

REGIONAL WASTEWATER TREATMENT CONSOLIDATION STUDY

Final Report

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PREPARED FOR

Naugatuck Valley Council of Governments

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1.0 WASTEWATER REGIONALIZATION STUDY SUMMARY

1.1 BACKGROUND AND OVERVIEW

The Naugatuck Valley Council of Governments (NVCOG) commissioned a wastewater treatment study involving five municipalities in the region: Derby, Ansonia, Seymour, Beacon Falls, and Naugatuck. The study was funded through the Connecticut Office of Policy and Management (OPM), under the Regional Performance Incentive Program, under GCS Section 4-124s. The primary goal of this study was to identify the potential for greater economic efficiencies and cost savings through regionalization of wastewater treatment.

Under the existing setup, each of the five communities in the study area has its own water pollution control facility, along with associated collection systems consisting of sewers and pumping stations. Regionalization alternatives, which would combine treatment to reduce the number of plants, offers the potential to reduce capital and operating expenses for the local communities through consolidating infrastructure and sharing staff resources.

This regionalization study has been performed in two phases.

1.1.1 Study Phases

In the first phase of the study, the population and wastewater flow rates and loading for each of the five communities were projected over a 20-year planning period. Also, initial budgetary level capital expenditure projections over that planning horizon were identified for the “base case” scenario of no regionalization. During this first phase of the study, a “long list” of 23 regionalization alternatives was identified for further study.

During the second phase of this study, the long list of 23 alternatives was reduced to a “short list” of the six alternatives considered to be the most advantageous. These six alternatives were then developed further, to assess feasibility as well as projected capital and operating costs. A Recommended Regional Alternative was selected after comparing the present worth costs (which includes capital costs and operations and maintenance costs) of each short-listed regionalization alternative with the base case of no regionalization. Final development of the Recommended Regional Alternative was conducted as a part of Task 4 and is summarized in this report.

1.1.2 Workshops

A series of interactive workshops were conducted at strategic milestones throughout the study to present tentative findings and get input from representatives of the five communities, NVCOG, OPM and the Connecticut Department of Energy and Environment (DEEP). The following workshops were held in connection with this study.

- Workshop No. 1: Held on May 30, 2018, to present population projections and recent flow data, and to get input from the stakeholders before developing the long list of regionalization alternatives.
- Workshop No. 2: Held on December 11, 2018, to discuss preliminary assessment of the condition of existing infrastructure in each of the five communities, and capital

improvements required over the two-year planning period if no regionalization is implemented.

- Workshop No. 3: Held on June 25, 2020, to present and discuss the short list of six regionalization alternatives to be further developed.
- Workshop No. 4: Held on February 6, 2021, to present and discuss the recommended alternative.

It is also noted that numerous other informational meetings were held with the communities during the course of this study. These meetings provided opportunity to review the project objectives, initial findings and to obtain input from the communities directly involved.

1.1.3 Environmental Impact Evaluation

An Environmental Impact Evaluation (EIE) is being prepared as a part of this study, to present the environmental impacts of the proposed project, including socio-economic impacts, for review and comment by government agencies and the public.

1.2 PHASE 1 OF THE REGIONALIZATION STUDY

The projected population for the five communities over a 20-year planning period, from the Connecticut State Data Center, is summarized in Table 1-1 below. For the purpose of this study, CSDC projections were adjusted based on input from local officials to allow for modest anticipated growth, as discussed in Technical Memorandum (TM) No. 1, in the appendix of this report.

Table 1-1 CT State Data Center Population Projections to 2040

Municipality	US Census 2010	Based on CT State Data Center Population Projections (published August 31, 2017)				
		2020	2030	2035	2040	Percent increase, 2040 vs. 2020
Derby	12,902	13,251	13,803	13,959	14,082	6.3%
Ansonia	19,249	19,841	20,648	20,890	21,067	6.2%
Seymour	16,540	16,798	16,924	16,852	16,753	-0.3%
Beacon Falls	6,049	6,421	6,587	6,591	6,587	2.6%
Naugatuck	31,862	32,212	32,638	32,372	31,854	-1.1%
TOTAL	86,602	88,523	90,600	90,664	90,343	2.1%

Currently, the average flows to the treatment plants in each of the five communities are less than half of the design permitted capacity, on an annual average basis, as indicated in Table 1-2 below.

Table 1-2 Annual Average (AA) Flow: Actual (2015-2017) vs. Permitted Capacity

Municipal WPCF	Average Annual (AA) Flow, 2015-2017 (MGD)	Permitted AA Design Capacity (MGD)	2015-2017 AA Flow as Percent of Permitted Capacity
Derby	1.3	3.5	37%
Ansonia	1.57	3.5	45%
Seymour	0.97	2.93	33%
Beacon Falls	0.31	0.71	44%
Naugatuck	4.61	10.3	45%

With the exception of Beacon Falls, all of the communities in this study have older collection systems that are plagued with high infiltration and inflow (I/I). This results in very high peak flows to the treatment plants. Two of the communities are under Orders to reduce I/I from their collection systems. The Derby treatment plant has been unable to treat peak wet weather flows in the past. Overall, collection system investment has generally been lacking.

1.2.1 Plant Condition Assessments

Under this study, an initial wastewater infrastructure condition assessment was conducted for each of the five communities, in 2018. This provided a high-level summary of the condition of existing wastewater treatment and collection system facilities based on site visits, interviews and review of existing reports. The Ansonia treatment plant is in overall satisfactory condition, following a major upgrade completed in 2011. However, the other four plants are in fair to poor condition; and will require major upgrades in the near future. In the case of Derby, this could approach full replacement of the plant. The details of this assessment are provided in TM No. 2, in the appendix of this report. Major findings are summarized as follows, by community.

1.2.1.1 Derby

The Derby Water Pollution Control Facility (WPCF) serves 95% of the population of Derby, plus a small portion of Seymour. The WPCF provides secondary treatment with nitrogen reduction, plus seasonal disinfection (chlorination/dechlorination). Since it discharges to a tidally impacted portion of the Housatonic River, there is no requirement for phosphorus removal. Sludge is dewatered, then trucked offsite for incineration and disposal. This facility was built in 1964, with a major upgrade to secondary treatment in 1973, followed by more limited upgrades in the 1980's and 1990's. Due to its age and poor condition, this facility is overdue for a major plant upgrade. The plant site is constrained, with little room available for expansion. The City is under Order from DEEP to upgrade the plant.

Systems at the Derby WPCF that will require major upgrades in the very near future include: influent pump station and preliminary screening system, grit removal facility, aeration basins and blower system, secondary clarifiers and flow splitter structure, control building, secondary control building, sludge processing, numerous pumping systems, plant-wide SCADA and electrical systems and numerous subsystem upgrades and repairs throughout the plant.

Derby's collection system is old, with approximately 70% of the sewers being old vitrified clay (VC) pipe, characteristically with serious defects. The collection system is leaky, with high peak wet weather flows. The sewerage collection system is under USEPA Order requiring major improvements, including I/I correction and an approved CMOM plan. Based on age and condition of the collection system, an annual program of pipeline and structures renewal is required; this should include a 'catch up' period during the first five years followed by a reduced yet sustained program of investment thereafter. It is noted that Derby has recently been taking positive action in the upgrade of its collection system and major pumping stations.

1.2.1.2 Ansonia

The Ansonia WPCF serves 98% of the population of Ansonia plus small portions of Derby, Seymour, and Woodbridge. The WPCF provides secondary treatment with BNR (nitrogen removal) and UV disinfection. Since the plant discharges to the Naugatuck River, which is fresh water, seasonal phosphorus removal is required. Primary sludge and thickened waste activated sludge are removed by tanker trucks, for offsite dewatering and incineration. The plant has demonstrated consistent good performance in meeting discharge permit effluent quality requirements. The plant was constructed in 1968 and upgraded to provide secondary treatment in 1970. The most recent extensive plant upgrade was completed in 2011. The plant equipment is in satisfactory condition overall. The Ansonia plant site has the advantage of having available room for adding major plant processes if needed. This site is much less constrained than that of the Derby facility.

The Ansonia WPCF upgrades required (under the base case, with no regionalization) are much less extensive than those needed at Derby, and include: adding a second UV channel, adding a second mechanical screen at the headworks, upgrading the effluent pump station to meet peak flows, and making improvements to its sludge thickening and pumping facilities.

In general, improvements to the Ansonia collection system have been deferred for many years. As a result, this will require a period of catch-up for replacing and repairing pipes, followed by a sustained annual capital improvements program for buried infrastructure. Work on the City's two major sewerage pump stations will also be needed.

1.2.1.3 Seymour

The Seymour WPCF serves most of the population of Seymour, along with a small section of Oxford. The plant was built in the 1970's, with a significant upgrade in the early 1990's. It provides secondary treatment with BNR for nitrogen reduction, with chlorination/dichlorination for disinfection. Since this plant discharges to the Naugatuck River, which is fresh water, seasonal phosphorus removal also is required, which is accomplished through chemical addition. Dewatered sludge is trucked offsite for incineration and disposal. The WPCF site is on a very narrow site bounded by State Route 8 and the Naugatuck River. Due to these geographic constraints, there is limited room available for adding major new facilities at this site.

Due to the age of this facility and the length of time since the last major upgrade, much of the mechanical and electrical equipment is at the end of its useful life and in need of replacement or upgrade. Upgrades to the Seymour WPCF that would be needed to maintain reliability and sustain continued operation, under the base case scenario with no regionalization, include: complete mechanical refurbishment of the headworks, influent pump station; replacement of the sludge pumps and sludge processing facilities; mechanical upgrades to the primary and

secondary clarifiers; refurbishing the primary control building; and upgrade of the plant-wide SCADA and electrical systems.

The Seymour wastewater collection system is old, with 23% of the sewers being VC pipe. The system is leaky, with high I/I and in need of significant replacement or upgrade of existing piping. Similar to Derby and Ansonia, it is recommended that a more accelerated program (for approximately five years) be undertaken to replace and rehabilitate sewer lines and structures. This should be followed by a sustained annual program of capital improvements to maintain system reliability and further reduce excess I/I flows.

1.2.1.4 Beacon Falls

The Beacon Falls WPCF was built in 1971, with the most recent upgrade done in 1994. The plant is due for a major upgrade, which the town has been planning to implement in phases starting in the near future.

The condition assessment identified the following systems as needing major upgrade or replacement: headworks, influent pump station, BNR system, clarifiers, rotary drum thickeners, and plant-wide electrical and SCADA.

The collection system is relatively new, with approximately two-thirds of it being installed within the past 20 years. Beacon Falls plans to focus on collection system needs after the plant upgrade is addressed.

1.2.1.5 Naugatuck

The Naugatuck WPCF serves the Borough of Naugatuck and adjacent portions of Middlebury, Oxford, Beacon Falls, and Prospect. The original plant was upgraded to secondary treatment in the 1970's. The WPCF is also the site of a regional solids processing facility that includes bulk sludge delivery, liquid sludge storage, dewatering via centrifuge or belt filter press, and incineration. High strength side stream flows from the regional solids processing facility contribute significantly to plant loading.

A Facilities Plan completed in 2017 identified capital improvements needed at the WPCF. These include upgrading following systems: scum collection on the primary clarifiers, secondary clarifiers, BNR, and phosphorus removal.

Naugatuck has an older collection system with much VC pipe. The community is under an Order with DEEP regarding the collection system operations and maintenance, and I/I control.

1.2.1.6 Phase 1 Base Case Costs

Table 1-3 below presents the base case budget capital costs that were identified for the study communities during Phase 1. Costs for the plants, the collection system pipelines and structures, and the larger collection system pump stations are presented. The costs represent high level estimates based on experience with other comparable-sized facilities, limited on-site reviews and parametric considerations (such as \$/gallon for treatment or \$/LF for collection system rehabilitation). The plant costs would be needed immediately for Derby and Seymour. For Ansonia, the plant improvements would be needed in approximately 10 years. The collection system costs would occur over the 20-year study period with an accelerated level of rehabilitation required in the initial five-year period for all three communities. It is noted that plant upgrade costs

were revised in Phase 2; this was as a result of further facilities development work and needs identification at the treatment plants.

