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Wastewater Management Plan Report

Town of Coventry WPCA

December 2023

Tighe&Bond

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Executive Summary

The Town of Coventry owns and operates a Water Pollution Control Facility (WPCF) that treats wastewater generated in the area around Coventry Lake and then discharges it to the groundwater through a series of Rapid Infiltration Basins (RIBs). The WPCF is designed and permitted to treat an average daily flow of 200,000 gallons per day.

This Wastewater Management Plan was developed to evaluate the most appropriate method for upgrading Coventry's existing wastewater treatment system. The following two alternatives were evaluated:

- Upgrade the existing WPCF and its discharge to achieve compliance with current Connecticut Department of Energy and Environmental Protection (CTDEEP) standards.
- Cease on-site treatment and repurpose the TOWN's WPCF to pump (and possibly temporarily store peak flows of) wastewater to the Windham WPCF.

In order to plan for the future and evaluate the above alternatives, an evaluation of the existing wastewater infrastructure needs over the next 20 years was completed. This work included:

- Evaluation of areas of need within the Town that may require sewers in the future and development of an estimate of the associated future flows and loads.
- Evaluate the capacity of the existing WPCF and two existing pump stations to accept future flows and loads.
- Perform a desk-top level assessment of Infiltration/Inflow (I/I) within the TOWN's existing sewer collection system.

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Overview of Collection and Treatment System

Collection System:
The sewer collection system flowing to the Coventry Plant consists of approximately 16 linear miles of sewer mains, 460 manholes, two pump stations, and 53 Town-owned residential grinder pumps. Service is currently provided to 982 properties which equates to a total of 1,182 equivalent dwelling units (EDUs) per the Town's billing system. A breakdown of this piping is presented in Table EX-1:

TABLE EX-1
Gravity Sewer Summary

Diameter (in)	Approximate Length (LF)
8	57,270
10	1,530
12	20,720
15	4,000
18	1,580
Total	**85,100**

The gravity sewers flowing to the treatment plant were constructed in 1985 concurrent with the treatment plant construction. Sewers around Waungumbaug Lake were constructed in 2006.

Sanitary sewers are also being proposed in the northwestern corner of the Town, along Route 44. This collection system will flow to the Town of Manchester WPCF for treatment. Evaluation of the Route 44 collection system is not a part of this project.

A majority of the sewer service area consists of residential parcels surrounding Wangumbaug Lake. Many of the homes are seasonal, although some have transitioned to year round residences.

Pump Stations:
Coventry also has two Town owned pump stations, as summarized in Table EX-2. The Lakeview Drive station pumps flow to a gravity sewer that flows to the Avery Shores Station. Flow from the Avery Shores Station is pumped east to a gravity sewer on Edgewater Drive. From this point, flow is entirely gravity to the Coventry Treatment plant.

TABLE EX-2
Pump Station Summary

Pumping Station	Year Put into Service	Capacity[1] (gpm)	Description	Force Main Length/Material
Lakeview Drive	2006	330	Submersible	860 LF Ductile Iron Pipe
Avery Shores	2006	386	Submersible	1,200 LF Ductile Iron Pipe

[1]One Pump Running

Each pump station is a submersible station that contains the following major components:

1. Below-grade concrete wet well where the pumps are located.
2. Below-grade concrete valve vault containing the discharge piping, isolation valves, and flow meter.
3. Above-grade precast concrete building with vinyl siding that houses the electrical components and controls, as well as an emergency generator.

Treatment Plant:
The existing Water Pollution Control Facility (WPCF) is a primary treatment plant with a permitted capacity of 200,000 gallons per day (gpd). The WPCF treats flow from the Coventry collection system, which is primarily residential in nature.

The WPCF was originally constructed in 1985, and is located in the southeast corner of Town, bordered by the Willimantic River to the east and south, Route 31 to the west, and residential parcels to the north.

The major treatment processes at the WPCF include preliminary treatment (screening and grit removal), influent pumping (after screening), and primary treatment. Primary effluent is discharged to rapid infiltration basins (RIBs). Sludge from the treatment process is collected and disposed of as a liquid offsite. Treated effluent is discharged to the groundwater from the bottom of the RIBs.

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Existing Wastewater Flows and Loads

Treatment plant monthly operating reports for a 4 year period from 2019-2022 were reviewed, and a summary of the best estimate of existing wastewater flows and loads was developed. This summary is presented in Table EX-3.

TABLE EX-3
Historical WPCF Flows and Loads

Parameter	4 Year Data Range (2019-2022)	2 Year Data Range (2019-2022)	Use as Basis of Design for Existing Flows and Loads
Daily Flow (GPD – Taken Daily)			
Annual Average	**142,939**	141,192	**143,000**
Maximum Month	**306,550**	306,550	**307,000**
Maximum	**518,668**	518,668	**520,000**
BOD5 (lbs/day – Taken weekly)			
Annual Average	165	**188**	**190**
Maximum Month	364	**364**	**370**
Maximum	546	**546**	**550**
TSS (lbs/day – Taken Monthly)			
Annual Average	184	**218**	**220**
Maximum Month	475	**475**	**480**
Maximum	1127	**1127**	**1130**
TKN-N (lbs/day – Taken Monthly)			
Annual Average	47.8	**49.2**	**50.0**
Maximum	63.8	**63.8**	**64.0**
TP-P (lbs/day – Taken Monthly)			
Annual Average	**6.3**	5.5	**6.4**
Maximum	**19.8**	8.8	**20.0**

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Future Sewer Service Area

Water use data for customers already connected to the Coventry sewer system was used as the basis for estimating base and future sanitary flows. Base flows are as presented in Table EX-3. Future flows must be estimated prior to evaluating future treatment plant needs. To accomplish this, input on additional connections within the existing Sewer Service Area as well as areas that should be considered for the future sewer service area were discussed at a meeting held on March 13, 2023, with representatives from the Eastern Highland Health District, Coventry WPCA, and the Coventry Land Use Director and Zoning Enforcement Officer. Based on the results of this discussion, a proposed Future Sewer Service Area Map was developed. Areas of future sewer expansion fell into 3 main categories: Infilling within the existing sewer area, areas of need, and areas of future development.

Future Wastewater Flows and Loads

Future flows were based upon the following assumptions:

- All parcels within the boundaries defined on the updated Sewer Service Area that could feasibly be developed that are currently not connected to the sewer system will connect at some point over the 20-year planning period. Lots that have poor soils, wetlands and/or steep slopes will not be considered for future sewer service unless there is already a structure constructed on the property.
- Future average flows (and loads) for each customer class would be consistent with existing flows: residential (86 gpd), commercial (310 gpd), and industrial (584 gpd). This assumption is generally consistent with assuming that the expanded sewer area will have a low occupancy rate (86/75 = 1.1 person per EDU) consistent with the existing sewered area and much less that the remaining Town as a whole (2.65 persons per EDU). This appears to be a reasonable assumption as long as the service area remains close to the lake.
- An additional flow of 2,400 gpd was added to account for the 28 EDUs that could be constructed adjacent to St. Mary's church.
- An allowance for future flows should be added in the event that Accessory Dwelling Units (ADU) are allowed to be constructed. The minimum lot size needed to allow an ADU to be added is 25,000 square feet (0.57 acres). After discussion with the Town, it was determined that all lots greater than 0.57 acres would have the potential to add an ADU and that one fifth of these would construct ADU at some point in the future at a flow rate of 50 gpd per unit. A total of 204 parcels are greater than 0.57 acres. Assuming 1/5 of these parcels construct an ADU at 50 gpd/ADU equates to a future flow of 2,100 gpd (rounded) or 24 EDUs.
- An allowance of 10,000 gpd of flow would be added in case an increased density housing development is proposed at any location. This is equivalent to 117 EDUs.
- An allowance of 10,000 gpd of flow will be added due to the planned construction of a water tower in the downtown area, which increases the potential for additional commercial customers to be added in the Sewer Service Area. This is equivalent to 116 EDUs.
- An allowance of 5,200 gpd of flow will be added for a proposed 60 unit apartment building (60 EDUs) at 112 Woodlawn Road.
- An allowance of 15,000 gpd was also added for future I/I, as required by TR-16. This value is based upon an estimate of 7 miles of new 8-inch sewers being constructed and an infiltration rate of 250 gallons per day per inch mile of new sewer line.

As indicated in Table EX-4, the projected future average daily flow will increase by 107,800 gpd of sanitary flow plus 15,000 gpd of I/I for a new total of 265,800 gpd. This will be rounded to 266,000 going forward.

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TABLE EX-4
20-Year Projected Wastewater Flows – Annual Average

Existing Flows (Sanitary and I/I)	143,000
New Residential	59,600
New Commercial	14,900
New Industrial	3,600
St. Mary's Church	2,400
Accessory Dwelling Units	2,100
Increased Density Development	10,000
Water Tower Flows	10,000
Woodlawn Road Apartments	5,200
Total New Sanitary Flows	107,800
Total Current and Future Sanitary Flows	250,800
Allowance for Future Infiltration	15,000
Total 20-Year Projected Flow	***265,800***

Based on the future service area and the above future flows analysis, it is expected that flows and loads will increase by 1,092 EDUs. To be conservative, it was estimated that loads would increase by the equivalent of 1,201 people based on an occupancy of 1.1 persons per EDU which is 10% higher than the existing sewer service area.

Using the flows developed above, the TR-16 loading factors, and the above factor for increases in occupancy, the future annual average flows and loads are anticipated to increase as indicated in Table EX-5.

TABLE EX-5
Future WPCF Flows and Loads

Item	BOD5 (lb/day)	TSS (lb/day)	TKN Nitrogen (lb/day)	Phosphorus (lb/day)	Flow (Gal/Day)	Peak Hour Flow (GPM)
Existing Plant Flows/Loads	190	220	50	6.5	143,000	850
Future Customer Flows/Loads	208	245	48	7.2	108,000	110
Future I/I Allowance					15,000	80
Future Avg Design Criteria	**398**	**465**	**98**	**13.7**	**266,000**	**1,040**
Change over Existing	*109%*	*111%*	*100%*	*115%*	*88%*	*22%*
Change over WPCF Design	*-12%*	*-13%*	*18%*	*-19%*	*33%*	*81%*

Peak Flows coming into the plant are mainly caused by I/I during rain events. Because the feasibility and/or cost of future treatment/conveyance alternatives will be impacted by the peak flow that must be treated or conveyed, the plant's hourly flow data was used to understand them, and also estimate the on-site volume of storage that would be required if the peak flow treated on site or conveyed away from the plant was to be limited.

A detailed analysis was conducted to evaluate peak flows coming into the plant, as well as the potential for the existing clarigesters to be used for storage during peak flow events. This analysis concluded that for the purpose of a conservative design, the existing peak flows are 850 gpm and peak flows will increase to 1,040 gpm. This is 25% higher than 830 gpm which is recommended by TR-16 based on the Max Day Peak being 4.5 times the future average daily flow of 184 gpm/266,000 gpd.

Collection System Evaluation

A desktop evaluation of the amount of existing infiltration and inflow within the Coventry sewer system was completed by Martinez Couch & Associates (MCA) as a subconsultant to Tighe & Bond. Pump station and treatment plant flow information as well as historical rainfall data from the Town's SCADA system was used to evaluate flows over a three year period (2019-2022). A total of three subareas were evaluated:

- Sewers that flow to the Avery Shores Pump Station
- Sewers that flow to the Lakeview Drive Pump Station
- Sewers that flow directly to the Coventry WPCF.

A summary of the findings from this evaluation are as follows:

<u>Infiltration</u>

- A Seasonal variation of flow trends was observed in the collection system, which is indicative of an I/I response to increased rainfall and seasonal groundwater variations.
- Summer flows are lower than Spring flows for all flows evaluated.
- The collection system shows evidence of infiltration from the spring to the summer flows in the data analyzed. The peak infiltration is approximately 50,000 gpd. When compared to the size of the collection system, this amount of infiltration would not warrant additional investigations at this time.
- Approximately one-half the peak infiltration is noted to come from the Lakeview Drive Pump station sewer shed (24,000 gpd), and half coming from the gravity collection system flowing to the WPCF (22,400 gpd). Minimal infiltration (3,600 gpd) was observed coming from the Avery Shores pump station sewer-shed.

<u>Inflow</u>

- The use of rainfall data from the Hartford Brainard airport introduces variability in the repeatability of the analysis. Due to geographic separation, the total values of rainfall and hourly intensities may vary. This is seen in inconsistent volumes of inflow for rainfall events of similar magnitude and duration.
- For data after March of 2022, the inflow responses appear to have reduced significantly. This indicates that the modifications to the Avery Shores pump station eliminated a major source of inflow in the system.
- Even with the lack of reliable rainfall data, the data shows several significant rain events in 2021 with inflow volumes of 240,000 to 320,000 gallons with peak inflow rates of 24,000 to 32,000 gallons per day. For 2022, all rain events that were studied show inflow volumes of 15,000 to 100,000 gallons with peak inflow rates of 3,000 to 10,000 gallons per day.

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Conclusions and Recommendations

- The amount of infiltration appears to be well below the levels that would justify additional investigations at this time.
- There is an inflow component to the flows. However, the responses appear to have decreased since the recent pump station modifications. The storm response and inflow volumes identified after March 2022 do not appear to be at levels that justify further investigations. It was concluded that if flow responses continue to follow this trend, no additional study seems necessary. If an increase in storm response flows is observed in the future, smoke testing should be performed.

Pump Station Evaluation

As presented in Table EX-5, the average future flow to the Coventry WPCF is 266,000 GPD. As part of the Wastewater Management Planning, an evaluation of the existing pump station capacities was conducted to determine if the Avery Shores and Lakeview Drive Pump Stations have adequate capacity to accept these future flows.

Existing and future parcel counts for lots connected to each station were used as the basis for estimating future flows. Future peak flows were compared to the current pump station pump rate to determine if adequate capacity exists. Conclusions reached are as follows:

- Future peak flows to the Lakeview Drive Pump Station were estimated at 187 GPM. This is less than 60% of the pump station's capacity of 330 GPM. Therefore, the Lakeview Drive Pump Station has adequate capacity under future flow conditions.
- Future peak flows to the Avery Shores pump station were estimated at 405 GPM which is 5% higher than the current pump station capacity of 386 GPM and likely within the margin of error for this very conservative estimate. The estimate is conservative because of the following reasons:

  1) The peak flow from Lakeview will be sustained only several minutes before the pumps cycle off for several more minutes (because Lakeview's peak flow incoming flow is only 60% of its capacity). This means the peak flow entering the Avery Shores pump station's wet well from Lakeview Station will be reduced and sustained for a short period of time relative to the capacity of the Avery Shores wet well's working volume.
  2) It is not known when or if all lots within the Sewer Service Area will connect to the Coventry sewer system.

  For this reason, it can be concluded that the Avery Shores pump station appears to have adequate capacity for existing and future flows. This evaluation should be performed again when the pumps are replaced.

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Existing Treatment Plant Assessment

Section 5 of this report includes a detailed discussion of the condition and capacity of the following major components of the existing treatment plant:

- Preliminary Treatment
  - Screening
  - Pumping
  - Influent Flow Metering
  - Grit Removal
- Clarigester Evaluation
- Solids Handling
- Plant and Potable Water System
- Disinfection

<u>Available Space for New Facilities</u>:
If the decision is made to upgrade the existing WPCF, adequate space must exist for the construction of all new facilities, and provisions must be made to allow for treatment processes to remain operational during construction.

The WPCF currently utilizes 8 rapid infiltration basins (RIBs) for groundwater discharge. Discussions with operating personnel have confirmed that it is currently not feasible to take one or more RIBs out of service without impacting the ability of the plant to dispose of effluent during periods of high flow. The treatment plant site has a large area at the southern end of the property, part of which is being used as a cornfield.

Therefore, the following assumptions were made to allow the plant construction to move forward while still maintaining eight RIBs in service:

- Basin No. 1, which is adjacent to the existing headworks building, would be decommissioned. This space will be used for the construction of an upgraded treatment plant. Additional space between the clarigesters and the RIBS will be used if necessary.

- Basin No. 6 will be expanded to the southwest, partially into the cornfield to make up for the loss of Basin 1. Once the treatment plant upgrade is complete, the basin extension will be demolished, and the cornfield can be re-established.

<u>Additional Recommended Improvements</u>:
The existing wastewater treatment plant was constructed in 1985, and as such, there are a number of components that are at or nearing the end of their useful life. In addition to the treatment process components, an inspection of the existing treatment plant components was conducted from a structural and electrical perspective in March 2023. Input was also sought from operating staff on specific improvements that should be made. The following recommended basic improvements will be considered under the treatment plant upgrade and Windham connection alternatives as applicable.

<u>Structural</u>:

- Replacement of the existing roof and provide new exterior stairway for access.
- New larger door on the south side of the building near the laboratory.
- Interior painting of CMU walls in the generator room.
- Recoat concrete floors in the truckway.
- Repair building exterior where woodpecker damage was noted.

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Electrical:

- New electric utility service downstream of the existing transformer, including a new Main Circuit Breaker, Distribution Panelboard, replacement of the MCC and a new Lighting Panelboard
- LED Lighting Upgrades
- Conduit and Wire as applicable

Treatment Plant Analysis: Common Components

Discharge Location:
In order to handle the future flows and loads, the Coventry WPCF must be able to treat flows received to the level required for following alternatives considered in this study:

1. Fully Treat on-site and discharge to the groundwater
2. Fully Treat on-site and discharge to the Willimantic River

For the groundwater alternative, an evaluation was conducted of the feasibility of constructing a large-scale on-site subsurface wastewater absorption system (SWAS) to replace existing rapid infiltration basins (RIBs) which currently discharge treated municipal wastewater flows from the Coventry, CT Water Pollution Control Facility (WPCF). This analysis confirmed that maintaining the current groundwater discharge for the Coventry WPCF is not feasible. A technical memo summarizing this work was submitted to and approved by the Town and the Connecticut DEEP in July 2022. Therefore, the evaluation of a new treatment facility will therefore assume a new discharge to the Willimantic River.

Future Discharge Permit Limits/Design Criteria:
A discharge to the Willimantic River will require a NPDES discharge permit, which according to the CT DEEP is permittable because the river is a Class B river. A summary of the design criteria to be used in the treatment plan evaluation is summarized in Table EX-6 below:

TABLE EX-6
Surface Water Discharge Criteria

BOD5 (mg/l)	TSS (mg/l)	Ammonia Nitrogen (mg/l)	Total Nitrogen (mg/l)	Phosphorus (mg/l)	E-Coli (Col/100ml)	DO (mg/l min)
30	30	2	6	0.7	410	5

Common Upgrade Improvements:
Under all secondary treatment technologies, the influent pump station and headworks would be generally upgraded as follows:

- New headworks Influent Screens.
- The existing pump station with 8″ discharge pipes and 8″ suction pipes from the existing wet wells would not require modification if three pumps are selected to handle future flows.
- A larger influent magnetic flow meter would be provided.
- The existing aerated grit removal system would remain.

- The balance of the treatment process would ideally flow by gravity from the effluent of the existing aerated grit chamber.
- Three influent pumps would be provided to handle the design peak flow of 1050 gpm.

Secondary Treatment Technology Evaluation

Several secondary treatment technologies screened for potential use at the Coventry WPCF. This screening was performed to select the top three technologies that are the most appropriate to small system such as Coventry's, given the influent loads, anticipated effluent criteria, and the fact that the existing plant is currently staffed with 2 operators.

Specific technologies evaluated are as follows:

- Conventional Activated Sludge – Modified Ludzack-Ettinger (MLE)
- Conventional Activated Sludge – A20 Process – Anaerobic/Anoxic/Oxic
- Conventional Activated Sludge – 4 Stage Bardenpho
- Conventional Activated Sludge – 5 Stage Bardenpho
- Rotating Biological Contactors with Add On Treatment
- Moving Bed Biofilm Reactors
- Conventional Activated Sludge – Oxidation Ditch
- Sequencing Batch Reactors
- AquaNereda: Granular Sludge in an SBR Configuration
- Membrane Reactors

The three alternatives shortlisted are listed below, along with their advantages and disadvantages:

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Activated Sludge Oxidation Ditch

Advantages

- Does not require fine screens to protect equipment.
- Long solids retention time enhances removal of organic matter and suspended solids and also lowers sludge yield rates to minimize solids production.
- Resilient in handling temporary spikes in organic loading or hydraulic flow without significant adverse effects on treatment performance.
- Ease of operation and maintenance.
- Require relatively shallow tanks (on the order of 10-12 feet) that would not require deep excavation at Coventry's site allowing them to be constructed at a similar hydraulic grade line as the existing aerated grit chamber (allowing it to be reused).
- No Nitrate recycle pumps required.
- Larger size allows storage of sludge in the process prior to dewatering.

Disadvantages

- Requires larger footprints compared to SBR and MBR.

- Requires downstream clarifiers for solids separation.

Manufacturers

- EIMCO (now Ovivo) Carrousel® (Jewett City, Simsbury CT, UCONN, New Canaan, Suffield, Ansonia, Westport some without primary clarification with mechanical aerators.
- Evoqua Orbal® with brush style aerators (Seabrook NH)

SBR

Advantages

- Does not require fine screens to protect equipment.
- Small footprint and no need for a downstream clarifier or return sludge pumping.
- Lower energy consumption due to intermittent operation and the absence of continuous aeration during non-aeration phases.

Disadvantages

- Complex control and operation
- Peak flow handling required adjustment in cycle times and/or necessitating larger tank size and/or storage to handle for peak flow events.
- Potential for foaming and scum formation, which is difficult to remove (and often is not) due to the varying water level in the tank.
- Generates thinner sludge during each cycle, requiring an unthickened sludge storage tank prior to sludge thickening process. Such a tank would have to be aerated if biological phosphorus removal was to be relied upon.
- The batch nature of the process relies on deep/taller tanks (on the order of 20 feet) that would require the tanks be constructed deep into the ground (allowing reuse of the existing grit chamber) or up higher (requiring new a new aerated grit chamber).
- The batch nature of the process results in the rising and falling of the water level in the reactors effluent flows that start and stop. To allow for efficient disinfection (assuming UV), this would require additional post equalization tanks and either additional pumping or automated modulating valves. The net result of this could be a deep (buried) UV system and effluent pipe to the river or a tall SBR structure with new grit chamber and larger HP influent pumps.

Manufacturers

- Aqua Aerobics markets a true batch SBR system that is common in CT (Thomaston @ 1.3 MGD, Plainville @ 2.2 MGD) and is now being designed in Orleans MA
- Jet Tech (Shelton).

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MBR
Advantages
• Small footprint and no need for a downstream clarifier. • Resilience in handling temporary spikes in organic loading or hydraulic flow without significant adverse effects on treatment performance. • Reduced sludge production, resulting in potential cost saving for sludge disposal.
Disadvantages
• Fine screening is required. The Coventry WWTP headworks has spatial constraints that will likely prevent the installation of any form of fine screen system within the current headworks building. This would likely require a new headworks building to accommodate a fine screen. Even with Fine screening, hair can build up in tanks. • Higher capital and operating cost due to membrane modules inclusion and regular cleaning and replacement requirements. Higher peak flows require more membranes driving up costs for membranes of peak flow storage. • Higher energy consumption due to continuous aeration for biological process and membrane scouring. • Sensitive to high concentration of solids, grease, and certain chemicals, potentially requiring preliminary treatment. • Lower sludge settling rate, which may require chemical addition.
Manufacturers
• Ovivo with Kabota Plate Membranes (Wayland, MA @ 0.052 MGD, Westford MA @ 0.1 MGD, Southborough MA @ 0.03 MGD) • Zenon (were replaced after 6 years with Lane Christiansen in Redding/Georgetown CT)

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Short List Vendor Evaluation Methodology

In order to determine the most appropriate secondary treatment process to be used in Coventry, formal quotes were obtained from equipment vendors for each of the 3 short-listed technologies. Reference is made to Section 6 for a detailed description of the assumptions made for the purposes of evaluating the vendor responses, and specific information contained within each vendor response.

The matrix and ranking system illustrated in Table EX-7 was developed to summarize the above discussion and evaluate the three alternatives based on subjective criteria to allow them to be compared and ranked. A rank scoring system from 0 to 5 was used, with the lowest score being the best:

TABLE EX-7
Treatment Process Alternatives Comparison and Ranking Summary

Criteria	SBRs	Oxidation Ditches	MBRs
Impact on Existing Plant Processes	**Moderate +2** (All fit within RIB #1 near existing process building. All require construction of temporary RIB to allow taking RIB # 1 off line)		
New Buildings	**Not Required +2** Possibly aeration blowers and automatic valves may be protected in confined space)	**Not Required +0** (if Submersible RAS Pumps are used)	**Required +5**
Odor Potential	**Low +3** (Flow into raw wastewater in SBR's has an air gap)	**Low +2** (Submerged inlet)	**Low +2** (Submerged inlet)
Capability of meeting possible future regulated contaminants	**Low +3** (Post EQ pumps could be replaced with high head pumps if needed to pump to add on process)	**Low +4**	**Low +4**
Process Complexity	**Medium +4**	**Low +0**	**High +5**
Maintenance	**Medium-High +3** (3 Floating Decanters, 3 Floating Mixers, 6 retrievable/1 fixed aeration grids, 3 WAS pumps. 4+ Blowers, 2 Post EQ pumps, many instruments)	**Medium +1** (10 submersible mixers, 2 aerators, 4 RAS/WAS pumps, 2 clarifiers)	**High +5** (Membranes are maintenance intensive)
Reliability	**Medium +3** (Many components to be controlled on hourly basis)	**High +0** (Can run in hand)	**Low +5** (Membranes subject to fouling, Fine screens, multiple auto controls)
Total Ranking Score	20	9	28
Relative "Subjective Cost" Score	20/20 = 1.00	9/20 = 0.45	N/A - Eliminated

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After reviewing both MBR proposals and performing the subjective ranking criteria presented above, the evaluation of MBRs was eliminated from moving forward into the life cycle cost analysis.

Upgraded Plant Classification

The CT DEEP utilizes a point and scoring system to determine the classification of a wastewater treatment facility. Points are assigned based upon the proposed flows and treatment processes. The scoring of an upgraded Coventry plant was reviewed with the CT DEEP on November 11, 2023, and a copy of the scoring sheet is included in Appendix D. Based on the agreed upon scoring, it is not possible to keep the plant as a Class II facility. The plant, if upgraded, would become a Class 3 plant regardless of the selection of the short listed secondary technology.

Staffing Impacts

A staffing analysis for an upgraded Coventry WPCF was developed using the guidance document developed by the New England Interstate Water Pollution Control Commission (NEIWPCC) titled "The Northeast Guide for Estimating Staffing at Publicly and Privately Owned Wastewater Treatment Plants," dated November 2008. The results of the analysis are as follows:

- Analysis of staffing requirements for the current Coventry WPCF in accordance with the NEIWPCC guidance document indicates a need for approximately 1.2 operation/maintenance staff. This staffing guide includes only the plant and not the collection system and pump stations, which explains in part why this is less than the current staffing level of 2 personnel.
- Analysis of staffing requirements for the upgraded Coventry WPCF in accordance with the NEIWPCC guidance document indicated a need for the following staffing requirements at current flows:
  - SBR: 2.6: (less existing 1.2) suggesting adding 1.4 staff
  - Oxidation Ditch: 2.4 (less existing 1.2) suggesting adding 1.2 staff

Assuming the WPCF is upgraded, the current WPCF staffing levels would need to be increased by at least 1 possibly 2 personnel (more likely if an SBR was selected).

Treatment Plant Cost Evaluation

Capital Costs:
Construction costs for each treatment plant were developed based on the following costs and assumptions:

- Each treatment plant option would require the construction of a temporary RIB followed by decommissioning of all RIBs.
- Excavation, dewatering, and backfill for all new treatment plant components. Allowances for sheeting were included for tanks .
- Demolition of existing equipment, piping and valves in the influent pump room, lower level, and solids handling room following by installation of new equipment/piping/valves as applicable.
- New influent mechanical bar screen, slide gates, plus an allowance for a future washer/compacter.
- Three new dry pit submersible influent pumps.
- Secondary treatment equipment specific to the SBR and Oxidation Ditch technology with specific components as discussed above.
- New spray system for the SBR option
- Solids Handling system (RDT, pumps, blowers, polymer and odor control) as discussed above.
- UV Disinfection
- New plant water system

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- Chemical feed systems for phosphorus removal and sludge handling including provisions for a safety shower and eyewash station.
- Structural improvements including process tanks, equipment pads, ladders, railings, handrails, hoists and pipe supports.
- Building improvements including new roof, exterior stair access new door, and exterior repairs
- Painting of generator room walls and coating of concrete floors in truckway
- Electrical improvements including new generator, new electric utility service where applicable, new Main Circuit Breaker, Distribution Panelboard, replacement of the MCC and a new Lighting Panelboard
- LED Lighting Upgrades.
- VFDs
- Effluent flow metering
- Headworks and secondary process control systems with Mission Monitoring.

Equipment costs are based upon quotes obtained from manufacturers and/or pricing received on similar treatment plant projects in Connecticut. Suitable markups were added for costs of manufacturer's services, and installation. All construction costs are based on the December 2023 ENR Construction Cost Index of 13514.7.

The following costs were also applied to the capital cost of each option:

- 20% Contractor Overhead and Profit
- 30% Contingency
- 20% Engineering

<u>Operation and Maintenance Costs</u>:
Operation and Maintenance costs were estimated as follows:

The existing Coventry WPCA budget was used as a starting point for each option, since many of the existing costs associated with staffing and maintenance of the collection system, pump station, and treatment plant site will remain. Additions and deductions to applicable budget line items were made as detailed below.

- Additional staffing costs based upon the NEIWPCC staffing spreadsheets.
- Chemical costs for nutrient removal and solids handling.
- Electrical costs for operating the various treatment facility components were estimated based upon the expected KW usage and operating times at a rate of $0.15/KWH based upon data from Coventry's 2021-2023 electric bills. Adjustments in usage were also made for equipment shut down at the plant.
- Laboratory costs based upon the expected sampling requirements for a Class III secondary treatment facility. An allowance for future influent, effluent and PFAS sampling was included in both the SBR and OD alternatives based upon the guidance received from the CT DEEP.
- Sludge disposal costs were estimated based upon a unit charge of $0.30/gallon.

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A life cycle (present worth) cost analysis was performed for each alternative based upon a 20-year service life, including capital costs, equipment replacement costs, and annual O&M costs. O&M costs are based upon flows, loads, and estimates of labor, energy, chemical use, and sludge disposal.

For the purposes of this analysis, it was assumed that flows and loads will increase to 50% of the planned growth 10 years after construction, and the remaining growth will take place by the 20th year.

All operation and maintenance costs were estimated annually over a 20 year planning period, with O&M costs starting one year after the capital costs. The present worth costs are determined based upon a 2.5% discount rate (Department of Interior Federal Water Resources Planning published rate for FY2023) and an assumed 10 year breakeven inflation rate of 2.22% (according to the Federal Reserve December 1, 2023). This method allows the use of current O&M costs in the present worth calculations. The net present worth of the O&M costs was utilized in evaluating the costs of each alternative.

Funding Impacts

The CTDEEP has indicated that the upgraded treatment plant option would qualify for a 20% conventional grant, as well as a 30% nutrient reduction grant. It is noted that the grant percentages do not apply to the same costs. The 30% grant is only for the cost of the components of wastewater treatment projects which relate directly to the denitrification or phosphorus reduction processes. The 20% conventional grant would apply to the remaining applicable project costs.

DEEP has also indicated that the project will be required to go through an eligibility determination in order to establish the final grant percentage. The balance of the project costs would qualify for a 2% loan under DEEP's funding program.

All projects funded by the State's Clean Water Program are required to comply with the Buy America Build America (BABA) Requirements. This program requires that all iron, steel, manufactured products and construction products used on a project be produced in the United States. This is an expansion of the American Iron and Steel (AIS) requirement which requires the use of iron and steel products that are produced in the United States. The BABA requirements expand into equipment components, many of which were previously produced overseas. As a result, some manufacturers have indicated that equipment costs can be increased by as much as 40% if BABA requirements apply.

BABA requirements apply to all projects that initiated planning after May 14, 2022. The Town of Coventry issued the Request for Qualifications for this Wastewater Management Report on March 31, 2022. Therefore, a request for a waiver was made to the DEEP in December 2022. In an email dated December 14, 2022, the DEEP concurred that BABA requirements do not apply to the Coventry project. However, the project will be required to meet AIS requirements. When developing costs for the treatment plant options, it was assumed that a 30% grant would be obtained for costs associated with denitrification and phosphorus removal. The remaining costs would qualify for a 20% grant.

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Windham Connection Alternatives Analysis

Wastewater Flows/Connection Specifics:

A meeting was held with the Town of Windham on June 26, 2023. At the meeting, the following items related to Coventry's proposed connection were discussed and generally agreed upon:

- The Windham treatment plant has the capacity to accept Coventry's future average, maximum month and peak flows of 266,000 gpd, 559,000 gpd and 1.5 MGD (rounded to 1050 gpm for the purpose of this evaluation), respectively.
- Influent screening will be required.
- Odor control in the force main may be required.
- A flow meter must be installed to measure flows being sent to Windham for billing purposes.
- The connection to the Windham collection system should take place at a manhole located on the north side of Route 6. This 21" RCP interceptor will be able to handle the proposed peak flows without any capacity issues. No Windham pump stations will be impacted if the connection is made at this location, as all flow is gravity from this point to the treatment plant.

New Pump Station and Force Main:
It was determined that the new pump station would be located on the existing treatment plant site. A detailed hydraulic evaluation concluded that three new pumps would be provided. This evaluation also concluded that dual 8-inch diameter force mains should be installed to transmit the flow to Windham.

Force Main Routing

A total of 5 force main routes were evaluated to pump flow from the Coventry plant to Windham, as listed below:

Alternative 1: Route 32
Alternative 1 involves crossing the Willimantic River and New England Central Railroad directly east of the WPCF. Once beyond the railroad tracks, the force main would be installed through an easement area until it reaches Route 32. The sewer force main would then run south down Route 32 and then go off-road along Route 6 to the proposed connection manhole.

Alternative 2: Route 31
Alternative 2 involves installing the sewer force main along Memorial Drive and onto Route 31. The force main would then run south down Route 31 and cross the Willimantic River and the New England Central Railroad. At the end of Route 31, the sewer force main would continue south down Route 32 and then go off-road along Route 6 to the proposed connection manhole.

Alternative 3A: Depot Road (Bridge Attachment)
Alternative 3A involves installing the sewer force main briefly along Memorial Drive and then going through an easement area north of the plant along the Willimantic River, towards Depot Road. The sewer force main pipe must be installed at least 75 feet away from the WPCF potable water well. The force main could potentially be attached to the railroad bridge crossing just south of Depot Road. Attaching the force main pipe to the bridge would allow the pipeline

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to cross both the Willimantic River and the railroad in one location, however, easements would need to be obtained on either side of the bridge crossing for this alternative to be feasible. Once across the bridge, the force main would then run from Depot Road to Route 32, head south down Route 32 and then go off-road along Route 6 to the proposed connection manhole.

Alternative 3B: Depot Road
Alternative 3B is the same layout as Alternative 3A with the only difference being that the sewer force main would cross the railroad bridge and the river from Depot Road, with two crossings required in lieu of a single crossing.

Alternative 4: Jude Lane
Alternative 4 involves installing the sewer force main along Memorial Drive briefly and then south through the treatment plant until it is in line with Jude Lane to the east. The force main would then cross the Willimantic River and railroad tracks before exiting along Route 32. The sewer force main would then run south down Route 32 and then go off-road along Route 6 to the proposed connection manhole.

A detailed evaluation of the proposed pipeline installation methodologies and construction feasibility concluded the following construction approach:

Alternative 1: Horizontal directional drill under Willimantic River and railroad to CT Route 32, open cut excavation along CT Route 32, cross Cider Mill Brook via open cut excavation, then open cut along CT Route 32 and CT Route 6.

Alternative 2: Open cut excavation on Memorial Drive and CT Route 31, cross Willimantic River via bridge attachment, cross railroad via horizontal directional drilling, open cut excavation along CT Route 31 and 32, cross Cider Mill Brook via open cut excavation, then open cut along CT Route 32 and CT Route 6.

Alternative 3A: Not feasible.

Alternative 3B: Open cut excavation on Memorial Drive and off-road to Depot Road, cross Willimantic River and railroad via horizontal directional drilling, open cut along Depot Road and CT Route 32, cross Cider Mill Brook via open cut excavation, then open cut along CT Route 32 and CT Route 6.

Alternative 4: Not feasible

Windham Connection Cost Evaluation:

Capital costs for the installation of the force main along the feasible routing alternatives were developed based upon unit prices from recently bid Tighe & Bond projects, CTDOT 2023 Estimating Guidelines and input received from directional drilling contractors using the following assumptions:

- Installation of two 8-inch diameter force mains.
- Temporary paving will be 2-inches thick within local roads and 4-inches in state roads.
- Final paving will be 4-inches thick in local roads and 9-inches thick in state roads. Full width mill/overlay would also take place on State roads.

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- A rock excavation quantity based upon 6" of rock removal along each force main route. The presence of rock and quantity of rock removal must be confirmed during final design.
- Horizontal direction drilling was assumed to cost $600/LF plus an additional $200,000 for the railroad casing. An additional cost of $75/LF was added for the new force main piping to be installed within the casing.
- Air release manholes are required at high points along each route and would be installed at a cost of $15,000 for the valve and manhole.
- Allowances for Clearing and Grubbing, (2%), Mobilization/Demobilization (10%), Maintenance and Protection of Traffic (3.5%).
- Railroad fees of $4,500 per crossing plus $20,000 for railroad flagmen.
- Easement costs of $15,000 per parcel as indicated by the Town of Coventry.
- An allowance for the DOT Encroachment Permit Bond in the amount of $12/$1,000 of contract value.

<u>Capital costs for the new Pump Station</u> are based upon the following assumptions:

- New mechanical influent bar screen, with an allowance for a future washer/compacter.
- Three new pumps with VFD controls will be installed in the existing pump room, along with new discharge piping and valves as required.
- Flow meter with SCADA connection to Windham.
- A new generator will be provided, to be installed outside the building.
- Miscellaneous electrical and building improvements as discussed above.
- Bioxide odor control system for the force main.
- Decommissioning of the existing RIBS and the temporary RIB.
- Demolition of the existing aerated grit chamber and clarigesters mechanisms.

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All construction costs are based on the December 2023 ENR Construction Cost Index of 13514.76

The following costs were applied to the capital cost of each route:

- 20% Contractor Overhead and Profit
- 30% Contingency
- 20% Engineering

<u>Operation and Maintenance costs</u> were estimated as follows:

- The existing Coventry WPCA budget was used as a starting point for each option, as many of the existing costs associated with staffing and maintenance of the collection system, pump station, and treatment plant site will remain. Additions and deductions where applicable to the budget line items were made as detailed below.
- Staffing costs will remain unchanged.
- Chemical costs of $18,600 per year for force main odor control were added based upon 34 gallons bioxide being used for six months per year at a cost of $3/gallon.

- Electrical costs for operating the new pumps and related equipment (mechanical screen) were estimated based upon the expected KW usage and operating times at a rate of $0.15/KWH based upon data from Coventry's 2021-2023 electric bills. Adjustments in usage were also made for equipment shut down at the plant.
- Laboratory and sludge disposal costs will be eliminated.

A life cycle (present worth) cost analysis was performed based upon the assumptions listed for the treatment plant upgrade.

<u>Windham Fees</u>:

Connection Fee:
An initial connection fee of $2,100,000 was calculated based upon Windham's fee schedule and the number of sewered parcels in Coventry.

Annual Usage Fee:
The annual usage fee for treatment at the Windham wastewater treatment facility will be based upon the total amount of flow sent by the Town for treatment. Windham has indicated that the charge rate will be based upon Windham's cost to treat the wastewater, which is currently $6,454 per million gallons.

Flows were estimated to be 150,000 gpd during the first year. For the purposes of this evaluation, it was assumed that flows will gradually increase, with 50% of the future connections taking place by Year 10, and 100% of the future connections taking place by Year 20. An inflation factor of 2.2% was also applied to the annual charge being assessed.

Share of Future Plant/Collection System Upgrade Costs:
Windham has indicated that the Town of Coventry would be expected to pay for a share of improvements to both the treatment facility and collection system. Specifics of such an agreement, if this option is recommended, would be developed during the negotiation phase.

The payment cost would be based upon the percentage of flow being sent by Coventry. At this time, it is unclear as to whether the percentage would be based on the Windham plant's design flow of 5.5 MGD (2.8% +/-), or the actual flow which is currently 2.0 MGD (7.5%). For the purposes of this evaluation, it was assumed that an improvement project of $2,000,000 would take place at the treatment plant and/or collection system at Years 5, 10, 15 and 20, with Coventry paying 5.25% of the costs, which is the approximate average of the existing and future flow percentages.

Funding Impacts

The CTDEEP has indicated that the Windham connection option is expected to qualify for a 20% conventional grant due to the fact that it is a wastewater conveyance project. However, DEEP has also stated that the project will be required to go through an eligibility determination in order to establish the final grant percentage. The balance of the project costs would qualify for a 2% loan under DEEP's funding program.

As with the treatment plant costs, BABA Requirements do not apply, but the project will be required to meet AIS requirements.

Construction cost estimates were therefore adjusted based upon the assumption of a 20% grant being awarded.

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Pipeline Cost Summary

A summary of the construction costs for the 3 Route Alternatives is presented in Table EX-8. As noted, Alternative 2: Route 31 is the least expensive, followed by Alternative 1: Route 32 and then Alternative 3B: Depot Road.

TABLE EX-8
Opinion of Probable Construction Cost (OPCC) – Summary of Three Routes

Route	Total Cost
Alternative 1: Route 32	$19,410,000
Alternative 2: Route 31	$18,650,000
Alternative 3B: Depot Road	$20,240,000

Inasmuch as the Route 31 Alternative is the least expensive, it is recommended that this alternative be selected to move into the final comparison phase to determine if Coventry should connect to Windham or upgrade its plant.

Summary of Total Life Cycle Project Costs

Life cycle costs for all three alternatives were developed as listed below.

