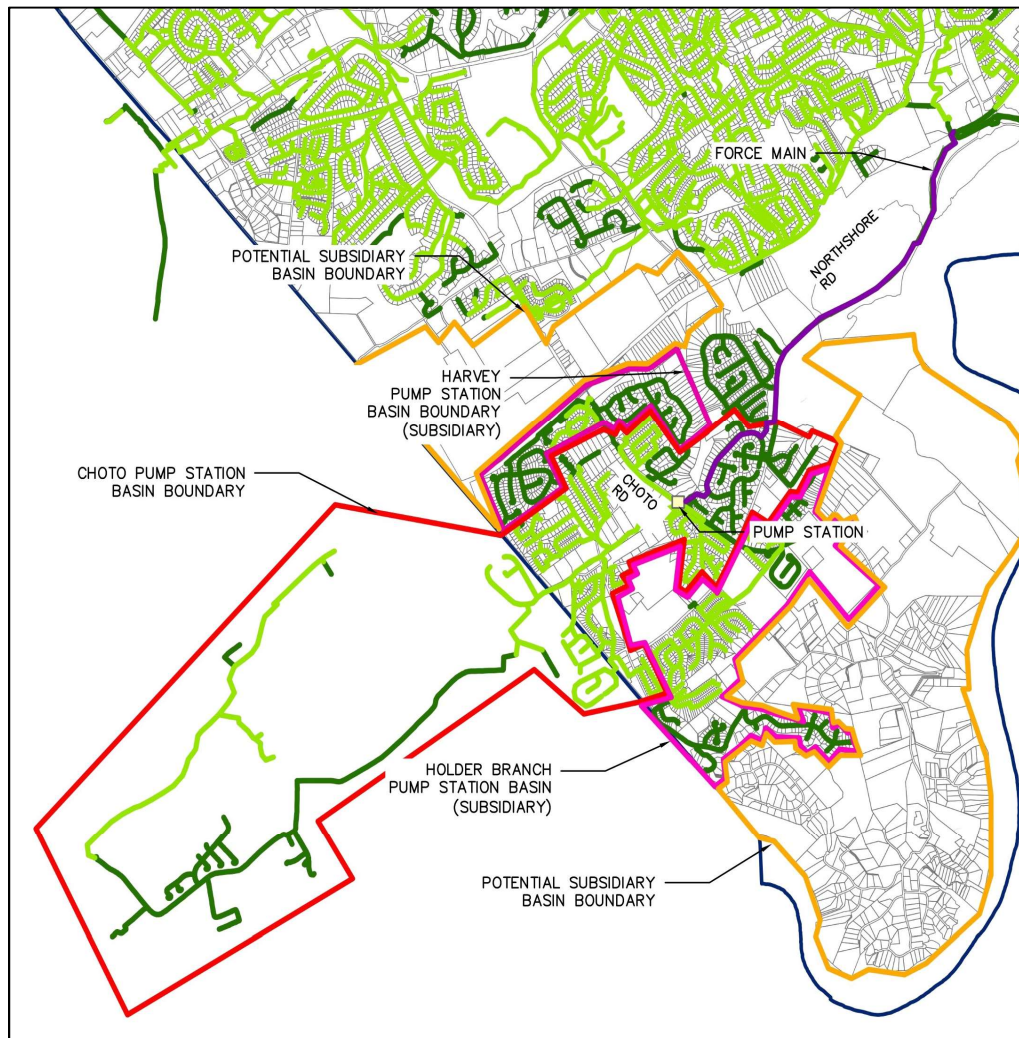


Fig. 2.1. Choto Peninsula Existing and Potential Pump Station Basins (FUD Report)

Existing zoning information was obtained from Knoxville Geographic Information Services (KGIS) and was used as a supplement to the projected zoning information. (Figure 2.2). The project area is primarily zoned as Agricultural. The existing zone information was used to verify that the planned zone density had not already been exceeded by the current density such as some areas currently zoned Planned Residential. Finally, other zones such as Rural Commercial, Neighborhood Commercial, and General Business provide essential information about the location of commercial businesses, which were considered separately for wastewater demand.

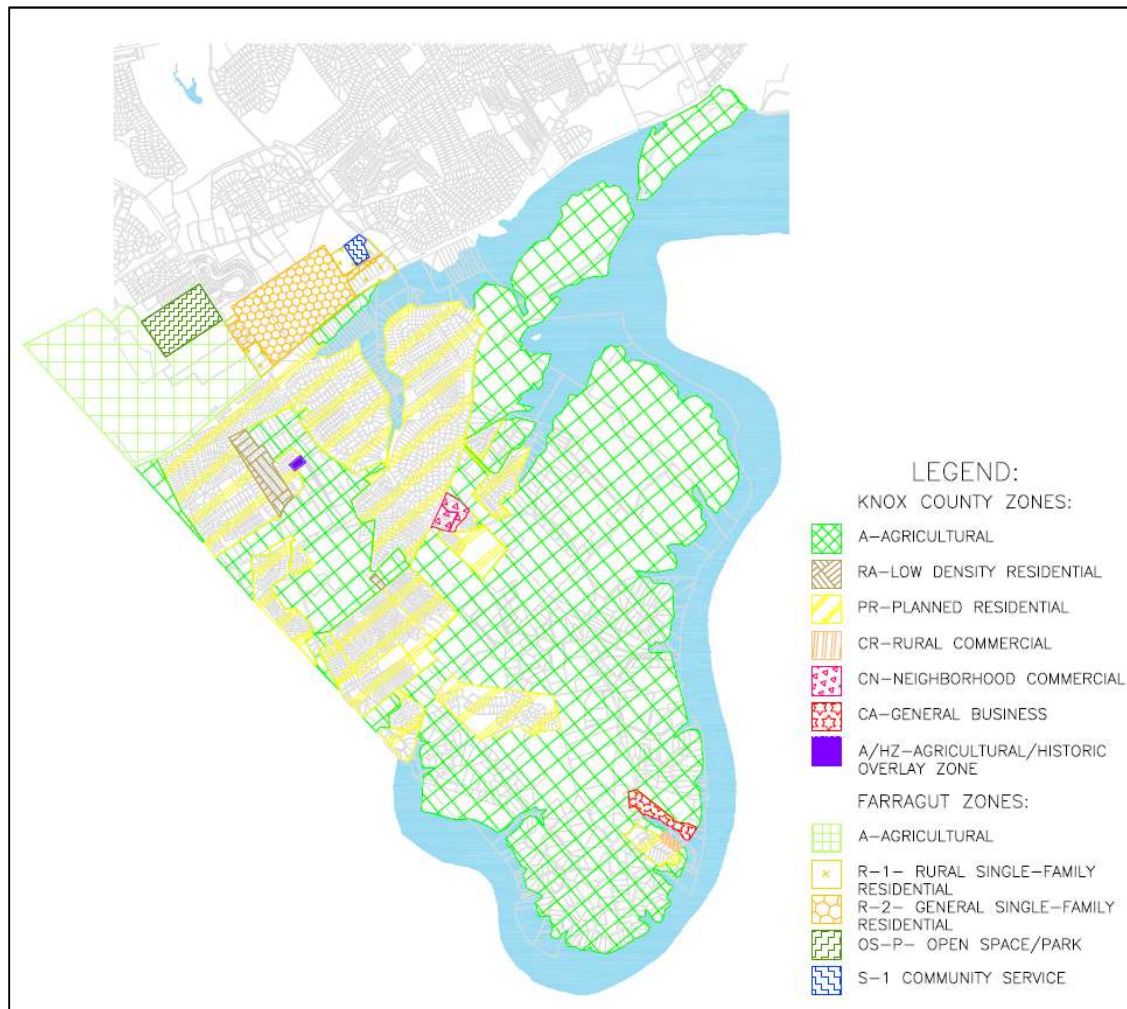


Fig. 2.2. Choto Peninsula Zoning of Existing Conditions

Projected zoning information was used to estimate the quantity of dwelling units that will likely be built in an area (Figure 2.3). This information came from two sources. For areas within Knox County, information was obtained from Knox County’s sector plan from KGIS similar to the existing zoning information. The sector plan is updated to reflect the expected development in the next 15 years, which matches the design year of this project (Knoxville- Knox County Planning 2020). For areas within Farragut’s city limits, Farragut’s “Future Land Use Map” was used as a reference which has a design year of 2025 (2019). This map was used despite the time difference because no planning maps closer to the design year of 2035 were available. This is not expected to significantly influence the accuracy of the demand estimate since most of the area within the potential expanded basin is classified as Open Space/Cluster Residential which requires limitations on building and makes zone changes unlikely (Town of Farragut 2012).

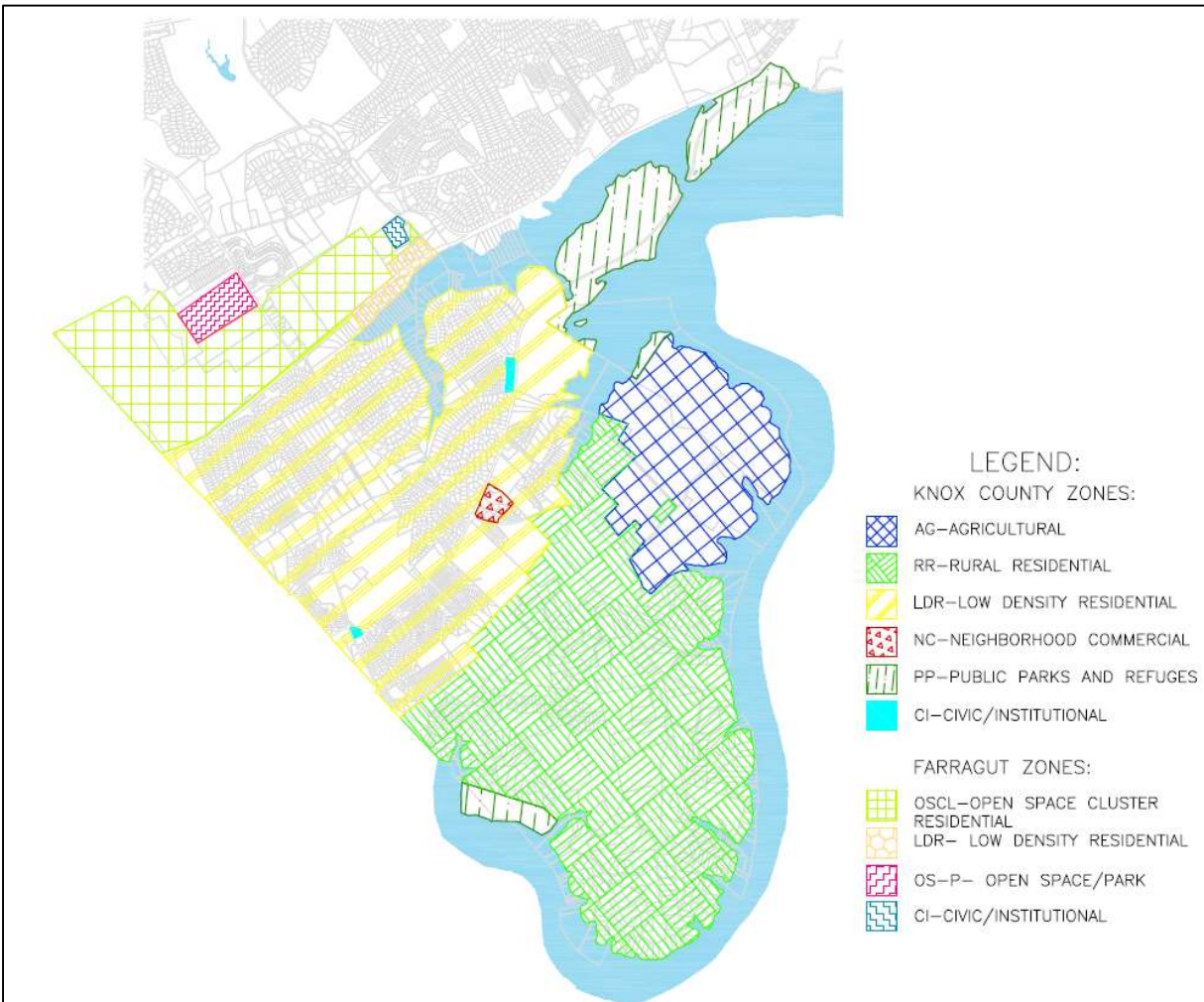


Fig. 2.3. Choto Peninsula Projected Zoning

Most of the projected zones already have associated dwelling unit density recommendation or maximums that were used for dwelling unit estimation. Some zones such as Parks and Recreation in Farragut or Public Parks and Refuges indicate no growth will occur, so these areas were excluded from demand estimations. Other projected zones such as Civic/Institutional indicate the location of a planned or currently existing community buildings which, like commercial establishments, were identified and considered separately.

One approximately 25-acre area zoned as Open Space in Farragut's future land use map was replaced with the Open Space/Cluster Residential due to being insignificant in size and having a shape difficult to properly map. Please see **Appendix A** for a map showing the location of this area. This change applies to the map and calculations for water demand. Any errors produced by this scenario are insignificant compared to the size of the demand estimation.

The number of existing unsewered dwelling units inside the project area was determined by counting houses in developed areas on satellite maps. Any area that was not expected to develop beyond its current usage in the next 15 years was included in the counting. This included any

unsewered area currently zoned as Planned Residential since these areas already had houses or parcel lines spaced to build houses. In addition, a scan of the project area with Google Maps obtained additional areas sufficiently developed that were outside of Planned Residential areas. Engineering judgement was used instead of an exact criterion for the required density to be counted. Areas that only had sparsely placed houses were not included since these areas could be developed into a neighborhood.

For areas not sufficiently dense to include in existing house counts, potential dwelling units were estimated based on the land area and associated zoning. First the area to consider for development was determined. As previously mentioned, any area inside of the Harvey basin, primary Choto basin, and the Holder Branch basin (excluding three agricultural plots) were excluded. In addition, any area included in the existing houses count was excluded. Any commercial area was excluded because of existing establishments. Finally, a section of the peninsula was excluded due to expected difficulties building on the steep topography. The area excluded can be found in **Appendix A**. Any area inside of the project area that was not excluded by any of these criteria was included in the estimation calculation. The area by zone type was found with an overlay of the projected zone map over the expected growth area (Table 2.1 and Figure 2.4).

Table 2.1. Area of Expected Development Per Zone.

	Planned Zone	Area (acre)
Knox County	Parks and Refuges	49
	Rural Residential	745
	Agricultural	487
	Low Density Residential	176
Farragut	Farragut Park	44
	Farragut Civic/Institutional	8
	Farragut OS/CL	497
	Total	2006

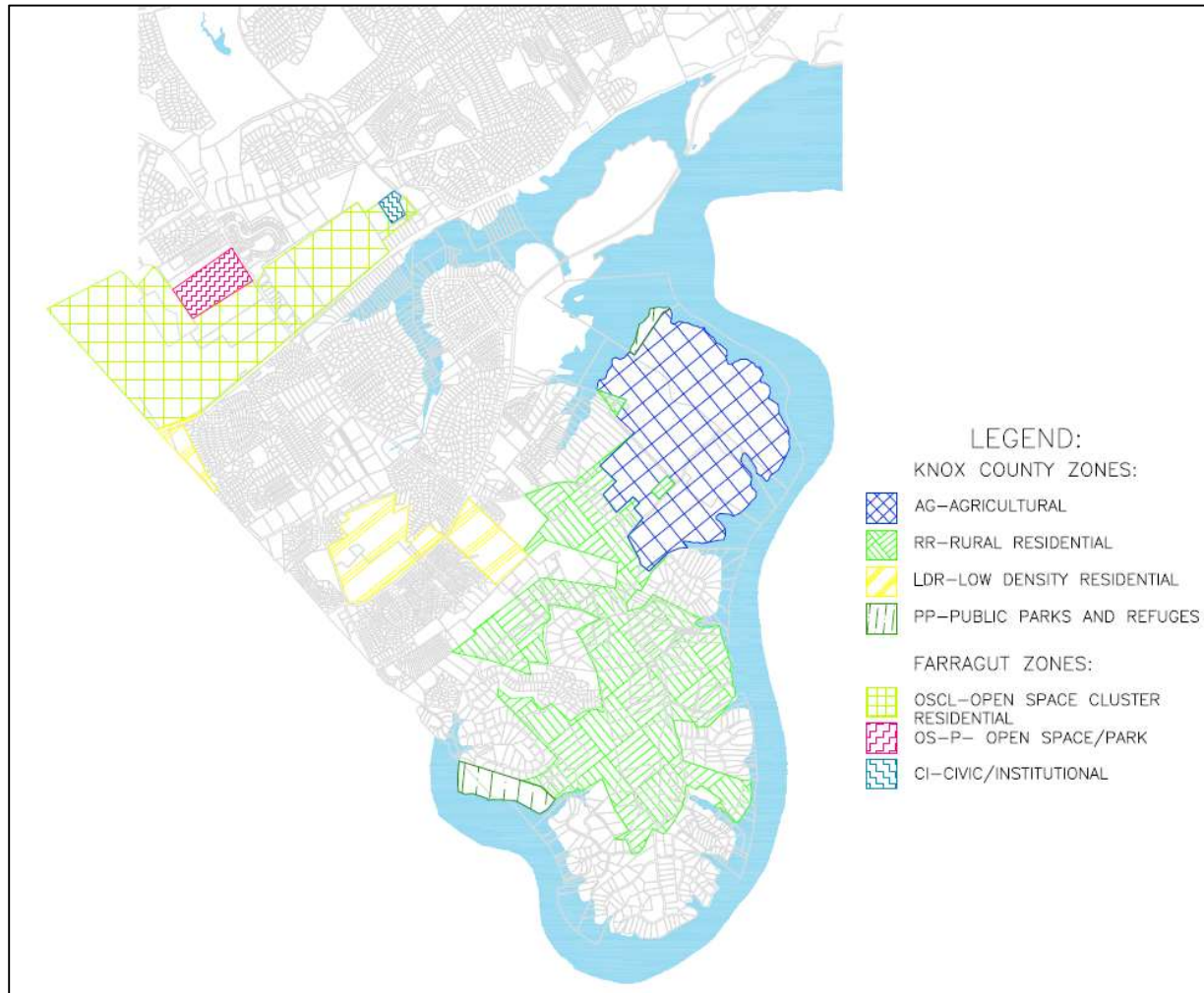


Fig. 2.4. Zoning Used in Dwelling Unit Estimation Calculations

Each zone had an expected dwelling unit density selected based on the description of the planned zone (Table 2.2). For the Knox County planned zones, this number was the recommend max dwelling units per acre. Based on the experience of professional developing engineer Russ Rackley, a reduction to 60% of the maximum dwelling units was taken as the expected fully built-out density. A fully built density was based on FUD's expectation for the area. The only zone that did not follow this procedure is the Agricultural planned zone since it did not have a recommend maximum density. Instead, this zone was assigned a density roughly similar to that of the existing development inside of agricultural zones on the Choto Peninsula. Planned zones in Farragut give a range of housing density allowed (Town of Farragut 2012). For these zones, a value close to the middle of the range was selected. Calculations for dwelling unit estimation can be found in **Appendix A**.

Table 2.2. Assigned Dwelling Unit Density per Zone

	Planned Zone	Expected Dwelling Units per Acre
Knox County	Public Parks and Refuges	0
	Agricultural	0.3
	Rural Residential	0.6
	Low Density Residential	3
Farragut	Parks and Recreation	0
	Civic/Institutional	0
	Open Space/Cluster Residential	3

Projected wastewater demand was calculated by combining the existing demand with the projected new demand generated from the dwelling unit estimation. A summary of assumptions and calculated demand is provided in Table 2.3. The existing demand was obtained from FUD based on current flows into the Choto pump station. The projected new demand was taken as a function of the number of estimated households. Existing businesses and institutions were considered negligible due to the small quantity which were not already sewered. The population was assumed to be linearly related with the number of dwelling units. The design value for average household size was based on the “Residential End Uses of Water” but was rounded up to account for possible local variation (Deoreo et. al 2016). E/One’s standard estimation for wastewater demand per capita was selected as the design value. The End Uses of Water survey verifies the accuracy of E/One’s assumption with its similar indoor water use value (Deoreo et. al 2016). A typical relationship between peak hour and average flow was assumed from literature (Hammer and Viessman 1993). Calculations for wastewater demand can be found in **Appendix B**.

Table 2.3. Wastewater Demand Calculations Summary

Parameter	Design Value
Existing Flow (gpd)	342000
New Dwelling Units	2614.4
New Connections to Existing Dwelling Units	351
Household Size	3
Wastewater Production per Capita	65
Peak Hour: Average Hour Ratio	3
Projected Average Hour Wastewater Flow (gpd)	920370
Projected Peak Hour Wastewater Flow (gpd)	2761110

3.0 Design of Low-Pressure System

The design of the proposed low-pressure system on the peninsula was determined based on existing properties and estimated future development using the Design Assistant provided by E/One. The system was designed using zone length, maximum main line elevation, and minimum pump elevation for each zone along the low-pressure layout to calculate minimum flow velocity, pipe size, and total required head. Flow velocities were referenced with current Tennessee Department of Environment and Conservation (TDEC) standards to reduce pipe erosion and sediment deposition.

Designing for the existing demand of the peninsula was based on the sectioning of the area in relation to the major roadways that currently exist on the peninsula. There are three main roadways that currently exist on the peninsula. These are Early Road, Warrior Trail, and Choto Road. These main roadways have other roadways branching off that help create these three major sections. The reason the peninsula needs to be sectioned in this way is so that the E/One Design Assistant can have proper inputs for design. Figure 3.1 shows the determination of these three sections based on the three major roadways running along the area of the peninsula. With these areas, the design assistant then requires the sectioning of different zones within these major areas spanning the length of the roadway in that section. These zones are determined by both the length of the roadway in that section and the presence of an intersection. The E/One Design Assistant recommends lengths of 1,000 feet per zone before an intersection. If an intersection is present, the Design Assistant recommends ending the zone at that point.

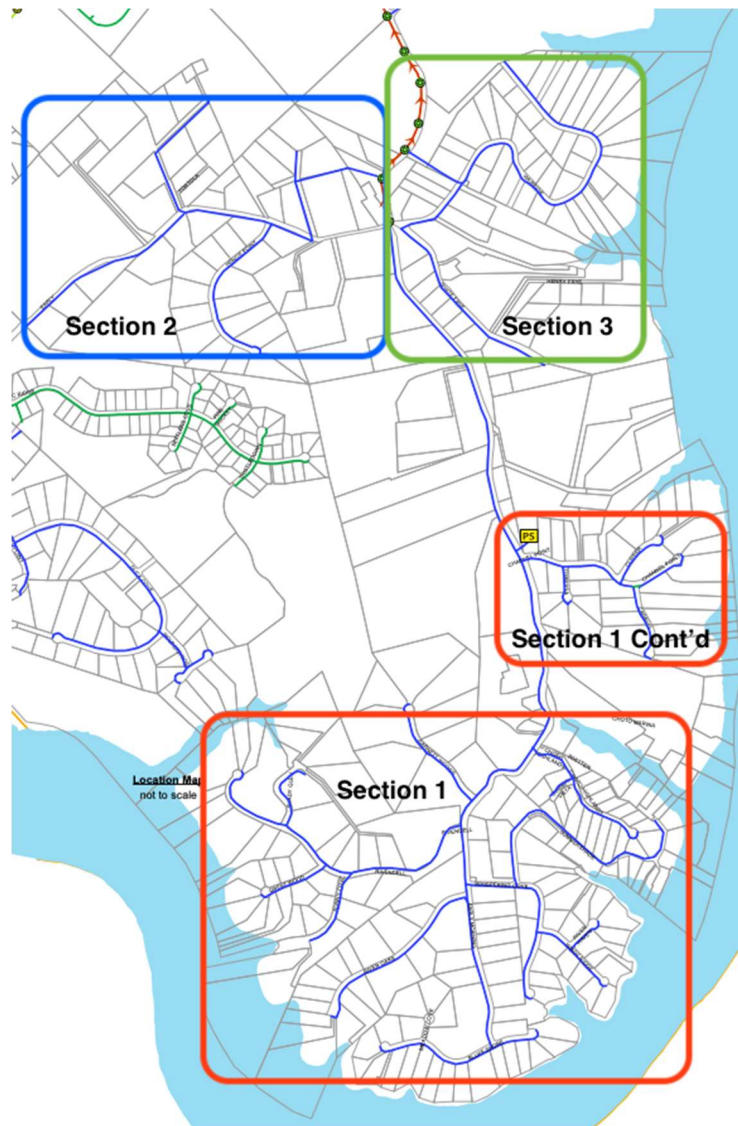


Fig. 3.1. Sectioned Areas of Peninsula for E/One Design Assistant

The zones in the E/One Design Assistant require information specific to each zone to compute the low-pressure design. The user input data consists of the length of the sub-main in that zone, the number of houses with accompanying E/One Simplex Grinder Pumps in that zone, maximum flow per pump (11 gpm), capacity of each pump (200 gpd), minimum pump elevation, and maximum elevation for the sub-main in that zone. The length of the sub-main for each zone was determined using the measurement tool in Civil3D on a FUD supplied map that was to scale to the original. The number of houses was determined by the actual existing house counts along the roadways. This information was determined using Google Maps. The maximum flow per pump and the capacity of each pump were both supplied by FUD. The minimum pump elevation and the maximum elevation for the sub-main in the zone were both determined using KGIS topographical maps (KGIS 2010). With this information input into the system, the E/One Design Assistant

computes the information for the low-pressure system in each zone. This information consists of pipe size, maximum velocity, maximum flow, total dynamic head, friction loss, and other hydraulic information for the pressurized network in that area. The design assistant will also red flag any zones that have too high of dynamic head or have either too high or too low maximum velocity. This is due to the elevation differences of the minimum pump elevation and the maximum elevation of the sub-main in that zone. It can also be due to an unsatisfactory pipe size for that section. Shortening of the zones to allow for a more minor total dynamic head will correct this issue. Also, manipulating the pipe size in that zone can correct the issue of maximum velocity. Once all zones are acceptable, the design assistant can create reports for the finished low-pressure system in each section. The reports for the three low-pressure sections for existing properties on the peninsula can be found in E/One Software Reports 1-3 in **Appendix C**. The hand calculations for the design assistant can also be found in **Appendix C**.

Once the low-pressure system servicing existing properties was developed, it was determined that an additional pump station was required to overcome static head caused by the excessive elevation difference between Channel Point Drive and Early Road. This can be seen in the E/One Software Report 5 – Page 1 in **Appendix C**. The total dynamic head for zones 4, 7, 10, 13, and 15 in the report all have a total dynamic head requirement higher than 185 ft. The low-pressure sewer network would not be able to pump wastewater over the hill due to excessive head requirements. By adding a pump station, the system has enough power to overcome static head requirements. This can be seen in the E/One Software Report 1 in **Appendix C**.

A low-pressure system was also designed for areas without existing roadways or houses by considering estimated future demand for that area. This future demand was proposed for the section of the peninsula shown in Figure 3.2. This area in the Northeast portion of the peninsula is currently undeveloped. It was assumed that there would be future development in that area based on the developability of the land and the proximity to water. To design for future development of this section, the low-pressure system along the one existing roadway, Prater Lane, was designed using the E/One Design Assistant. The system was designed using the future demand calculated using the existing zoning in that area. The future demand would discharge into Prater Lane which would then discharge into the gravity main leading to the Holder Branch pump station. It was determined that the system in that area would utilize a 5 in. pipe and accommodate an average of 3,200 GPD. This can be seen in the E/One Software Report 4 in **Appendix C**.

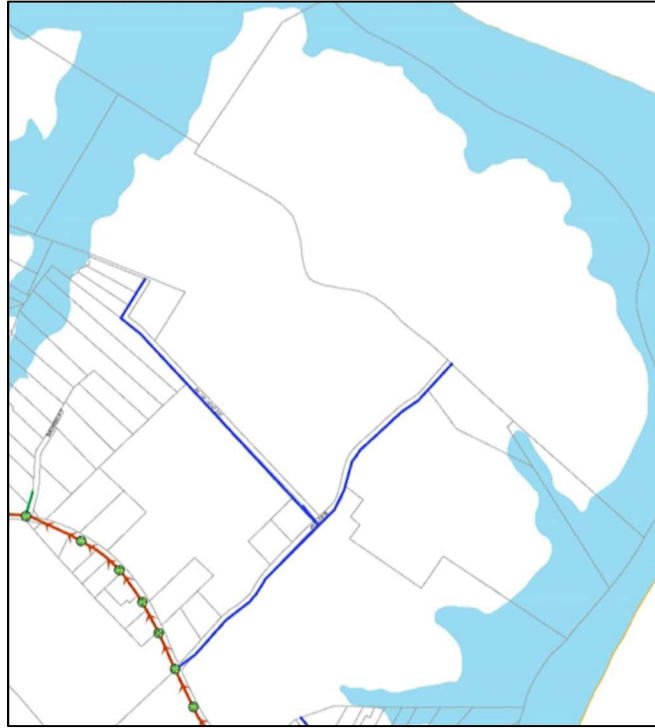


Fig. 3.2. Northeast Undeveloped Area of Peninsula for Future Low-Pressure Demand

4.0 Pump Station Evaluation and Proposal

The proposed wastewater system for the Choto Peninsula will require upgrades to existing pump stations, and the construction of a new pump station to appropriately accommodate future demand caused by new developments. Choto Peninsula is currently divided into three main basins, each served by its respective pump station. The northern portion of the peninsula is currently developed with no expected future development and is served by the Harvey Road pump station. The existing developments on the southern portion of the peninsula are currently served by the Holder Branch pump station, which will require upgrades due to the increased sewage flow caused by future developments on the peninsula. The proposed low-pressure sewer system on the southern portion of the peninsula will require the construction of a new pump station located at the intersection of Choto Road and Channel Point Drive. This proposed pump station will be referred to as Channel Point pump station. The three pump stations will discharge all sewage flows into the Choto Road pump station, which will pump directly to the FUD WRRF. The Choto Road pump station has been completely redesigned to accommodate future sewage demand. The three pump stations are shown in Figure 4.1.

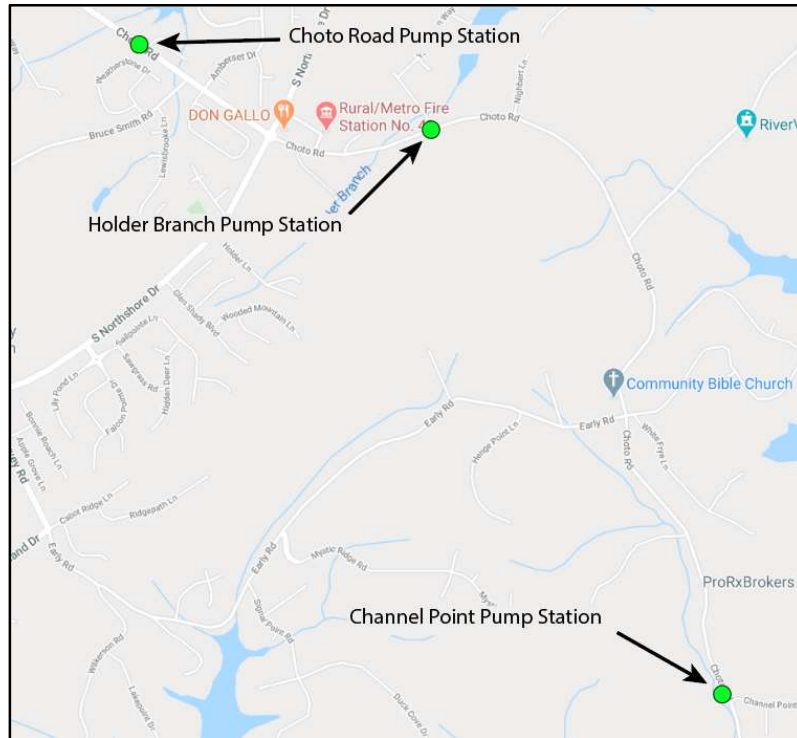


Fig. 4.1. Pump Station Locations

Pump stations were evaluated based on future flow rates with hydraulic calculations that considered the existing hardware at each station. The Hazen-Williams equation was used to determine frictional losses throughout the force main, and the Darcy-Weisbach equation for minor losses was used to calculate in-station losses. Total required pump head was determined by using Bernoulli's equation. Whenever it was determined that pumps needed to be replaced or chosen for a new pump station, Gorman-Rupp pumps were favored over other companies because of their wide application throughout the industry. Gorman-Rupp is used by FUD and other local utilities. In addition, Gorman-Rupp has intuitive software for pump selection.

4.1 Holder Branch Pump Station

The existing Holder Branch pump station currently serves a small basin on the western portion of the peninsula and will require upgrades to accommodate future flow demands. A summary of the pump station characteristics is provided in Table 4.1. The station was designed larger than required to accommodate future developments on the southern portion of the peninsula, in addition to providing an extra factor of safety due to the proximity of a nearby stream. The station was evaluated by performing hydraulic calculations that utilized existing pump station equipment with future flow velocities. The existing capacity can accommodate the increased flows caused by future developments, so the only upgrade required is the replacement of the existing pumps with two new pumps that can accommodate the increased flow demand. The proposed pumps have the same size inlet and outlet diameter as the existing pumps, so all in-station piping should be reused

if not showing significant signs of wear. The existing 12 in. force main exiting the station will maintain flow velocities within the 2-15 ft/s standards required by TDEC.

Table 4.1. Holder Branch Pump Station Summary

Parameter	Existing	Proposed
Number of Pumps	2	2
Manufacturer	Gorman Rupp	Gorman Rupp
Model	T8A3-B	T8A3S-B
Design Point	750 gpm @ 70 ft	1,475 gpm @ 72.5 ft
Speed (rpm)	1,785	1,250
Wet well Diameter (ft)	8	8
Wet well Depth (ft)	18.5	18.5
Wet well Capacity (gal)	6,956	6,956
HP	50	50
Voltage	480	480
Force main size/material	12 in.	12 in.
Force main length	2,630'	2,630'
Average Daily Flow (gpm)	78	402
Peak Hourly Flow (gpm)	460	1,440

4.2 Channel Point Pump Station

The installation of a low-pressure sewer on the southern portion of the peninsula will require the construction of a new pump station due to the excessive slope of Choto Road. The Channel Point station will be constructed on the corner of Choto Road and Channel Point Drive. The pump station will discharge into a force main that will follow Choto Road until the intersection of Choto Road and Early Road. The force main will discharge into a manhole at the intersection of Choto Road and Early Road, and a proposed gravity sewer will discharge into the Holder Branch Pump Station. The length of the force main was selected based on the distance from the proposed station to the highest point on Choto Road. The force main could have continued, but it was determined that gravity would be a better choice since the entire route from the high point on Choto Road to Holder Branch pump station is downhill. Additional low-pressure sewers located adjacent to the force main on Choto Road will discharge directly into the force main. The force main can be tapped to accommodate the discharge of other pressurized sewers that might be constructed in the future.