Table 1-3 Projected 20-Year Wastewater Expenditures, Base Case (If No Regionalization)

	Derby	Ansonia	Seymour	Beacon Falls	Naugatuck	Total
Water Pollution Control Facility	\$70.0M	\$15.0M	\$40.0M	\$14.0M	\$55.0M	\$194.0M
Collection System	\$8.0M	\$10.3M	\$8.5M	\$3.1M	\$18.5M	\$48.4M
Large Pumping Stations	\$4.2M	\$3.0M	\$2.0M	\$0.5M	\$1.0M	\$10.7M
TOTAL	\$82.2M	\$28.3M	\$50.5M	\$17.6M	\$74.5M	\$253.1M

1.2.2 Long List of Regional Alternatives

At the conclusion of Phase 1 of this study, a 'long list' of 23 regionalization alternatives was identified as summarized in Table 1-4 below. For each alternative, a basic conveyance corridor was identified for major interconnection trunk sewers and force mains needed to connect the plants. Multiple pipeline routes were also identified within the conveyance corridors. The long list of regional alternatives was then carried into Phase 2 for development and evaluation. Phase 2 would start with a screening out process of the less feasible long-list regional alternatives such that a short-list would be identified. The short-list regional alternatives would then undergo more detailed development and evaluation such that a recommended alternative would be identified.

Table 1-4 Long List of 23 Regionalization Alternatives Identified in Phase 1 of the Study

No.	Alternative Description
1	Beacon Falls to Naugatuck
2	Beacon Falls to Seymour
2a	Beacon Falls to Seymour, I/I Reduction
3	Derby to Ansonia
3a	Derby to Ansonia, I/I Reduction
4	Derby to Ansonia, Effluent Pumped to Housatonic River
4a	Derby to Ansonia, I/I Reduction, Effluent Pumped to Housatonic River
5	Derby and Seymour to Ansonia
5a	Derby and Seymour to Ansonia, I/I Reduction
5b	Derby and Seymour to Ansonia, Effluent Pumped to Housatonic River
5c	Derby and Seymour to Ansonia, I/I Reduction, Effluent Pumped to Housatonic River
6	Derby to Seymour and Ansonia
6a	Derby to Seymour and Ansonia, I/I Reduction
8	Ansonia to Derby
8a	Ansonia to Derby, I/I Reduction
9	Seymour and Ansonia to Derby
9a	Seymour and Ansonia to Derby, I/I Reduction
10	Seymour to Ansonia, Part of Ansonia to Derby
10a	Seymour to Ansonia, Part of Ansonia to Derby, I/I Reduction
11	Beacon Falls and Seymour to Ansonia, Part of Ansonia to Derby
11a	Beacon Falls and Seymour to Ansonia, Part of Ansonia to Derby, I/I Reduction
12	Beacon Falls, Seymour, and Ansonia to Derby
12a	Beacon Falls, Seymour, and Ansonia to Derby, I/I Reduction

1.3 PHASE 2 OF THE REGIONALIZATION STUDY

The 23 regional alternatives that had been identified in Phase 1 were further developed through a two-step screening process to come up with the recommended alternative. The first step was to reduce the long list of alternatives to a short list of six alternatives that were considered most promising. The three primary drivers of this screening process were: (1) feasibility of aggressive I/I reduction, (2) availability of viable conveyance corridors, and (3) physical site limitations for WPCF process layouts. The final step was to further develop the six short listed alternatives, to come up with costs versus the base case (no regionalization) to determine the recommended alternative.

The process of reducing the long list of 23 alternatives to a short list of six alternatives is documented in TM No. 3, included in Appendix C of this report. Due to the difficulty in establishing a reliable pipeline from Beacon Falls to either Naugatuck or Seymour, this eliminated both Beacon Falls and Naugatuck from further consideration in cost-effective regionalization.

1.3.1 Inflow and Infiltration

Each one of the plants included in this study will need collection system improvements regardless of changes in flows and wastewater characteristics associated with regionalization. Community-wide inflow and infiltration (I/I) programs need to be undertaken in all the communities, and some of these are currently underway. The results of these programs need to be regularly

monitored. In part, this will allow the communities to reevaluate the need and degree to implement aggressive I/I mitigation measures, reducing impacts to the wastewater system. Reducing I/I flow will result in smaller infrastructure (e.g., pipes, pumping, treatment structures), will better align with treatment objectives and performance and result in lower capital and O&M costs.

In order to assess the impact of wet weather on collection system flows, a limited flow monitoring study was conducted from April 14 through May 13, 2020. This involved flow measurement at selected manholes in Derby, Ansonia, and Seymour. Rainfall data was also collected at Ansonia and as recorded at the Sikorsky Airport weather station. The measured flows were compared to incoming plants flows. Flows observed were generally typical for late spring, with average flows moderately high compared with other times of the year. Flow rates declined steadily throughout the monitoring period, due to lack of precipitation, also indicating the drawdown of the water table throughout the study area. More data is needed to more accurately evaluate flow characteristics in these systems, however these limited results confirmed significant I/I response during storm events. Details of I/I evaluations and flow monitoring are provided in TM No. 3 and TM No. 4, in Appendix C and Appendix D of this report.

1.3.2 Short List of Regional Alternatives

The six short-listed alternatives are listed in Table 1-5 below. Note that four of the alternatives are based on a regional WPCF in Ansonia, while the other two are based on locating the regional WPCF in Derby. The Ansonia-based regional approaches include alternatives to discharge either to the Naugatuck River at Ansonia (which would require phosphorus reduction); or pumping to the existing Derby outfall to the Housatonic River (which would eliminate the need for phosphorus reduction). Fresh water rivers such as the Naugatuck River are considered phosphorus-limited for eutrophication purposes, while salt and brackish marine environments are not.

Table 1-5 Short List of Wastewater Regionalization Alternatives

No.	Alternative Description
Ansonia Regional Alternatives	
3	Derby to Ansonia
4	Derby to Ansonia, Effluent Pumped to Housatonic River
5	Derby and Seymour to Ansonia
5b	Derby and Seymour to Ansonia, Effluent Pumped to Housatonic River
Derby Regional Alternatives	
8	Ansonia to Derby
9	Seymour and Ansonia to Derby

1.3.3 Short List Alternatives Development and Recommendation

Based on preliminary process layouts, the Ansonia plant site provides adequate space to accommodate the expanded facilities required to handle additional combined flows from both Derby and Seymour, utilizing conventional treatment processes. However, the Derby WPCF plant site is more constrained and would require intensification treatment technologies that can be accommodated within a smaller footprint. Therefore, for Regional Alternatives No. 8 and 9, which feature a regional WPCF in Derby, preliminary site layouts and costs were developed based on using ballasted activated sludge (based on BioMag) and integrated fixed film activated

sludge (IFAS) for secondary treatment. This is discussed in detail in TM Nos. 3 and 4, included in Appendix C and Appendix D of this report.

The six short-listed regional alternatives and the base case scenario were financially compared to each other using the present worth cost method of analysis. The present worth cost method allows for monetary costs associated with capital expenditures and O&M costs over the planning period (25 years) to be expressed as a present equivalent value, enabling alternatives to be compared. The basis for this present worth analysis is provided in TM No. 4 in Appendix D. The alternative with the lowest present worth cost is the most favorable as compared to the others. Table 6-1 in TM 4 shows the present worth cost results for all six short-listed regional alternatives and the base case scenario.

The two Derby-based regionalization alternatives, Regional Alternatives No. 8 and 9, were both found to be more expensive than the base case alternative of no regionalization. Therefore, these two alternatives were eliminated from further consideration. The four remaining regional alternatives, which feature a central WPCF in Ansonia, all were found to be more cost-effective than the base case of no regionalization.

TM 4 identified that regionalizing wastewater treatment would be more cost effective than staying with the status quo, the base case scenario, and that a regional WPCF at Ansonia is the most cost-effective alternative. A regional plan for wastewater consolidation will involve a different organizational structure than for the base case. While a detailed review of various regional structures was not conducted here, it is understood that a new Regional Wastewater Authority created, pursuant to Section 22a-500 of the Connecticut General Statutes will allow for more complete centralized accountability of wastewater service and performance in complying with federal, state and local laws and regulations; this is particularly the case when the Regional Authority is in control of all assets, including the WPCF, the collection system networks and pump stations.

Capital projects for a regional wastewater authority are eligible for state SFR funding as are projects from municipalities and other utility structures. However, a regional authority facility project would receive higher priority points by CT DEEP (because of greater populations served and commensurate water quality benefits), often making these projects more fundable than base case scenario projects. Additionally, a regional facility will receive a 25% grant from DEEP for an initial eligible project as compared to a 20% grant for an eligible base case project.

It is recommended that a new Regional Wastewater Authority would be formed as part of this plan (pursuant to Section 22a-500 of the Connecticut General Statutes) and that it would be able to utilize SRF funding to finance the recommended project identified herein, with SRF funding opportunity for 25% grant and the remainder in low interest loans (20-year term at 2.0% interest) for its eligible capital cost. Implementing a regional solution would also improve the likelihood of obtaining SRF funding, due to higher priority points available based on greater population served, as well as positive impacts to water quality.

Refer to Table 1-6 below for the present worth cost summary of the four alternatives centered at Ansonia and the base case scenario taking eligible state SRF funding into account, which involves a combination of grants and low-interest loans for eligible capital costs. The table also shows the net savings versus the base case. When likely grant funding is taken into consideration, the

most advantageous Regional Alternatives is No. 5b. Similar savings are projected for Regional Alternative No.4, the second most advantageous alternative evaluated.

Table 1-6 Present Worth Cost Comparison of Ansonia Regionalization Alternatives versus Base Case

No.	Regionalization Alternative	Regional Alternative Costs (\$M)					Base Case, No Regionalization	Base Case Costs (\$M)					Present Worth Savings in Regionalization (\$M)	
		Capital		O&M	Total			Capital		O&M	Total			
		With 0% Grant	With 25% Grant		With 0% Grant	With 25% Grant		With 0% Grant	With 20% Grant		With 0% Grant	With 20% Grant	With No Grants	With Grants ¹
3	Derby to Ansonia	\$78.2	\$58.7	\$57.5	\$135.7	\$116.2	WPCF's Remain in Derby, Ansonia	\$85.7	\$68.6	\$67.9	\$153.6	\$136.5	\$17.9	\$20.3
4	Derby to Ansonia; Effluent Pumped to Housatonic	\$71.1	\$53.3	\$57.1	\$128.2	\$110.4	WPCF's Remain in Derby, Ansonia	\$85.7	\$68.6	\$67.9	\$153.6	\$136.5	\$25.4	\$26.0
5	Derby & Seymour to Ansonia	\$125.8	\$94.4	\$74.2	\$200.0	\$168.6	WPCF's Remain in Derby, Ansonia, Seymour	\$118.1	\$94.5	\$95.6	\$213.7	\$190.1	\$13.7	\$21.5
5b	Derby & Seymour to Ansonia; Effluent Pumped to Housatonic	\$117.9	\$88.4	\$73.8	\$191.7	\$162.2	WPCF's Remain in Derby, Ansonia, Seymour	\$118.1	\$94.5	\$95.6	\$213.7	\$190.1	\$22.0	\$27.9

Clarifications

- (1) Costs for 0% grant scenario taken from Table 6-1 - Base Case and Regional Alternatives Comparison of TM 4 (Draft) Regional Wastewater Alternatives Short List Development (12/23/2020)
- (2) SRF grant funding for capital costs is set at 25% for regional alternatives, and 20% for non-regional alternatives.
- (3) No costs are included in the above table related to overall utility system administration.

The costs in Table 1-6 include the impact of anticipated SRF grant monies for each of the regionalization alternatives as well as the base case. Regional WPCAs created, under Section 22a-500 of the Connecticut General Statutes will have their initial infrastructure project such as those described herein eligible for 25% grant funding, while non-regional solutions (each community upgrading its own WPCF) would be eligible for 20% grant funding. As indicated on the table, the most cost-effective solution is Regional Alternative 5b, with a projected present worth savings of \$27.9M, compared to the base case of no regionalization.