- Capital costs for the treatment plant alternatives include all initial construction. The Windham connection capital costs include the force main construction cost presented in Table 7-8 plus the costs of work needed to construct the new pump station at the treatment plant site, as well as initial construction fees that would have to be paid to Windham.
- Operation and Maintenance costs represent the net present worth of expected costs over the 20 year planning period for this project.
- An estimate of potential grants to be received from the CT DEEP was subtracted off of each option based upon the qualifying percentages discussed earlier.

A summary of the total life cycle costs for the three alternatives is presented in Table EX-9.

As indicated in Table EX-9 the present worth life cycle cost of the Windham alternative is much more costly than upgrading the plant and going to a river discharge. Therefore, it is recommended that the Windham alternative be dropped.

It should be noted that the planning level capital costs and present worth O&M costs include many assumptions. These are discussed in detail in the appropriate sections above. For the on-site treatment plant alternative, conservative assumptions and opportunities for value engineering were discussed that could result in the lowering of the cost to Coventry. Scaling back future growth plans could also further reduce capital costs for the on-site treatment alternative. For the Windham alternative, it was noted that there is a less than conservative assumption and risk for increased costs in regard to connection fees for future Coventry users. Scaling back future growth plans would have little cost impact on capital cost for the Windham alternative. These points further support the conclusion that pursuing on-site treatment is in Coventry's best interest.

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Table EX-9
Detailed Life Cycle Cost Analysis Summary

Tighe&Bond
Engineers | Environmental Specialists

				Alternative #1		Alternative #2		Alternative #3	
				Discharge to the River		**Discharge to the River**		**Force Main to Windham**	
				Sequencing Batch Reactor		Oxidation Ditch			
Item	Category			% Grant	Cost	% Grant	Cost	% Grant	Cost
	Contract #1 Plant Construction Costs								
1	General Conditions (10% of Items 2 to 19)[1]			23%	$1,295,000	23%	$1,313,000	20%	$264,000
2	Construct Temporary RIB			25%	$170,000	25%	$170,000	20%	$136,000
3	Decommissioing of RIBs			20%	$444,000	20%	$444,000	20%	$438,000
4	Demolition & Site Civil - General			20%	$547,000	20%	$377,000	20%	$170,000
5	Demolition & Site Civil - Secondarys			25%	$1,236,000	25%	$528,000		
6	Outfall to River from Main Plant Area			20%	$460,000	20%	$460,000		
7	Concrete Tanks - Secondary Treatment			25%	$3,234,000	25%	$3,678,000		
8	Concrete Tanks - Solids Handling			20%	$404,000	20%	$679,000		
9	Concrete - Other			20%	$39,000	20%	$30,000	20%	$5,000
10	Plant Process Improvements - Headworks			20%	$731,000	20%	$731,000	20%	$812,000
11	Plant Process Improvements - Primary Clarifiers/Storage			20%	$910,000	20%	$300,000		
12	Plant Process Improvements - Secondary Equipment Installed			25%	$1,846,000	25%	$2,942,000		
13	Plant Process Improvements - Additional Process Equipment			20%	$84,000	20%	$84,000		
14	Plant Process Improvements - Solids Handling Equipment			20%	$903,000	20%	$903,000		
15	Plant Process Improvements - Chemical Feed System Equipment			30%	$89,000	30%	$89,000	20%	$141,000
16	Plant Process Improvements - Disinfection Equipment			20%	$269,000	20%	$255,000		
17	Other Plant Improvements			20%	$311,000	20%	$311,000	20%	$311,000
18	Electric & Controls Improvements			20%	$537,000	20%	$531,000	20%	$630,000
19	Electric & Controls Improvements - Secondary Treatment			25%	$739,000	25%	$614,000		
20	**Subtotal (Above Items)**				**$14,200,000**		**$14,400,000**		**$2,900,000**
21	Contractor Overhead & Profit[1]	at	20%	23%	$2,840,000	23%	$2,880,000	20%	$580,000
22	Subtotal (Above 2 Items)				$17,040,000		$17,280,000		$3,480,000
23	Contingency [1]	at	30%	23%	$5,120,000	23%	$5,190,000	20%	$1,050,000
24	Subtotal (Above 2 Items)				$22,160,000		$22,470,000		$4,530,000
25	Engineering Cost[1]	at	20%	23%	$4,440,000	23%	$4,500,000	20%	$910,000
26	**Total Contract #1 Plant Construction Costs (Above 2 Items)**				**$26,600,000**		**$26,970,000**		**$5,440,000**
27	**Total Contract #2 Force Main Costs**				**$0**		**$0**	20%	**$18,650,000**
28	**Connection Fee for Current Customers - Paid to Windham**				**$0**		**$0**	0%	**$2,100,000**
29	**Total Capital Costs (Above 3 Items, Rounded)**				**$26,600,000**		**$27,000,000**		**$26,200,000**
	O&M Costs (20 Years Present Worth)								
30	Current WPCA Budget (Year 0 @ $502,000)				$9,757,000		$9,757,000		$9,757,000
31	Additional Staffing Costs				$2,449,000		$2,099,000		$0
32	Additional Chemical Costs				$120,000		$127,000		$363,000
33	Additonal Electrical Utility Costs				$511,000		$618,000		$15,000
34	Additonal Permit Compliance Costs (Lab, Permit fees etc)				$391,000		$371,000		($207,000)
35	Additional Sludge Disposal Costs				$1,507,000		$1,339,000		($606,000)
36	Additional Equipment Maintenance				$861,000		$861,000		$328,000
37	Sewer User Fees - Paid to Windham				$0		$0		$8,214,000
38	Allowance for Capital Improvements Assessments - Paid to Windham				$0		$0		$406,000
39	Future Sewer Connection Fees - Paid to Windham				$0		$0		$0
40	**Total O&M Costs (20 Years Present Worth)**				**$15,600,000**		**$15,200,000**		**$18,300,000**
41	**Total Present Worth Capital & O&M Costs (Items 29+40)**				**$42,200,000**		**$42,200,000**		**$44,500,000**
42	Potential Grants (Based on Percentages given above, rounded)				$10,000,000		$10,200,000		$5,660,000
43	**Present Worth Cost to Town after Grants (Item 41-42)**				**$32,200,000**		**$32,000,000**		**$38,840,000**
44	**Town Capital Costs less Potential Grants (Item 29-42)**				**$16,600,000**		**$16,800,000**		**$20,540,000**

Notes
1 - Grant percentage for these items based on weighted average of grant eligibility for other items

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Recommended Alternative

As indicated in Table EX-9, the present worth cost of the two on-site treatment alternatives are nearly identical and the difference is small compared to the expected accuracy of these planning level cost estimates.

The recommended technology for the river discharge should be based on both costs and other subjective criteria. In order to recommend a technology for on-site treatment and river discharge, the relative dollar cost score was developed by dividing both of the life cycle costs for each alternative by the highest cost. As shown in Table EX-10, the numbers are identical, so each technology receives a dollar cost score of 1.0.

TABLE EX-10
Alternatives Comparison Table

Costs	SBR	Oxidation Ditch
Capital Cost	$26,600,000	$27,000,000
20 Yr - O&M Cost	$15,600,000	$15,200,000
Total Life Cycle Cost	$42,200,000	$42,200,000
Relative "Dollar Cost" Score	42.2/42.2 = 1.0	42.2/42.2 = 1.0

The relative subjective cost score developed in Table EX-7 was then combined with the dollar cost share. To combine these two relative costs, a weighting of 70% for the "Dollar Costs" and 30% for the "Subjective Costs" was assumed, with the lowest total value being the recommended alternative. CT DEEP has accepted this weighting for the selection of technologies for phosphorus upgrades subjective in recent projects in Vernon and Southington. This method/evaluation is summarized in Table EX-11 and concludes that the technology with the lowest total relative cost is the Oxidation Ditch.

TABLE EX-11
Alternatives Scoring Comparison Table

Relative Cost Score	SBR	Oxidation Ditch
Dollar (From Table 8-5)	1.0	1.00
Subjective (from Table 6-2)	1.0	0.45
Total Relative Cost = 0.8*Dollar +0.2*Subjective	**1.0**	**0.89**

Based on this above analysis, on-site treatment with a river discharge using oxidation ditch technology is the recommended approach for addressing the current concerns with the operation of the existing RIBs at the Coventry Plant.

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Section 1 Introduction

1.1 Background

The Town of Coventry owns and operates a Water Pollution Control Facility (WPCF) that treats wastewater generated in the area around Coventry Lake and then discharges it to the groundwater through a series of Rapid Infiltration Basins (RIBs). The WPCF is designed and permitted to treat an average daily flow of 200,000 gallons per day.

This Wastewater Management Plan was developed to evaluate the most appropriate method for upgrading Coventry's existing wastewater treatment system. The following two alternatives were evaluated:

- Upgrade the existing WPCF and its discharge to achieve compliance with current Connecticut Department of Energy and Environmental Protection (CTDEEP) standards.
- Cease on-site treatment and repurpose the TOWN's WPCF to pump (and possibly temporarily store peak flows of) wastewater to the Windham WPCF.

In order to plan for the future and evaluate the above alternatives, an evaluation of the existing wastewater infrastructure needs over the next 20 years was completed. This work included:

- Evaluation of areas of need within the Town that may require sewers in the future and development of an estimate of the associated future flows and loads.
- Evaluate the capacity of the existing WPCF and two existing pump stations to accept future flows and loads.
- Perform a desk-top level assessment of Infiltration/Inflow (I/I) within the TOWN's existing sewer collection system.

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1.2 Project Regulatory History

The Town of Coventry had been under order by the State of Connecticut to address pollution issues around Waungumbaug Lake. The following is a list of past consent orders issued to the Town of Coventry by the State of Connecticut:

1. Original Order No. 916, dated February 22, 1971: required the Town to design, bid and construction collection and treatment system improvements by January 31, 1976.
2. Modified Order No. 916, dated December 15, 1975: modified the order to include the preparation of a Facilities Plan and modified the date of construction completion to September 30, 1979.
3. Modified Order No. 916, dated September 27, 1976: modified the date of construction completion to July 31, 1981.

4. Superior Court Ruling dated September 30, 1983: ordered the completion of the construction of new facilities no later than August 31, 1986.
5. Consent Order WC-5243 dated April 9, 1998, replaced by Consent Order WC-5247 dated January 4, 1999: required the evaluation of wastewater disposal needs for small residential properties around Waungumbaug Lake through the completion of an engineering report, followed by the construction of all recommended improvements by December 31, 2004.
6. Certificate of Compliance for Consent Order WC-5247 dated January 2, 2013, confirmed that the Town was now in compliance with the Consent Order.

1.3 Current Project Drivers

The existing WPCF groundwater discharge permit from the CTDEEP expired in 1995 and because of evidence of wastewater seepage near the RIBS along the Willimantic River, CTDEEP has not renewed the permit. Based on discussions with CTDEEP, they would not issue a new permit until the Town identifies an optimal long term wastewater management plan that will require the Town to address the issue of seepage from the RIBS. They also noted that the WPCF does not meet current standards for treatment and on-site ground disposal systems further complicating the permitting process. This Wastewater Management Plan is intended to develop, evaluate and provide recommendations for a cost effective solution to this issue.

At this time, it appears that a new discharge permit to either groundwater or surface water will require that the WPCF be upgraded from a primary treatment process to a secondary treatment process with nutrient removal capability. Based on preliminary discussions with CTDEEP, it is anticipated that a new surface discharge permit will have limits similar to the Windham WPCF and a new groundwater discharge permit would require that the WPCF meet current design standards.

The second alternative to be evaluated consists of ceasing treatment and discharge of treated water at the existing WPCF and pumping untreated wastewater to Windham.

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Section 2
Existing Wastewater Infrastructure Overview

This section presents a brief overview of the existing wastewater collection and treatment facilities within the Town of Coventry. An overview of these facilities along with sewered parcels is presented in Figure 2-1.

2.1 Collection System

The sewer collection system flowing to the Coventry Plant consists of approximately 16 linear miles of sewer mains, 460 manholes, two pump stations, and 53 Town-owned residential grinder pumps. Service is currently provided to 982 properties which equates to a total of 1,182 equivalent dwelling units (EDUs) per the Town's billing system. A breakdown of this piping is presented in Table 2-1:

TABLE 2-1
Gravity Sewer Summary

Diameter (inches)	Approximate Length (linear foot)
8	57,270
10	1,530
12	20,720
15	4,000
18	1,580
Total	**85,100**

The gravity sewers flowing to the treatment plant were constructed in 1985 concurrent with the treatment plant construction. Sewers around Waungumbaug Lake were constructed in 2006.

Sanitary sewers are also being proposed in the northwestern corner of the Town, along Route 44. This collection system will flow to the Town of Manchester WPCF for treatment. Evaluation of the Route 44 collection system is not a part of this project.

A majority of the sewer service area consists of residential parcels surrounding Wangumbaug Lake. Many of the homes are seasonal, although some have transitioned to year round residences. Approximately 367 sewer customers have water service provided by the Connecticut Water Company. The remaining customers receive water from wells. In addition to the residential customers, there are 17 commercial and 2 industrial properties, 3 Town owned schools and 1 church currently connected to the sewer system.

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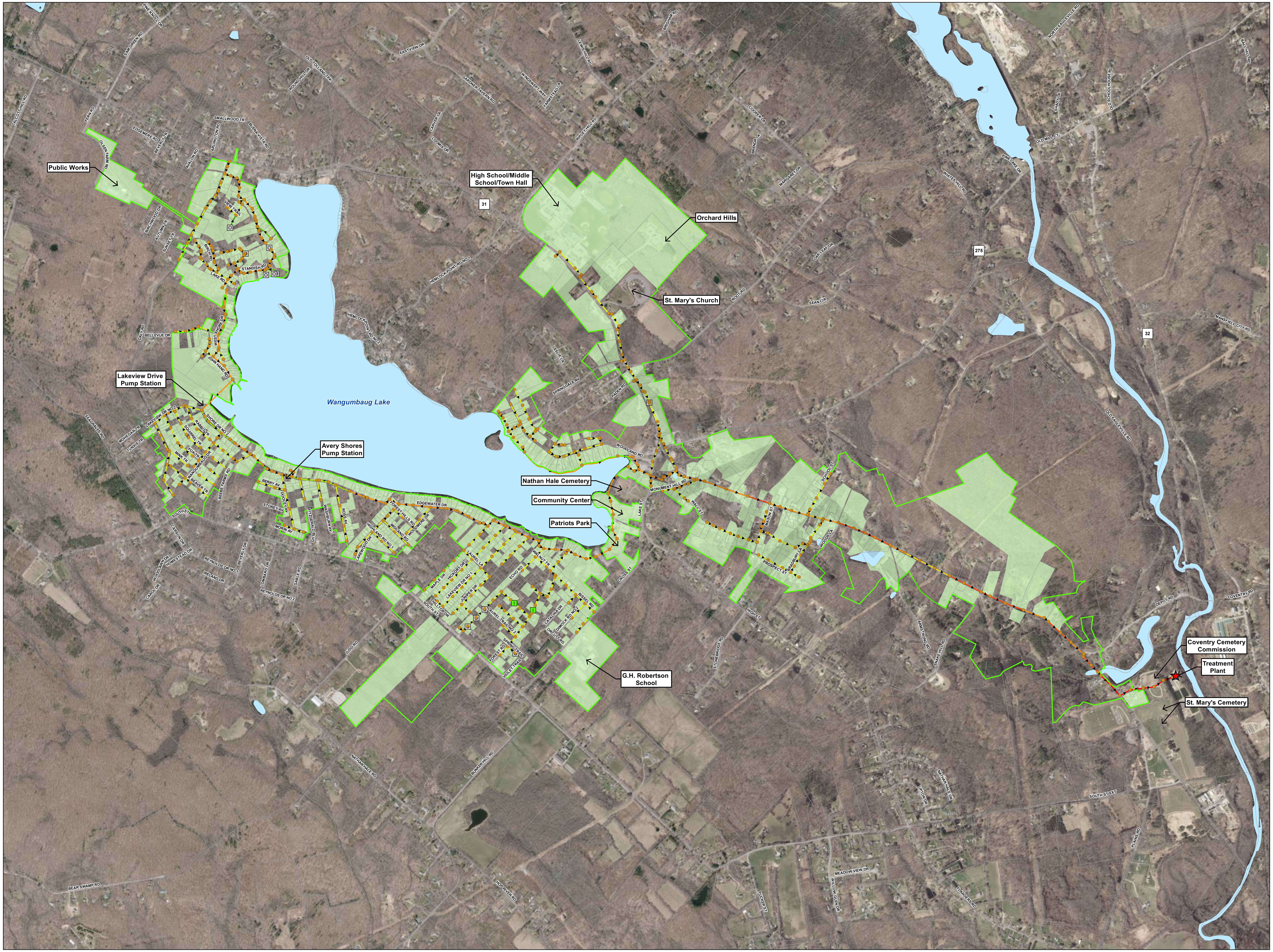

G:\GIS\CT\CoventryCT\MXD\SewerExpansion\CoventrySanitarySewer_24x36_ExistingArea.mxd

FIGURE 5-2
TOWN OF COVENTRY
WASTEWATER
COLLECTION AND
TREATMENT FACILITIES

LEGEND

- Sewer Manhole
- Other
- Impervious Collar
- Treatment Plant

Main Diameter (Inches)

- 1.25
- 2
- 6
- 8
- 10
- 12
- 15
- 18

- Existing Sewer Service Area Boundary
- Sewered

LOCUS MAP

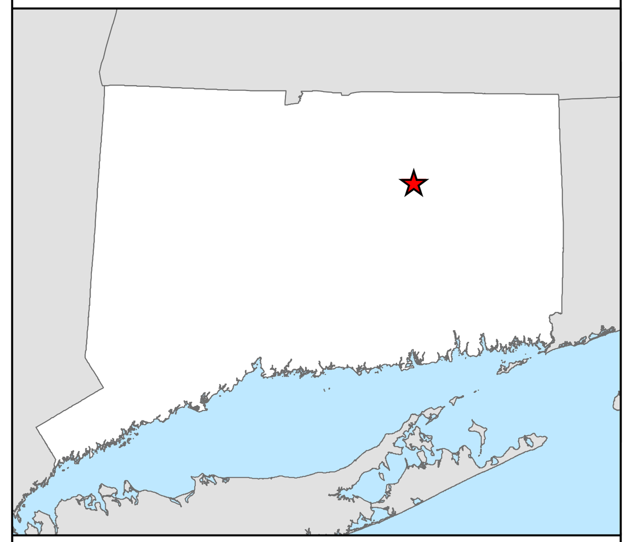

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FIGURE 1

Existing Sanitary Sewer Service Area

December 2023

Tighe&Bond

2.2 Pump Stations

Coventry also has two Town owned pump stations, as shown in Figure 2-1 and summarized in Table 2-2. The Lakeview Drive station pumps flow to a gravity sewer that flows to the Avery Shores Station. Flow from the Avery Shores Station is pumped east to a gravity sewer on Edgewater Drive. From this point, flow is entirely gravity to the Coventry Treatment plant.

TABLE 2-2
Pump Station Summary

Pumping Station	Year Put into Service	Capacity[1] (gpm)	Description	Force Main Length/Material
Lakeview Drive	2006	330	Submersible	860 LF Ductile Iron Pipe
Avery Shores	2006	386	Submersible	1,200 LF Ductile Iron Pipe

[1]One Pump Running

Each pump station is a submersible station that contains the following major components:

1. Below-grade concrete wet well where the pumps are located.
2. Below-grade concrete valve vault containing the discharge piping, isolation valves, and flow meter.
3. Above-grade precast concrete building with vinyl siding that houses the electrical components and controls, as well as an emergency generator.

2.3 Water Pollution Control Facility

The existing Water Pollution Control Facility (WPCF) is a primary treatment plant with a permitted capacity of 200,000 gallons per day (gpd). The WPCF treats flow from the Coventry collection system, which is primarily residential in nature.

The WPCF was originally constructed in 1985, and is located in the southeast corner of Town, bordered by the Willimantic River to the east and south, Route 31 to the west, and residential parcels to the north.

The major treatment processes at the WPCF include preliminary treatment (screening and grit removal), influent pumping (after screening), and primary treatment. Primary effluent is discharged to rapid infiltration basins (RIBs). Sludge from the treatment process is collected and disposed of as a liquid offsite. Treated effluent is discharged to the groundwater from the bottom of the RIBs.

A more detailed description of the existing treatment plant processes is presented in Section 5.

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Section 3
Sewer Service Area, Flows and Loads

3.1 Introduction

The first task to be completed as part of Wastewater Management Planning is determining existing and future wastewater flows and associated loadings.

The evaluation discussed in this Section was completed by Tighe & Bond, and the results summarized in Technical Memorandum No. 1, which was submitted to the Town of Coventry and the CT DEEP for review and approval in May 2023. CT DEEP comments were received in June 2023, with all comments being addressed as part of this report.

3.2 Existing Wastewater Flows and Loads

3.2.1 Treatment Plant Design Criteria

Based on the WPCF's existing groundwater discharge permit and documentation included in the Hydrogeological Study prepared by Fuss & O'Neill, Inc. (F&O) dated June 3, 1985, the Coventry WPCF was designed for the average flows and loads indicated in Table 3-1. Table 3-1 also indicates the population equivalent of these loadings which are based on the flow and loading factors included in NEIWPCC's "Guides for the Design of Wastewater Treatment Works" (TR-16), the design standard used for designing WPCFs in Connecticut.

TABLE 3-1
WPCF Design Flows and Loads

Parameter	Design Value	Loading Factor (per capita)	Population Equivalent
BOD5 (lb/day)	450	0.17	2,647
TSS (lb/day)	533	0.20	2,667
TKN Nitrogen (lb/day)	83	0.04	2,075
Phosphorus (lb/day)	17	0.006	2,833
Flow (gal/day)	200,000	75	2,667
Peak Hour Flow (gpm)	486	-	

Notes:
[1]_The permit requires facility planning when flows or loads reach 90% of permit. It is not clear if this is over one year or one month. There is no mention of design loads, only average day flow. Connecticut surface water discharge (NPDES) Permits are based on a 180-day average.
[2]- F&O Appendix states RIB loading and assumes clarigesters will provide 30% BOD removal and 70% TSS removal. Plant influent loadings were back calculated from these values. Peak Flow was listed as 700,000 gpd which equals 486 gpm.
[3]- The 75 gal/day/capita factor was assumed (in part based on data developed below based on current water usage) and does not include inflow and infiltration, which presumably should have been included. Alternately, Fuss & O'Neill assumed a lower factor than 75 gal/day and added inflow and infiltration on top of that calculation. TR-16 recommends that usage be considered in the range of 70 to 100 gal/day/capita.

Based on Table 3-1, the estimated full time equivalent population planned to be served by the WPCF equates to approximately 2,650 people. The assumed population divided by the 1,182 EDUs currently connected to the sewer system suggests an occupancy rate of 2.24 people per EDU. This suggests that, in the mid-1980s, Fuss & O'Neill may have assumed an occupancy which was lower than Coventry's current occupancy rate of 2.65 (12,458 people / 4,710 households) as listed in the Town's July 2020 Plan of Conservation and Development

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3.2.2 Historical Flows and Loads

Treatment plant monthly operating reports for a 4 year period from 2019-2022 were reviewed, and a summary of the best estimate of existing wastewater flows and loads was developed. This summary is presented in Table 3-2. Wastewater flows are comprised of sanitary flows and loads from sewer users, and flow from infiltration and inflow (I/I). I/I is derived from rainwater and groundwater entering the sewer system, and I/I flows are typically higher during high groundwater and wet weather periods in the spring and fall. Loads are measured in the middle of the week, so the data in Table 3-2 includes adjustments for increased weekend loads that are averaged out over the week.

TABLE 3-2
Summary of Existing WPCF Flows and Loads – Annual Average

Item	BOD5 (lb/day)	TSS (lb/day)	TKN Nitrogen (lb/day)	Phosphorus (lb/day)	Flow (Gal/Day)	Peak Hour Flow (GPM)
Existing I/I Estimate					50,000[1]	850[2]
Average Weekday	190	220	49.2	6.4	93,000[1]	
Average Weekend Add	3.8	4.4	1.0	0.1	Included	
Overall Average	**194**	**224**	**50**	**6.5**	**143,000**	**850**[2]
Percent of Design	*43%*	*42%*	*61%*	*38%*	*72%*	*175%*
Estimated Population	*1,140*	*1,122*	*1,255*	*1,088*	*1,240*	

Notes:
1- See Section 3.2.3 for the breakdown of the "Base Sewer Flows" and the estimate of I/I
2- See Section 3.6 for the review of the estimated peak hour flows

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As shown in Table 3-2, the existing BOD loadings to the WPCF suggest that the full time year round equivalent population served is around 1,150 people. This also suggests that the average year round occupancy rate of the 1,182 EDUs currently served is ~ 1.0 people per EDU, which is much lower than that of the Town as a whole and what was assumed in the original design.

Due to the small area served by the WPCF and the fact that the sewer service is a lake front community, a review was conducted of the seasonal variation in average flows/load, maximum month variation in flows/loads (calculated based on a 30-day rolling average), and weekend versus weekday variations in flow. Additional data is presented in Table 3-3, Table 3-4, Figure 2-1, Figure 3-1, and Figure 3-2, from which the following was concluded:

- Flows and loads are lower in the summer time. Lower flows are expected due to the reduction/absence of I/I. The lower load values suggests that the three schools (and possibly University of Connectcut student rentals during the academic year) may be a larger factor in the plant loads than summer occupancy of seasonal homes.
- Existing maxmimum month loads (indicated in Table 3-3 and shown in Figure 2) as a rolling average are approximately twice the annual average loads. This factor will be used when estimating future maximum monthly loads. The means that there are periods of time when the average occupancy of the EDUs served doubles from ~1.0 to ~2.0 persons per dwelling, which is closer to the average overall occupancy of the Town itself.

Figure 3-1: WPCF Temperature Readings

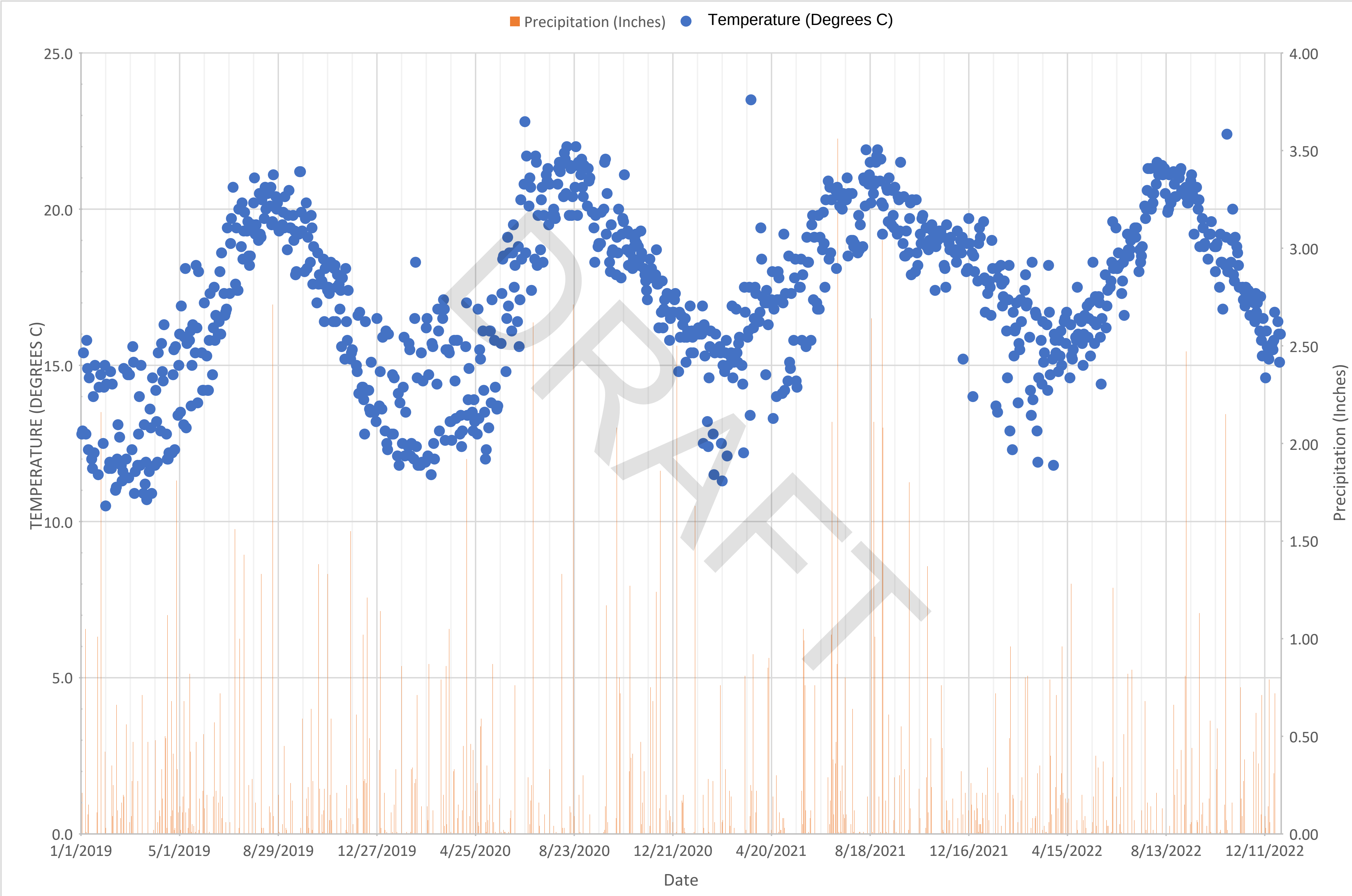
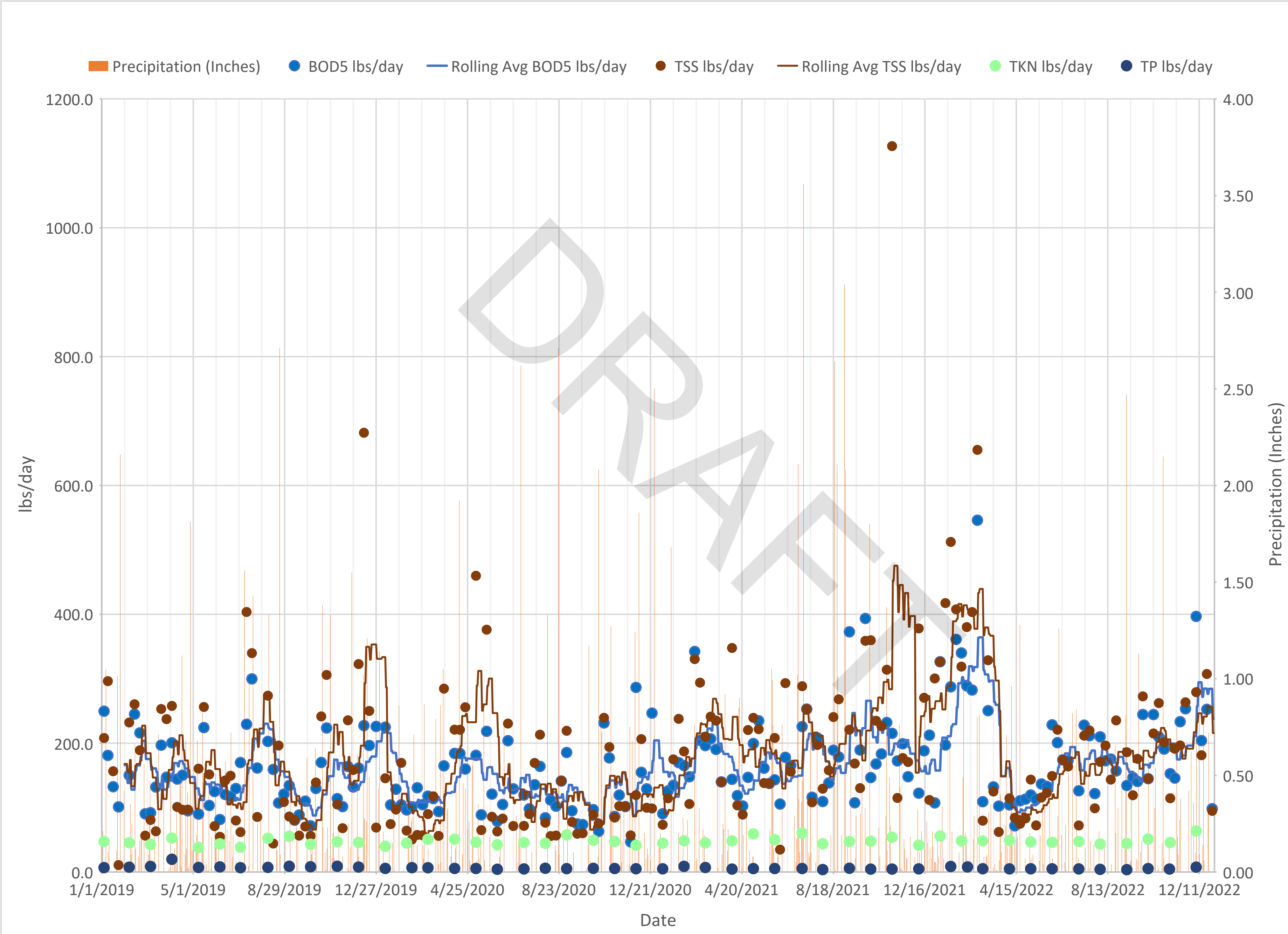


Figure 3-2: WPCF Historical Influent Loading (lbs/day)



- Weekend flows were ~5% higher than weekday flows. Assuming this increase is comprised of sanitary flows and not I/I, it is estimated that sanitary flows and loads were 7% higher on weekends. Thus, on average over the year, WPCF loads would be 2% higher than the estimates of loads suggested based on WPCF sampling data, which is only conducted on Wednesdays. Weekend versus weekday flows are discussed further in the next section.
- Wastewater temperatures entering the plant typically vary from 11°C to 22°C.
- There was a significant change in the loads between 2019-2020 and 2021-2022. Due to the potential impact of Covid on working and living patterns during the study period, we took a conservative approach and used the higher loadings from these two periods for the basis of design. It is noted that these higher loadings were consistent with the data set going back to 2012 provided by operators in their percent removal spreadsheets.
- Base Infiltration on average over the year appears to be around 35 gallons per minute (gpm), or 50,000 gallons per day.

Peak flows are estimated (and significant) and are further discussed in Section 3.6.

TABLE 3-3
Historical WPCF Flows and Loads

Parameter	4 Year Data Range (2019-2022)	2 Year Data Range (2019-2022)	Use as Basis of Design for Existing Flows and Loads
Daily Flow (GPD – Taken Daily)			
Annual Average	**142,939**	141,192	**143,000**
Maximum Month	**306,550**	306,550	**307,000**
Maximum	**518,668**	518,668	**520,000**
BOD5 (lbs/day – Taken weekly)			
Annual Average	165	**188**	**190**
Maximum Month	364	**364**	**370**
Maximum	546	**546**	**550**
TSS (lbs/day – Taken Monthly)			
Annual Average	184	**218**	**220**
Maximum Month	475	**475**	**480**
Maximum	1127	**1127**	**1130**
TKN-N (lbs/day – Taken Monthly)			
Annual Average	47.8	**49.2**	**50.0**
Maximum	63.8	**63.8**	**64.0**
TP-P (lbs/day – Taken Monthly)			
Annual Average	**6.3**	5.5	**6.4**
Maximum	**19.8**	8.8	**20.0**

TABLE 3-4
Historical WPCF Flows and Loads – Seasonal Variation 2019-2022

	Average Daily Flow (Gal/day) (Taken Daily)	Average BOD5 (lbs/day) (Taken Weekly)	Average TSS (lbs/day) (Taken Weekly)	TKN-N (lbs/day) (Taken Monthly)	TP-P (lbs/day) (Taken Monthly)
Nov- Feb	152,065	187	220	47.6	6.6
July & August	122,667	161	165	47.5	5.3
Difference	29,398	26	55	0.1	1.3
Per Capita change	420	152	274	3	213

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3.2.3 Base Sanitary Flows

A key part of this project involves estimating existing and future sanitary flows as well as I/I into the Coventry WPCF. A variety of data was used as part of this task, including:

- GIS parcel data provided by the Capital Region Council of Governments (CRCOG), last updated in 2021.
- Sewer infrastructure (pipes, service lines, and manholes) provided by the Town of Coventry.
- GIS zoning information provided by the Town of Coventry, last updated 2013.
- Tabular data provided by the Coventry Water Pollution Control Department included:
    - Water consumption data from the Connecticut Water Company for sewered customers who also have water service– 2021 billing containing 4 quarters worth of water use.
    - Town of Coventry Sewer Customer lists. Detailed information was provided as follows:
        - Parcels connected to the sewer system.
        - Parcels within the sewer service area that are not connected.
        - Empty lots, including buildable, non-buildable and uninhabitable parcels.

Sewered parcels include customers served by Connecticut Water and sewer customers on well water. The Town of Coventry bills customers for sewer service at a flat annual rate per Equivalent Dwelling Unit (EDU). A typical single family home represents 1 EDU. An apartment building containing 6 apartments would represent 6 EDUs. A listing of EDUs per parcel was also provided by the Town.

Tighe & Bond used the GIS information and customer lists to identify parcels connected to the sewerage system, the total number of sewer customers, and those parcels within the sanitary sewer service area that are not connected to the sewer. Water usage data was used as the basis for estimating existing flows on a basis of gallons per customer per day. The specific methodology used is as follows:

- The list of sewer customers was joined to parcels in GIS, using the property address and tax map/lot number values as a basis for determining a match.
- This analysis concluded that there are a total of 898 parcels connected to the Coventry sewer system. The 898 parcels equate to 1,182 EDUs currently being billed for sewer service.
- A large majority of the 982 parcels connected to the sewer system are residential users. There are 17 commercial customers primarily located on Main Street (Route 31). The Town currently has one industrial user (Teleflex-CT Devices, Inc.) located at 2 locations (1275 and 1295 Main Street).
- The water consumption data provided by Connecticut Water was separated into residential, commercial, and industrial customer classes. In some cases, a full four quarters worth of water usage was not listed, likely due to new customers. Where applicable, a full four quarters worth of water usage was approximated using the available data. The water consumption data was summarized and averaged to establish an average daily water usage for each class (residential, commercial, and industrial customers).
- A summary of average water use for residential, commercial and industrial customers is summarized in Table 3-5.

TABLE 3-5
2021 Annual Average Water Use for Sewered Connecticut Water Customers

	Average Customer Water Use (gallons/day/customer)
Residential	96
Commercial	310
Industrial	584

Typically, 100% of the residential water use does not enter into the sanitary sewer system, especially during the summer months when water is used for lawn watering and irrigation. On average, irrigation accounts for approximately 10-15% of the measured annual water use. In New England irrigation is seasonal, typically occurring between May and September. Therefore, residential wastewater flows were estimated as 90% of the residential water usage, and wastewater flows for both commercial and industrial customers were estimated at 100%.

It is noted that the expected water usage is on the order of 75 gallons per day per capita. The residential usage rate, once adjusted for the number of EDUs suggests an average occupancy of approximately 1.1 (86/75) person per EDU. The average wastewater flows for each class of sewer customer were then applied to all customers (including those on wells) to estimate the total domestic flows. For residential customers, flows were assigned based upon the number of EDUs per property, which is more reflective of expected sewer flows. Table 3-6 summarizes the results of these calculations.

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TABLE 3-6
Estimated Existing Domestic and Commercial/Industrial Wastewater Flows

	Number of Sewer Users	Number of EDUs	Wastewater Flow per EDU/Customer (gallons/day)	Estimated Base Wastewater Flow (gallons/day)
Residential	879	1,142	86	98,300
Commercial	17	36	310	5,300
Industrial	2	4	584	1,200
Total Base Flows	**982**			**104,800**

Sanitary flows represent base wastewater, without infiltration and inflow (I/I) taken into account. Theoretically, the base wastewater values calculated in Table 3-6 should approximate the average flows of 93,000 gallons per day presented in Table 3-2.

As stated earlier, some of the properties connected to the sewer system may be seasonally used, in some may be weekends only or increased usage on weekends. Therefore, an evaluation was made of treatment plant flow observed on weekdays (Monday – Friday) versus weekend days (Saturday and Sunday).

A review was also conducted of flow noted during the summer months (July and August) of 2020 and 2022. These years were chosen because no significant rain events took place that would cause an increase of flows associated with I/I.

Treatment plant flow data information for weekends/summer months compared to year round values is summarized in Table 3-7.

TABLE 3-7
Comparison of Treatment Plant Flow Data

	Overall Flows (gpd)	Weekday Flows (gpd) (Mon-Fri)	Weekend Flows (gpd) (Sat/Sun)
Average Daily Flow: 2019-2022	142,920	141,595	146,242
Minimum Daily Flow: 2019-2022		80,647	90,833
Summer 2020 (dry summer)		99,066	104,563
Summer 2022 (dry summer)		96,133	99,840

An upgraded treatment plant or pump station should be capable of handling year round flows from the existing and future sewered area. Table 3-7 clearly shows higher flows during the weekend. In addition, the summer weekend flows are generally consistent with the base wastewater flows calculated in Table 3-6.

Therefore, the unit flows in Table 3-6 will be considered valid for the purpose of estimating future sewer flows from similar properties.

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3.3 Proposed Future Sewer Service Area

Input on additional connections within the existing Sewer Service Area as well as areas that should be considered for the future sewer service area were discussed at a meeting held on March 13, 2023, with representatives from the Eastern Highland Health District, Coventry WPCA, and the Coventry Land Use Director and Zoning Enforcement Officer. Based on the results of this discussion, the Proposed Future Sewer Service Area Map was developed as presented in Figure 3-3. Future sewer expansion will fall into 3 main categories, each of which is discussed below.

3.3.1 Infilling within Existing Sewer Service Area

The existing sewer service area currently includes parcels surrounding Wangumbaug Lake. New lots around the lake are currently not being developed in part due to the presence of wetlands, steep grades and/or poor soils that prohibit the installation of septic systems and thus prohibit development from taking place due to zoning requirements that require the lot be suitable for septic even if sewer is available. To prevent overdevelopment of small lots surrounding the lake, it was decided that existing lots that have poor soils, wetlands and/or steep slopes will not be considered for future sewer service unless there is already a structure constructed on the property. All other lots will be assumed to be able to connect to the sewer system in the future and will be included in future sewer flow estimates.