The need for a pump station at the intersection of Choto Road and Channel Point Drive was determined based on the excessive static head requirement from Channel Point Drive to Early Road. The proposed low-pressure sewer network does not produce enough head to overcome the elevation change, so the Channel Point pump station is required to deliver sewage to the top of the hill. The station was designed based on the static head requirement, and the dynamic head requirement calculated based on the force main and in-station piping. The proposed pump station dimensions and pump specifications are provided in Table 4.2. The station will utilize two

submersible pumps in a cylindrical pre-cast concrete wet well. The pumps will follow a typical lead-lag cycle, and odor control measurements might be required due to the potential for a moderate detention time caused by low flow demand. The decision to have a high capacity for the station is due to the proximity of a nearby stream. The need for odor control shall be evaluated after construction of the pump station.

Table 4.2. Channel Point Pump Station Summary

Parameter	Proposed
Number of Pumps	2
Manufacturer	Gorman-Rupp
Model	SF4D-1
Design Point (1 Pump)	370 gpm @132 ft
Impeller Size (in)	11
Wet well diameter (ft)	8
Wet well Depth (ft)	15
Wet well Capacity (gal)	797
HP	18.5
Force Main Size/Material	8 in. PVC
Force Main Length (ft)	3,750
Average Daily Flow (gpm)	48
Peak Hourly Flow (gpm)	143

4.3 Choto Road Pump Station

The existing Choto Road pump station is the point in which all sewage from the Choto Peninsula basin discharges before being pumped to the FUD WRRF, and significant upgrades will be required to accommodate future sewage demand. The existing station is undersized for the proposed sewage demands and utilizes an outdated wet well dry pit configuration. The proposed pump station utilizes a submerged pump wet well system with a singular wet well. Due to the large area requirement of the proposed wet well, a pre-cast concrete wet well will not be utilized. Instead, a rectangular cast-in-place wet well will be constructed to reduce costs and logistics associated with manufacturing and transporting an excessively large cylindrical wet well. A summary of the existing and proposed Choto Road pump station is provided in Table 4.3.

Pump station calculations were performed by considering the incremental development of the peninsula in order to reduce detention times and reduce energy consumption. The number of pumps for the station was increased from 2 to 3 to accommodate the increased flows, while also minimizing the retention time of the station. Since development of the peninsula will occur in stages, flow demands will remain low and increase as construction is completed. The station can utilize one pump running to accommodate low flows, and two pumps running can accommodate peak flow rates associated with the full development of the peninsula. This configuration allows the station to remain efficient during the initial stages of development.

Table 4.3. Choto Road. Pump Station Existing Conditions

Parameter	Existing	Proposed
Number of Pumps	2	3
Manufacturer	Fairbanks Morse	Gorman-Rupp
Model	6-B5443	SF6D-X
Design Point (1 Pump)	1,550 gpm @ 55 ft	1,450 gpm @ 53 ft
Design Point (2 Pumps)	--	1,950 gpm @ 66 ft
Impeller Size (in)	--	9.75
Wet well Diameter (ft)	8	--
Wet well Length (ft)	--	16
Wet well Width (ft)	--	16
Wet well Depth (ft)	20.9	13
Wet well Capacity (gal)	23,574	23,766
HP	50	25
Force Main Size/ Material	16"	16 in. DIP
Force Main Length (ft)	15,022'	15,022
Average Daily Flow (gpm)	240	640
Peak Hourly Flow (gpm)	720	1,918

The Choto Road pump station currently discharges into a force main that follows Northshore Drive and transports sewage to the WRRF and does not require replacement or upgrades. The current force main is oversized for existing flows and will accommodate the increased flows associated with the complete development of the peninsula. Future flows fall within the range required by TDEC. The force main is buried underneath the roadway and incorporates a bridge crossing, so the construction of a replacement force main would be costly and negatively impact traffic flows in the area. If the existing force main does not exhibit signs of excessive inflow and infiltration, then the existing force main should continue to be utilized by the proposed pump station.

5.0 Gravity Sewer Design

Due to a change in the topography at a point along Choto Road in the southern part of the peninsula, the incorporation of a 5,800-foot gravity sewer will be necessary between multiple sections of low-pressure sewer and the Holder Branch pump station. Manhole locations were selected by marking points along the road at a chosen spacing increment suggested by TDEC's sewer design guide. The elevations at each manhole along Choto Rd., were retrieved from KGIS topography data. These elevations were used to determine the slope of each pipe section between the manholes and calculate the required design parameters including pipe diameter, flow, and velocity. The appropriate pipe diameter was chosen by determining the smallest diameter that produces a minimum flow velocity. While the gravity sewer system can contain pipes of varying diameters, there cannot be a decrease in pipe diameter in the direction of the flow.

5.1 Pipe and Manhole Layout.

The gravity system was designed in sixteen 360-foot sections along a strip of Choto Road, with manholes at each junction. A manhole spacing of 360 feet on center was determined to be the most efficient distance to maintain a path along the road. The gravity sewer begins at the intersection of Choto and Early Roads and ends at the newly proposed Holder Branch Pump Station. The justification for the use of a gravity system along this portion of Choto Road, is that from this local peak, water would flow downhill for just over a mile. Gravity sewers are typically preferred when possible due to easier maintenance and overall cost. In accordance to the TDEC Sewage Design Criteria, sewer pipes are to be laid on crushed stoned bedding in a trench backfilled with crushed stone up to 1 ft below the ground surface.

5.2 Hydraulic Design.

Assumptions were made to begin the design process, including those for Manning's roughness coefficient, depth to crown, and infiltration. Manning's equation, shown in **Appendix E** requires the use of a roughness coefficient, n . It is typically assumed in design of gravity sewers to be 0.13 referring to reinforced concrete pipe (RCP), but due to the need for pipe diameters of less than 21 inches. An n -value of 0.011 was chosen to be the roughness coefficient for PVC pipes. The table showing the iterated flow velocities can be found in **Appendix E**.

Using KGIS topography data, the elevation at each manhole was determined to calculate pipe slope. The slope was taken as the difference in elevations between manholes divided by 360 feet. Using Manning's equation, the flow and slope were considered for each 360-foot section to determine the necessary pipe diameters. To do this, an iterative process was required where the input was the pipe diameter and the comparative output was velocity. Solids deposition is controlled by a minimum flow velocity in the pipes. The velocity is known as "self-cleaning velocity" as it is the minimum flow velocity that will remove and prevent solids buildup from deposition or side wall cohesion.

The minimum velocity calculated from a selected diameter is to be 2.0 feet per second. At any point where a selected diameter gave a velocity less than 2.0 feet per second, a smaller diameter was required. It was determined that the first eight sections of pipe would be 8-in, followed by a 12-in pipe, then seven sections of 15-in pipes. The decision to choose these diameters is based on the smallest diameter possible that delivers no less than 2.0 feet per second flow. If all pipe sections were to be of a single diameter, the smallest diameter that would allow sufficient flow velocities would be 15-in. It is more cost-efficient to choose the smallest pipe possible for each individual section. The gravity sewer design must account for four inflows at four different manholes. A 39,000 gallons per day (gpd) flow will be the initial flow at the beginning of the gravity system at manhole MH-0. At manholes MH-1, MH-2, and MH-8, additional flows of 12,800, 11,800, and 34,000 gpd will feed into the system. Tables showing the manhole elevations, slopes, and flows can be found in **Appendix E**.

5.3 Manhole Sizing.

The manholes were sized according to the procedure shown in **Appendix E**, Figure 5.2. All pipe lengths are well over the given minimum L. The manholes were selected to be 48 inches in diameter with a 22-in surface opening. Manholes are to be precast concrete units and dropped in place.

6.0 Civil Site Development for Pump Stations

A site plan was created for both the existing Choto station and the new Channel Point pump station to accommodate for the proposed wet wells and pumps. The stations' sites shared similar design considerations. Both stations were evaluated for the optimal location to install the new wet wells. For the Choto station, this included replacement of existing influent lines and manholes. For the Channel Point station, this included selection of the site location and shape since there was no existing FUD property in the required region. The selected site layouts were evaluated to verify that maintenance trucks could properly turn and reach the wet wells. To cover the new paved area in the site plans, a simple Portland cement concrete slab was designed. The earthwork required for both stations was also estimated. In addition to these shared geometric and construction considerations, the design of a small storm drain at the Channel Point station was required for proper drainage.

A location inside of the existing property line which avoids damage to existing equipment was selected for the new wet well at the Choto pump station. The wet well will be placed just outside of the existing station and will have the slab, fence, and grading extended to match the new wet well's area. An alternative location for the new wet well was inside of the existing slab but was rejected due to concerns with fitting maintenance trucks in the fenced area and to avoid unnecessary excavation of the existing slab (Figure 6.1). Since the new wet well location is separated from the existing wet well, construction can occur without disturbing the existing equipment, maintaining wastewater flow during construction. Avoiding the existing equipment provides the additional advantage that certain equipment which is still sufficient for the needs of the proposed wet well, such as the bioxide tank and chemical scrubber units, can be reused after the addition of new connections to the wet well.

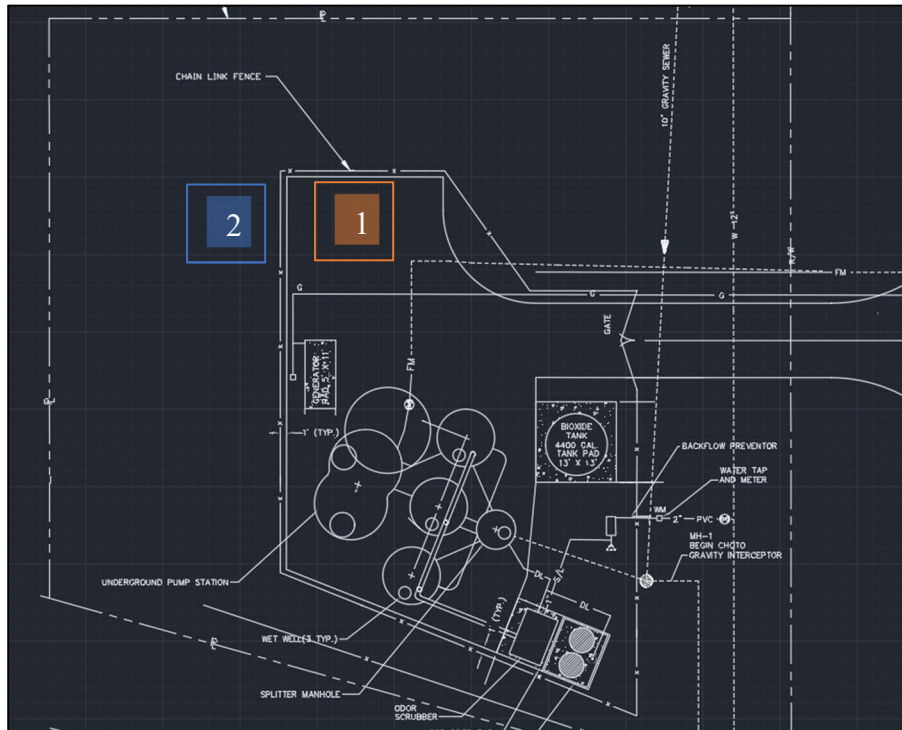


Fig. 6.1. Choto Pump Station Wet Well Location Alternatives.

Since the proposed wet well and the influent sewer pipes are on opposite sides of the existing wet well, the influent pipes were redirected. Choto pump station has two influent sewer lines that were previously combined at a manhole near the eastern side of the existing wet well. Using the original manhole to feed wastewater into the new wet well was not possible since the new connecting pipe would be oriented through the existing wet well. To correct this, the southern force main will be dropped into gravity sewer and extended north. The extension pipe will intersect with the existing gravity sewer at the location of a second new manhole. After combining flows, the sewer will be directed west into a final manhole and then dropped into the wet well. This design around the slab prevents both unnecessary earthwork due to the increased elevation and the need to replace most of the slab. The grade and diameter of the new sewer lines were set to match the existing influent line. Since the influent sewer lines were adequate for the project's expected future demand, the existing pipe parameters should be sufficient. This assumption was verified for grade, which was higher than the TDEC minimum, and pipe diameter which was able to carry the maximum peak flow for the pump station as shown in **Appendix F** (TDEC 2016). The material of pipes was selected to be ductile iron to match the existing influent pipe. The manhole diameters were selected based on the Tennessee Department of Transportation's (TDOT) manhole design criteria (TDOT "D-MH"). The drops from influent to effluent pipes for each manhole were kept minimal to ensure that the final influent pipe into the wet well was above the required high-water alarm level. A check was performed in Civil3D to verify that no pipes were higher than the TDEC minimum depth of 2.5 ft (TDEC 2016). Unlike the influent pipes, the effluent force main's

proximity to the proposed wet well and orientation allow a simple connection directly to the existing force main. Table 6.1 displays a summary of important variables for the new influent pipe and manholes.

Table 6.1. Choto Pump Station Influent Pipe and Manhole Design Summary

Parameter	Design Value
Pipe Grade	0.44%
Pipe Diameter (in.)	20
Pipe Material	Ductile Iron
Choto Station Design Flow (gpm)	1918
Pipe Capacity Flow (gpm)	3784
Manhole Diameters (ft)	4-5
Manhole Depths (ft)	7.5-9.5

The Channel Point pump station location was selected to both minimize impact on existing development and the amount of earthwork required. Head requirements mandated a pump station within a certain proximity (Figure 6.2). Two sites were considered for this location. Site location 1 was on the eastern side of the road. This site was advantageous because it was relatively flat, already cleared of trees, and near the exact point that the low pressure in the pipes required a pump. Ultimately, location 1 was not chosen due to concerns with community resistance as the site had a neighborhood-entrance monument and was in the driveway of one household. Site location 2 was chosen instead. The area's topography is not as level or clear, but it has no development and is still close to the required point based on head.

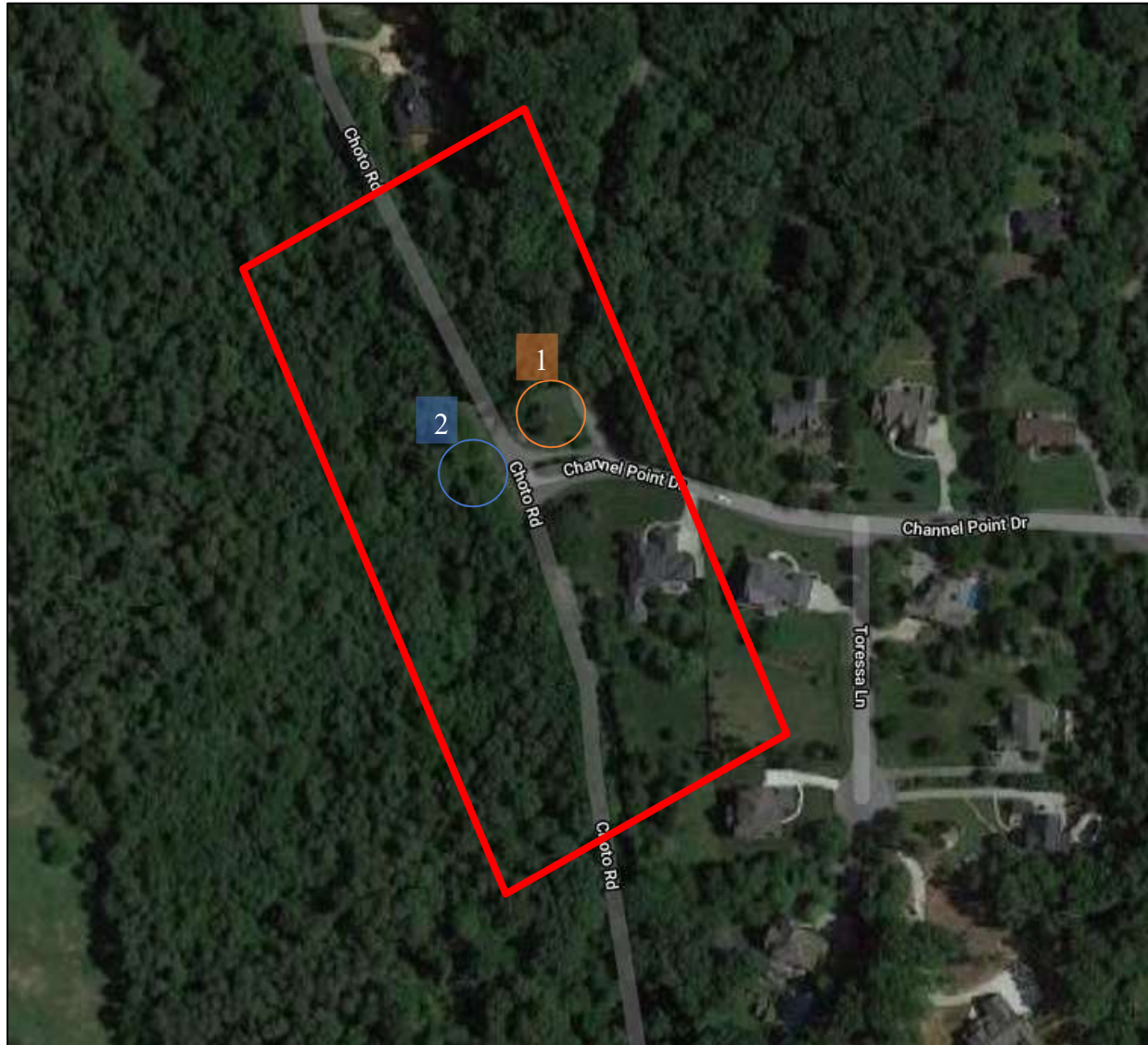


Fig. 6.2. Channel Point Pump Station Location Alternatives.

The station shape was selected to provide a comfortable turning radius for maintenance trucks. Alternative station shapes were considered that would allow for complete reversal of a maintenance truck within the site but were not selected due to the large expansion in necessary site area needed. Outside of this shape a fence and 2:1 slope will be built. The property purchase requirement for this plan is approximately a 98-foot by 62-foot rectangular shape with an area of 0.14 acres.

The slab for new pavement in the stations was designed following ACI's Guide for the Design and Construction of Concrete Parking Lots. This guide was selected as suitable since parking lots share a similar purpose of temporarily holding heavy vehicles. Since a full geotechnical study was not available, a fine grain type subgrade was assumed based on soil maps for the area (USDA 2020). Before construction, soil samples should be gathered to confirm this

assumption. The pavement was selected to be the minimum recommended depth to minimize cost. This selection is justified due to the rarity of traffic into the pump station. The calculations for the slab design may be found in **Appendix F**. The final design requests 4.5 in. of Portland cement concrete on top of 4.5 in. of granular aggregate.

6.1 Turning Radius Check

The designs of both pump stations were checked to verify that an appropriate turning radius for maintenance trucks was provided. The manufacturer's datasheet for the FUD maintenance trucks provides the turning radius data used in this evaluation (Western Star 2018). All calculations assume that the driver will either make multiple three-point turns, reverse into the station, or reverse out of the station. Designing the station for space for simple U-turns would not be feasible based on economic constraints. The method used to verify turning radius was plotting a circle with the minimum turning radius in AutoCAD. If this circle could be placed without causing collisions or running off of the slab, then the minimum turn requirement was considered satisfied. Similar calculations with circles of various radiuses were performed to find the optimal turn radius which brings the truck closest to the wet well. Clearances were also calculated as a measure of required precision of the turn. This calculation was based on the range in feet at which the turn could be started and be completed successfully. All calculations account for both the width of the tires and the overhang of the maintenance truck while turning. Both pump stations were found to have adequate allowances for turning (Table 6.2). Diagrams and calculations for expected turns can be found in **Appendix F**.

Table 6.2. Permitted Turning Radius Summary

Station	Min. Turn Possible	Horizontal Clearance for Min. turn (ft)	Range of Possible Turns (Wall to Wall radius ft)	Optimal Turn radius (Wall to Wall Radius ft)	Optimal Turn Horizontal Clearance (Wall to Wall Radius ft)
Choto	Yes	16.6	193	242	8.1
Channel Point	Yes	12.2	13.0	Same as min	Same as min

6.2 Earthwork

Earthwork quantities were estimated for the two sites and found to mostly require fill (Table 6.3). The method of estimation was comparison of two surfaces generated from KGIS and proposed contour lines in Civil3D. Due to the linear approximation of the Civil3D software for elevations, less cut may be required at the Choto pump station than reported. The Civil3D results did not

include excavation for the wet wells. Shoring will be used for the Choto wet well which requires no sloping (Section 8). For Channel Point a 3/4:1 slope was permitted since the soil in the area is a Class A loam (USDA 2020, OSHA 2020, Virginia Tech 2020). Earthwork Volume calculations for the wet wells can be found in **Appendix F**. Figures 6.3 and 6.4 show heat maps for the sites where darker shades represent more required fill and lighter shades represent less fill or more for the Choto station.

Table 6.3. Earthwork Summary

Station	Cut (yd ³)	Fill (yd ³)
Choto	8.6	67.1
Choto with Wet Well	195.8	67.1
Channel Point	0	556.5
Channel Point with Wet Well	720.4	556.5

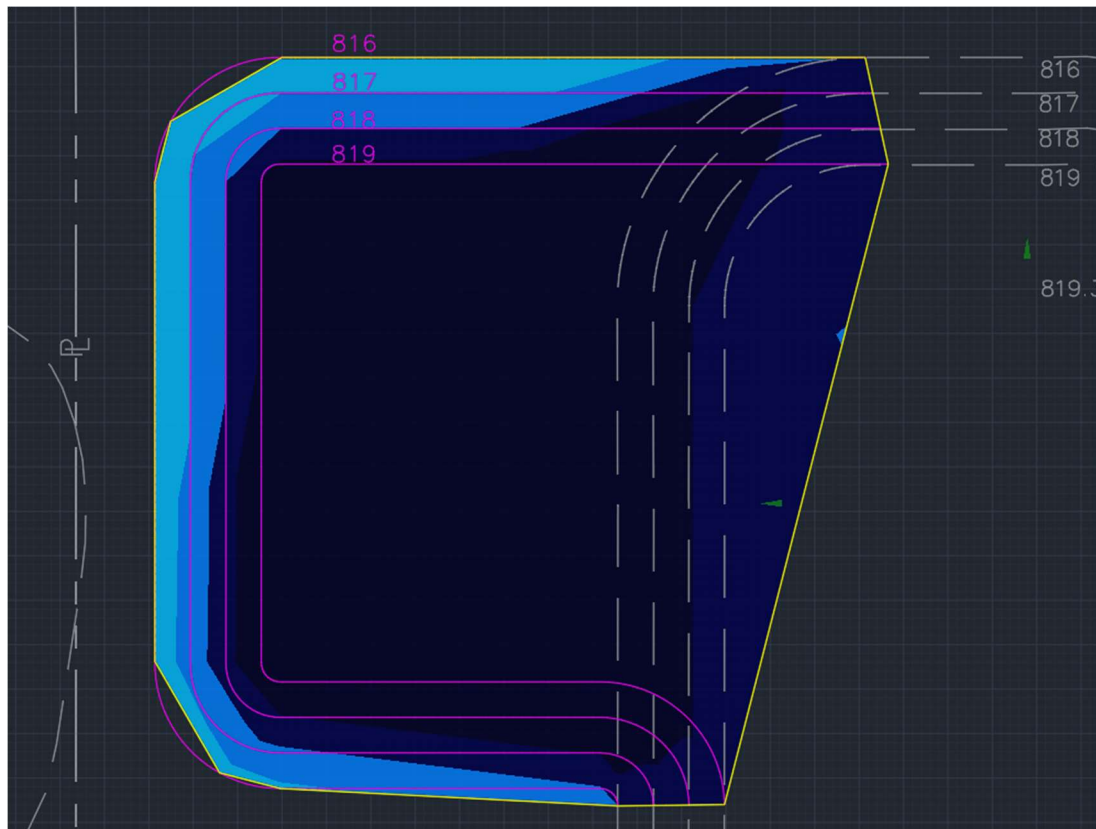


Fig. 6.3. Choto Required Cut and Fill Heat Map.

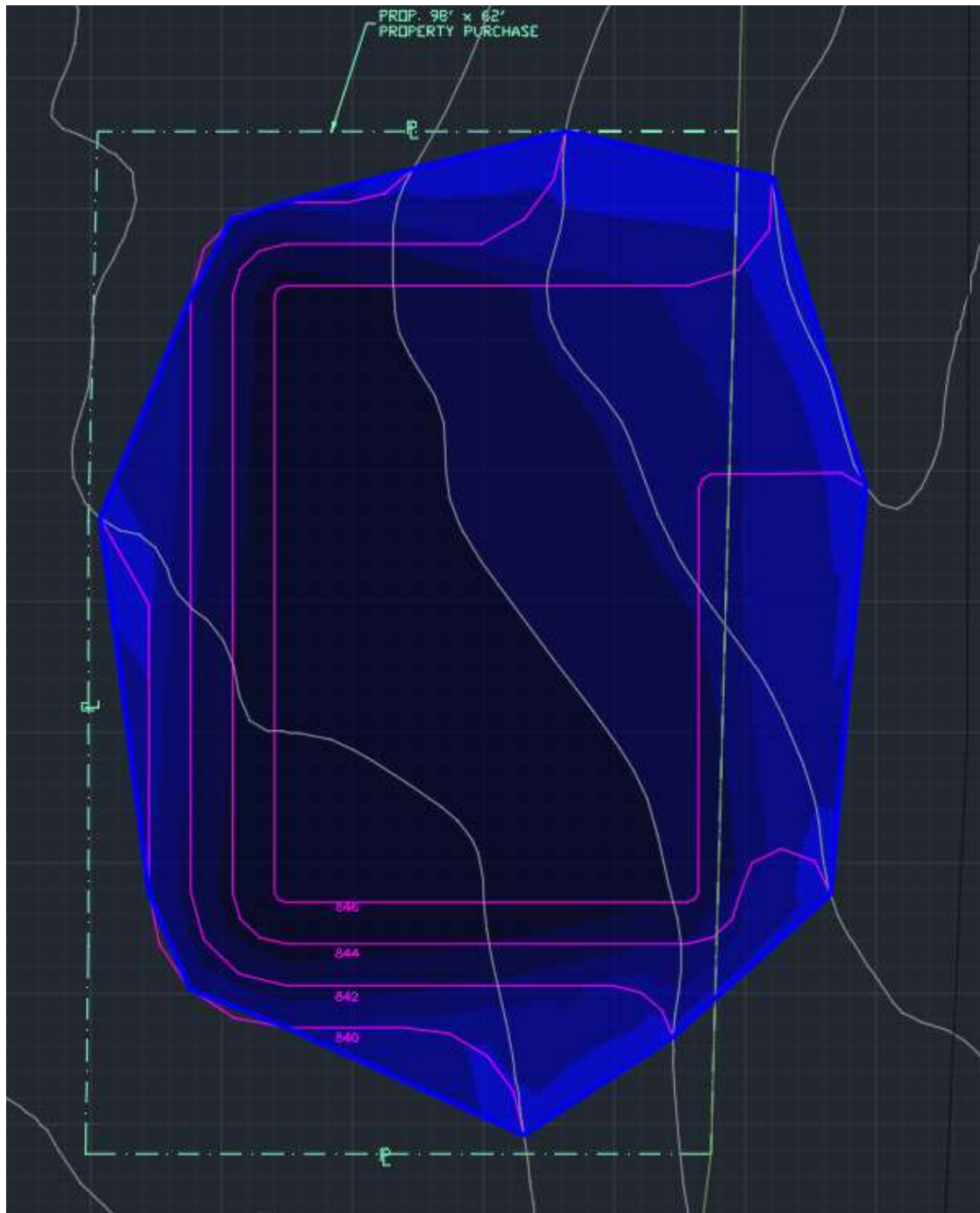


Fig. 6.4. Channel Point Required Fill Heat Map.

6.3 Channel Point Storm Drain

Due to the concern of standing water flooding the driveway to the Channel Point pump station, a small storm drain was designed to move water off the pavement. The slope of the driveway was set to 5% across the driveway into the storm drain. The procedure listed in TDOT's Drainage Manual was used to size the storm drain (TDOT *Drainage*). Civil3D was used to trace all points

which fed into the storm drain. For the purposes of determining the watershed area, any point landing on the driveway or on the immediately adjacent segment of road up to the center line would be considered as flowing into the storm drain. The rational formula, kinetic wave method, and Manning's equation were used to determine the design flow, pipe slope, and pipe diameter. The pipe diameter was excessive for the requirements of this watershed, but common design convention does not size storm pipes smaller than 12 in (ODOT 2020). Calculations for the storm drain can be found in **Appendix F**. Table 6.4 summarizes the design parameters for the storm drain.

Table 6.4. Storm Drain Design Summary

Parameter	Design Value
Watershed Area (ft ²)	942
Average Watershed Flow Velocity (ft/s)	3.2
Time of Concentration	<1s
Peak Design Flow (ft ³ /s)	0.15
Capacity Flow (ft ³ /s)	2.2
Slope	6.25%
Pipe Diameter (in.)	12
Pipe Material	PVC
Pipe Length (L.F)	8

7.0 Structural Design for Choto Road Pump Station

The proposed Choto Road pump station will be a cast-in-place square wet well with a total capacity of 23,766 gal. The wet well design utilized the design Guide for Cantilevered Retaining Walls First edition, Risa Foundation 11.0, ACI318-14, and foundation design Principles and practices second edition. The side walls of the square wet-well were designed as cantilevered retaining walls and Ko was utilized for the resultant of the lateral earth pressure because of the rigidity gained from the other walls making each wall act as a stiff/rigid member. This use of Ko was the only difference in comparison to the design of a cantilevered retaining wall. The load cases considered for design of these side walls were based on the different conditions encountered during construction. The first load case considered was backfilled walls with a dry wet well as this would be the controlling load case for the vertical rebar size and spacing requirements and the wall thickness. The second load case considered the leakage check for the wet-well full of water to max capacity with no backfill. The second load case controlled the second vertical reinforcement size and spacing.

The slab of the wet well was designed using Risa foundation 11.0 and then checked with hand calculations. The controlling load case for the wet well slab considered the walls as strip

loads, the tank at maximum capacity and the three pumps as point loads. This load case determined the longitudinal and transverse reinforcement of the slab.

8.0 Geotechnical Design for Pump Stations

The main parameter for the geotechnical design of Choto Road Pump station was the bearing capacity of the soil. To ensure the soil could bear the weight of the wet well, Terzaghi's bearing capacity equation was used. Conservative parameters based on the area were taken to determine the friction angle, unit weight of soil, and cohesion. The allowable bearing capacity of the soil exceeded the required and no further compaction was required, and a small subgrade layer of stone will be laid for construction of the slab. After the required and allowable bearing capacity were found, the over consolidation ratio was utilized to determine K_o that allowed the structural calculation of the side walls to be completed.

The location of the new wet well for the Choto Rd. Pump station is directly next to the current wet wells on site. Due to the risk of collapse caused by the location of the new wet well, shoring was required for the excavation. Soldier piles with timber lagging were the selected shoring method for the excavation due to experience of TVWS. The characteristics of channel point pump station site allows for an open excavation and will not require shoring. Based on the soil type in the area, the side walls of the excavation will be at an angle of 53 degrees.

9.0 Cost Estimation

The cost estimation for this project was developed to include the expenses associated with the materials and installation of the low-pressure system, the gravity system, the updated Choto Road pump station, the new Channel Point Pump Station along with the associated site and structural work.

The estimated cost for the low-pressure system was derived in association with Wascon Inc. The cost was broken down into the kits and installation associated with the E/One Simplex Grinder Pumps that connect to each house that ties into the low-pressure system along with the different pipe sizes that comprise the low-pressure system. The unit costs for each of these components were acquired through Wascon Inc. which is a distributor of the E/One low-pressure system. The breakdown of the low-pressure system cost estimation can be seen in **Appendix H**. The total cost for the low-pressure system was calculated to be \$3,281,758.12.

To estimate the costs associated with the construction of the gravity sewer along Choto Road, accurate and up-to-date lists of typical material and labor unit costs were obtained. Four main components were reported on for this portion – piping, manhole units, excavation, and backfill. Piping cost estimates tend to have much greater variability due the availability of pipe material alternatives. Pipe material production can vary greatly due to the fluctuating prices of metals and industrial materials. Backfill pricings can fluctuate directly due to the availability and “mineability” of rock for crushed stone. Material pricing often follows local economics. Naturally, the United States has abundant lime, granite, and shale rock formations; the same is true for East Tennessee and surrounding Appalachian states. The preferred code for backfill calls for No. 57

Crushed Stone (Class B) for all sewer lines. Backfill includes a minimum bed thickness of 6 inches and covering the pipe trench up to a depth of 12 inches from the ground surface (TDEC). This stone suffices for all pipe materials, however, only PVC will be used in the gravity portion of the Choto Peninsula system. Excavation costs typically remain steady as most is due to the costs of human labor and machinery but will locally rise and fall with changing economies and inflation. The cost estimate for the gravity system can be seen in **Appendix H**.

Costs associated with both the Choto and Channel Point pump station site designs were estimated primarily with 2019 TDOT average item costs (TDOT 2019 “Average”). Some item unit prices were retrieved from the 2018 average item costs list when 2019 data was not available. Typical unit densities of granular aggregate and granular fill and backfill were selected based on TDOT’s recommend values (TDOT *Roadway*). There are 2 items where TDOT data was not available. First, the control panel for the Channel Point pump station was selected based on typical costs from a manufacturer’s website (Septic Solutions Inc. 2020). Second, the SCADA equipment for the Channel Point pump station was valued based on estimates from another city’s SCADA plan budget for their wastewater system (City of St. Charles 2017). The breakdown of the site design cost estimation can be seen in **Appendix H**.