Upgrades to the sewerage collection system pipelines, manholes and structures are required for all three communities. This is the same whether for the base case or for regionalization purposes. As such this collection system rehabilitation program is part of the recommended plan. Historically there has been a lack of concerted upkeep and rehabilitation of these buried systems. The collection systems of all three communities; Derby, Ansonia and Seymour experience excessive infiltration and inflows (I/I) which can overwhelm the systems during more intense storm events, including at the WPCFs. Derby is under Consent Order by US EPA to develop a CMOM plan and an I/I control plan. A recent collection system inspection by DEEP indicated that maintenance was lacking overall at Ansonia. Neither Ansonia nor Seymour have undertaken community wide I/I investigation and control programs for over 15 years. All three collection systems require prolonged investment to rehabilitate the pipe systems and reduce excessive I/I flow.

Under the recommended plan, the Derby and Seymour WPCF's would be decommissioned and converted to conveyance pumping stations. The Seymour pumping station would have a headworks consisting of mechanical screening and degritting, and the Derby pumping station would have a mechanical screenings facility. Conveyance pipeline routes from the Seymour plant to the Ansonia regional WPCF, and from the Derby plant to Ansonia, are shown on Figure 1-1 and Figure 1-2. The Seymour route also would require one intermediate pumping station, as shown. The proposed conveyance pipeline routes generally follow local roads and avoid major infrastructure rights of way as much as possible. Where major highway (State Route 8) and railroad crossing is required, this could be accomplished through pipe jacking and boring, to minimize traffic disruption during construction. With Recommended Regional Alternative 5b, the treated effluent pipeline from the Ansonia WPCF to the Derby outfall on the Housatonic River would largely follow the same route as the conveyance pipeline from Derby to Ansonia.

Conceptual design drawings of the conveyance pipelines were prepared during the development of Recommended Regional Alternative 5B and are included in Appendix A.



Figure 1-1 Seymour to Ansonia Conveyance Pipeline Route



Figure 1-2 Derby to Ansonia Conveyance Pipeline Route

The proposed site layout for the Ansonia WPCF under recommended Regional Alternative 5B is shown on Figure 1-3 below. Expansion and upgrades needed to accommodate the increased flow from Seymour and Derby will include the addition of one more each of the following treatment units: grit chamber, primary clarifier, secondary clarifier, and UV disinfection channel unit. The new regional plant will also require a new sludge thickening, storage and pumping facility. The recommended plan for sludge processing is to haul thickened liquid sludge offsite for further processing, incineration and ultimate disposal.

An updated conceptual design drawing of the Ansonia regional plant was prepared during the development of recommended Regional Alternative 5B and is included in Appendix A.

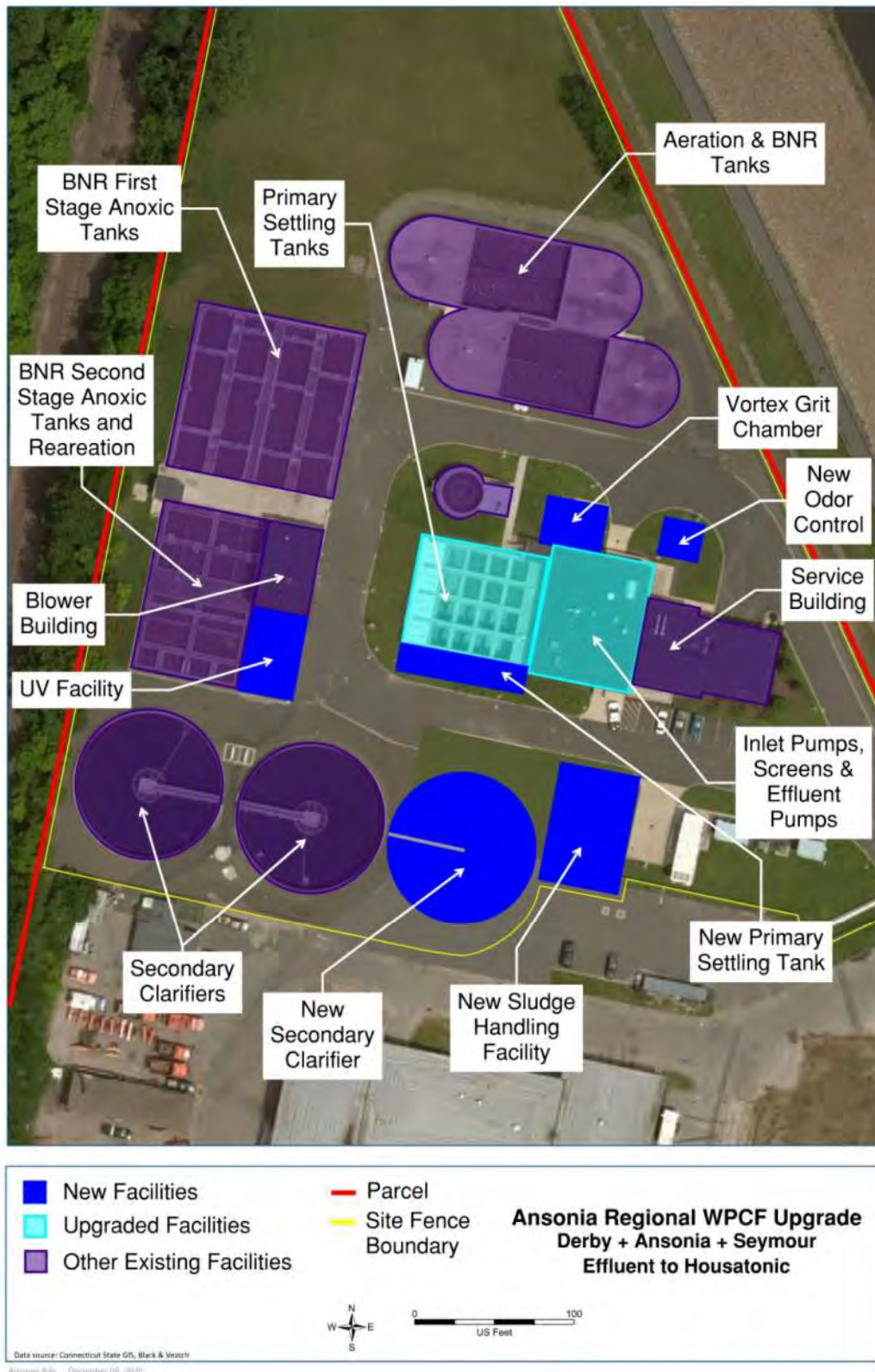


Figure 1-3 Ansonia Plus Derby and Seymour Site Layout

2.0 RECOMMENDED REGIONAL ALTERNATIVE

The recommended Regional Alternative (5b) was developed further with the focus on the feasibility of implementing the work during construction. This section summarizes the additional development work undertaken on the recommended regional alternative (5b).

All existing wastewater infrastructure including the sewerage collection system pipe networks, pumping stations, wastewater treatment plants, and treated effluent outfalls must be maintained in operating condition throughout construction. The feasibility of regionalization at Ansonia was further defined through a better understanding of its constructability, anticipated construction phasing and sequencing, and known constraints. These factors were then used to develop an overall conceptual program schedule.

2.1 ANSONIA REGIONAL PLANT UPGRADES

2.1.1 Phasing and Maintenance of Plant Operations

Expansion and upgrades at Ansonia will be conducted while the plant is operating. Local flow bypassing and temporary treatment measures will be required to implement the work, however flow to and from the plant must always be maintained because the sewer system cannot be isolated. The following sections discuss specific phasing and sequencing measures by area and upgrade. This will continue to be defined during the project's design phase.

2.1.1.1 Plant Influent

The Ansonia plant influent works is expected to remain unmodified. Introducing wastewater flow from Derby and Seymour is discussed later. Construction phasing and constraints discussed below regarding flow isolation and bypass associated with the raw wastewater screenings, influent pumping, and grit removal areas may be mitigated by installing influent bypass pumping systems to discharge directly into the influent channel of the primary settling tanks. However, temporary wastewater screenings removal may also be necessary. Bypass pumping systems may be the most effective strategy to isolate several areas of the plant on an as-needed basis throughout construction; this will be confirmed and finalized during the project design phase.

2.1.1.2 Screenings

Currently there is one mechanical bar screen and a parallel bypass channel with a manually cleaned bar rack. The plan is to replace the fixed bar rack with a new mechanical bar screen which will work side-by-side with the existing mechanical screen. Both the mechanical screen and manual bar rack can be isolated with slide gates. However, installation of a new gate will be required to route flow and this will be facilitated by isolating the bypass channel with the existing slide gates to allow for the work to be conducted while flow is maintained through the existing mechanical screen. The second screen will require a skylight to be cut into the roof over the new screen for installation and future removal.

There is not enough floor space nor room between the two screen channels to accommodate dedicated screening grinder/compactors for each screen. Therefore, a conveyor will be required to collect and convey the screenings to a replacement screenings grinder/compactor. During the interim period when the new bar screen is operational but before the conveyor and screening grinder/compactor is in place, screenings will be collected manually and dumped into a

roll-off cart for disposal. See Figure 2-1 Ansonia's Bar Rack Demolition (left) and New Screening System (right) for a conceptual view of the new screenings system.

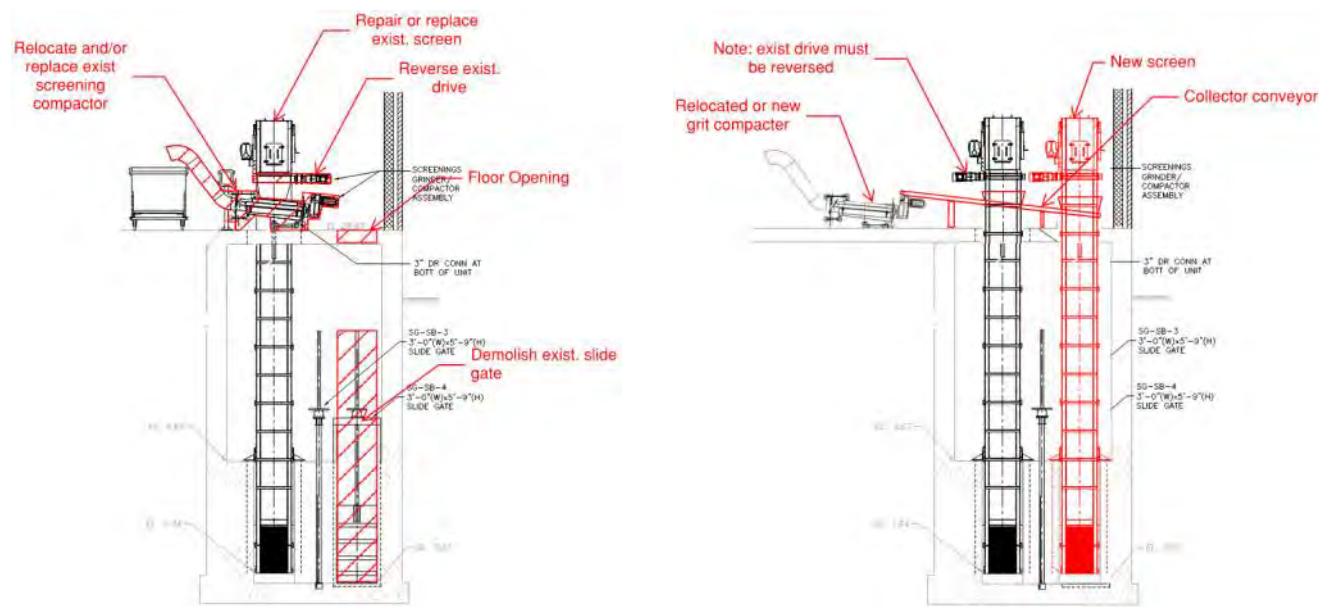


Figure 2-1 Ansonia's Bar Rack Demolition (left) and New Screening System (right)

2.1.1.3 Influent Pumps

The influent pumps will need to be replaced with larger capacity pumps to handle the incoming flows from Derby and Seymour. The existing pump suction inlets are 45-degree bells rather than 90-degree bells. This reduces the available submergence and may not be adequate once the plant flows are higher with Derby and Seymour connected in. Additionally, the existing inlet pipe sizes limit the flow capacity available from larger pumps. It is anticipated that the 45-degree suction bells will need to be replaced with larger 90-degree suction bells to increase the pump station capacity. Hydraulic analysis during design will confirm the adequacy of the existing pump station piping and all major process piping here and throughout the plant.