3.3.2 Areas of Need

Based upon input from the Eastern Highland Health District, the following areas outside of the existing Sewer Service Area were recommended to be included in the updated Sewer Service Area due to the difficulty in maintaining a health compliant septic system. Specific areas include:

- The small lots adjacent to the lake on Hemlock Lane, Hemlock Point Drive, Knoll Drive, and Cheney Lane that are adjacent to the lake.
- Small lots on the northeast side of the lake including Birchwood Drive, Autumn Trail, Sunset Trail, Buena Vista Road, Beverly Train and Edgemere Road, including the small lots adjacent to the lake south of Main Street.
- Small lots on Lamotte Road, Paden Road and Hinman Road.
- Streets on the south side of the lake and existing sewered area should also be considered for future sewer service, including Reynolds Drive, Woodlawn Drive, Lombard Drive, and Carol Drive.

It is noted that properties within these areas will be subject to the same criteria mentioned earlier: lots that have poor soils, wetlands and/or steep slopes will not be considered for future sewer service unless there is already a structure constructed on the property.

3.3.3 Areas of Future Development

Consideration must also be given to including parcels within the revised Sewer Service Area where development is expected to take place. Specifically:

- For consistency, the future sewered area should include all lots in Special Planning Areas 6, 8, 9, and 11 as shown in Map 12 of Coventry’s Plan of Conservation and Development. Areas 6, 8 and 9 are in an Industrial Heritage Overlay District. Area 11 is in an RD (Rural Development) zone, so an industrial use is feasible. Therefore,

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lots in Area 11 that are not Town owned (total of 6 lots) were assigned industrial use flows.

- A 28 acre parcel owned by St. Mary's church is currently a cornfield, however, there is the potential that this may developed at some point in the future. The development extent is not known at this time. It was agreed that this parcel would be assumed to be developed into the number of parcels allowed by current zoning (GR40). This represents a flow equal to 28 residential parcels.

As discussed in the next section, anticipated flows from these future development areas (roughly 6,000 gpd) are minor.

3.4 Future Flows

Future flows were based upon the following assumptions:

- All parcels within the boundaries defined on the updated Sewer Service Area Map that could feasibly be developed that are currently not connected to the sewer system will connect at some point over the 20-year planning period. Lots that have poor soils, wetlands and/or steep slopes will not be considered for future sewer service unless there is already a structure constructed on the property.

- Future average flows (and loads) for each customer class would be consistent with existing flows: residential (86 gpd), commercial (310 gpd), and industrial (584 gpd). This assumption is generally consistent with assuming that the expanded sewer area will have a low occupancy rate (86/75 = 1.1 person per EDU) consistent with the existing sewered area and much less that the remaining Town as a whole (2.65 persons per EDU). This appears to be a reasonable assumption as long as the service area remains close to the lake.

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A total of 747 parcels were determined to be eligible to connect to the sewerage system. Cemeteries were not considered feasible. The parcels were broken down into residential/commercial/industrial customer based upon zoning and listed use of each lot as shown in the GIS data. Parcels with missing information were assumed to be residential customers.

A large majority of parcels are single family residential lots. Where GIS data indicated a parcel contained a multi-family residence, the flow value was multiplied accordingly. Thus, flows listed represent EDU flows.

A breakdown of parcels by class and estimated future flows is presented in Table 3-8.

TABLE 3-8
Summary of Unsewered Parcels and Future Wastewater Flows

Zoning Category	Unsewered Parcels/EDUs	Wastewater Flow per User (gallons/day)	Estimated Wastewater Flow (gallons/day)
Residential	693	86	59,600
Commercial	48	310	14,900
Industrial	6	584	3,600
Total	**747**		**78,100**

In addition to the above, the following flows will be added to specific parcels as listed below (increasing the total EDU count by 284 EDUs to a total of 1032 EDUs):

- An additional flow of 2,400 gpd was added to account for the 28 EDUs that could be constructed adjacent to St. Mary's church.
- An allowance for future flows should be added in the event that Accessory Dwelling Units (ADU) are allowed to be constructed. The minimum lot size needed to allow an ADU to be added is 25,000 square feet (0.57 acres). After discussion with the Town, it was determined that all lots greater than 0.57 acres would have the potential to add an ADU and that one fifth of these would construct ADU at some point in the future at a flow rate of 50 gpd per unit. A total of 204 parcels are greater than 0.57 acres. Assuming 1/5 of these parcels construct an ADU at 50 gpd/ADU equates to a future flow of 2,100 gpd (rounded) or 24 EDUs.
- An allowance of 10,000 gpd of flow would be added in case an increased density housing development is proposed at any location. This is equivalent to 117 EDUs.
- An allowance of 10,000 gpd of flow will be added due to the planned construction of a water tower in the downtown area, which increases the potential for additional commercial customers to be added in the Sewer Service Area. This is equivalent to 116 EDUs.
- An allowance of 5,200 gpd of flow will be added for a proposed 60 unit apartment building (60 EDUs) at 112 Woodlawn Road.
- An allowance of 15,000 gpd was also added for future I/I, as required by TR-16. This value is based upon an estimate of 7 miles of new 8-inch sewers being constructed and an infiltration rate of 250 gallons per day per inch mile of new sewer line.

As indicated in Table 3-9, the projected future average daily flow will increase by 107,800 gpd of sanitary flow plus 15,000 gpd of I/I for a new total of 265,800 gpd. This will be rounded to 266,000 going forward.

TABLE 3-9
20-Year Projected Wastewater Flows – Annual Average

Existing Flows (Sanitary and I/I)	143,000
New Residential	59,600
New Commercial	14,900
New Industrial	3,600
St. Mary's Church	2,400
Accessory Dwelling Units	2,100
Increased Density Development	10,000
Water Tower Flows	10,000
Woodlawn Road Apartments	5,200
Total New Sanitary Flows	107,800
Total Current and Future Sanitary Flows	250,800
Allowance for Future Infiltration	15,000
Total 20-Year Projected Flow	***265,800***

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3.5 Future Loads

Based on the future service area and the above future flows analysis, it is expected that flows and loads will increase by 1,092 EDUs. To be conservative, it was estimated that loads would increase by the equivalent of 1,201 people based on an occupancy of 1.1 persons per EDU which is 10% higher than the existing sewer service area.

Using the flows developed above, the TR-16 loading factors, and the above factor for increases in occupancy, the future annual average flows and loads are anticipated to increase as indicated in Table 3-10.

TABLE 3-10
Future WPCF Flows and Loads

Item	BOD5 (lb/day)	TSS (lb/day)	TKN Nitrogen (lb/day)	Phosphorus (lb/day)	Flow (Gal/Day)	Peak Hour Flow (GPM)
Existing Plant Flows/Loads	190	220	50	6.5	143,000	850
Future Customer Flows/Loads	208	245	48	7.2	108,000	110
Future I/I Allowance					15,000	80
Future Avg Design Criteria	**398**	**465**	**98**	**13.7**	**266,000**	**1,040**
Change over Existing	*109%*	*111%*	*100%*	*115%*	*88%*	*22%*
Change over WPCF Design	*-12%*	*-13%*	*18%*	*-19%*	*33%*	*81%*

For future maximum month loads (of which BOD5 and TKN-N are the key criteria for designing a treatment system for nutrient removal), it was assumed that peaking factors (over the average loads) will be the similar to the existing peaking factors for parameters where the plant collects weekly data (BOD5 and TSS). Where monthly data was only available, and no averaging was possible to compute a 30 day rolling average (resulting in a less accurate estimate), we increased the TKN-N peaking factor to be consistent with the factor for BOD5 (for a conservative design), and for phosphorus, we reduced the peaking factor to split the difference between the existing and that for BOD. The existing and future maximum month peaking factors and loads as summarized in Table 3-11.

TABLE 3-11
Plant Influent Maximum Month Flows and Loads

Item	BOD5 (lb/day)	TSS (lb/day)	TKN Nitrogen (lb/day)	Phosphorus (lb/day)	Flow (Gal/Day)	Peak Hour Flow (GPM)
Existing Max Month	370	480	64	20	307,000	850
Existing Peaking Factor	*1.9*	*2.2*	*1.3*	*3.1*	*2.1*	
Future Peaking Factor	*1.9*	*2.2*	*1.9*	*2.5*	*2.1*	
Future Design Criteria – Max Month	**760**	**1020**	**192**	**34**	**559,000**	**1,040**
Change over Existing	105%	113%	192%	73%	82%	22%
Change over Original Design	69%	91%	125%	102%	180%	81%

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3.6 Peak Flows

Peak flows coming into the plant are mainly caused by I/I during rain events. Because the feasibility and/or cost of future treatment/conveyance alternatives will be impacted by the peak flow that must be treated or conveyed, the plant's hourly flow data was used to understand them, and also estimate the on-site volume of storage that would be required if the peak flow treated on site or conveyed away from the plant was to be limited.

Under both scenarios, the plant headworks and influent pump station would be sized to handle the anticipated influent peak flows. Existing tankage (such as the existing clarigesters, each with a volume of ~53,000 gallons) could be converted to a temporary storage tank to store flow that is in excess of the capacity of the receiving treatment or conveyance system. Under this section we are not considering or discussing the use of any tanks as a day-to-day equalization tank for balancing diurnal flows.

3.6.1 Existing Conditions

Hourly operating data from January 2019 through March 2023 was reviewed to characterize the peak flows pumped by the influent pump station at the treatment plant. This data is summarized in Figure 3-4.

Figure 3-4 is based on hourly average data. As shown, there were three events over the four plus year period where the influent flow exceeded (or came close to exceeding) the 600 gpm limit of the influent flow meter over a one hour period. In order to understand what happened during these periods, the SCADA data for the flows and wet well levels on a minute by minute basis (on 9/2/2021) was reviewed to estimate flows delivered by the influent pump station pumps. This also required discussions with the treatment plant operators to determine their recollection of wet well levels and the use of portable pump(s) as indicated in Figure 3-5. During these high flow events, operator's notes reflected that:

- On 7/9/2021, portable pump(s) were used over a 4 hour period. SCADA (one hour average) data showed no instances of the flow meter maxed out, however, there were 4 hours with flows around 525 gpm (from approximately 11 AM to 3 PM). It appears that the portable pumps prevented the flow meter from maxing out or water levels coming above the floor (for a prolonged period of time).

- On 7/10/2021, portable pump(s) were used over a 2 ½ hour period. SCADA (one hour average) data showed only one hour with the flow meter maxed out, but there were 2 adjacent hours with flows around 575 gpm (approximately 12 AM to 2 AM). On this date, a high water mark was recorded at 1 foot above headworks room floor.

- On 9/2/2021, portable pump(s) were used over a 4 hour period. SCADA (one hour average) data showed 4 hours with the flow meter maxed out, and 3.5 hours with a level above 10 feet on this day. Operators recall that water levels likely were at or slightly above floor level.

Based upon this information, it appears reasonable to conclude the following:

- The peak flow during the two later events was 150 gpm higher than the 600 gpm recorded. This is based on the projected flow rate from the pumps at the higher wet well level.

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Figure 3-4 WPCF Historical Hourly Flows

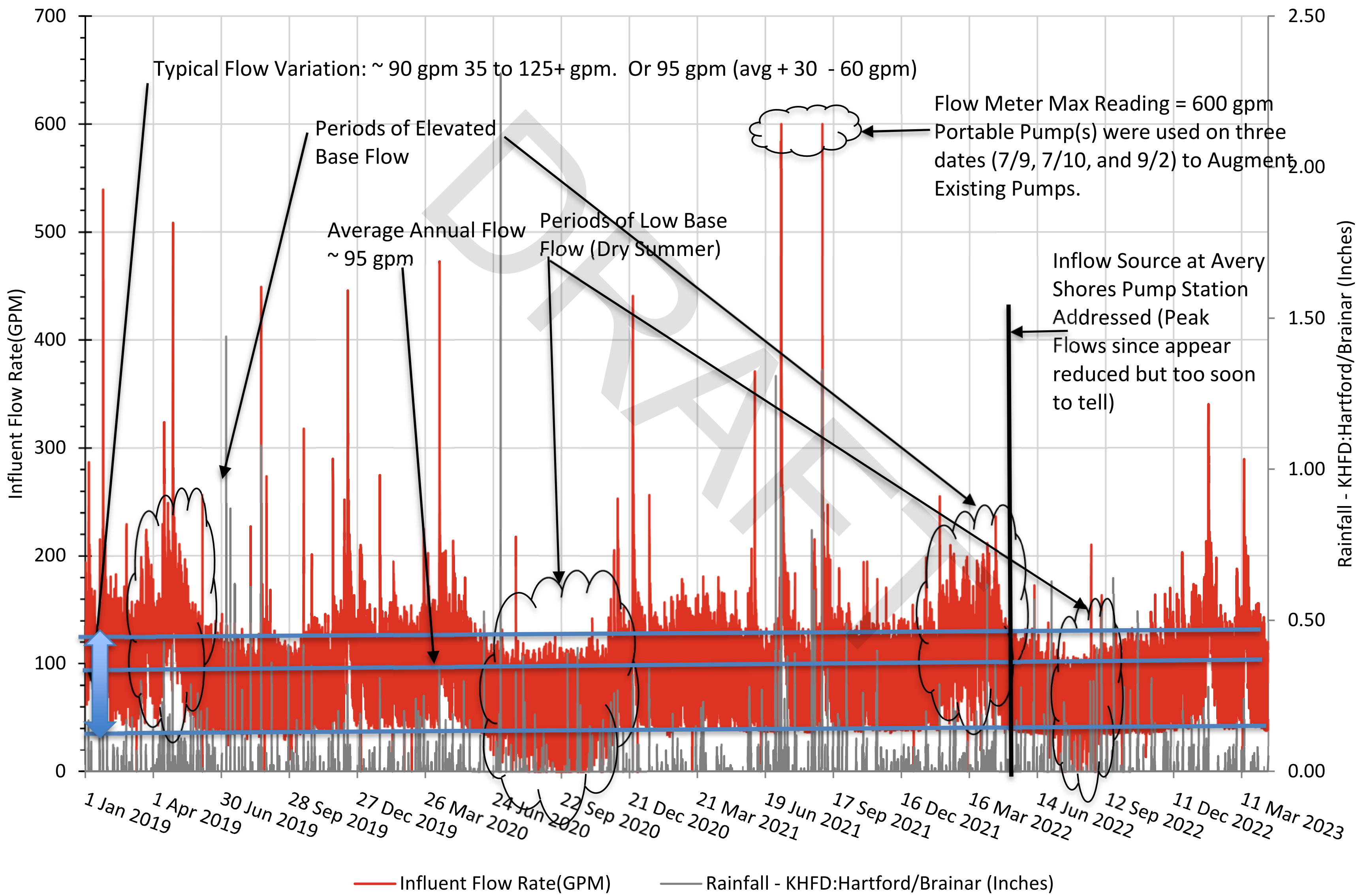
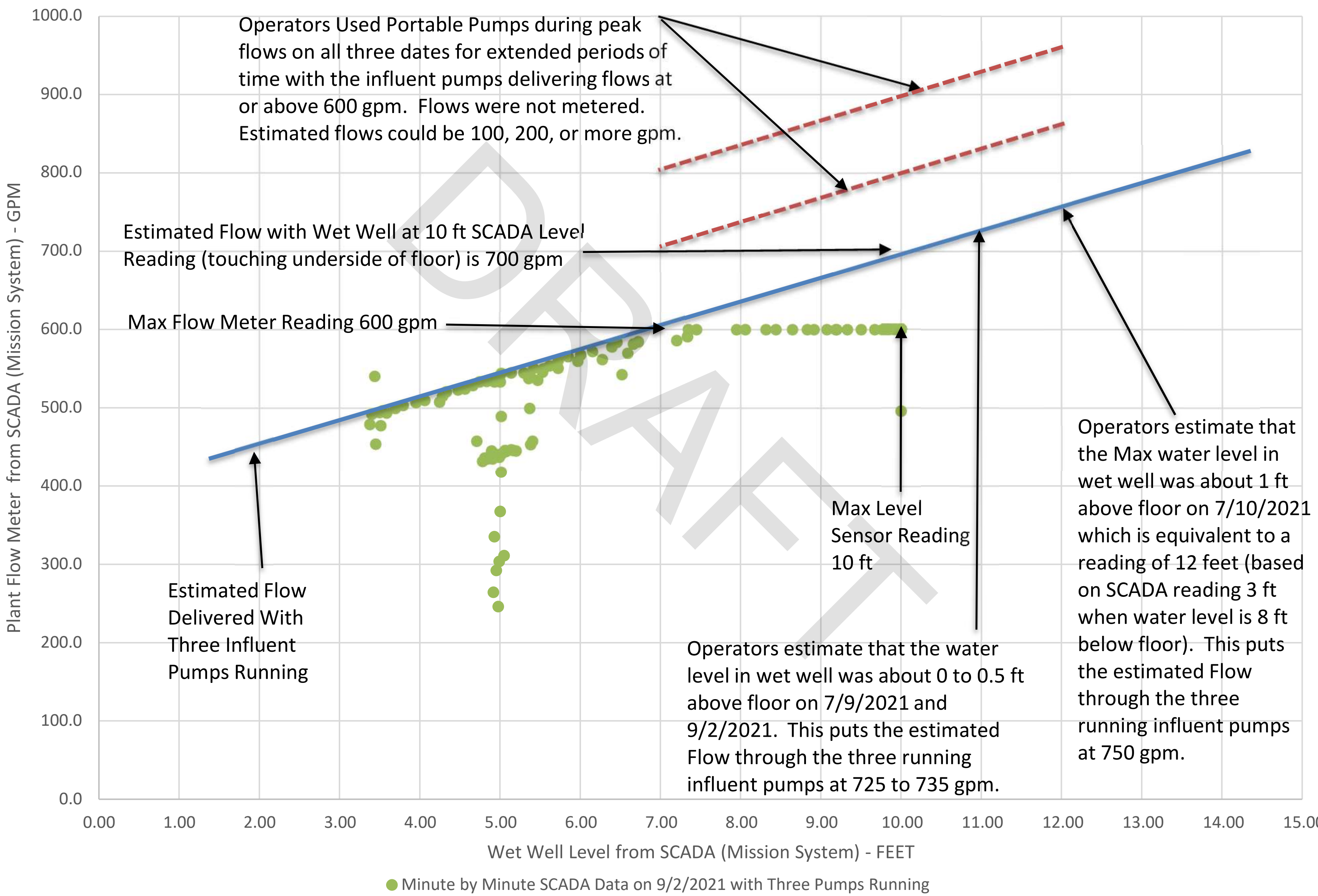


Figure 3-5 Peak Flow Estimate for High Flows Summer of 2021



Total added flow during the 9/2/21 flow event was 22,000 gallons, based upon ~4 hours at an average additional flow of 75% of the added 125 gpm peak flow.

The plant's peak influent flow during these events would also include the flow delivered by the portable pump(s), which was not metered and roughly estimated by the operators to be in the range of 300 to 400 gpm based on the pump's maximum capacity. Before this flow can be added to the flows above, the following factors must be considered:

- The required pumping head during the event will reduce the pump's capacity to below the maximum capacity, possibly significantly.
- The use of the portable pumps would likely reduce the wet well level slightly, which in turn reduces the flow from the influent pumps from an estimated peak of 725 gpm to 700 gpm.
- On all three dates, WPCF's throughout the state reported raw sewage overflows/bypasses in their collection systems and at their plants (https://portal.ct.gov/DEEP/Municipal-Wastewater/CT-Sewage-Right-to-Know) due to unusually heavy rains. This suggests that the weather exceeded typical "design" expectations.
- In Spring of 2022, a significant inflow source was removed from the collection system.

Based on the above, for the purpose of determining an existing "design" peak hour flow for this study it was decided to conservatively add an additional 100 gpm of peak flow (for a total flow of 600+150+100 = 850 gpm) to reduce surcharging of water in the wet well and bypassing of influent screening that will occur during such an event.

For the purpose of the peak flow analysis below, it was assumed that the added flow from the portable pumps was 12,000 gallons (100 gpm for 2 hours) for a total volume of 22,000 + 12,000 = 34,000 gallons that not recorded by the influent flow meter during these events.

With the above corrections, the plant's data was analyzed to estimate how often and what volume of wastewater would need to be stored if the flows to be treated locally or remotely were limited to specific peak flows. The results are summarized in Table 3-12.

TABLE 3-12
Existing Peak Flow Analysis (4.25 year period)

Peak Flow Capacity Limitation (gpm)	Period of Time Flow Exceeded Peak Capacity (Hrs)	Longest Consecutive Hours Flow Exceeded Peak Capacity (Hrs)	Storage Volume Required for the Longest Time Event[1] (Gals)
599	3	2	34,000
550	7	4	46,000
500	14	5	59,000
450	16	6	78,000
400	32	6	102,000

[1] To store influent flow in excess of the Peak Flow Capacity Limitation. Flow and Times were calculated from real time hourly data collected Mission system and corrected as discussed above.

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The data suggests that existing flows could be stored using the volume of the two clarigesters, as long as the peak flow to the treatment system is not limited to less than 400 gpm. The water stored in the clarigesters could then be bled back into the influent wet well (pumping would be required to use the full capacity of the tanks) once the peak flow subsides. If the three high flow events with flows exceeding 600 gpm (when many plants in CT bypassed) are disregarded, then storage volumes would decrease by 34,000 gallons.

3.6.2 Future Conditions

Based on the increase in the sewer service area, the peak flows to the plant are anticipated to increase by 190 gpm based on the following assumptions:

- Increase in Infiltration (15,000 gpd): this equates to a flow of 10 gpm.
- Increase in Peak Sanitary Flow from new customers: the existing average flow variation is 90 gpm above existing infiltration flows based upon existing average sewage flows of 93,000 gpd as listed in Table 3-2. New customer wastewater flows are therefore expected to add an additional 108,000 gpd, which equates to an additional peak flow of ~110 gpm (90 gpm existing peak flows x 108,000 gpd future wastewater flow / 93,000 existing wastewater flow).
- Increase in Inflow due to new sewers: Assumed at 70 gpm (an increase of 10% over the existing estimated peak inflow of 700 gpm). Although this may be considered a small increase, it also assumes some credit for inflow reduction including flows already removed at the Avery Shores Pump Station.
- Total increase in peak flow = 10 gpm + 110 gpm + 70 gpm = 190 gpm.
- Total existing plus future inflow = 850 gpm + 190 gpm = 1,040 gpm.

With the above new estimate in peak flow, the plant's data was analyzed to estimate how often and what volume of future wastewater would need to be stored if the flows to the treatment system or conveyance system was limited to certain peak flows and the results are provided in Table 3-13.

TABLE 3-13
Projected Future Peak Flow Analysis (over same 4.25 year period)

Treatment/Conveyance System Peak Flow Capacity (gpm)	Period of Time Flow Exceeded Peak Capacity (Hrs)	Longest Time Flow Exceeded Peak Capacity (Hrs)	Storage Volume Required for the Longest Time Event[1] (Gals)
599	31	6	97,000
550	52	12	125,000
500	72	14	163,000
450	133	32	223,000
400	312	39	295,000

[1] – To store influent flow in excess of the Treatment/Conveyance System Peak Capacity

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This analysis suggests that the two existing clarigesters, with a combined storage volume of 106,000 gallons, could be relied upon to store only flow in excess of ~ 575 gpm assuming they were not used for any other purpose and a sewer system bypass was to be avoided for during the most extreme storm events during the study period. If the three high flow events with flows exceeding 600 gpm (when many plants in CT bypassed) are disregarded, then storage volumes would decrease by 34,000 gallons and the clarigesters could be used to store flows in excess of ~ 530 gpm.

3.6.3 Peak Flow Summary

The above analysis concluded that for the purpose of a conservative design, the existing peak flows are 850 gpm and peak flows will increase to 1,040 gpm. This is 25% higher than 830 gpm which is recommended by TR-16 based on the Max Day Peak being 4.5 times the future average daily flow of 184 gpm/266,000 gpd.

Major storms in 2021 (that overwhelmed a majority of the States sewage collection systems) contributed over 250 gpm, and storm flow volumes in excess of the plant's metering capacity were estimated at 44,000 gallons or more. It could be argued that a less conservative design is appropriate and should be considered in the future when more supporting data is available.

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Section 4
Collection System and Pump Station Evaluations

The main focus of this Wastewater Management Plan is to determine the best method for future wastewater treatment and disposal for the Town of Coventry. However, as part of this project, a desktop evaluation was completed to quantify the amount of infiltration and inflow within the collection system. In addition, the two existing pump stations were evaluated to determine if they have adequate capacity to accept the proposed future flows. Each evaluation is discussed individually below.

4.1 Desktop Infiltration/Inflow Evaluation

A desktop evaluation of the amount of existing infiltration and inflow within the Coventry sewer system was completed by Martinez Couch & Associates (MCA) as a subconsultant to Tighe & Bond. Detailed results of this analysis are presented in Technical Memo No. 2, which is included in Appendix A of this report.

MCA used pump station and treatment plant flow information as well as historical rainfall data from the Town's SCADA system to evaluate flows over a three year period (2019-2022). Rainfall data was based upon published data from the Hartford Brainard Airport. A total of three subareas were evaluated:

- Sewers that flow to the Avery Shores Pump Station
- Sewers that flow to the Lakeview Drive Pump Station
- Sewers that flow directly to the Coventry WPCF.

Prior to the start of the evaluation, a kickoff meeting was held with the Town of Coventry to review existing conditions within the collection system and potential problem areas. One item of note was that in late 2022 the Town discovered and removed a significant inflow source from the Avery Shores Pump Station.

Infiltration is caused by groundwater entering the sewer system from leaking joints and manholes and is most prevalent during the spring months when groundwater levels are higher. Therefore, nighttime flows were reviewed during multiple periods of dry weather in both the spring and summer months to quantity the extent of infiltration within the collection system.

Inflow is excess flows that enter the system during a rain event from connections such as roof leaders, sump pumps, or catch basins that may be connected to the sewers. Therefore, five rain events were selected for evaluation, and a comparison of flows before and during each rain event was made to quantify the volume of inflow within the sewer system.

A summary of the findings from this evaluation are as follows:

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Infiltration

- A Seasonal variation of flow trends was observed in the collection system, which is indicative of an I/I response to increased rainfall and seasonal groundwater variations.
- Summer flows are lower than Spring flows for all flows evaluated.
- The collection system shows evidence of infiltration from the spring to the summer flows in the data analyzed. The peak infiltration is approximately 50,000 gpd. When compared to the size of the collection system, this amount of infiltration would not warrant additional investigations at this time.
- Approximately one-half the peak infiltration is noted to come from the Lakeview Drive Pump station sewer shed (24,000 gpd), and half coming from the gravity collection system flowing to the WPCF (22,400 gpd). Minimal infiltration (3,600 gpd) was observed coming from the Avery Shores pump station sewer-shed.

Inflow

- The use of rainfall data from the Hartford Brainard airport introduces variability in the repeatability of the analysis. Due to geographic separation, the total values of rainfall and hourly intensities may vary. This is seen in inconsistent volumes of inflow for rainfall events of similar magnitude and duration.
- For data after March of 2022, the inflow responses appear to have reduced significantly. This indicates that the modifications to the Avery Shores pump station eliminated a major source of inflow in the system.
- Even with the lack of reliable rainfall data, the data shows several significant rain events in 2021 with inflow volumes of 240,000 to 320,000 gallons with peak inflow rates of 24,000 to 32,000 gallons per day. For 2022, all rain events that were studied show inflow volumes of 15,000 to 100,000 gallons with peak inflow rates of 3,000 to 10,000 gallons per day.

Recommendations

- The amount of infiltration appears to be well below the levels that would justify additional investigations at this time. In Metcalf & Eddy's text "*Wastewater Engineering: Collection and Pumping of Wastewater*" the suggestion for system infiltration threshold rates which are not excessive is 1500 gallons per day per inch-mile (gpd/idm) of sewer. The estimated infiltration in the Coventry system is 297 gpd/idm.
- There is an inflow component to the flows. However, the responses appear to have decreased since the recent pump station modifications. The storm response and inflow volumes identified after March 2022 do not appear to be at levels that justify further investigations.
- If the flow responses continue to follow this trend, no additional study seems necessary. If an increase in storm response flows is observed in the future, a smoke testing program is a potential option for a low-cost investigation to identify inflow sources. Typically, smoke testing can be performed for under $0.50/l.f., at a rate of 10,000 to 12,000 linear feet of sewer mainline per day. The testing may identify cross-connected catch basins, connected roof leaders, broken cleanouts, failing manholes, and sink holes that can be cost-effectively repaired.

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4.2 Pump Station Capacity Evaluation

As presented in Section 3.5, Table 10, the average future flow to the Coventry WPCF is 266,000 GPD. These flows will come from sewer extensions, as well as additional connections to the sewer from lots that are already within the Sewer Service Area. As part of the Wastewater Management Planning, an evaluation of the existing pump station capacities was conducted to determine if the Avery Shores and Lakeview Drive Pump Stations have adequate capacity to accept these future flows.

4.2.1 Methodology

The number of sewered parcels connecting to each pump station under existing and future conditions was confirmed based upon GIS data as outlined in Section 3. Ground topography was used to determine parcels that could connect to each station under future conditions. This resulted in the following:

- Parcels on the northwest side of Wangumbaug Lake north and south of Route 31 would connect to the Lakeview Drive Pump Station.
- Parcels on the south side of Wangumbaug Lake to the west of Lakewood Drive as well as pumped flow from Lakeview Drive would connect to the Avery Shores Pump Station.
- Parcels on the south side of Wangumbaug Lane to the east of Lakewood Drive would connect to gravity sewers that flow directly to the Coventry WPCF.
- Parcels along Hemlock Point Road would require a pump station that would pump flow to the north, connecting to sewers that flow directly to the WPCF.

Existing and future average sewer flows were estimated by using the value of 86 gpd/parcel calculated as outlined in Section 3.4, Table 8. The resulting existing average flow value was compared to dry weather, summertime pump station flows obtained from SCADA system data to determine if the values were relatively consistent.

Future peak flows were estimated by applying a peaking factor as recommended in TR-16 with the exception that peak flow coming into the Avery Shore Pump Station from Lakeview was conservatively assumed to be the Lakeview Drive pump's capacity of 330 GPM.

Peak flows were then compared to the existing pump rate at each pump station. If the peak flow is less than the pump rate, then the station is determined to have adequate capacity.

4.2.2 Findings and Conclusions

Findings and conclusions of the pump station evaluation are as follows:

Lakeview Drive Pump Station

Existing Conditions: There are currently 230 parcels connected to the Lakeview Drive Pump Station. Using a flow of 86 gallons per day per parcel results in an average flow of just under 20,000 gallons per day (GPD). This is consistent with flow data from August of 2022.

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Future Conditions: An additional 278 parcels have the capability to connect to the sewer system under future conditions. 508 future parcels represents a future average daily flow of 43,700 gpd. After applying the peaking factor, this equates to a peak flow of 187 GPM.

Avery Shores Pump Station

Existing Conditions: There are currently 112 parcels connected to the Lakeview Drive Pump Station. Using a flow of 86 gallons per day per parcel results in an average flow of just under 10,000 gallons per day. Adding in the flow from the Lakeview Drive Pump Station results in a total average daily flow of approximately 30,000 GPD, which is slightly less than the flow data from August of 2022.

Future Conditions: An additional 201 parcels have the capability to connect to the sewer system under future conditions. 313 future parcels equates to a future average daily flow of 27,000 gpd, or a peak flow of 105 GPM. Adding in the pumped flow from Lakeview Drive (330 GPM) equates to a future peak flow of 405 GPM.

Conclusions

Future peak flows to the Lakeview Drive Pump Station were estimated at 187 GPM. This is less than 60% of the pump station's capacity of 330 GPM. Therefore, the Lakeview Drive Pump Station has adequate capacity under future flow conditions.

Future peak flows to the Avery Shores pump station were estimated at 405 GPM which is 5% higher than the current pump station capacity of 386 GPM and likely within the margin of error for this very conservative estimate. The estimate is conservative because of the following reasons:

1) The peak flow from Lakeview will be sustained only several minutes before the pumps cycle off for several more minutes (because Lakeview's peak flow incoming flow is only 60% of its capacity). This means the peak flow entering the Avery Shores pump station's wet well from Lakeview Station will be reduced and sustained for a short period of time relative to the capacity of the Avery Shores wet well's working volume.
2) It is not known when or if all lots within the Sewer Service Area will connect to the Coventry sewer system.

For this reason, it can be concluded that the Avery Shores pump station appears to have adequate capacity for existing and future flows. This evaluation should be performed again when the pumps are replaced.

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Section 5
Existing Treatment Plant Assessment and Capacity Evaluation

There are a number of secondary treatment technologies available for consideration to upgrade the Coventry WPCF. Prior to screening and selecting these secondary technologies, it is necessary to evaluate the existing processes at the plant including preliminary treatment, pumping, primary clarification, solids handling, plant water systems, and space available for the new treatment processes.

5.1 Preliminary Treatment Processes

Preliminary treatment processes at the plant include screening, pumping, influent flow metering, and grit removal. Each process is discussed individually below.

Screening

Influent screening is performed in the plant's influent channel in the area above the wet well of the influent pump station. Stop gates direct flow to either the channel grinder in a 24" wide channel or a manually cleaned bar rack (with 1-5/8" openings) in an 18" wide channel. It is a small space with limited ability to add new equipment. Access to the area is via spiral stairs.

Field observations are as follows:

- The headworks inlet channel has a depression for a rock trap. The basket that went into the rock trap prior to the comminutor is no longer used as it did not provide a benefit. Consideration should be given to filling this in.

- The original comminutor in the 24" wide channel was replaced by a JWC muffin monster channel grinder which they maintain by replacing the cutter box and gear every 6 years at a cost of around $11,000 each time.

- The Town is interested in, and in 2020 obtained quotes for, augmenting the grinder with one of the following two options:

  1. JWC – Auger Monster (less Grinder) at a unit cost of $73,169:
     - To be installed in the channel after the existing channel grinder. The manual bar rack would remain as backup.
     - ¼" (6 mm) perforated plate screen. Hydraulic capacity of 535 gpm (770,000 gpd) of peak flow with an unspecified upstream freeboard.
     - Discharge of screenings approximately 5 feet above lower-level floor so operators would need to hoist or carry the screenings up the stairs.

  2. JWC – Monster Chain and Rake at a unit cost of $157,602
     - Installed to replace existing manual bar rack. The channel grinder would remain.

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- ½" Bar screen spacing. Hydraulic capacity of 700 gpm (1,000,000 gpd) of peak flow with estimated upstream freeboard of 23" (0% blinding) to 20" (50% blinding)
- Retrofit in the existing channel.
- Discharge of screenings approximately 4 feet above upper floor at grade level so operators would not need to hoist or carry the screenings up the stairs.

Pumping

The original three 5-HP influent pumps (which required seal water) are now 7.5-HP Wilo dry pit submersible pumps. The plant has one spare 7.5 HP pump ready to install and one spare older 5 HP Wilo pump from a prior pump change.

Technical Memorandum #1 documented that peak flows routinely exceed the capacity of the influent pumps, assuming two are duty and one is out of service. A review of operating data from the Town's Mission System indicated the following:

- The firm capacity of the Pump Station (2 pumps running) is 440 +/- gpm. This is based upon existing SCADA data for new pumps running at a typical high wet well level of 5 feet.

- The maximum capacity of the Pump Station (3 pumps running) is 600 +/- gpm. This is based upon existing SCADA data for new pumps running at an excessively high wet well level of 7.5 feet.

- During the Summer of 2021 with 3 pumps running and wet well levels exceeding 10 feet, the flows exceeded the flow meter's 600 gpm capacity for a long period of time.

It should be noted that excessively high wet well levels will result in bypassing the influent screening which could be detrimental to certain future wastewater treatment processes.

New and larger pumps would be required in the upgrade to handle the added flow in order to meet the future peak flows.

Operating staff indicated that the wet well is cleaned every 2 years or so to remove grease and rags. Since the wet well is a single chamber, this is done by using the WPCA's portable pumps and hoses to pump the flow up to RIB#1 or the aerated grit chamber, thus bypassing the wet well and influent screening grinder. Should the use of the RIBs be discontinued, the plant upgrade plans should include a means to allow the flow to be bypassed. Options to deliver the bypassed flow include:

- Use the existing clarigesters (as well as peak flow storage) for temporary storage. This would likely be the most cost effective solution if they are not used for primary clarification and/or sludge storage and least risk as the flow could be later passed through the headworks form screening and grit removal. As discussed below, they will have adequate capacity for this purpose during dry weather.

- Use the aerated grit chamber that flows to the new treatment process. This is less desirable as there would be no screening unless a method was devised to incorporate screening into the bypass pumping system.

Influent Flow Metering

The WPCF's influent flow is metered using a 4-inch magnetic flow meter located in the force main from the influent pumps to the grit chamber. The flow meter is original and only records and reports flow up to 600 gpm. The flow meter should be replaced to record higher flows. In the short term, the Town should consider re-spanning it when it is next calibrated. At the current max flow of 600 gpm, the velocity in the force main is 15 feet per second which may or may not be acceptable to the existing meter. There is no bypass or isolation valve around the existing flow meter, so replacing it would need to be done with bypass pumping.

Grit Removal

Grit removal is performed by a 7 ft wide, 14 ft long, 7 ft side water depth aerated grit chamber that is located outside downstream of the influent pump station. The chamber contains diffusers that are supplied by two (one duty, on standby) locally mounted outdoor enclosed 3 HP blowers each with a capacity of 69 ICFM. Below the diffusers (at the 7 ft side water depth), is an additional 3 foot depth grit storage channel of reduced width that can store approximately 100 cubic feet of grit. A portion of the flow from the blowers is used to aerate the adjacent flow distribution box. There are no air flow meters at this location, so it is not possible to tell how much flow is typically diverted.

The grit chamber contains a bypass channel and a drain. Operators report that the drain valve is broken.

A comparison of the grit chamber's characteristics compared to TR-16 requirements is as follows:

- The chamber has a length to width ratio of 2:1 which is less than the minimum ratio of 3:1 recommended by TR-16.
- The width to depth ratio is 1:1 which is close to the 0.8:0.9 ratio recommended by TR-16.
- At the future peak hour flow of 1,050 gpm, the hydraulic retention time is approximately 5 minutes. Which is within TR-16's recommended range of 3 to 10 minutes.
- TR-16 recommends 3 to 8 SCFM per foot of tank length requiring a blower capacity in the range of 42 to 104 SCFM, so the existing blower's size appears adequate.

Operators have reported good performance, however, grit collected in the clarigesters is pumped off using truck mounted pumps, so pump wear would not be noticed by operators. The grit chamber is cleaned every 2 years and several yards of grit (and rags that get wrapped around the piping and valve stems) are removed.

Aerating wastewater just prior to a treatment process designed for biological nitrogen and phosphorus removal is less than desirable as will be discussed in the next section.

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5.2 Clarigester Evaluation

The existing clarigesters currently perform rudimentary primary clarification at the plant. These units were evaluated to understand how they may or may not be utilized in the WPCF upgrade.

Physical Conditions

- The Clarigesters were designed to act like a combination of a clarifier and a digester in the same 20 ft diameter footprint. The clarifier portion has a 10 foot side water depth, and the digester underneath it has approximately 12 feet of sludge storage/digester depth. They are in essence similar to an Imhoff tank (an older technology that is now rarely used in treatment plants), but with a low floor slope and a mechanism to scrape solids from the clarifier to the lower level and also mix the lower level.

- The volume of the clarifier is approximately 28,200 gallons, and the volume of the digester is 23,500 gallons. If the center baffle separating the two sections were removed, the storage volume of each combined clarifier and digester would be approximately 53,200 gallons. Thus, at an influent flow rate of 200,000 gpd, the two clarigesters would be able to store influent flow for half of a day if necessary for emergency storage purposes.

- The condition of these tanks was assessed by Corrosion Probe in 2019. A copy of the inspection report is attached as part of Appendix B. The report indicated that there were no major corrosion concerns with the clarifier, but maintenance coating work was recommended for all steel components. The digester shows significant acidic attack to the concrete surfaces and aggressive corrosion of steel components with substantial coating failure, indicating a need for restoration and coating work over the next 12-15 months or less. The report did not offer a budget for the repair costs, however the Town estimated cost for the restoration was $250,000 in 2019 dollars, including cleaning and coating the concrete in the lower digester portions of the tank. With inflation, costs are likely $300,000 in December 2023 dollars.

- If the existing clarigesters are used for flow equalization/storage, then all the steel mixing structures should be demolished. The concrete should be rehabilitated in the lower digester portion of the tanks and steel remaining, if any, will have to be cleaned and coated. A pump would be needed to fully empty the storage tanks.

Operations

- Scum collected in the clarigester drops into the scum well which also serves as a valve vault for the clarigester. It is returned to the head of the plant through the 6” drain valve which is left open for this purpose. If the scum well was used to store scum, it would collect rags and debris at a much faster rate than it does now. There is also a sump pump in this chamber. Despite remaining empty, operators recall no issues with freezing of the sample, drain, and overflow lines and their related valves (6 total). This could be because they are rarely used.

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- Every month 6,000 gallons of sludge are hauled away from the plant with approximately 3,000 gallons removed from each clarigester. This suggests that the average age of sludge in each digester is 23,500 gals / 3,000 gal/month = ~8 months. From September 2018 through December 2022, the average solids content of the sludge was 6%, with typical values ranging from 2 to 10%. For plants disposing of liquid sludge from primary and secondary treatment (after mechanical thickening with polymer), a 5 to 6% solids content is typically a good target, as thicker sludge is less pumpable and can be difficult to load and unload from trucks. Primary sludge (such as that from Coventry) that is only gravity thickened is typically more pumpable and can be thicker.

- A review of the clarifier influent and effluent concentrations and sludge disposal volumes for four years suggests the following:

    1. The efficiency of the clarifiers in removing pollutants was less than typical published values of what would be expected in an Imhoff tank, and less than what was assumed by Fuss & O'Neill in their design as summarized in Table 5-1.

TABLE 5-1
Clarigester Removal Efficiency

Criteria	Actual Last 4 Years	Imhoff Tank Typical	Fuss & O'Neill's Design Assumption
TSS	65%	60%	70%
BOD5	15%	30%	30%
TKN-N	5%	0%	0%
P	5%	0%	0%

2. There was a 44% increase in plant TSS loading from 2019 /2020 to 2021/2022. There was a corresponding 44% increase in TSS removed from the clarigesters during the same time period, providing a high level of confidence in the data.