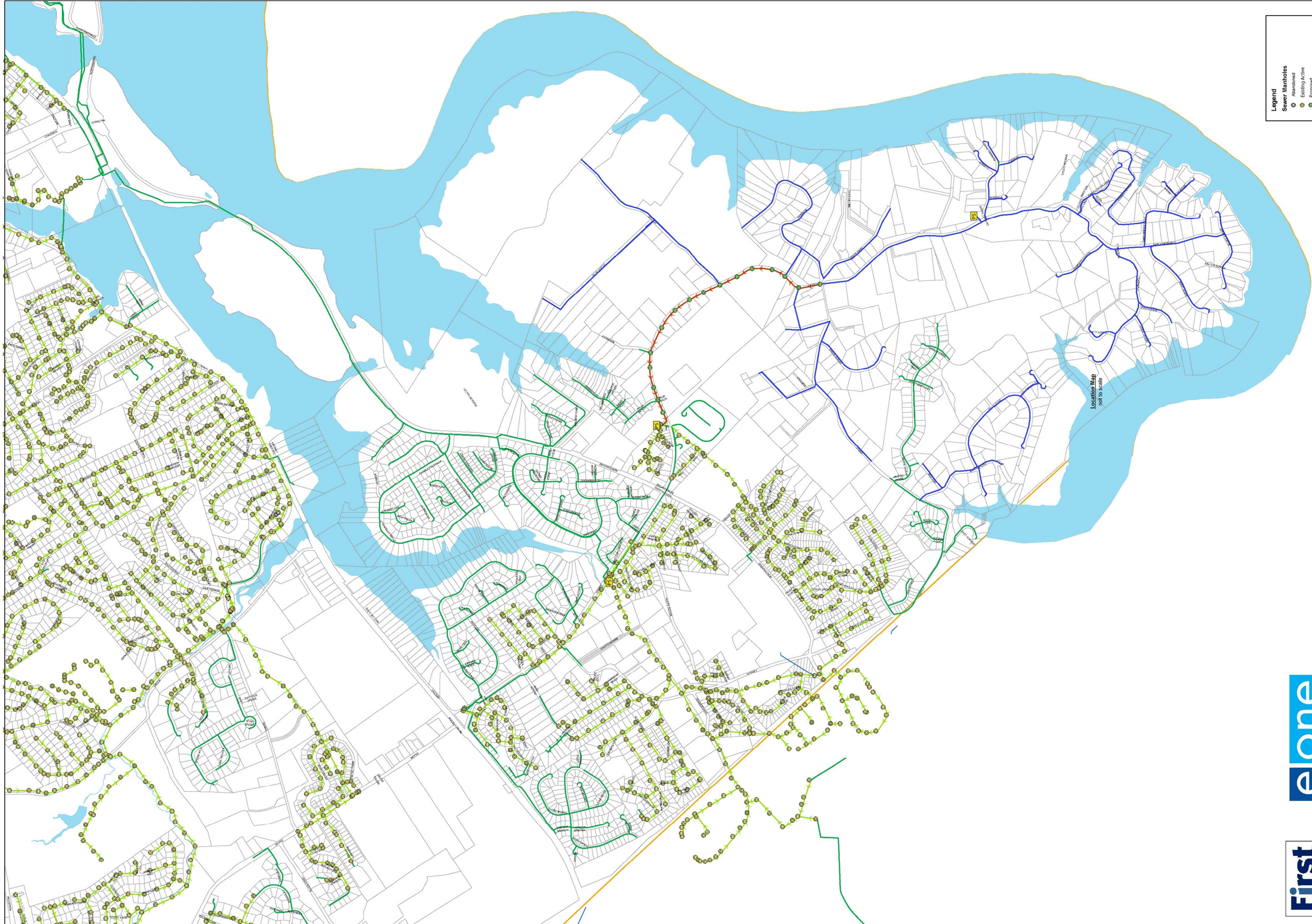
The cost estimate for the Choto wet well was based on the average cost of construction and shoring materials in the area. TVWA contacted local contractors in Tennessee that build cast-in-place structures using concrete and rebar as well as build soldier pile shoring. After these prices were determined, they were multiplied by the quantities to get the total cost for the wet well. The breakdown of the Choto wet well construction cost estimation can be seen in **Appendix H**.

The total cost estimates for the Choto Road, Channel Point, and Holder Branch pump stations were determined based on similar projects provided by FUD. This estimate includes the costs detailed for the site design. The Channel Point pump station is estimated to have an approximate bid price between \$750,000 and \$1,200,000. The Choto Road Pump station is estimated to have an approximate bid price between \$2,000,000 and \$3,000,000. These prices include all station piping, the two pumps, excavation, wet well, force main, and additional construction costs. The cost for the Holder Branch Pump station upgrades will be based on the individual cost for replacing two pumps. No additional components will be required for the Holder Branch pump station. The actual construction costs for these pump stations will need to be determined by the contractor. The total cost for the full system on the peninsula is estimated to be \$7,966,306.

10.0 Conclusion

The Choto Peninsula is currently serviced by the First Utility District of Knox County, and has many undeveloped parcels of land that have the potential to be developed within 10-15 years. Current developments along the southern portion of the peninsula utilize septic tanks and are not serviced by FUD. The proposal outlined in this report includes the design of a low-pressure sewer system network that will service existing and future properties. A proposal for Channel Point pump station will deliver flow from the low-pressure system to the Holder Branch pump station, and all

flows will discharge into the Choto Road pump station. The Holder Branch pump station will require the replacement of the existing pumps to accommodate flow, but the existing structure and force main will remain. The Choto Road pump station will require the existing wet well to be abandoned in place, and a new wet well will be constructed adjacent to the current station on the existing property. The proposal outlined in this report will assist FUD with the planning of future developments, and the construction of low-pressure sewers that will service existing properties will provide land developers easy access to utilities.



Legend

Sewer Manholes

- Abandoned
- Existing Active
- Proposed

Sewer Gravity Mains

- Existing Sewer Gravity Mains
- Proposed Sewer Gravity Mains

Sewer Pressurized Mains

- Existing Sewer Pressurized Mains
- Proposed Sewer Pressurized Mains

FLUD DBO River/Feature

FLUD Boundary

eone
ENVIRONMENT ONE CORPORATION

TVWS
TENNESSEE VALLEY WASTEWATER SOLUTIONS

First
UTILITY DISTRICT
OF KNOX COUNTY
Where community comes first.

1 inch = 800 feet
Date: 6/5/2019
By: April Conner



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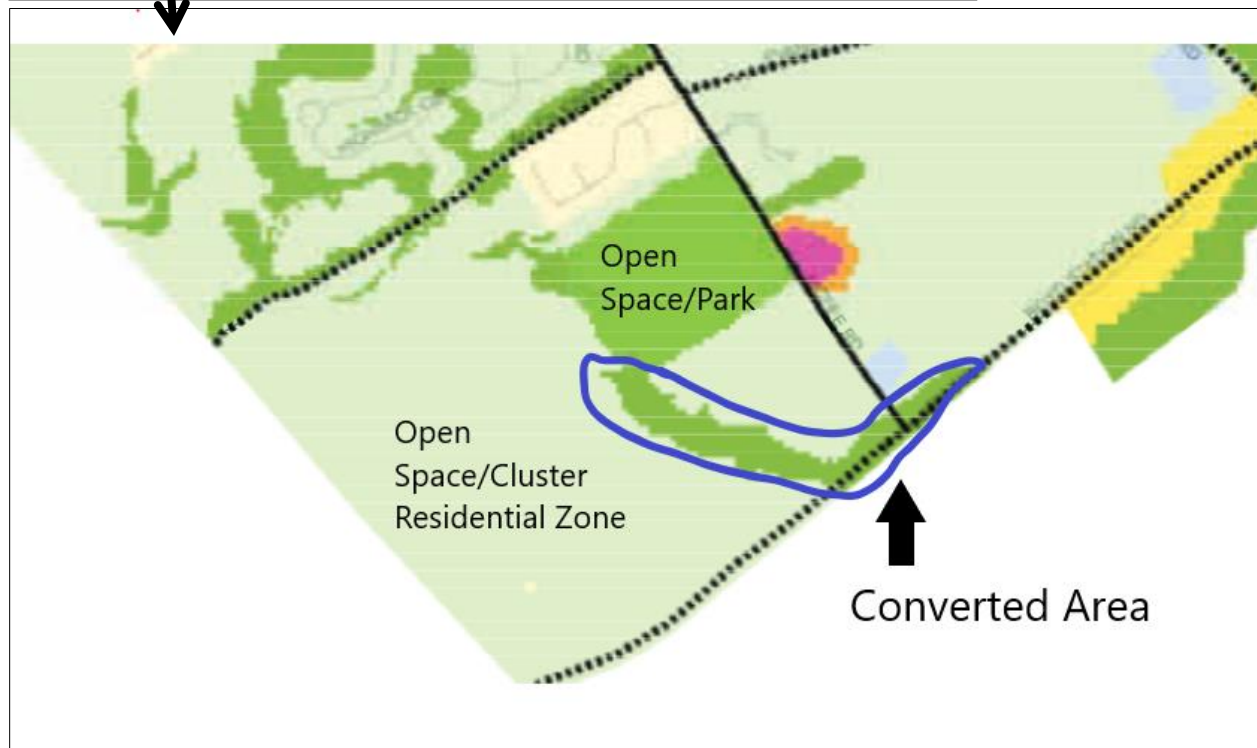
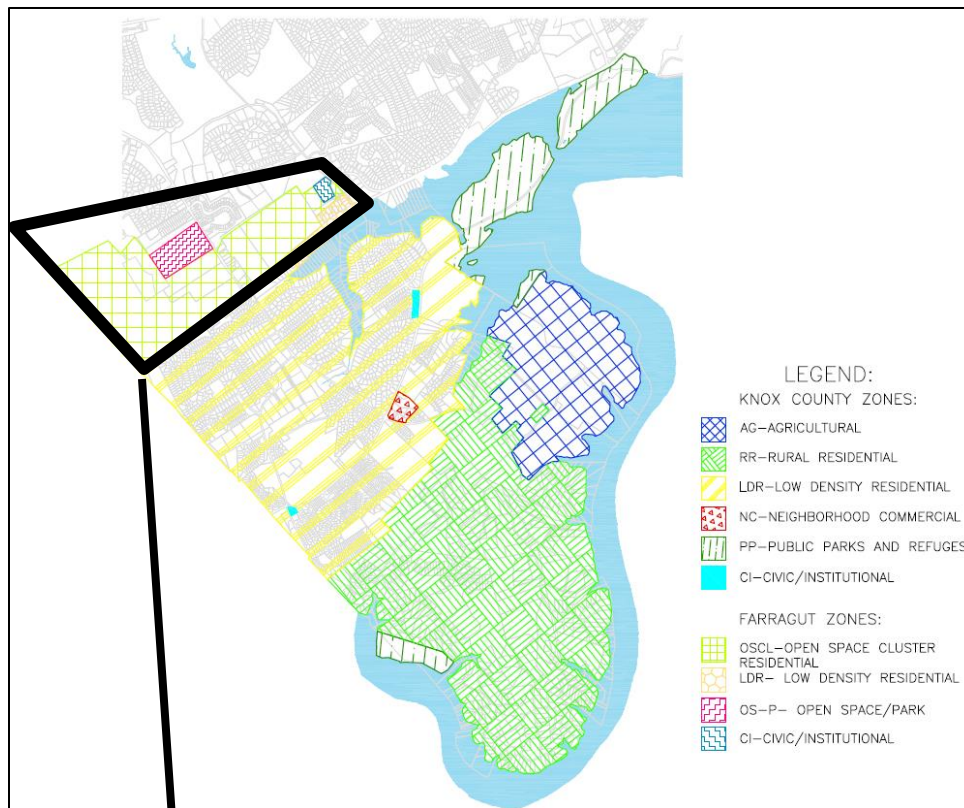
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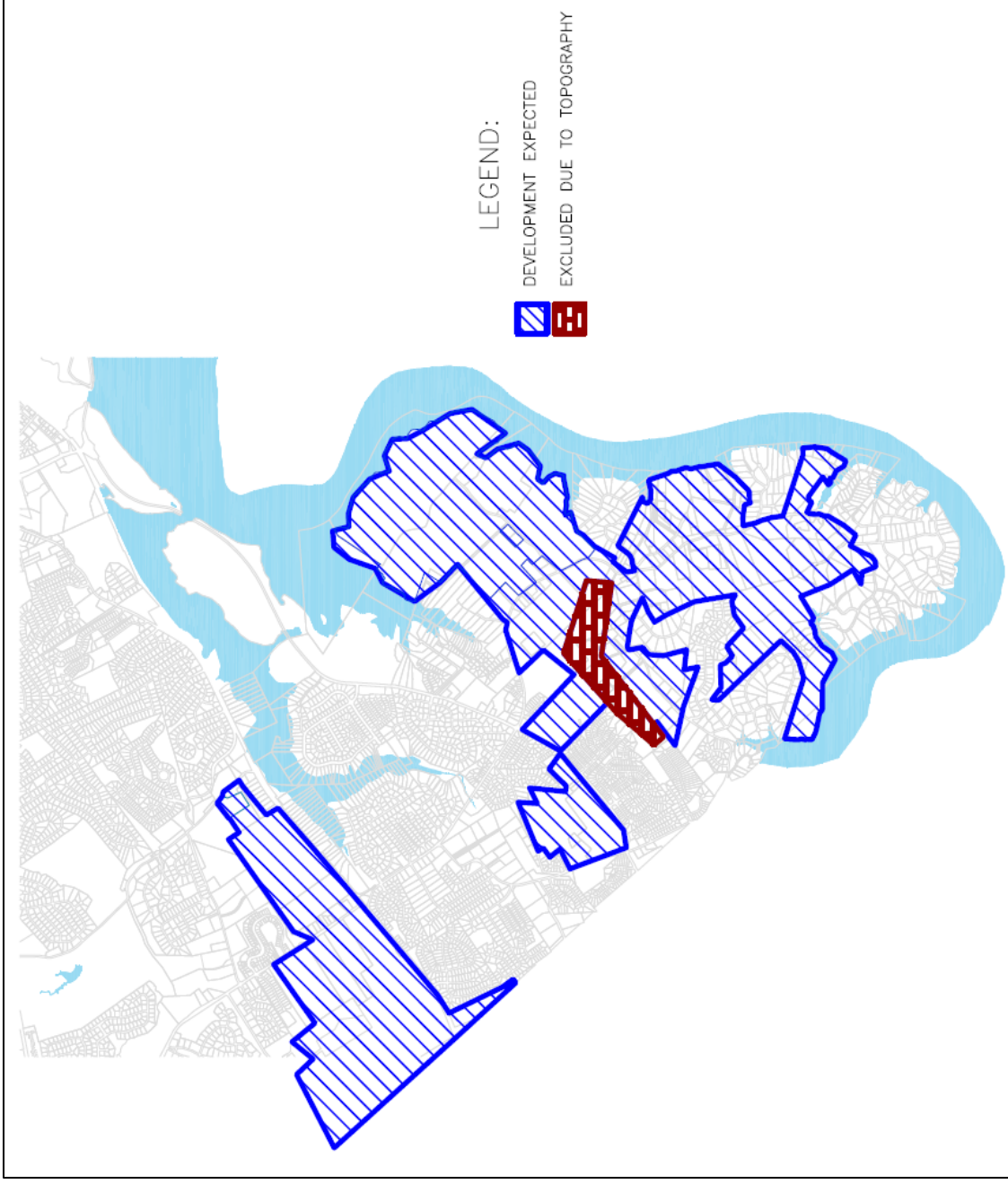
{https://www.ehss.vt.edu/programs/EXC_sloping_benching.php#1} (Accessed April 20th, 2020).

Western Star Inc. (2018). “Vehicle Specifications Summary-Turning Radius.”

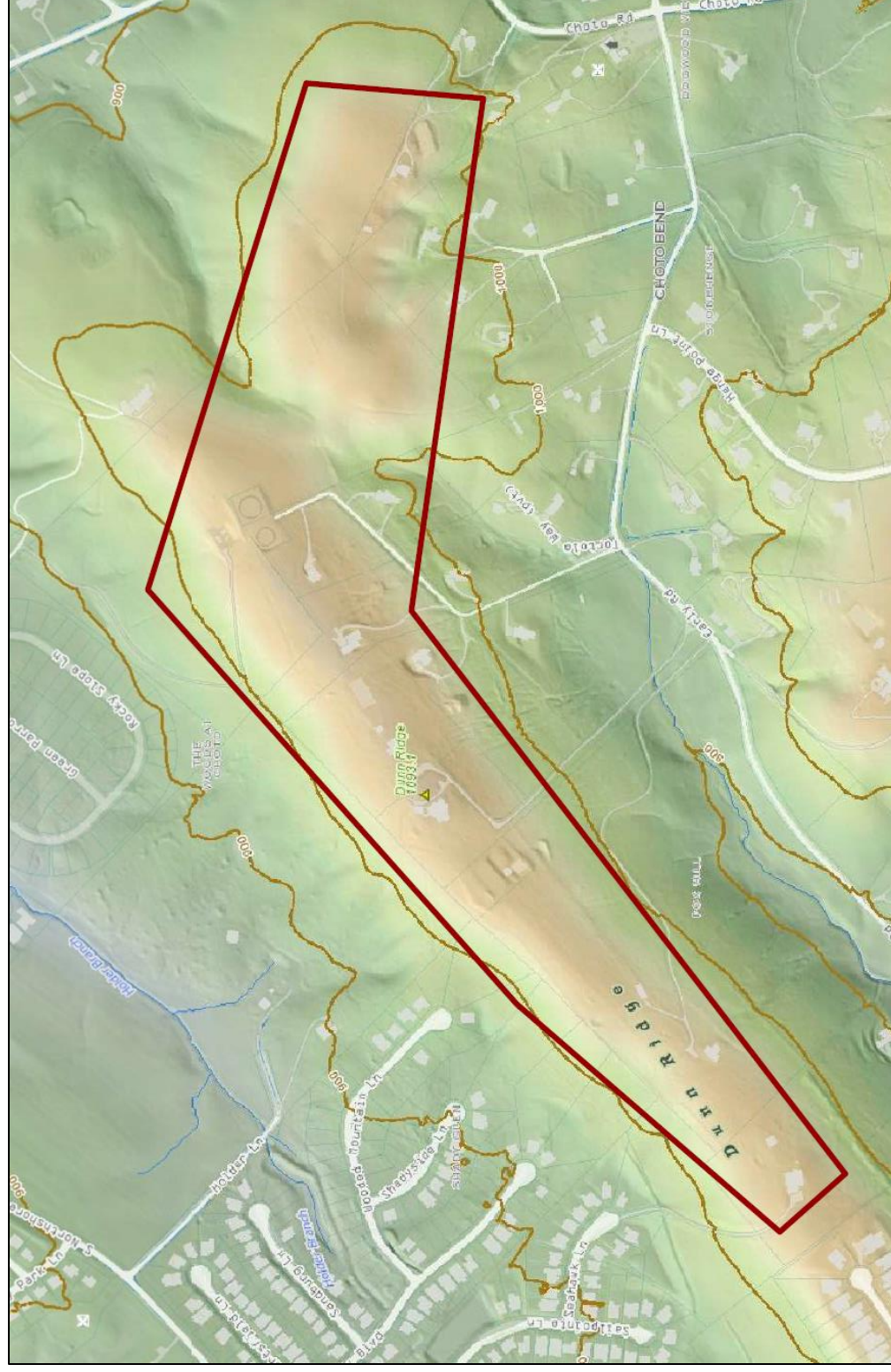
Appendix A. Dwelling Unit Estimations



This map shows the area in Farragut which was converted from Open Space/Park zoning to Open Space/ Cluster Residential zoning for the purposes of this project.



Above is a map depicting the areas included in the undeveloped-area dwelling unit estimation calculations. See the following map for a zoomed in map of the excluded area.



Above is a map depicting the area excluded from the dwelling unit estimations due to the extreme topography. The area starts approximately at the 1000 ft contour line and quickly rises to the peak of almost 1100 ft at the top of Dunn Ridge.

Calculations Performed By:
Jonathan West

Part 1: Totalling Areas by Zone

[illegible]

Part 2: Multiplying Areas by dwelling unit density

Zone	Area (acre)	Recommened Dwelling units per acre per growth plan	Expected	
			Actual dwelling units per acre	Expected houses
Parks and Refuges	49		0	0
Rural Residential	745		1	0.6
Agricultural	487	n/a		0.3
Low Density Residential	176		5	3
Farragut Park	44		0	0
Farragut Civic/Institutional	8		0	0
Farragut OS/CL	497			3
Total	2006			2614.4

Based on existing density

<----

ZONE ID MAP

LEGEND:

KNOX COUNTY ZONES:



AG-AGRICULTURAL

RR-RURAL RESIDENTIAL

LDR-LOW DENSITY RESIDENTIAL

PP-PUBLIC PARKS AND REFUGES

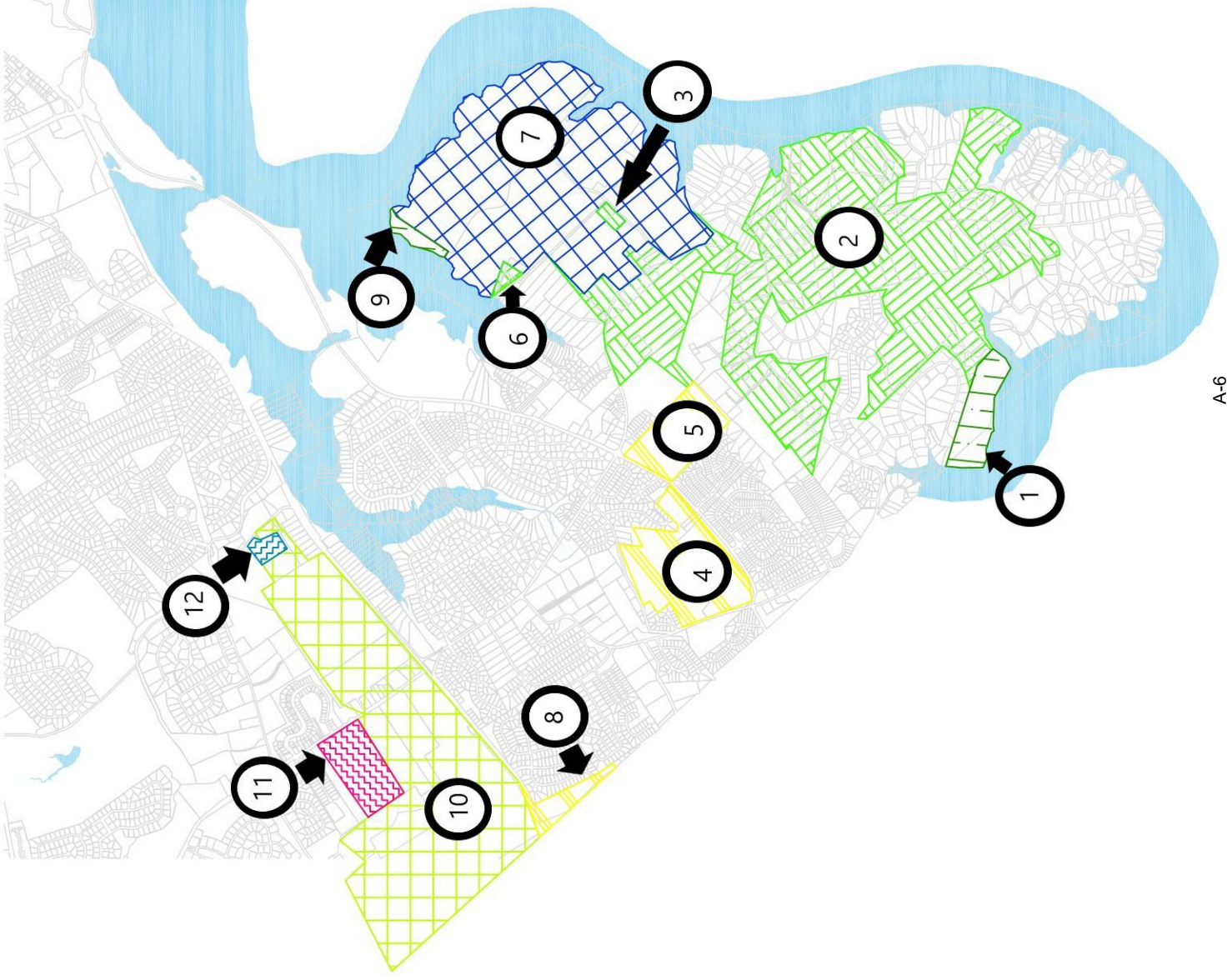
FARRAGUT ZONES:



OSCL-OPEN SPACE CLUSTER
RESIDENTIAL

OS-P- OPEN SPACE/PARK

CI-CIVIC/INSTITUTIONAL



EXISTING ZONES

LEGEND:

KNOX COUNTY ZONES:

- A-AGRICULTURAL
- RA-LOW DENSITY RESIDENTIAL
- PR-PLANNED RESIDENTIAL
- CR-RURAL COMMERCIAL
- CN-NEIGHBORHOOD COMMERCIAL
- CA-GENERAL BUSINESS
- A/HZ-AGRICULTURAL/HISTORIC OVERLAY ZONE

FARRAGUT ZONES:

- A-AGRICULTURAL
- R-1- RURAL SINGLE-FAMILY RESIDENTIAL
- R-2- GENERAL SINGLE-FAMILY RESIDENTIAL
- OS-P- OPEN SPACE/PARK
- S-1 COMMUNITY SERVICE



CI-CIVIC/INSTITUTIONAL

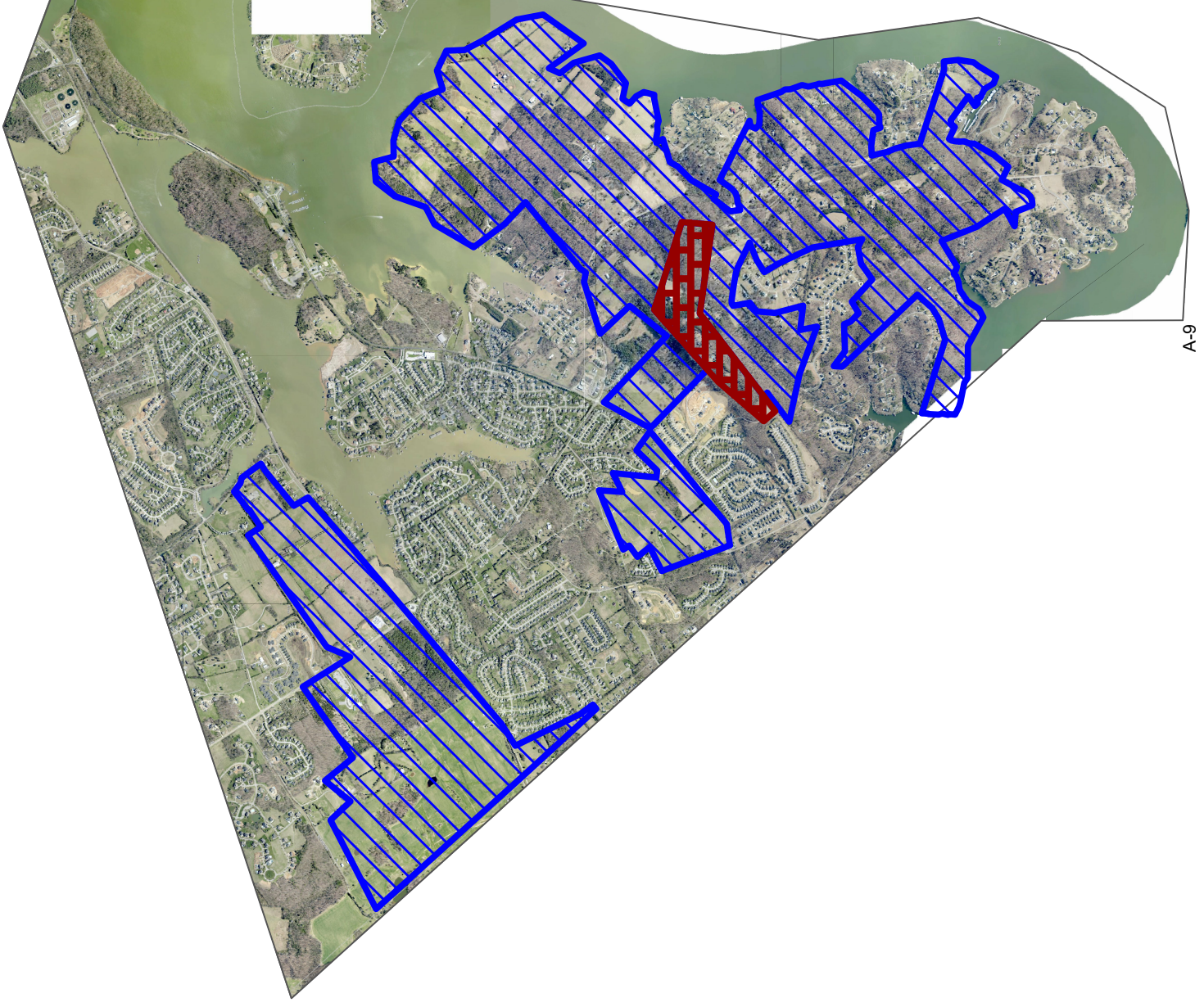
Expected Development Map

LEGEND:



DEVELOPMENT EXPECTED

EXCLUDED DUE TO TOPOGRAPHY



PROJECTED ZONES
OVERLAID ON EXPECTED
DEVELOPMENT



A-10

LEGEND:

KNOX COUNTY ZONES:



AG-AGRICULTURAL



RR-RURAL RESIDENTIAL



LDR-LOW DENSITY RESIDENTIAL



PP-PUBLIC PARKS AND REFUGES

FARRAGUT ZONES:



OSCL-OPEN SPACE CLUSTER
RESIDENTIAL



OS-P- OPEN SPACE/PARK



CI-CIVIC/INSTITUTIONAL

Appendix B. Wastewater Demand Estimations

Calculations Originally performed by:
Tyler Galyon
Reformatted by:
Jonathan West

Existing Demand		
	GPD	GPM
AVG	342000	237.5
Peak	2052000	1425

Pump station Demand Calculations

Future Developments (Houses)	2615
Existing Houses	351
Persons per Household	3
Per Capita WW Demand (gpcd)	65
Population	8898
Peak Hourly/Design Avg	3.00
Minimum Daily/Design Avg	0.33

$$=(\text{Future Houses} + \text{Existing Houses}) \times \text{Persons Per Household}$$

Wastewater Demand		
	GPD	GPM
Average	920370	639.15
Peak Hourly	2761110	1917.44
Minimum	303722	210.92

$$\begin{aligned} &= (\text{Population} \times \text{Per capita Demand}) + \text{Existing Average Flow} \\ &= \text{Average Flow} \times \text{Peak Hourly Factor} \\ &\text{\#NAME?} \end{aligned}$$

Appendix C. E/One Software Reports and Calculations

Calculations Corresponding to the eOne Software - Lower Section of the Peninsula (eOne Software Report 1):

Friction Loss Factor (ft/100 ft) for Zone 1 (Hazen-Williams):

$$h_{100ft} = \frac{0.2083 \left(\frac{100}{C} \right)^{1.852} q^{1.852}}{d_h^{4.8655}}$$

$$d_h = \text{hydraulic diameter} = \frac{4A}{P} = \frac{4(\pi(1'')^2)}{\pi(2'')} = 2''$$

C = 150 for SDR11 HDPE pipe

$$q = \text{Max \# of Sim. Cps} \times \text{Max Flow Per Pump} = 2 \times 11 \text{ GPM} = 22 \text{ GPM}$$

$$h_{100ft} = \frac{0.2083 \left(\frac{100}{150} \right)^{1.852} (22 \text{ GPM})^{1.852}}{(2'')^{4.8655}} = 1.03 \text{ ft/100ft}$$

Design Software calculated 1.19 ft/100 ft on account of differences in rounding.

Maximum Velocity (FPS) for Zone 1:

$$V = \frac{q}{A} = \frac{0.0490164 \text{ ft}^3/\text{s}}{0.021817 \text{ ft}^2} = 2.25 \text{ FPS} > 2 \text{ FPS} \checkmark$$

$$q = 22 \frac{\text{gal}}{\text{min}} \left| \frac{0.13368 \text{ ft}^3}{1 \text{ gal}} \right| \left| \frac{1 \text{ min}}{60 \text{ sec}} \right| = 0.0490164 \text{ ft}^3/\text{s}$$

$$A = \pi(1'')^2 = 3.1416 \text{ in}^2 \left| \frac{1 \text{ ft}}{12 \text{ in}} \right|^2 = 0.021817 \text{ ft}^2$$

Design Software calculated 2.38 FPS on account of differences in rounding.

All other data is user inputted.