The inlet pipes will be isolated on the wet well side with bulkheads installed during low flow conditions and then dewatered to perform the concrete and mechanical work. Once the inlet piping is replaced, the new influent pumps and associated piping and valves can be installed in the pump room. The new piping will be tied into the existing discharge line if capacity allows. If the discharge pipe must be upsized, it will be installed parallel to the existing line, which would allow for the first pump to be commissioned on the new line while the existing line remains in service as the remaining pumps are replaced and connected to the new line.

Replacement of the raw sewage pumps with larger pumps will add extra heat load to the pump station area. Building HVAC will need to be upgraded to accommodate the added heat load from the larger pumps. The electrical system capacity will also need to be reviewed for adequacy and/or upgrade.

2.1.1.4 Grit Removal

The increased flow from Derby and Seymour will require additional grit removal capacity. This is best accomplished by duplicating the existing vortex grit chamber with a new treatment tank adjacent to the existing tank. While there is available area on the site both east and west of the existing grit chamber both present challenges. For the purposes of this study, the east side was selected for the new tank; this will match the flow direction through the existing vortex grit chamber and will also take advantage of the existing inlet and outlet channels. The new grit chamber will be constructed prior to tying into the existing inlet and outlet flow channels. Flow to the new tank will be routed through a pipe and inlet channel, and flow from the new tank will be routed around the existing tank through an outlet channel. See Figure 2-2 for a conceptual level depiction of the expanded grit chamber facility.

After the new grit chamber is constructed, the existing grit chamber will be isolated and its incoming flow bypassed directly to the primary settling tanks while the existing inlet pipe is connected to a new pipe to split flow to the new grit chamber inlet channel. Flow through the existing tank will be isolated in a similar fashion to connect the new outlet channel to the existing outlet channel that continues to the primary settling tanks.

An underground 24-inch diameter HDPE air inlet pipe to the downstream odor control system will need to be routed around the existing grit tank. This line will need to be taken out of service temporarily to construct the new grit chamber. Ultimately, this line may be replaced or removed altogether in conjunction with the area odor control system which services the Headworks building. The odor control system which serves the headworks building is not working effectively and needs to be upgraded or replaced.

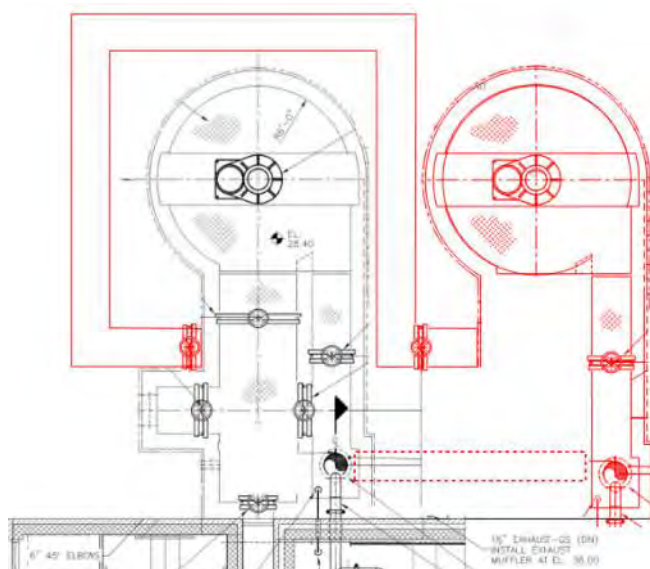


Figure 2-2 Ansonia's Grit Removal System Expansion

2.1.1.5 Primary Clarifiers

The new primary clarifier planned for the expanded Ansonia regional WPCF will be located at the southside of the other primary clarifiers and, while the new tank will match the other four primary tanks in size and dimension, it will be constructed in isolation of the existing tanks. This

will allow the other tanks to remain in service during construction. The existing primary scum pump station and valve vault are located in the area of the new tank and would be relocated west of the facility prior to constructing the new primary clarifier; this will call for isolating scum flow temporarily while the new pump station is put in service. The existing 20-inch plant effluent line is also located in this area; it will be relocated and likely upsized in conjunction with the effluent pump station upgrade discussed later.

Once construction of the new primary clarifier is complete, the existing channel end wall would be demolished to allow flow to the new tank. Sludge and scum draw off piping will be tied into the existing piping for these flows coming from the other primary clarifiers.

2.1.1.6 Secondary Clarifiers

The new third secondary clarifier will be constructed east of the two existing secondary clarifiers. The existing sludge handling facility is located in this area and will be decommissioned and demolished prior to beginning work on the new clarifier. The new sludge handling facility is discussed later. A portion of the neighboring recycling center pavement area will be needed to accommodate the new secondary clarifier. A new secondary clarifier flow distribution box will also be required to split flow evenly to the three tanks.

The new clarifier and flow distribution box will be constructed offline while the other two secondary clarifiers and distribution box remain in service. Flow to the new distribution box will be routed from the 36-inch second stage anoxic tanks effluent pipeline at the south end of the secondary process; this line will be isolated until the new clarifier is ready to be placed online. Flow from the new clarifier will be routed to the 36-inch UV facility inlet pipe. Scum flow from the new clarifier will be piped to the existing secondary scum pump station.

The new secondary clarifier will be placed online by opening flow to the new distribution box. Once the new tank is online, the inlet pipes for the existing two tanks will be tied to the new distribution box one by one. After all three tanks are in service with the new distribution box, the existing distribution box will be decommissioned and demolished.

2.1.1.7 UV Facility

The existing UV facility was originally constructed within the center channel of the previous three-channel chlorine contact tank; the other two channels were filled with structurally compacted fill and capped with concrete slabs. The current arrangement of the single channel UV system limits the flexibility of this system particularly during times when equipment malfunction has occurred and critical maintenance is required. A new second UV channel will be required as part of the expanded regional plant. To accommodate the new UV channel, the abandoned channel on the west side of this area will be opened up and rehabilitated. Flow to the new UV channel will be routed from the existing 36-inch inlet in conjunction with the secondary clarifier tie in. Flow out of the new UV channel will discharge into the existing plant effluent channel where it will continue on to the plant discharge.

2.1.1.8 Effluent Pump Station at Ansonia

Currently this pump station includes two plant effluent pumps; these will need to be replaced with larger units to accommodate the higher peak flows from the regional WPCF. Currently, this pump station is used intermittently and called into service during times of higher tides and/or

higher flow periods. However, as part of the recommended plan, this pump station will be converted to a conveyance pump station to discharge fully treated wastewater effluent through a new pipeline to the Derby outfall at the Housatonic River.

Design guidance for redundancy states that the system needs to be able to operate with the largest pump out of service. This would require that each of the two replacement pumps be capable of the full pumping capacity to meet peak flow conditions and also to be able to turn down to meet average and even low flows. In order to provide reliability to meet the design flows it will be necessary to add a third and perhaps even a fourth plant effluent pump. This will require construction of a second wet well that is hydraulically connected to the existing wet well. Since the existing effluent line from the UV Facility will need to be re-routed to construct the new primary clarifier, a new effluent pump station may be more feasible than expansion of the existing facility. Ultimate decision will be made during the design phase.

2.1.1.9 Sludge Thickening and Handling Facility

A new sludge thickening and handling facility will be constructed near the existing sludge facility. As noted previously, the existing sludge handling facility is located in the area planned for the new secondary clarifier. It is anticipated that the new sludge handling facility will be located generally east of the new secondary clarifier. The new sludge handling facility will be constructed and placed into service before the existing facility is decommissioned and demolished.

2.1.2 Effluent Discharge to Housatonic River

As part of the recommended plan, fully treated plant effluent will be conveyed south to discharge to the Housatonic River near the existing Derby wastewater plant. The existing Ansonia plant effluent pump station will be converted to a conveyance pump station and upgraded and expanded as described previously.

The plant effluent pipe at the Ansonia plant site will be upsized to accommodate increased flows from Derby and Seymour. This will be installed from the UV facility in conjunction with other yard piping changes associated with the new primary and secondary clarifiers; as this work is performed, plant effluent flow will be maintained through the existing pipe.

The new treated effluent conveyance pipeline to the Housatonic River at Derby will be constructed independently from the Ansonia plant expansion and upgrade work. As noted previously, this new pipeline would follow the same route as the conveyance pipeline from Derby to Ansonia and will terminate at the existing Derby plant outfall on the Housatonic River. It is currently envisioned that at times of high tide cycles, the hydraulic grade line would need to be raised for discharge to the river. This would be accomplished by the existing City of Derby stormwater pump station which also serves as a plant effluent pump station under high river level conditions. While a detailed review of this facility has not been conducted, it is likely that the capacity of this pump station would need to be increased to accommodate peak flows from the combined treated effluent discharged from the Ansonia regional WPCF. Specific expansion and development determinations for this pump station will be made during the project design phase.

2.1.3 Introducing Wastewater Flow from Derby and Seymour

Depending on final pipeline routing and hydraulic requirements at the site of the Ansonia regional WPCF, wastewater flow from Derby and Seymour will be introduced into the expanded plant in either a common line or in two separate lines. The final determination will be made during the design phase. Flow from Derby will not be de-gritted and must be introduced into the Ansonia plant upstream of the grit removal tanks; therefore, for a single line or combined line with Derby, the flow will empty at or near the influent pump station. Flow from Seymour will be de-gritted and could discharge directly at or near the primary settling tanks, depending on hydraulic conditions. Flow from both communities will be screened prior to conveyance and therefore do not have to empty ahead of the Ansonia screening facility.

Flow from Derby and Seymour will be introduced after all upgrade and expansion work is complete at the Ansonia WPCF. After the conveyance pipeline and pump station systems are in place, tested, and initially started up using clean water to prove functionality, wastewater flow will be conveyed to Ansonia at a low, manageable rate. Flow will then be steadily increased over multiple days to demonstrate functionality and treatment at the newly expanded and upgraded plant. This time will be used to adjust and calibrate equipment and processes across the regionalized network.

After full flow (e.g. Derby and Seymour) is being conveyed to Ansonia, the entire infrastructure system will be run under normal operating conditions for an extended demonstration test period during which time high flow events would be anticipated to prove-out the upper limits of the system. The wastewater facilities at Derby and Seymour will remain fully operational during this time, with the ability to stop conveying flow to Ansonia in case of any emergency or equipment failure during commissioning of the new regional infrastructure. After an extended period of successful operation of the entire regional system, the existing plants at Derby and Seymour can be decommissioned.

2.2 DERBY CONVEYANCE PUMP STATION UPGRADES

The existing influent pump station and screenings removal system will be converted to a conveyance pump station and screening facility for discharge of Derby wastewater to Ansonia under the recommended Regional Alternative.

2.2.1 General Phasing and Plant Operations During Construction

The Derby WPCF will continue to operate until all construction work at the Ansonia regional WPCF, the two wastewater conveyance pipelines to Ansonia, and the treated effluent line back to Derby are completed. The Derby plant's influent pump station will be converted to a conveyance pump station. This will require bypass pumping around the pump station to allow for these improvements to be completed, tested, and placed into service while the collection system and remainder of the plant remain in operation. The raw wastewater screening facility is contiguous with the influent pump station and work would occur at both once the pumping complex has been temporarily bypassed for construction purposes.

Raw wastewater enters the plant through two existing influent sewers (24-inch and 18-inch diameter) which converge in a junction chamber that is part of and just outside of the main pump station building. One slide gate in the junction chamber can be used to isolate the influent sewers from the pump station. The bypass pumping system required must be flexible enough to

accommodate low flows as well as expected peak flows for the duration of the temporary bypass period. Initial indications are that the chamber is too small to fit submersible bypass pumps within it or to accommodate multiple suction lines required to operate an above ground bypass pumping station.