3. From 2019 to 2022, sampling results of the influent and effluent of the clarigesters indicated that they removed an average of 113 dry pounds per day of solids from the wastewater. During that same period, sludge disposal sampling suggested that an average of 107 dry pounds per day of sludge solids were removed from the clarigesters. This difference of less than 5% is well within the margin of error of the estimate and is not conclusive evidence of sludge being digested.

Based on the above, there is little to no evidence that the Clarigesters actually digest sludge to any significant degree, which would result in a benefit of solids reduction. If methane is generated, it is vented to air.

The clarigesters have very poor performance as far as BOD5 removal goes. Industry benchmarking would suggest that 40% BOD5 removal should occur across the clarigesters to accompany the high level of TSS removal that is taking place. This is clear in Figure 5-1, which compares the performance of the clarigesters against industry standards for primary clarification.

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It may be that the digester portion of the clarigester may actually be acting as a fermenter, possibly producing volatile fatty acids (VFAs) that migrate up into the clarifier section and into the effluent. Although VFAs may be beneficial for enhanced biological phosphorus removal if the plant is upgraded, the continued operation of the clarigesters will not provide a significant benefit in reducing the required capacity of a future secondary treatment system.

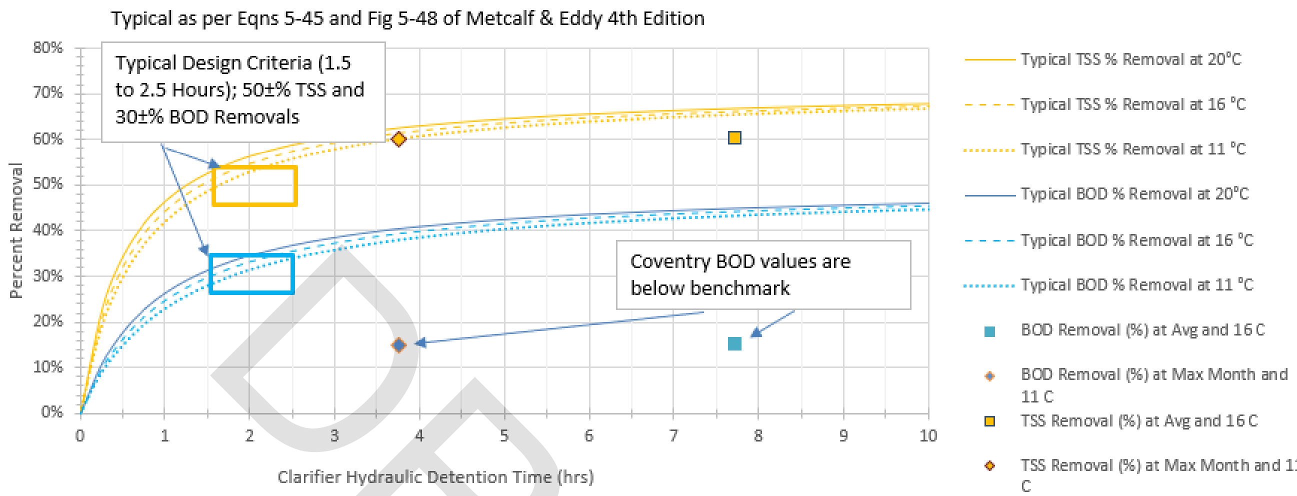

FIGURE 5-1: Comparison of Clarigester Performance vs Typical Primary Clarifiers

Clarigester Recommendations

Although the Coventry treatment plant was originally constructed with primary clarification, it is recommended that serious consideration be given to abandoning primary clarification for the following reasons:

- Very small plants such as Coventry requiring ammonia and/or nutrient removal are often designed without primary clarifiers to simplify operations by reducing the complexity of the plant, the number of processes, the types of sludge that must be handled, and to reduce odors. Examples of these plants can be found in Salisbury and Thompson CT.

- Many small plants with primary clarifiers do not use them, as the additional carbon (BOD5) helps with nitrogen removal and also simplifies operations. Example plants include Plainville and Simsbury CT.

- Eliminating primary clarification reduces odorous primary sludge.

- The operation of the existing clarigesters would not remove significant amounts of BOD5.

- As discussed in Section 3.6, the existing Clarigesters can provide value by potentially using them for peak flow storage.

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5.3 Solids Handling Processes

The solids handling system located adjacent to the Lab/Control/Electrical Room and the sludge feed pumps located in a space in the lower level have not been used in years as the plant currently hauls liquid sludge. It is unknown if this system is functional.

- The sludge feed pumps are Komline Sanderson piston pumps rated for 38 gpm at 20 ft of total head and 10 ft of suction lift. These pumps are too small to load sludge into trucks, thus the trucks currently self-load sludge using their own pump. The pumps could possibly be used to fully drain the bottom half of the clarigesters in about one half a day if they are still functional. The upper half of the clarigesters would drain by gravity. Operators reported that this was attempted once for sludge and the pumps ran but were not able to fully drain the tanks which would require a suction lift of 13 to 14 feet. Given the age of these pumps, consideration should be given to replacing one or both of these with a suction lift pump such as a Gorman Rupp pump if used for this purpose.

- The solids dewatering system consists of the following equipment, which appears to have been designed to generate a lime stabilized sludge suitable for landfilling or land application:
    - Polymer makeup and feed system.
    - Solmat sludge press and supporting equipment.
    - Lime feed system to stabilize the sludge and reduce potential for odors.
    - Drop chute into the truck loading bay below.

No operational/capacity information is available for the solids dewatering system equipment. Given that current options for sludge disposal (either liquid or cake) in Connecticut focus on incineration, it does not appear to have any value for use in the future. Therefore, given its age and unknown functionality, serious consideration should be given to demolishing this equipment and using the space for relocating electrical equipment out of the existing control room/lab, or for thickening liquid sludge.

If treatment at the plant continues in the future and the plant is upgraded, depending on the secondary treatment technology selected, for a treatment plant of this size without primary clarifiers, it is expected that the volume of sludge generated will increase significantly, due to the increase in plant load and increased BOD5 removal. The number of truckloads of sludge could more than double; increasing from one a month to one a week or so.

Based on the preliminary assumptions stated above, it is recommended that consideration be given to a future solids handling system based on the following, with the understanding that the economic viability must be confirmed during the preliminary design phase of the plant upgrade:

- The ability to store and load trucks with sludge that has been thickened (from 0.4% to 1% solids content typically generated by a secondary treatment process) to 5±% solids content (consistent with current sludge disposal solids content). Transportation costs are minimized when large 6,500 gallon capacity trucks can be fully loaded within 20 minutes or so with a 300-gpm capacity pump. The current

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sludge hauler comes with 3,000 gallon septage trucks that can self-load and that full size tanker trucks have been able to access the site in the past to load sludge.

- The active storage volume of a single thickened sludge storage tank should be on the order of one to two weeks of sludge generation or the volume of two truckloads of sludge, whichever is larger.
- The sludge in the thickened sludge storage tank should be mixed. A blower and air diffusers in the tank will help reduce odors, reduce tank corrosion, and also maintain a sludge a uniform consistency within the tank. The blowers could be cycled on and off to save energy, with run times adjusted based on the operator's experience.
- The waste activated sludge removed from the secondary process should be thickened using either membranes or a rotary drum thickener.
- If allowed by the secondary treatment process selected (and to reduce capital and operating costs), it is assumed that ideally sludge would be wasted periodically (for 6 to 8 hours one to two days a week) directly from the secondary treatment process to the thickening process. This avoids the need for a mixed thickened sludge storage tank, a common strategy used at smaller plants.
- It is noted that if the treatment plant is designed to biologically remove phosphorus (and minimize chemical use), the sludge would have to be aerated at all times to keep the oxygen levels high enough to keep phosphorus from being released.

It is also noted that some secondary treatment processes (including Sequencing Batch Reactors) will likely need a mixed unthickened sludge storage tank due to the nature of their operation.

5.4 Plant and Potable Water System

Plant (non-potable) Water is derived from potable water system using a backflow preventer. Operators report that the pressure and flow is too low at the existing hose bibs for effective cleaning. It may be possible to address this to a small degree by increasing the potable water system pressure. However, real improvements will likely require larger piping.

Plant water, which original drawings call out as Flushing water, is/was used for the following:

- Hose bibs for cleaning tanks around the plant
- Flushing connections at the sludge line from the clarigesters
- Influent Pump Seal water (discontinued)
- Solid Handling (discontinued)

Depending on how plant water is used in the future, Connecticut's Cross Connection Control Manual, (which is a guidance manual for water system purveyors and regulators) suggests that the use of just a backflow preventer in the future is not acceptable. This is because an "air gap" is recommended between all potable water lines and equipment or systems which may be subject to contamination. The guidance manual is clear that an

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"Air Gap" is the only acceptable protection for "Sewage Pumps/Sewage Connected Equipment". Continued use of potable water for plant water in the future should be reviewed by the local health officials.

Potable water is provided by a 110 foot deep gravel well with a 2 HP 3-phase submersible pump set 80 foot deep with a 1.5" PVC riser that is approximately 470 feet away from the building via 2" water line. The well yielded 35 gpm when constructed in 1986 and the pump was replaced in 2017 and tested at 12.5 gpm. Operators believe that the pump was rated for 21 to 28 gpm, but this has not been confirmed.

The 211 gallon (800 Liter) bladder style hydro-pneumatic tank (precharged to 30 psi) is located in the Boiler room. The normal operating pressure for this type of system is 40 to 60 psi, and a review of the recent replacement pump invoice confirms that the settings were 42 to 60 psi. Operators confirmed with testing that pump and switches are operating between 42 and 58 psi which is generally consistent with design intent. The pressure drop of the 2" copper potable piping between the well and the main building is estimated at 5 psi for a sustained flow of 30 gpm.

Based on the CT Public Health Code 19-13-B51d, and the assumption that the wells' withdrawal rate is between 10 and 50 gpm, the minimum required separation distances from the well are as follows:

- 150 ft from sewage disposal or other source of pollution
- 75 ft from a sewer line constructed of extra heavy cast iron pipe with leaded joints or equal.
- 50 ft from high water mark of surface water body or drain carrying surface water or foundation drain.

At this time, it is unclear if the plant water and potable water systems have sufficient capacity for an emergency shower/eyewash which requires a supply of tepid water for 15 minutes at 20 gpm. This is required if corrosive chemicals are used on-site for disinfection or phosphorus removal. Other possible requirements for plant water in future systems include fine screening, fluidizing grit for removal, sludge thickening systems, and cleaning of larger tanks in the future. The volume of water required for these purposes can be significant.

5.5 Disinfection

The current treatment plant does not disinfect the effluent. Pathogen reduction is accomplished in the subsurface groundwater discharge. If the treatment plant is upgraded for a surface water discharge, it is assumed that UV disinfection would be used as it should have no problem reaching the anticipated effluent criteria for Fecal coliform with the anticipated effluent quality without the need for additional treatment/filtration that would be required for more stringent effluent criteria.

5.6 Available Space for New Facilities

If the decision is made to upgrade the existing WPCF, adequate space must exist for the construction of all new facilities, and provisions must be made to allow for treatment processes to remain operational during construction.

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The WPCF currently utilizes 8 rapid infiltration basins (RIBs) for groundwater discharge. Discussions with operating personnel have confirmed that it is currently not feasible to take one or more RIBs out of service without impacting the ability of the plant to dispose of effluent during periods of high flow. The treatment plant site has a large area at the southern end of the property, part of which is being used as a cornfield.

Therefore, the following assumptions were made to allow the plant construction to move forward while still maintaining eight RIBs in service:

- Basin No. 1, which is adjacent to the existing headworks building, would be decommissioned. This space will be used for the construction of an upgraded treatment plant. Additional space in between the existing clarigesters and the RIBS will be used if necessary.

- Basin No. 6 will be expanded to the southwest, partially into the cornfield to make up for the loss of Basin 1. Once the treatment plant upgrade is complete, the basin extension will be demolished, and the cornfield can be re-established.

An overview of this suggested approach is presented in Figure 5-2.

It is also noted that the existing treatment plant construction drawings make note of Archaeological sites located between the treatment plant and the river in the vicinity of the operations building. These were preserved during construction and are located outside of the fenced area. It is assumed that no expansions are appropriate in these areas.

5.7 Additional Recommended Improvements

The existing wastewater treatment plant was constructed in 1985, and as such, there are a number of components that are at or nearing the end of their useful life. In addition to the treatment process items discussed above, an inspection of the existing treatment plant components was conducted from a structural and electrical perspective in March 2023. Input was also sought from operating staff on specific improvements that should be made. The following recommended basic improvements will be considered under the treatment plant upgrade and Windham connection alternatives as applicable. These will be updated further in Sections 6 and 7.

<u>Structural</u>:

- Replacement of the existing membrane roof and provide new exterior stairway for access.
- New larger door on the south side of the building near the laboratory.
- Interior painting of CMU walls in the generator room
- Recoat concrete floors in the truckway.
- Repair building exterior where woodpecker damage was noted.

<u>Electrical</u>:

- New electric utility service downstream of the existing transformer, including a new Main Circuit Breaker, Distribution Panelboard, replacement of the MCC and a new Lighting Panelboard
- LED Lighting Upgrades
- Conduit and Wire as applicable

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FIGURE 5-2
WPCF AVAILABLE SPACE FOR CONSTRUCTION

LEGEND

- Area of Potential Temporary RIB Extension
- Area For New WWTP Facilities
- CT Municipal Boundary

LOCUS MAP

0 75 150
Feet
1:1,800

NOTES

1. Based on 2019 Statewide Orthophotography, Courtesy of CTECO.

Coventry WPCF
Coventry, Connecticut

December 2023

Tighe&Bond

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Section 6
Treatment Plant Alternatives Analysis and Recommendations

6.1 Treatment Plant Discharge

In order to handle the future flows and loads, the Coventry WPCF must be able to treat flows received to the level required for following alternatives considered in this study:

1. Fully Treat on-site and discharge to the groundwater
2. Fully Treat on-site and discharge to the Willimantic River

For the groundwater alternative, an evaluation was conducted of the feasibility of constructing a large-scale on-site subsurface wastewater absorption system (SWAS) to replace existing rapid infiltration basins (RIBs) which currently discharge treated municipal wastewater flows from the Coventry, CT Water Pollution Control Facility (WPCF).

The analysis was conducted to determine the feasibility of a continued groundwater discharge as part of an upgraded wastewater treatment facility. The analysis was based upon the assumption that the existing RIBs adjacent to the Willimantic River do not meet current CTDEEP standards. Because of this, a new groundwater discharge will require abandonment of the RIBs and installation of a subsurface disposal system.

The selection and preliminary sizing of a subsurface disposal system was based upon a desk-top evaluation of open land within and adjacent to the existing WPCF property. The evaluation considered offsets to property boundaries, proximity to water supplies and structures, published soil information and groundwater elevation data, and available GIS data for watercourses, waterbodies, flood zones, wetlands, critical habitat, and natural diversity database (NDDB) areas.

The anticipated size of SWAS was developed utilizing guidelines presented in the Connecticut Department of Energy and Environmental Protection (DEEP) Guidance Manual for Large-Scale On-Site Wastewater Renovation Systems (DEEP Guidance Manual).

Reference is made to the Technical Memorandum attached at the end of this Report as Appendix C which confirms that maintaining the current groundwater discharge for the Coventry WPCF is not feasible. This was submitted to and approved by the Town and the Connecticut DEEP in July 2022.

The evaluation of a new treatment facility will therefore assume a new discharge to the Willimantic River and is discussed in the following subsections.

6.2 Future Discharge Permit Limits/Design Criteria

A discharge to the Willimantic River will require a NPDES discharge permit, which according to the CT DEEP is permittable because the river is a Class B river. It is already receiving NPDES discharges from the Town of Stafford and Uconn WPCFs. The Town of Windham WPCF discharges into the Shetucket River near the confluence of the two rivers. For planning purposes, DEEP has recommended considering NPDES permit limits similar

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to the other plants currently discharging these rivers. Therefore, based upon a review of these other permits, the following limits will be included when evaluating a future on-site treatment facility:

- BOD & TSS:
  - 30 mg/l Average Monthly and 50 mg/l Max day are standard limits in the NPDES permit program. It should be noted that in order to achieve nutrient removal, the treatment system will have to routinely achieve less than 5-10 mg/l in the effluent.
- Total Nitrogen:
  - A review was conducted of DEEP's nitrogen trading program, which targets a reduction in the total pounds of nitrogen discharge from pre-2000 levels by ~60% per EPAs TMDL. If loads are assumed to remain unchanged from that time and all nitrogen discharged from the existing WPCF to groundwater eventually makes its way to Long Island Sound, this would mean decreasing the pounds per day of nitrogen discharged from the WPCF from 47 to 18 pounds per day, which at a flow of 260,000 gpd is 7 mg/l.
  - At this time, it is unclear if the discharge will receive a hard permit limit for nitrogen or if it could be included in the State's Nitrogen trading program. To be conservative, it will be assumed that the treatment system will need to achieve the lower limit of the 6 to 8 mg/l range that was suggested by DEEP.
  - Therefore, for planning purposes, a limit of 6 mg/l will be assumed which at a flow of 260,000 gpd is 13.0 lbs/day.
- Total Phosphorus:
  - DEEP's 2014 Interim Phosphorus Strategy capped the pounds per day being discharged from the existing WPCFs in the Willimantic River to levels seen from 2001 and 2007 in order to prevent the "enrichment factor" of phosphorus in the river from exceeding the maximum target factor of 8.4 mg/l. Because Coventry's discharge was to groundwater (and it is generally assumed that soil absorbed the phosphorus), the discharge from Coventry WPCF was not included in the study (even though there is evidence some phosphorus in the groundwater may have entered the river).
  - As indicated in Figure 6 of Appendix B, the other WPCFs in the river were capped at existing discharge levels on a pounds per day basis. Also as indicated in Figure 6, an argument could be made that if Coventry's phosphorus discharge to the river was included in the strategy, Coventry would also have been capped at 6.5 lbs/day while maintaining an enrichment factor of less than 8.4 in the river. This loading at future flows of 260,000 gpd is a concentration of <3.0 mg/l.
  - For planning purposes, it was assumed that an effluent concentration of 0.7 mg/l (at 260,000 gpd, this is 1.52 lbs/day) would be required which is equivalent to the average historical discharge concentration for Stafford's WPCF and slightly less than Windham's at ~ 1.0 mg/l. It is noted that Windham technically discharges to the Shetucket River and this is not clear in Figure 6.

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  - It is noted that if the limit is as low at 0.5 mg/l, then this could likely be achieved with only additional chemical impacting only operational costs. It is also noted that Coventry should argue for a limit as high as 2.5 mg/l which is similar to UConn's historical discharge level.
- Ammonia:
  - The following limits exist for the other WPCFs in the river:
    - Stafford: No Limit in Winter (Nov thru May) and 3-4 mg/l in Summer
    - UConn: 20 mg/l in Winter (Nov thru May) and 2 mg/l in Summer
    - Windham: (No Limits)
  - For planning purposes and to meet the Total Nitrogen Limit, it was assumed Ammonia would need to be 2 mg/l year round.
- Metals:
  - While the Uconn WPCF has no limits on Copper, Stafford and Windham do have limits of 0.355 and 0.614 kg/day for the 30 day average as well as maximum daily limits.
  - For planning purposes, it is assumed that treatment for metals will not be needed. Any further metals evaluation is beyond the scope of this study. The plant does not collect metals data for its current effluent. Specific discharge permit requirements related to metals will ultimately be dependent on future industrial expansion, if any. DEEP has indicated that limits on heavy metals will be contingent on future industry and water quality data collected at the plant, evaluated based on current water quality regulations.
- Disinfection: Seasonal Bacteria.
  - It will be assumed that Coventry will match Stafford, Windham & UConn's WPCF Limits of 410 colonies per 100 ml for E-Coli.
- Dissolved Oxygen:
  - CTDEEP has indicated that an expanded plant will be required to meet the dissolved oxygen (DO) requirement for Class B waters of not less than 5 mg/l at any time.
  - This may trigger the need for post aeration under certain secondary treatment technologies.

A summary of the design criteria to be used in the treatment plan evaluation is summarized in Table 6-1 below:

TABLE 6-1
Surface Water Discharge Criteria

BOD5 (mg/l)	TSS (mg/l)	Ammonia Nitrogen (mg/l)	Total Nitrogen (mg/l)	Phosphorus (mg/l)	E-Coli (Col/100ml)	DO (mg/l min)
30	30	2	6	0.7	410	5

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6.3 Influent Pump Station and Headworks

Under all secondary treatment technologies, the influent pump station and headworks would be generally upgraded as follows:

- New headworks Influent Screens would be provided as discussed in Section 5. However, it is recognized that some treatment technologies being considered might require finer screening that may not be achievable in the available existing footprint.
- The existing pump station with 8" discharge pipes and 8" suction pipes from the existing wet wells would not require modification if three pumps are selected to handle future flows. This is because the piping appears to have been designed with future growth in mind.
- A larger influent magnetic flow meter would be provided.
- The existing aerated grit removal system, having sufficient capacity, would remain as discussed in Section 5.
- The balance of the treatment process would ideally flow by gravity from the effluent of the existing aerated grit chamber.
- Three new larger influent pumps would be provided to handle the design peak flow of 1050 gpm as discussed below.

The influent pump criteria were developed at the same time as selecting pumps for the force main to Windham as discussed in Section 7. The initial goal was to consider three pumps capable of providing 525 gpm at 40 feet of THD operating during wet weather with smaller jockey pumps being used to handle dry weather flows with near continuous pumping.

However, as discussed in detail for the Windham pump selection, the use of smaller jockey pumps that could turn down to the low flow that enters the plant was found to be impractical. The larger pumps turndown on VFDs was likely limited to 2:1 meaning that the treatment process would likely see flows vary between 0 gpm (pump off) and 260 gpm (Pump running at minimum speed) during dry weather. This means that the starting and stopping of flows will need to be considered while designing the downstream treatment processes, especially the disinfection systems and plant water systems.

Based on the criteria specified above, the vendor provided a selection for three pumps designed to pump at a rate of 525 gpm each.

6.4 Treatment Plant Technologies Overview/Screening

6.4.1 Nitrogen and Phosphorus Removal Overview

This section describes various biological treatment processes commonly used for nitrogen and phosphorus removal. This preliminary screening has been performed in order to develop a short-list of potential alternatives for Coventry's WPCF which will be evaluated in more detail in the next phase of this project.

An overview of basic nitrogen and phosphorus removal is presented below.

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- Total Nitrogen is composed of organic and inorganic forms of nitrogen, and is measured as the sum of organic, ammonia, nitrite and nitrate forms (all measured "as nitrogen"). Biological nitrogen removal is a two-step process which depends on the conversion of ammonia and organic nitrogen to nitrate in an aerobic environment (nitrification), and subsequent conversion of nitrate to nitrogen gas in an anoxic environment (denitrification). Because the microorganisms performing biological denitrification require an energy source (e.g., BOD), denitrification is typically practiced in a pre-anoxic zone to take advantage of the readily available carbon source in the influent ahead (in either space or time) of the aerated zone where most of the BOD removal and nitrification occurs. Because nitrification is the first step in the nitrogen removal process, the nitrate rich recycle stream from the aerobic zone is recycled to the pre-anoxic zone to achieve denitrification. For treatment processes that include a post-anoxic zone, an external source of carbon like methanol or glycerin is often added as an energy source. Nitrogen removal can also occur endogenously, but this occurs at a much slower rate and requires larger process tanks.

- Phosphorus removal typically occurs through chemical treatment via the addition of a coagulant (generally an iron or aluminum salt), biological treatment, or a combination of both, followed by solids removal.

  - Biological phosphorus removal is a two-step activated sludge process requiring an anaerobic zone followed by an aerobic zone. In the anaerobic zone, phosphate accumulating organisms (PAOs) will release phosphorus. The released phosphorus, as well as much of the influent phosphorus, is subsequently taken up by PAOs in the aerobic zone as a means of storing energy for the future growth that occurs under anaerobic conditions. The phosphorus is removed when the PAOs are settled/filtered and subsequently removed from the system with the waste solids. As noted, PAOs release phosphorus under anaerobic conditions. Therefore, the waste sludge containing PAOs removed from the activated sludge process and any solids handling processes must be maintained under aerobic conditions. If this is not the case, then the decant from any thickening or dewatering processes must be treated chemically to remove the released phosphorus.
  - Chemical phosphorus removal consists of precipitating soluble phosphorus through the addition of coagulating chemicals and formation of settleable particles. Coagulants typically consist of metal salts, particularly aluminum and iron salts such as aluminum sulfate (alum), polyaluminum chloride (PACL,) and ferric chloride. The ability of coagulants and soluble phosphorus to form precipitates depends on factors such as pH and the coagulant dose. Insoluble metal-phosphate precipitates are subsequently removed in the clarifiers (or other solids wasting processes). Additional solids separation processes (settling and filtration) can also be added downstream of the secondary clarifiers for additional "low level" phosphorus removal. This is referred to as tertiary phosphorus removal.

Generally, biological phosphorus removal has been shown to reliably achieve effluent total phosphorus (TP) values of 1.0 mg/L or less, while chemical phosphorus removal alone or in combination with biological phosphorus removal has been shown to achieve effluent TP values of approximately 0.5 mg/L. Effluent concentrations less than 0.5 mg/L would typically require a tertiary treatment process that will not be considered for Coventry.

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For the limits expected for Coventry, it is expected that DEEP would want chemical phosphorus removal included in the design to maintain required effluent TP permit limits during periods of time when biological phosphorus removal, if considered, does not perform well. Most plants prefer the use of PACL for phosphorus removal when dosing into the treatment process because it consumes very little alkalinity compared to the other common alternatives. Loss of alkalinity can interfere with nitrification and nitrogen removal.

The treatment plant currently does not collect alkalinity data. For the remainder of this evaluation, it is assumed that the plant has sufficient alkalinity for the required nitrification and that PACL will be used for chemical phosphorus removal. It is recommended that alkalinity and pH measurements be made in the future if on-site treatment is considered.

6.4.2 Secondary Treatment Technology Evaluation

Several secondary treatment technologies screened for potential use at the Coventry WPCF. This screening was performed to select the top three technologies that are the most appropriate to small system such as Coventry's, given the influent loads, anticipated effluent criteria, and the fact that the existing plant is currently staffed with 2 operators.

Specific technologies evaluated are as follows:

- Conventional Activated Sludge – Modified Ludzack-Ettinger (MLE)
- Conventional Activated Sludge – A20 Process – Anaerobic/Anoxic/Oxic
- Conventional Activated Sludge – 4 Stage Bardenpho
- Conventional Activated Sludge – 5 Stage Bardenpho
- Rotating Biological Contactors with Add On Treatment
- Moving Bed Biofilm Reactors
- Conventional Activated Sludge – Oxidation Ditch
- Sequencing Batch Reactors
- AquaNereda: Granular Sludge in an SBR Configuration
- Membrane Reactors

An overview of each treatment technology was presented in Technical Memorandum No. 3 which was submitted to and approved by the Town of Coventry and the Connecticut DEEP in July 2023. A copy of this memo is included in Appendix B. Please refer to Appendix B for descriptions of each of the treatment technologies listed above.

6.4.3 Short List of Treatment Processes

As stated earlier, the Coventry WPCF has a relatively small flow rate and will require both total nitrogen and total phosphorus removal. Based on its treatment capacity, the anticipated discharge limits for nitrogen (N) and phosphorus (P), and the anticipated space constraints, the following treatment processes were excluded for further analysis:

- Modified Ludzack-Ettinger (MLE): The MLE process results in higher typical effluent TN concentrations than Coventry WPCF's expected discharge limits. Additionally, the MLE process is not specifically optimized for phosphorus removal.

- A2O, 4 and 5 stage Barden Pho Processes: These processes with their many tanks and monitoring requirements are more difficult to operate than an oxidation ditch which can perform a similar function while being more robust and simpler to operate.
- RBCs with Add Ons: With new construction, it does not make sense to consider multiple processes to achieve the desired nutrient removal.
- MBBRs: These are not commonly used in Municipal Wastewater treatment and there are none currently operating in Connecticut. There is also a risk of releasing media to the environment. While these can help reduce the treatment system footprint, other technologies (MBRs) can also significantly reduce the system footprint if space is a concern.
- AquaNereda SBR: AquaNereda SBR requires a higher capital cost and provides a level of treatment efficiency that exceeds the necessary requirements for the Coventry WPCF. The system also relies heavily on instrumentation which would result in high maintenance costs for a system sized such as Coventry. The applicability of using this system in Coventry was discussed with the manufacturer, who also felt that for Coventry's size, the increased cost and complexity over a traditional SBR (which they also sell) was not justified.

The three alternatives shortlisted are listed below, along with their advantages and disadvantages:

Activated Sludge Oxidation Ditch

Advantages

- Does not require fine screens to protect equipment.
- Long solids retention time enhances removal of organic matter and suspended solids and also lowers sludge yield rates to minimize solids production.
- Resilient in handling temporary spikes in organic loading or hydraulic flow without significant adverse effects on treatment performance.
- Ease of operation and maintenance.
- Require relatively shallow tanks (on the order of 10-12 feet) that would not require deep excavation at Coventry's site allowing them to be constructed at a similar hydraulic grade line as the existing aerated grit chamber (allowing it to be reused).
- No Nitrate recycle pumps required.
- Larger size allows storage of sludge in the process prior to dewatering.

Disadvantages

- Requires larger footprints compared to SBR and MBR.
- Requires downstream clarifiers for solids separation.

Manufacturers

- EIMCO (now Ovivo) Carrousel® (Jewett City, Simsbury CT, UCONN, New Canaan, Suffield, Ansonia, Westport some without primary clarification with mechanical aerators.
- Evoqua Orbal® with brush style aerators (Seabrook NH)

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SBR

Advantages

- Does not require fine screens to protect equipment.
- Small footprint and no need for a downstream clarifier or return sludge pumping.
- Lower energy consumption due to intermittent operation and the absence of continuous aeration during non-aeration phases.

Disadvantages

- Complex control and operation
- Peak flow handling required adjustment in cycle times and/or necessitating larger tank size and/or storage to handle for peak flow events.
- Potential for foaming and scum formation, which is difficult to remove (and often is not) due to the varying water level in the tank.
- Generates thinner sludge during each cycle, requiring an unthickened sludge storage tank prior to sludge thickening process. Such a tank would have to be aerated if biological phosphorus removal was to be relied upon.
- The batch nature of the process relies on deep/taller tanks (on the order of 20 feet) that would require the tanks be constructed deep into the ground (allowing reuse of the existing grit chamber) or up higher (requiring new a new aerated grit chamber).
- The batch nature of the process results in the rising and falling of the water level in the reactors effluent flows that start and stop. To allow for efficient disinfection (assuming UV), this would require additional post equalization tanks and either additional pumping or automated modulating valves. The net result of this could be a deep (buried) UV system and effluent pipe to the river or a tall SBR structure with new grit chamber and larger HP influent pumps.

Manufacturers

- Aqua Aerobics markets a true batch SBR system that is common in CT (Thomaston @ 1.3 MGD, Plainville @ 2.2 MGD) and is now being designed in Orleans MA
- Jet Tech (Shelton).

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MBR

Advantages

- Small footprint and no need for a downstream clarifier.
- Resilience in handling temporary spikes in organic loading or hydraulic flow without significant adverse effects on treatment performance.
- Reduced sludge production, resulting in potential cost saving for sludge disposal.

Disadvantages

- Fine screening is required. The Coventry WWTP headworks has spatial constraints that will likely prevent the installation of any form of fine screen system within the current headworks building. This would likely require a new headworks building to accommodate a fine screen. Even with Fine screening, hair can build up in tanks.

- Higher capital and operating cost due to membrane modules inclusion and regular cleaning and replacement requirements. Higher peak flows require more membranes driving up costs for membranes of peak flow storage.
- Higher energy consumption due to continuous aeration for biological process and membrane scouring.
- Sensitive to high concentration of solids, grease, and certain chemicals, potentially requiring preliminary treatment.
- Lower sludge settling rate, which may require chemical addition.

Manufacturers

- Ovivo with Kabota Plate Membranes (Wayland, MA @ 0.052 MGD, Westford MA @ 0.1 MGD, Southborough MA @ 0.03 MGD)
- Zenon (were replaced after 6 years with Lane Christiansen in Redding/Georgetown CT)

6.5 Short List Vendor Evaluation Methodology

In order to determine the most appropriate secondary treatment process to be used in Coventry, formal quotes were obtained from equipment vendors for each of the 3 short-listed technologies.

6.5.1 Evaluation Assumptions

The following assumptions were made for the purposes of evaluating the vendor responses:

- Flow from the existing aerated grit tanks to the new secondary treatment processes would be by gravity using a new connection to the aerated grit tank. High Flows would also flow by gravity to the clarigesters using a new telescopic valve installed through the floor of the aerated grit chamber effluent channel.

- All tanks required for the secondary treatment processes would be concrete constructed on site by the Contractor and not supplied by the vendor.

- A geotechnical /hydrogeological analysis conducted during design would confirm the following assumptions based on currently available boring logs:

    - Piles would not be required under new structures. (the existing building does not have piles).
    - The new tankage can be constructed in RIB #1 after removal of 1 foot of organically impacted material at the top of the RIB. That material can potentially be stockpiled on-site and used to level out the area in RIB #1 around the new structures.
    - Stormwater run-off from newly developed areas could be directed to the existing RIBs for infiltration after minor treatment if required.

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- Blowers for process aeration and/or solids handling systems, if required, would be pad mounted outdoors, similar to the existing aerated grit system blowers but placed under cover.

- Sophisticated monitoring of nutrients (using phosphorus or nitrate analyzers) would not be considered.

- Without primary clarifiers or chemical addition for phosphorus removal, the gross sludge yield from the secondary process used by the vendors would likely be in the range of 0.85 to 1.0 lbs/VSS per lbs BOD removed. This would depend in part on the solids residence time (SRT), with longer times residence times resulting in a lower yield.

- The solids handling system (not provided by the secondary treatment process vendors) would include two aerated sludge storage tanks along with the related blower systems and air diffusers. Additional solids handling components/assumptions include:

  - One tank would hold unthickened waste activated sludge (WAS) and have a capacity of 10 days of storage at the concentration generated by the specific secondary treatment process at future maximum month flow. This is a conservatively sized to handle additional solids at annual average flows if chemical phosphorus removal is required. The size of the system would be similar for all of the alternatives.
  - The second tank would hold thickened waste activated sludge (TWAS). This tank would be covered to keep out rain and have a minimum capacity of two 6,500 gallon truckloads of TWAS at 5% solids or the volume of TWAS that could be generated from the WAS tank volume, whichever is larger.
  - A 100 gpm rotary drum thickener (RDT) running at ~75 gpm (which can process approximately 27,000 gals of WAS in one shift with a positive displacement feed pump of the same capacity. This equates to roughly a long 3-day weekend of sludge production at a rate of 9,000 gpd.
  - A thickened sludge pump operating at a rate of 15 gpm to send thickened sludge to the thickened sludge holding tank.
  - A 300 gpm truck loading pump to load a 6500 gallon truck in around 20 minutes.
  - The pumps and RDT would be located in the lower level of the existing building (likely in the existing sludge truckway) at elevation 280.5. The storage tanks would have a high water level at around elevation 295.5 which is generally consistent with grades near the existing clarigesters. The RDT sludge feed pump would require a three way motorized valve to prevent siphoning to the RDT. When pumping up from the TWAS tank, the static head would be as high as 14 feet. When pumping to the RDT, the static head will vary from near zero to negative 14 feet.
  - The WAS tanks would be aerated using coarse bubble diffusers for both keeping biological phosphorus in suspension and keeping the sludge uniformly mixed. The tanks would be around 13 feet deep (high water level of 12 feet and a

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working depth of approximately 11 feet) and will be aerated at a rate of 15 to 20 scfm per 1000 cubic feet using two blowers on variable speed drives.

- The two blowers would be rated for outdoor installation and will be valved to service either tank. In the event of a blower failure, valves would allow the blower to service one tank at a time (mixing WAS while dewatering, mixing TWAS periodically and prior to truck loading). Based on the above criteria, the sizing assumptions are as follows:
  - For a 94,000 gallon WAS tank with a maximum water depth of 12 feet and 11 to 11.5 feet of submergence, 20 SCFM per 1000 cf equates to 250 scfm at approximately 6 psi. A 10 HP blower with a usage of ~155 KWH per day if run at full speed is assumed.
  - For a 19,000 gallon TWAS tank with a maximum water depth of 12 feet and 11 to 11.5 feet of submergence, 20 SCFM per 1000 cf equates to 50 scfm at 6 psi. A 3 HP blower with a usage of ~31 KWH per day is assumed.
  - In order to use two identical blowers, a rate of 15 SCFM per 1000 cf was assumed for the WAS tank. It was also assumed that the second blower could be turned down or excess air sent to the aerated grit tank. Thus, each blower would be sized for 190 scfm at 6 psi. A 7.5 HP blower with a usage of ~116 KWH per day if run at full speed was assumed. The corresponding TWAS tank energy consumption is ~24 KWH per day if run at min speed.
  - In practice, the blowers will likely be run at lower speeds and may be cycled on and off. A 50% usage factor was used for estimating electrical consumption.
- Although it might be argued that for such a small plant it may not make sense to consider mechanical sludge thickening (reducing capital costs but increasing operation costs), this decision would be left for a value engineering exercise for final design should on-site treatment with a river discharge be selected. Reducing the size of the WAS tank and RDT can also be an option.

- Provisions for odor control as part of the plant upgrade alternative evaluation were limited to just the RDT operations in the building for the following reasons:
  - There is no odor control at the plant now and no history of complaints despite the following:
    - The wet well is ventilated to atmosphere
    - The aerated grit chamber is uncovered
    - The primary clarifier is uncovered.
    - There is a large area of Rapid Infiltration Basins
  - In the proposed upgrade, the following changes should reduce existing odors:
    - Elimination of primary sludge which is the sludge that typically produces objectionable odors.
    - The RIBs, a significant source of odors, will also be eliminated.
  - The waste activated sludge will be kept aerated and fresh and should have a similar odor profile to the biological process tanks that will be uncovered.
  - There is a lack of odor receptors in the area.

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  - If odor control is needed in the future, it would likely be required only on the covered thickened sludge holding tank, in case the blowers are shut off to save energy and sludge stops remaining fresh. Provisions could be made for adding activated Carbon based odor control.

- There is sufficient readily biodegradable BOD to drive denitrification in the secondary process to the required effluent total nitrogen levels, thus, a supplemental carbon source (methanol, micro-C, or similar product) would not be required. However, allowing room for such a source should be accommodated in the final design layout should it be needed in the future.

- Enhanced Biological phosphorus removal (if proposed by the vendors) would be seasonally backed up by a chemical based phosphorus removal system.
  - Based on the anticipated influent to the plant, and assuming no enhanced biological phosphorus removal (only typical uptake thru assimilation at 2% by weight of the wasted volatile solids), the following effluent concentrations would be expected without chemical removal:
    - Current ADF = 3.7 mg/l
    - Future ADF = 4.2 mg/l
  - Assuming no biological phosphorus removal, coagulant would be dosed at the following rates and costs to reduce the effluent to 0.6 mg/l assuming a molar ratio of 2.2:
    - Current ADF = 12 gallons per day ($54 per day at current costs)
    - Future ADF = 27 gallons per day ($122 per day at current costs)

    The costs above are based upon the coagulant being poly aluminum chloride / EPIC 58 from Holland Chemical. There are other options that consume more alkalinity, which is less than desirable.
  - Enhanced biological phosphorus removal performs best with higher BOD:P ratios with 20:1 (or 30:1 for some references) being the minimum recommended. Based on the load analyses discussed in Tables 3-3 and 3-10, Coventry's ratios are:
    - Current ADF = 194:6.5 = 30:1
    - Future ADF =398:13.7 = 29:1
    - Current MMF = 370:20 = 18.5:1
    - Future MMF =760:34 = 22:1
  - Based on average day conditions, it appears that enhanced biological phosphorus removal is viable and should be considered, at least initially. Based on the maximum month conditions, chemical addition will be required as backup to enhanced biological phosphorus removal. The following assumptions were made:
    - If enhanced biological phosphorus removal was proposed, it was assumed that two months of chemical addition per year at the ADF rates discussed above would be required. In practice, the dosing would likely be less per day and occur over a longer period of time using the same amount of chemical.

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      - If enhanced biological phosphorus removal was not proposed, it was assumed that 7 months of chemical addition per year at the ADF rates discussed above consistent with the seasonal (April through October) limit would be required.

      - Since the beginning of the phosphorous season is generally consistent with the coldest plant influent temperatures and there is the possibility of a future winter limit, the vendors were not advised that the phosphorus limit was seasonal.

      - It is also recommended that chemicals be stored indoors in case of a future year round limit, and because some of the chemicals can freeze.

   - Although it might be argued that for such a small plant, it may not make sense to consider enhanced biological phosphorus removal (reducing tankage and/or tank sizes), but increasing operation costs, this decision would be left for a value engineering exercise for final design should on-site treatment with a river discharge be elected.

   - Based on average day conditions, it is expected that chemical addition for phosphorus removal would increase the secondary process sludge yield by 10 to 20%.

- On-site chemical storage and pumping areas should be limited to the extent possible. Larger volumes of storage might need to be located outdoors for the following reasons:

   - The site is supplied by well water in limited quantities that cannot adequately supply an emergency shower directly for the time period required. Therefore, each area will require an emergency shower/eyewash that is supplied by a tepid water supply that can provide 20 gpm for 15 minutes. This will require at least one package system with a 300-gallon tank, pump, automatic fill system and possibly water heater. It is assumed that this system would be located in the lower level of the existing building (possibly in the boiler room).

   - The well also cannot adequately supply a sprinkler system. To meet code without sprinkling a building (which would be prohibitively expensive unless the local code official is willing to grant an exemption), the volume of chemicals that can be put inside a building would be limited to:

      - 500 gallons of "process tankage"

      - 900 gallons of "storage"

      - It could be argued that Alum or PAC tanks in a room are "storage" that feed the "secondary treatment process" located elsewhere.

   - For phosphorus removal with 27 gpd of chemical usage (no enhanced biological phosphorus removal at average future conditions), 900 gallons equates to 33 days of storage. 30 days of storage is the minimum volume that is typically required. Ideally, this can be provided in 2 450-gallon tanks. With enhanced biological phosphorus removal, chemical usage should be lower and storage should last much longer.