Total Dynamic Head (ft):

$$\text{Static Head (ft)} = 966.4 - 891.8 = 74.6$$

$$\begin{aligned} \text{Total Dynamic Head (ft)} &= \text{Static Head (ft)} + \text{Accum Fric Loss (ft)} \\ &= 74.6 \text{ ft} + 69.95 \text{ ft} \\ &= 144.55 \text{ ft} < 185 \text{ ft} \quad \checkmark \end{aligned}$$



Environment One Corporation

Pressure Sewer Preliminary

Cost and Design Analysis

For

Choto Peninsula Low Pressure Sewer System Analysis

Prepared For:
Tennessee Wastewater Solutions

Knoxville, TN TN 37922 US

Tel:

Fax:

Prepared By: Alexander Gothard

March 9, 2020

PRELIMINARY PRESSURE SEWER - PIPE SIZING AND BRANCH ANALYSIS

Choto Peninsula Low Pressure Sewer System Analysis 1 - Lower Section

Prepared By:
Alexander Gothard

March 9, 2020

Zone Number	Connects to Zone	Number of Pumps in Zone	Accum Pumps in Zone	Gals/day per Pump	Max Flow Per Pump (gpm)	Max Sim Ops	Max Flow (GPM)	Pipe Size (inches)	Max Velocity (FPS)	Length of Main in this Zone	Friction Factor (ft/100 ft)	Friction Loss This Zone	Accum Friction Loss (feet)	Max Main Elevation	Minimum Pump Elevation	Static Head (feet)	Total Dynamic Head (ft)
This spreadsheet was calculated using pipe diameters for: SDR11HDPE																	
Friction loss calculations were based on a Constant for inside roughness "C" of: 150																	
1.00	3.00	3	3	200	11.00	2	22.00	2.00	2.38	245.00	1.19	2.91	69.95	966.40	891.80	74.60	144.55
2.00	3.00	7	7	200	11.00	3	33.00	2.00	3.57	377.00	2.52	9.50	76.54	966.40	878.70	87.70	164.24
3.00	5.00	5	15	200	11.00	4	44.00	3.00	2.19	780.00	0.65	5.07	67.04	966.40	884.70	81.70	148.74
4.00	5.00	6	6	200	11.00	3	33.00	2.00	3.57	421.00	2.52	10.61	72.58	966.40	867.70	98.70	171.28
5.00	11.00	13	34	200	11.00	6	66.00	3.00	3.29	1,533.00	1.38	21.11	61.97	966.40	889.50	76.90	138.87
6.00	11.00	17	17	200	11.00	4	44.00	3.00	2.19	1,863.00	0.65	12.11	52.97	966.40	866.40	100.00	152.97
7.00	9.00	6	12	200	11.00	4	44.00	3.00	2.19	841.00	0.65	5.47	52.29	966.40	835.30	131.10	183.39
8.00	7.00	6	6	200	11.00	3	33.00	2.00	3.57	334.00	2.52	8.41	60.70	966.40	865.70	100.70	161.40
9.00	11.00	8	37	200	11.00	6	66.00	3.00	3.29	433.00	1.38	5.96	46.82	966.40	890.00	76.40	123.22
10.00	9.00	17	17	200	11.00	4	44.00	3.00	2.19	2,656.00	0.65	17.26	64.08	966.40	850.00	116.40	180.48
11.00	16.00	4	130	200	11.00	9	99.00	4.00	2.98	3,138.00	0.86	26.97	40.86	966.40	896.40	70.00	110.86
12.00	11.00	14	38	200	11.00	6	66.00	3.00	3.29	350.00	1.38	4.82	45.68	966.40	877.80	88.60	134.28
13.00	12.00	8	8	200	11.00	3	33.00	2.50	2.44	1,463.00	0.99	14.55	60.23	966.40	845.00	121.40	181.63
14.00	12.00	10	10	200	11.00	4	44.00	3.00	2.19	614.00	0.65	3.99	49.67	966.40	886.30	80.10	129.77
15.00	12.00	6	6	200	11.00	3	33.00	2.50	2.44	1,260.00	0.99	12.53	58.21	966.40	840.00	126.40	184.61
16.00	16.00	1	195	200	11.00	11	121.00	5.00	2.39	3,124.00	0.44	13.89	13.89	915.90	818.80	97.10	110.99
17.00	16.00	17	22	200	11.00	5	55.00	3.00	2.74	1,392.00	0.98	13.68	27.57	966.40	829.20	137.20	164.77
18.00	17.00	5	5	200	11.00	3	33.00	2.00	3.57	285.00	2.52	7.18	34.75	966.40	891.80	74.60	109.35
19.00	16.00	15	33	200	11.00	6	66.00	3.00	3.29	1,942.00	1.38	26.74	40.63	966.40	843.30	123.10	163.73
20.00	19.00	5	5	200	11.00	3	33.00	2.00	3.57	483.00	2.52	12.17	52.80	982.70	946.10	36.60	89.40
21.00	19.00	6	6	200	11.00	3	33.00	2.00	3.57	569.00	2.52	14.34	54.97	966.40	902.70	63.70	118.67
22.00	19.00	7	7	200	11.00	3	33.00	2.00	3.57	364.00	2.52	9.17	49.80	966.40	874.30	92.10	141.90
23.00	16.00	9	9	200	11.00	3	33.00	2.00	3.57	1,153.00	2.52	29.05	42.94	966.40	912.90	53.50	96.44

PRELIMINARY PRESSURE SEWER - ACCUMULATED RETENTION TIME (HR)
Choto Peninsula Low Pressure Sewer System Analysis 1 - Lower Section

Prepared By:
 Alexander Gothard

March 9, 2020

Zone Number	Connects to Zone	Accumulated Total of Pumps this Zone	Pipe Size (inches)	Gallons per 100 lineal feet	Length of Zone	Capacity of Zone	Average Daily Flow	Average Fluid Changes per Day	Average Retention Time (Hr)	Accumulated Retention Time (Hr)
This spreadsheet was calculated using pipe diameters for: SDR11HDPE										
1.00	3.00	3	2.00	15.40	245.00	37.74	600	15.90	1.51	200
2.00	3.00	7	2.00	15.40	377.00	58.07	1,400	24.11	1.00	8.64
3.00	5.00	15	3.00	33.47	780.00	261.04	3,000	11.49	2.09	8.12
4.00	5.00	6	2.00	15.40	421.00	64.85	1,200	18.51	1.30	7.13
5.00	11.00	34	3.00	33.47	1,533.00	513.04	6,800	13.25	1.81	6.33
6.00	11.00	17	3.00	33.47	1,863.00	623.48	3,400	5.45	4.40	5.04
7.00	9.00	12	3.00	33.47	841.00	281.45	2,400	8.53	2.81	7.63
8.00	7.00	6	2.00	15.40	334.00	51.45	1,200	23.33	1.03	6.51
9.00	11.00	37	3.00	33.47	433.00	144.91	7,400	51.07	0.47	7.54
10.00	9.00	17	3.00	33.47	2,656.00	888.87	3,400	3.83	6.27	3.70
11.00	16.00	130	4.00	55.31	3,138.00	1,735.73	26,000	14.98	1.60	9.97
12.00	11.00	38	3.00	33.47	350.00	117.13	7,600	64.88	0.37	3.23
13.00	12.00	8	2.50	22.57	1,463.00	330.20	1,600	4.85	4.95	3.60
14.00	12.00	10	3.00	33.47	614.00	205.48	2,000	9.73	2.47	8.55
15.00	12.00	6	2.50	22.57	1,260.00	284.39	1,200	4.22	5.69	6.06
16.00	16.00	195	5.00	84.50	3,124.00	2,639.86	39,000	14.77	1.62	9.28
17.00	16.00	22	3.00	33.47	1,392.00	465.85	4,400	9.45	2.54	1.62
18.00	17.00	5	2.00	15.40	285.00	43.90	1,000	22.78	1.05	4.17
19.00	16.00	33	3.00	33.47	1,942.00	649.92	6,600	10.16	2.36	5.22
20.00	19.00	5	2.00	15.40	483.00	74.40	1,000	13.44	1.79	3.99
21.00	19.00	6	2.00	15.40	569.00	87.64	1,200	13.69	1.75	5.77
22.00	19.00	7	2.00	15.40	364.00	56.07	1,400	24.97	0.96	5.74
23.00	16.00	9	2.00	15.40	1,153.00	177.59	1,800	10.14	2.37	4.95
										3.99

Choto Peninsula Low Pressure Sewer System Analysis 1 - Lower Section

Prepared by : Alexander Gothard

On: March 9, 2020

Notes :

This is the lower section of the low pressure system analysis for the peninsula.

Analysis based upon drawings and data provided. Station recommendations are preliminary.

GPD values impact retention times only, not line sizing or hydraulics. GP laterals to be 1.25".

General recommendations for valve placement are: clean out valves at intervals of approximately 1,000 ft and at branch ends and junctions; isolation valves at branch junctions; and air release valves at peaks of 25 ft or more and/or at intervals of 2,000 to 2,500 ft. Lateral kits comprised of a ball and check valve are required to be installed between the pump discharge and street main on all installations. Laterals should be located as close to the public right of way as possible.

<<<< END OF NOTES >>>>



Environment One Corporation

Pressure Sewer Preliminary

Cost and Design Analysis

For

Choto Peninsula Low Pressure Sewer System Analysis

Prepared For:
Tennessee Wastewater Solutions

Knoxville TN 37922 US

Tel:

Fax:

Prepared By: Alexander Gothard

March 9, 2020

Budgetary Low Pressure Sewer System Costs

Choto Peninsula Low Pressure Sewer System Analysis 2 - Northwest Developed Section

	<u>Quantity</u>	<u>Description</u>	<u>Unit Cost</u>	<u>Installation</u>	<u>Sub Total</u>
Pumps	59	Lateral (Boundary) Kits	\$0.00	0.00	\$11,800.00
	59	Lateral (Boundary) Installation	\$0.00	0.00	\$103,250.00
	59	Pump/Panel Installation	\$0.00	0.00	\$103,250.00
	59	Simplex Grinder Pump Stations	\$0.00	0.00	\$143,311.00
					<u>\$361,611.00</u>

Piping	1,830	2.00" Pipe	\$0.00	2,744.85	\$2,744.85
	5,361	3.00" Pipe	\$0.00	46,908.75	\$46,908.75
	1,767	4.00" Pipe	\$0.00	4,876.92	\$4,876.92
					<u>\$54,530.52</u>

Number of Connections	<u>59</u>		
Total Per Connection	<u>\$6,129.00</u>	Total (w/o other)	<u>\$361,611.00</u>
Grand Total Per Connection	<u>\$7,053.25</u>	Grand Total (including other)	<u>\$416,141.52</u>

Note: The System Costs above are based on piping sized for, and Grinder Pumps manufactured by Environment One Corporation.

PRELIMINARY PRESSURE SEWER - PIPE SIZING AND BRANCH ANALYSIS

Choto Peninsula Low Pressure Sewer System Analysis 2 - Northwest Developed Section

Prepared By:
Alexander Gothard

March 9, 2020

Zone Number	Connects to Zone	Number of Pumps in Zone	Accum Pumps in Zone	Gals/day per Pump	Max Flow Per Pump (gpm)	Max Sim Ops	Max Flow (GPM)	Pipe Size (inches)	Max Velocity (FPS)	Length of Main in this Zone	Friction Factor (ft/100 ft)	Friction Loss This Zone	Accum Friction Loss (feet)	Max Main Elevation	Minimum Pump Elevation	Static Head (feet)	Total Dynamic Head (ft)
This spreadsheet was calculated using pipe diameters for: SDR11HDPE																	
Friction loss calculations were based on a Constant for inside roughness "C" of: 150																	
32.00	33.00	6	6	200	11.00	3	33.00	2.00	3.57	1,102.00	2.52	27.76	58.18	998.00	876.40	121.60	179.78
33.00	35.00	6	12	200	11.00	4	44.00	3.00	2.19	865.00	0.65	5.62	30.42	998.00	937.00	61.00	91.42
34.00	35.00	10	10	200	11.00	4	44.00	3.00	2.19	1,381.00	0.65	8.97	33.77	1,037.00	925.70	111.30	145.07
35.00	37.00	3	25	200	11.00	5	55.00	3.00	2.74	884.00	0.98	8.68	24.80	998.00	925.70	72.30	97.10
36.00	37.00	17	17	200	11.00	4	44.00	3.00	2.19	1,753.00	0.65	11.39	27.51	1,042.30	978.40	63.90	91.41
37.00	38.00	2	44	200	11.00	6	66.00	3.00	3.29	478.00	1.38	6.58	16.12	998.00	934.60	63.40	79.52
38.00	40.00	6	54	200	11.00	7	77.00	4.00	2.32	885.00	0.54	4.78	9.54	998.00	934.60	63.40	72.94
39.00	38.00	4	4	200	11.00	3	33.00	2.00	3.57	727.90	2.52	18.34	27.88	998.00	934.60	63.40	91.28
40.00	40.00	5	59	200	11.00	7	77.00	4.00	2.32	882.00	0.54	4.76	4.76	998.00	953.00	45.00	49.76

PRELIMINARY PRESSURE SEWER - ACCUMULATED RETENTION TIME (HR)
Choto Peninsula Low Pressure Sewer System Analysis 2 - Northwest Developed Section

Prepared By:
Alexander Gothard

March 9, 2020

Zone Number	Connects to Zone	Accumulated Total of Pumps this Zone	Pipe Size (inches)	Gallons per 100 lineal feet	Length of Zone	Capacity of Zone	Average Daily Flow	Average Fluid Changes per Day	Average Retention Time (Hr)	Accumulated Retention Time (Hr)
This spreadsheet was calculated using pipe diameters for: SDR11HDPE										
Gals per Day per Dwelling										
32.00	33.00	6	2.00	15.40	1,102.00	169.74	1,200	7.07	3.39	200
33.00	35.00	12	3.00	33.47	865.00	289.48	2,400	8.29	2.89	10.23
34.00	35.00	10	3.00	33.47	1,381.00	462.17	2,000	4.33	5.55	6.83
35.00	37.00	25	3.00	33.47	884.00	295.84	5,000	16.90	1.42	9.48
36.00	37.00	17	3.00	33.47	1,753.00	586.67	3,400	5.80	4.14	3.94
37.00	38.00	44	3.00	33.47	478.00	159.97	8,800	55.01	0.44	6.66
38.00	40.00	54	4.00	55.31	885.00	489.52	10,800	22.06	1.09	2.52
39.00	38.00	4	2.00	15.40	727.90	112.12	800	7.14	3.36	2.08
40.00	40.00	59	4.00	55.31	882.00	487.86	11,800	24.19	0.99	5.44
										0.99

Choto Peninsula Low Pressure Sewer System Analysis 2 - Northwest Develo

Prepared by : Alexander Gothard

On: March 9, 2020

Notes :

This is the Northwest section of the low pressure system analysis.

Analysis based upon drawings and data provided. Station recommendations are preliminary.

GPD values impact retention times only, not line sizing or hydraulics. GP laterals to be 1.25".

General recommendations for valve placement are: clean out valves at intervals of approximately 1,000 ft and at branch ends and junctions; isolation valves at branch junctions; and air release valves at peaks of 25 ft or more and/or at intervals of 2,000 to 2,500 ft. Lateral kits comprised of a ball and check valve are required to be installed between the pump discharge and street main on all installations. Laterals should be located as close to the public right of way as possible.

<<<< END OF NOTES >>>>



Environment One Corporation

**Pressure Sewer Preliminary
Cost and Design Analysis
For**

Choto Peninsula Low Pressure Sewer System Analysis

**Prepared For:
Tennessee Wastewater Solutions**

Knoxville TN 37922 US

Tel:

Fax:

Prepared By: Alexander Gothard

March 9, 2020

PRELIMINARY PRESSURE SEWER - PIPE SIZING AND BRANCH ANALYSIS
Choto Peninsula Low Pressure Sewer System Analysis 3 - Northeast Developed Section

Prepared By:
 Alexander Gothard

March 9, 2020

Zone Number	Connects to Zone	Number of Pumps in Zone	Accum Pumps in Zone	Gals/day per Pump	Max Flow Per Pump (gpm)	Max Sim Ops	Max Flow (GPM)	Pipe Size (inches)	Max Velocity (FPS)	Length of Main this Zone	Friction Factor (ft/100 ft)	Friction Loss This Zone	Accum Friction Loss (feet)	Max Main Elevation	Minimum Pump Elevation	Static Head (feet)	Total Dynamic Head (ft)
This spreadsheet was calculated using pipe diameters for: SDR11 HDPE																	
41.00	42.00	15	15	200	11.00	4	44.00	3.00	2.19	2,064.00	0.65	13.41	40.27	997.10	934.10	63.00	150
42.00	43.00	46	61	200	11.00	7	77.00	4.00	2.32	4,393.00	0.54	23.70	26.86	997.10	856.10	141.00	103.27
43.00	43.00	3	64	200	11.00	7	77.00	4.00	2.32	586.30	0.54	3.16	3.16	988.00	900.00	88.00	167.86
																	91.16

PRELIMINARY PRESSURE SEWER - ACCUMULATED RETENTION TIME (HR)
Choto Peninsula Low Pressure Sewer System Analysis 3 - Northeast Developed Section

Prepared By:
Alexander Gothard

March 9, 2020

Zone Number	Connects to Zone	Accumulated Total of Pumps this Zone	Pipe Size (inches)	Gallons per 100 lineal feet	Length of Zone	Capacity of Zone	Average Daily Flow	Average Fluid Changes per Day	Average Retention Time (Hr)	Accumulated Retention Time (Hr)
This spreadsheet was calculated using pipe diameters for: SDR11HDPE										
41.00	42.00	15	3.00	33.47	2,064.00	690.75	3,000	4.34	5.53	10.91
42.00	43.00	61	4.00	55.31	4,393.00	2,429.91	12,200	5.02	4.78	5.39
43.00	43.00	64	4.00	55.31	586.30	324.30	12,800	39.47	0.61	0.61

Choto Peninsula Low Pressure Sewer System Analysis 3 - Northeast Develop

Prepared by : Alexander Gothard

On: March 9, 2020

Notes :

This is the Northeast developed section of the low pressure system analysis.

Analysis based upon drawings and data provided. Station recommendations are preliminary.

GPD values impact retention times only, not line sizing or hydraulics. GP laterals to be 1.25".

General recommendations for valve placement are: clean out valves at intervals of approximately 1,000 ft and at branch ends and junctions; isolation valves at branch junctions; and air release valves at peaks of 25 ft or more and/or at intervals of 2,000 to 2,500 ft. Lateral kits comprised of a ball and check valve are required to be installed between the pump discharge and street main on all installations. Laterals should be located as close to the public right of way as possible.

<<<<< **END OF NOTES** >>>>>



Environment One Corporation

**Pressure Sewer Preliminary
Cost and Design Analysis
For**

Choto Peninsula Low Pressure Sewer System Analysis

**Prepared For:
Tennessee Wastewater Solutions**

Knoxville TN 37922

Tel:

Fax:

Prepared By: Alexander Gothard

March 9, 2020

Budgetary Low Pressure Sewer System Costs

Choto Peninsula Low Pressure Sewer System Analysis 4 - Northeast Undeveloped Section

	<u>Quantity</u>	<u>Description</u>	<u>Unit Cost</u>	<u>Installation</u>	<u>Sub Total</u>
Pumps	171	Lateral (Boundary) Kits	\$0.00	0.00	\$0.00
	171	Lateral (Boundary) Installation	\$0.00	0.00	\$299,250.00
	171	Pump/Panel Installation	\$0.00	0.00	\$299,250.00
	171	Simplex Grinder Pump Stations	\$0.00	0.00	\$415,359.00
					<u>\$1,013,859.00</u>

Piping	2,580	4.00" Pipe	\$0.00	7,121.63	\$7,121.63
	3,788	5.00" Pipe	\$0.00	34,087.50	\$34,087.50
					<u>\$41,209.13</u>

Number of Connections	<u>171</u>		
Total Per Connection	<u>\$5,929.00</u>	Total (w/o other)	<u>\$1,013,859.00</u>
Grand Total Per Connection	<u>\$6,169.99</u>	Grand Total (including other)	<u>\$1,055,068.13</u>

Note: The System Costs above are based on piping sized for, and Grinder Pumps manufactured by Environment One Corporation.

PRELIMINARY PRESSURE SEWER - PIPE SIZING AND BRANCH ANALYSIS

Choto Peninsula Low Pressure Sewer System Analysis 4 - Northeast Undeveloped Section

Prepared By:
Alexander Gothard

March 9, 2020

Zone Number	Connects to Zone	Number of Pumps in Zone	Accum Pumps in Zone	Gals/day per Pump	Max Flow Per Pump (gpm)	Max Sim Ops	Max Flow (GPM)	Pipe Size (inches)	Max Velocity (FPS)	Length of Main this Zone	Friction Factor (ft/100 ft)	Friction Loss This Zone	Accum Friction Loss (feet)	Max Main Elevation	Minimum Pump Elevation	Static Head (feet)	Total Dynamic Head (ft)
This spreadsheet was calculated using pipe diameters for: SDR11HDPE																	
44.00	44.00	102	171	200	11.00	10	110.00	5.00	2.17	3,787.50	0.37	14.11	14.11	892.70	825.50	67.20	150
45.00	44.00	69	69	200	11.00	7	77.00	4.00	2.32	2,580.30	0.54	13.92	28.03	940.00	840.00	100.00	128.03

PRELIMINARY PRESSURE SEWER - ACCUMULATED RETENTION TIME (HR)
Choto Peninsula Low Pressure Sewer System Analysis 4 - Northeast Undeveloped Section

Prepared By:
Alexander Gothard

March 9, 2020

Zone Number	Connects to Zone	Accumulated Total of Pumps this Zone	Pipe Size (inches)	Gallons per 100 lineal feet	Length of Zone	Capacity of Zone	Average Daily Flow	Average Fluid Changes per Day	Average Retention Time (Hr)	Accumulated Retention Time (Hr)
This spreadsheet was calculated using pipe diameters for: SDR11HDPE										
44.00	44.00	171	5.00	84.50	3,787.50	3,200.53	34,200	10.69	2.25	2.25
45.00	44.00	69	4.00	55.31	2,580.30	1,427.25	13,800	9.67	2.48	4.73

Choto Peninsula Low Pressure Sewer System Analysis 4 - Northeast Undeveloped

Prepared by : Alexander Gothard

On: March 9, 2020

Notes :

This is the undeveloped Northeast section of the peninsula.

Analysis based upon drawings and data provided. Station recommendations are preliminary.

GPD values impact retention times only, not line sizing or hydraulics. GP laterals to be 1.25".

General recommendations for valve placement are: clean out valves at intervals of approximately 1,000 ft and at branch ends and junctions; isolation valves at branch junctions; and air release valves at peaks of 25 ft or more and/or at intervals of 2,000 to 2,500 ft. Lateral kits comprised of a ball and check valve are required to be installed between the pump discharge and street main on all installations. Laterals should be located as close to the public right of way as possible.

<<<<< END OF NOTES >>>>>

PRELIMINARY PRESSURE SEWER - PIPE SIZING AND BRANCH ANALYSIS

Choto Peninsula Low Pressure Sewer System Analysis 1 - Lower Section

Prepared By:
Alexander Gothard

March 9, 2020

Zone Number	Connects to Zone	Number of Pumps in Zone	Accum Pumps in Zone	Gals/day per Pump	Max Flow Per Pump (gpm)	Max Sim Ops	Max Flow (GPM)	Pipe Size (inches)	Max Velocity (FPS)	Length of Main this Zone	Friction Factor (ft/100 ft)	Friction Loss This Zone	Accum Friction Loss (feet)	Max Main Elevation	Minimum Pump Elevation	Static Head (feet)	Total Dynamic Head (ft)
This spreadsheet was calculated using pipe diameters for: SDR11HDPE																	
Friction loss calculations were based on a Constant for inside roughness "C" of: 150																	
1.00	3.00	3	3	200	11.00	2	22.00	2.00	2.38	245.00	1.19	2.91	85.92	966.40	891.80	74.60	160.52
2.00	3.00	7	7	200	11.00	3	33.00	2.00	3.57	377.00	2.52	9.50	92.51	966.40	878.70	87.70	180.21
3.00	5.00	5	15	200	11.00	4	44.00	3.00	2.19	780.00	0.65	5.07	83.01	966.40	884.70	81.70	164.71
4.00	5.00	6	6	200	11.00	3	33.00	2.00	3.57	421.00	2.52	10.61	88.55	966.40	867.70	98.70	187.25
5.00	11.00	13	34	200	11.00	6	66.00	3.00	3.29	1,533.00	1.38	21.11	77.94	966.40	889.50	76.90	154.84
6.00	11.00	17	17	200	11.00	4	44.00	3.00	2.19	1,863.00	0.65	12.11	68.94	966.40	866.40	100.00	168.94
7.00	9.00	6	12	200	11.00	4	44.00	3.00	2.19	841.00	0.65	5.47	68.26	966.40	835.30	131.10	199.36
8.00	7.00	6	6	200	11.00	3	33.00	2.00	3.57	334.00	2.52	8.41	76.67	966.40	865.70	100.70	177.37
9.00	11.00	8	37	200	11.00	6	66.00	3.00	3.29	433.00	1.38	5.96	62.79	966.40	890.00	76.40	139.19
10.00	9.00	17	17	200	11.00	4	44.00	3.00	2.19	2,656.00	0.65	17.26	80.05	966.40	850.00	116.40	196.45
11.00	16.00	4	130	200	11.00	9	99.00	4.00	2.98	3,138.00	0.86	26.97	56.83	966.40	896.40	70.00	126.83
12.00	11.00	14	38	200	11.00	6	66.00	3.00	3.29	350.00	1.38	4.82	61.65	966.40	877.80	88.60	150.25
13.00	12.00	8	8	200	11.00	3	33.00	2.50	2.44	1,463.00	0.99	14.55	76.20	966.40	845.00	121.40	197.60
14.00	12.00	10	10	200	11.00	4	44.00	3.00	2.19	614.00	0.65	3.99	65.64	966.40	886.30	80.10	145.74
15.00	12.00	6	6	200	11.00	3	33.00	2.50	2.44	1,260.00	0.99	12.53	74.18	966.40	840.00	126.40	200.58
16.00	16.00	1	195	200	11.00	11	121.00	5.00	2.39	6,717.00	0.44	29.86	29.86	915.90	818.80	97.10	126.96
17.00	16.00	17	22	200	11.00	5	55.00	3.00	2.74	1,392.00	0.98	13.68	43.54	966.40	829.20	137.20	180.74
18.00	17.00	5	5	200	11.00	3	33.00	2.00	3.57	285.00	2.52	7.18	50.72	966.40	891.80	74.60	125.32
19.00	16.00	15	33	200	11.00	6	66.00	3.00	3.29	1,942.00	1.38	26.74	56.60	966.40	843.30	123.10	179.70
20.00	19.00	5	5	200	11.00	3	33.00	2.00	3.57	483.00	2.52	12.17	68.77	982.70	946.10	36.60	105.37
21.00	19.00	6	6	200	11.00	3	33.00	2.00	3.57	569.00	2.52	14.34	70.94	966.40	902.70	63.70	134.64
22.00	19.00	7	7	200	11.00	3	33.00	2.00	3.57	364.00	2.52	9.17	65.77	966.40	874.30	92.10	157.87
23.00	16.00	9	9	200	11.00	3	33.00	2.00	3.57	1,153.00	2.52	29.05	58.91	966.40	912.90	53.50	112.41

Appendix D. Pump Station/Force Main Hydraulic Calculations

WASTEWATER DEMAND

Choto rd. Pump Station Flow Demands

Existing Discharge into choto Pump Station (gpd)	342000
Future Developments (Houses)	2615
Low-Pressure Houses	351
Persons per Household	3
Per Capita WW Demand (gpcd)	65
Population	8898
Peak Hourly/Design Avg	3.00
Minimum Daily/Design Avg	0.33

Average Low-Pressure Flow (gpm)	47.53
Average Future Flow (gpm)	354.11
Average Existing Flow (gpm)	237.50
Total Average Flow (gpm)	639.15

Peak Hourly Flow (gpm)	1917.44
------------------------	---------

Channel Point Pump Station

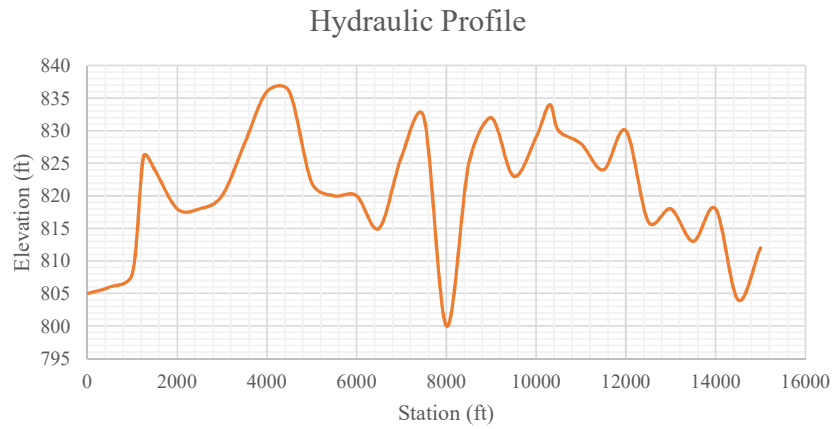
Number of Houses in Section	351
Average Low-Pressure Flow (gpm)	47.53
Peak Hourly flow (gpm)	142.59375

STATIC HEAD CALCULATIONS

Choto Rd. Pump Station

Hydraulic Profile	
Station	Invert Elevation (ft)
15	805
500	806
1000	808
1250	826
1500	824
2000	818
2500	818
3000	820
3500	828
4000	836
4500	836
5000	822
5500	820
6000	820
6500	815
7000	826
7500	832
8000	800
8500	825
9000	832
9500	823
10000	829
10300	834
10500	830
11000	828
11500	824
12000	830
12500	816
13000	818
13500	813
14000	818
14500	804
15000	812

Total Length (ft)	15022
Total Static Head (ft)	36



Channel Point Pump Station

Hydraulic Profile	
Station	Invert Elevation (ft)
0	846
250	850
500	860
750	880
1000	892
1250	908.1
1500	900
1750	890
2000	883
2250	887
2500	890
2750	900
3000	920
3250	930
3500	940
3625	966.1
Total Length (ft)	3625
Total Static Head (ft)	120.1

Gravity Sewer

Hydraulic Profile	
Station	Invert Elevation (ft)
0	966
250	946
500	940
750	930
1000	930
1250	920
1500	910
1750	900
2000	892.7
2250	890
2500	880
2750	880
3000	874.8
3250	874.8
3500	875.6
3750	870
4000	868
4250	862
4500	844.8
4750	840
5000	838
5250	836.4
5500	836.9
5750	834.8
6000	845.1
6250	860.9
6500	868.1
6750	880.8
7000	870
7250	860
7500	850
7750	840
8000	839.4
8250	833
8500	830
8750	828
9000	826
9250	824
9500	820

Choto Rd. Pump Station WETWELL SIZING

Pump	On (ft above lwl)	Off (ft above lwl)	delta h	avg elev (ft above lwl)
lead	3	0	3	1.5
lag	4	2	2	3

Lead Pump	
Cycle Time (min)	15
Number of Pumps	1
Incremental Flow (gpm)	1450.00
Volume (g)	5437.5
delta h	3
Plan area (ft2)	242.31

min height of water (ft)	3
--------------------------	---

Wet Well Depth (ft)	13
Plan Area (ft2)	244.40
Volume of Wet Well (gal)	23765.63
Average Volume (gal)	5437.5
Retention Time (min)	8.51

Lag Pump	
Cycle Time (min)	15
Number of Pumps	2
Incremental Flow (gpm)	1950.00
Volume (g)	3656.25
delta h	2
Plan area	244.40

Minimum Dimensions	
L (ft)	15.63
W (ft)	15.63

Channel Point Pump Station WETWELL SIZING

Pump	On (ft above lwl)	Off (ft above lwl)	delta h	avg elev (ft above lwl)
lead	3	0	3	1.5
lag	4	2	2	3

Lead Pump	
Cycle Time (min)	15
Number of Pumps	1
Incremental Flow (gpm)	400.00
Volume (g)	1500
delta h	3
Plan area (ft2)	66.84

Lag Pump	
Cycle Time (min)	15
Number of Pumps	2
Incremental Flow (gpm)	450.00
Volume (g)	843.75
delta h	2
Plan area	56.40

min height of water (ft)	3
--------------------------	---

Minimum Dimensions	
L (ft)	8.18
W (ft)	8.18

Wet Well Depth (ft)	15
Plan Area (ft2)	66.84
Volume of Wet Well (gal)	7500
Average Volume (gal)	1500
Retention Time (min)	31.56

Holder Branch Pump Station Evaluation

Existing Pumps - Operating Point	
Head	Flow
70 ft	750 gpm

Existing Avg Flow (gpm)	78
Existing Peak Design Flow (gpm)	460
Future Houses	2615
Low-Pressure Houses	351
Future Avg Flow (gpm)	401.65
Total Future Peak Flow (gpm)	1438.94

Total Capacity (gal)	20,868
----------------------	--------

Parameter	Existing	Proposed
Number of Pumps	2	2
Manufacturer	Gorman Rupp	Gorman Rupp
Model	T8A3-B	T8A3S-B
Design Point	750 gpm @ 70 ft	1475 gpm @ 72.5 ft
Speed (rpm)	1785	1250
Wetwell Diameter (ft)	8	8
Wetwell Depth (ft)	18.5	18.5
Wetwell Capacity (gal)	6956	6956
HP	50	50
Voltage	480	480
Force main size/material	12 in.	12 in.
Force main length	2630'	2630'
Average Daily Flow (gpm)	78	402
Peak Hourly Flow (gpm)	460	1440

Choto Pump Station Summary

Existing Discharge into choto Pump Station (gpd)	342000
Future Developments (Houses)	2615
Low-Pressure Houses	351
Persons per Household	3
Per Capita WW Demand (gpcd)	65
Population	8898
Peak Hourly/Design Avg	3
Minimum Daily/Design Avg	0.33

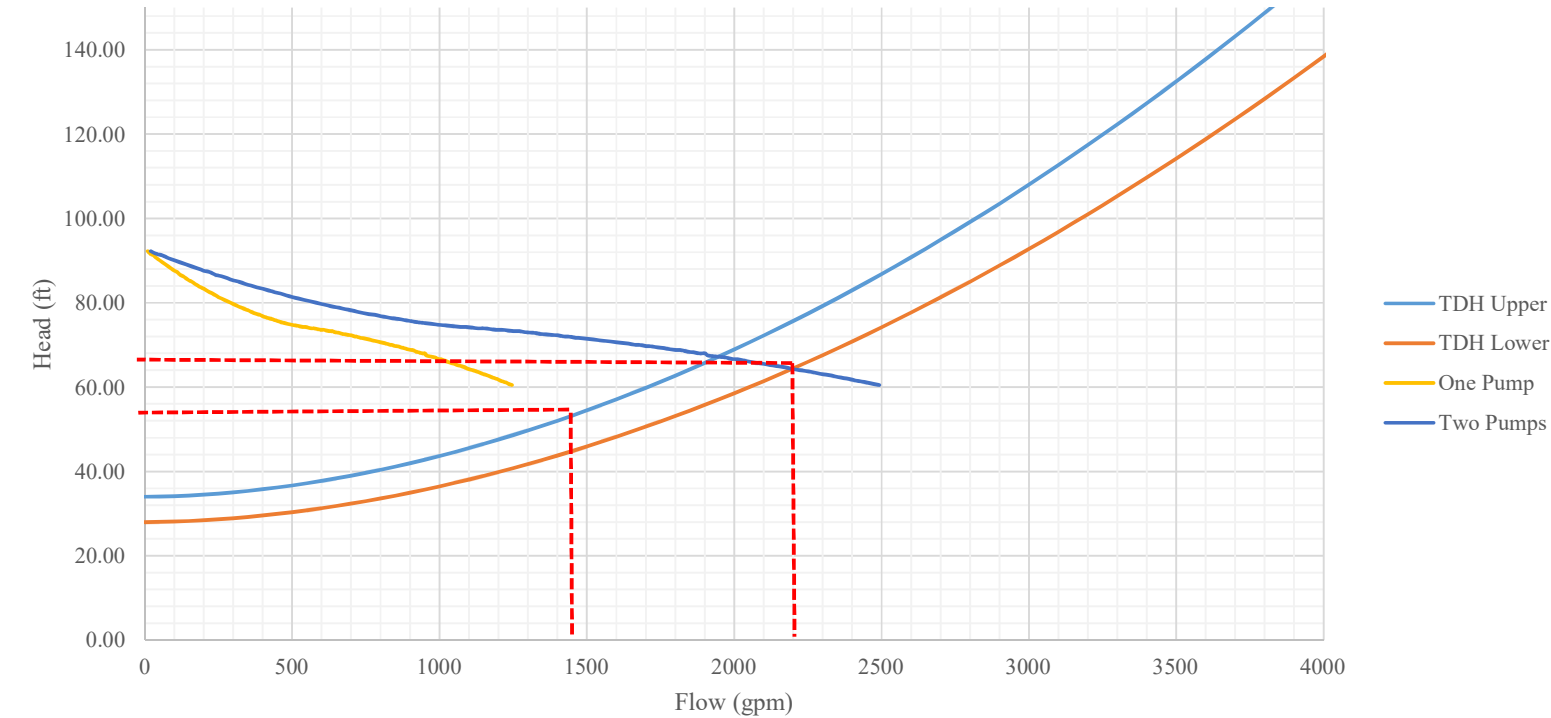
Average Low-Pressure Flow (gpm)	47.53
Average Future Flow (gpm)	354.11
Average Existing Flow (gpm)	237.5
Total Average Flow (gpm)	639.15