It was determined that plant flow could be bypassed with a main temporary pump system drawing suction from the junction chamber while a second, smaller temporary pump system would take suction from a manhole several hundred feet upstream on the 24-inch diameter incoming sewer. This pump station would operate intermittently and will be called into duty when flows to the system are higher. Both bypass pump stations would discharge flow directly to a temporary screening facility just upstream of the grit chamber and which would discharge to the grit chamber. This will allow all construction modifications to be made within the pump station and screening facility without being hindered by phasing and will also allow the plant to continue normal operations.

2.2.1.1 Raw Wastewater Influent Pumps and Piping

The existing influent pumps do not have adequate discharge head to meet the increased pumping requirements to deliver flow to the Ansonia regional plant. It is envisioned that the existing pumps will be replaced with dry pit submersible sewage pumps. In addition to making this facility more flood proof, this style pump will eliminate concerns related to maintaining long shafts and their bearings, and thus will increase overall pumping reliability. This style of pump will also free up floor space at the pump building's upper level which will be needed to accommodate the larger size electrical equipment. The pumps will be VFD driven so that they can be paced with varying inflows.

While the pump station is out of service, the existing piping and valves will be replaced, and a stub out will be left in the yard for connection to the conveyance pipeline to Ansonia.

2.2.1.2 Raw Wastewater Screening Facility

The existing bar rack influent channels will be demolished to the exterior foundation walls and slab while the pump station is out of service. New channels reconfigured for mechanical screens will be constructed. Once the channels have been constructed, continuous belt or climber type screens will be installed to lift the screenings to the upper level. The captured screenings would be conveyed to a screenings/washer/compactor system located outside the existing building which in turn would discharge to a roll-off container. Excess liquid from the washer compactor would be diverted back to the inflow. The washer compactor will be situated in a small building enclosure contiguous to the pump building. It is likely that the existing plant emergency generator will need to be relocated to

accommodate the washer compactor building addition. Figure 2-3 shows the upper level plan of the new mechanical screening facility currently envisioned at Derby.

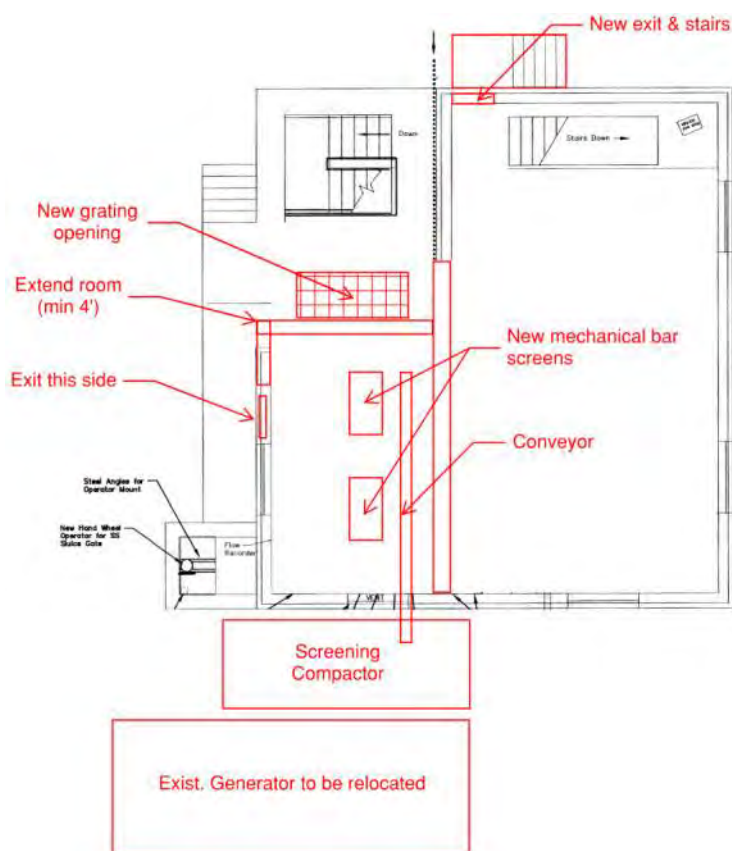


Figure 2-3 Derby's New Screening Facility - Upper Level

2.2.1.3 Electrical and Ventilation Systems

When a dry well raw wastewater pump station is unventilated or has a ventilation rate of less than 6 air changes per hour (ac/hr), the area is classified as Class 1 Division 2 per NFPA 820, requiring motors and other electrical components to be explosion proof rated to that classification. If the ventilation rate meets or exceeds 6 ac/hr the area becomes unclassified and explosion proof equipment is not required. NFPA 820 also states that the area over the wet well and screenings area would require a ventilation rate of 12 ac/hr to attain an area classification of Class 1 Division 2 and, ventilation rates of less than 12 ac/hr would leave the area as Class 1 Division 1 which requires stricter safety measures to be incorporated into the electrical systems design.

The new mechanical screens will require that holes be opened in the floor slab over the screening area and wet well. This will create a source of ambient air communication between the two areas such that the entire area would require the higher ventilation rate. To address this, the electrical room including the motor control center will be isolated and a wall constructed providing physical isolation between the pumping and screenings areas. This will mean that ventilation for the screenings portion of the building will require 12 ac/hr for the screenings portion of the pump station and 6 ac/hr for the pump station side of the building. Note that the entrance to the pump station side of the building will need to be relocated as the screenings area will expand

across the existing doorway. Doorways between the two areas would not be allowed according to code without an intermediate chamber. New electrical and HVAC equipment will be provided to meet the air change requirements and increased motor loads from the larger pumps. See headworks demolition plan in Figure 2-4 and Figure 2-5.

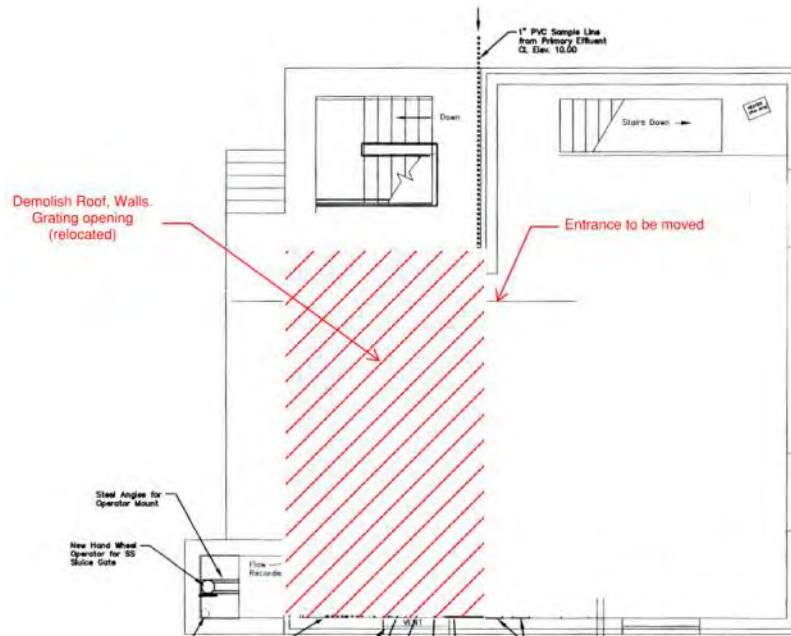


Figure 2-4 Derby's Headworks Roof and Structural Demolition

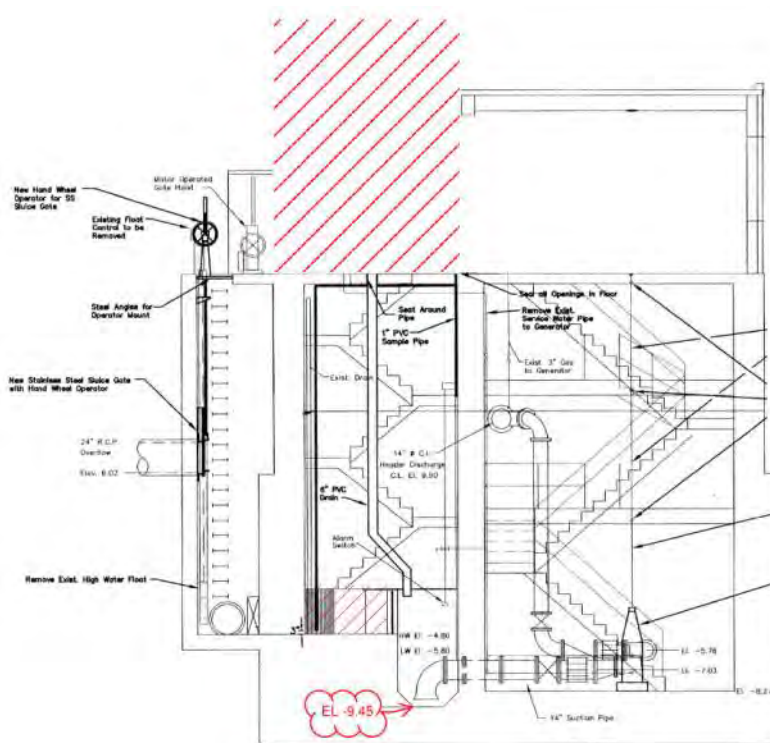


Figure 2-5 Derby's Headworks Building Demolition (section)

2.2.2 Derby Conveyance Pump Station Commissioning

Upgrades to the Derby pump station and screening facility will be conducted while incoming wastewater is being routed to downstream treatment systems with a bypass pumping system. This bypass pump system will be maintained throughout construction, startup, and commissioning with the ability to stop pumping flow to the Ansonia regional WPCF at any time during commissioning of the new regional infrastructure. The upgraded conveyance pump station and screening system will be tested using clean water to demonstrate full functionality and performance capabilities prior to conveying raw wastewater to Ansonia.

After the Derby and Seymour conveyance pipeline and pump station systems are in place, tested, and started up, and after all upgrades at Ansonia are completed and tested, wastewater flow will be conveyed to Ansonia at a low, manageable rate and then steadily increased over multiple days. This transfer from bypass system to conveyance will be possible through isolation gates that can be closed at any time with the City's wastewater flow re-directed from going to the Ansonia facility back to the Derby plant for treatment. A minimal amount of wastewater will continue to be processed at Derby during commissioning to maintain process operations throughout the plant in case operation at full capacity must resume.

After all Derby wastewater flow is conveyed to Ansonia, the conveyance pump station and pipeline will be run under normal operating conditions for an extended demonstration test period during which time varying flow events would be anticipated to prove-out the system under a range of conditions. After an extended period of successful operation of the entire regional system, the existing Derby wastewater plant can be decommissioned.

2.3 SEYMOUR CONVEYANCE PUMP STATION UPGRADES

The existing influent pump station, screenings removal system, and grit removal system will be converted to a conveyance pump station and headworks facility to convey Seymour wastewater to Ansonia under the recommended Regional Alternative.

2.3.1 General Phasing and Plant Operations During Construction

2.3.1.1 Bypass Pumping

The screening and grit removal systems at Seymour are located in a recessed area approximately 12 feet below grade level, upstream of the influent pump station. The area is configured with multiple channels routing flow through the screening and grit removal areas, with several bypass and overflow routes for maintenance and high flow conditions. There is ample space available in this area of the plant to establish a bypass pumping system that can convey incoming wastewater flow directly to the primary treatment tanks while upgrades are made to the screening and grit removal treatment systems, and while the influent pump station is converted to a conveyance pump station. This bypass pump system would also be used during commissioning of the Seymour to Ansonia conveyance pipeline system.

2.3.1.2 Screening and Grit Removal

Raw wastewater is currently screened through a catenary coarse bar screen that has been enclosed using a light metal frame system covered with corrugated fiberglass panels. Screenings are discharged into rolling bins with liner bags. The bags are then lifted to grade using an overhead monorail and dumped into containers for hauling to a landfill. The single screen will be replaced with two mechanical screens with finer openings to reduce operation and maintenance of equipment downstream including the raw sewage pumps. An evaluation to determine the final screen spacing will be based on hydraulic considerations including approach velocity. It appears that the existing bypass channel would allow a second screen to be installed before removing and replacing the existing catenary bar screen. Thus, as two mechanically cleaned screens are to be installed, the first should be installed in the bypass channel before removing and replacing the unit currently in service.