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  - If any other chemicals are used, they would likely need to be located outside or in an additional fire separated structure.

  - It was also assumed that the chemical storage tanks and pumps would be located in the lower level of the existing building, near the existing sludge pumps or possibly in the sludge truckway.

- The alkalinity entering the plant is sufficient to allow for nitrification and denitrification as long as Poly aluminum chloride (PAC) is used for chemical phosphorus removal.

  - This should be verified if on-site treatment is selected. It is noted that most plants in Connecticut that nitrify and denitrify do not require alkalinity addition. Coventry is currently in the process of collecting and analyzing samples to confirm this assumption.
  - Based on the alkalinity requirements for nitrification and denitrification, and the expected plant loadings, it is estimated that the influent wastewater alkalinity would need to be ~ 150 to 180 mg/l as $CaCO_3$.
  - If alkalinity addition is needed, raising the alkalinity by 30 mg/l would require approximately 10 gallons per day of 25% sodium hydroxide solution which could be stored outdoors.

- Disinfection would be seasonal, in the form of an open channel ultraviolet (UV) system, to be installed outdoors under a canopy or in a FRP building. This will either be an in-concrete channel or in-pipe fabricated channel installation, depending on the hydraulic grades. This approach helps to keep the treatment area compact and reduces the need for more on-site chemicals.

- An Effluent Flow meter would be installed either before or after the UV system to monitor flows for permit compliance and UV flow pacing. The type and location of the flow meter will depend on the hydraulics of the selected secondary treatment technology. If the process flow is pumped up, it could be a magnetic flow meter. If flow is by gravity, then a Parshall flume would likely be feasible.

- A new Plant Water System will be installed to meet the required process water demands at the plant. The major components will be located in the lower level of the existing building near the existing sludge pumps. The system will deliver disinfected plant effluent and consist of the following components:

  - Two skid-mounted VFD powered pumps (one duty one standby) with pressure control capable of delivering 10 to 40 gpm continuously at a pressure of 70 psi. Water requirements for the plant are expected to be as follows:
    - 11 gpm @ 60 psi for spray water when running the Solids Handling System's RDT. The RDT will have a booster pump to increase the pressure to 100 psi.
    - 5 gpm @ 60 psi for mechanical screening
    - Up to 40 gpm for tank cleaning and pipe flushing @ 70 psi (10 psi higher than the existing potable/well water based plant water system).

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    - Additional assumptions are as follows:
        - Low pressure sump pumps drawing water from the launder could be used for scum sprays in the center wells of any proposed clarifiers.
        - A portion of the tank cleaning and pipe flushing flow budgeted above could be used for manually set-up scum sprays in SBRs or Aeration Tanks if needed.
        - Protected potable water will be used for making up polymer when running the RDT. This could be provided though the backflow preventor connected to the existing potable water system. This approach is generally acceptable for connections to chemical feed systems but is subject to local Health Department Review.
    - A hydropneumatic tank to allow the pumps to shut off when demand is low and hold the pressure steady when usage changes.
    - Additional plant water yard hydrants would be added near the secondary processes to facilitate tank cleaning.
    - A disinfected effluent flow-through storage tank. After disinfection, a chamber would be installed to store effluent and be used by a plant water system. A storage volume of 1,000 gallons should be sufficient to provide the anticipated maximum demand of the plant water system for up to 25 minutes. This chamber likely to be a precast manhole structure 6 feet deep and 6 foot in diameter.
- The new discharge to the river would be at the far (southeast) end of the plant beyond the RIBs. It is expected that the existing 16″ ductile iron RIB distribution pipe could be repurposed as an effluent pipe and lined to seal off the pipes that currently distribute effluent to each RIB. The distribution pipe would send the treated effluent flow to the RIBs while the plant is being started up. Temporary bypass pumping and piping would be required for a short period after startup occurs to allow for the lining. A new 16″ outfall pipe would be constructed (cut and cover) from the endpoint of the existing distribution pipe south past the southernmost RIB. The outfall would then turn towards the river and be installed through the current embankment to either the edge of the river, or within the existing river channel. Specifics of how and where the discharge terminates would be determined during the design phase after input is obtained from the CTDEEP. Justification for this approach is based upon the following:
    - The capacity of the pipe is adequate. With a slope of 0.24%, the full pipe capacity is 1,700 gpm. At 650 gpm, the depth of flow in the pipe would be less than 7 inches.
    - This approach minimizes the risk of excavating between and/or adjacent to the RIBs while they are in service.
    - The length of time and cost of the bypass would be low, limited to a few weeks until the RIBs drain, and the pipe is lined/modified.
    - The treatment plant hydraulics can fit in between the existing hydraulic grade of the aerated grit chamber (weir elevation 296.17) and the RIB distribution pipe estimated backwater elevation (288.2 = 287.58 invert + ~7″) at distribution box #1 near the SW corner of RIB #1. This represents

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approximately 8 feet of head loss across the entire secondary system unless pumping is included in the alternative. This appears doable for a gravity system with up to three processes – Aeration Tank, Clarifier, UV Disinfection but will require careful coordination and evaluation during design phase.

- The location of the effluent pipe through the embankment is as recommended in the report entitled "Slope Evaluation Adjacent to Infiltration Basin No. 2" dated August 2023 Revised to 10-09-2034" prepared by Karl F. Acimovic, P.E. & L.S. that was provided by the Coventry WPCF staff.

- The plant's existing electrical systems would be upgraded with a new motor control center (MCC) to power all new equipment. The utility has sufficient power at the pole adjacent to the building. If the vendor proposed that their scope included VFDs, it would likely be negotiated out of their scope so it could be located in the MCC and furnished by the electrical contractor. The new MCC would be located in the existing (unused) solids handling room adjacent to the existing control room/office. This will provide additional space in the control room/office for a new control panel and eliminate the current code violations for the electrical equipment.

- A SCADA system would be provided and used to monitor and adjust the control of the plant. Specific components would include the following:
  - A new control panel to operate the influent pumps, with an operator interface terminal.
  - The Secondary Treatment Systems Vendor Control Panel, with an operator interface terminal.
  - A Rotary Drum Thickener Control panel to monitor the sludge tank level and control the RDT - with an operator interface terminal.
  - A mission based telemetry system to allow operators to remotely monitor the status of the plant and alarms in the main control panel.

- Space in the existing building will be identified for the needed lab equipment to be used for process control This could be in the existing control room/office or possibly in the existing generator room if a larger outdoor generator is provided in the upgrade.

6.5.2 Upgraded Plant Classification

The CT DEEP utilizes a point and scoring system to determine the classification of a wastewater treatment facility. Points are assigned based upon the proposed flows and treatment processes. The scoring of an upgraded Coventry plant was reviewed with the CT DEEP on November 11, 2023, and a copy of the scoring sheet is included in Appendix D. Based on the agreed upon scoring, it is not possible to keep the plant as a Class II facility. The plant, if upgraded, would become a Class 3 plant regardless of the selection of the short listed secondary technology.

6.6 Review of SBR Responses

Three proposals for SBRs were received. Specific details of each are as follows:

<u>Aqua Aerobics:</u>

- 3 Rectangular SBR Basins (total volume of 0.694 MG)

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- 1 Post Equalization Tank (usable volume of 0.030 MG)
- Total Cost: $917,000.

Aqua Aerobics Alternate:

- 2 Rectangular SBR Basin Design (usable volume of 0.680 MG)
- 1 Post Equalization Tank (usable volume of 0.070 MG)
- Total Cost: $790,000

Evoqua/Jet Tech Omnipac:

- Circular donut design with 3 SBR Basins, 1 Post Equalization tank and inner sludge digester for a total cost of $3,400,000 (which includes the steel tank).
- This SBR design was proposed to operate at a much lower Mixed Liquor Suspended Solids concentration (~1/2) and much smaller (2/3) reactor volume than that of Aqua Aerobic's 3 Basin proposal. It also came with a recommendation to add disc filters on the effluent.

For the purposes of evaluating the SBR technology feasibility, it was determined that the Aqua Aerobics 3 basin proposal would be evaluated further, noting that this proposal was the only one consistent with the vendor RFP and plan to evaluate the SBR technology using field constructed concrete tanks. This vendor also has more experience in Connecticut with SBRs. If on-site treatment with three SBRs is found to be the recommended approach and the project moves forward, the other alternatives could be considered as a value engineering alternative during the formal design phase of the project.

The layout and hydraulic profile of this alternative is provided in Figures 6-1 and 6-2. The annotated Aqua proposal is included in Appendix E. The following summary also help provide the level of complexity of the system:

Aqua Aerobics' 3-Basin Proposal includes the following specific components:

- 3 Motorized influent valves to allow flow to be directed to one SBR at a time
- 3 7.5-HP SBR Mixers with mooring equipment (to mix each reactor)
- 3 SBR Decanters (to draw clarified effluent from the reactor to the post EQ tank)
- 3 2.4-HP SBR submersible pumps (to pump waste sludge from each reactor)
- 6 retrievable Fine bubble diffuser assemblies (2 per SBR reactor to provide oxygen to the process)
- 3 50-HP Blowers requiring indoor installation with isolation and check valves (to supply air to the diffusers)
- 3 3-HP Post EQ Tank submersible Pumps with related valves (to pump effluent to the UV system)
- 1 Post EQ Fixed Fine Bubble diffuser assembly (to keep the tank mixed and prevent solids build up)
- 1 5-HP Post EQ Blower requiring indoor installation with isolation and check valves. To provide air to the Post Eq diffusers, 2 are recommended.

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FIGURE 6-1
EXISTING TREATMENT PLANT SITE- SBR LAYOUT

LEGEND

Approximate Parcel Boundary

CT Municipal Boundary

LOCUS MAP

N

0 20 40

Feet

1:480

NOTES

1. Based on 2019 Statewide Orthophotography, Courtesy of CTECO.

Coventry WPCF
Coventry, Connecticut

December 2023

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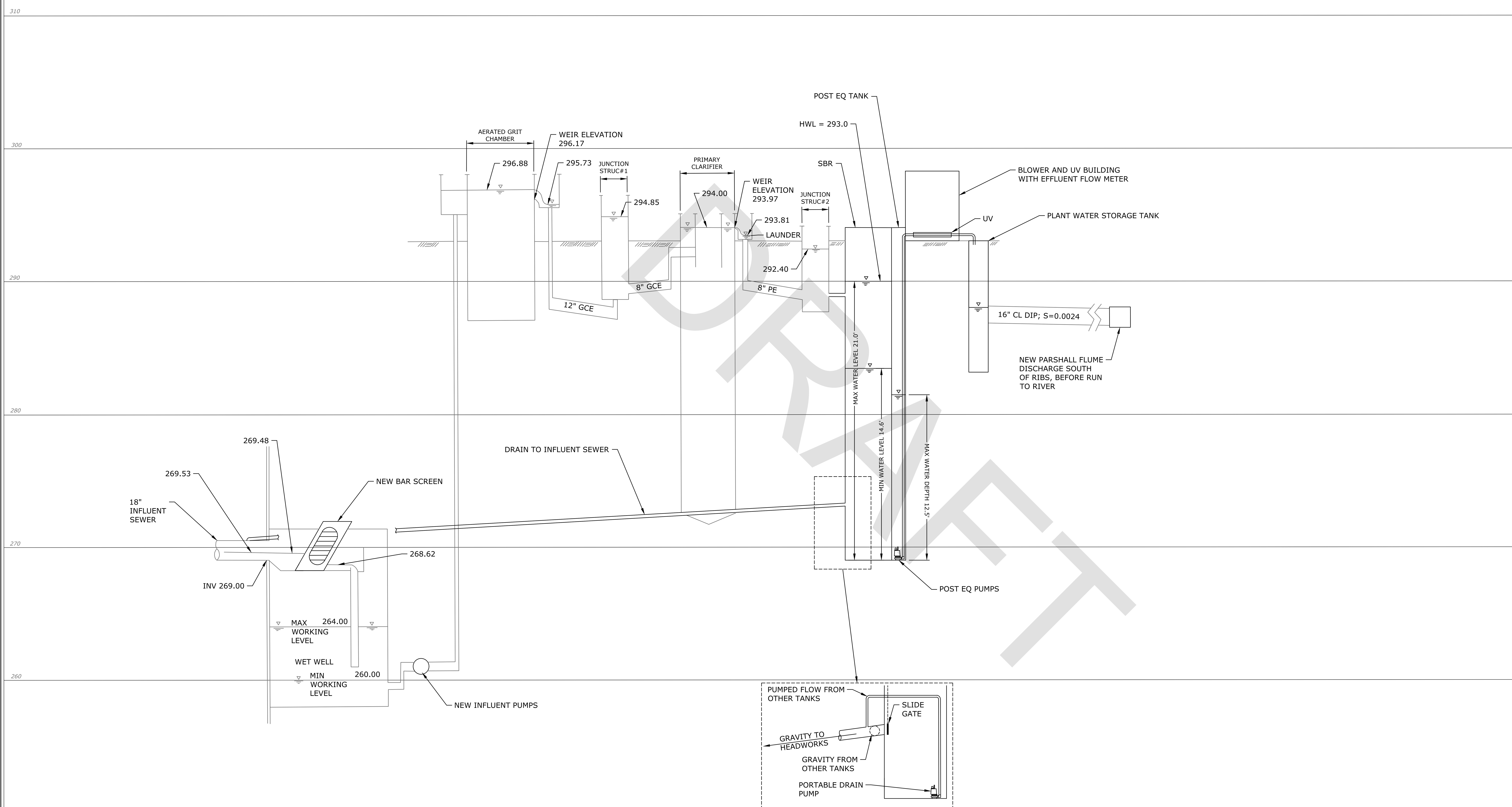

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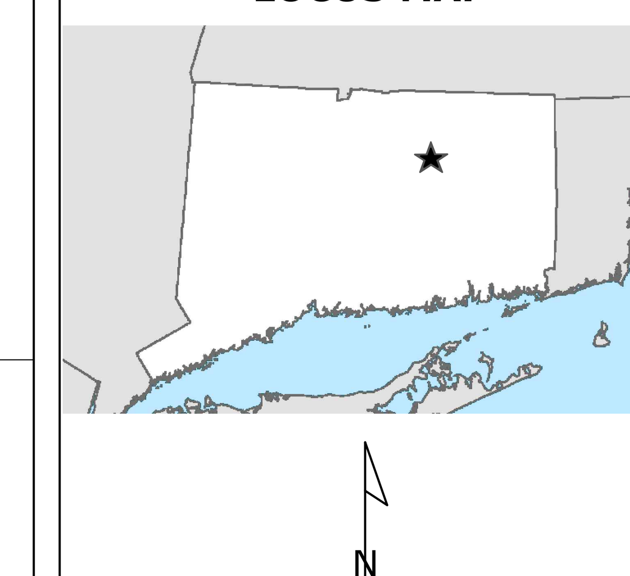
Figure 6-2
SBR Layout
Hydraulic Profile

LEGEND

LOCUS MAP



NO SCALE

NOTES

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Coventry, Connecticut

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C-5080

- Instrumentation Package (with mounting equipment for the 3 SBRs and 1 Post EQ Tank)
  - 4 level sensors
  - 4 float switches
  - 3 Dissolved Oxygen sensors & display.
- System Control Panel – Allen Bradley Compactlogix PLC with Operator interface
- Engineering and Startup Assistance

Aqua's Proposal assumes that the following is to be provided by others, some of which is illustrated in the referenced figures:

- Concrete Tanks including SBR perimeter walkways, stairs, and safety railings as required.
- Portable Hoists to pull the pumps. It is assumed that pulling the mixers and decanters will require riggers with cranes)
- Interconnecting Piping (external to reactors – three influent lines, one effluent line to UV disinfection, WAS piping and scum piping if used)
- Scum collectors or sprays to manage scum (if used)
- Equipment for draining the SBR and Post EQ tanks by gravity to the headworks and related piping. This should not be required often as fine bubble diffusers are retrievable for maintenance, but it will allow all or a portion of the MLSS in the tank be returned to the operating SBRs while rescreening the tank contents. Due to the deep depth of the tanks, it was assumed that the following equipment would be required:
  - 4 slide gates (1 per SBR and 1 for Post EQ tank) to drain most of the tank by gravity.
  - 1 portable submersible pump (with 3 discharge pipes and slide rails) and related piping to fully drain the 3 SBR tanks.
  - It was assumed draining the post EQ tank, which would likely only require a short period of time, would be performed by first partially draining one SBR, then diverting flow to the empty SBR or the clarigesters. The post EQ tank pump would be used to drain it to the extent possible.
- Motor Starters/VFDs for all equipment.
- Structures to house Blowers. A FRP building was assumed.
- Installation.

Based on a review of the proposal and Aqua Aerobic's Assumptions (see annotations in the Appendix), it was assumed that their proposed cost would increase by 15% when the final scope is adjusted to formal required design conditions.

A majority of the required equipment for this alternative will be located outdoors with the possible exceptions of the following:

- Influent Valves – to be located in a heated above grade enclosure.
- 3 Aeration Blowers/motorized valves and Post EQ blowers – to be located on a concrete pad outdoors within a covered enclosure.

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- Electrical Equipment (Motor Starters/VFDs) and System Control Panel – to be located within the existing building.

A discussion of the pertinent design details for the SBRs follows:

- The proposed design was based on an solids retention time (SRT) of 22.6 days and MLSS of 4,000 mg/l at future maximum month flows and loads with an estimated total sludge yield of 0.964 lbs WAS per lb of BOD removed, the process when operating at design capacity is expected to produce 7,900 gallons per day of waste activated sludge at ~1% solids or about 660 dry pound per day. At current flows and loads, the MLSS would likely be lower and waste activated sludge thinner. Sludge mass/volumes will be higher if alum is added to aid in phosphorus removal.

- The design appears to assume complete biological phosphorus removal (does not mention chemical addition) however the sludge yield does appear to be slightly elevated. Backup phosphorus removal might require a slight increase in tank capacity.

- Other than specifying the total cycle and decant times of each SBR at 6 Hours and 75 minutes respectively, the proposal did not describe how the cycle times will be allocated. It is assumed that with three reactors, each cycle will include 2 hours of fill (including anoxic and anaerobic times and aerated time), ~2+ hours of react (with aeration cycling on an off), ~0.5+/- hours of settle (allowing solids to drop to bottom of tank and a portion of the solids wasted), and ~ 1.25 hours of decant (clear water is drawn from surface of reactor) time.

- The Control panel will be responsible for opening and closing inlet valves to the SBR tanks, valves that control the decanting as well as controlling the decant and waste pumps on an hourly basis, modifying the control sequence during high flows, and starting and stopping the mixers and blowers. Like all alternatives, the control panel will also be responsible for maintaining suitable oxygen levels in the process by monitoring the dissolved oxygen sensor and controlling the blower's speed and/or on-time. In the event of a control system (the PLC, many automated valves, and level sensors) failure, the process will need to be operated manually on an hourly basis (assuming sufficient staff are available to do so) until it can be repaired.

- The SBR has an aerated post equalization tank. It is assumed that this would be sufficient to raise the effluent dissolved oxygen to at least 5 mg/l prior to the discharge to the river per DEEP requirements. This is before considering oxygen transfer that will also occur at downstream UV weirs or gravity piping to the river.

- With an SBR, having a thickened sludge storage tank is required because the sludge must be wasted based on the time schedule of the SBR.

- Energy costs were projected by the equipment vendor at 571 KWH/day at future maximum month conditions. The manufacturer did not assume any additional solids loading due to chemical additions for phosphorus removal.

- While reviewing the plant classification criteria with DEEP as discussed above, DEEP raised the following concerns about SBRs as proposed.

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- DEEP had concerns that some SBR plants in CT (e.g. Shelton) don't perform well due to lack of grease removal (e.g lack of primary clarification). Scum removal from SBRs is a challenge because of the varying tank levels. To address this concern, the following additional assumptions were made:
  - The clarigesters would be refurbished with new mechanisms and reused for primary clarification.
  - Primary effluent will flow by gravity to the SBR basins through the existing 12" pipe.
  - The SBR's reactor size would remain unchanged (assuming the clarigester's BOD removal balance added peak flow to the SBRs.
  - The sludge production and disposal requirements from the SBRs would be reduced by 15% from the values discussed above.
  - Primary sludge would still be generated and managed as it is currently.
  - The SBR would need to be installed 2 feet deeper and be two feet taller (increasing excavation and concrete costs) in order to fit within the existing hydraulic profile and keep the top of the SBR consistent with existing nearby grades.
- They had concerns that SBRs may not be able to reach a lower effluent total nitrogen limit of 3 mg/l. However, the decision was made to move the evaluation forward based upon the 6 mg/l discharge limit.

6.7 Review of Oxidation Ditch Responses

Proposals were requested from two vendors, but only one proposal was received for oxidation ditches. Specific details are as follows:

Ovivo Carrousel System

- 2 oxidation ditches with anaerobic and anoxic tanks (total volume 0.446 MG)
- 2 35-foot diameter spiral blade clarifiers (usable volume of 0.173 MG)
- Total Cost: $1,520,000

Evoqua Orbal System – no proposal received.

The layout and hydraulic profile of the Ovivo system is provided in Figures 6-3 and 6-4, and their annotated proposal is included in Appendix E. The following summary also help provide the level of complexity of the system:

Ovivo's Carrousel System proposal includes the following specific components:

- 2 25-HP Excell Aerators with VFDs to mix and deliver oxygen to the process
- 10 2-HP Submersible Mixers: 6 for the anaerobic zones and 4 for the anoxic zones
- 2 "EliminatIR" motorized gates to control the internal recycle flow of nitrate into the anoxic zone based upon instrument readings
- Instrumentation Package with mounting equipment for the 2 trains consist of:
  - 2 Dissolved Oxygen sensors & display
  - 2 ORP Probes
- System Control Panel – Allen Bradley Compactlogix PLC with Operator interface

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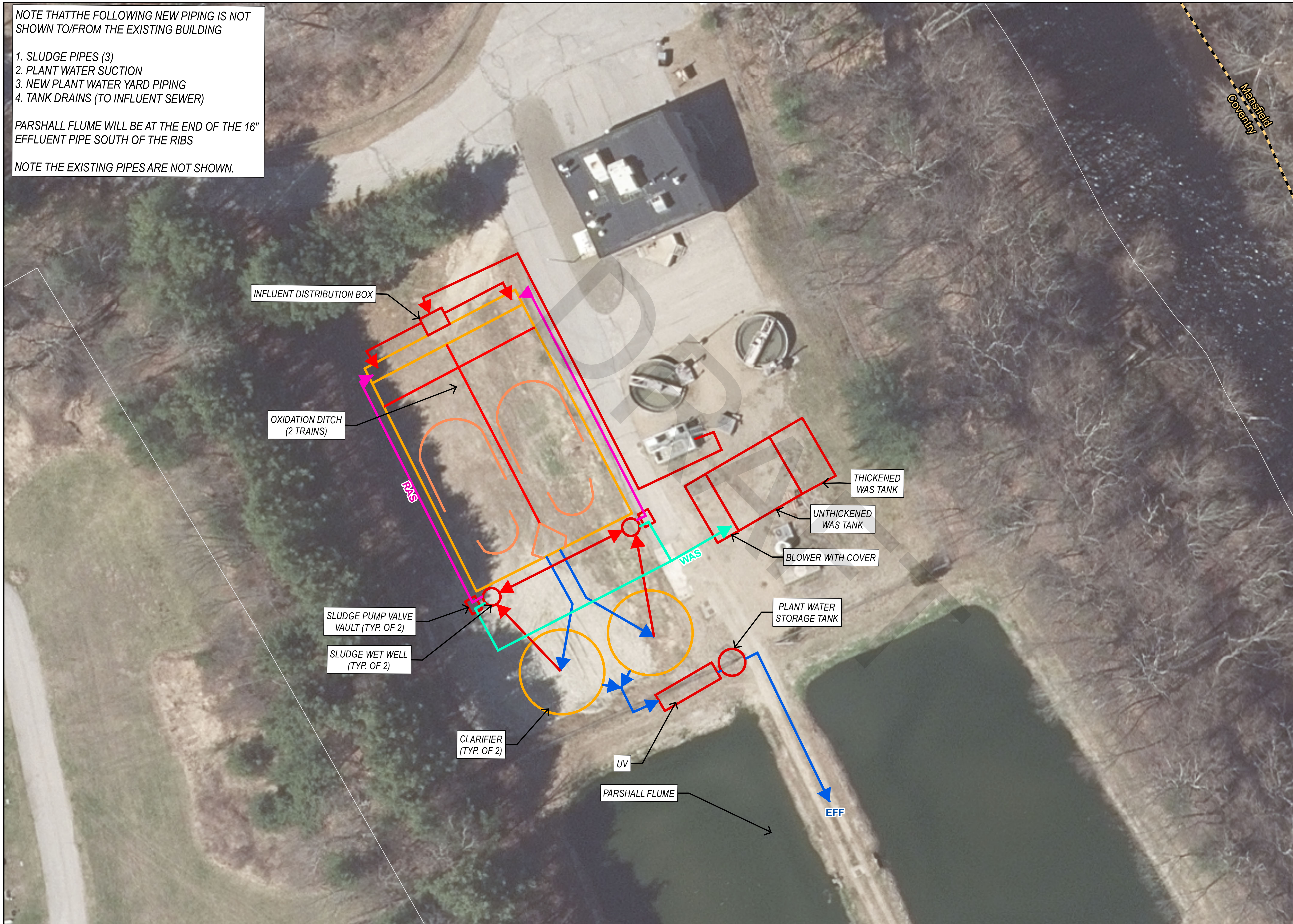

FIGURE 6-3 EXISTING TREATMENT PLANT SITE- OXIDATION DITCH LAYOUT

LEGEND

Approximate Parcel Boundary

CT Municipal Boundary

LOCUS MAP

N

0 20 40

Feet

1:480

NOTES

1. Based on 2019 Statewide Orthophotography, Courtesy of CTECO.

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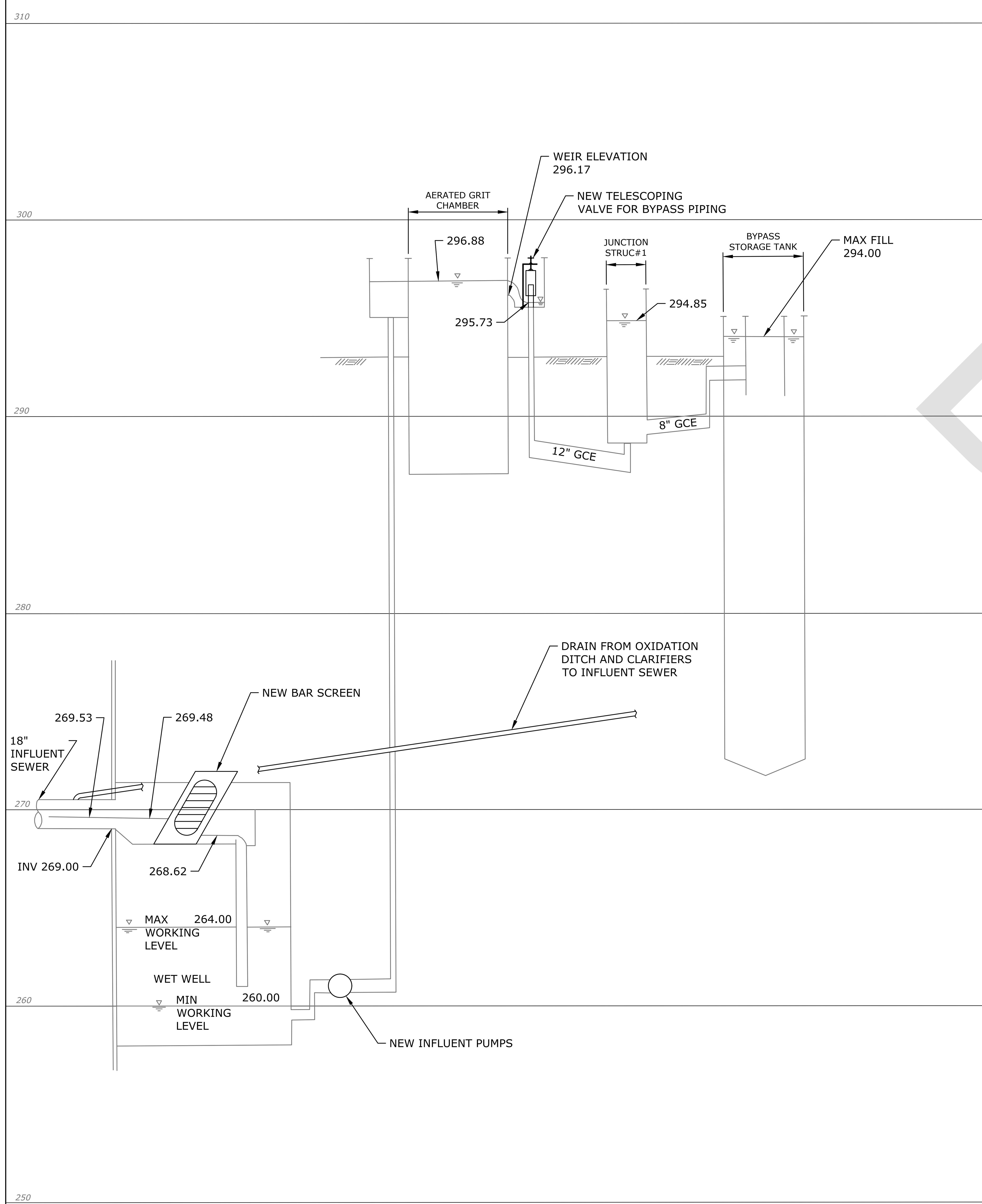


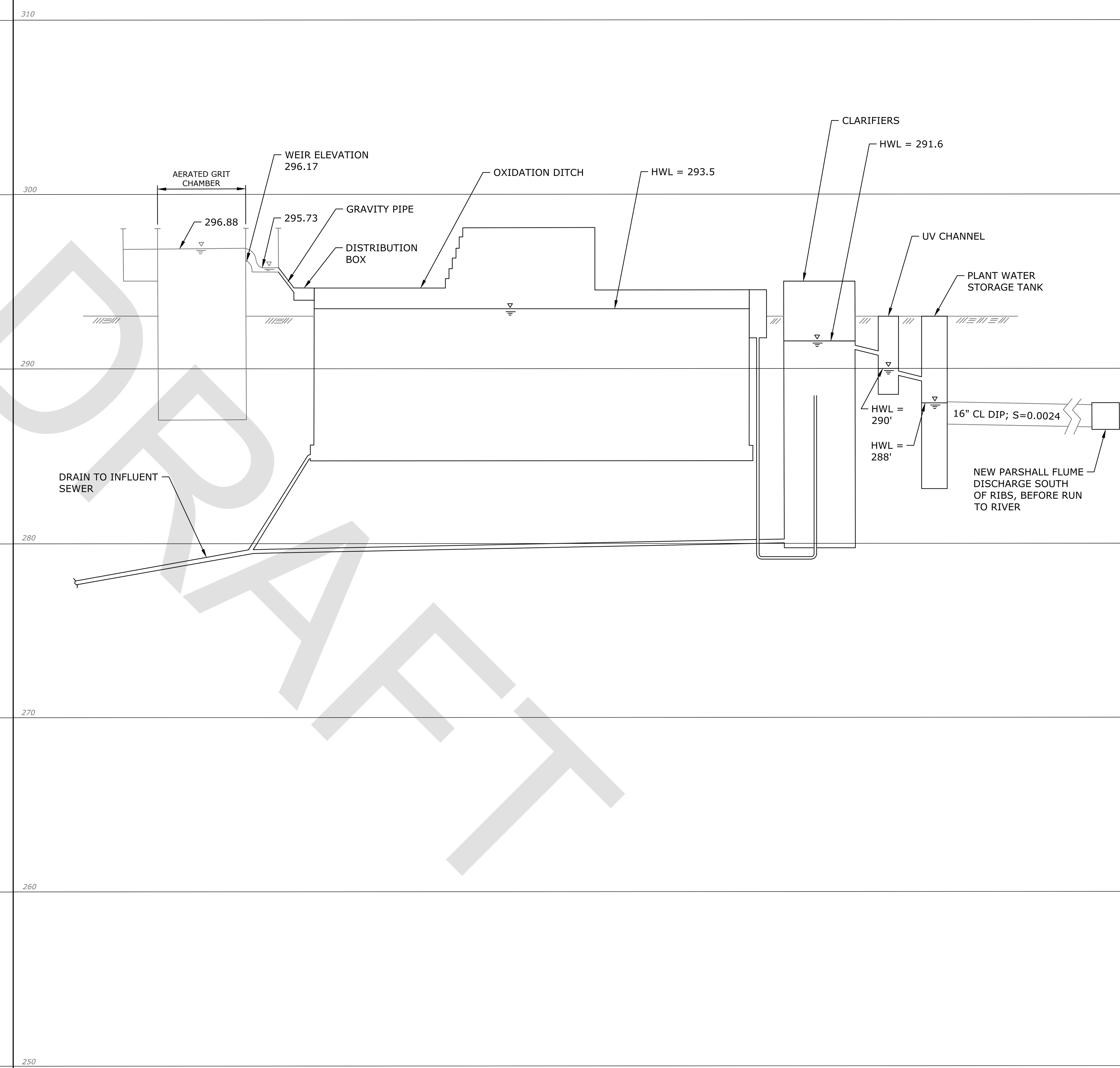
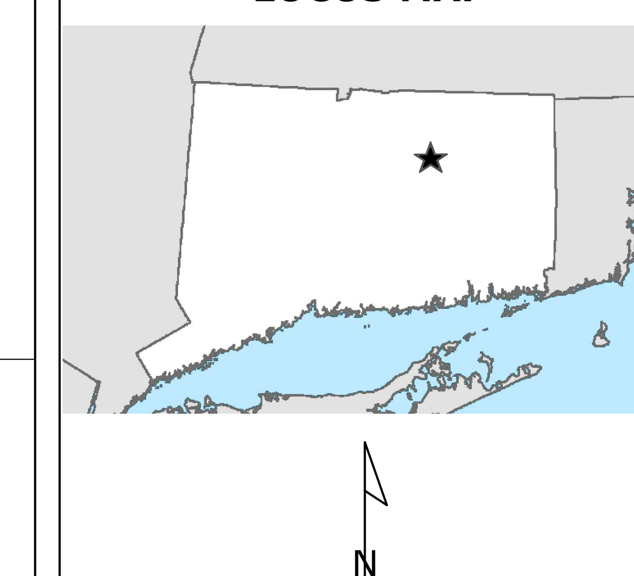


Figure 6-4
Oxidation Ditch
Hydraulic Profile

LEGEND

LOCUS MAP



NO SCALE

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- 2 Spiral Blade Clarifier Mechanisms (Painted Carbon Steel) with Control Panel, Weirs and Baffles, walkway truss with handrail.
- Engineering and Startup Assistance

Ovivo’s Carrousel System proposal assumes that the following is provided by others:

- Influent Splitter Box with 4 manually operated isolation gates. This includes 2 extra gates that were added to allow for step feed during high flow conditions.
- Oxidation Ditch and related tank concrete & guardrails
- 2 8-foot effluent manual weir gates
- Mixer in the distribution box for the clarifiers
- 2 18” manual isolation slide gates in the distribution box for the clarifiers
- Clarifier Concrete, including the launders
- Clarifier perimeter guardrails
- Clarifier accessories:
    - FRP current density baffles
    - FRP Launder Covers to control algae growth
- One precast 6' diameter scum pit to handle scum from both clarifiers that could be manually decanted to the head of the plant along with concentrated scum that is removed periodically by a vacuum truck or manually pumped over to the sludge tank was assumed.
- Sprays commonly used to break up scum at the inner well estimated to provide 5-10 gpm per clarifier with a sump pump in the effluent launder will supply water at low pressure.
- Return & Waste Sludge Pumps and Structures to house them (these are required to return sludge from the clarifiers to the oxidation ditch and send waste sludge to the thickened sludge storage tank). This includes 4 300-gpm pumps (estimated at 7.5 HP), and related valves. For now, it is assumed that two 8’ diameter wet wells as deep as the clarifiers with 2 submersible pumps in each and an adjacent valve vault. If this alternative is selected, consideration should be also given to locating non-submersible pumps and related valves and flow meters in an accessible vault designed for human occupancy.
- 3 Flow meters:
    - 2 RAS (magnetic)
    - 1 WAS (magnetic)
    - 1 Effluent Parshall flume after the UV for permit compliance and UV flow pacing. In this case, the Parshall flume will be installed close to the end of effluent line in order to get enough hydraulic difference.
- Interconnecting Piping external to the oxidation ditches & clarifiers. These include influent and effluent lines, RAS & WAS lines, scum piping, and one effluent line to the UV disinfection system.
- 4 slide gates (1 per ditch and 1 per clarifier) for draining the tank by gravity to the headworks.
- Equipment for draining each ditch (and related tanks) and the clarifiers by gravity to the headworks and related piping. This will be required when taking a train or

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clarifier off line because it is not needed or for maintenance and it will allow all or a portion of the MLSS in the tank to be returned to the operating train while rescreening the tank contents. Due to the relative shallow depth of the tanks, it is assumed that all flow would be by gravity and the following equipment would be required:

  - 2 slide gates (1 per ditch)
  - 2 buried gate valves (1 per clarifier from a bottom drain)

- Motor Starters/VFDs for all equipment (except for the 2 50 HP oxidation ditch mixer aerators)
- Installation /field welding.

Based on the review of the proposal and Ovivo's assumptions (see annotations in Appendix E), it was determined that the scope was well defined, with much of the work being done by others. Therefore, it was concluded that their final scope and cost would not need adjustment.

A majority of the required equipment for this alternative will be located outdoors with the possible exception of the following:

- Electrical Equipment including the Motor Starters/VFDs and System Control Panel will be located inside the existing building.
- The RAS pumps could have been constructed inside the basement of a new building. However, for this analysis it was that submersible pumps and vaults would be used to keep costs reasonable. This could be re-considered during the design phase if this technology is selected.

A discussion of the pertinent design details for the Oxidation Ditches follows:

- The proposed design was based on an aerated SRT of 12 days and MLSS of 4,000 mg/l at future maximum month flows and loads with an estimated total sludge yield of 1.10 lbs WAS per Lb of BOD removed, the process is expected to produce 9,400 gallons per day of waste activated sludge at 1% solids or about 782 dry pound per day. The vendor proposal assumed that some level of alum dosing and solids generation is included in these numbers.

- The design was based on biological phosphorus removal but also assumed that supplemental chemical phosphorus removal would be required due to the low P to BOD ratio. As a result, a higher sludge yield coefficient was assumed to reflect the additional solids generated. If this alternative is selected for implementation, the life cycle cost of adding the 6 anaerobic tanks and operating them should be compared to the life cycle cost of the increased sludge disposal costs and likely slightly larger tanks and clarifier required. Chemical would be dosed into the common effluent channel of the two ditches (/clarifier distribution box) and consideration should be given to adding a (gentle) mixer in that location.

- The design sized the aerators so that one train could treat the maximum month load with the other train/aerator out of service, which would require doubling the MLSS. Both trains and aerators would be required to treat the maximum day load.

- The Control Panel will be responsible for controlling the speed of the RAS pumps (which are typically flow paced), and possibly starting and stopping or adjusting the speed anerobic and anoxic mixers (this is not required by easily done useful

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for optimizing operations. Like all alternatives, the control panel will also be responsible for maintaining suitable oxygen levels in the process by monitoring the dissolved oxygen sensor and controlling the aerator/mixers speeds. In the event of a control system failure (the PLC), the process could likely be run in hand with adjustments on a daily basis (more often if wet weather) until it can be repaired with a slight reduction in nutrient removal performance.

- Oxidation Ditches typically operate at low dissolved oxygen levels (1.0 mg/l). In order to raise the effluent dissolved oxygen to at least 5 mg/l prior to the discharge to the river per DEEP requirements, it was conservatively assumed that oxygen transfer at the clarifier weirs and UV weirs would not be sufficient. There may be some oxygen transfer within the gravity piping to the river, however, it is impractical for the operators to monitor this for compliance purposes.

  Therefore, an allowance of $75,000 is included to add a post aeration chamber and the related blowers & diffusers prior to or after the clarifiers. This could also possibly be located in the distribution box after the ditch.

- With an Oxidation Ditch and clarifiers, having a thickened sludge storage tank is preferred but not required because the sludge can be stored in the process tanks (mostly in the clarifier blankets), and there is less concern with under sizing the unthicken sludge tank. It was conservatively assumed that a sludge storage tank would be provided.

- Energy costs were projected by the equipment vendor at 581 KWH/day at future maximum month conditions. This does not include the costs of operating the RAS and WAS Pumps, which would add an estimated 60 KWH/day at maximum month.

- While reviewing the plant classification criteria with DEEP as discussed above, DEEP did not appear concerned about the lack of primary clarifiers or future lower permit limited for this alternative. This is because the secondary clarifiers should be able to handle the grease. Post anoxic and aeration tanks could be added in the future (assuming room is included during the design) to reach a lower effluent Total nitrogen limit of 3 mg/l if a lower limit lower is ever required.

6.8 Review of Membrane Bioreactor (MBR) Responses

Two proposals for MBRs were received. Specific details are as follows:

<u>Veolia ZeeWeed</u>:

- 3 Trains (total volume of 0.270 MG).
- Each train includes a pre-anoxic tank, pre-aeration tank, and post anoxic tank.
- The 3 trains share 2 MBR tanks (which act as the clarifier), each with 4 membrane cassettes containing a total of 168 hollow fiber membrane modules.
- The proposal was unclear on redundancy on how it would handle peak flow with portions of the modules out-of-service.
- Total cost: $1,810,000.

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Kubota:

- 2 Trains (total volume of 0.447 MG).
- Each train includes a pre-anoxic tank, pre-aeration tank, post anoxic tank.
- The 2 trains share 3 MBR tanks (which act as the clarifier), each with 5 submerged flat plate membrane units designed to handle peak flow with one MBR tank out of service.
- Total: cost of $2,391,000.
- Unfortunately, the Kubota system was sized based on maximum daily loads instead of maximum month loads as were all other vendor proposals. Therefore, the decision was made to focus the discussion below on Veolia's Zeeweed proposal and then draw comparisons when appropriate.