Peak Hourly Flow (gpm)	1917.44
------------------------	---------

Force Main Diameter Selected (in)	16
-----------------------------------	----

Total Length (ft)	15022
Total Static Head (ft)	36

Pump Manfr. :	Gorman Rupp
Model:	SF6D-X
Impeller Size:	9.75" Impeller
HP:	25 hp



Pump	On (ft above lwl)	Off (ft above lwl)	delta h	avg elev (ft above lwl)
lead	3	0	3	1.5
lag	4	2	2	3

Lead Pump	0
Cycle Time (min)	15
Number of Pumps	1
Incremental Flow (gpm)	1450
Volume (g)	5437.5
delta h	3
Plan area (ft2)	242.31

min height of water (ft)	3
--------------------------	---

Wet Well Depth (ft)	13
Plan Area (ft2)	244.40
Volume of Wet Well (gal)	23765.63
Average Volume (gal)	5437.50
Retention Time (min)	8.51

Lag Pump	0
Cycle Time (min)	15
Number of Pumps	2
Incremental Flow (gpm)	1950
Volume (g)	3656.25
delta h	2
Plan area	244.40

Minimum Dimensions	0
L (ft)	15.63
W (ft)	15.63

Parameter	Existing	Proposed
Number of Pumps	2	3
Manufacturer	Fairbanks Morse	Gorman-Rupp
Model	6-B5443	SF6D-X
Design Point (1 Pump)	1550 gpm @ 55 ft	1450 gpm @ 53 ft
Design Point (2 Pumps)	--	1950 gpm @ 66 ft
Impeller Size (in)	--	9.75
Wetwell Diameter (ft)	8	--
Wetwell Length (ft)	--	16
Wetwell Width (ft)	--	16
Wetwell Depth (ft)	20.9	13
Wetwell Capacity (gal)	23574	23,766
HP	50	25
Force Main Size/ Material	16"	16 in. DIP
Force Main Length (ft)	15022'	15,022
Average Daily Flow (gpm)	240	640
Peak Hourly Flow (gpm)	720	1918

Channel Point Pump Station Summary

Number of Houses in Section	351
Average Low-Pressure Flow (gpm)	47.53

Peak Hourly flow (gpm)	142.59
------------------------	--------

Force Main Diameter Selected (in)	8
-----------------------------------	---

Total Length (ft)	3625
Total Static Head (ft)	120.1

Pump Manfr. :	Gorman Rupp
Model:	SF4D-1
Impeller Size:	11 in
HP:	18.5 HP

Pump	On (ft above lwl)	Off (ft above lwl)	delta h	avg elev (ft above lwl)
lead	3	0	3	1.5
lag	4	2	2	3

Lead Pump	0
Cycle Time (min)	15
Number of Pumps	1
Incremental Flow (gpm)	400
Volume (g)	1500
delta h	3
Plan area (ft2)	66.84

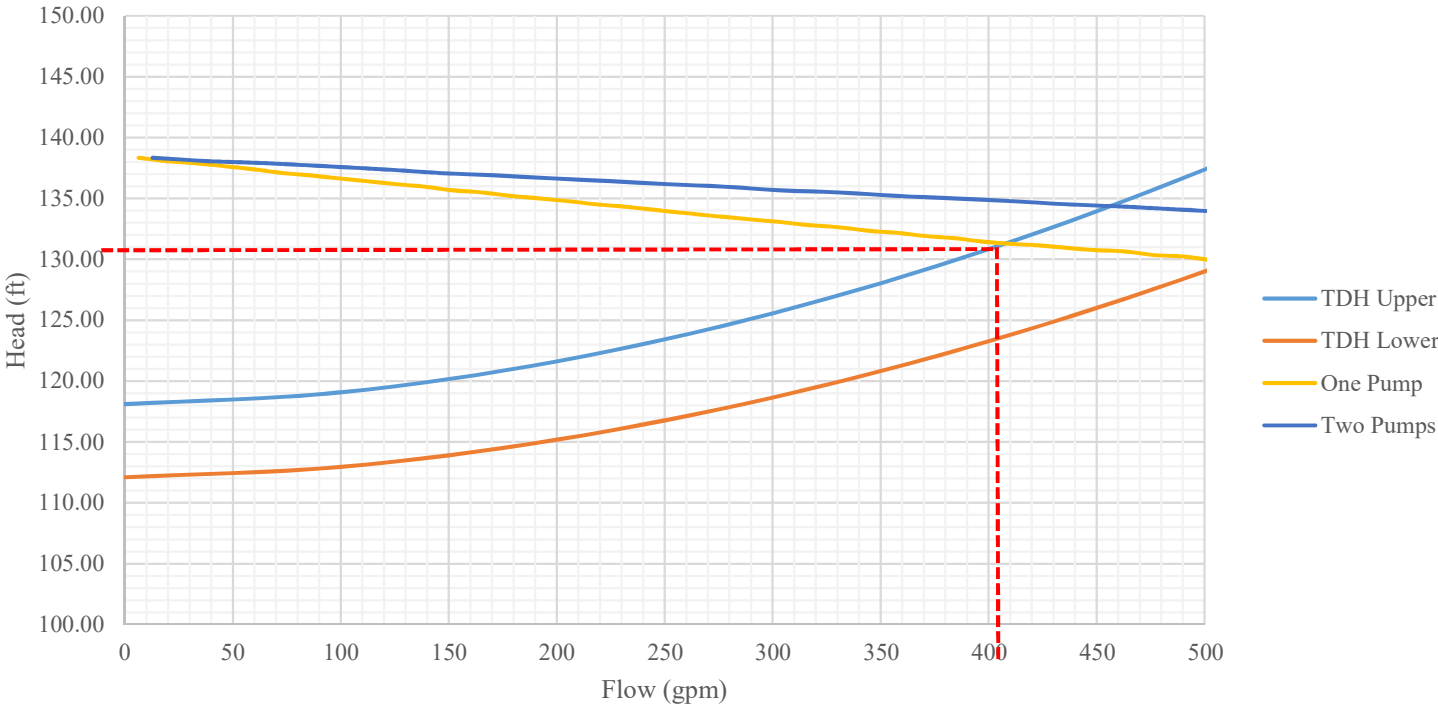
Lag Pump	0
Cycle Time (min)	15
Number of Pumps	2
Incremental Flow (gpm)	450
Volume (g)	843.75
delta h	2
Plan area	56.40

min height of water (ft)	3
--------------------------	---

Minimum Dimensions	0
L (ft)	8.18
W (ft)	8.18

Wet Well Depth (ft)	15
Plan Area (ft2)	66.84
Volume of Wet Well (gal)	7500
Average Volume (gal)	1500
Retention Time (min)	31.56

Parameter	Proposed
Number of Pumps	2
Manufacturer	Gorman-Rupp
Model	SF4D-1
Design Point (1 Pump)	370 gpm @132 ft
Impeller Size (in)	11
Wet well diameter (ft)	8
Wetwell Depth (ft)	15
Wetwell Capacity (gal)	797
HP	18.5
Force Main Size/Material	8 in. PVC
Force Main Length (ft)	3,625
Average Daily Flow (gpm)	48
Peak Hourly Flow (gpm)	143



Pump DESIGN FLOWS

$$1 \text{ Pump} = 1450 \text{ GPM}$$

$$2 \text{ Pumps} = 1950 \text{ GPM}$$

FORCE MAIN SIZING

Min Flow

$$V = \frac{Q}{A} = \frac{1450 \text{ G}}{\text{min}} \cdot \frac{1 \text{ Ft}^3}{7.48 \text{ G}} \cdot \frac{1 \text{ min}}{60 \text{ sec}} \cdot \frac{4}{\pi} \cdot \frac{1}{\left(\frac{16}{12}\right)^2} = \underline{2.3 \frac{\text{Ft}}{\text{s}}}$$

Max Flow

$$\frac{V}{A} = \frac{Q}{A} = \frac{1950}{7.48(60)} \cdot \frac{4}{\pi} \left(\frac{1}{\left(\frac{16}{12}\right)^2} \right) = \underline{3.11 \frac{\text{Ft}}{\text{s}}}$$

$$\text{Pump CYCLE TIME} = 15 \text{ min}$$

$$\text{INCREMENTAL FLOW} = 1950 \text{ GPM}$$

$$\# \text{ OF PUMPS} = 2$$

$$\Delta h = 2 \text{ Ft}$$

$$V = \frac{15 \text{ min} \cdot 1950 \text{ gpm}}{4 \cdot 2} = \underline{3656.25 \text{ gal}}$$

$$\text{Area} = \frac{3656.25 \text{ gal} \text{ Ft}^3}{7.48 \text{ gal} \cdot 2 \text{ Ft}} = \underline{244.4 \text{ Ft}^2}$$

$$\text{DIAMETER OF WELL} = \sqrt{\frac{4(244.4 \text{ Ft}^2)}{\pi}} = \underline{17.64'}$$

$$\text{TOTAL VOLUME} = Ah = 13 \text{ Ft} (244.4 \text{ Ft}^2) (7.48 \frac{\text{g}}{\text{Ft}^3})$$

$$V = \underline{23,765.6 \text{ gal}}$$

$$\text{RETENTION TIME } \theta = \frac{V}{Q} = \frac{5437.5 \text{ gal}}{639.15 \text{ gpm}} = \underline{8.5 \text{ min}}$$

EXISTING AVG. FLOW = 237.5 GPM
EXISTING PEAK FLOW = 1,425 GPM

PROPOSED FUTURE AVG = 640 GPM
PROPOSED FUTURE PEAK FLOW = 1,918 GPM

$$\text{STATIC HEAD} = \text{MAX ELEV} - \text{MIN ELEV} = 836' - 800' = \underline{\underline{36 \text{ Ft}}}$$

REQ'D HEAD AVG. FLOW

1 Pump Design flow = 1000 GPM
2 pumps Design flow = 2000 GPM

$$h_f = 10.47 L \left(\frac{Q}{C} \right)^{1.852} D^{-4.87}$$

$$h_f = 10.47 (15022 \text{ Ft}) \left(\frac{1000 \text{ GPM}}{140} \right)^{1.852} (16)^{-4.87}$$

$$h_f = 8.25 \text{ Ft}$$

$$h_f = 10.47 (15022 \text{ Ft}) \left(\frac{2000 \text{ GPM}}{140} \right)^{1.852} (16)^{-4.87}$$

$$h_f = 29.74 \text{ Ft}$$

MINOR LOSSES

$K = 5.06$ BASED ON STATION PIPING

$$h_m = \frac{K V^2}{2g} = (5.06) \left[\frac{\left(\frac{1000}{(7.48 \times 60)} \right)^2}{\frac{\pi}{4} \left(\frac{16 \text{ in}}{12 \text{ in}} \right)^2} \right] \left(\frac{1}{2(32.2 \frac{\text{Ft}}{\text{s}^2})} \right)$$

$$h_m = 0.20 \text{ Ft}$$

$$h_m = (5.06) \left[\frac{\left(\frac{2000}{(7.48 \times 60)} \right)^2}{\frac{\pi}{4} \left(\frac{16}{12} \right)^2} \right] \left(\frac{1}{2(32.2 \frac{\text{Ft}}{\text{s}^2})} \right)$$

$$h_m = 0.80 \text{ Ft}$$

$$\text{TDM 1 Pump} = 8.25 \text{ Ft} + 0.20 \text{ Ft} + 36 \text{ Ft} = 44.45 \text{ Ft}$$

$$\text{TDM 2 Pumps} = 29.74 \text{ Ft} + 0.80 \text{ Ft} + 36 \text{ Ft} = 66.54 \text{ Ft}$$

SF Series[®]
VARIOUS PATENTS APPLY

**Solids Handling Electric Motor Driven
Submersible Pump
Model
SF4D & SF4D-X
Size 4"**



APPROVED
X-PROOF MOTORS FOR CLASS I,
DIV. 1, GROUPS C&D LOCATIONS
(X-Proof Version Only)

The Gorman-Rupp SF Series[®] channel pumps provide superior pumping efficiency while maintaining a 3-inch (76,2 mm) spherical solids passage.

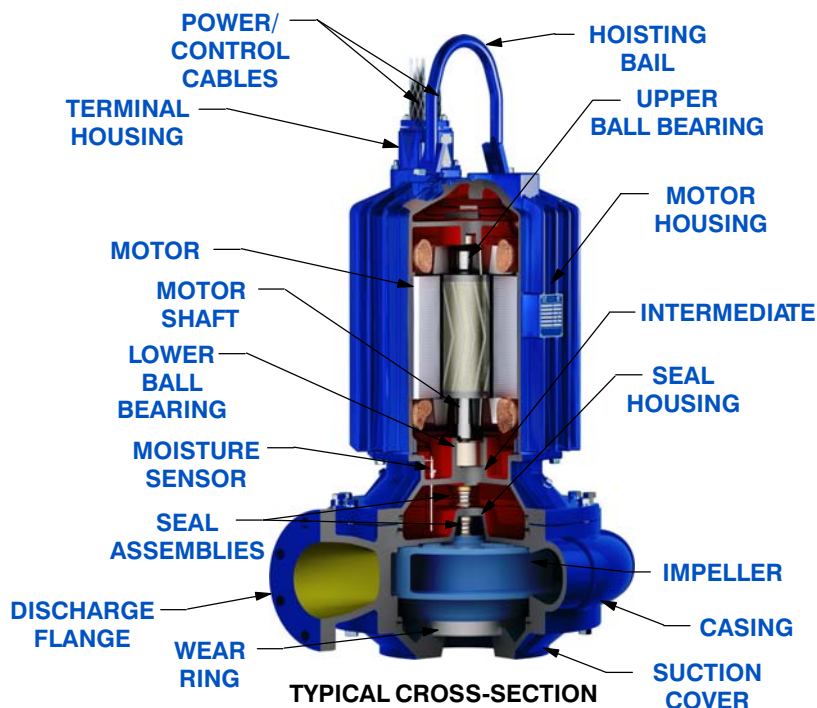
Key Features:

- NEMA premium efficiency motors
- Externally adjustable face clearance
- Press-fit motor with finned motor housing provides superior motor cooling, even in a submersed state, resulting in increased motor life
- NEMA Class H motor insulation
- Direct wired power cable leads
- Standard moisture detection in both the seal and motor chambers
- CSA-C/US and Factory Mutual (FM) Approved for Class I, Division I, Group C and D Haz Loc Applications (X-Proof Model)



Optional Accessories:

- G-R Hard Iron Impeller
- Control Panel (Consult [SF Series Motor Data Reference](#) for Customer-Supplied Control Panel)
- Submersible Transducer or Ball Type Float Switches
- Slide Rail Installation Components
 - Guide shoe ^w/Rubber Seal
 - Base Elbow and Rail Brackets
- Trash Stand Installation Components
 - Trash Stand
 - Discharge Elbow



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PUMP DETAILS

Pump Model	SF4D & SF4D-X	
Agency Approvals	Consult Factory	
Discharge Flange	4" ANSI w/ Flat Face	
Impeller Type / Material	Two Vane Enclosed / Ductile Iron	
Wear Ring Material	Ductile Iron	
Spherical Solids Handling	3" (76 mm) Non-Deformable Diameter	
Motor Housing (Including Seal Plate and Intermediate)	Gray Iron 30	
Pump Casing	Ductile Iron	
Motor Shaft	17-4 PH Stainless Steel	
O-Rings	Buna-N and Fluorocarbon (DuPont Viton® or Equivalent)	
Bearings: Type/Lubrication	Upper	Single Row Ball/Permanent Lubrication
	Lower	Double Row Ball/Permanent Lubrication
External Hardware	303/304 Stainless Steel	
External Surface Protection	Epoxy Paint	
Seal Type	Tandem, Mechanical, Oil Lubricated Upper, Self-Lubricated Lower	
Sealing Faces: Rotating/Stationary	Upper	Carbon/Ni-Resist
	Lower	Silicon Carbide/Silicon Carbide
Seal Elastomers	Buna-N Upper, Fluorocarbon (DuPont Viton® or Equivalent) Lower	
Max. Liquid Temperature	104°F (40°C)	

MOTOR DETAILS

Speed	60 Hz, 1750 RPM	
Motor Design	Inverter-Duty Rated, Air-Filled Enclosure, Squirrel Cage, Induction Start	
Motor Efficiency Rating	NEMA Premium Efficiency (IEC IE3)	
Insulation	Class H, Rated 356°F (180°C)	
Max. Submergence	65 Feet (20 Meters)	
Min. Submergence	1/2 Motor Covered	
Max. Starts per Hour	10	
Max. Rated Output Power	75.0 HP (55.9 kW)	
Amp Draw @ Max. Rated Power: Full Load/ Locked Rotor		
	460V/3P	86.0/542.0
	575V/3P	69.0/434.0
NEMA Motor Code	G	
Service Factor	1.15	
Efficiency: 100%/75%/50% Load	94%/96%/94%	
Power Factor: 100%/75%/50% Load	0.85/0.81/0.73	
Field Connection	Across-the-Line	
Moisture Sensor	Single Probe Monitoring Both Seal and Motor Chambers	
Thermal Overload	One Thermostat per Phase, Self-Resetting	

CABLE DETAILS

Voltage		460V	575V
Power Cable	No. Cables/Conductors per	1/6	1/6
	Gauge	(3) 1 AWG PWR & (3) 6 AWG GND/GC	(3) 6 AWG PWR & (3) 10 AWG GND/GC
	Type/Material	GGC/CPE	GGC/CPE
	Nominal Cable O.D.	1.5" (38.1 mm)	1.05" (26.7 mm)
Control Cable	No. Cables/Conductors per	1/4	
	Gauge	14 AWG	
	Type/Material	SOOW/CPE	
	Nominal Cable O.D.	0.61" (15.5 mm)	
Length		32 Feet (10 Meters) Standard, 164 Feet (50 Meters) Maximum	



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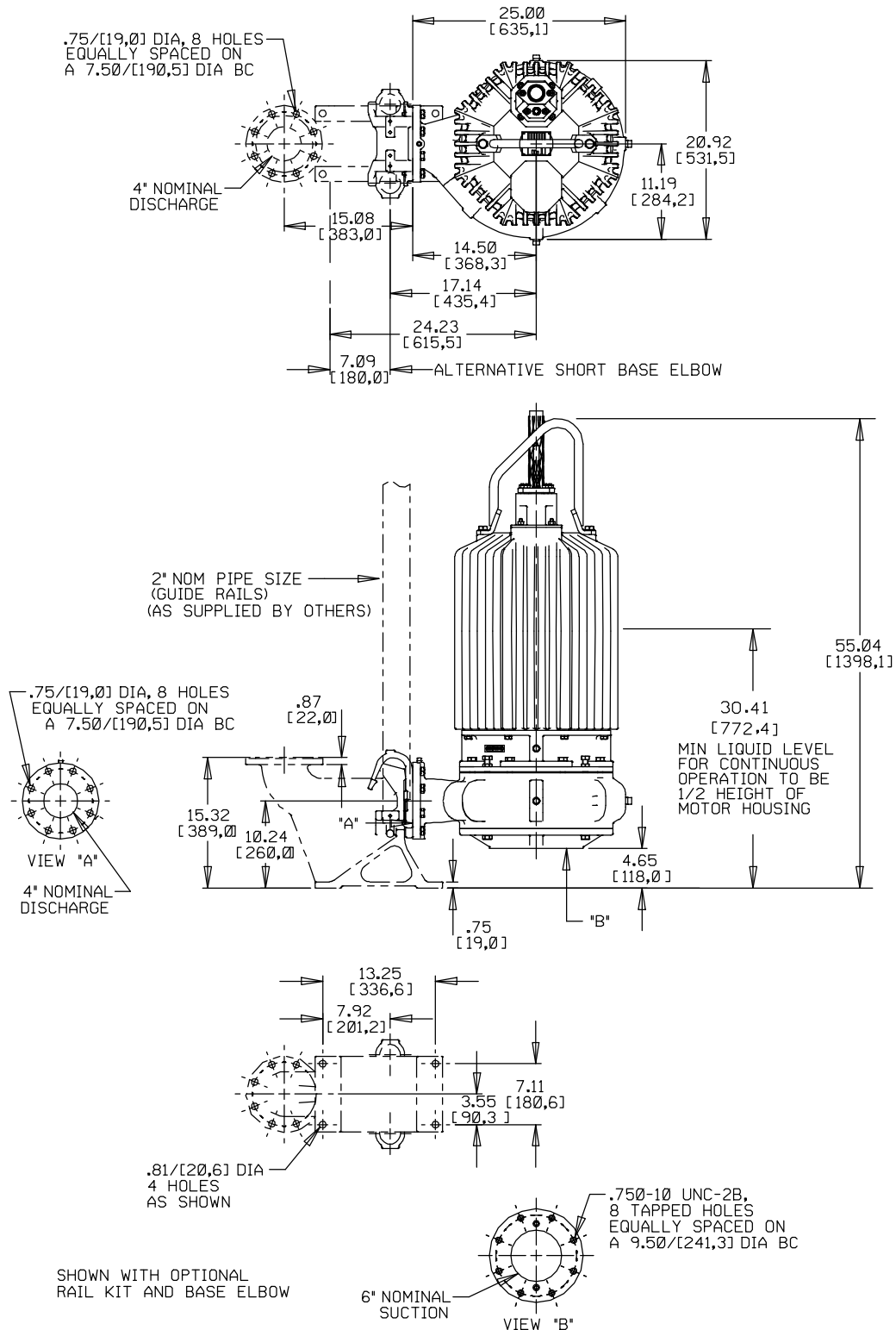
Specification Data

SECTION 133, PAGE 230.2

**MODEL SF4D & SF4D-X
SLIDE RAIL VERSION
APPROXIMATE
DIMENSIONS and WEIGHTS**

NET WT: (pump only)
(comb. cable wt. per meter [3.2 ft.])
SHIPPING WT: (pump only)
CRATE SIZE:

1316 LBS. [596,9 KG.]
3.4 LBS. [1,54 KG.]
1366 LBS. [619,6 KG.]
29.5 CU. FT. [0,83 CU. M.]



DIMENSIONS:
INCHES
[MILLIMETERS]

SHOWN WITH OPTIONAL
RAIL KIT AND BASE ELBOW



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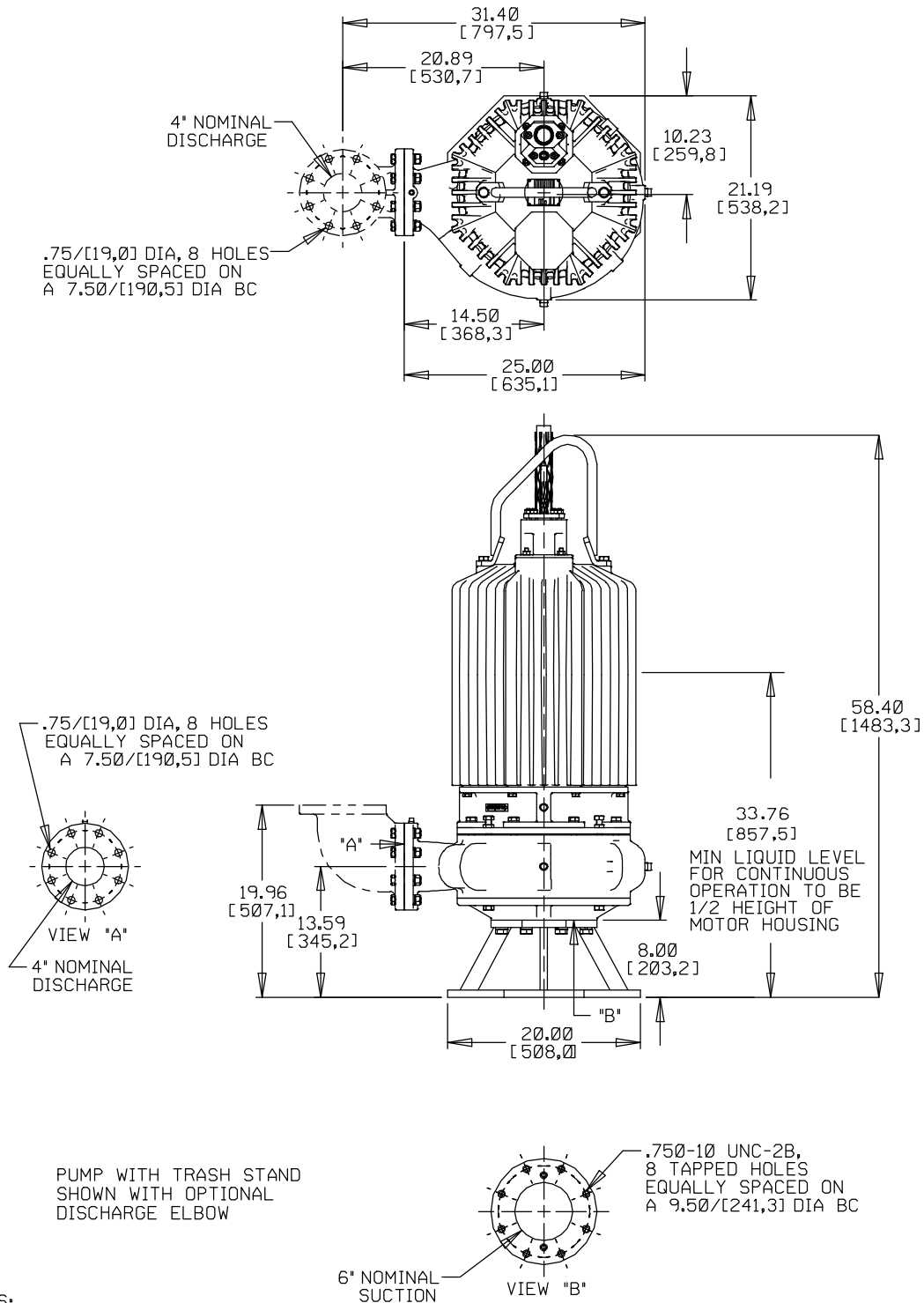
Specification Data

SECTION 133, PAGE 230.3

**MODEL SF4D & SF4D-X
TRASH VERSION
APPROXIMATE
DIMENSIONS and WEIGHTS**

NET WT: (pump only)
(comb. cable wt. per meter [3.2 ft.])
SHIPPING WT: (pump only)
CRATE SIZE:

1371 LBS. [621,9 KG.]
3.4 LBS. [1,54 KG.]
1421 LBS. [644,5 KG.]
31.4 CU. FT. [0,89 CU. M.]



DIMENSIONS:
INCHES
[MILLIMETERS]

PUMP WITH TRASH STAND
SHOWN WITH OPTIONAL
DISCHARGE ELBOW

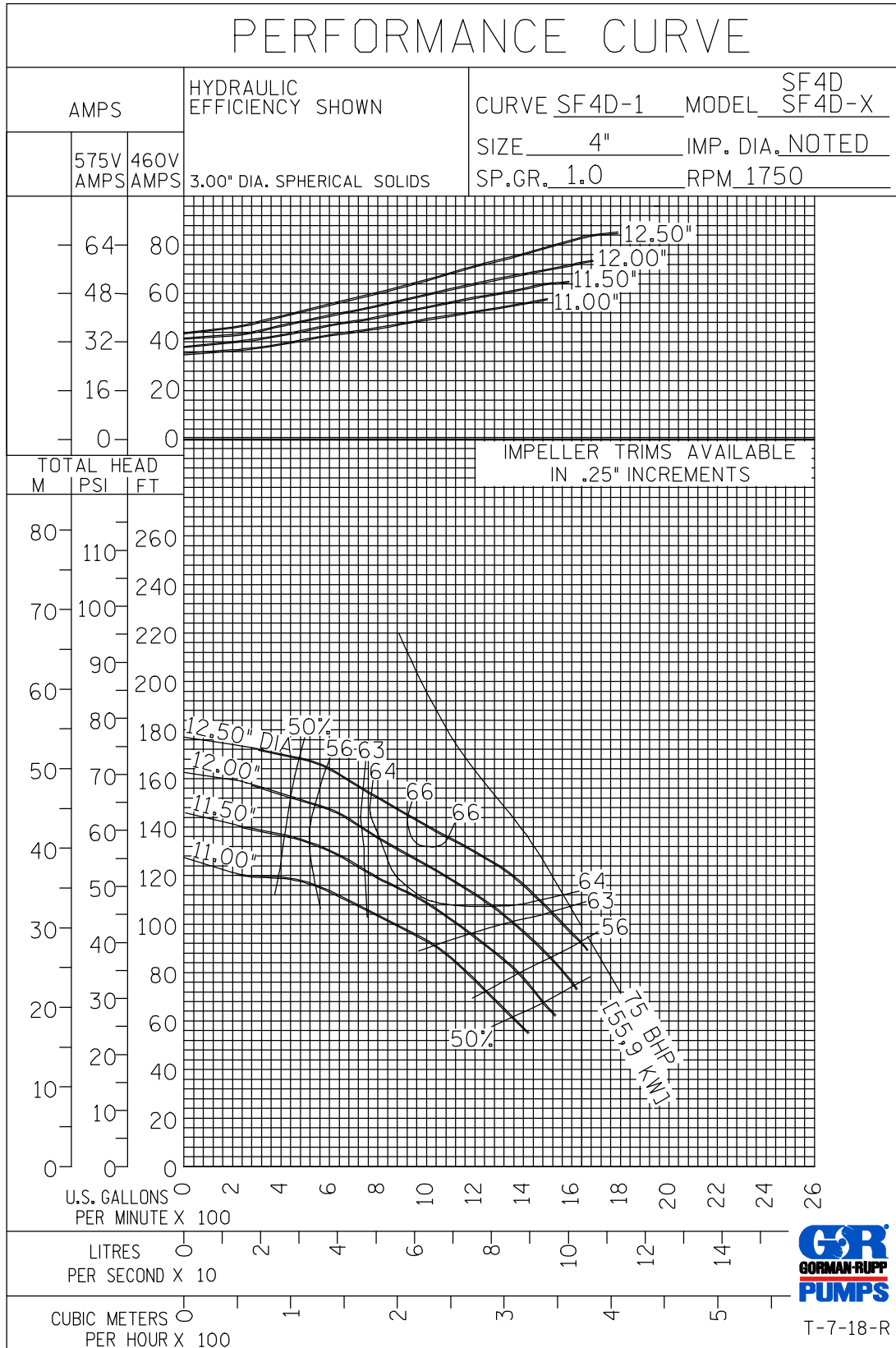


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SF Series[®]
VARIOUS PATENTS APPLY

**Solids Handling Electric Motor Driven
Submersible Pump
Models
SF6D & SF6D-X
Size 6"**



APPROVED
X-PROOF MOTORS FOR CLASS I,
DIV. 1, GROUPS C&D LOCATIONS
(X-Proof Version Only)

The Gorman-Rupp SF Series[®] channel pumps provide superior pumping efficiency while maintaining a 3-inch (76,2 mm) spherical solids passage.