The catenary bar screen is old technology that works relatively well; however, over time the heavy chain links collect rags creating nuisance maintenance. Catenary screens are manufactured for fine screens however as previously noted the chains are heavy and require higher power consumption than other screening technologies. They are also somewhat limited in terms of their lifting height. Continuous belt type screens as well as step screens can be used to lift screenings to the required level. Climber type screens could also be considered.

The existing grit removal system consists of a cross collector in a channel that draws settled grit to a chain and bucket elevator that lifts the captured grit into a grit separator at grade level. Removal and replacement of the grit collection system will require dewatering the grit channel. This will require bypass pumping to the primary treatment tanks. While the channel is dewatered, the channel can be cleaned and inspected. Any structural defects can be repaired during this time.

During final design it is recommended that construction of an equipment access platform be considered for construction at grade essentially at the top of wall elevation for the main structure with the mechanical screen system discharging above grade. This would allow for the

new screens to be housed in a protected enclosure at grade. While this approach adds additional lifting height to the bar screen it would reduce the need to access the lower level and simplify the removal of solids. Mounting the equipment at this height would allow the direct discharge of the screenings to a washer/compactor which would discharge to a roll-off container. This system would reduce odors and minimize solids handling. This arrangement would also allow the equipment to be accessed for maintenance without going down to the lower level. It should be noted that the lower bearings of a continuous screen would be immersed in the channel. Regardless of whether the screening equipment is brought to grade or not, ventilation will need to be upgraded to meet current code. According to NFPA 820, ventilation in this area would need to be 12 air changes per hour to attain Class 1 Division 2. Otherwise the raw wastewater headworks environment makes this area classified as Class 1 Division 1.

2.3.1.3 Influent Pump Station

The existing pumps are vertically mounted end suction sewage type that are shaft driven with the motors being mounted on the main pump level. New close coupled pumps with submersible motors are recommended in lieu of the shafted pumps. The benefits of this style pump is the elimination of maintenance associated with the long pump shafts, increased reliability and a more flood proof facility. Ventilation of the pump station will be confirmed in accordance with NFPA 820 as a dry well pump station requires a ventilation rate of six air changes per hour to be considered unclassified. This ventilation rate would be the same for both the lower and upper areas of the pump station.

2.3.2 Seymour Conveyance Pump Station Commissioning

Upgrades to the Seymour pump station will be conducted while incoming wastewater is being routed to the primary clarifiers with the bypass pumping system. This bypass pump system will be maintained throughout construction, startup, and commissioning with the ability to stop conveying flow to Ansonia at any time during commissioning of the new regional infrastructure. The upgraded conveyance pump station, screening system, and grit removal system will be tested using clean water to demonstrate full functionality and performance capabilities prior to pumping raw wastewater from Seymour to Ansonia. The intermediate booster pump station is described later.

After the Derby and Seymour conveyance pipeline and pump station systems are in place, tested, and initially started up, and after all upgrades at the Ansonia regional WPCF are completed and tested, wastewater flow will be conveyed to Ansonia at a low, manageable rate and then steadily increased over multiple days. This transfer of wastewater from the bypass pump system to the conveyance pipeline will be accomplished with isolation gates that can be closed at any time and the wastewater flow redirected back to the Seymour plant for treatment if necessary. A minimal amount of wastewater will continue to be processed at Seymour during commissioning to maintain process operations throughout the plant in case operation at full capacity must resume.

After full flow is being conveyed to the Ansonia regional WPCF, the system will be run under normal operating conditions for an extended demonstration test period during which time high flow events would be anticipated to prove-out the upper limits of the system. After an extended period of successful operation of the entire regional system, the existing Seymour plant can be decommissioned.

2.4 CONVEYANCE PIPELINE ROUTING AND BOOSTER PUMPING STATION

2.4.1 Pipeline Alignments

The pipeline alignments identified in the Task 3 TM were developed further; the focus of this effort was to avoid conflict with existing infrastructure and to minimize challenges that are anticipated during construction. The alignment of the influent pipeline into the Ansonia regional plant's headworks was also better defined. The incoming line was routed around the aeration oxidation ditch structures to minimize existing plant piping relocation and associated site restoration. Up-to-date survey and mapping of the pipeline routes will be undertaken during design. These will capture all underground and overhead utilities and will serve as the basis to establish the final routes for the two conveyance pipelines. The influent pipe alignment at the Ansonia WPCF can be seen in sheet 6 of 14 of the pipeline conceptual design drawings in Appendix A.

2.4.2 Booster Pumping Station

An intermediate booster pump station will be required on the Seymour conveyance pipeline approximately two miles away from Seymour WPCF to convey flow to the Ansonia regional WPCF. The pump station is envisioned to be a one level facility with a wet well, electrical room, and mechanical room. The wet well will be designed to handle surge flows that can occur during extreme wet weather events. The site will include a driveway, parking spaces, and a standby electrical power generator.

A concept level location and site layout of this pump station can be seen in sheet 8 of 14 of the pipeline conceptual design drawings in Appendix A. At this stage of the study, an unoccupied space at 30 Maple Street in Ansonia has been identified as a potential location for this pump station. The pump station can be located elsewhere in this approximate area. The final location will need to be determined based on land availability and acquisition, as well as hydraulics, environmental restrictions, and other considerations.

2.4.2.1 Booster Pump Station Commissioning

The booster pump station will be commissioned with the Seymour conveyance pipeline. Initial startup and testing will be conducted using clean water. Once the systems have successfully completed initial startup, wastewater will be introduced into the pipeline and conveyed at a minimal rate and gradually increased over several days of testing to prove-out full functionality.

2.4.3 Utility Coordination

A limited utility coordination review was conducted along the pipeline routes using available mapping, aerial imagery, and design references to identify potential major utilities in the area that may conflict with the new conveyance pipelines. No high voltage electrical lines or natural gas transmission lines are within the pipeline alignments which would pose the highest risk and cost during construction. Through an in-person and virtual drive-through of the routes, along with review of utility maps where available, it was confirmed that most streets within the alignments have several underground utilities, overhead power lines, and lighting poles. Caroline Street and Water Street in Derby and Pershing Drive and Jackson Street in Ansonia were identified as streets with many utilities where extensive coordination will be required during construction to maintain and reroute services.

Although no major utility conflicts are apparent, a detailed utility coordination effort will need to be undertaken during design to identify and accommodate all utilities along the conveyance route. Alternate route variations are available along adjacent streets that can be adopted as needed during design.

2.4.4 Derby and Seymour Flow Convergence

To facilitate construction near the Ansonia regional plant, wastewater flow from Derby and Seymour in separate lines will combine into one gravity sewer that will cross under the railroad and into the Ansonia plant site; this connection can be seen in sheet 6 of 14 of the pipeline conceptual design drawings in Appendix A. At this junction point, the pipeline coming from Seymour will be a gravity sewer and the pipeline from Derby will be a force main being pumped. Before these flows meet, the pressurized flow from Derby must exit to atmosphere to match hydraulic conditions with Seymour flow and to prevent surcharging. The two pipelines will discharge into a junction box designed to take the wastewater flows from each community so that these flows combine and stabilize prior to entering the Ansonia plant.

It is also noted that there is an existing sewer line in proximity to the conveyance pipeline that comes into the plant from Pershing Drive. The flows conveyed from Derby and Seymour might be able to be combined in that existing sewer line ahead of the railroad crossing; this could potentially avoid a new pipe crossing under the railroad and would also minimize routing conflicts with existing site piping and duct banks. However, the available capacity of the existing sewer line and associated hydraulic constraints will need to be evaluated and confirmed with the new conveyance pipeline flow conditions to determine whether the existing pipeline may be tied into to improve constructability and phasing constraints.

2.4.5 Trenchless Crossings

Five segments along the conveyance corridors were identified as requiring trenchless crossings to maintain use of other infrastructure. These segments are either within the right-of-way of the railroad or State highway Route 8 and will require close coordination and permitting with their governing agencies. After analyzing pertinent trenchless construction methods available to perform these crossings, the “jack and bore” method was determined as suitable for each of these locations. This method generally consists of excavating a jacking pit and a receiving pit, and then jacking a casing pipe forward from the jacking pit to the receiving pit while simultaneously excavating soil with an auger boring machine. After the casing pipe is installed between the two pits, the wastewater conveyance pipeline is installed within the casing pipe and the annular space is filled with grout.

The segments of the pipelines anticipated to be constructed using the jack and bore method are generally described in Table 2-1 and shown on the pipeline conceptual design drawings in Appendix A.

Table 2-1 Jack and Bore Segments

Segment	Launch Pit	Receiving Pit	Approximate Length	Right of Way	Community
From Derby WPCF to Caroline St	Property owned by the City of Derby	Derby WPCF	120 ft	Railroad	Derby
Along merge ramp under Route 8	Merge ramp shoulder/lane	Merge ramp shoulder/lane.	150 ft	Route 8	Derby
From Pershing Dr. to Ansonia WPCF	Private property	Ansonia WPCF	190 ft	Railroad	Ansonia
From Derby Ave to Route 8 merge ramp	Private property	Private property/merge ramp shoulder	350 ft	Route 8	Ansonia & Seymour
From Seymour WPCF to Derby Ave	Property owned by the State of Connecticut	Seymour WPCF	420 ft	Railroad & Route 8	Seymour

Construction within the highway and railroad right-of-way poses a safety risk to the traffic-carrying ability and physical integrity of associated structures. The railroad owner and CT DOT will have specific requirements for this work that will need to be incorporated into design, including maximum crossing angles, minimum vertical and horizontal clearances between the new utility and existing road structures, and casing strength. Standard values were assumed for the purposes of this study.

2.4.6 Property Access

Pipelines are routed through private property and several easements and property acquisitions will be required for initial construction and future pipeline maintenance. For conceptual purposes at this level of study, the following properties have been identified for acquisition or easement securement.

2.4.6.1 Property Acquisitions

The following property has been identified for the booster pump station.

1. 30 Maple Street, Ansonia: The booster station will be located in the northeast area of the parcel which is currently unoccupied. Approximately 0.30 acres of the parcel currently owned by 30-38 Maple Street Associates LLC will need to be acquired.

2.4.6.2 Easements

The following properties are within the pipeline route or access will be needed for jacking pits.

1. Main Street parcel, Derby: The launch pit for the railroad crossing into Derby's WPCF and a portion of the force main leaving the WPCF will be routed along this parcel. This is an unoccupied parcel currently owned by the City of Derby.

2. 116 Pershing Drive, Ansonia: The launch pit for the railroad crossing into Ansonia's WPCF and a portion of the gravity sewer going into the WPCF will be routed along this parcel. This is a vegetated parcel currently owned by Lemko Indep Citizens Assoc Inc.
3. 814 Derby Ave, Seymour: The launch pit for Route 8 crossing and a portion of the force main going to Ansonia will be routed along this parcel. This is an empty parcel that is part of the parking lot for Tri-Town Plaza owned by CT Pro Tri Town Plaza LLC et al.
4. 560 Wakelee Avenue and Route 8 shoulder, Ansonia/Seymour: These parcels are parallel to the Naugatuck River and within proximity of Route 8; these are vegetated parcels and certain parts are considered protected open space. The receiving pit for Route 8 crossing and a portion of the pipeline will be in these parcels. 560 Wakelee Ave is owned by the City of Ansonia and easements with the Town of Seymour will be required to build in proximity to Route 8.
5. 731 Derby Avenue, Seymour: The launch pit for the Route 8 crossing into Seymour's WPCF and a portion of the force main will be routed along this parcel. This parcel is owned by the State of Connecticut and has a sand storage structure.

2.4.7 Permit Needs

Several permits will be required for the conveyance pipeline work. At this stage of planning, major permits required have been identified. A detailed permit need assessment will need to be conducted during design based on the final pipeline route, utility coordination, and environmental and municipal requirements. Table 2-2 lists the anticipated permits required for the work.