The layout and hydraulic profile of the Veolia alternative is provided in Figures 6-5 and 6-6. The annotated proposals are included in Appendix E. The following summary also helps to provide the level of complexity of the MBR system.

Veolia's proposal is presented in a manner that would allow initial construction of 2 trains to handle current flows and loads, and a third train to be constructed in the future to handle future flows and loads if necessary. The built out (3 train) proposal includes the following:

- 6 1.2-HP submersible mixers in the pre-anoxic & post anoxic tanks: 1 per tank per train.
- 3 15-HP RAS Pumps. It is not clear if the pumps are submersible or dry pit that would require a lower level basement in building.
- 3 3-HP Internal Recirculation Pumps.
- 3 10-HP rotary lobe (reversible) Permeate Pumps – each to be skid mounted with related piping, valves, and instrumentation as listed below.
- 3 Sets of fixed fine bubble diffusers assemblies.
- 3 30-HP* Process Blowers (2 duty, 1 standby) to be located indoors.
- 3 20-HP* MBR Blowers (2 duty, 1 Standby) to be located indoors.
- 2 5-HP Air Compressors with Air Dryers & accessories (1 duty 1 standby) to be located indoors.
- 3 Chemical Injection Systems with skid mounted chemical feed pumps (1 duty 1 standby) with related pipe, valves etc.).
    - Alum/Coagulant – For phosphorus removal
    - Micro-C – Carbon source to drive nitrate removal in anoxic zone
    - Sodium Hydroxide– for alkalinity addition
- 2 Chemical Injection Systems with 1 duty chemical feed pump with related pipe, valves, etc.) for 2x a week and annual cleaning of membranes in place.
    - Sodium Hypochlorite
    - Citric Acid
- Instrumentation Package with mounting equipment for all of the process equipment

FIGURE 6-5
EXISTING TREATMENT PLANT SITE- MBR LAYOUT

LEGEND

Approximate Parcel Boundary

CT Municipal Boundary

LOCUS MAP

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Feet
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1. Based on 2019 Statewide Orthophotography, Courtesy of CTECO.

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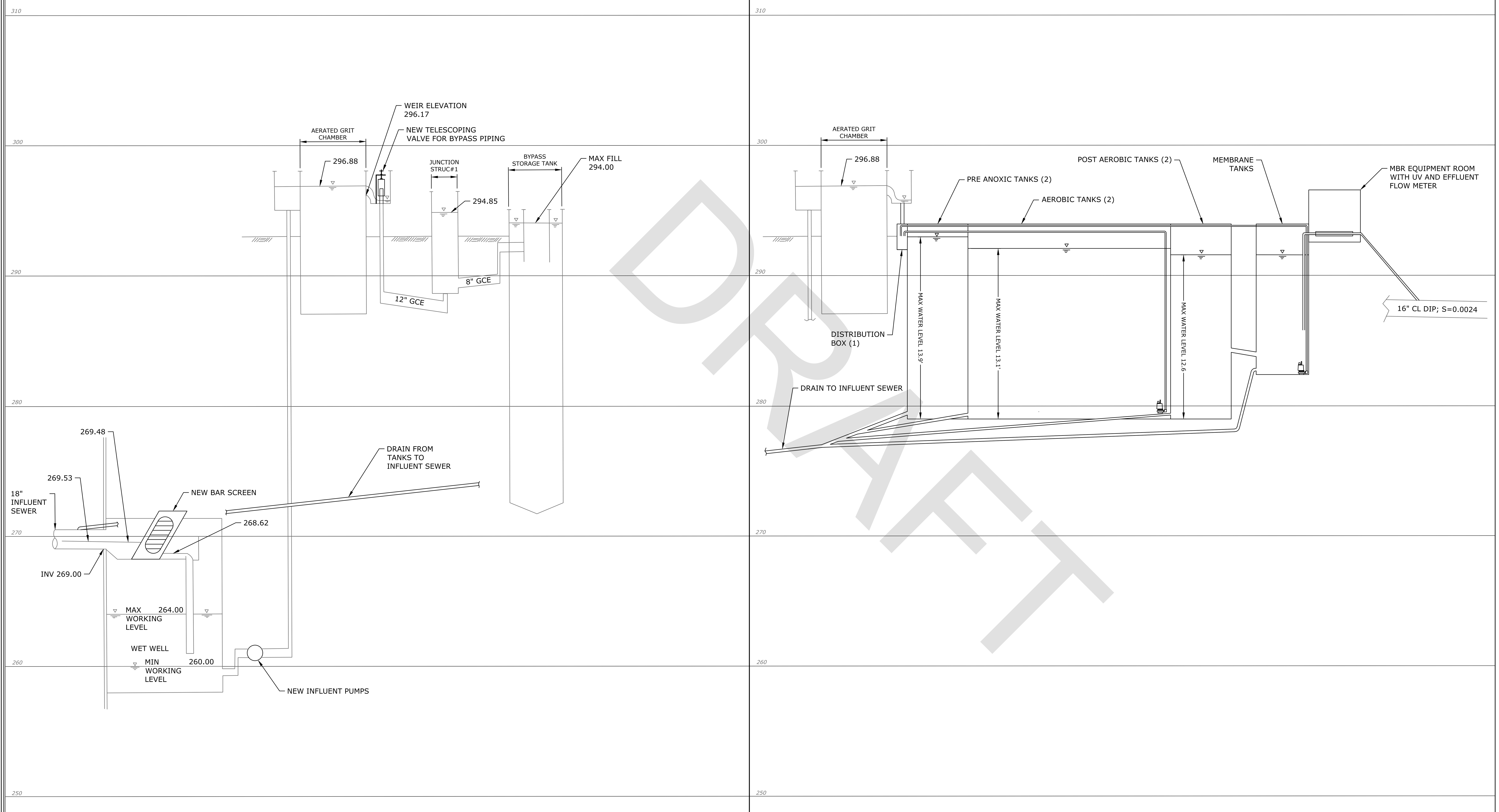
Figure 6-6
MBR Layout
Hydraulic Profile



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LEGEND

LOCUS MAP

NO SCALE

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  - 3 dissolved oxygen sensors.
  - 3 pH Sensors
  - 1 WAS magnetic flow meter
  - 3 RAS magnetic flow meters
  - 1 Temperature transmitter
  - 1 level transmitter – backpulse tank
  - 2 Permeate flow meters
  - 2 Permeate turbidity meters
  - 3 (Permeate) pressure transmitters
  - 5 day tank level switches for chemicals
- System Control Panel – Allen Bradley PLC with Operator interface
- Engineering and Startup Assistance
- 10 Year Membrane Warranty (Prorated after 2 years)

Kubota's proposal was similarly (if not more) complex and did not specify any of the equipment horse powers.

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Veolia's proposal (Kubota's was similar) assumed that the following is to be provided by others:

- Fine (2 mm) Screens would be required for this process. These will not fit in the existing influent channel.
- A flow mixing/splitting structure with a mixer and three weir gates to mix RAS with the incoming flow and then evenly split to flow to the three trains. Consideration should be given to making this part of a new fine screening building.
- Concrete Tanks
- 2000 SF Building to enclose the membrane tank and process equipment.
- Chemical Tanks (for Alum, Micro-C, Sodium Hypochlorite).
- Equipment for draining each train by gravity to the headworks and related piping. This will be required when taking a train off-line when it is not needed or for maintenance and it will allow all or a portion of the MLSS in the tank to be returned to the operating train while rescreening the tank contents. Due to the moderate depth of the tanks, it was assumed that all flow would be by gravity and following equipment would be required:
  - 2 slide gates (one per membrane tank)
  - 3 slide gates (one per train to handle pre-anoxic, oxic and post anoxic)
- Interconnecting piping external to the MBR trains. These include influent and effluent lines, RAS & WAS lines, scum piping, and one effluent line to the UV disinfection system.
- Interconnecting piping internal to the MBR trains. These include RAS piping, internal (nitrate) recycle piping, recycles piping influent and effluent lines, RAS & WAS lines, scum collectors.
- Motor Starters/VFDs for all equipment (except mixer/Aerators)
- Installation

Discussion of the pertinent design details for the MBRs follows:

- A minority of the required equipment for this alternative will be located outdoors. Therefore, a new building will be required to enclose the pieces of equipment listed below, as this equipment may not fit in or be practical to locate in the existing building.
  - Fine Screens
  - Membrane Tanks
  - Chemical Tanks and Injection pumps
  - 4 permeate pumps
  - 3 MBR and 3 Pre Aeration Blowers – Due to the high number of blowers, it does not make sense to locate these units outside. It is also assumed that indoor units are less expensive.
  - Electrical Equipment (Motor Starters/VFDs) and System Control Panel – could be located within the existing building.
- The proposed design was based on an aerated SRT of 13.5 days (HRT= 10.7 Hrs @ MMF), and a MLSS of 8,300 mg/l at future maximum month flows and loads. Although these was some discrepancy in the data presented, they appeared to base the design on an estimated total sludge yield of >1.6 lbs WAS per Lb of BOD removed with the process is expected to produce 15,000 gallons per day of waste activated sludge at 1% solids or about 1,270 dry pounds per day which was assumed to include chemical sludge from Alum addition. It is unclear where the vendors were proposing adding the chemical – but it is likely at the end of the train. It is not clear if PAC could be used instead of Alum for phosphorus removal without impacting the membranes.

- The Control Panel will be responsible for controlling the speed of the RAS and internal recycle pumps (which are typically flow paced), and the speed of the Permeate pumps (based on the tank level). It will also be used to air scour the membranes. Like all alternatives, the control panel will also be responsible for maintaining suitable oxygen levels in the process by monitoring the dissolved oxygen sensor and controlling the aerator/mixers speeds. In the event of a control system (PLC , control valves, level sensors), the process could likely be run in hand with adjustments on a hourly basis (assuming sufficient staff are available to do so) until it can be repaired.

- The MBR has an aerated membrane tank in which air is used to scour the membranes. It was assumed that this would be sufficient to raise the effluent dissolved oxygen to at least 5 mg/l prior to the discharge to the river per DEEP requirements. This is before considering oxygen transfer that will also occur at downstream UV weirs or gravity piping to the river.

- Energy costs were projected by the equipment vendor at 416 KWH/day at current average day conditions. To compare this to other energy consumption estimates done at future maximum month conditions, their estimate was adjusted to 1410 KWH/day at future maximum month conditions. Note that this is about 60% of Kubota's energy costs of 2,255 KWH/day which makes sense as Kubota's design was overdesigned by the same factor.

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- This design assumed that chemical phosphorus removal would still be required, as the design does not include any provisions for anerobic conditions due to the high amount of aeration required for the process.

- While reviewing the plant classification criteria with DEEP as discussed above, it was indicated that, as will be discussed below, MBRs were not being considered further in the evaluation. Therefore, there was no MBR related discussion on scum or future nitrogen permit limits.

6.9 Subjective Criteria Evaluation Matrix

The matrix and ranking system illustrated in Table 6-2 was developed to evaluate the three alternatives based on subjective criteria to allow them to be compared and ranked. A scoring system from 0 to 5 was used, with the lowest score being the best. This factor will be used in Section 8 to confirm the recommended technology.

TABLE 6-2
Treatment Process Alternatives Comparison and Ranking Summary

Criteria	SBRs	Oxidation Ditches	MBRs
Impact on Existing Plant Processes	**Moderate +2** (All fit within RIB #1 near existing process building. All require construction of temporary RIB to allow taking RIB # 1 off line)		
New Buildings	**Not Required +2** Possibly aeration blowers and automatic valves may be protected in confined space)	**Not Required +0** (if Submersible RAS Pumps are used)	**Required +5**
Odor Potential	**Low +3** (Flow into raw wastewater in SBR's has an air gap)	**Low +2** (Submerged inlet)	**Low +2** (Submerged inlet)
Capability of meeting possible future regulated contaminants	**Low +3** (Post EQ pumps could be replaced with high head pumps if needed to pump to add on process)	**Low +4**	**Low +4**
Process Complexity	**Medium +4**	**Low +0**	**High +5**
Maintenance	**Medium-High +3** (3 Floating Decanters, 3 Floating Mixers, 6 retrievable/1 fixed aeration grids, 3 WAS pumps. 4+ Blowers, 2 Post EQ pumps, many instruments)	**Medium +1** (10 submersible mixers, 2 aerators, 4 RAS/WAS pumps, 2 clarifiers)	**High +5** (Membranes are maintenance intensive)
Reliability	**Medium +3** (Many components to be controlled on hourly basis)	**High +0** (Can run in hand)	**Low +5** (Membranes subject to fouling, Fine screens, multiple auto controls)
Total Ranking Score	20	9	28
Relative "Subjective Cost" Score	20/20 = 1.00	9/20 = 0.45	N/A - Eliminated

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After reviewing both MBR proposals and performing the subjective ranking criteria presented above, the evaluation of MBRs was eliminated from moving forward into the life cycle cost analysis for the following reasons:

- The one main advantage of MBR technology is that it can solve the problem of fitting into sites with limited available space. Based on the layouts of all three alternatives evaluated, this is not an advantage for Coventry as the other alternatives fit easily in the available space.
- The MBR process has high levels of operating costs for energy, screenings disposal, cleaning activities associated with the membranes, and the replacement of membranes which have a limited life.
- The process requires 2 mm fine screens which would likely require its own building after prescreening and the influent pump station to pass the required flow. This alternative will also generate significantly more screenings than the existing screens which will drive up disposal costs.
- The process has significantly more controls and instrumentation than other processes being considered and is therefore much more complex to operate and maintain.
- The presence of membranes may reduce flexibility in selecting an alternative coagulant if the plant were to receive an aluminum limit in the future.

6.10 Staffing Impacts

A staffing analysis for an upgraded Coventry WPCF was developed using the guidance document developed by the New England Interstate Water Pollution Control Commission (NEIWPCC) titled "The Northeast Guide for Estimating Staffing at Publicly and Privately Owned Wastewater Treatment Plants," dated November 2008. NEIWPCC developed this guidance document to update the 1973 EPA staffing guide titled "Estimating Staffing for Municipal Wastewater Treatment Facilities." Wastewater treatment has seen many changes since the publication of EPA's 1973 document demonstrating the need for an up-to-date guide such as NEIWPCC's document. Treatment processes, technologies, control techniques, residual handling and terminology are all examples of changes that have occurred in wastewater treatment since 1973. These changes and others are reflected in NEIWPCC's staffing guide.

The NEIWPCC staffing guide was developed through surveys and pilot studies of plants located throughout New England and New York. The survey results were used to make the staffing estimate tables included in the guidance document. A review was conducted with a substantial group of experienced superintendents, regulators, and consultants drawn from throughout New England to affirm consensus with respect to approach and to confirm that the guidance documents yielded accurate results for estimating staffing at wastewater treatment facilities.

NEIWPCC developed seven charts that help estimate staffing requirements with the following titles:

1. Basic and Advanced Operations and Processes
2. Maintenance
3. Laboratory Operations
4. Biosolids/Sludge Handling

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5. Yardwork
6. Automation/SCADA
7. Considerations for Additional Plant Staffing

Within these charts NEIWPCC lists various processes, tasks, and activities that are commonly carried out at wastewater treatment facilities. An estimate is given of how many hours a day are required by an employee to conduct these various activities. The amount of time required is directly related to the design flow of the treatment plant.

Charts 1 through 5 were used to establish an estimate of the daily hours required to operate and maintain the Coventry WPCF. Charts 6 and 7 do not give a numerical value of hours but rather give an insight into the extent of automation at the WPCF as well as additional staffing considerations. The daily hours corresponding to Charts 1 through 5 were translated into annual hours required based on the number of shifts operated at the WPCF.

The current Coventry WPCF falls under NEIWPCC's "One Shift Plant" category, which is any facility that has one shift a day, five days a week. When using the "One Shift Plant" category, the daily hour estimates for a given task in Charts 1 through 5 were multiplied by 260 to get annual hours. NEIWPCC then recommends dividing the estimated annual hours by 1,500 hours per year (EPA's estimate of the number of hours worked by a single employee a year) to estimate the number of employees required. The number of hours worked a year by an employee assumes a 5-day work week, 29 days of vacation, sick leave, holidays, and 6.5 hours per day of productive work.

Design flows, flow diagrams, unit processes, and other information pertaining to the existing Coventry WPCF were used by Tighe & Bond when filling out these charts to establish a baseline staffing estimate. Adjustments to the baseline analysis were then made based on NEIWPCC Charts 6 and 7 as well as information obtained during our review of the WPCF. The results of the treatment personnel staffing requirements are included in the following sections.

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6.10.1 Existing Staffing Requirements

Analysis of staffing requirements for the current Coventry WPCF in accordance with the NEIWPCC guidance document's charts 1 through 5 indicates a need for approximately 1.2 operation/maintenance staff. This staffing guide includes only the plant and not the collection system and pump stations, which explains in part why this is less than the current staffing level of 2 personnel.

The NEIWPCC guidance document recognizes that staff estimating is not an exact science. To assist with this analysis, the NEIWPCC guidance document includes Chart 6 (automation/SCADA) and Chart 7 (additional considerations for plant staffing) which do not give numerical staffing estimates but rather point out areas where consideration should be given when refining the baseline estimate from Charts 1 through 5. When completing the analysis, approximations were made for processes at the WPCF that were not listed. For example, with no RIBs listed in the chart, sand filters were selected.

6.10.2 Proposed Upgrade Staffing Requirements

The proposed upgrade of the Coventry WPCF will include a significant increase in the number of processes and equipment that must be monitored and maintained, as well as lab analyses and reporting. This necessitates the need for an updated staffing analysis which includes determining if the plant will remain in the "One Shift Plant" category. The next possibly applicable alternative is "One Shift Plus Plant" which is staffed more than one shift five days a week. This could mean one person working a 4 hr shift on weekends and holidays. For the purpose of this analysis, it was assumed that the plant would remain a "One Shift Plant".

Analysis of staffing requirements for the upgraded Coventry WPCF in accordance with the NEIWPCC guidance document's charts 1 through 5 indicated a need for the following staffing requirements at current flows:

- SBR: 2.6: (less existing 1.2) suggesting adding 1.4 staff
- Oxidation Ditch: 2.4 (less existing 1.2) suggesting adding 1.2 staff

6.10.3 Staffing Conclusions

Assuming the WPCF is upgraded, the current WPCF staffing levels would need to be increased by at least 1 possibly 2 personnel (more likely if an SBR was selected). The NEIWPCC reports for each of the scenarios discussed above are included in Appendix F To account for this in the cost evaluation for the two alternatives, added staffing levels of 1.2 and 1.4 staff at $80,000 per year for the OD and SBR alternatives respectively were assumed.

6.11 Cost Evaluation

6.11.1 Construction Costs

Construction costs for each treatment plant were developed based on the following costs and assumptions:

- Each treatment plant option would require the construction of a temporary RIB followed by decommissioning of all RIBs. A discussion of RIB decommissioning is presented in Section 7.11.
- Excavation, dewatering, and backfill for all new treatment plant components. Allowances for sheeting were included for tanks .
- Demolition of existing equipment, piping and valves in the influent pump room, lower level, and solids handling room following by installation of new equipment/piping/valves as applicable.
- New influent mechanical bar screen, slide gates, plus an allowance for a future washer/compacter.
- Three new dry pit submersible influent pumps.
- Secondary treatment equipment specific to the SBR and Oxidation Ditch technology with specific components as discussed above.
- New spray system for the SBR option
- Solids Handling system (RDT, pumps, blowers, polymer and odor control) as discussed above.
- UV Disinfection
- New plant water system

- Chemical feed systems for phosphorus removal and sludge handling including provisions for a safety shower and eyewash station.
- Structural improvements including process tanks, equipment pads, ladders, railings, handrails, hoists and pipe supports.
- Building improvements including new roof, exterior stair access new door, and exterior repairs
- Painting of generator room walls and coating of concrete floors in truckway
- Electrical improvements including new generator, new electric utility service where applicable, new Main Circuit Breaker, Distribution Panelboard, replacement of the MCC and a new Lighting Panelboard
- LED Lighting Upgrades.
- VFDs
- Effluent flow metering
- Headworks and secondary process control systems with Mission Monitoring.

Equipment costs are based upon quotes obtained from manufacturers and/or pricing received on similar treatment plant projects in Connecticut. Suitable markups were added for costs of manufacturer's services, and installation. All construction costs are based on the December 2023 ENR Construction Cost Index of 13514.76.

6.11.2 O&M Costs

Operation and Maintenance costs were estimated as follows:

The existing Coventry WPCA budget was used as a starting point for each option, since many of the existing costs associated with staffing and maintenance of the collection system, pump station, and treatment plant site will remain. Additions and deductions to applicable budget line items were made as detailed below.

- Additional staffing costs based upon the NEIWPCC staffing spreadsheets discussed in Section 6.10.
- Chemical costs for nutrient removal and solids handling were estimated based upon the usage rates discussed above and manufacturer quoted pricing.
- Electrical costs for operating the various treatment facility components were estimated based upon the expected KW usage and operating times at a rate of $0.15/KWH based upon data from Coventry's 2021-2023 electric bills. Adjustments in usage were also made for equipment shut down at the plant.
- Laboratory costs based upon the expected sampling requirements for a Class III secondary treatment facility. An allowance for future influent, effluent and PFAS sampling was included in both the SBR and OD alternatives based upon the guidance received from the CT DEEP.
- Sludge disposal costs were estimated based upon a unit charge of $0.30/gallon. This based on the following:
    - The plant's current disposal costs are $0.33/gallon.
    - The plant improvements should result in a similar sludge thickness (% solids) but allow faster loading of larger trucks that will cut hauling costs. If

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it is assumed Coventry is now paying approximately $400 per load in transportation costs, then 40% of the current budget is for sludge hauling and 60% for disposal. Allowing for larger loads should cut hauling costs in half which will reduce the current rate by 20%.

- It is acknowledged that sludge disposal costs are expected to rise in the future. Based on the above hauling costs, the sludge disposal costs are approximately $950 per dry ton of solids. In comparison, Windham is currently paying $425 per dry ton and expects that costs will go up. Nearby plants in Massachusetts were recently quoted $650 per dry ton for disposal at Upper Blackstone, as Rhode Island incinerator capacity has recently diminished.
- To be conservative, it was assumed that the total savings on hauling and disposal costs would only be 10% of current costs even though the above comparison of both types of costs suggests savings could be more.

A life cycle (present worth) cost analysis was performed for each alternative based upon a 20-year service life, including capital costs, equipment replacement costs, and annual O&M costs. O&M costs are based upon flows, loads, and estimates of labor, energy, chemical use, and sludge disposal.

For the purposes of this analysis, it was assumed that flows and loads will increase to 50% of the planned growth 10 years after construction, and the remaining growth will take place by the 20th year.

All operation and maintenance costs were estimated annually over a 20 year planning period, with O&M costs starting one year after the capital costs. The present worth costs are determined based upon a 2.5% discount rate (Department of Interior Federal Water Resources Planning published rate for FY2023) and an assumed 10 year breakeven inflation rate of 2.22% (according to the Federal Reserve December 1, 2023). This method allows the use of current O&M costs in the present worth calculations. The net present worth of the O&M costs was utilized in evaluating the costs of each alternative.

6.11.3 Capital Cost Adders

The following costs were applied to the capital cost of each option:

- 20% Contractor Overhead and Profit
- 30% Contingency
- 20% Engineering

The cost estimates developed are considered to be consistent with Class 4 estimates as defined by the Estimate Classification system of the American Association of the Advancement of Cost Engineering International (AACE International), formerly known as the American Association of Cost Engineers (AACE). The cost estimates were developed without detailed engineering data and are considered approximate. Class 4 estimates are normally expected to be accurate within minus 30 percent to plus 50 percent (-30% / +50%). This range implies that there is a high probability that the final project cost will fall within the range. The contingencies included in these cost estimates are a provision for unforeseeable additional costs within the general bounds of the project scope, particularly where experience has shown that unforeseeable costs are likely to occur. Thus, contingencies are used as a means to reduce the risk of possible cost overruns.

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6.11.4 Funding Impacts

The CTDEEP has indicated that the upgraded treatment plant option would qualify for a 20% conventional grant, as well as a 30% nutrient reduction grant. It is noted that the grant percentages do not apply to the same costs. The 30% grant is only for the cost of the components of wastewater treatment projects which relate directly to the denitrification or phosphorus reduction processes. The 20% conventional grant would apply to the remaining applicable project costs.

DEEP has also indicated that the project will be required to go through an eligibility determination in order to establish the final grant percentage. The balance of the project costs would qualify for a 2% loan under DEEP's funding program.

All projects funded by the State's Clean Water Program are required to comply with the Buy America Build America (BABA) Requirements. This program requires that all iron, steel, manufactured products and construction products used on a project be produced in the United States. This is an expansion of the American Iron and Steel (AIS) requirement which requires the use of iron and steel products that are produced in the United States. The BABA requirements expand into equipment components, many of which were previously produced overseas. As a result, some manufacturers have indicated that equipment costs can be increased by as much as 40% if BABA requirements apply.

BABA requirements apply to all projects that initiated planning after May 14, 2022. The Town of Coventry issued the Request for Qualifications for this Wastewater Management Report on March 31, 2022. Therefore, a request for a waiver was made to the DEEP in December 2022. In an email dated December 14, 2022, the DEEP concurred that BABA requirements do not apply to the Coventry project. However, the project will be required to meet AIS requirements. When developing costs for the treatment plant options, it was assumed that a 30% grant would be obtained for costs associated with denitrification and phosphorus removal. The remaining costs would qualify for a 20% grant.

6.12 Desktop Environmental Alternatives Analysis

An assessment of the environmental impacts and permitting requirements for the upgrade of the Coventry plant is presented in the Technical Memorandum that is included in Appendix G of this Report.

6.13 Recommended Secondary Treatment Technology

Upon completion of calculating construction and O&M costs for the SBR and OD technologies, it was noted that the costs were generally similar. For that reason, it was decided that the life cycle costs of both options should be compared to the Windham connection option. This discussion, along with a final recommendation, is presented in Section 8.

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Section 7
Windham Connection Alternatives Analysis

This Section presents the analysis of Windham Connection Alternatives and makes a recommendation for the pipeline route to move forward into the final evaluation phase.

7.1 Windham Connection Requirements

A meeting with the Town of Windham was held on June 26, 2023. At the meeting, the following items related to Coventry's proposed connection were discussed and generally agreed upon:

<u>Wastewater Flows/Connection Specifics:</u>

- The Windham treatment plant was designed for an average flow of 5.5 MGD. Current daily flows to the plant are 1.9 MGD. There are no capacity concerns in Windham accepting Coventry's future average, maximum month and peak flows of 266,000 gpd, 559,000 gpd and 1.5 MGD (rounded to 1050 gpm for the purpose of this evaluation), respectively.
- Coventry should avoid large slugs of wastewater being sent to the plant. Pumping should try to be continuous except for when flows drop very low overnight.
- Influent screening will be required.
- Odor control in the force main may be required.
- A flow meter must be installed to measure flows being sent to Windham for billing purposes.

<u>Connection Location</u>:

The connection to the Windham collection system should take place at a manhole located on the north side of Route 6. This 21" RCP interceptor will be able to handle the proposed peak flows without any capacity issues. No Windham pump stations will be impacted if the connection is made at this location, as all flow is gravity from this point to the treatment plant.

<u>Connection Costs</u>:

If Coventry connects to the Windham collection system, three separate fees will be assessed:

- Initial connection fee
- Annual User Fees
- Cost Sharing for future upgrades at the Windham treatment plant.

Fees will be discussed further as part of Section 7.13.

7.2 Pump Station Requirements

Under this alternative, all flow from Coventry must be pumped to the Windham collection system. Prior to the evaluation of connection routes, a suitable location for a new pump station must be considered. This section discusses and recommends a location for the new pump station.

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7.2.1 Pump Station Size and Components

A new pump station must be capable of handling future peak flows from the Coventry collection system. The station must also contain all of the features required by Windham. For the purposes of this evaluation, it has been assumed that the pump station will have the following components:

- Pumps capable of handling the future peak flows from Coventry with one pump being out of service as required by TR-16 (New England Design Standards). The pumps would be equipped with variable frequency drives to allow the flow to be ramped up and down to match incoming flows, thus avoiding sending large slugs of flow to Windham.
- Emergency power in the form of an on-site generator
- Influent bar rack
- Flow meter on the outgoing force main
- Force Main odor control

If a new pump station were to be constructed, it is assumed that a new building will be required to house the generator and electrical controls.

7.2.2 Pump Station Location

Two potential locations were identified for the new pump station. Each location is discussed further below:

<u>Existing WPCF Site</u>:
The primary and ideal pump station site location is the existing WPCF site. The existing wet well would remain, the influent chamber would be equipped with a new bar rack, and the influent pump room would be retrofitted for the new pumps and discharge piping. The number of pumps required must be determined during the final design phase. A new generator would also be installed, sized as needed for the new pumping system.

Maintaining the existing treatment plant operational while the new pump station is being constructed would have to be incorporated as part of the final design phase.

<u>120 Main Street, Coventry</u>:
The secondary pump station site evaluated is located at the end of the access drive for 120 Main Street, adjacent to the Willimantic River. Connecting to the new pump station would require an extension of the existing 18-inch gravity sewer main south along Route 31 and onto the 120 Main Street property. A new pump station would have to be constructed, including a wet well, discharge piping, and a building to house the generator and electrical controls.

It should be noted that this location is within the flood zone of the Willimantic River. Thus, environmental permits would be required, and all critical components would have to be installed 3 feet above the 100 year flood zone.

7.2.3 Pump Station Recommended Location

It is recommended that the new pump station be constructed at the existing treatment plant site, for the following reasons:

- Installing the pump station at the existing treatment plant by modifying the existing building and wet well avoids construction of an entirely new facility.
- Construction at the plant keeps the new pump station outside of the flood zone of the Willimantic River.
- A pump station at the plant will require a new force main to be installed from the treatment plant south along Route 31. However, the cost per foot to install a force main should be significantly less than that of an 18-inch gravity trunk sewer.
- The Town already owns the treatment plant site. Thus, no additional easements would be required.

7.3 Force Main Routing

Five feasible routes were identified to transport wastewater from the existing WPCF site to the Windham connection manhole. The routes include pipeline installation within State Route 31, State Route 32, and along the State Route 6 corridor. All five routes begin at the WPCF site, and all routes are identical from the intersection of Routes 31 and 32 to the connection manhole on the north side of CT Route 6 to the east of the Mansfield Avenue overpass.

See Figure 7-1 for a summary of layouts for all 5 alternatives. A brief description of each is as follows:

Alternative 1: Route 32
Alternative 1 involves crossing the Willimantic River and New England Central Railroad directly east of the WPCF. Once beyond the railroad tracks, the force main would be installed through an easement area until it reaches Route 32. The sewer force main would then run south down Route 32 and then go off-road along Route 6 to the proposed connection manhole.

Alternative 2: Route 31
Alternative 2 involves installing the sewer force main along Memorial Drive and onto Route 31. The force main would then run south down Route 31 and cross the Willimantic River and the New England Central Railroad. At the end of Route 31, the sewer force main would continue south down Route 32 and then go off-road along Route 6 to the proposed connection manhole.

Alternative 3A: Depot Road (Bridge Attachment)
Alternative 3A involves installing the sewer force main briefly along Memorial Drive and then going through an easement area north of the plant along the Willimantic River, towards Depot Road. The sewer force main pipe must be installed at least 75 feet away from the WPCF potable water well. The force main could potentially be attached to the railroad bridge crossing just south of Depot Road. Attaching the force main pipe to the bridge would allow the pipeline to cross both the Willimantic River and the railroad in one location, however, easements would need to be obtained on either side of the bridge crossing for this alternative to be feasible. Once across the bridge, the force main would then run from Depot Road to Route 32, head south down Route 32 and then go off-road along Route 6 to the proposed connection manhole.

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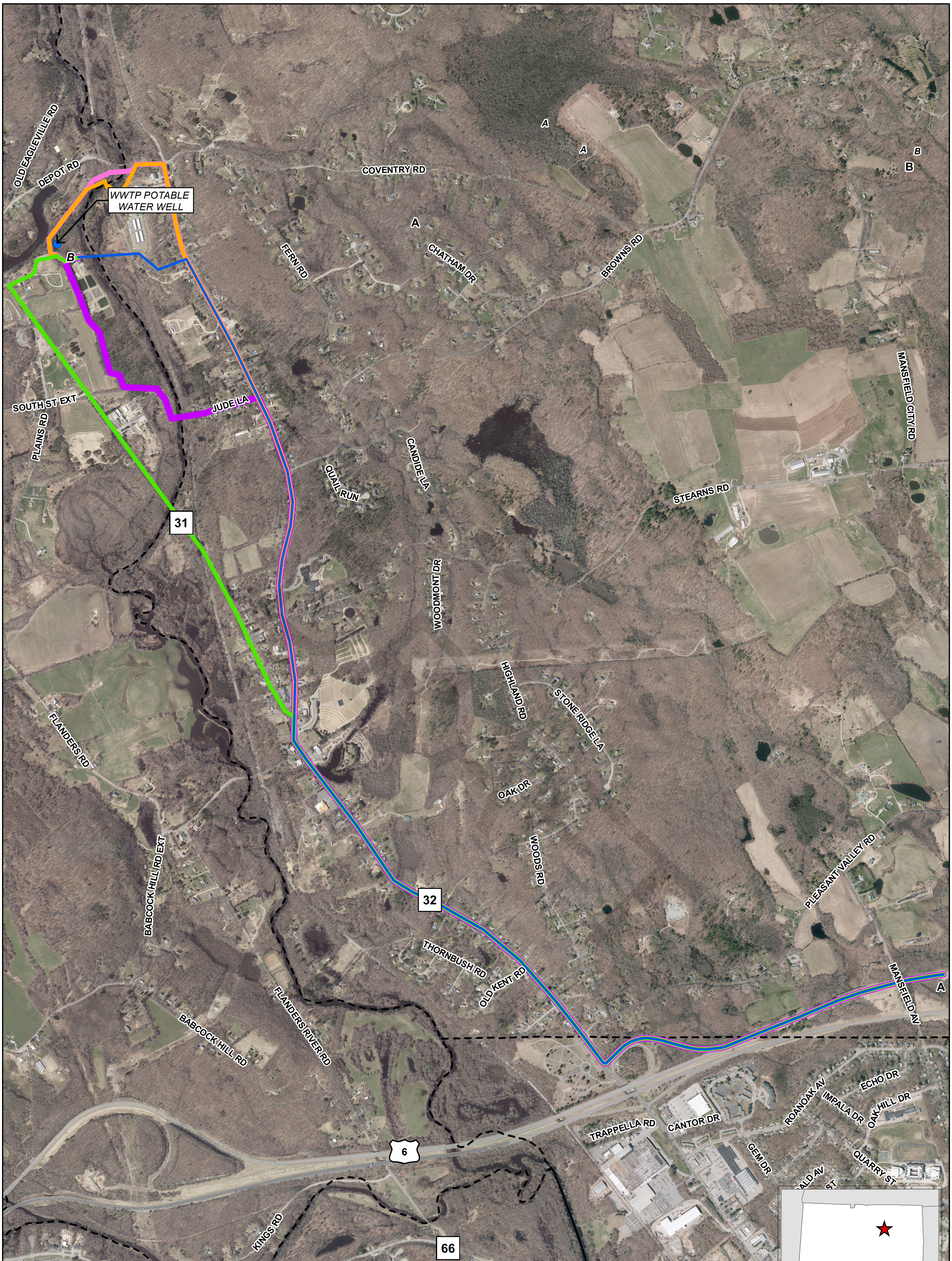

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- WWTP Potable Water Well
- CT Municipal Boundary
- Alternative 1
- Alternative 2
- Alternative 3A
- Alternative 3B
- Alternative 4

1:15,600

0 650 1,300

Feet

Tighe&Bond

1. Based on 2019 Statewide Orthophotography, Courtesy of CTECO.

FIGURE 7-1
FORCE MAIN ROUTE ALTERNATIVES OVERVIEW

Coventry WWTP
Coventry, Connecticut

December 2023

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One item of note for this layout would be that treatment plant record drawings identify an archeological site north of the plant. See Section 10 Environmental Resource Areas for further discussion.

Alternative 3B: Depot Road
Alternative 3B is the same layout as Alternative 3A with the only difference being that the sewer force main would cross the railroad bridge and the river from Depot Road, with two crossings required in lieu of a single crossing.

Alternative 4: Jude Lane
Alternative 4 involves installing the sewer force main along Memorial Drive briefly and then south through the treatment plant until it is in line with Jude Lane to the east. The force main would then cross the Willimantic River and railroad tracks before exiting along Route 32. The sewer force main would then run south down Route 32 and then go off-road along Route 6 to the proposed connection manhole.

Easements would need to be acquired in order to cross the river and railroad tracks to get the sewer force main to Route 32. Properties on Jude Lane have well water, septic systems, and are within close proximity to each other. Special considerations such as the easement location and horizontal directional drilling location would need to be made if the Town was to pursue this alternative.

7.4 Force Main and Pump Sizing

With a recommended pump station site established and connection routes identified, the force main size(s) and associated pumping rates need to be determined. The hydraulic analyses performed included the evaluation of a single force main as well as dual force mains. A discussion of each alternative is presented below.

7.4.1 Single Force Main Evaluation

The following assumptions were made for the purpose of estimating the pumping requirements and sizing of a single force main:

- All flow would be screened prior to pumping, as required by Windham.
- There would be no grit removal at the new pump station. Grit removal is not required by Windham, and it would be costly to install as the plant's existing grit removal occurs after the existing influent pump station. As a result, it is important to maintain reasonably high velocities in the force main to prevent the deposition of grit. If possible, a minimum velocity of 2 (to 3) fps should be achieved on a daily basis to push the grit through the long and likely undulating force main.
- The rate of peak flow sent through the force main will be minimized to help to reduce the required pipe size, pumping heads, and pump horsepower.
- A single force main length of 23,200 ft, which is the length of the longest feasible alternative (3B), will be installed.
- Any horizontally drilled sections would be installed with dual force mains per the request of CTDEEP. This will allow service to be maintained at a reduced capacity if repairs must be done to one of the force mains. Force mains will be designed with sufficient diameter to achieve similar head losses as those estimated here.

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- Minor loss coefficients of 7 and a Hazen Williams coefficient of 130.

- A water to wire pumping efficiency of 65%, for the purpose of evaluating the pumping options discussed below.

- Consistent with the peak flow sizing of the on-site treatment alternative and Windham's willingness to take 100% of Coventry's peak flow, the former clarigesters may (or may not) be used to limit the peak flow sent through the force main to 650 gpm. The pump station would still have to pump the peak flow of 1050 gpm, but it would send that a portion of that flow to the former clarigesters, possibly at a lower pressure. The following rational illustrates the need for this assumption:

    1) Windham does not want surging flow. Therefore, an approach which sizes the pumps and force main to run with one pump turning on and off during normal flows at a velocity of 3 fps or greater cannot be used. The impacts of pipe diameter on pump rate and required motor horsepower are presented in Table 7-1.

TABLE 7-1
Hydraulic Conditions at a Velocity of 3 fps

Force Main Diameter	Flow (gpm)	Friction Loss (ft)	Estimated Total Pump Electrical Horsepower Required
6-inch	360	1,900	400
8-inch	470	475	125
10-inch	700	160	40
12-inch	1050	70	25

    2) As stated in Section 3.6, it appears feasible for the existing clarigesters to store peak flows in excess of 650 gpm. This use of the clarigesters could theoretically help to reduce the peak hour flow pumped to Windham from 1050 gpm to 650 gpm. This reduction in the peak flow for a given force main size also helps to reduce the friction loss in the pipe and required pump horsepower. It may also allow for the use of a smaller diameter pipe that would be less prone to solids deposition and odor generation (as will be discussed in the next section).

    3) In order to pump continuously as requested by Windham and achieve a velocity of 2-3 fps for a reasonable time each day, it is necessary to characterize the daily "dry weather" flow variation. From Section 3.6, we know that:

        - Current average day flows are ~95 gpm with daily dry weather peaks at 125 to 150 gpm.

        - Future average day flows are expected to grow to ~184 gpm with daily dry weather peaks at 240 gpm to 290 gpm.

    4) The flows required to achieve 2 to 3 fps in the force main are as follows:

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- 6” Force main: 170 to 360 gpm. This is reasonably close to the daily peak flows.
- 8” force main: 310 to 470 gpm. This is well above (double) current daily peak flows.
- 10” force main: 470 to 700 gpm. This would only be achieved a few times a year during wet weather.
- 12” force main: 700 to 1050 gpm, which would only be seen every few years.

5) During the peak flow conditions, the friction losses in the force main would be reduced significantly by using the clarigesters to reduce peak flows. There is also a significant reduction in pump horsepower as noted below.

TABLE 7-2
Force Main Hydraulic Condition Comparison

Force Main Diameter	Pump Horsepower at 1050 GPM	Pump Horsepower at 650 GPM	Friction Loss (ft) at 1050 gpm	Friction Loss (ft) at 650 gpm
6-inch	785	200	1,900	775
8-inch	195	35	470	195
10-inch	65	20	160	65
12-inch	30	10	65	30

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Tighe & Bond discussed this project with a local pump vendor, who advised that dry pit raw wastewater pump selections offered in their product line (and typically used in CT and New England) have the following limitations:

- Are available up to a total dynamic head (TDH) of 250 to 275 feet.
- If that have a TDH greater than 100 ft. they risk high vibration if not operated with the pump’s preferred operating range.
- Don’t turn down below 100 gpm (which is consistent with 3 ft/sec in a 4” force main which is the minimum size used in the industry) and these are typically submersibles.

Based on the above, the following conclusions can be made:

- The single 6” force main option is ruled out as impractical given the high pump horsepower (HP) required.
- The single 12” force main option appears to be impractical given the very low velocity, very high solids deposition potential, and odor generation potential. However, this was evaluated further in the analysis below under the assumption that grit removal might be installed prior to the pump station, thereby allowing lower velocities in the force main.
- Sending Coventry’s peak flow of 1050 gpm to Windham appears to have little value unless 1) multiple force mains are considered, and 2) it is cost effective for Windham to construct a new grit removal system on the incoming sewer, which is unlikely.

The following additional assumptions were made to continue the evaluation:

- The total dynamic head (TDH) delivered by the new pumps will be based on the force main friction losses added to the Static Head Lift of 65 feet. This lift is conservative based on the difference in the lowest anticipated wet well elevation (~260 ft) and the highest elevation on the force main (estimated at Elev ~325 ft); a difference of 65 ft.