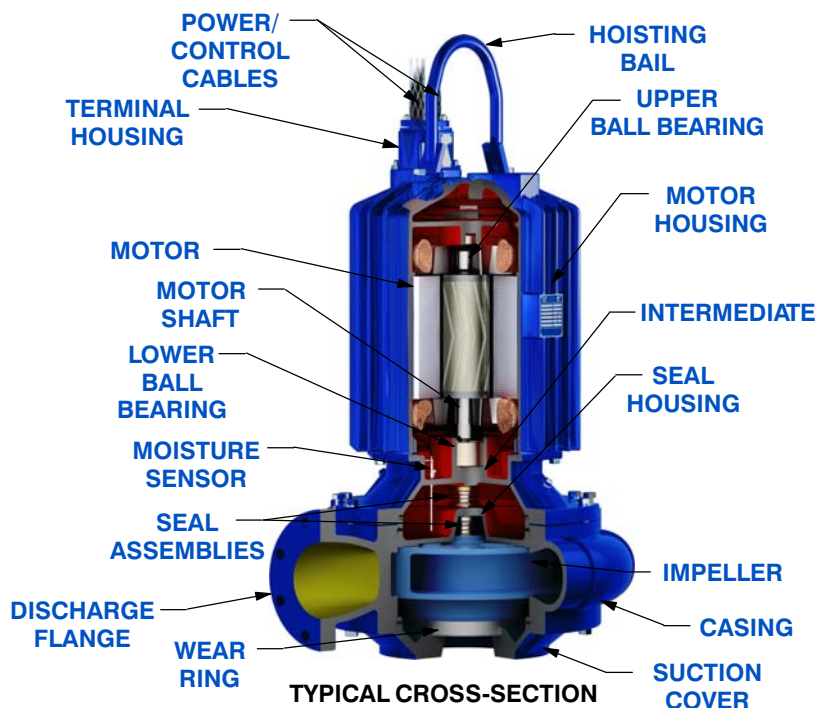
Key Features:

- NEMA premium efficiency motors
- Externally adjustable face clearance
- Press-fit motor with finned motor housing provides superior motor cooling, even in a submersed state, resulting in increased motor life
- NEMA Class H motor insulation
- Direct wired power cable leads
- Standard moisture detection in both the seal and motor chambers
- CSA-C/US and Factory Mutual (FM) Approved for Class I, Division I, Group C and D Haz Loc Applications (X-Proof Model)



Optional Accessories:

- G-R Hard Iron Impeller
- Control Panel (Consult [SF Series Motor Data Reference](#) for Customer-Supplied Control Panel)
- Submersible Transducer or Ball Type Float Switches
- Slide Rail Installation Components
 - Guide shoe w/Rubber Seal
 - Base Elbow and Rail Brackets
- Trash Stand Installation Components
 - Trash Stand
 - Discharge Elbow



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PUMP DETAILS

Pump Model	SF6D & SF6D-X	
Agency Approvals	Consult Factory	
Discharge Flange	6" ANSI w/ Flat Face	
Impeller Type / Material	Two Vane Enclosed/ Ductile Iron	
Wear Ring Material	Ductile Iron	
Spherical Solids Handling	3" (76 mm) Non-Deformable Diameter	
Motor Housing (Including Seal Plate and Intermediate)	Gray Iron 30	
Pump Casing	Ductile Iron	
Motor Shaft	17-4 PH Stainless Steel	
O-Rings	Buna-N and Fluorocarbon (DuPont Viton® or Equivalent)	
Bearings: Type/Lubrication	Upper	Single Row Ball/Permanent Lubrication
	Lower	Double Row Ball/Permanent Lubrication
External Hardware	303/304 Stainless Steel	
External Surface Protection	Epoxy Paint	
Seal Type	Tandem, Mechanical, Oil Lubricated Upper, Self-Lubricated Lower	
Sealing Faces: Rotating/Stationary	Upper	Carbon/Ni-Resist
	Lower	Silicon Carbide/Silicon Carbide
Seal Elastomers	Buna-N Upper, Fluorocarbon (DuPont Viton® or Equivalent) Lower	
Max. Liquid Temperature	104°F (40°C)	

MOTOR DETAILS

Speed	60 Hz, 1750 RPM	
Motor Design	Inverter-Duty Rated, Air-Filled Enclosure, Squirrel Cage, Induction Start	
Motor Efficiency Rating	NEMA Premium Efficiency (IEC IE3)	
Insulation	Class H, Rated 356°F (180°C)	
Max. Submergence	65 Feet (20 Meters)	
Min. Submergence	1/2 Motor Covered	
Max. Starts per Hour	10	
Max. Rated Output Power	50.0 HP (37,3 kW)	
Amp Draw @ Max. Rated Power: Full Load/ Locked Rotor		
	460V/3P	60.0/425.0
	575V/3P	48.0/340.0
NEMA Motor Code	H	
Service Factor	1.15	
Efficiency: 100%/75%/50% Load	93%/90%/88%	
Power Factor: 100%/75%/50% Load	0.86/0.77/0.68	
Field Connection	Across-the-Line	
Moisture Sensor	Single Probe Monitoring Both Seal and Motor Chambers	
Thermal Overload	One Thermostat per Phase, Self-Resetting	

CABLE DETAILS

Power Cable	Voltage	All
	No. Cables/Conductors per Gauge	1/6
	Type/Material	(3) 6 AWG PWR & (3) 10 AWG GND/GC
	Nominal Cable O.D.	GGC/CPE 1.05" (26,7 mm)
Control Cable	No. Cables/Conductors per Gauge	1/4
	Type/Material	14 AWG
	Nominal Cable O.D.	SOOW/CPE 0.61" (15,5 mm)
	Length	32 Feet (10 Meters) Standard, 164 Feet (50 Meters) Maximum



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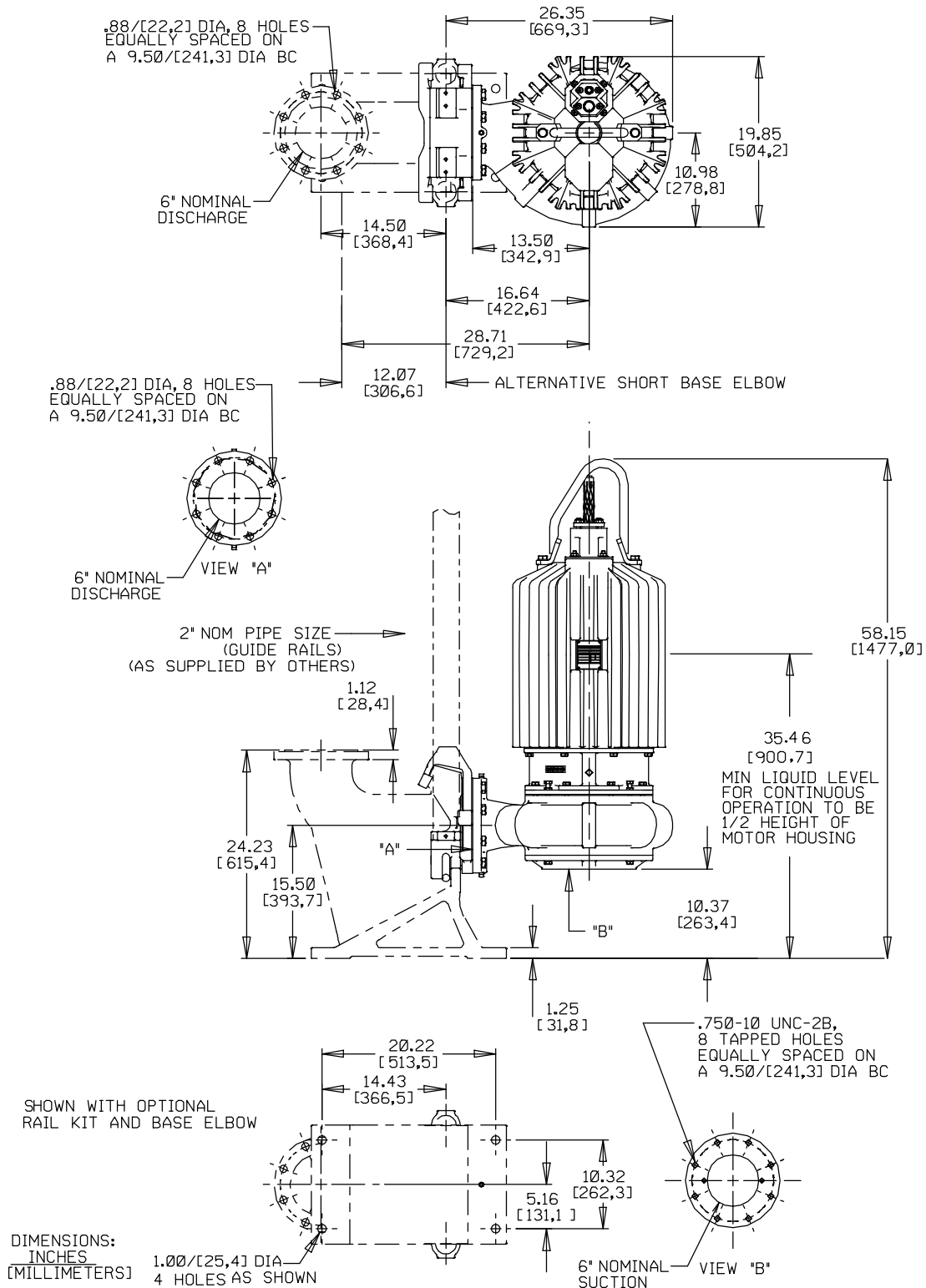
Specification Data

SECTION 133, PAGE 330.2

MODEL SF6D & SF6D-X SLIDE RAIL VERSION APPROXIMATE DIMENSIONS and WEIGHTS

NET WT: (pump only)
(comb. cable wt. per meter [3.2 ft.])
SHIPPING WT: (pump only)
CRATE SIZE:

1075 LBS. [487,6 KG.]
3.4 LBS. [1,54 KG.]
1175 LBS. [533 KG.]
31 CU. FT. [0,88 CU. M.]



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SECTION 133, PAGE 330.3

NET WT: (pump only)
(comb. cable wt. per meter [3.2 ft.])
SHIPPING WT: (pump only)
CRATE SIZE:

1180 LBS. [525,2 KG.]
3.4 LBS. [1,54 KG.]
1330 LBS. [603,3 KG.]
28.5 CU. FT. [0,81 CU. M.]

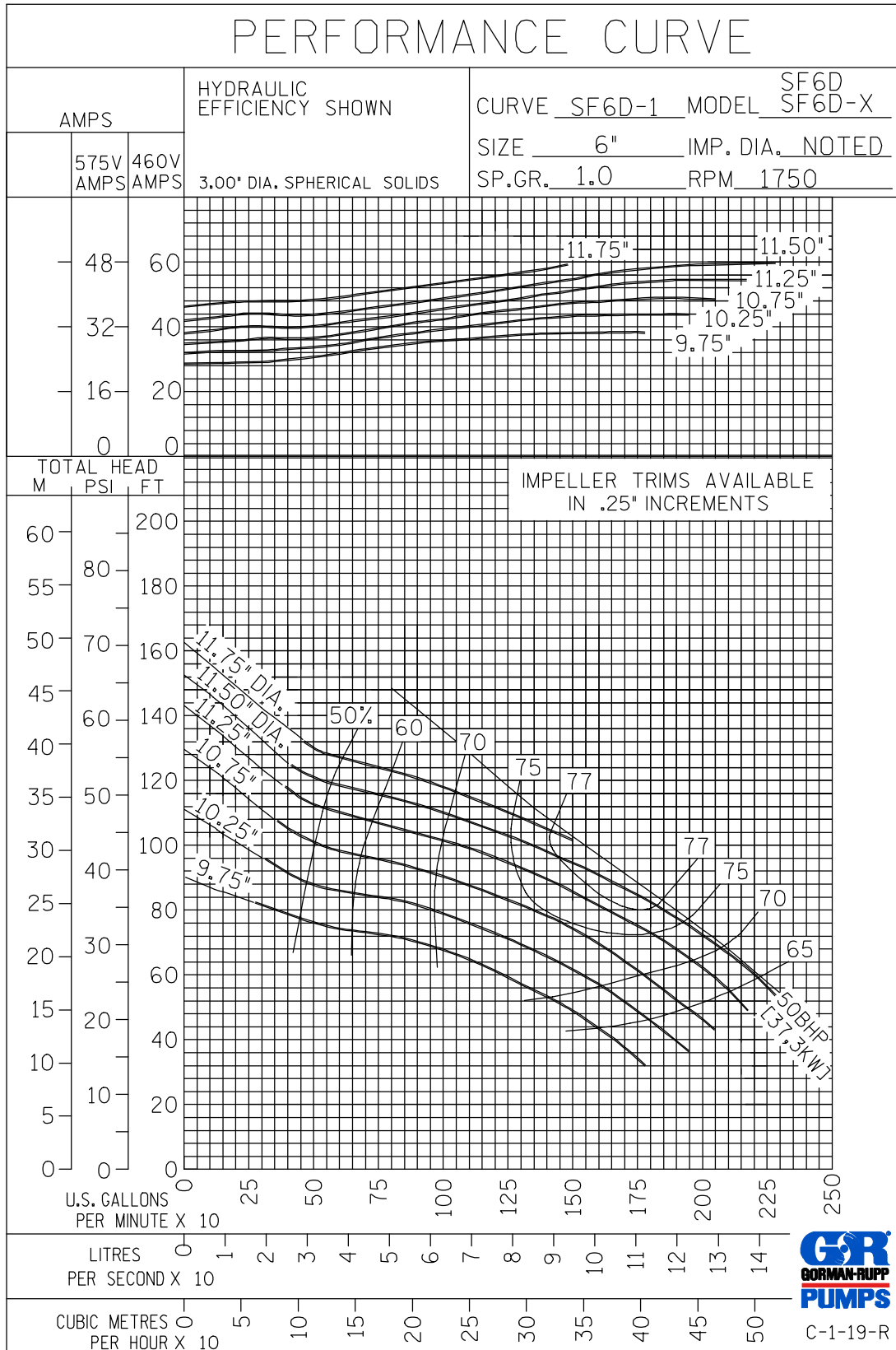


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Self Priming Centrifugal Pump

**Super
T SERIES**

VARIOUS PATENTS APPLY

Model T8A3S-B

Size 8" x 8"



PUMP SPECIFICATIONS

Size: 8" (203 mm) ASA Flanged Suction x 8" (203 mm) NPT Female Discharge.

Casing: Gray Iron 30.

Maximum Operating Pressure 83 psi (572 kPa).*

Semi-Open Type, Two Vane Impeller: Ductile Iron 65-45-12.

Handles 3" (76.2 mm) Diameter Spherical Solids.

Impeller Shaft: Alloy Steel 4150.

Shaft Sleeve: Alloy Steel 4130.

Replaceable Wear Plate: Gray Iron 30.

Removable Adjustable Cover Plate: Gray Iron 30; 94 lbs. (43 kg).

Flap Valve: Neoprene w/Nylon and Steel Reinforcing.

Seal Plate: Gray Iron 30.

Radial and Thrust Bearings: Open Double Row Ball.

Bearing Housing: Gray Iron 30.

Bearing and Seal Cavity Lubrication: SAE 30 Non-Detergent Oil.

Flanges: Gray Iron 30.

Gaskets: Buna-N, Compressed Synthetic Fibers, PTFE, Cork and Rubber.

O-Rings: Buna-N.

Hardware: Standard Plated Steel.

Brass Pressure Relief Valve.

Bearing and Seal Cavity Oil Level Sight Gauges.

Optional Equipment: Metal Bellows Seal. Automatic Air Release Valve. High Pump Temperature Shutdown Kit. 120V/240V Casing Heater.

Gray Iron 30 Spool Flanges:

8" ASA Discharge (**Specify Model T8A3S-B /F**).

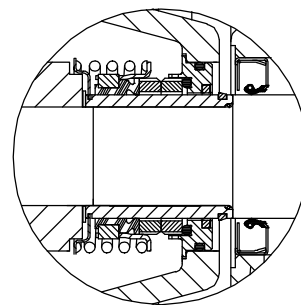
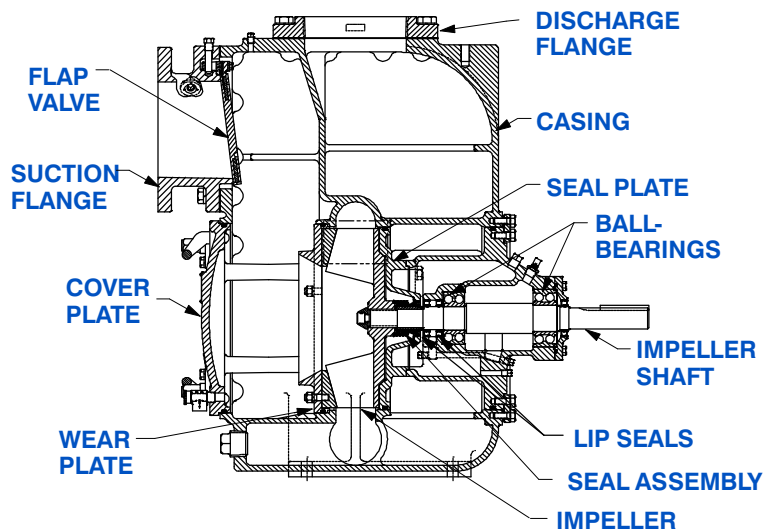
200 mm DIN 2527 (PN 16) Suction and Discharge

(**Specify Model T8A3S-B /FM**).

**Consult Factory for Applications Exceeding Maximum Pressure and/or Temperature Indicated.*



Shown with Optional Discharge Spool Flange (Available in ASA or DIN Standard Sizes).



SEAL DETAIL

Cartridge Type, Mechanical, Oil-Lubricated, Double Floating, Self-Aligning. Tungsten Titanium Carbide Rotating and Stationary Faces. Stainless Steel 316 Stationary Seat. Fluorocarbon Elastomers (DuPont Viton® or Equivalent). Stainless Steel 18-8 Cage and Spring. Maximum Temperature of Liquid Pumped, 160°F (71°C).*



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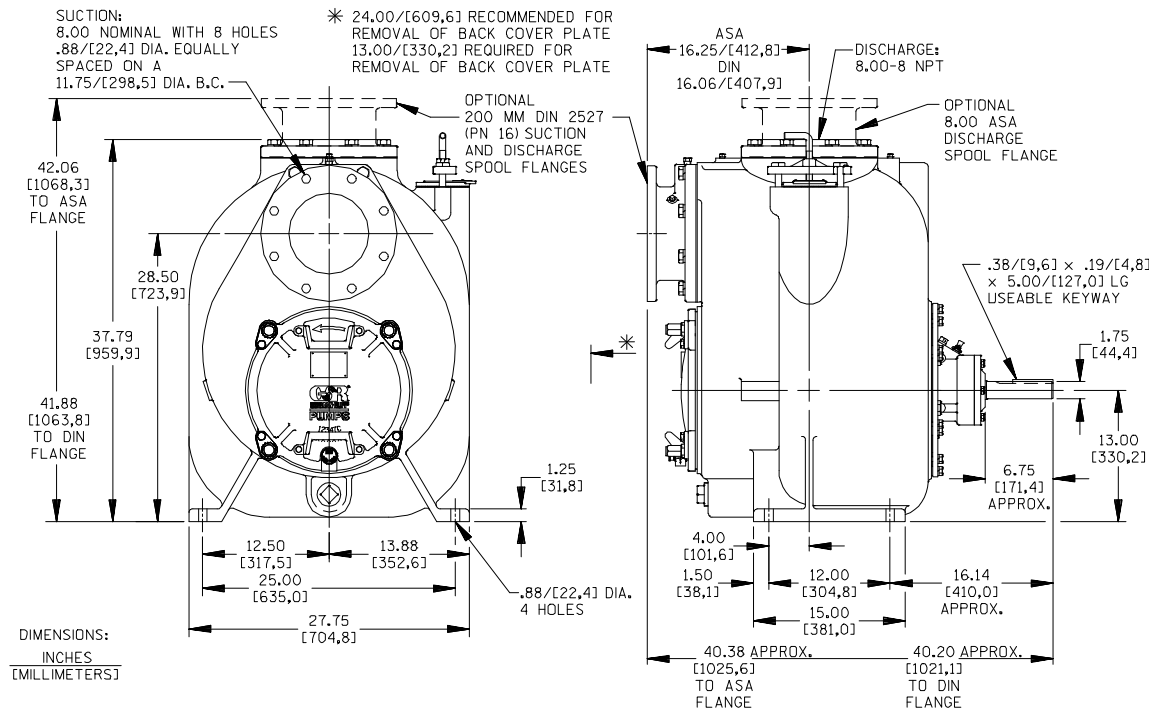
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Specification Data

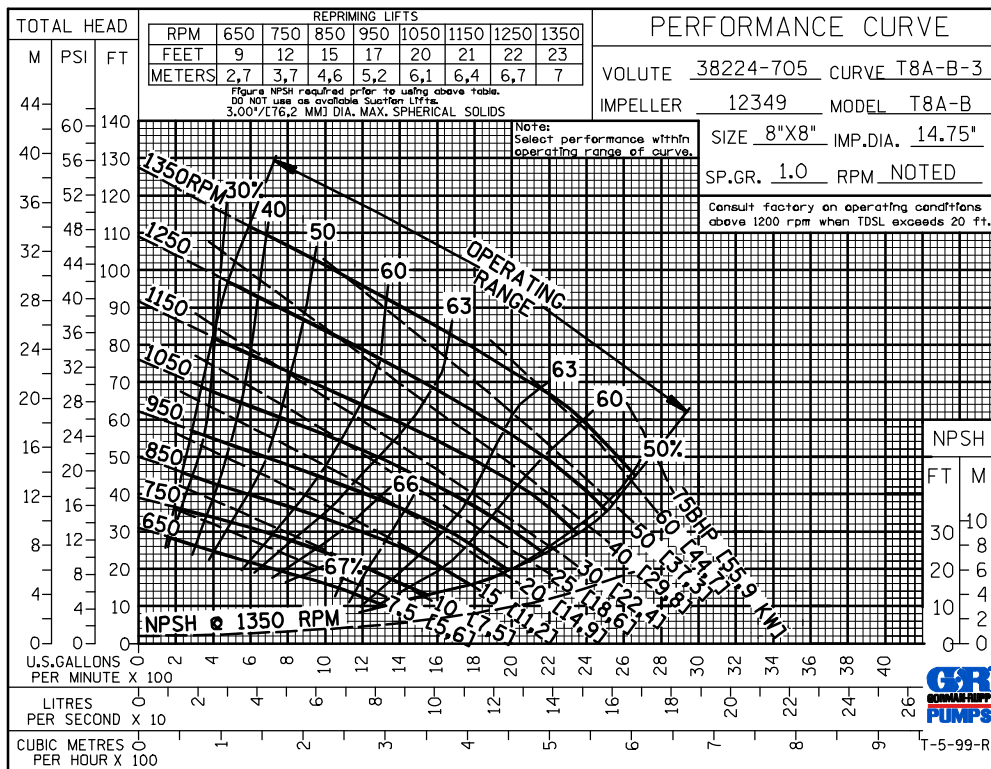
SECTION 55, PAGE 2300

APPROXIMATE DIMENSIONS and WEIGHTS

NET WEIGHT: 1295 LBS. (587 KG.)*
SHIPPING WEIGHT: 1399 LBS. (635 KG.)*
EXPORT CRATE: 47.9 CU. FT. (1,36 CU. M.)
***ADD 35 LBS. (15,9 KG.) W/EACH SPOOL FLANGE**



OPTIONAL ASA OR DIN STANDARD SPOOL FLANGES AVAILABLE



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Appendix E. Gravity Sewer Section Calculations Along Choto Rd.

APPENDIX E

Given			
Sewer Demand	=	39000	gpd
Infiltration	=	500	gpd/mile/in-dia
Minimum depth of cover to crown	=	5	ft
Crown elevation at MH1		937.0	ft
n		0.011	-
1 cfs		646,272	gpd
Assumptions			
Qmax at 0.8*dia			
$d = 1.56 [n Q / K S^{0.5}]^{3/8}$ at Qmax			

DESIGN CALCULATION TABLES

Table E.1. Manhole Elevation and Slope

Elevation		Δ	Slope
Upper	Lower		
966.1	942.5	23.6	0.0656
942.5	930.5	12.0	0.0333
930.5	927.6	2.9	0.0081
927.6	912.4	15.2	0.0422
912.4	898.4	14.0	0.0389
898.4	891.3	7.1	0.0197
891.3	879.5	11.8	0.0328
879.5	877.5	2.0	0.0056
877.5	874.8	2.7	0.0075
874.8	873.5	1.3	0.0036
873.5	868.5	5.0	0.0139
868.5	857.4	11.1	0.0308
857.4	841.0	16.4	0.0456
841.0	837.6	3.4	0.0094
837.6	836.8	0.8	0.0022
836.8	835.0	1.8	0.0050

Table E.3. Pipe Profile Elevations

Manhole #	Street Elevation (ft)		Crown Elevation (ft)		Invert Elevation (ft)	
	Upper	Lower	Upper	Lower	Upper	Lower
0	966.1	942.5	960.1	936.5	952.1	899.5
1	942.5	930.5	936.5	924.5	928.5	887.5
2	930.5	927.6	924.5	921.6	916.5	884.6
3	927.6	912.4	921.6	906.4	913.6	869.4
4	912.4	898.4	906.4	892.4	898.4	855.4
5	898.4	891.3	892.4	885.3	884.4	848.3
6	891.3	879.5	885.3	873.5	877.3	836.5
7	879.5	877.5	873.5	871.5	865.5	834.5
8	877.5	874.8	871.5	868.8	863.5	831.8
9	874.8	873.5	868.8	867.5	856.8	820.5
10	873.5	868.5	867.5	862.5	853.5	809.5
11	868.5	857.4	862.5	851.4	848.5	798.4
12	857.4	841.0	851.4	835.0	837.4	782.0
13	841.0	837.6	835.0	831.6	821.0	778.6
14	837.6	836.8	831.6	830.8	817.6	777.8
15	836.8	835.0	830.8	829.0	816.8	776.0

Table E.2. Gravity Sewer Design Pipe Diameter Calculations

Pipe #	Manhole		Elevation		Pipe Length	Street Slope	Min. Slope	Design Dia.	Average Sewage Flow	Average Sewage Flow	Infil.	Cum. Infiltration Flow	Infiltration Flow	Q _{avg}
	Upper	Lower	Upper	Lower	(ft)	(ft/ft)	(ft/ft)	(in)	(gpd)	(cfs)	(gpd)	(gpd)	(cfs)	(cfs)
A	0	1	966.1	942.5	360	0.066	0.004	8	39000	0.060	2.73	2.73	0.000004	0.060
B	1	2	942.5	930.5	360	0.033	0.004	8	50800	0.079	2.73	5.45	0.000008	0.079
C	2	3	930.5	927.6	360	0.008	0.004	8	63600	0.098	2.73	8.18	0.000013	0.098
D	3	4	927.6	912.4	360	0.042	0.004	8	63600	0.098	2.73	10.91	0.000017	0.098
E	4	5	912.4	898.4	360	0.039	0.004	8	63600	0.098	2.73	13.64	0.000021	0.098
F	5	6	898.4	891.3	360	0.020	0.004	8	63600	0.098	2.73	16.36	0.000025	0.098
G	6	7	891.3	879.5	360	0.033	0.004	8	63600	0.098	2.73	19.09	0.000030	0.098
H	7	8	879.5	877.5	360	0.006	0.004	8	63600	0.098	2.73	21.82	0.000034	0.098
I	8	9	877.5	874.8	360	0.008	0.002	12	97600	0.151	4.09	25.91	0.000040	0.151
J	9	10	874.8	873.5	360	0.004	0.002	14	97600	0.151	4.77	30.68	0.000047	0.151
K	10	11	873.5	868.5	360	0.014	0.002	14	97600	0.151	4.77	35.45	0.000055	0.151
L	11	12	868.5	857.4	360	0.031	0.002	14	97600	0.151	4.77	40.23	0.000062	0.151
M	12	13	857.4	841.0	360	0.046	0.002	14	97600	0.151	4.77	45.00	0.000070	0.151
N	13	14	841.0	837.6	360	0.009	0.002	14	97600	0.151	4.77	49.77	0.000077	0.151
O	14	15	837.6	836.8	360	0.002	0.002	14	97600	0.151	4.77	54.55	0.000084	0.151
P	15	16	836.8	835.0	360	0.005	0.002	14	97600	0.151	4.77	59.32	0.000092	0.151
Q	16	PS	835.0	825.0	360	0.028	0.002	14	97600	0.151	4.77	64.09	0.000099	0.151

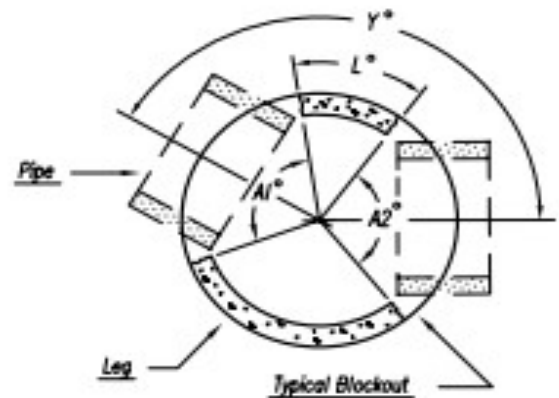
***For 80% full**

Q _{design}	d _{cal}	d _{set}	Q _{full}	V _{full}	Q _{des} ÷ Q _{full}	y/d	θ	A	P	R	AR ^{2/3}	S ^{1/2}	Q _{trial} - Q _{des}	Q _{trial}	V _{design}
(cfs)	(in)	(in)	(cfs)	(fps)	(cfs)			(ft ²)	(ft)	(ft)			(cfs)	(cfs)	(fps)
0.241	2.91	8	3.66	10.48	0.066	0.115	1.381	0.022	0.460	0.048	0.003	0.256	-0.140	0.102	4.58
0.314	3.65	8	2.61	7.47	0.121	0.160	1.647	0.036	0.549	0.066	0.006	0.183	-0.169	0.145	4.02
0.394	5.18	8	1.28	3.67	0.307	0.185	1.778	0.044	0.593	0.075	0.008	0.090	-0.298	0.096	2.16
0.394	3.79	8	2.93	8.41	0.134	0.214	1.924	0.055	0.641	0.085	0.011	0.205	-0.099	0.295	5.38
0.394	3.85	8	2.82	8.07	0.140	0.285	2.255	0.082	0.752	0.109	0.019	0.197	0.107	0.501	6.09
0.394	4.38	8	2.01	5.75	0.196	0.314	2.378	0.094	0.793	0.118	0.023	0.140	0.034	0.428	4.57
0.394	3.98	8	2.59	7.41	0.152	0.320	2.406	0.096	0.802	0.120	0.023	0.181	0.181	0.574	5.96
0.394	5.55	8	1.06	3.05	0.370	0.344	2.505	0.106	0.835	0.127	0.027	0.075	-0.123	0.270	2.55
0.604	6.16	12	3.65	4.64	0.166	0.134	1.499	0.063	0.749	0.084	0.012	0.087	-0.464	0.140	2.24
0.604	7.07	14	3.82	3.57	0.158	0.187	1.791	0.139	1.045	0.133	0.036	0.060	-0.311	0.293	2.11
0.604	5.49	14	7.49	7.00	0.081	0.194	1.825	0.146	1.064	0.137	0.039	0.118	0.012	0.616	4.23
0.604	4.73	14	11.15	10.43	0.054	0.224	1.971	0.179	1.150	0.155	0.052	0.176	0.621	1.225	6.86
0.604	4.39	14	13.56	12.68	0.045	0.275	2.207	0.239	1.287	0.185	0.078	0.213	1.632	2.236	9.37
0.604	5.90	14	6.17	5.77	0.098	0.302	2.326	0.272	1.357	0.200	0.093	0.097	0.618	1.222	4.50
0.604	7.74	14	2.99	2.80	0.202	0.312	2.370	0.285	1.383	0.206	0.099	0.047	0.028	0.632	2.22
0.604	6.65	14	4.49	4.20	0.135	0.334	2.467	0.313	1.439	0.218	0.113	0.071	0.480	1.084	3.46
0.604	4.82	14	10.59	9.90	0.057	0.350	2.532	0.333	1.477	0.226	0.124	0.167	2.179	2.783	8.35

PIPE SIZE	A ° PER MANHOLE BARREL SIZE						BLOCKOUT HEIGHT
	48"	60"	72"	84"	96"	120"	
	deg	deg	deg	deg	deg	deg	ft
<u>ROUND PIPE</u>							
12	52	41	34	29	26	20	1.7
15	60	47	39	33	29	23	2.0
18	72	56	46	39	34	27	2.3
24	94	72	59	49	43	34	2.9
30	122	89	72	60	52	41	3.5
36	*	110	86	72	62	48	4.1
42		138	102	84	72	56	4.7
* * 48		*	122	97	82	64	5.3
* * 54			153	113	94	72	5.8
* * 60			*	133	107	80	6.4
<u>ARCH PIPE</u>							
18	84	65	53	45	39	31	2.0
24	113	84	68	57	49	39	2.5
30	*	107	84	70	60	47	3.0
36		*	105	86	73	57	3.4
42			129	102	85	66	3.8
48			*	124	101	76	4.3
54				*	115	85	4.7
* * 60					140	97	5.2

* Select a larger diameter manhole.

* * These sizes are too tall for a standard 6' tall manhole barrel section. Check with the manufacturer for other possibilities like bends and tees.



TYPICAL PLAN SECTION

MANHOLE SIZE	MINIMUM L°
48"	32
60	29
72	24
84	24
96	24
120	24

Figure E.2. Typical manhole plan section and minimum angle L

MANHOLE SIZING PROCEDURE

1. Assume a manhole size and determine a minimum leg angle (MINIMUM L°) from Table 1.
2. For the assumed manhole size, determine the degrees required (A°) for each pipe opening from Table 2 .
3. Find the leg angle (L°) between pipe openings. $L° = Y° - 0.5 (A1° + A2°)$. See TYPICAL PLAN. L° must be larger than or equal to MINIMUM L° given in table 1. $L° \geq \text{MINIMUM } L°$.
4. Use Table 2 and check the manhole height. The remaining full barrel height must be 1.3' minimum. See example and notes.

Figure E.1. Manhole sizing procedure and minimum values for angle A based on pipe and manhole diameters

Table E.4. Calculation check for 48-in manhole conformance

48-in Dia.		Y																	
MH	X	Y	PVC1		PVC2		PVC3		K1+K3	K1+K2 *	K2+K3 *	deg	X	0.5*(K1+K3)	Conforms?	X + 0.5*(K2+K3)	Y	360 - 0.5*(K1+K2)	Conforms?
			D _{Pipe1}	K1	D _{Pipe2}	K2	D _{Pipe3}	K3	2	2	2								
0	179	-	6	31	-	-	8	37	34.0	-	-	180	179	34	✓	-	-	-	-
1	127	256	8	37	6	31	8	37	37.0	34.0	34.0	180	127	-	-	161.0	256	326.0	✓
2	76	184	8	37	6	31	8	37	37.0	34.0	34.0	180	76	-	-	110.0	184	326.0	✓
3	149	-	8	37	-	-	8	37	37.0	-	-	180	149	37	✓	-	-	-	-
4	153	-	8	37	-	-	8	37	37.0	-	-	180	153	37	✓	-	-	-	-
5	177	-	8	37	-	-	8	37	37.0	-	-	180	177	37	✓	-	-	-	-
6	176	-	8	37	-	-	8	37	37.0	-	-	180	176	37	✓	-	-	-	-
7	177	-	8	37	-	-	8	37	37.0	-	-	180	177	37	✓	-	-	-	-
8	79	194	8	37	6	31	8	37	37.0	34.0	34.0	180	79	-	-	113.0	194	326.0	✓
9	170	-	8	37	-	-	12	47	42.0	-	-	180	170	42	✓	-	-	-	-
10	164	-	12	47	-	-	14	53	50.0	-	-	180	164	50	✓	-	-	-	-
11	165	-	14	53	-	-	14	53	53.0	-	-	180	165	53	✓	-	-	-	-
12	167	-	14	53	-	-	14	53	53.0	-	-	180	167	53	✓	-	-	-	-
13	166	-	14	53	-	-	14	53	53.0	-	-	180	166	53	✓	-	-	-	-
14	171	-	14	53	-	-	14	53	53.0	-	-	180	171	53	✓	-	-	-	-
15	140	-	14	53	-	-	14	53	53.0	-	-	180	140	53	✓	-	-	-	-

* The average of K1, K2 and K2, K3 are represented in the appendix calculations by A and B , respectively.