Table 2-2 Anticipated Required Permits

Permit	Description
Building Permit	Allows excavation and construction of any structure.
Zoning Permit	Allows the use of a property, building or parcel for an identified purpose.
Site Plan Approval	Used to determine if site plans comply with appropriate regulations, preserve appearance of neighborhood, and any potential nuisances to abutters; a public hearing will most likely be required to obtain this approval.
Soil and Erosion Sediment Control Plan	A certified plan will be needed depending on the size of excavation and effect the excavation will have on wetlands, water course, drainage system, building or adjoining streets and properties.
Wetland Permits	Needed if construction is in close proximity or would disturb wetland areas.
Flood Hazard Area Permit	Needed if construction is within a city or town flood hazard area.
Encroachment Permit	Allows construction of any structure within the right-of way of a state highway.
Construction within Railroad ROW Permit	Allows for construction within 200 feet of Metropolitan Transit Authority railroad.
Street Excavation/Obstruction Permit	Allows excavation and/or obstruction in city streets; traffic control planning will be necessary to meet permit requirements.
Street Tunneling/Jacking Permit	Allows tunneling or jacking in city/town streets.
Occupancy of Street License	License required to build and obstruct city/town streets and public spaces as long as safe and convenient passage around any structure is provided.
Contaminated Soil and/or Sediment Management	Authorizes staging, transfer and temporary storage of contaminated soil and/or sediments generated during construction.
Construction Stormwater General Permit	Requires implementation of a Stormwater Pollution Control Plan to prevent movements of sediments off construction sites to nearby bodies of water.
Utility Coordination Process	Focused on resolving any utility conflicts including coordination with existing utilities, relocation plans, and test pit programs.

2.5 SCHEDULE

A conceptual program implementation schedule was prepared based on the development of the recommended Regional Alternative and is shown in Figure 2-6 below. The overall implementation timeline is estimated to be approximately six years. The schedule starting point assumes that all regional governance and stakeholder engagement has been established, and that the program is approved by all governing agencies for design and construction.



Figure 2-6 Preliminary Implementation Schedule - Recommended Regionalization Alternative

3.0 CLOSING

Further detailed planning and engineering work of the facilities comprising the recommended plan will require site specific data development, including geotechnical and environmental investigation programs, and field surveys and mapping. A detailed flow measuring program is also required such that all facilities are properly sized for both current and design horizon conditions.

It is noted that the work associated with the conveyance pipelines will be coordinated closely with other street projects that are planned within the three communities, Seymour, Ansonia, and Derby. This will allow for an integrated program to be developed on streets where possible competing construction projects would occur. Close coordination will serve to minimize multiple sequential and unnecessary work on streets which would otherwise result in disruptions impacting neighborhoods, including residents and commercial interests. The work to upgrade Route 34 in downtown Derby is one such example. The buried sewer pipeline work will be coordinated such that a complete and integrated plan can be implemented that serves the needs of both the Route 34 reconstruction and downtown redevelopment project and this wastewater regionalization program.

The recommended wastewater regionalization plan is shown in schematic form on Figure 3-1.

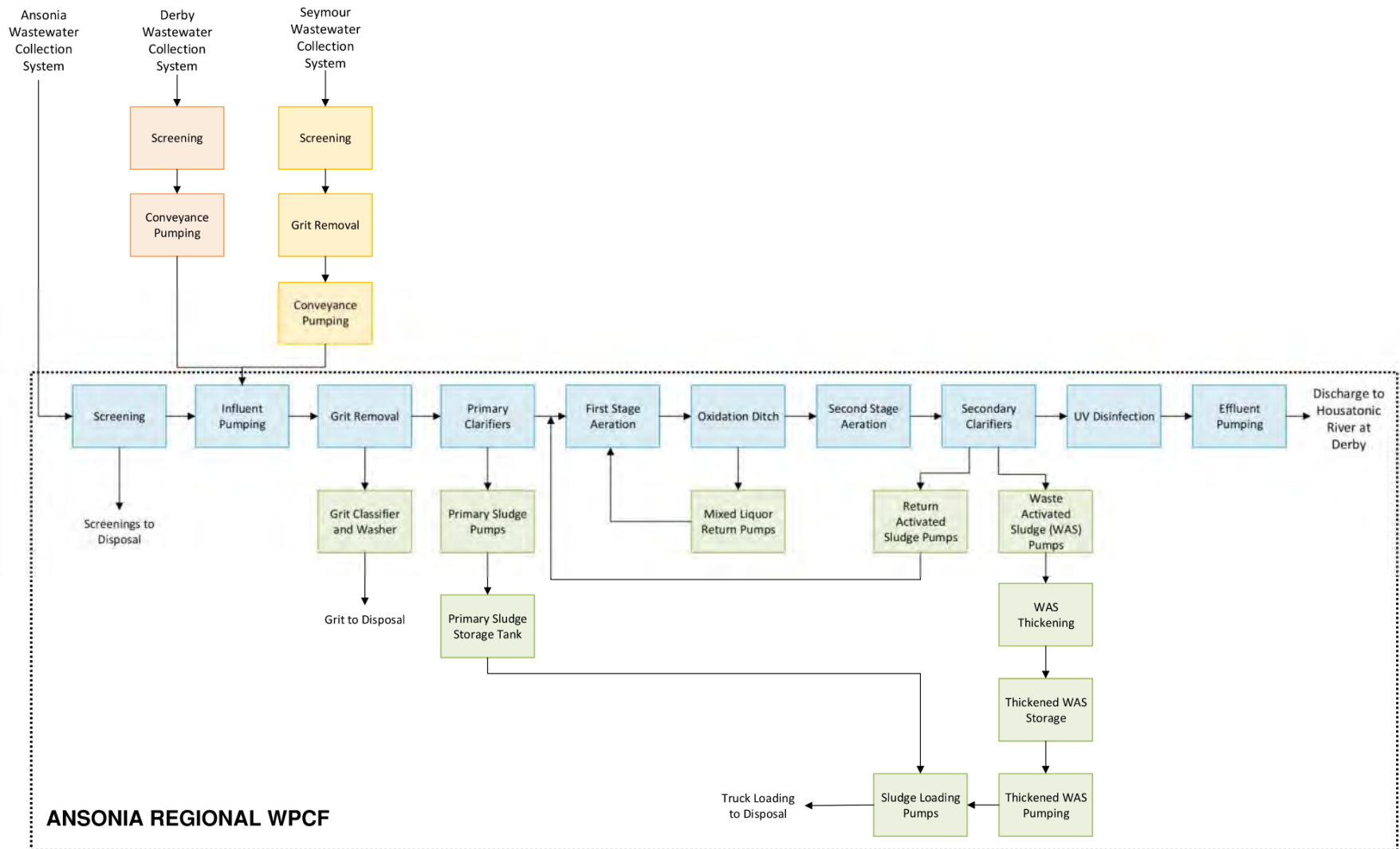
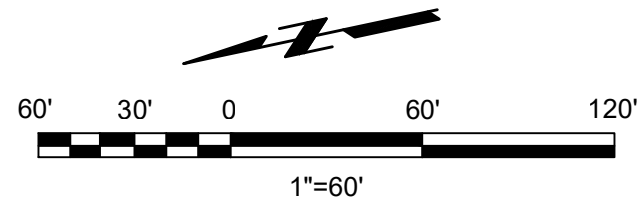