- Windham will accept pumps cycling on and off at a minimum flow of 120 gpm. This is equivalent to a pump station with 4" force main discharging a three feet per second.

- Depending on the size of the force main, the pump station will need to:

  1) Include both dry weather (low head) and wet weather (high head) pumps which will be abbreviated as DWPs and WWPs.
  2) Divert flow in excess of 650 gpm to the clarigesters by opening a three-way valve on a pump discharge manifold that will send up to 400 gpm of flow from the DWPs to the splitter box for the existing clarigesters. This will allow the DWPs to operate at a much lower pressure. A schematic is presented in Figure 7-2 below:

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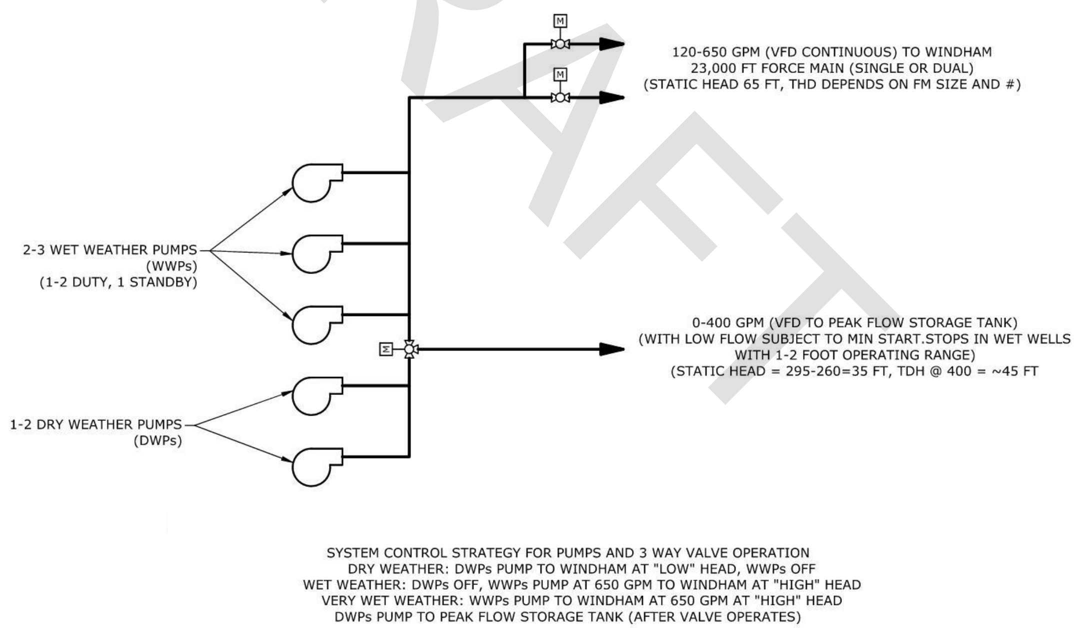

FIGURE 7-2
System Control Strategy for Pumps and 3 way Valve Operation

<u>Conclusions:</u>
Based on these assumptions, the pumping needs for each size force main assuming only one force main were estimated as follows:

TABLE 7-3
Single Force Main Hydraulic Condition Comparison

Force Main Diameter/Flow	Velocity (fps)	TDH (ft)	Total Electrical Pump Horsepower Required for Pumping
8-inch @120 GPM	0.8	75	3
8-inch @ 400 GPM	2.6	145	25
8-inch @ 650 GPM	4.1	260	50
10-inch @120 GPM	0.5	70	3
10-inch @ 400 GPM	1.6	95	15
10-inch @ 650 GPM	2.7	130	35
12-inch @120 GPM	0.3	67	3
12-inch @ 400 GPM	1.1	76	12
12-inch @ 650 GPM	2.4	110	25
12-inch @ 1050 GPM	3.0	130	55

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From this information, and other operating costs estimated, it was concluded that none of the single force main alternatives are desirable for the following reasons:

- The 12" force main option is eliminated from further consideration based on the fact that the force main velocity will rarely be high enough rate to prevent solids deposition. The only way this diameter becomes feasible is through the construction of a new headworks building containing screening and grit removal. The energy savings associated with a fewer number of lower horsepower pumps does not offset this expense.

- With a 10" force main, the required horsepower could potentially be provided using two duty (1 spare) ~20 HP pumps. However, low velocities in the force main during dry weather flows create the likelihood for solids deposition. 20 HP pumps are also twice the horsepower of the existing treatment plant pumps. Thus, it may not be feasible to install all five pumps within the existing pump room.

- The 8" force main is also eliminated from further consideration, as the required head is beyond the outer range of typical wastewater pumps. In addition, it is highly unlikely that the two duty (1 spare) 25 HP pumps can fit within the existing pump room.

7.4.2 Dual Force Main Evaluation

Due to results of the analysis for the single force main option, the feasibility of installing two parallel force mains of the same size for the entire distance was evaluated. Since some of the bridge /river crossing options would likely require splitting a single force main into two sections at the crossings, this approach would greatly simplify the complexity of these crossings and eliminate the need to site two valve vaults at each crossing.

The dual force main design approach summarized in the table below suggests that the second force main would automatically be placed into service at one half of the peak flow capacity. In practice, the pumps will be able to deliver more than the stated flow and velocity with just one force main in service and the second force main will not be brought online until the pumps are at their maximum flow out in order to optimize the flushing of the force main.

TABLE 7-4
Dual Force Main Hydraulic Condition Comparison

Force Main Diameter/Flow	No. of Force Mains in Service	Velocity (fps)	TDH (ft)	Total Electrical Pump Horsepower Required for Pumping
6-inch @120 GPM	1	1.4	100	5
6-inch @ 325 GPM	1	3.7	282	35
6-inch @ 325 GPM	2	1.8	125	15
6-inch @ 650 GPM	2	3.7	282	70
8-inch @120 GPM	1	0.8	75	5
8-inch @ 325 GPM	1	2.1	120	15
8-inch @ 325 GPM	2	1.0	80	10
8-inch @ 650 GPM	2	2.1	120	30

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From this data, it can be concluded that:

- Dual 6” force mains are not feasible, as the required head is beyond the outer range of typical wastewater pumps. In addition, the proposed horsepower (requiring two duty (1 spare) 35 HP pumps) is significantly greater than that of the existing pumps, so it is unlikely the new units would be able to fit into the existing pump room.

- Dual 8” force mains appear to be the best available option for Coventry to send wastewater flow to Windham.
  - The 30 HP required could likely be provided using two ~15 HP duty WWP’s (and one 1 standby) at only twice the HP of the existing pumps.
  - Reasonable velocities can be achieved in the force main and it appears that the risk for solids deposition is low enough to rule out the need to add grit removal. As long as the second force main is not put online until high flows, this dual 8” force main option will have higher velocities and reduced solids deposition as compared to the single 10” force main option.
    - At current flows, the pumps will likely run a significant time at the assumed min speed of 120 gpm so the velocities will increase from 0.4 to 0.8 ft/sec.
    - Assuming the pumps are designed assuming the second force main is not placed into service until flow is higher than 325 gpm, the velocity will reach at least 2.1 ft/s. With a good pump selection, it might be possible to achieve closer to 3 ft/s with two duty pumps in service before placing the second force main in service.

    - It should be noted that design flow 325 gpm is higher than the 260 gpm that the plant's existing pumps push influent flow through the existing 8" force main to the grit chamber during dry weather (while they cycle on and off) and there is a significant vertical section of this pipe. At the flow of 260 gpm there has not been a problem with grit or rag buildup in the existing 8" piping.

  - The range in required TDH is not excessive, and the peak head is only slightly higher than 100 ft lowering the risk for vibration in the pumps.

7.4.3 Preliminary Pump Selection

Preliminary wet weather and dry weather (dry pit submersible) pump selections were requested from a vendor for the dual 8" force main option selected. They offered two options for the wet weather pumps:

- Three 25 HP pumps capable of delivering 650 gpm (with BEP of 48%)
- Three 31 HP pumps capable of delivering 800 gpm (with a BEP of 53%)

Based on our review of the selections, the following was concluded:

- The three wet weather pump selections would fit in the pump room where the existing pumps are located.
- The pumps were much less efficient than our initial assumptions and it would not remain within the pumps preferred operating range at the low range where the pumps might be expected to run most ~50% of the time. This is not desirable.
- The wet weather pump would not turn down to 120 gpm. The lowest turndown they would recommend for these pumps is above 130 gpm but is it running at 50% of the pumps BEP at 40 Hz and would lead to premature wear and failure as the pump would be running at minimum speed.
- The dry weather pump(s) would need to be small and submersible (but not dry pit) which would be difficult to install in the station. This is because for their smaller pumps with this range of flow, they are oil cooled and are not recommended to run continuously in a dry pit system.

Based on selections provided, the wet weather/dry weather pump approach was abandoned.

7.4.4 Final Pump Selection

It was concluded that the concept of diverting flow to the clarigesters will be too complicated and require a level of control that is less than desirable. An alternate conceptual design approach was therefore considered for the pumps, with the goal of having the pumps fit in the existing pump room. The following was assumed:

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- Three pumps (two duty, one standby) rated at 525 gpm each with a Capacity of pumping 100% of the flow (1050 gpm) to Windham.

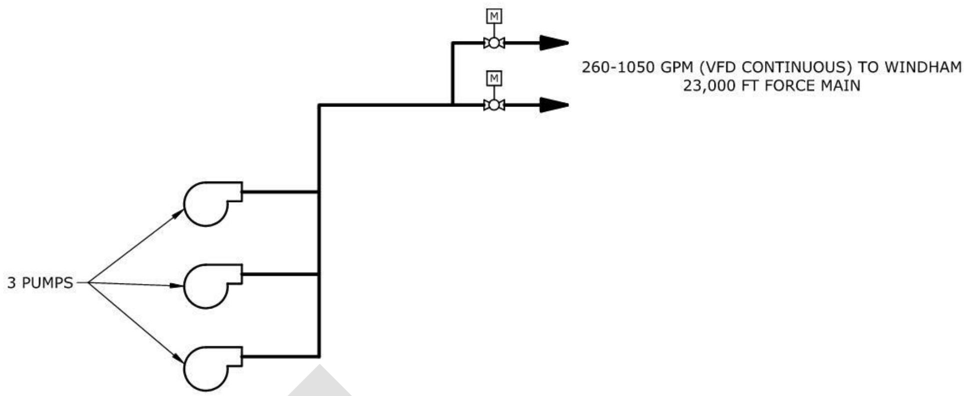

FIGURE 7-3
Recommended System Control Strategy with No Overflow to Clarigesters

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- Air release valves would be required at all force main high points in order to prevent trapped air from reducing the capacity of the pipeline. Vacuum relief valves would also be required at some of the high points to allow portions of the line to flow by gravity. The combination of the portions of the force main that would flow by gravity during low flows and the use vacuum relief valves will also smooth out the flow entering Windham system when Coventry's pumps start and stop.

- Windham would accept the pumps cycling on and off at around 260 gpm (the approximate flow rate of Coventry's existing pumps). This is a turn down of ~2:1 which would hopefully be in the preferred operating range of the pump across all flows. The following is noted:

  - It is likely that the distance travelled through the collection system in Windham and the portions of the force main that would flow by gravity would attenuate the peaks of flow entering their plant a flow variation much less than 260 gpm. During high flows in wet weather, there would be no starting and stopping of flow.

  - If sending flow to Windham was selected as the feasible alternative, more advanced pumping systems or control systems could be considered during design if Windham was concerned about flow starting and stopping at 260 gpm.

  During peak flows, the last 5,000 feet of the force main flow by gravity due to the slope of the force main allowing a reduction in the TDH (including static head) requirements. At lower flows, there would be a similar reduction in head and the majority of the force main would flow by gravity from the high point about 7,500 feet from the pump station. These assumptions would need to be verified during design.

- 260 gpm in an 8 inch pipe represents a velocity of 1.7 ft per second, which is much less than 3 ft/sec recommended in TR-16. This flow is considered feasible because

Coventry runs their existing pumps at this flow rate now and has not reported problems with grit buildup in the 8" vertical force main from the influent pumps to the grit tank and this pumping is post screening.

Based on the above assumptions, final design criteria for the pumps was developed and summarized in Table 7-5. The pump selection from the vendor was a set of 3 - 40 HP dry pit submersible pumps that operate at 3600 RPM and appeared to meet the criteria (operate at 260 gpm close to the pumps BEP). It was also confirmed that the pumps would fit in the existing pump room. A budget cost of $111,000 was provided by the vendor.

TABLE 7-5
Recommended Final Pump Selection

Force Main Diameter/Total Flow	No. of 8" Force Mains in Service	Velocity (fps)	TDH (ft)	Total Electrical Pump Horsepower Required for Pumping
8-inch @260 GPM	1	1.7	80	10
8-inch @ 525 GPM	1	3.4	130	60
8-inch @ 525 GPM	2	1.7	80	20
8-inch @ 1050 GPM	2	3.4	130	60

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7.5 Force Main Odor Control

The length of the proposed force main to Windham necessitates an evaluation for potential odors due to the travel time from Coventry to Windham. The evaluation estimates the hydrogen sulfide potential within the dual 8" force main option at current flows and loads. Under future flow conditions, the potential will likely be similar as retention times will decrease but the domestic load will grow faster than I/I.

The Pomeroy formula was used for this evaluation. The formula creates a model of the biological slime on the pipe walls and within the flow stream to convert sulfate present in the water into (hydrogen) sulfide in the presence of BOD and with the lack of other forms of oxygen (e.g. dissolved oxygen or nitrate). The model also estimated chemical usage using commonly accepted factors.

Major assumptions used in the model are as follows:

- The chemical is injected only into the active force main at the pump station. There are no other injection locations.
- The force main is 8" in diameter with a length of 23,200 ft as discussed previously.
- Wastewater flows and strengths are as discussed in Section 3 (0.143 MGD Avg, 160 mg/l BOD). As flows increase, the residence time in the force main decreases and so does the chemical usage. For the purpose of evaluating the potential cost of odor control, no growth in flows and loads was conservatively assumed.
- Summer and Winter Temperatures of 12 to 22 deg C.
- Influent Dissolved Oxygen concentration in the incoming sewer of 2 mg/l.

- The operation of air release and vacuum relief valves in the force main would contribute no oxygen into the wastewater during normal flows as the pumps cycle on and off. If oxygen were introduced, this would reduce H2S formation and chemical usage.

- Bioxide (a source of oxygen in the form of a liquid calcium nitrate solution) would be injected into the force main at the pump station at a dosing rate of 1.3 gal of sulfite per pound of sulfide forming potential. This is often the first choice of operators in this situation, therefore other chemicals were not considered. A standard bioxide system is fully automated and is supplied with a VFD and modem to allow operators to monitor various parameters including gallons of chemical used, alarm set points, and hourly chemical dose curves. It is also possible for operators to adjust the bioxide dose remotely as needed.

The model estimated a H2S formation potential of 15 to 35 lbs per day based on winter and summer conditions. This suggests dosing of 15 to 45 gallons of Bioxide per day. Our experience is that the actual doses can be much less (often none in the winter) so the supplier was contacted for their recommendations.

Their recommendation was to assume a chemical dose of 34 gallons per day, with the solution being used for half the year. This cost will be added into the overall cost estimate for the Windham Connection Alternative.

It should also be noted that the generation of H2S in the inactive force main cannot be controlled. H2S will be formed until the available sulfate is consumed. Therefore, it is important that both force mains be protected from corrosion, which is most likely to occur at high points where the air release (and vacuum relief) valves will be installed.

- When the inactive force main is turned on, there will be a slug of H2S into Windham's collection system and at the air release valves. Due to the few times a year this may be required, and remote location, this should not present a problem.
- If the second force main is activated during high flows associated with wet weather events, the flow from the force main already in service will help to dilute H2S concentrations in the Windham system and the slug will be a very short term one.

If the active force mains are changed during dry weather conditions, the impact will be more pronounced. Therefore, this should be avoided to the extent possible, especially during low flow, dry weather conditions, typically expected during the summer.

7.6 Pipeline Installation Methodologies

Most pipeline construction projects are accomplished using traditional excavation equipment. However, this type of installation may not be feasible for crossing the Willimantic River and Railroad tracks, and/or passing through easement areas. A discussion of potential pipeline installation methods is presented in the next section.

7.6.1 Open Cut Excavation

The open cut excavation method involves excavating down to a specific depth required, installing the pipeline, and backfilling the trench to grade. Tree and brush clearing will be necessary for off-road pipeline installations prior to performing the open cut excavation

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method. Backfill and restoration requirements will vary based on the location and permitting requirements associated with them.

7.6.2 Directional Drilling

Horizontal directional drilling is a trenchless method of pipeline construction that involves using a drill rig to drill a pilot hole and cut a path for the new pipeline. After the pilot hole is drilled, different sized reamers are used to expand the diameter of the bore hole. Once the bore hole is large enough, the new pipeline is pulled through.

Geotechnical explorations in the form of soil borings are required prior to the design of a horizontal directional drilling installation. Drillers would need, at minimum, one boring on either side of the proposed crossing to at least the maximum depth of the proposed pipeline. It is preferable to get an additional boring halfway across the crossing, but this may be infeasible as this may be located in the Willimantic River, within close proximity to the New England Central Railroad, or the terrain may be inaccessible for a drill rig to get to the location. The recommended decision as to drill in or out of rock will be determined during the design phase after geotechnical explorations have occurred.

7.6.3 Jack/Bore

The Jack and Bore trenchless method of pipeline installation is used for short pipe runs in stable and dry soils without large boulders. Two pits are excavated: a sending pit and a receiving pit. A jack and bore machine is then placed in the sending pit and cuts a hole though the ground while simultaneously pushing the new pipeline in place, exiting at the receiving pit. After the pipeline is installed, the pits are backfilled and restored accordingly.

7.7 Bridge Crossing Feasibility

Tighe & Bond investigated the feasibility of crossing ten bridge structures as part of this study. This effort included acquiring and reviewing available record plans, inspection reports, and load ratings for each structure, and reviewing field conditions. The structures are each described in detail below. See Figure 7-4 for locations of bridge crossings.

Depot Road over Willimantic River (Bridge 05989)
This structure is a 138-foot long, three-span prestressed concrete adjacent deck unit structure supported on stone masonry piers and abutments. The stone masonry substructures were reused from the previous bridge. The structure has a "scour critical" appraisal rating of 3, indicating that the theoretical calculated or presumed scour depth for the design storm is deeper than existing foundations, and may undermine the structure resulting in a loss of support. The structure is a liability and should be avoided. It may be bypassed using directional drilling under the river.

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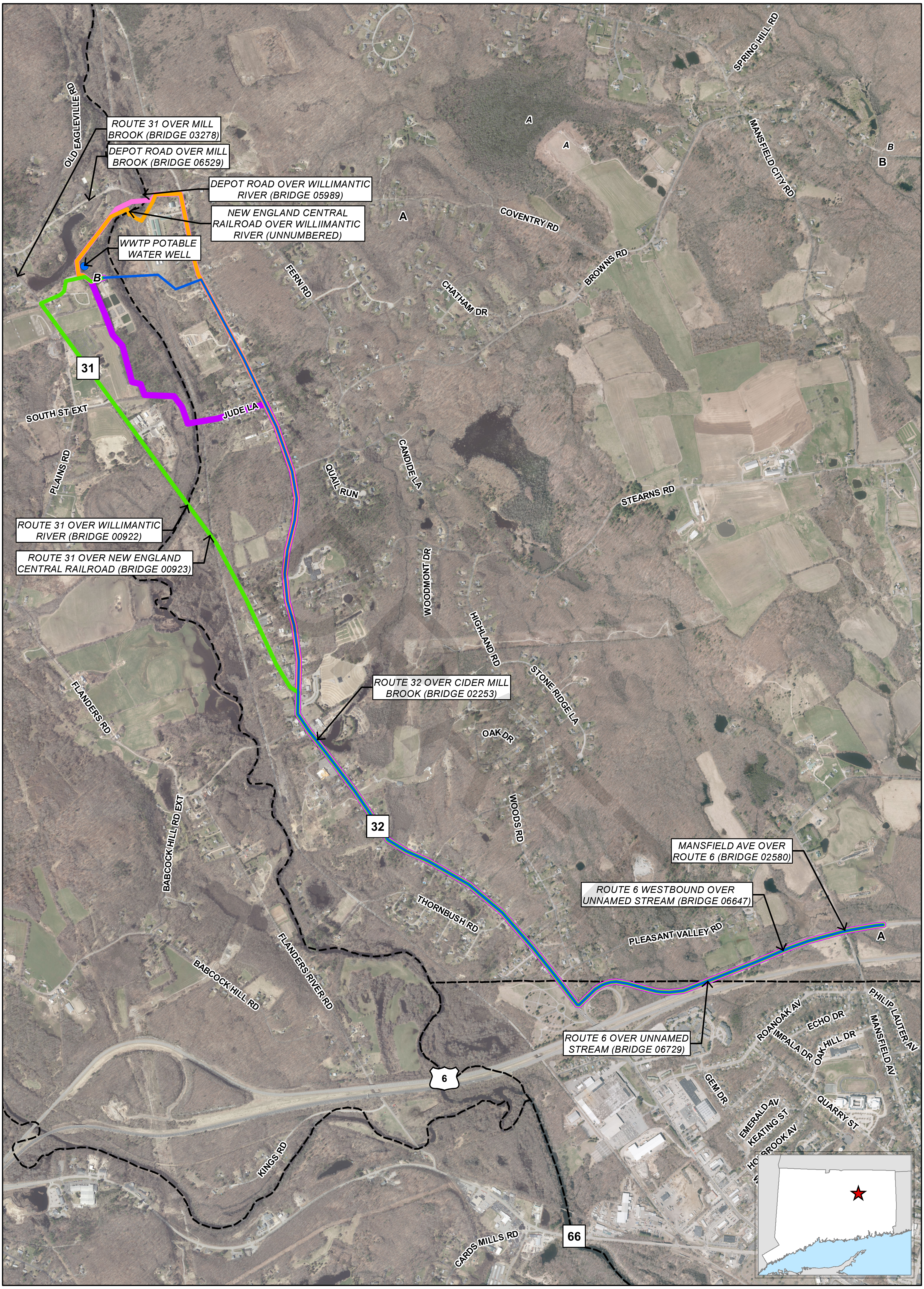

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- WWTP Potable Water Well
- CT Municipal Boundary
- Alternative 1
- Alternative 2
- Alternative 3A
- Alternative 3B
- Alternative 4

1:16,800

0 650 1,300

Feet

Tighe&Bond

1. Based on 2019 Statewide Orthophotography, Courtesy of CTECO.

FIGURE 7-4
ROUTE ALTERNATIVES OVERVIEW BRIDGE CROSSING LOCATIONS

Coventry WWTP
Coventry, Connecticut

December 2023

G:\GIS\CT\CoventryCT\MXD\WWTP\Route Alternatives\CoventryCT_WWTP_WindhamConnection_AllRoutes_RRCrossing.mxd [Exported By: AFreudenberg, 12/6/2023, 3:52:11 PM]

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Depot Road over Mill Brook (Bridge 06529)
This structure is a 20-foot span precast concrete arch supported on strip footings. Like the Depot Road bridge over the Willimantic River, the structure has a "scour critical" appraisal rating of 3 and should be avoided. The structure may be bypassed using open cut or directional drilling under the brook or a separate utility bridge. Due to the difficulty of crossing this bridge during initial review, this bridge crossing wasn't recommended as an alternative for further review.

New England Central Railroad over Willimantic River
This structure is a 100-foot span steel truss carrying a single track of freight rail. The bridge is not inspected by CTDOT since it is privately owned. The truss may have an open corridor for a utility pipe; however, a bridge attachment is not recommended. Any construction and subsequent inspection would require coordination with the freight rail owner, which is anticipated to be time-consuming. Lack of CTDOT inspection also means that the burden of regular inspection of a pipe crossing would fall on the utility owner. The structure may be bypassed using directional drilling under the river.

Route 31 over Mill Brook (Bridge 03278)
This structure is a 24-foot long, two-span stone masonry and concrete arch. The bridge is rated in fair overall condition (5), based on the condition of the superstructure and substructure. The rating appears to be generous, and considering its substandard geometry and bridge rails, it is a likely candidate for near-term replacement. Additionally, the stone masonry fasciae have not been maintained, making the structure a poor candidate for a bridge attachment. The structure may be bypassed using directional drilling under the brook or a separate utility bridge. Due to the difficulty of crossing this bridge during initial review, this bridge crossing wasn't recommended as an alternative for further review.

Route 31 over Willimantic River (Bridge 00922)
This structure is a 126-foot span steel multigirder bridge supported on reinforced concrete abutments. The structure is rated in satisfactory overall condition (6) based on the condition of the deck and superstructure. The bridge also has a healthy inventory load rating of 1.17 for the current AASHTO design loading. The steel plate girders have a web depth of 64 inches, and there appears to be adequate room to support a utility main underbridge in any of the four bays between the girders.

Route 31 over New England Central Railroad (Bridge 00923)
This structure is a 79-foot span steel girder-floorbeam-stringer bridge supported on tall reinforced concrete abutments. The structure is rated in fair overall condition (5), based on the condition of the steel superstructure. The bridge also has a substandard load rating, with girders, floorbeams and stringers all having rating factors less than 1.0. Since the floorbeams are full-depth, it is infeasible to install a utility pipe beneath the bridge. The fracture-critical configuration and substandard load rating make it so that the State would not permit a utility to be attached to either fascia. The only feasible options to cross the railroad at this location appear to be directional drilling or jack and bore.

Route 32 over Cider Mill Brook (Bridge 02253)
This structure is a 15-foot span concrete arch. No record plans are available. While the structure is rated in satisfactory overall condition (6), the rating appears to be generous. The wingwalls exhibit severe surface spalling, and the arch and wingwalls exhibit map cracking throughout. The arch has shallow cover, and it would not be feasible to install a

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pipe in the fill above it. The parapets are in good condition (7) and could be candidates for a fascia bridge attachment. The inspection report photos don't show any ledge in the area, so open cut or directional drilling under the brook adjacent to the arch also appear to be feasible options at this time.

Open cut excavation is the least expensive option and was therefore selected as the most appropriate installation method for this crossing. With open cut excavation through the brook, a water control plan will need to be set during construction.

Route 6 over Unnamed Stream (Bridge 06729)
This culvert is an 8'-10" x 6'-1" asphalt-coated, corrugated structural plate pipe arch with about 8 feet of cover beneath the westbound barrel of Route 6. The pipe arch is rated in poor overall condition (4), making it likely that the State will line or replace it in the near future. The structure could be crossed via directional drilling beneath the existing culvert or bypassed by open cut or directional drilling under the stream beside the existing inlet headwall. Open cut excavation could either above the structure or under the stream beside the existing inlet headwalls is the least expensive option and was selected for this crossing. Specific routing would be determined during final design.

Route 6 Westbound Over Unnamed Stream (Bridge 06647)
This culvert is a 10'-3" x 6'-9" asphalt-coated, corrugated structural plate pipe arch with approximately 4 to 5 feet of cover beneath the westbound barrel of Route 6. While the pipe arch is rated in satisfactory overall condition (6), the State may eventually line or replace it, as they have been doing with much of their corrugated metal structure inventory. Because of the relatively shallow cover, it is not advisable to install a utility pipe in the roadbed over the culvert. The structure could be crossed via directional drilling beneath the existing culvert or bypassed by open cut or directional drilling under the stream beside the existing inlet headwall. Open cut excavation could either above the structure or under the stream beside the existing inlet headwalls is the least expensive option and was selected for this crossing. Specific routing would be determined during final design.

Mansfield Avenue over Route 6 (Bridge 02580)
This structure is a 135-foot span steel box girder bridge that spans overhead above the proposed connection. According to record plans, the abutments for this bridge are supported on spread footings that are set back from the road. A simple, open cut trench along Route 6 could easily avoid existing substructures.

Summary of Bridge Crossing Feasibility
Table 7-6 lists the feasibility and recommended installation method for each bridge crossing.

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TABLE 7-6
Comparison of Bridge Crossings Feasibility

Bridge Crossing Location	Feasible	Method of Installation
Depot Road over Willimantic River (Bridge 05989)	Yes	Directional Drilling
Depot Road over Mill Brook (Bridge 06529)	No	N/A
New England Central Railroad over Willimantic River (Unnumbered)	No	N/A
Route 31 over Mill Brook (Bridge 03278)	No	N/A
Route 31 over Willimantic River (Bridge 00922)	Yes	Bridge Attachment
Route 31 over New England Central Railroad (Bridge 00923)	Yes	Directional Drilling
Route 32 over Cider Mill Brook (Bridge 02253)	Yes	Open Cut Excavation
Route 6 over Unnamed Stream (Bridge 06729)	Yes	Open Cut Excavation
Route 6 Westbound over Unnamed Stream (Bridge 06647)	Yes	Open Cut Excavation
Mansfield Ave over Route 6 (Bridge 02580)	Yes	Open Cut Excavation

7.8 Construction Route Feasibility Evaluation

With the bridge and stream crossing evaluation work completed, a review of each of the five pipeline route alternatives was conducted to determine if the overall pipeline route is feasible as well as the associated construction methodologies to use. Each route alternative is discussed individually below.

Alternative 1 (Steep Willimantic River embankment and railroad crossing)
Due to the proximity of a very steep embankment directly east of the treatment facility site, crossing the Willimantic River itself, and crossing the railroad tracks immediately east of the Willimantic River, it is not feasible to perform open cut excavation for this crossing. The length of the crossing is too great for the jack and bore trenchless excavation method. Horizontal directional drilling would be required for this crossing. After the crossing is complete, open cut excavation would be performed for the remainder of the pipeline installation along CT Route 32 and CT Route 6.

To complete the horizontal directional drilling, a drill rig would need to set up on the east side of the railroad and Willimantic River and end on the other side of treatment plant. One drill setup will be completed to cross underneath both the railroad and the Willimantic River. The approximate drill length for this location would be between 1,000 and 1,200 feet long.

Alternative 2 (Drill under railroad on CT Route 31)
The open cut excavation method would be used on Memorial Drive and south on CT Route 31 until the pipeline reaches the Willimantic River. Upon reaching the Willimantic River bridge crossing, the pipeline would be attached to the bridge structure and continue south on CT Route 31. After briefly installing the pipeline south of the Willimantic River, horizontal directional drilling would be required to drill underneath the railroad crossing, adjacent to the CTDOT Bridge 00923. CTDOT does not allow horizontal directional drilling to be performed directly underneath the bridge structure. The jack and bore trenchless method would not be feasible due to the steep grades along the river embankments and the length of drilling would be too great. Open cut excavation is not allowed to cross the railroad.

To complete the horizontal directional drilling, a drill rig would set up to cross underneath the railroad. The approximate drill length for the railroad crossing is 600 feet. After the

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crossing is complete, open cut excavation would be performed for the remainder of the pipeline installation along CT Route 32 and CT Route 6.

Alternative 3A (Bridge Attachment near Depot Road)
This alternative is deemed to not be feasible as any construction and subsequent inspection of a pipeline installed within the existing bridge would require coordination with the freight rail owner, which is anticipated to be time-consuming. Lack of CTDOT inspection also means that the burden of regular inspection of a pipe crossing would fall on the Town. Alternative 3A is therefore eliminated from further consideration.

Alternative 3B (Drill under railroad and Willimantic River near Depot Road)
The open cut excavation method would be used on Memorial Drive and north through easements towards Depot Road along the Willimantic River. Once near the railroad and for efficiency, horizontal directional drilling would be used to cross underneath both the railroad and the Willimantic River. Jack and bore trenchless method is not recommended as it would require two setups to cross each feature. The approximate drill length for this crossing would be 600 feet. Open cut excavation method is not possible to cross the railroad or the Willimantic River. After the crossing is complete, open cut excavation would be performed for the remainder of the pipeline installation along Depot Road, CT Route 32, and CT Route 6.

Alternative 4 (Drill under railroad and Willimantic River to Jude Lane)
The open cut excavation method would be used briefly on Memorial Drive and then off-road south through the treatment plant until it is in line with Jude Lane to the east. Open cut method is not possible to cross the Willimantic River and railroad to get to pipeline to Jude Lane. Horizontal directional drilling and the jack and bore trenchless excavation methods are not feasible to cross the Willimantic River and the railroad as drilling near the houses, septic systems, and water wells on Jude Lane would not be allowed. Due to the crossing not being feasible, this option is not recommended. Alternative 4 is therefore eliminated from further consideration.

Summary of Route Alternative Feasibility

Alternative 1: Horizontal directional drill under Willimantic River and railroad to CT Route 32, open cut excavation along CT Route 32, cross Cider Mill Brook via open cut excavation, then open cut along CT Route 32 and CT Route 6.

Alternative 2: Open cut excavation on Memorial Drive and CT Route 31, cross Willimantic River via bridge attachment, cross railroad via horizontal directional drilling, open cut excavation along CT Route 31 and 32, cross Cider Mill Brook via open cut excavation, then open cut along CT Route 32 and CT Route 6.

Alternative 3A: Not feasible.

Alternative 3B: Open cut excavation on Memorial Drive and off-road to Depot Road, cross Willimantic River and railroad via horizontal directional drilling, open cut along Depot Road and CT Route 32, cross Cider Mill Brook via open cut excavation, then open cut along CT Route 32 and CT Route 6.

Alternative 4: Not feasible.

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7.9 Permitting Requirements

7.9.1 CTDOT Permitting

Work being completed within the CTDOT right-of-way on CT Route 31, CT Route 32, and CT Route 6 warrant special requirements. Any CTDOT bridge crossings will require review by the CTDOT bridge group during design. Horizontal directional drilling under any CTDOT structure is not allowed, however, drilling adjacent to a structure is acceptable. A summary of general requirements are as follows:

- Traffic control during construction based upon on alternating one way traffic.
- There will be no laydown areas within the right-of-way and all equipment and materials must be out of the right-of-way during the nighttime hours.
- Setup, laydown, and protection of drill rigs within the right-of-way will be allowed but will require prior approval from CTDOT.
- Typical CTDOT working hours are from 8:30AM to 4:00PM. Anything outside those hours will require special requirements for construction.
- Road restoration requirements will depend on the age of the road and location of the pipeline and will either be full width or curb to centerline roadway restoration.
- Asphalt thickness will need to either match the existing thickness of the roadway or be 9-inches thick, whichever is greater. CTDOT does not have any records of the existing conditions of CT Route 31 or CT Route 32.
- Any inlayed road markings, rumble strips, disturbed curbing, disturbed guardrail, and any other existing features will need to be placed back in place during restoration efforts.

7.9.2 New England Central Railroad Permitting

General Permitting Requirements:
Prior to commencing entry or construction activities within the New England Central Railroad right-of-way, a Right of Entry Permit or Contractor Occupancy/Access Agreement application must be submitted and approved. The standard term for the agreement is 60 days and the permit application fee is $1,750. The standard processing time is 6 to 8 weeks but if expedited processing is required, the fee is $2,500 and the expedited processing time is 1 to 2 weeks. The application must include latitude and longitude with an aerial map of the proposed worksite, and complete scope of work. Upon approval, A Railroad Protective Liability Application must be filled out to prove certificate of insurance.

A Utility Occupancy License (Pipeline) must be obtained to initiate the installation of an underground pipeline within the New England Central Railroad right-of-way. An Application for Underground Pipeline Crossing or Parallelism of Railroad Property and or Track must be submitted along with a non-refundable $1,000 Application Fee and a non-refundable $1,750 Engineering Review Fee. Prior to the executed agreement, the first-year rental payment, deposit, and relevant proof of insurance (outlined in the agreement) are to be submitted prior to execution on behalf of the railroad.

Sewer Force Main Pipeline Railroad Crossing Requirements
A summary of specific sewer force main pipeline railroad crossing requirements include the following items. An additional casing pipe spanning the entire right-of-way length is required for sewer force main pipeline applications. The minimum cover from the base of the rail to the top of the pipe is 25 feet. Flagmen and railroad inspectors will be required

during work within the right-of-way and during the sewer force main installation. The pipeline location will need to be documented during installation and marked with weatherproof signs after it has been installed.

7.10 Desktop Environmental Analysis

A desktop review of environmental resource areas near the route alternatives was performed. The results of this analysis are summarized in a Technical Memo that is attached to this report as Appendix G

7.11 Decommissioning the Existing Treatment Plant

If the treatment plant is converted to just a pump station, then the unused components of the existing treatment facilities should be decommissioned. It is assumed that the following would take place:

- The new pumps would be installed within the existing pump room.
- The Aerated Grit Chambers would be demolished and the mechanisms within the existing Clarigesters would be removed.
- Sludge Pumps/Sludge Dewatering System: this system is currently inoperable and would serve no use under the pump to Windham option. The equipment would be removed and disposed of.
- The visible piping/valves between the RIBS would be demolished, and the top layer of soil within the RIBS would potentially be removed, followed by the installation of clean fill, topsoil and seed.

Primary effluent has been disposed of within each RIB since the treatment plant was constructed in 1982. Prior to the decommissioning of each basin, it is recommended that the top layer of soil within each basin be tested for a variety of pollutants including metals, petroleum based compounds (VOCs, SVOCs), ETPH, and pesticides. The presence of PCBs is unlikely; however, PCB testing should also be performed because if the soil must be removed from the site, the receiving facility will require this information.

If any pollutants are found to be above RSR (Remediation Standard Regulation) limits, then the soil will have to be removed and disposed of in accordance with the requirements for the pollutant in question. The cost evaluation includes an allowance of $6,000 for 12 tests within the existing 8 RIBs. It also conservatively assumed that 1 foot of soil in 50% of the RIBs will have to be removed and disposed of as contaminated material at a cost of $100/ton. Soils that remain in place will be treated with lime for disinfection purposes within the soil removal budget stated above.

The water level within the existing RIBs at the treatment plant was continuing to rise at the time that this report was prepared, and the Town was considering the potential construction of an additional RIB for temporary storage. Therefore, an allowance for the construction of a temporary RIB as discussed in Section 6 (but smaller in size) was included as part of the cost for the Windham connection alternative to be conservative.

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7.12 Staffing Impacts

Under this alternative, no changes in staffing are expected. Duties currently performed in collecting samples and monitoring water levels for permit compliance reporting as well as checking on the RIBS and Clarigesters, and sludge disposal will be replaced by activities associated with maintaining/operating the Chemical feed system and the force main's air and vacuum relief valves. A minimum of two operators will be required to maintain these valves, which will be located in confined spaces.

7.13 Cost Evaluation

7.13.1 Construction Costs

Costs for the installation of the force main along the feasible routing alternatives were developed based upon unit prices from recently bid Tighe & Bond projects, CTDOT 2023 Estimating Guidelines and input received from directional drilling contractors using the following assumptions:

- Installation of two 8-inch diameter force mains.
- Temporary paving will be 2-inches thick within local roads and 4-inches in state roads.
- Final paving will be 4-inches thick in local roads and 9-inches thick in state roads. Full width mill/overlay would also take place on State roads.
- A rock excavation quantity based upon 6" of rock removal along each force main route. The presence of rock and quantity of rock removal must be confirmed during final design.
- Horizontal direction drilling was assumed to cost $600/LF plus an additional $200,000 for the railroad casing. An additional cost of $75/LF was added for the new force main piping to be installed within the casing.
- Air release manholes are required at high points along each route and would be installed at a cost of $15,000 for the valve and manhole.
- Allowances for Clearing and Grubbing, (2%), Mobilization/Demobilization (10%), Maintenance and Protection of Traffic (3.5%).
- Railroad fees of $4,500 per crossing plus $20,000 for railroad flagmen.
- Easement costs of $15,000 per parcel as indicated by the Town of Coventry.
- An allowance for the DOT Encroachment Permit Bond in the amount of $12/$1,000 of contract value.

Costs for the new Pump Station at the existing treatment plant site are based upon the following assumptions:

- New mechanical influent bar screen, with an allowance for a future washer/compacter.
- Three new pumps with VFD controls will be installed in the existing pump room, along with new discharge piping and valves as required.
- Flow meter with SCADA connection to Windham.
- A new generator will be provided, to be installed outside the building.

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- Miscellaneous electrical and building improvements as discussed in Section 6.
- Bioxide odor control system: System dosing installation costs can range from \$60,000 - \$90,000 depending upon the specific application. A unit cost of \$75,000 was assumed for Coventry.
- Decommissioning of the existing RIBS and the temporary RIB.
- Demolition of the existing aerated grit chamber and clarigesters mechanisms.

All construction costs are based on the December 2023 ENR Construction Cost Index of 13514.76.

7.13.2 O&M Costs

Operation and Maintenance costs were estimated as follows:

- The existing Coventry WPCA budget was used as a starting point for each option, as many of the existing costs associated with staffing and maintenance of the collection system, pump station, and treatment plant site will remain. Additions and deductions where applicable to the budget line items were made as detailed below.
- Staffing costs will remain unchanged.
- Chemical costs of \$18,600 per year for force main odor control were added based upon 34 gallons bioxide being used for six months per year at a cost of \$3/gallon.
- Electrical costs for operating the new pumps and related equipment (mechanical screen) were estimated based upon the expected KW usage and operating times at a rate of \$0.15/KWH based upon data from Coventry's 2021-2023 electric bills. Adjustments in usage were also made for equipment shut down at the plant.
- Laboratory and sludge disposal costs will be eliminated.

A life cycle (present worth) cost analysis was performed based upon a 20-year service life, including capital costs, equipment replacement costs, and annual O&M costs. O&M costs are based upon flows, loads, and estimates of labor, energy, and chemical use.

For the purposes of this analysis, it was assumed that flows and loads will increase to 50% of the planned growth 10 years after construction, and the remaining growth will take place by the 20th year.

All operation and maintenance costs were estimated annually over a 20 year planning period, with O&M costs starting one year after the capital costs. The present worth costs are determined based upon a 2.5% discount rate (Department of Interior Federal Water Resources Planning published rate for FY2023) and an assumed 10 year breakeven inflation rate of 2.22% (according to the Federal Reserve December 1, 2023). This method allows the use of current O&M costs in the present worth calculations. The net present worth of the O&M costs was utilized in evaluating the costs of each alternative.