ESTIMATED COSTS CALCULATION TABLES

Table E.5. Open Trench Excavation Cost Estimates for Choto Rd. Gravity Sewer

Entering Manhole #	Pipe Dia. (in)	Pipe Area (SF)	Pipe Length (LF)	Required Width		Trench Width		Depth		Volume		Cost	
				d + 16" (in)	1.25 * d (in)	max{(d+16),(1.25*d)} (in)	(ft)	Backfill (ft)	Trench (ft)	Backfill (CY)	Excavation (CY)	Backfill P = \$181 / CY	Excavation P = \$6.27 / CY
1	8.0	0.35	360.0	24.0	10.0	24.0	2.0	5.0	6.0	133	160	\$ 24,133	\$ 1,003
2	8.0	0.35	360.0	24.0	10.0	24.0	2.0	5.0	6.0	133	160	\$ 24,133	\$ 1,003
3	8.0	0.35	360.0	24.0	10.0	24.0	2.0	5.0	6.0	133	160	\$ 24,133	\$ 1,003
4	8.0	0.35	360.0	24.0	10.0	24.0	2.0	5.0	6.0	133	160	\$ 24,133	\$ 1,003
5	8.0	0.35	360.0	24.0	10.0	24.0	2.0	5.0	6.0	133	160	\$ 24,133	\$ 1,003
6	8.0	0.35	360.0	24.0	10.0	24.0	2.0	5.0	6.0	133	160	\$ 24,133	\$ 1,003
7	8.0	0.35	360.0	24.0	10.0	24.0	2.0	5.0	6.0	133	160	\$ 24,133	\$ 1,003
8	8.0	0.35	360.0	24.0	10.0	24.0	2.0	5.0	6.0	133	160	\$ 24,133	\$ 1,003
9	12.0	0.79	360.0	28.0	15.0	28.0	2.3	6.0	7.0	187	218	\$ 33,787	\$ 1,365
10	14.0	1.07	360.0	30.0	17.5	30.0	2.5	6.0	7.0	200	233	\$ 36,200	\$ 1,463
11	14.0	1.07	360.0	30.0	17.5	30.0	2.5	6.0	7.0	200	233	\$ 36,200	\$ 1,463
12	14.0	1.07	360.0	30.0	17.5	30.0	2.5	6.0	7.0	200	233	\$ 36,200	\$ 1,463
13	14.0	1.07	360.0	30.0	17.5	30.0	2.5	6.0	7.0	200	233	\$ 36,200	\$ 1,463
14	14.0	1.07	360.0	30.0	17.5	30.0	2.5	6.0	7.0	200	233	\$ 36,200	\$ 1,463
15	14.0	1.07	360.0	30.0	17.5	30.0	2.5	6.0	7.0	200	233	\$ 36,200	\$ 1,463
16	14.0	1.07	360.0	30.0	17.5	30.0	2.5	6.0	7.0	200	233	\$ 36,200	\$ 1,463
PS	14.0	1.07	360.0	30.0	17.5	30.0	2.5	6.0	7.0	200	233	\$ 36,200	\$ 1,463
										2,853 CY	3,364 CY	Total \$ 516,453	\$ 21,095

*based on fully open trench instead of trenchless technology

*backfill unit cost based on TDOT Avg. Unit Prices 2019 for "Bedding Material (Pipe) Class B"

Table E.6. Pipe and Manhole Cost Estimates

Pipe Dia.	Quantity	Length (LF)	Total Length (LF)	Unit Cost	Subtotal Cost
8-in	8	360	2,880	\$ 100	\$ 288,000
12-in	1	360	360	\$ 100	\$ 36,000
14-in	7	360	2,520	\$ 100	\$ 252,000
Total			5,760		\$ 576,000

Manhole Size	Quantity	Unit Cost	Subtotal Cost
48-in	16	\$ 6,000	\$ 96,000

Pipe	\$ 576,000
Manholes	\$ 96,000
Total	\$ 672,000

Table E.7. Choto Rd. Gravity Sewer Total Estimated Cost

Task	Cost
Excavation	\$ 21,095
Backfill	\$ 516,453
Pipe	\$ 576,000
Manholes	\$ 96,000
Total	\$ 1,209,548

2/26/20

- ① Assume Q_{max} at 0.8(dia.), Assume n , Calculate S

$$a) Q = \frac{1.486}{n} A R_h^{2/3} S_o^{1/2}$$

$$b) A = \frac{\pi D^2}{4}$$

$$c) R_h = \frac{A}{P}$$



$$d) d = 1.56 \left[\frac{n Q}{S^{1/2}} \right]^{3/8} \quad \text{(metric)}$$

$$AR_h^{2/3} = \frac{n Q}{K S^{1/2}}$$

Velocity for min flow

 $K=1$, metric

②

y/D	θ , rad	A , m^2	P , m	R , m	$AR_h^{2/3}$

Spreadsheet

$$a) \theta = 2 \cos^{-1} \left(1 - \frac{2y}{D} \right)$$

$$b) A = \frac{1}{8} (\theta - \sin \theta) D^2$$

$$c) P = \frac{\theta D}{2}$$

$$d) R = \frac{A}{P} = \frac{1}{4} \left(1 - \frac{\sin \theta}{\theta} \right) D$$

e) Flow depth from y/D & selected D

$$y = \left(\frac{y}{D} \right) D$$

 $y_{min} = 6 \text{ ft}$

$$f) V = \frac{Q}{A}$$

$$V_{min} = 2 \frac{\text{ft}}{\text{s}} \left(\frac{0.3048 \text{ m}}{1 \text{ ft}} \right) = \underline{\underline{0.61 \text{ m/s}}}$$

g) Ensure that $v > v_{min}$ $y = 0.8 \text{ Dia}$

$$h) \text{ Check } V_c = \frac{V_c^* K_n U_{*c} d^{1/6}}{n \sqrt{g}}$$

 V_c^* from Figure 4.11
(critical Velocity)1st MH: $Q = 39,000 \text{ Gpd}$

104.05' from 966.1 BM

2nd MH: $Q = 11,800 \text{ Gpd}$

350' from 1st

3rd MH: $Q = 12,800 \text{ Gpd}$

350' from 2nd

4th MH: $Q = 34,000 \text{ Gpd}$

$$S = \frac{\Delta h}{L} = \frac{966.1 - 834.8}{5900}$$

$$= 0.022254 = 2.225\% \quad \text{Average Slope}$$

$$Q = \frac{1.486}{0.011} \left(\frac{\pi D^2}{4} \right) \left(\frac{\pi D^2}{4P} \right)$$

Appendix F. Civil Site Development for Pump Stations

Pipe at capacity at full pipe with
no pressure per TDEC

$$Q = \left(\frac{1.49}{n} \right) A R^{\frac{2}{3}} \sqrt{S} \quad \text{Manning's Equation (No pressure)}$$

$$Q = \left(\frac{1.49}{0.014} \right) \left(\pi (0.83 \text{ ft})^2 \right) \left(\frac{1.67 \text{ ft}}{4} \right)^{\frac{2}{3}} \sqrt{0.0044}$$

$$n = 0.014 \quad (\text{Chow 1959})$$

$$Q = 684.43 \frac{\text{ft}^3}{\text{s}}$$

$$Q = 3784 \text{ gpm}$$

Reference: ACI 330 R-01

Use Table 2.1 assuming

Assume Fine Grain Soils Predominate in Subgrade

(Based on USDA Soil
Maps-Full Geotechnical
Study Required Before
Construction)

Take average k value

$$k = \frac{(120 + 75) \text{ pci}}{2} = 97.5 \text{ pci} \approx 100 \text{ pci}$$

From Table 2.4, Assumptions A-1

traffic conditions ($ADTT = 1$)

- Concrete thickness between 4-5 in.
depending on concrete strength.

- Strength unknown assume typical value
of normal ($f_c = 4000 \text{ psi}$)

$$M_R = 2.3 f_c^{2/3}$$

$$M_R = 2.3 (4000 \text{ psi})^{2/3}$$

$$M_R = 579.6 \text{ psi} \approx 580 \text{ psi}$$

From Table 2.4

concrete depth = 4.5 in.

Use Table 2.2 to select subbase

OP.A - 4 in. - thinnest possible

$$k_{add} = 30 \text{ pci'}$$

$$k_{new} = 97.5 \text{ pci' } + 30 \text{ pci'}$$

$$k_{new} = 127.5 \text{ pci'}$$

Not high enough to enter new

k -category on table 2.4

OP.B - 12 in. - thickest recommended

$$k_{add} = 90 \text{ pci'}$$

$$k_{new} = 97.5 \text{ pci' } + 90 \text{ pci'}$$

$$k_{new} = 187.5 \text{ pci'}$$

Close enough to evaluate

$k = 200$ category on Table 2.4

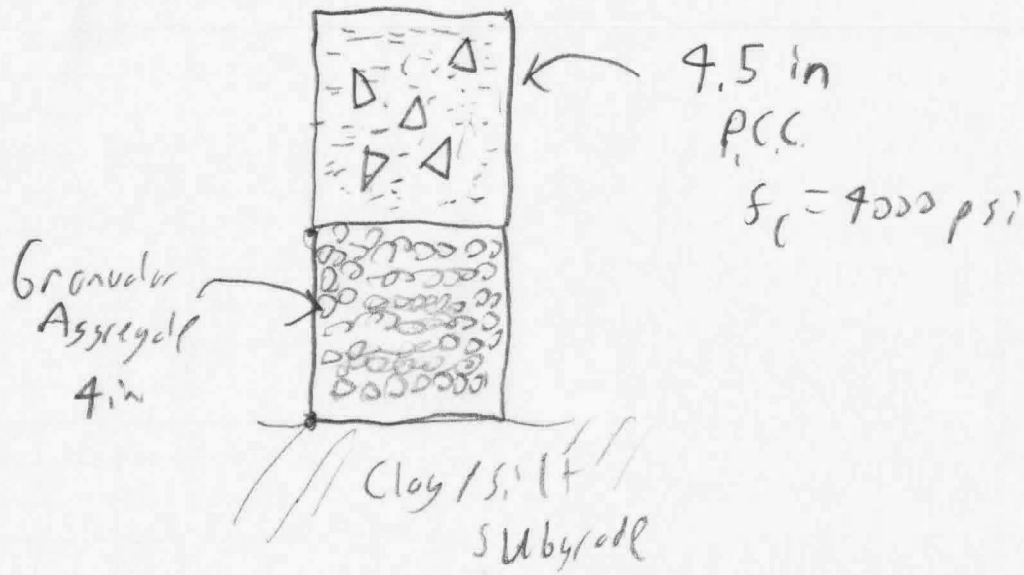
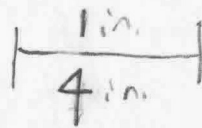
New concrete thickness = 4.5 in

No change 

Use option A to reduce cost

Subbase depth = 4 in

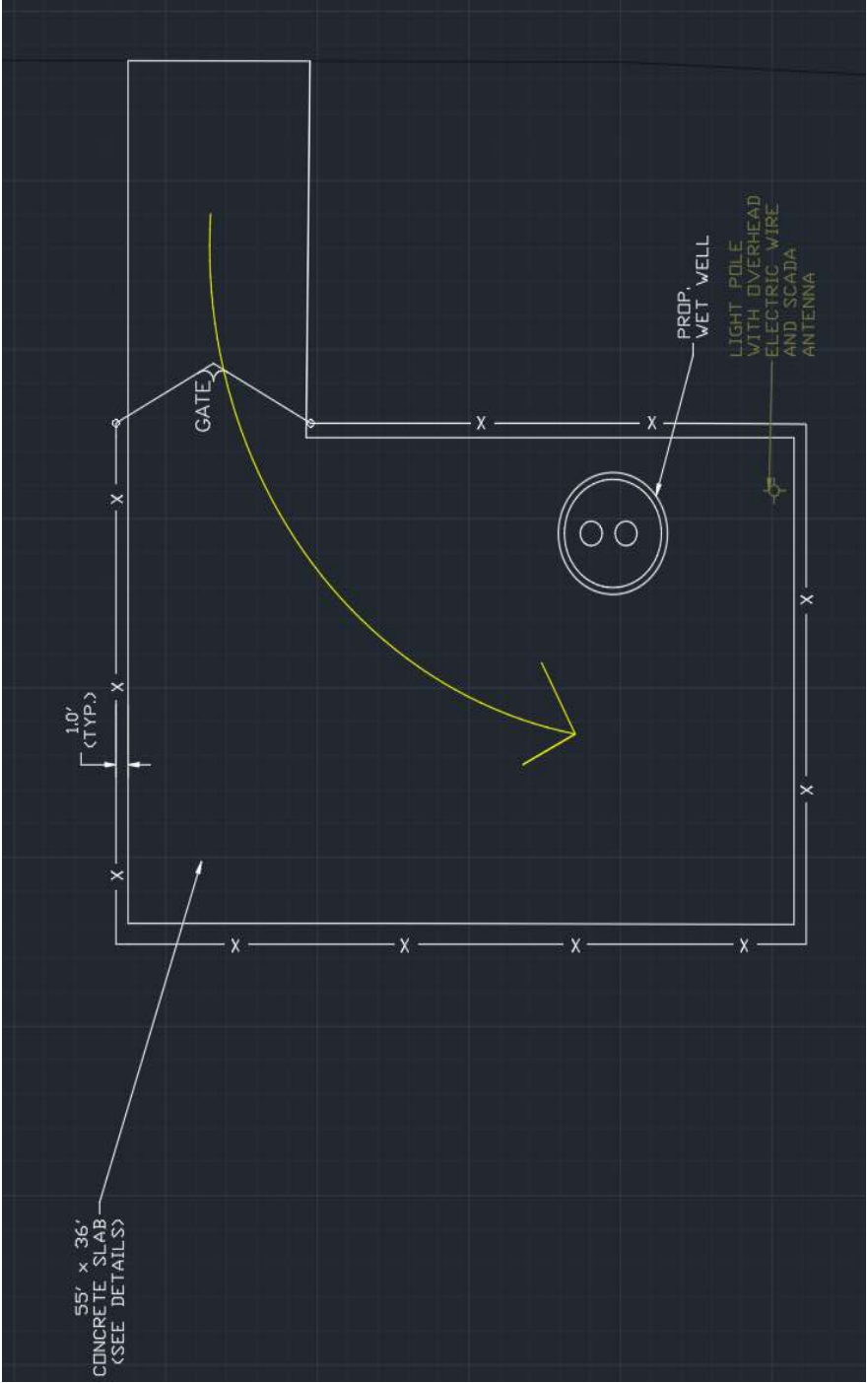
Sketch



Turning Radius Calculations

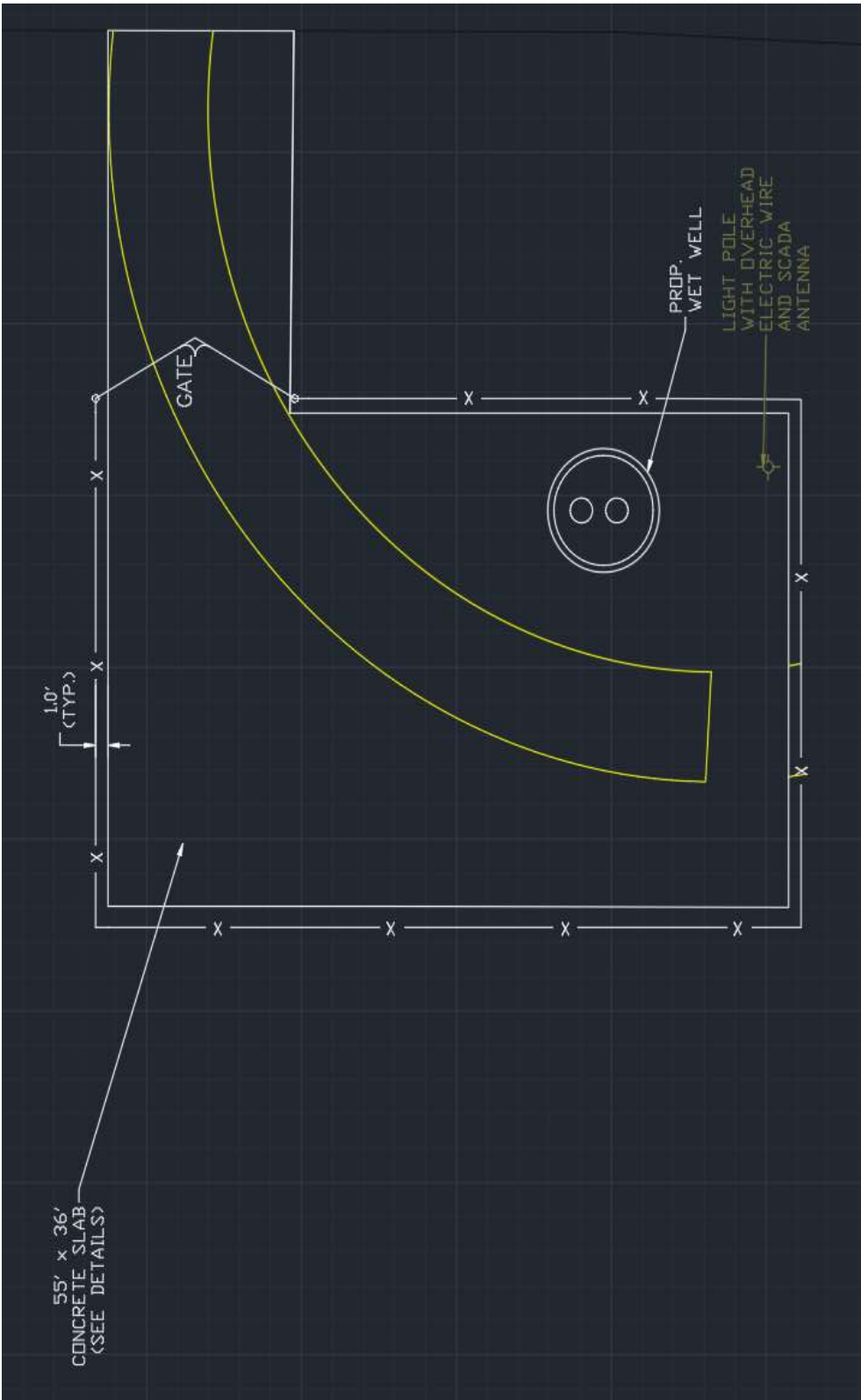
Calcs Performed by:	Jonathan West
Min. Wall to Wall Radius (Ft)	48.85
Min. Curb to Curb Radius (ft)	44.75
distance between edge of bumper and edge of pavement (ft)	4.1
Channel Point Pump Station	
Evaluated Turn:	

=Min. Curb to Curb Radius- Min Wall to Wall Radius



Minimum Turn	
Wall to Wall Radius (Ft)	48.85
Curb to Curb Radius (ft)	44.75
Initial min distance from wet well	6
Clearance to western edge of fence (ft)	9.1
Clearance to edge of curb (ft)	12.2
The minimum of the above two clearnaces is the range at which the driver can take this turn and still make it.	

$$= (\text{Clearance to edge of curb}-1\text{ft})+\text{Wall Radius-Curb Radius}$$



Maximum Turn	
Wall to Wall Radius (ft)	61.8
Curb to Curb Radius (ft)	54
Initial min distance from wet well (ft)	10.2
Clearance to western edge of fence (ft)	4.2
Clearance to edge of curb (ft)	11'
The minimum of the above two clearances is the range at which the driver can take this turn and still make it.	

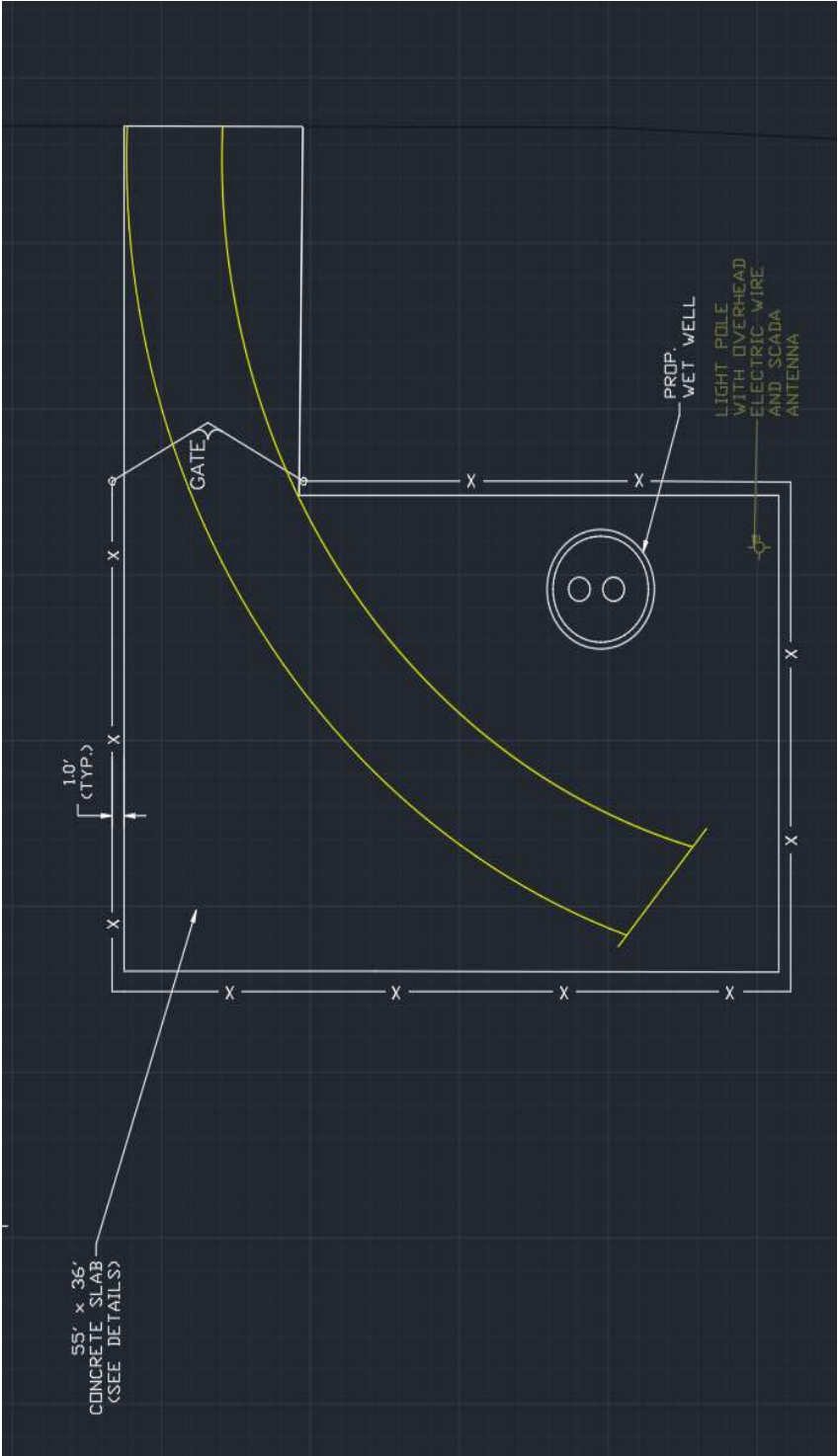
$\frac{1}{2}$ = (Clearance to edge of curb-1ft)+Wall Radius-Curb Radius

Range of turns possible	
Wall to Wall Radius (ft)	13.0
Curb to Curb Radius (ft)	9.3

=Wall Radius for max turn-Wall Radius for min turn

=Curb Radius for max turn-Curb Radius for min turn

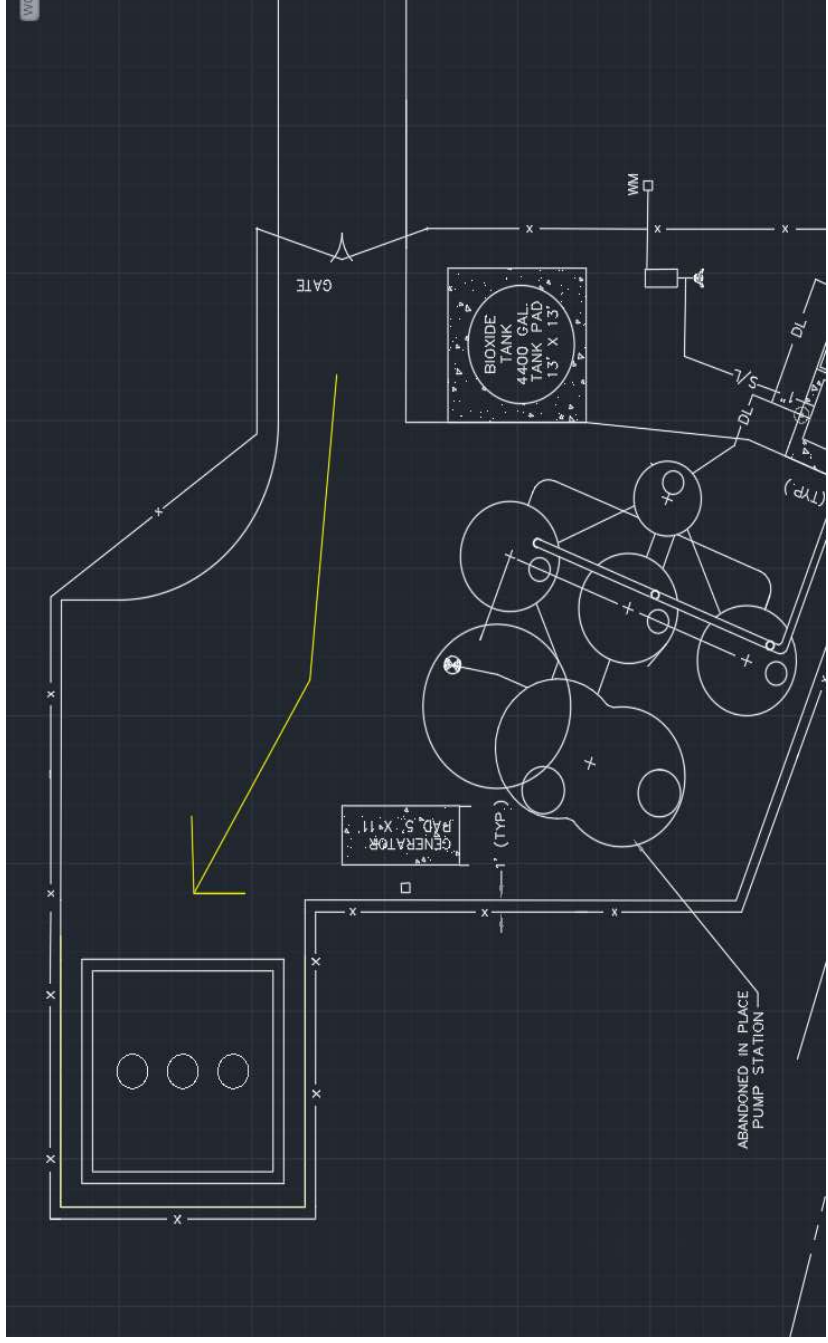
Min turn is closer to pump than max. Min is optimal



Calcs Performed By:	Jonathan West
Min. Wall to Wall Radius (Ft)	48.85
Min. Curb to Curb Radius (ft)	44.75
distance between edge of bumper and edge of pavement (ft)	4.1

=Min. Curb to Crub Radius- Min Wall to Wall Radius

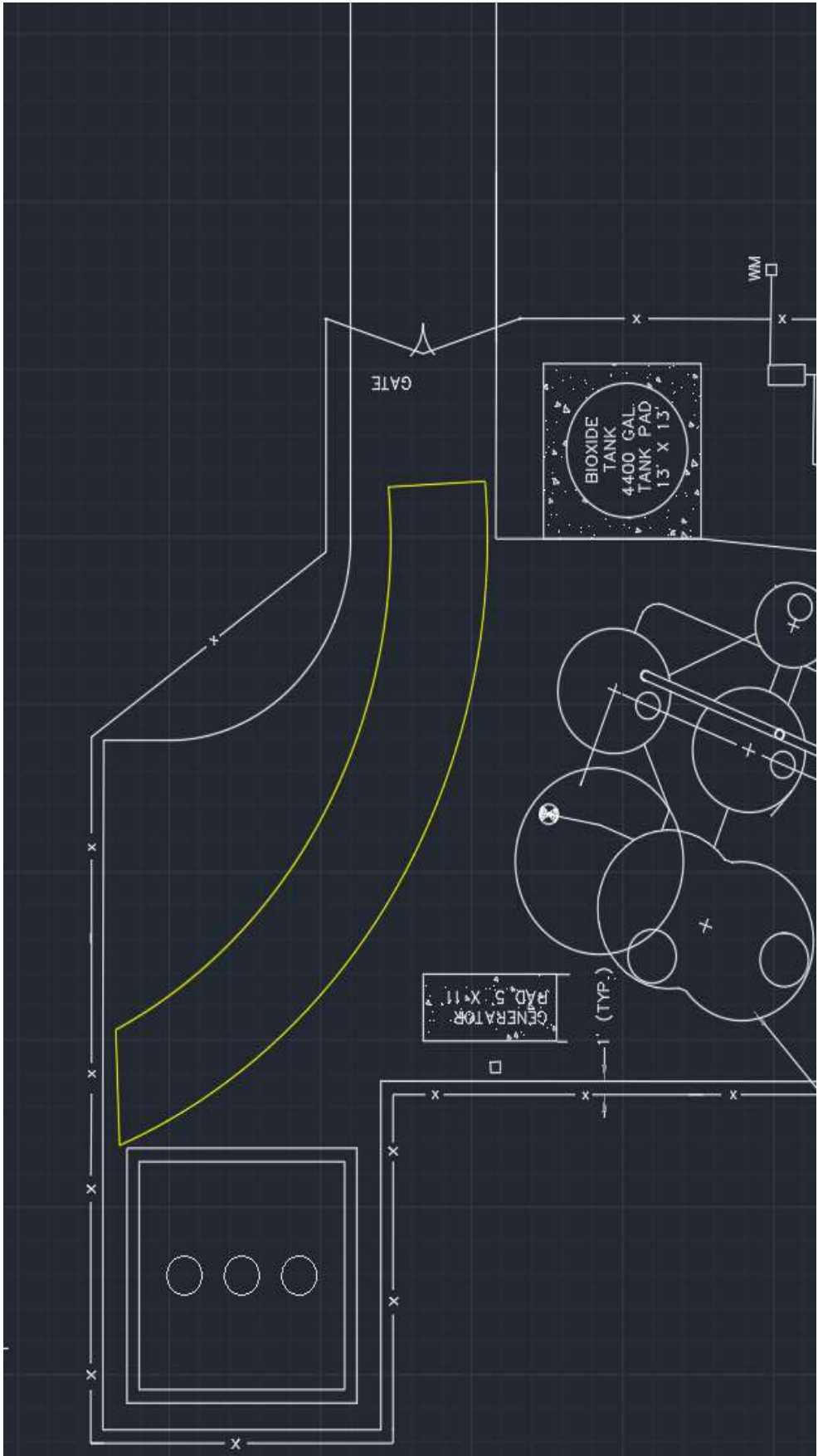
Channel Point Pump Station
Evaluated Turn:



Minimum Turn	
Wall to Wall Radius (Ft)	48.85
Curb to Curb Radius (ft)	44.75
Initial min distance from wet well (ft)	0.4
Clearance to eastern edge of fence (ft)	18.8
Clearance to edge of curb (ft)	16.6

$$=12.5 \text{ ft} + (\text{Dist. Btwn edge of bumper and edge of pavement})$$

The minimum of the above two clearances is the range at which the driver can take this turn and still make it.



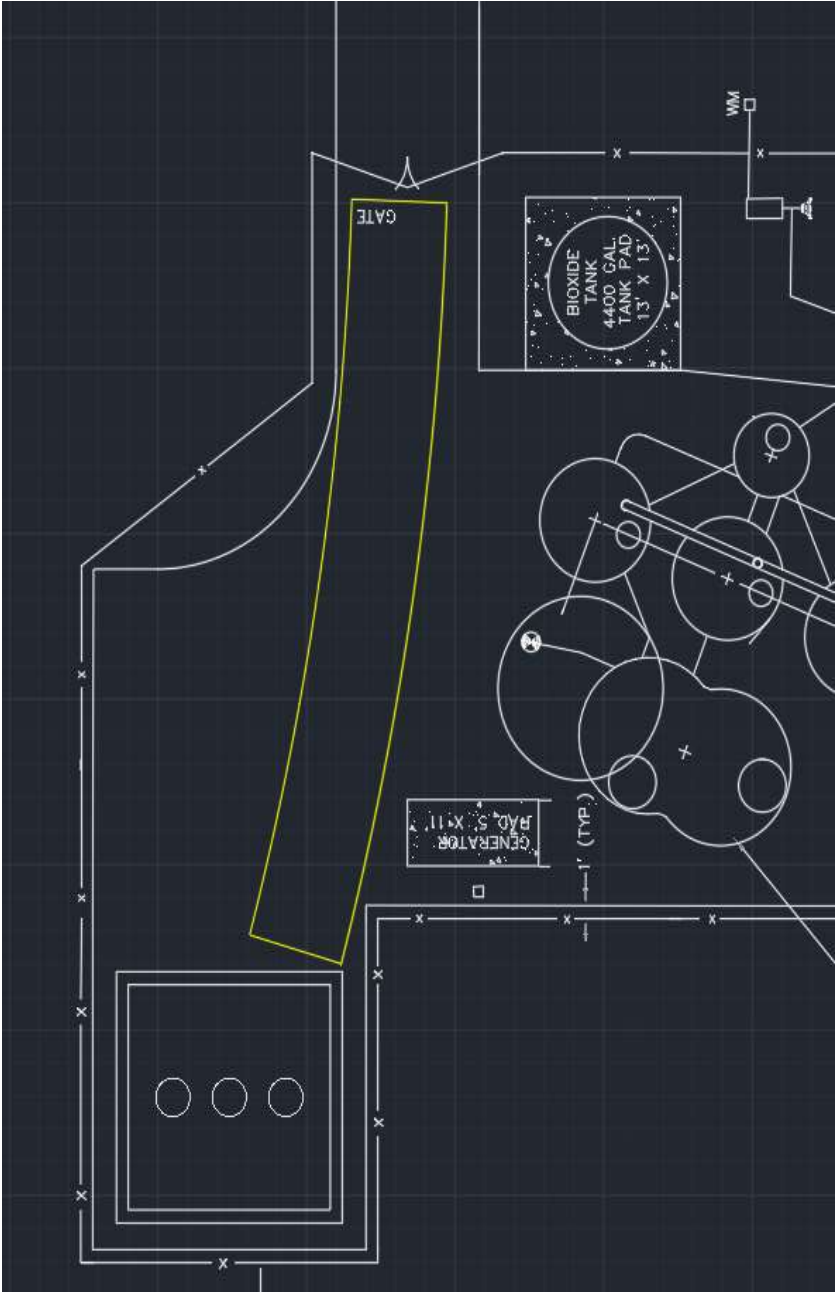
Maximum/optimal Turn	
Wall to Wall Radius (Ft)	242
Curb to Curb Radius (ft)	234
Initial min distance from wet well (ft)	0.1
Clearance to eastern edge of fence (ft)	17
Clearance to edge of curb (ft)	8.1
The minimum of the above two clearances is the range at which the driver can take this turn and still make it.	