7.13.3 Fees Charged by Windham

As stated in Section 7.1, the Town will be required to pay Windham a connection fee, annual fees for treatment, and share in the cost of any future upgrades at the Windham treatment facility. Costs added into the pumped alternative are as follows:

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Connection Fee:
Windham currently charges connection fees for all new sewer users connecting to the sewer system. The fee structure is as follows:

- $1,800 per single family residence
- $1,400 per unit for multi-family residences
- $900 per 1,000 square feet for commercial and industrial users. Schools, and municipal buildings would also fall into this category.

The initial connection fee was calculated based upon the parcel count and EDU data presented in Table 3-6. All single family parcels were assessed a fee of $1,800. Residential parcels with more than 1 EDU were charged a rate of $1,400 times the number of EDUs. All commercial, industrial and municipal parcels were charged at the commercial/industrial rate with the square footage estimated based upon aerial imaging.

A total connection fee of $2,100,000 was calculated using the methodology discussed above. It should be noted that Windham had indicated (and then confirmed) that connection fees for future connections to the Coventry sewer system would be paid directly to Coventry with no requirement that a fee be paid to Windham. This unusual approach would save Coventry money if all of Coventry's planned growth were to occur and we did not include the cost for future connections to Coventry in our cost analysis.

Annual Usage Fee:
The annual usage fee for treatment at the Windham wastewater treatment facility will be based upon the total amount of flow sent by the Town for treatment. Windham has indicated that the charge rate will be based upon Windham's cost to treat the wastewater, which is currently $6,454 per million gallons.

Flows were estimated to be 150,000 gpd during the first year. It is not known how fast future connections to the system will be constructed. Therefore, for the purposes of this evaluation, it was assumed that flows will gradually increase, with 50% of the future connections taking place by Year 10, and 100% of the future connections taking place by Year 20. An inflation factor of 2.2% was also applied to the annual charge being assessed.

Share of Future Plant/Collection System Upgrade Costs:
The Windham Treatment Facility was originally constructed in 1959, upgraded in 1974 to secondary treatment and in 2010 for nutrient removal. While a major upgrade to the wastewater treatment facility is not currently planned, the Town is beginning a large scale improvement to its collection system, and minor upgrades to the plant take place at regular intervals.

Windham has indicated that the Town of Coventry would be expected to pay for a share of improvements to both the treatment facility and collection system. Specifics of such an agreement, if this option is recommended, would be developed during the negotiation phase.

The payment cost would be based upon the percentage of flow being sent by Coventry. At this time, it is unclear as to whether the percentage would be based on the Windham plant's design flow of 5.5 MGD (2.8% +/-), or the actual flow which is currently 2.0 MGD (7.5%). For the purposes of this evaluation, it was assumed that an improvement project of $2,000,000 would take place at the treatment plant and/or collection system at Years

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5, 10, 15 and 20, with Coventry paying 5.25% of the costs, which is the approximate average of the existing and future flow percentages.

7.13.4 Capital Cost Adders

The following costs were applied to the capital cost of each option:

- 20% Contractor Overhead and Profit
- 30% Contingency
- 20% Engineering

The cost estimates developed are considered to be consistent with Class 4 estimates as defined by the Estimate Classification system of the American Association of the Advancement of Cost Engineering International (AACE International), formerly known as the American Association of Cost Engineers (AACE). The cost estimates were developed without detailed engineering data and are considered approximate. Class 4 estimates are normally expected to be accurate within minus 30 percent to plus 50 percent (-30% / +50%). This range implies that there is a high probability that the final project cost will fall within the range. The contingencies included in these cost estimates are a provision for unforeseeable additional costs within the general bounds of the project scope, particularly where experience has shown that unforeseeable costs are likely to occur. Thus, contingencies are used as a means to reduce the risk of possible cost overruns.

7.13.5 Funding Impacts

The CTDEEP has indicated that the Windham connection option is expected to qualify for a 20% conventional grant due to the fact that it is a wastewater conveyance project. However, DEEP has also stated that the project will be required to go through an eligibility determination in order to establish the final grant percentage. The balance of the project costs would qualify for a 2% loan under DEEP's funding program.

As with the treatment plant costs, BABA Requirements do not apply, but the project will be required to meet AIS requirements.

Construction cost estimates were therefore adjusted based upon the assumption of a 20% grant being awarded.

7.14 Force Main Installation Cost Summary

A summary of the construction costs for the 3 Route Alternatives is presented in Table 7-7, and a detailed summary is presented in Table 7-8. As noted, Alternative 2: Route 31 is the least expensive, followed by Alternative 1: Route 32 and then Alternative 3B: Depot Road.

TABLE 7-7
Opinion of Probable Construction Cost (OPCC) – Summary of Three Routes

Route	Total Cost
Alternative 1: Route 32	$19,410,000
Alternative 2: Route 31	$18,650,000
Alternative 3B: Depot Road	$20,240,000

7.15 Recommended Connection Route

The cost of the three force main routes evaluated are in the range of $18 - $20 million dollars. As shown in the Environmental Desktop Analysis presented in Appendix G, the permitting levels are relatively similar. The effort in obtaining a permit for the Route 31 Alternative may be slightly less since the Willimantic River crossing is proposed to take place via a bridge crossing as opposed to drilling underneath the river.

Inasmuch as the Route 31 Alternative is also the least expensive, it is recommended that this alternative be selected to move into the final comparison phase to determine if Coventry should connect to Windham or upgrade its plant.

This evaluation, along with a calculation of present worth lifecycle costs is presented in Section 8.

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Table 7-8
Force Main Alternatives
Detailed Opinion of Probable Construction Cost

This is an engineer's Opinion of probable Construction Cost (OPCC). Tighe & Bond has no control over the cost or availability of labor, equipment or materials, or over market conditions or the Contractor's method of pricing, and that the estimates of probable construction costs are made on the basis of the Tighe & Bond's professional judgment and experience. Tighe & Bond makes no guarantee nor warranty, expressed or implied, that the bids or the negotiated cost of the Work will not vary from this estimate of the Probable Construction Cost

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		Alternative 1: CT Route 32				**Alternative 2: CT Route 31**				**Alternative 3B: Depot Road (Drilling)**			
Bid Item No.	Bid Item Description	Quantity	Unit	Estimated Unit Cost	Extended Total	Quantity	Unit	Estimated Unit Cost	Extended Total	Quantity	Unit	Estimated Unit Cost	Extended Total
1	Mobilization/Demobilization (10%)	1	LS	$879,000.00	$879,000.00	1	LS	$847,000.00	$847,000.00	1	LS	$915,000.00	$915,000.00
2A	Pump Station Force Main - (State Road)	13700	LF	$160.00	$2,192,000.00	14300	LF	$160.00	$2,288,000.00	15100	LF	$160.00	$2,416,000.00
2B	Pump Station Force Main - (Local Road)	0	LF	$160.00	$0.00	1300	LF	$160.00	$208,000.00	850	LF	$160.00	$136,000.00
2C	Pump Station Force Main - (Off Road)	6,950	LF	$135.00	$938,250.00	5,300	LF	$135.00	$715,500.00	7,200	LF	$135.00	$972,000.00
2D	Pump Station Force Main - (Redundant Drilling)	1,200	LF	$100.00	$120,000.00	600	LF	$100.00	$60,000.00	600	LF	$100.00	$60,000.00
3	Air Release Valve/Manhole	7	Each	$15,000.00	$105,000.00	6	Each	$15,000.00	$90,000.00	7	Each	$15,000.00	$105,000.00
4	Clearing and Grubbing (2%)	1	LS	$176,000.00	$176,000.00	1	LS	$170,000.00	$170,000.00	1	LS	$183,000.00	$183,000.00
5	Maintenance + Protection of Traffic (3.5%)	1	LS	$344,000.00	$344,000.00	1	LS	$336,000.00	$336,000.00	1	LS	$364,000.00	$364,000.00
6	Rock Excavation	1,200	CY	$200.00	$240,000.00	1,200	CY	$200.00	$240,000.00	1,300	CY	$200.00	$260,000.00
7	Sediment and Erosion Control	1	LS	$149,000.00	$149,000.00	1	LS	$116,000.00	$116,000.00	1	LS	$154,000.00	$154,000.00
8	Process Gravel Subbase	4,123	CY	$60.00	$247,375.00	4,514	CY	$60.00	$270,850.00	4,800	CY	$60.00	$288,000.00
9	Granular Backfill Material	2,199	CY	$60.00	$131,933.33	2,408	CY	$60.00	$144,453.33	2,560	CY	$60.00	$153,600.00
10	Temporary Pavement	2,256	Tons	$160.00	$360,960.00	2,363	Tons	$160.00	$378,080.00	2,557	Tons	$160.00	$409,120.00
11	Permanent Pavement - State Road	6,638	Tons	$160.00	$1,062,080.00	6,638	Tons	$160.00	$1,062,080.00	7,316	Tons	$160.00	$1,170,560.00
11A	Permanent Pavement - Local Road	0	Tons	$160.00	$0.00	264	Tons	$160.00	$42,240.00	173	Tons	$160.00	$27,680.00
11B	State Road Milling	40,417	SY	$5.00	$202,085.00	38,850	SY	$5.00	$194,250.00	44,606	SY	$5.00	$223,030.00
12	State Road Overlay (Full Width)	6,912	Tons	$160.00	$1,105,920.00	6,644	Tons	$160.00	$1,063,040.00	7,628	Tons	$160.00	$1,220,480.00
13	Pavement Markings (4" White/Yellow Epoxy)	55,900	LF	$1.50	$83,850.00	58,800	LF	$1.50	$88,200.00	63,200	LF	$1.50	$94,800.00
14	Off Road Restoration	11,584	SY	$40.00	$463,360.00	8,834	SY	$40.00	$353,360.00	12,000	SY	$40.00	$480,000.00
15	Guide Rail Replacement	1,200	LF	$30.00	$36,000.00	1,800	LF	$30.00	$54,000.00	1,200	LF	$30.00	$36,000.00
16	Asphalt Curb Replacement	4,600	LF	$10.00	$46,000.00	1,400	LF	$10.00	$14,000.00	4,600	LF	$10.00	$46,000.00
17	WWTP HDD under Willimantic River	1	LS	$360,000.00	$360,000.00	0	LS	$360,000.00	$0.00	0	LS	$360,000.00	$0.00
17A	WWTP HDD under R/R	1	LS	$560,000.00	$560,000.00	0	LS	$560,000.00	$0.00	0	LS	$560,000.00	$0.00
17B	CT-31 Willimantic River Bridge Attachment	0	LS	$200,000.00	$0.00	1	LS	$200,000.00	$200,000.00	0	LS	$200,000.00	$0.00
17C	CT-31 R/R HDD	0	LS	$560,000.00	$0.00	1	LS	$560,000.00	$560,000.00	0	LS	$560,000.00	$0.00
17D	Depot Road HDD under R/R	0	LS	$380,000.00	$0.00	0	LS	$380,000.00	$0.00	1	LS	$380,000.00	$380,000.00
17E	Depot Road HDD under Willimantic River	0	LS	$180,000.00	$0.00	0	LS	$180,000.00	$0.00	1	LS	$180,000.00	$180,000.00
17F	CT-32 Cider Mill Brook Bridge Crossing Add on	300	LF	$150.00	$45,000.00	1	LS	$50,000.00	$50,000.00	1	LS	$50,000.00	$50,000.00
17G	CT-6 Culvert Crossings Add On	600	LF	$150.00	$90,000.00	2	Each	$2,500.00	$5,000.00	2	Each	$2,500.00	$5,000.00
18	Trafficmen	2,480	Hour	$90.00	$223,200.00	2,704	Hour	$90.00	$243,360.00	2,824	Hour	$90.00	$254,160.00
19	Railroad Flagmen	10	Shift	$2,000.00	$20,000.00	10	Shift	$2,000.00	$20,000.00	10	Shift	$2,000.00	$20,000.00
20	Easements				$30,000.00				$0.00				$45,000.00
21	Railroad Company Fees				$4,500.00				$4,500.00				$4,500.00
22	DOT Encroachment Permit Bond				$150,000.00				$144,000.00				$156,000.00
				Subtotal:	$10,365,513.33			Subtotal:	$9,961,913.33			Subtotal:	$10,808,930.00
				20% Contractor Overhead	$2,074,000.00			20% Contractor Overhead:	$1,993,000.00			20% Contractor Overhead:	$2,162,000.00
				Subtotal	$12,439,513.33			Subtotal	$11,954,913.33			Subtotal	$12,970,930.00
				30% Contingency	$3,731,854.00			30% Contingency:	$3,586,474.00			30% Contingency:	$3,891,279.00
				Subtotal	$16,171,367.33			Subtotal	$15,541,387.33			Subtotal	$16,862,209.00
				20% Engineering Cost	$3,234,273.47			20% Engineering Cost	$3,108,277.47			20% Engineering Cost	$3,372,441.80
				Total:	$19,405,640.80			Total:	$18,649,664.80			Total:	$20,234,650.80
				SAY:	**$19,410,000.00**			**SAY:**	**$18,650,000.00**			**SAY:**	**$20,240,000.00**

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Section 8 Development of Recommended Alternative

8.1 Alternatives to be Evaluated

As stated in Section 6, the decision was made to evaluate life cycle costs of both the SBR and Oxidation Ditch Treatment Plant Upgrade Alternatives. Section 7 concluded that the life cycle costs of the Windham alternative be based on the Route 31 force main route and then compared against the treatment plant Alternative.

8.2 Summary of Total Life Cycle Project Costs

Life cycle costs for all three alternatives were developed as discussed in Sections 6 and 7.

- Capital costs for the treatment plant alternatives include all initial construction. The Windham connection capital costs include the force main construction cost presented in Table 7-8 plus the costs of work needed to construct the new pump station at the treatment plant site, as well as initial construction fees that would have to be paid to Windham.
- Operation and Maintenance costs represent the net present worth of expected costs over the 20 year planning period for this project.
- An estimate of potential grants to be received from the CT DEEP was subtracted off of each option based upon the qualifying percentages discussed earlier.

A summary of the total life cycle costs for the three alternatives is presented in Table 8-1. A more detailed breakdown of the capital construction cost line items is included in Tables 8-2, 8-3 and 8-4. Note that the totals in Tables 8-2, 8-3 and 8-4 do not have the 10% markup factor applied to construction costs in Table 8-1, which explains the difference in totals for each alternative..

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8.3 Recommended Alternative and Costs

As indicated in Table 8-1 the present worth life cycle cost of the Windham alternative is much more costly than upgrading the plant and going to a river discharge. Therefore, it is recommended that the Windham alternative be dropped.

It should be noted that the planning level capital costs and present worth O&M costs include many assumptions. These are discussed in detail in the appropriate sections above. For the on-site treatment plant alternative, conservative assumptions and opportunities for value engineering were discussed that could result in the lowering of the cost to Coventry. Scaling back future growth plans could also further reduce capital costs for the on-site treatment alternative. For the Windham alternative, it was noted that there is a less than conservative assumption and risk for increased costs in regard to connection fees for future Coventry users. Scaling back future growth plans would have little cost impact on capital cost for the Windham alternative. These points further support the conclusion that pursuing on-site treatment is in Coventry's best interest.

As indicated in Table 8-1, the present worth cost of the two on-site treatment alternatives are nearly identical and the difference is small compared to the expected accuracy of these planning level cost estimates.

Table 8-1
Detailed Life Cycle Cost Analysis Summary

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			Alternative #1 **Discharge to the River** Sequencing Batch Reactor		Alternative #2 **Discharge to the River** Oxidation Ditch		Alternative #3 **Force Main to Windham**	
Item	Category		% Grant	Cost	% Grant	Cost	% Grant	Cost
	Contract #1 Plant Construction Costs							
1	General Conditions (10% of Items 2 to 19)[1]		23%	$1,295,000	23%	$1,313,000	20%	$264,000
2	Construct Temporary RIB		25%	$170,000	25%	$170,000	20%	$136,000
3	Decommissioing of RIBs		20%	$444,000	20%	$444,000	20%	$438,000
4	Demolition & Site Civil - General		20%	$547,000	20%	$377,000	20%	$170,000
5	Demolition & Site Civil - Secondarys		25%	$1,236,000	25%	$528,000		
6	Outfall to River from Main Plant Area		20%	$460,000	20%	$460,000		
7	Concrete Tanks - Secondary Treatment		25%	$3,234,000	25%	$3,678,000		
8	Concrete Tanks - Solids Handling		20%	$404,000	20%	$679,000		
9	Concrete - Other		20%	$39,000	20%	$30,000	20%	$5,000
10	Plant Process Improvements - Headworks		20%	$731,000	20%	$731,000	20%	$812,000
11	Plant Process Improvements - Primary Clarifiers/Storage		20%	$910,000	20%	$300,000		
12	Plant Process Improvements - Secondary Equipment Installed		25%	$1,846,000	25%	$2,942,000		
13	Plant Process Improvements - Additional Process Equipment		20%	$84,000	20%	$84,000		
14	Plant Process Improvements - Solids Handling Equipment		20%	$903,000	20%	$903,000		
15	Plant Process Improvements - Chemical Feed System Equipment		30%	$89,000	30%	$89,000	20%	$141,000
16	Plant Process Improvements - Disinfection Equipment		20%	$269,000	20%	$255,000		
17	Other Plant Improvements		20%	$311,000	20%	$311,000	20%	$311,000
18	Electric & Controls Improvements		20%	$537,000	20%	$531,000	20%	$630,000
19	Electric & Controls Improvements - Secondary Treatment		25%	$739,000	25%	$614,000		
20	**Subtotal (Above Items)**			**$14,200,000**		**$14,400,000**		**$2,900,000**
21	Contractor Overhead & Profit[1]	at 20%	23%	$2,840,000	23%	$2,880,000	20%	$580,000
22	Subtotal (Above 2 Items)			$17,040,000		$17,280,000		$3,480,000
23	Contingency [1]	at 30%	23%	$5,120,000	23%	$5,190,000	20%	$1,050,000
24	Subtotal (Above 2 Items)			$22,160,000		$22,470,000		$4,530,000
25	Engineering Cost[1]	at 20%	23%	$4,440,000	23%	$4,500,000	20%	$910,000
26	**Total Contract #1 Plant Construction Costs (Above 2 Items)**			**$26,600,000**		**$26,970,000**		**$5,440,000**
27	**Total Contract #2 Force Main Costs**			**$0**		**$0**	20%	**$18,650,000**
28	**Connection Fee for Current Customers - Paid to Windham**			**$0**		**$0**	0%	**$2,100,000**
29	**Total Capital Costs (Above 3 Items, Rounded)**			**$26,600,000**		**$27,000,000**		**$26,200,000**
	O&M Costs (20 Years Present Worth)							
30	Current WPCA Budget (Year 0 @ $502,000)			$9,757,000		$9,757,000		$9,757,000
31	Additional Staffing Costs			$2,449,000		$2,099,000		$0
32	Additional Chemical Costs			$120,000		$127,000		$363,000
33	Additonal Electrical Utility Costs			$511,000		$618,000		$15,000
34	Additonal Permit Compliance Costs (Lab, Permit fees etc)			$391,000		$371,000		($207,000)
35	Additional Sludge Disposal Costs			$1,507,000		$1,339,000		($606,000)
36	Additional Equipment Maintenance			$861,000		$861,000		$328,000
37	Sewer User Fees - Paid to Windham			$0		$0		$8,214,000
38	Allowance for Capital Improvements Assessments - Paid to Windham			$0		$0		$406,000
39	Future Sewer Connection Fees - Paid to Windham			$0		$0		$0
40	**Total O&M Costs (20 Years Present Worth)**			**$15,600,000**		**$15,200,000**		**$18,300,000**
41	**Total Present Worth Capital & O&M Costs (Items 29+40)**			**$42,200,000**		**$42,200,000**		**$44,500,000**
42	Potential Grants (Based on Percentages given above, rounded)		23%	$6,100,000	23%	$6,200,000	18%	$4,820,000
43	**Present Worth Cost to Town after Grants (Item 41-42)**			**$36,100,000**		**$36,000,000**		**$39,680,000**
44	**Town Capital Costs less Potential Grants (Item 29-42)**			**$20,500,000**		**$20,800,000**		**$21,380,000**

Notes
1 - Grant percentage for these items based on weighted average of grant eligibility for other items

Table 8-2
Sequencing Batch Reactor
Detailed Opinion of Probable Construction Cost

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Item	Category	Description	Unit Price	Quan	Units	Materials/ Equipment Cost	Equipment Installation	Total Cost
1	**General Conditions (10% of Items Below)**							
2	**Construct Temporary RIB**							
		Land Taking	$0	1	LS	$0		$0
		Land Clearing	$10,000	1	LS	$10,000		$10,000
		Grading/Excavation	$30	3340	CY	$100,200		$100,000
		Fencing	$50	580	LF	$29,000		$29,000
		Fence Gate to RIB	$1,000	1	LS	$1,000		$1,000
		Temporary RIB material	$30,000	1	LS	$30,000		$30,000
3	**Decommissioning of RIBS**							
		Decommissioning of Existing RIBs	$416,000	1	LS	$416,000		$416,000
		Decommissioning of Temporary RIB	$28,000	1	LS	$28,000		$28,000
4	**Demolition & Site Civil - General**							
		Demolition in Existing Building	$25,000	1	LS	$25,000		$25,000
		Demolition of Aerated Grit and Clarigester Mechanisms	$100,000	1	LS	$100,000		$100,000
		Bypass Pumping Allowance	$50,000	1	LS	$50,000		$50,000
		Excavation - Solids Handling	$50	1668	CY	$83,400		$83,000
		UV and Blower Building	$86,589	1	LS	$86,589	$34,636	$121,000
		Site Piping (also Air) including trenching	$167,500	1	LS	$167,500		$168,000
5	**Demolition & Site Civil - Secondarys**							
		Sheeting - SBRs for Deep 75 by 120 foot area	$60	12000	SF	$720,000		$720,000
		Dewatering	$100,000	1	LS	$100,000		$100,000
		Excavation - SBRs	$50	6672	CY	$333,600		$334,000
		Stairs/Steps	$20,000	1	LS	$20,000		$20,000
		Site Piping including trenching	$61,500	1	LS	$61,500		$62,000
6	**Outfall to River from Main Plant Area**							
		Rehab Existing Pipe Between RIBS - Asssume 16" Exist. D.I. Lines	$150	700	LF	$105,000		$105,000
		Bypass Pumping Allowance During Relining	$10,000	1	LS	$10,000		$10,000
		HDD Drilled Line to River and outfall structure	$345,000	1	LS	$345,000		$345,000
7	**Concrete Tanks - Secondary Treatment**							
		Concrete - SBR and Post EQ Tank Bottom Slab	$1,500	700	CY	$1,050,000		$1,050,000
		Concrete - SBR and Post EQ Tank Walls	$2,000	912	CY	$1,824,000		$1,824,000
		Concrete - SBR and Post EQ Tank Elev Slabs	$2,000	57	CY	$114,963		$115,000
		Concrete - Pad for SBR Blowers	$1,000	10	CY	$10,000		$10,000
		Concrete - Precast Valve Vault	$30,000	1	LS	$30,000		$30,000
		Hand Railings - Around Tanks	$250	800	LF	$200,000		$200,000
		Painting - Valve Vault	$5,000	1	LS	$5,000		$5,000
8	**Concrete Work - Solids Handling Tanks & Access**							
		Concrete - Tank Bottom Slab	$1,500	135	CY	$202,733		$203,000
		Concrete - Tank Walls	$2,000	88	CY	$176,586		$177,000
		Concrete - Pad for WAS Tank Blowers	$1,500	3	CY	$4,500		$5,000
		Railings - Around WAS Tank	$250	75	LF	$18,750		$19,000
9	**Concrete - Other**							
		Concrete Pad for New Electrical Generator	$5,000	1	LS	$5,000		$5,000
		Structural Improvements - Chemical Containment Areas	$20,000	1	LS	$20,000		$20,000
		Plant Water Holding Tank	$10,000	1	LS	$10,000	$4,000	$14,000
10	**Plant Process Improvements - Headworks**							
		Mechanical Bar Screen	$235,270	1	EA	$235,270	$94,108	$329,000
		Possible Future Washer Compactor	$65,930	1	EA	$65,930	$26,372	$92,000
		Influent Pumps	$27,217	3	EA	$81,650	$32,660	$114,000
		Piping and Valves and supports	$100,000	1	LS	$100,000	$40,000	$140,000
		Replace Aerated Grit Blowers	$15,000	2	EA	$30,000	$12,000	$42,000
		Replace Diffusers - Aerated Grit	$10,000	1	LS	$10,000	$4,000	$14,000
11	**Plant Process Improvements - Primary Clarifiers/Storage**							
		Rehabilitate Clarigesters - New Mechanisms	$200,000	2	EA	$400,000	$160,000	$560,000
		Rehabilitate Clarigesters - Piping Modifications/Struct Repair	$150,000	2	EA	$300,000		$300,000
		Piping & Valves to allow connection to truck loading pump	$50,000	1	LS	$50,000		$50,000

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Table 8-2 (Continued)
Sequencing Batch Reactor OPCC

12	**Plant Process Improvements - Secondary Equipment Installed**	*$1,054,010*					
	Aqua Aerobics SBR Equipment Package and Services	$1,212,111	1	LS	$1,212,111	$484,844	$1,697,000
	Slide Gates for SBR & Post Eq Drains	$8,000	4	EA	$32,000	$12,800	$45,000
	Portable Submersible Pump to Fully Drain the Tanks	$30,000	1	EA	$30,000	$12,000	$42,000
	Foam Spray System - Allowance	$33,000	1	LS	$33,000	$13,200	$46,000
	Portable Hoists	$2,000	1	LS	$2,000		$2,000
	Pipe and Equipment Support	$10,000	1	LS	$10,000	$4,000	$14,000
13	**Plant Process Improvements - Additional Process Equipment**						
	Plant Water System (2 Pumps, HP Tank)	$50,000	1	LS	$50,000	$20,000	$70,000
	Pipe Support and Equipment Supports	$10,000	1	LS	$10,000	$4,000	$14,000
14	**Plant Process Improvements - Solids Handling Equipment**						
	RDT	$247,250	1	EA	$247,250	$98,900	$346,000
	RDT Feed Pumps	$20,240	1	EA	$20,240	$8,096	$28,000
	Thickened Sludge Transfer Pump	$16,905	1	EA	$16,905	$6,762	$24,000
	Truck Loading Pumps	$29,670	1	EA	$29,670	$11,868	$42,000
	PD Blowers (w/ manual Butterfly valves)	$28,750	2	EA	$57,500	$23,000	$81,000
	Coarse Bubble Diffuser Set	$45,000	1	LS	$45,000	$18,000	$63,000
	Polymer System	$30,000	1	EA	$30,000	$12,000	$42,000
	Allowance For Valves, including Automatic Valve	$50,000	1	LS	$50,000	$20,000	$70,000
	Odor Control System	$42,360	1	EA	$42,360	$16,944	$59,000
	TWAS Tank Removable FRP Cover	$91,701	1	LS	$91,701	$36,680	$128,000
	Painting - Related Interior Walls and Piping	$20,000	1	LS	$20,000		$20,000
15	**Plant Process Improvements - Chemical Feed System Equipment**						
	Safety Shower Tepid Supply Package System	$26,000	1	LS	$26,000		$26,000
	Safety Shower and Related Piping	$2,000	1	LS	$2,000		$2,000
	Bulk Tank for PAC - 450 Gallons	$6,000	2	EA	$12,000	$4,800	$17,000
	Chemical Feed Pump for PAC	$7,800	2	EA	$15,600	$6,240	$22,000
	Related Piping/ Fill Ports / Related Instumentation	$10,000	1	LS	$10,000	$4,000	$14,000
	Painting - Chemical Containment	$8,000	1	LS	$8,000		$8,000
16	**Plant Process Improvements - Disinfection Equipment**						
	UV System Train - In Pipe	$75,929	2	EA	$151,858	$60,743	$213,000
	Protective Structure	$40,000	1	LS	$40,000	$16,000	$56,000
17	**Other Plant I (First 3 Items below same as Windham)**						
	Existing Building Roof Replacement	$200,000	1	LS	$200,000		$200,000
	General Improvements - New Elec Room incl HVAC	$50,000	1	LS	$50,000		$50,000
	New Door, Exterior repairs/stairs, painting, floor coating	$61,000	1	LS	$61,000		$61,000
18	**Electric & Controls Improvements**						
	Instrumentation - Headworks & Plant Water	$25,000	1	LS	$25,000		$25,000
	Instrumentation - Solids Handling	$25,000	1	LS	$25,000		$25,000
	Flow Meter - Effluent Ultrasonic for Parshall Flume	$10,000	1	EA	$10,000	$4,000	$14,000
	New Headworks/Misc Control System with Mission Monitoring	$60,000	1	LS	$60,000		$60,000
	VFDs - Influent pumps	$22,000	3	EA	$66,000		$66,000
	VFDs - Sludge Equipment	$31,900	1	LS	$31,900		$32,000
	New Generator & ATS	$90,500	1	LS	$90,500		$91,000
	Misc. Electrical (W & C) Dist Equipment	$224,100	1	LS	$224,100		$224,000
19	**Electric & Controls Improvements - Secondary Treatment**						
	VFDs - SBR Blowers	$28,867	3	EA	$86,600		$87,000
	VFDs - EQ Pumps	$9,433	3	EA	$28,300		$28,000
	VFDs - EQ Blower	$9,500	1	EA	$9,500		$10,000
	New Generator & ATS	$90,500	1	LS	$90,500		$91,000
	Misc. Electrical (W & C) Dist Equipment, Site Lighting	$522,900	1	LS	$522,900		$523,000
20	**Subtotal of Above Items**					**Sum Above**	**$12,953,000**

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Table 8-3
Oxidation Ditch
Detailed Opinion of Probable Construction Cost

Tighe&Bond
Engineers | Environmental Specialists

Item	Category	Description	Unit Price	Quan	Units	Materials/ Equipment Cost	Equipment Installation	Total Cost
1	**General Conditions (10% of Items Below)**							
2	**Construct Temporary RIB**							
		Same as SBRs	$170,000	1	LS	$170,000		$170,000
3	**Decommissioing of RIBS**							
		Same as SBRs	$444,000	1	LS	$444,000		$444,000
4	**Demolition & Site Civil Improvements - General**							
		Demolition in Existing Building	$25,000	1	LS	$25,000		$25,000
		Demolition of Aerated Grit and Clarigester Mechanisms	$100,000	1	LS	$100,000		$100,000
		Bypass Pumping Allowance	$50,000	1	LS	$50,000		$50,000
		Excavation	$50	1480	CY	$74,000		$74,000
		Site Piping including trenching	$128,125	1	LS	$128,125		$128,000
5	**Demolition & Site Civil Improvements - Secondarys**							
		Sheeting Allowance	$150,000	1	LS	$150,000		$150,000
		Excavation - Ditch	$50	2392	CY	$119,600		$120,000
		Excavation - Clarifiers	$50	930	CY	$46,500		$47,000
		Site Piping including trenching	$211,250	1	LS	$211,250		$211,000
6	**Outfall to River from Main Plant Area**							
		Same as SBR Option	$460,000	1	LS	$460,000		$460,000
7	**Concrete Tanks - Secondary Treatment**							
		Concrete - Ox Ditch Bottom Slab	$1,500	734	CY	$1,101,000		$1,101,000
		Concrete - Ox Ditch Tank Walls	$2,000	639	CY	$1,278,000		$1,278,000
		Concrete - Ox Ditch Elevated Slab	$2,000	103	CY	$206,000		$206,000
		Concrete - Clarifier Bottom Slab	$1,500	205	CY	$307,500		$308,000
		Concrete - Clarifier Tank Walls	$2,000	141	CY	$282,000		$282,000
		Concrete - Clarifier Launder	$2,000	30	CY	$60,000		$60,000
		Precast Concrete - Secondary Clarifier Scum Pit	$10,000	1	LS	$10,000	$4,000	$14,000
		Precast Concrete - Wet Wells for RAS/WAS Pumps	$15,000	2	EA	$30,000	$12,000	$42,000
		Precast Concrete - Valve Vaults for RAS/WAS Pumps	$20,000	2	EA	$40,000	$16,000	$56,000
		Railings - Around Clarifiers & Process Tanks	$250	1290	LF	$322,500		$323,000
		Painting - Valve Vaults	$8,000	1	LS	$8,000		$8,000
8	**Concrete Tanks - Solids Handling**							
		Concrete - Tank Bottom Slab	$2,000	115	CY	$230,393		$230,000
		Concrete - Tank Walls	$3,000	140	CY	$418,716		$419,000
		Concrete - Pad for WAS Tank Blowers	$1,500	3	CY	$4,500		$5,000
		Railings - Around WAS Tank	$250	100	LF	$25,000		$25,000
9	**Concrete - Other**							
		Concrete - Pad for New Electrical Generator	$5,000	1	LS	$5,000		$5,000
		Concrete - Plant Water Holding Tank	$10,000	1	LS	$10,000		$10,000
		Concrete Pad for SBR Blowers	$1,500	10	CY	$15,000		$15,000
10	**Plant Process Improvements - Headworks**							
		Same as for SBR	$731,000	1	LS	$731,000		$731,000
11	**Plant Process Improvements - Primary Clarifiers/Storage**							
		Allowance Submersible Drain pumps to Clarigesters	$75,000	2	EA	$150,000		$150,000
		Clarigester Minor Repairs @ 1/2 that for SBR	$75,000	2	EA	$150,000		$150,000

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Table 8-3 (Continued)
Oxidation Ditch OPCC

12	**Plant Process Improvements - Secondary Equipment Installed**						
	Ovivo Oxidation Ditch Equipment Package	$1,092,500	1	LS	$1,092,500	$437,000	$1,530,000
	Ovivo Clarifier Mechanism Equipment Package	$655,500	1	LS	$655,500	$262,200	$918,000
	Slide Gates - Influent Splitter Box	$5,000	4	EA	$20,000	$8,000	$28,000
	Slide Gates - OD Drains	$8,000	4	EA	$32,000	$12,800	$45,000
	Gate Valves - Buried for Draining Clarfiiers	$8,000	2	EA	$16,000	$6,400	$22,000
	Slide Gates (Weir) OD Effluent 8 ft	$30,000	2	EA	$60,000	$24,000	$84,000
	Mixer - Clarifier Distribution Box	$5,000	1	EA	$5,000	$2,000	$7,000
	Pump - Submersible WAS/RAS Sludge Transfer	$10,000	4	EA	$40,000	$16,000	$56,000
	Pump - Sump Pipe Type to prvde spray to center of clarifier	$3,000	2	EA	$6,000	$2,400	$8,000
	FRP - Clarifier Launder Cover Set	$33,707	2	EA	$67,414	$26,966	$94,000
	FRP - Clarifier Density Panels	$21,856	2	EA	$43,712	$17,485	$61,000
	Pump - Submersible Scum Decant Pump	$10,000	1	LS	$10,000	$4,000	$14,000
	Allowance for post aeration to raise Oxygen	$75,000	1	LS	$75,000		$75,000
13	**Plant Process Improvements - Additional Process Equipment**						
	Same as SBR Option	$84,000	1	LS	$84,000		$84,000
14	**Plant Process Improvements - Solids Handling Equipment**						
	Same as SBR Option	$903,000	1	LS	$903,000		$903,000
15	**Plant Process Improvements - Chemical Feed System Equipment**						
	Same as SBR Option	$89,000	1	LS	$89,000		$89,000
16	**Plant Process Improvements - Disinfection Equipment**						
	UV System Train - Open Channel	$52,157	2	EA	$104,314	$41,726	$146,000
	Parshal Flume	$3,000	1	EA	$3,000	$1,200	$4,000
	Protective Structure/Building	$75,000	1	LS	$75,000	$30,000	$105,000
17	**Other Plant Improvements**						
	Same as SBR Option	$311,000	1	LS	$311,000		$311,000
18	**Electric & Controls Improvements**						
	Instrumentation - Headworks & Plant Water	$25,000	1	LS	$25,000		$25,000
	Instrumentation - Solids Handling	$25,000	1	LS	$25,000		$25,000
	Flow Meter - RAS Pumps & WAS	$3,000	3	EA	$9,000	$3,600	$13,000
	Flow Meter - Effluent Ultrasonic for Parshall Flume	$12,000	1	EA	$12,000	$4,800	$17,000
	New Headworks/Misc Control System with Mission Monitoring	$60,000	1	LS	$60,000		$60,000
	VFDs - Influent pumps	$22,000	3	EA	$66,000		$66,000
	VFDs - Sludge Equipment	$31,900	1	LS	$31,900		$32,000
	New Generator & ATS	$83,500	1	LS	$83,500		$84,000
	Misc. Electrical (W & C) Dist Equipment	$209,400	1	LS	$209,400		$209,000
19	**Electric & Controls Improvements - Secondary Treatment**						
	VFDs - Aerator/Mixers - Included in Equip Vendor costs						
	VFDs - RAS/WAS Pumps	$10,200	4	EA	$40,800		$41,000
	New Generator & ATS	$83,500	1	LS	$83,500		$84,000
	Misc. Electrical (W & C) Dist Equipment, Site Lighting	$488,600	1	LS	$488,600		$489,000
20	**Subtotal of Above Items**					**Sum Above**	**$13,126,000**

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Table 8-4
Windham Connection: Treatment Plant Site Improvements
Detailed Opinion of Probable Construction Cost

Tighe&Bond
Engineers | Environmental Specialists

Item	Category	Description	Unit Price	Quan	Units	Materials/ Equipment Cost	Equipment Installation	Total Cost
1	**General Conditions (10% of Items Below)**							
2	**Construct Temporary RIB**							
		80% of what was used for SBR	$136,000	1	LS	$136,000		$136,000
3	**Decommissioing of RIBS**							
		Decommissioning of Existing RIBs	$416,000	1	LS	$416,000		$416,000
		Decommissioning of Temporary RIB	$22,400	1	LS	$22,400		$22,000
4	**Demolition & Site Civil - General**							
		Demolition in Existing Building	$20,000	1	LS	$20,000		$20,000
		Demolition of Aerated Grit Chamber/Clarigester Mechanisms	$100,000	1	LS	$100,000		$100,000
		Bypass Pumping Allowance	$50,000	1	LS	$50,000		$50,000
5	**Demolition & Site Civil - Secondarys**							
6	**Outfall to River from Main Plant Area**							
7	**Concrete Tanks - Secondary Treatment**							
8	**Concrete Tanks - Solids Handling**							
9	**Concrete - Other**							
		Concrete Pad for New Electrical Generator	$5,000	1	LS	$5,000		$5,000
10	**Plant Process Improvements - Headworks**							
		Mechanical Bar Screen	$235,270	1	EA	$235,270	$94,108	$329,000
		Possible Future Washer Compactor	$65,930	1	EA	$65,930	$26,372	$92,000
		Influent Pumps	$43,000	3	EA	$129,000	$51,600	$181,000
		Misc. Piping and Valves and Supports	$100,000	1	LS	$100,000	$40,000	$140,000
		Motorized Valves	$25,000	2	EA	$50,000	$20,000	$70,000
11	**Plant Process Improvements - Primary Clarifiers/Storage**							
12	**Plant Process Improvements - Secondary Equipment Installed**							
13	**Plant Process Improvements - Additional Process Equipment**							
14	**Plant Process Improvements - Solids Handling Equipment**							
15	**Plant Process Improvements - Chemical Feed System Equipment**							
		Safety Shower Tepid Supply Package System	$26,000	1	LS	$26,000		$26,000
		Safety Shower and Related Piping	$2,000	1	LS	$2,000		$2,000
		Chemical Feed Systems for Odor Control Sytem	$75,000	1	LS	$75,000	$30,000	$105,000
		Painting - Chemical Containment	$8,000	1	LS	$8,000		$8,000
16	**Plant Process Improvements - Disinfection Equipment**							
17	**Other Plant Improvements**							
		Existing Building Roof Replacement	$200,000	1	LS	$200,000		$200,000
		General Improvements - New Elec Room incl HVAC	$50,000	1	LS	$50,000		$50,000
		New Door, Exterior repairs/stairs, painting, floor coating	$61,000	1	LS	$61,000		$61,000
18	**Electric & Controls Improvements**							
		New Headworks Instrumentation	$20,000	1	LS	$20,000		$20,000
		New Headworks/Misc Control System with Mission Monitoring	$50,000	1	LS	$50,000		$50,000
		Influent Pump - VFDs	$22,000	3	EA	$66,000		$66,000
		New Generator & ATS	$105,000	1	LS	$105,000		$105,000
		Misc. Electrical @ 20% of Process	$389,000	1	LS	$389,000		$389,000
19	**Electric & Controls Improvements - Secondary Treatment**							
20	**Subtotal of Above Items**						**Sum Above**	**$2,643,000**

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As proposed in Technical Memorandum #3, the recommended technology for the river discharge would be based on both costs and other subjective criteria. In order to recommend a technology for on-site treatment and river discharge, the relative dollar cost score was developed by dividing both of the life cycle costs for each alternative by the highest cost. As shown in Table 8-5, the numbers are identical, so each technology receives a dollar cost score of 1.0.

TABLE 8-5
Alternatives Comparison Table

Costs	SBR	Oxidation Ditch
Capital Cost	$26,600,000	$27,000,000
20 Yr - O&M Cost	$15,600,000	$15,200,000
Total Life Cycle Cost	$42,200,000	$42,200,000
Relative "Dollar Cost" Score	42.2/42.2 = 1.0	42.2/42.2 = 1.0

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The relative subjective cost score developed in Table 6-2 was then combined with the dollar cost share. To combine these two relative costs, a weighting of 70% for the "Dollar Costs" and 30% for the "Subjective Costs" was assumed, with the lowest total value being the recommended alternative. CT DEEP has accepted this weighting for the selection of technologies for phosphorus upgrades subjective in recent projects in Vernon and Southington. This method/evaluation is summarized in Table 8-6 and concludes that the technology with the lowest total relative cost is the Oxidation Ditch.

TABLE 8-6
Alternatives Scoring Comparison Table

Relative Cost Score	SBR	Oxidation Ditch
Dollar (From Table 8-5)	1.00	1.00
Subjective (from Table 6-2)	1.00	0.45
Total Relative Cost = 0.7*Dollar +0.3*Subjective	**1.00**	**0.84**

Based on this above analysis, on-site treatment with a river discharge using oxidation ditch technology is the recommended approach for addressing the current concerns with the operation of the existing RIBs at the Coventry Plant.

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100% Recyclable