8.1 = 4 ft + (Dist. Btwn edge of bumper and edge of pavement)

Range of turns possible	
Wall to Wall Radius (ft)	193.2
Curb to Curb Radius (ft)	189.3

193.2 = Wall Radius for max turn-Wall Radius for min turn

189.3 = Curb Radius for max turn-Curb Radius for min turn



Chofs: No slope required - Shoring

$$V = L \cdot W \cdot d$$

$$V = (19\text{ ft})(19\text{ ft})(14\text{ ft})$$

$$V = 5054 \text{ ft}^3$$

$$V = 187.2 \text{ yd}^3 \text{ cut}$$

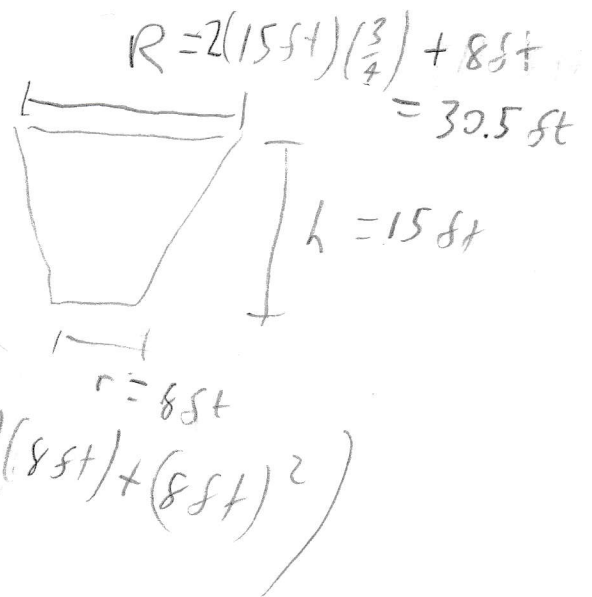
Channel Point: Slope $\frac{3}{4}$: 1

$$V = \frac{\pi h}{3} (R^2 + Rr + r^2)$$

$$V = \frac{\pi (15\text{ ft})}{3} \left((30.5\text{ ft})^2 + (30.5\text{ ft})(8\text{ ft}) + (8\text{ ft})^2 \right)$$

$$V = 19450 \text{ ft}^3$$

$$V = 720.4 \text{ yd}^3 \text{ cut}$$



Cut/Fill Report

Generated: 2020-04-20 11:26:01

By user: CEESDL.Remote12

Drawing: \\cad27.cee.utk.edu\CEE400.Fall2019\FUD Sewer Design\\cad27.cee.utk.edu\CEE400.Fall2019\FUD Sewer Design\Choto Site Proposed Plan.dwg

Volume Summary							
Name	Type	Cut Factor	Fill Factor	2d Area (Sq. Ft.)	Cut (Cu. Yd.)	Fill (Cu. Yd.)	Net (Cu. Yd.)
comp2	full	1.000	1.000	1519.36	8.61	67.09	58.48<Fill>

Totals				
	2d Area (Sq. Ft.)	Cut (Cu. Yd.)	Fill (Cu. Yd.)	Net (Cu. Yd.)
Total	1519.36	8.61	67.09	58.48<Fill>

* Value adjusted by cut or fill factor other than 1.0

Cut/Fill Report

Generated: 2020-03-09 19:13:48

By user: CEESDL

Drawing: \\cad27.cee.utk.edu\CEE400.Fall2019\FUD Sewer Design\\cad27.cee.utk.edu\CEE400.Fall2019\FUD Sewer Design\Channel Point Proposed Plan.dwg

Volume Summary							
Name	Type	Cut Factor	Fill Factor	2d Area (Sq. Ft.)	Cut (Cu. Yd.)	Fill (Cu. Yd.)	Net (Cu. Yd.)
comparison	full	1.000	1.000	5451.62	0.00	556.46	556.45<Fill>

Totals				
	2d Area (Sq. Ft.)	Cut (Cu. Yd.)	Fill (Cu. Yd.)	Net (Cu. Yd.)
Total	5451.62	0.00	556.46	556.45<Fill>

* Value adjusted by cut or fill factor other than 1.0

$$\text{Watershed Area} = 942 \text{ ft}^2$$

Use Rational Formula

$$Q = CiA$$

(- vast majority of watershed area is pavement Assume $C = 0.85$)

- Design for 50-year storm

(TDOT
Drainage
Manual
pg. 4-8)

$$T_c = \frac{L}{60 \times V}$$

← kinetic wave method

$$V = 3.2 \frac{\text{ft}}{\text{s}}$$

→

based on Average Paved slope of 0.025 ft/ft

(USDA Urban
Hydrology for
Small Watersheds
(1986))

$$T_c = \frac{40 \text{ ft}}{60 \cdot (3.2 \frac{\text{ft}}{\text{s}})}$$

$$T_c = 0.25$$

$$i = 8.0 \frac{\text{in.}}{\text{hr}}$$

← based on T_c

(Figure 4A-7
TDOT Drainage)

$$Q = (0.85) \left(8.0 \frac{\text{in.}}{\text{hr}} \right) (0.022 \text{ acre})$$

$$Q = 0.15 \frac{\text{ft}^3}{\text{s}}$$

Select Trial pipe size of 12 in. — Minimum for several design agencies (DOT)

$$\text{Outlet EL} = 844.5 \text{ ft}$$

$$\text{Inlet EL} = 845 \text{ ft}$$

$$S_p = \frac{I_{\text{inlet}} - I_{\text{outlet}}}{L_p}$$

$$S_p = \frac{(845 - 844.5) \text{ ft}}{8 \text{ ft}}$$

Top elev. is 835.5,
but drop is required
for catch basin

$$S_p = 0.0625 \frac{\text{ft}}{\text{ft}}$$

$$Q_{\text{cap}} = \frac{0.463}{n} D^{2.667} S_p^{0.5} \leftarrow \text{Manning's Equation}$$

$$n = 0.013 \text{ (PVC pipe)}$$

(DOT Drainage
Table 7-10)

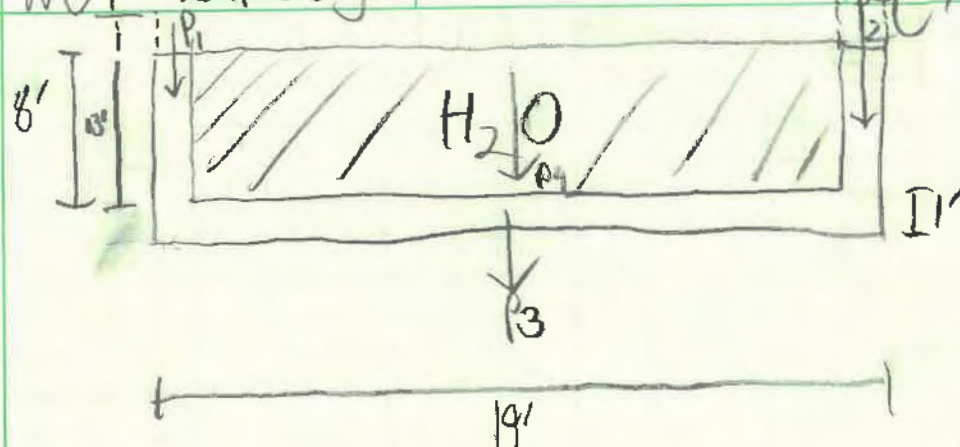
$$Q_{\text{cap}} = \frac{0.463}{0.013} (1 \text{ ft})^{2.667} (0.0625 \frac{\text{ft}}{\text{ft}})$$

$$Q_{\text{cap}} = 2.2 \frac{\text{ft}^3}{\text{s}}$$

$Q_{\text{cap}} \gg Q$ ✓ pipe size acceptable

Appendix G. Geotechnical and Structural Calculations for Pump Station Redesign

$\rho, \sigma, \tau, \kappa$



- Side walls
 $P_1 \& P_2 = (1')(13')(19')(150 \text{ pcf}) = 37.05 \text{ kips}$

- SLab
 $P_3 = (1') (14') (14') (150 \text{ pcf}) = 54.15 \text{ kips}$

- $$P_y = (17' \times 17' \times 8') 62.4 \text{ pcf} = 144.27 \text{ kips}$$

- bearing capacity
 $\gamma = 120 \text{ PCF}$, $C = 100$, $\phi = 27^\circ$

$$N_c = 29.2, N_q = 15.9, N_f = 12.5$$

- $$q = 1.3(29.2)(100 \text{ psf}) + 1680 \text{ psf}(15.9) + 0.4(120)(19')(125)$$
- $$= 3796 + 26712 + 11,400$$

$$q_{allow} = 41,908 \text{ psf}$$

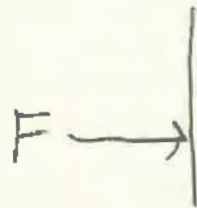
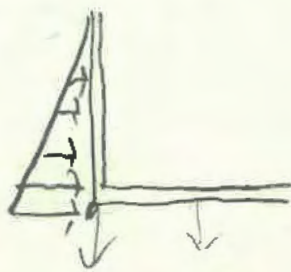
- $$q_{req} = \frac{\text{sidewalk} + \text{slab} + \text{water}}{A} = \frac{37.05 + 54.75 + 149.27}{192} = 652.27 \text{ psf} \checkmark \text{ O.K.}$$

$$OCR = \frac{652.27}{1680} = 0.388$$

$$\begin{aligned} K_0 &= (1 - \sin \phi) OCR^{\sin \phi} = \\ &= (1 - \sin 27^\circ) 0.388^{\sin(27^\circ)} \\ &= 0.355 \end{aligned}$$

Wet well design Side walls LC 1, side fill 3' empty

$$f'_c = 4,000 \text{ psi} \quad , \quad f_y = 60,000 \quad , \quad 13 \text{ ft wall}$$



$$F_{\text{resultant}} = \frac{1}{2} k_o \gamma (h')^2 = \frac{1}{2} (0.355) (120 \text{ pcf}) (13)^2$$

$$= 3,600 \text{ lbs}$$

$$\text{moment} = (13 \times 3,600) \left(\frac{1}{3} \right) =$$

$$= 15,600 \text{ ft-lbs}$$

Vertical reinforcement

$$V_u = 1.6 (F_{\text{resultant}}) = 1.6 (3,600)$$

$$V_u = 5,760$$

$$M_u = V_u (h_w) \left(\frac{1}{3} \right) = 5,760 (13') \left(\frac{1}{3} \right)$$

$$M_u = 24,960 \text{ ft-lbs} = 300$$

$$d = A_{\text{stem}} - 2'' - 0.25'' = 12 - 2' - 0.25 = 9.75 \text{ in}$$

$$C_1 = 1.7 (f'_c) b d / (2 f_y) = 1.7 (4) (12 \text{ in}) (9.75 \text{ in}) / (2 (60))$$

$$C_1 = 6.63$$

$$C_2 = 6.8 f'_c b M_u / (4 (0.9) f_y^2)$$

$$C_2 = 6.8 (4) (12) (300) / (4 (0.9) (60)^2)$$

$$C_2 = 7.55 \text{ in}^4$$

$$A_s = C_1 - \sqrt{C_1^2 - C_2} = 6.63 - \sqrt{6.63^2 - 7.55} = 0.59 \text{ in}^2/\text{ft}$$

$$\text{use } 1 \#7 \text{ bar/ft} \quad 0.6 \text{ in}^2/\text{ft} > 0.59 \text{ in}^2/\text{ft}$$

$$S \& T = 0.00186t = 0.0018 (12) (12) = 0.259 \text{ in}^2/\text{ft} \checkmark \text{OK}$$

Shear check

$$F \phi(h_w - d) = H \phi h_w (h_w - d)^2 / h_w^2$$
$$= 4542.72 (13 - \frac{9.75}{12})^2 / 13^2 = 3992.63 \text{ lbs}$$

$$V_u = 1.6 (3992.63 \text{ lbs}) = 6388.2$$

$$\phi V_c = \phi 2 \sqrt{F_c} b d = 0.75 (2) \sqrt{4000} (12 \times 9.75)$$

$$\phi V_c = 11,100 \text{ lbs} > 6388.2 \text{ lbs} \checkmark$$

longitudinal reinforcement = L_w

$$A_s = .002 b t = .002 (12)(12) = 0.288 \text{ in}^2/\text{ft}$$

$$('L_w') \text{ Bars } \times 5 @ 12'' = 0.31 \text{ in}^2/\text{ft} > 0.288 \text{ in}^2/\text{ft} \checkmark$$

vet well design

side wall

LC2 - leak check

$$f'_c = 4,000 \quad f_y = 60,000, \quad 8\text{ft wall}$$

$$\text{water} = 62.4 \text{ pcf}$$

$$k=1$$

$$F_{\text{resultant}} = \frac{1}{2}(k)(\gamma_{H_2O})(h')^2 = \frac{1}{2}(1)(62.4)(8')^2$$

$$\rightarrow = 1996.8165$$

$$\text{Moment} = 8'(1996.8)\left(\frac{1}{3}\right) =$$

$$= 5324.8 \text{ ft-lbs}$$

Vertical Reinforcement

$$V_u = 1.6(1996.8) = 3194.9165$$

$$M_u = 8'(3194.9165)\left(\frac{1}{3}\right) = 8519.7 \text{ ft-lbs}$$

$$d = 9.75'' - .25'' = 9.5''$$

$$C_1 = 1.7(f'_c)bd / 2f_y = 1.7(4)(12'')(9.5'') / 2(60)$$

$$C_1 = 6.46$$

$$C_2 = 6.8(f'_c) b M_u / 4(0.9) f_y^2$$

$$C_2 = 6.8(4)(12'')(5324.8(12)) / 4(0.9)60^2$$

$$C_2 = 1.6$$

$$A_s = C_1 - \sqrt{C_1^2 - C_2} = 6.46 - \sqrt{6.46^2 - 1.6} =$$

$$A_s = 0.13 \text{ in}^2/\text{ft}$$

~~4~~ bars @ 1'

$$\text{S\&T } 0.0018 \text{ bt} = .0018(12)(9.5) = .2052 \quad X$$

use 5 bars every ft

Development length

X7's

$$L_d = \left(\frac{3}{40} \frac{F_y}{\sqrt{F'_c}} \frac{\Psi_t \Psi_e \Psi_s}{\left(\frac{C_b + K_{tr}}{d_b} \right)} \right) d_b$$

$$L_d = \left(\frac{3}{40} \frac{60,000}{1.0 \sqrt{4,000}} \frac{(1.0)(1.0)(1.0)}{\left(\frac{1 + 0}{.875} \right)} \right) 0.875'' = 54.98'' = 4.6''$$

X4's

$$L_d = \left(\frac{3}{40} \frac{60,000}{1.0 \sqrt{4,000}} \frac{(1.0)(1.0)(0.8)}{\left(\frac{1 + 0}{.5} \right)} \right) =$$

X7'

$$L_d = 47 d_b = 47 (.875) = 41.13'' = 3.43' \approx 3.5'$$

X4's

$$L_d = 47 (.5) = 23.5'' \approx 2'$$

Shoring design Timber lagging

$$H_{wall} = 14'$$

$$b = \text{embed width} = 1.17'$$

$$\text{Pile spacing} = \text{Spile} = 9.5 \text{ ft}$$

$$\gamma_{soil} = 120 \text{ pcf}$$

$$\phi = 27^\circ$$

$$\alpha = \text{inclination of wall} = 0^\circ$$

$$\beta = \text{inclination of ground surface} = 0^\circ$$

$$q = 100 \text{ psf} = \text{surcharge}$$

$$\gamma_p = 1.5 \text{ load factor for earth pressure}$$

$$\gamma_{LL} = \text{Strength I LL factor} = 1.75$$

$$\phi_p = \text{passive earth pressure reduction factor} = 0.75$$

$$\phi_v = 1.0 = \text{shear reduction}$$

$$F_y = 50 \text{ ksi}$$

$$\gamma_f = 120 \text{ pcf}$$

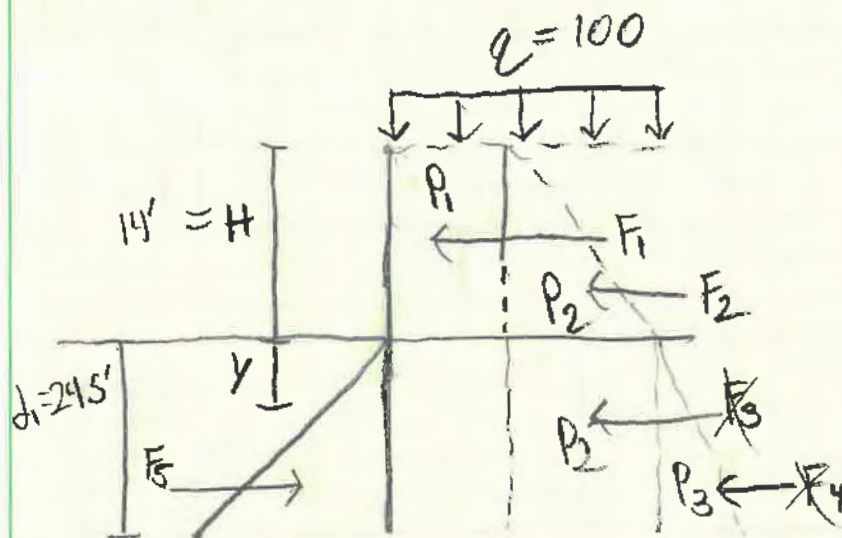
$$\phi_f = 27^\circ$$

$$K_a = \frac{\cos(\phi - \alpha)^2}{\cos(\alpha)^2 \cdot \cos(\phi_w + \alpha) \cdot \left(1 + \sqrt{\frac{\sin(\phi + \phi_w) \sin(\phi - \beta)}{\cos(\phi_w + \alpha) \cos(\beta - \alpha)}} \right)^2}$$

$$K_a = 0.376$$

$$K_p = \frac{\cos(\phi_f + \alpha_f)^2}{\cos(\alpha)^2 \cos(\phi_f - \alpha) \left[1 - \sqrt{\frac{\sin(\phi_f - \alpha_f) \sin(\phi_f - \beta)}{\cos(\alpha_f + \alpha_f) \cos(\beta_f - \alpha_f)}} \right]^2}$$

$$K_p = 2.663$$



- compute pressures

$$P_1 = K_a \cdot q = 37.55 \text{ PSF}$$

$$P_2 = K_a \cdot \gamma_{\text{soil}} \cdot H_{\text{wall}} = 630.88 \text{ PSF}$$

$$P_4 = K_p \cdot \gamma_{\text{soil}} \cdot d_1 = 7,830 \text{ PSF}$$

- compute forces

$$F_1 = \gamma_{LL} \cdot \text{Spile} \cdot P_1 \cdot H_{\text{wall}} = 8.74 \text{ kips}$$

$$F_2 = \gamma_p \cdot \text{Spile} \cdot 0.5 \cdot P_2 \cdot H_{\text{wall}} = 62.93 \text{ kips}$$

$$F_3 = \phi_p \cdot 3 \cdot b \left(K_p \cdot \frac{\gamma_{\text{soil}} \cdot d_1^2}{2} + 2 \cdot C_f \cdot d_1 \sqrt{K_p} \right) = 252.47 \text{ kips}$$

- moment about Hirse

$$\text{arm}_1 = d_1 + \frac{H_{\text{wall}}}{2} = 31.5 \text{ ft}$$

$$\text{arm}_2 = d_1 + \frac{H_{\text{wall}}}{3} = 24.17 \text{ ft}$$

$$\text{arm}_3 = \frac{d_1}{3} = 8.17 \text{ ft}$$

Σ moments about hinge

$$= F_1 \cdot Arm_1 + F_2 \cdot Arm_2 - F_3 \cdot Arm_3$$

$$= 48.94 \approx 0 \text{ close enough to zero } \checkmark$$

Max shear

$$F_1 + F_2 = 71.67 \text{ kips}$$

Depth to zero shear gives max moment
 $y = 13' \approx F_1 + F_2 - F_3 = 0.59 \text{ K close to zero}$

$$F_{1y} = \gamma_{LL} \cdot \text{Spile} \cdot P_{1y} \cdot H_{wall} = 8.74 \text{ kips}$$

$$F_{2y} = \gamma_p \cdot \text{Spile} \cdot 0.5 \cdot P_{2y} \cdot H_{wall} = 62.93$$

$$F_{3y} = \phi_p \cdot 3 \cdot b \left(k_f \cdot \frac{\gamma_f \cdot y^2}{2} + 2C_f \cdot y \sqrt{\gamma_f} \right) = 71.06$$

$$M_{max} = 978.55 \text{ K-ft controls pile selected}$$

$$P_{1y} = K_a \cdot q = 37.55 \text{ ps}$$

$$P_{2y} = K_a \cdot \gamma_{soil} \cdot H_{wall} = 630.88 \text{ psf}$$

Rock socket depth IF rock encountered @ ^{less than} embedment

- moment is conservatively taken at the top of rock

- $d_r = 6 \text{ ft}$

$$R_{req} = \frac{M_{max}}{\frac{d_r}{2}} = \frac{978.55}{3} = 355.84 \text{ kips}$$

$$R_{roce \text{ avail}} = \phi_p \cdot S_m \cdot d_r \cdot \frac{(d_r + \sqrt{2} \cdot b)}{(1 - \tan \beta_{roce})} = 369.8 \text{ kips}$$

$$369.8 \text{ kips} > 355.84 \text{ kips } \checkmark \text{ O.K.}$$

Shoring design timber lagging

Bending of pile

Try a HP16X141

assume wood lagging causes it to be fully braced.

$$b_f = 16.0 \text{ in}$$

$$I_x = 1870 \text{ in}^4$$

$$I_y = 599 \text{ in}^4$$

$$J_s = 12.9 \text{ in}^4$$

$$t_f = 0.875$$

$$h_o = 15.1 \text{ in}$$

$$S_x = 234$$

$$r_y = 6.7$$

$$t_w = 0.475$$

$$Z_x = 264 \text{ in}^3$$

$$t_{web} = .875$$

yielding

$$(F2-1) \quad M_n = M_p = F_y Z_x = 50 \text{ ksi} (264 \text{ in}^3) = 13,200 \text{ k-in}$$

$$M_n = 13,200 / 12 = 1100 \text{ k-ft}$$

$$\phi M_n = 1100 (0.9) = 990 \text{ k-ft}$$

$$L_p = 1.76 r_y \left(\sqrt{\frac{E F_y}{50}} \right) = 23.67 \text{ ft}$$

$L_b = 14'$, $L_b < L_p$, therefore LTB DOES NOT apply.

use HP16X141

• pressure ON wood facing

$$W_{lagging} = [\gamma_{soil} \cdot H_{wall} \cdot K_a \cdot \gamma_p] + [q \cdot K_a \cdot \gamma_{LL}]$$

$$W_{lagging} = 1012 \text{ PSF} = \text{max pressure}$$

$$W_{design} = 0.6 W_{lagging} = 607.22 \text{ PSF}$$

$$W_{moment} = \frac{[W_{design} \cdot (1ft) \cdot Spile^2]}{8} = 6.85 \text{ K-ft}$$

$$V_{wall} = \frac{[W_{design} \cdot (1ft) \cdot Spile]}{2} = 2.88 \text{ kips}$$

• WOOD lagging inputs

$F_b = 2350 \text{ psi}$ = reference design bending value

$F_v = 175 \text{ psi}$ = reference design shear value

$C_{fu} = 1.15$ = flat use factor

$C_m = 0.85$ = wet service factor

$C_L = 1.0$ = beam stability factor

$C_r = 1.15$ = repetitive member factor

$C_t = 1.0$ = temperature factor

$\phi_b = 0.85$ = Bending resistance factor

$\lambda_w = 1.0$ = time effect factor

$K_F = 2.54$ = format conversion factor

$b_v = 8 \text{ in}$ = breadth

$d_w = 4 \text{ in}$ = depth

$\phi_v = 0.75$ = shear resistance factor

$K_{Fv} = 2.88$ = format conversion factor

$C_{mv} = 0.97$ = wet service factor

Shoring design Timber lagging

6

• Bending

$$F_b = L_v \cdot \phi_v \cdot K_F \cdot F_b \cdot C_{Fu} \cdot C_m \cdot C_L \cdot C_F \cdot C_t$$

$$F_b = 5,700 \text{ psi}$$

$$S_{xv} = \frac{b \cdot d^2}{6} = 12 \text{ in}^3$$

$$f_b = \frac{M_{\text{moment}}}{S_{xv}} = 3,850 \text{ psi}$$

$$f_b < F_b \quad \checkmark \quad \text{O.K.}$$

• Shear

$$F_v = L_v \cdot \phi_v \cdot K_{Fv} \cdot F_v \cdot C_{Fv} \cdot C_t$$

$$F_v = 366.7 \text{ psi}$$

$$f_v = \frac{3 \cdot V_{\text{avg}}}{2 \cdot b_v \cdot d_v} = 135.2 \text{ psi}$$

$$f_v < F_v \quad \checkmark \quad \text{O.K.}$$

USE 4" x 8" rough cut southern
pine lagging.

Appendix H. Cost Estimation

LOW PRESSURE SYSTEM COST ANALYSIS

PART DESCRIPTION	QUANTITY	UNIT	PRICE PER UNIT (\$)	ESTIMATED PRICE (\$)
Lateral (Boundary) Kits	489.00	EACH	200.00	97800.00
Lateral (Boundary) Installation	489.00	EACH	1750.00	855750.00
Pump/Panel Installation	489.00	EACH	1750.00	855750.00
Simplex Grinder Pump Stations	489.00	EACH	2429.00	1187781.00
2.00" Piping	6061.00	L.F.	1.50	9091.50
2.50" Piping	2723.00	L.F.	2.01	5473.23
3.00" Piping	19829.00	L.F.	8.75	173503.75
4.00" Piping	12464.00	L.F.	2.76	34400.64
5.00" Piping	6912.00	L.F.	9.00	62208.00

TOTAL PRICE	\$3,281,758.12
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ESTIMATED COSTS CALCULATION TABLES

Table G.5. Open Trench Excavation Cost Estimates for Choto Rd. Gravity Sewer

Entering Manhole #	Pipe Dia. (in)	Pipe Area (SF)	Pipe Length (LF)	Required Width		Trench Width		Depth		Volume		Cost	
				d + 16" (in)	1.25 * d (in)	max{(d+16),(1.25*d)} (in)	(ft)	Backfill (ft)	Trench (ft)	Backfill (CY)	Excavation (CY)	Backfill P = \$181 / CY	Excavation P = \$6.27 / CY
1	8.0	0.35	360.0	24.0	10.0	24.0	2.0	5.0	6.0	133	160	\$ 24,133	\$ 1,003
2	8.0	0.35	360.0	24.0	10.0	24.0	2.0	5.0	6.0	133	160	\$ 24,133	\$ 1,003
3	8.0	0.35	360.0	24.0	10.0	24.0	2.0	5.0	6.0	133	160	\$ 24,133	\$ 1,003
4	8.0	0.35	360.0	24.0	10.0	24.0	2.0	5.0	6.0	133	160	\$ 24,133	\$ 1,003
5	8.0	0.35	360.0	24.0	10.0	24.0	2.0	5.0	6.0	133	160	\$ 24,133	\$ 1,003
6	8.0	0.35	360.0	24.0	10.0	24.0	2.0	5.0	6.0	133	160	\$ 24,133	\$ 1,003
7	8.0	0.35	360.0	24.0	10.0	24.0	2.0	5.0	6.0	133	160	\$ 24,133	\$ 1,003
8	8.0	0.35	360.0	24.0	10.0	24.0	2.0	5.0	6.0	133	160	\$ 24,133	\$ 1,003
9	12.0	0.79	360.0	28.0	15.0	28.0	2.3	6.0	7.0	187	218	\$ 33,787	\$ 1,365
10	14.0	1.07	360.0	30.0	17.5	30.0	2.5	6.0	7.0	200	233	\$ 36,200	\$ 1,463
11	14.0	1.07	360.0	30.0	17.5	30.0	2.5	6.0	7.0	200	233	\$ 36,200	\$ 1,463
12	14.0	1.07	360.0	30.0	17.5	30.0	2.5	6.0	7.0	200	233	\$ 36,200	\$ 1,463
13	14.0	1.07	360.0	30.0	17.5	30.0	2.5	6.0	7.0	200	233	\$ 36,200	\$ 1,463
14	14.0	1.07	360.0	30.0	17.5	30.0	2.5	6.0	7.0	200	233	\$ 36,200	\$ 1,463
15	14.0	1.07	360.0	30.0	17.5	30.0	2.5	6.0	7.0	200	233	\$ 36,200	\$ 1,463
16	14.0	1.07	360.0	30.0	17.5	30.0	2.5	6.0	7.0	200	233	\$ 36,200	\$ 1,463
PS	14.0	1.07	360.0	30.0	17.5	30.0	2.5	6.0	7.0	200	233	\$ 36,200	\$ 1,463
										2,853 CY	3,364 CY	Total \$ 516,453	\$ 21,095

*based on fully open trench instead of trenchless technology

*backfill unit cost based on TDOT Avg. Unit Prices 2019 for "Bedding Material (Pipe) Class B"

Table G.6. Pipe and Manhole Cost Estimates

Pipe Dia.	Quantity	Length (LF)	Total Length (LF)	Unit Cost	Subtotal Cost
8-in	8	360	2,880	\$ 100	\$ 288,000
12-in	1	360	360	\$ 100	\$ 36,000
14-in	7	360	2,520	\$ 100	\$ 252,000
Total			5,760		\$ 576,000

Manhole Size	Quantity	Unit Cost	Subtotal Cost
48-in	16	\$ 6,000	\$ 96,000

Pipe	\$ 576,000
Manholes	\$ 96,000
Total	\$ 672,000

Table G.7. Choto Rd. Gravity Sewer Total Estimated Cost

Task	Cost
Excavation	\$ 21,095
Backfill	\$ 516,453
Pipe	\$ 576,000
Manholes	\$ 96,000
Total	\$ 1,209,548

CHOTO RD PUMP STATION SITE PLAN

PART DESCRIPTION	QUANTITY	UNIT	PRICE PER UNIT (\$)	ESTIMATED PRICE (\$)	NOTES
Manhole 4-8 ft depth	1	Each	6200.00	6200.00	
Manhole 8-12 ft depth	2	Each	6230.00	12460.00	
20 in. DIP Gravity Sewer 6-12 ft depth	172	L.F.	375.00	64500.00	20 in. not availbe. 24 in. used for unit price
16 in. DIP Force Main	27	L.F.	112.25	3030.75	Pulled from 2018 Data
Granular Aggregate for Slab	12.9	Ton	56.00	722.40	599 ft^2 surface area *4in. Depth*1.75 tons/CY (TDOT DESIGN MANUAL)
PCC for Slab	8.3	C.Y.	144.20	1196.86	599 ft^2 surface area *4.5in. Depth
Chain Link Fence	90	L.F.	12.17	1095.30	
Excavaction (CUT)	67.1	C.Y.	6.27	420.72	Does not include wet well excavation, counted in wet well estimation
Granular fill	16.443	TON	31.90	524.53	Volume *2.03 ton/CY (TDOT DESIGN MANUAL)

CHOTO RD PUMP STATION STRUCTURAL PLAN

PART DESCRIPTION	QUANTITY	UNIT	PRICE PER UNIT (\$)	ESTIMATED PRICE (\$)
Concrete	48.03	C.Y.	108.00	5187.24
#4 BARS	1020	L.F.	4.32	4406.40
#5 BARS	884	L.F.	6.35	5613.40
#6 BARS	481	L.F.	9.27	4458.87
#7 BARS	1122	L.F.	13.12	14720.64
Form work	1948	S.F.	0.34	662.32
4 x 8 Lagging	2016	L.F.	3.25	6552.00
HP16x141	312	L.F.	50.00	15600.00
Wet Well Excavation	187.2	C.Y.	6.27	1173.74

TOTAL PRICE (SITE AND STRUCTURAL)	\$148,525.17
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CHANNEL POINT PUMP STATION SITE PLAN

PART DESCRIPTION	QUANTITY	UNIT	PRICE PER UNIT (\$)	ESTIMATED PRICE (\$)	NOTES
Catch Basin 0-4 ft	1	EACH	4500.00	4500.00	
12 in. PVC Pipe	8	L.F.	47.00	376.00	
Granular Aggregate for Slab	51.5	TON	56.00	2884.00	2386 ft^2 surface area *4in. Depth*1.75
PCC for Slab	33.1	C.Y.	144.20	4773.02	2386 ft^2 surface area *4.5in. Depth
Chain Link Fence	175	L.F.	12.17	2129.75	
CL Fence Gate	1	EACH	993.00	993.00	
Light on pole	1	EACH	43.81	43.81	
SCADA Equipment	1	EACH	25000.00	25000.00	PLC control typical costs from City of St. Charells
Power/Control Panel	1	EACH	1000.00	1000.00	Typical Price from manufactor at www.septicsolutions.com
Wet Well Excavation	720.4	C.Y.	6.27	4516.91	
Granular fill	1130	TON	31.90	36047.00	

TOTAL COST	\$82,263.49
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CHANNEL POINT PUMP STATION

PART DESCRIPTION	QUANTITY
4" DIA. DIP (FT)	30
4" PLUG VALVE	1
4" FLOW METER	1
4" GATE VALVE	2
4" PLUG VALVE	1
AIR RELEIF VALVE	1
4" 90DEG BEND	1
4" TEE	2
8' DIA. PRECAST CONCRETE WET WELL (3 PIECE 15' DEPTH)	1
FRP ENCLOSURE	1
ELECTRICAL PANEL	1
4" CHECK VALVE	1
GORMAN-RUPP SF4D-1	2
8" DIP (FORCE MAIN) (FT)	4000
LIQUID LEVEL PROBE	1
LIQUID LEVEL TRANSMITTER	1
BRACKETS	6

LOW BID	\$750,000
HIGH BID	\$1,200,000

CHOTO RD. PUMP STATION

PART DESCRIPTION	QUANTITY
8" DIA. DIP (FT)	75
8" 90 DEG BEND	2
8" CHECK VALVE	3
8" GV	3
8" PLUG VALVE	1
AIR RELEASE VALVE	1
8" TEE	3
GORMAN-RUPP SF6D-X	3
FRP ENCLOSURE	1
ELECTRICAL PANEL	1
8" FLOW METER	1
10"X8" REDUCER	1
LIQUID LEVEL PROBE	1
LIQUID LEVEL TRANSMITTER	1
BRACKETS	8

LOW BID	\$2,000,000
HIGH BID	\$3,000,000