Figure 3-1 Recommended Wastewater Regionalization Plan Schematic

APPENDIX A

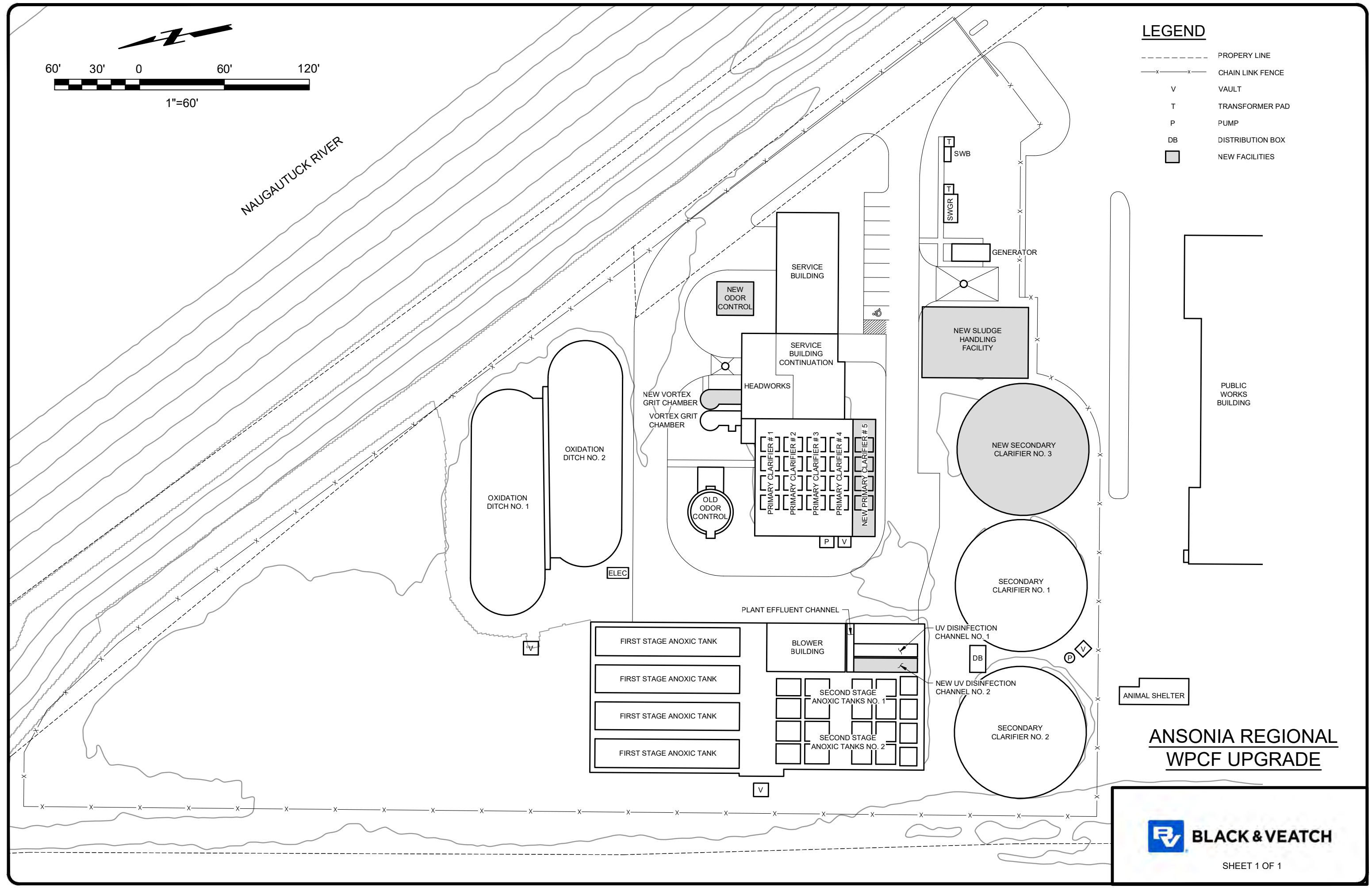
RECOMMENDED REGIONAL ALTERNATIVE CONCEPTUAL DESIGN DRAWINGS



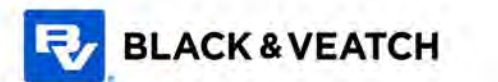
NAUGAUTUCK RIVER

LEGEND

- PROPERTY LINE
- x-x- CHAIN LINK FENCE
- V VAULT
- T TRANSFORMER PAD
- P PUMP
- DB DISTRIBUTION BOX
- NEW FACILITIES

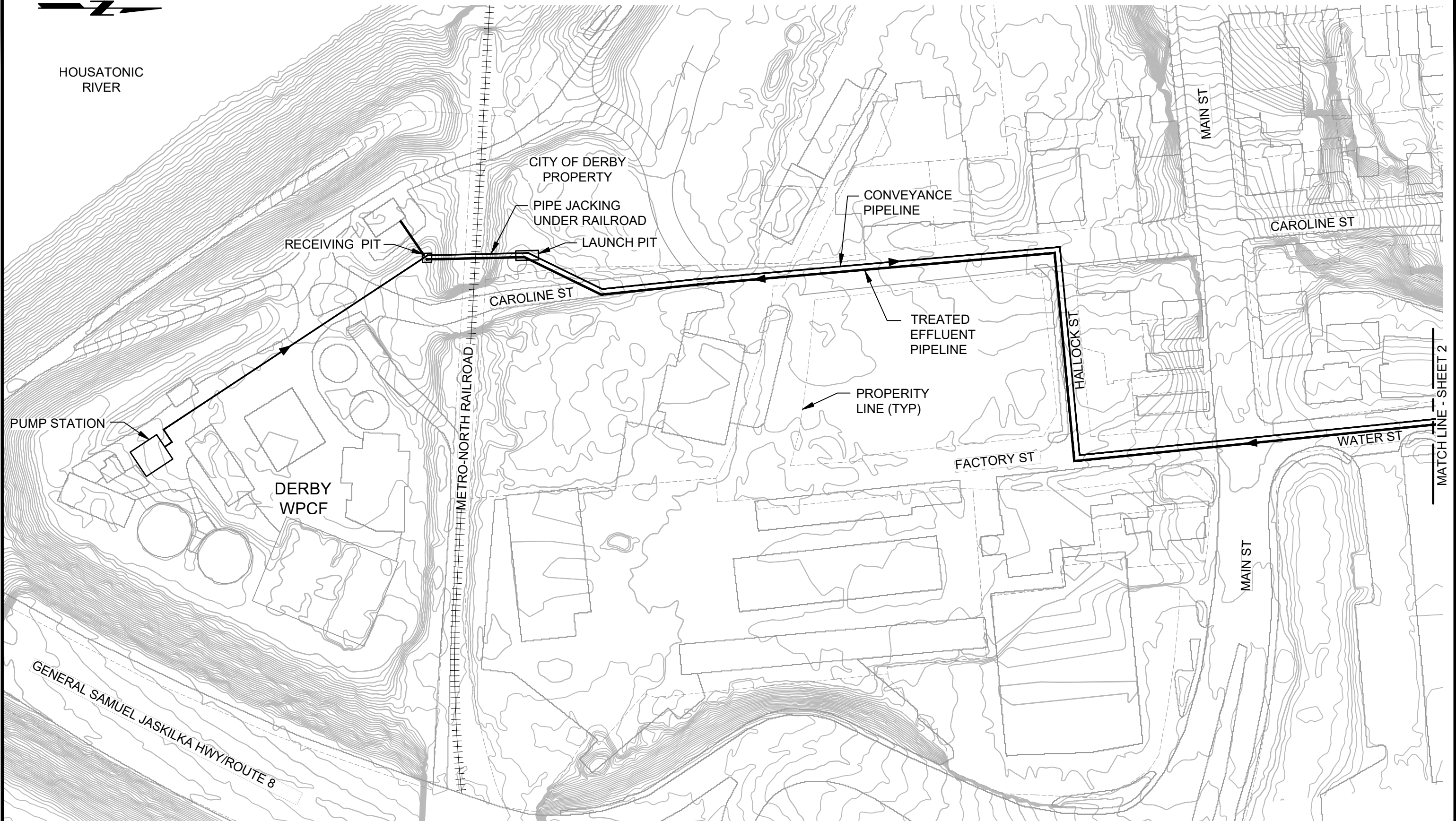


ANSONIA REGIONAL WPCF UPGRADE



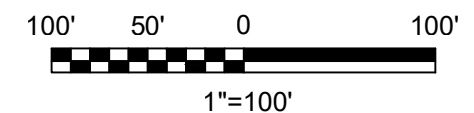


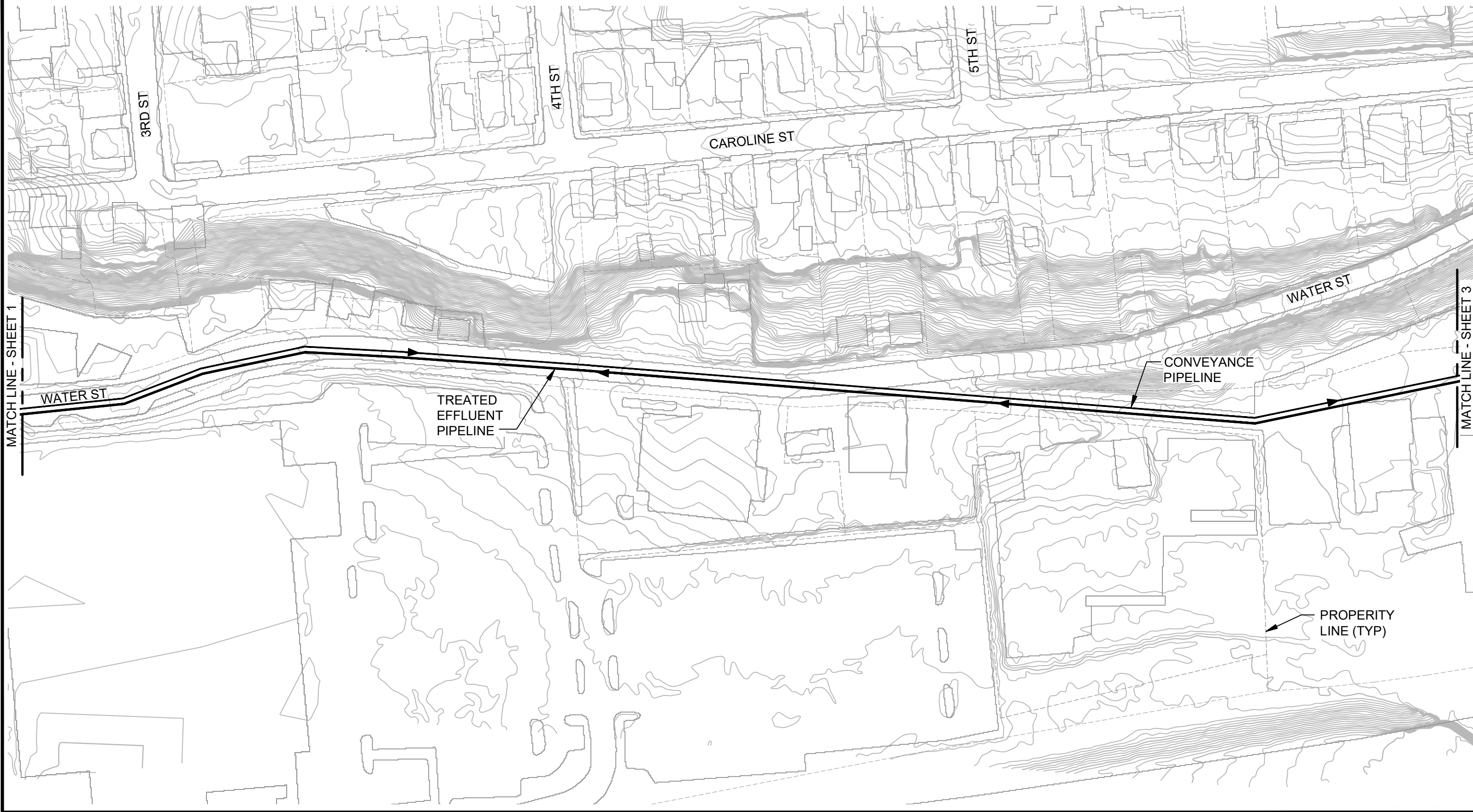
HOUSATONIC
RIVER



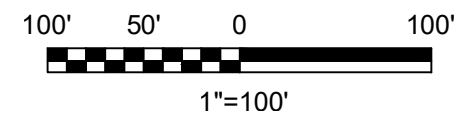
WASTEWATER REGIONALIZATION STUDY

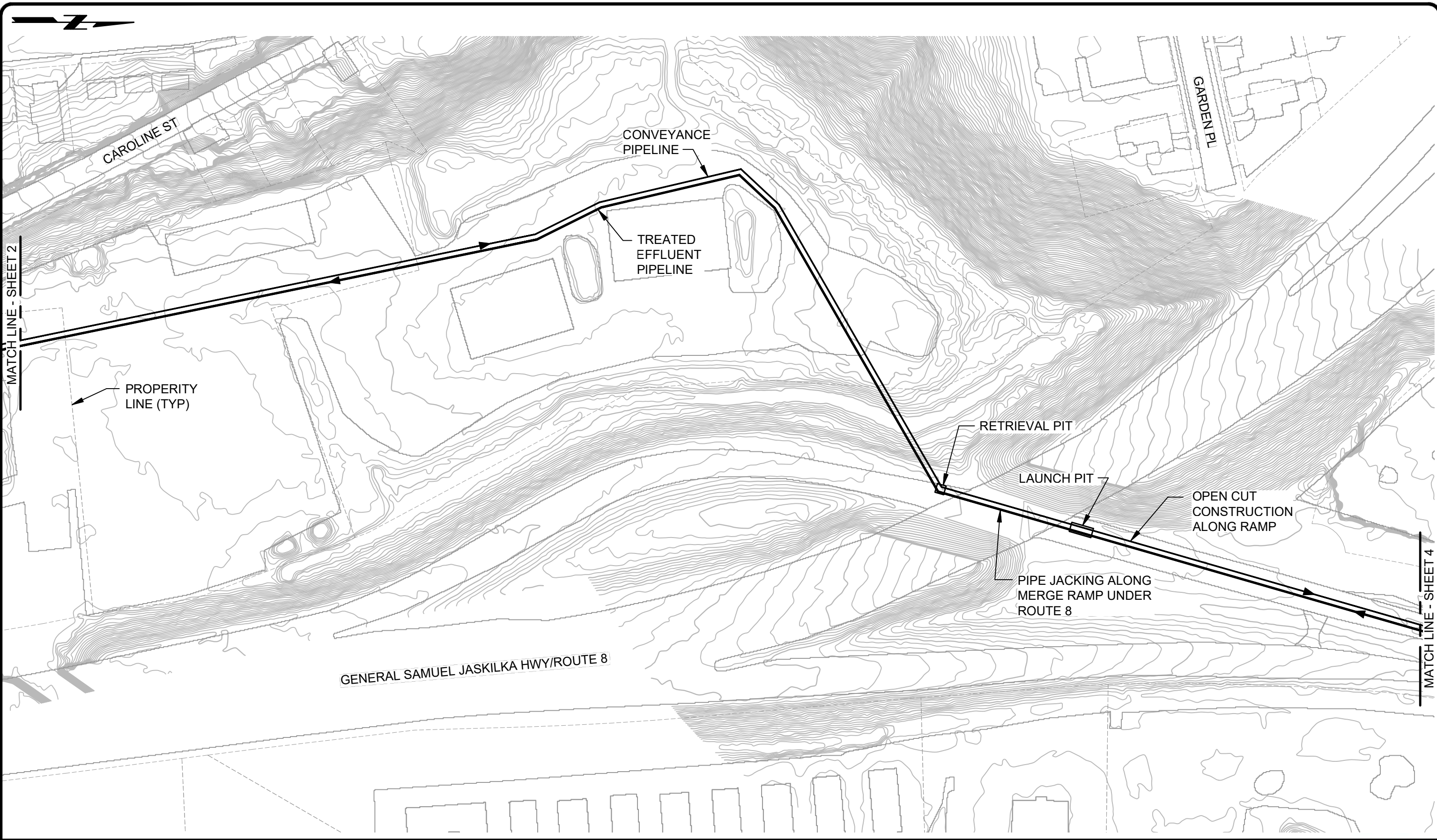
CONVEYANCE PIPELINE - DERBY TO ANSONIA
TREATED EFFLUENT - ANSONIA TO DERBY



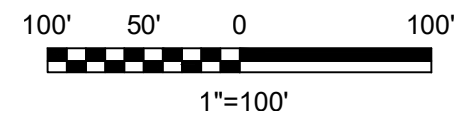


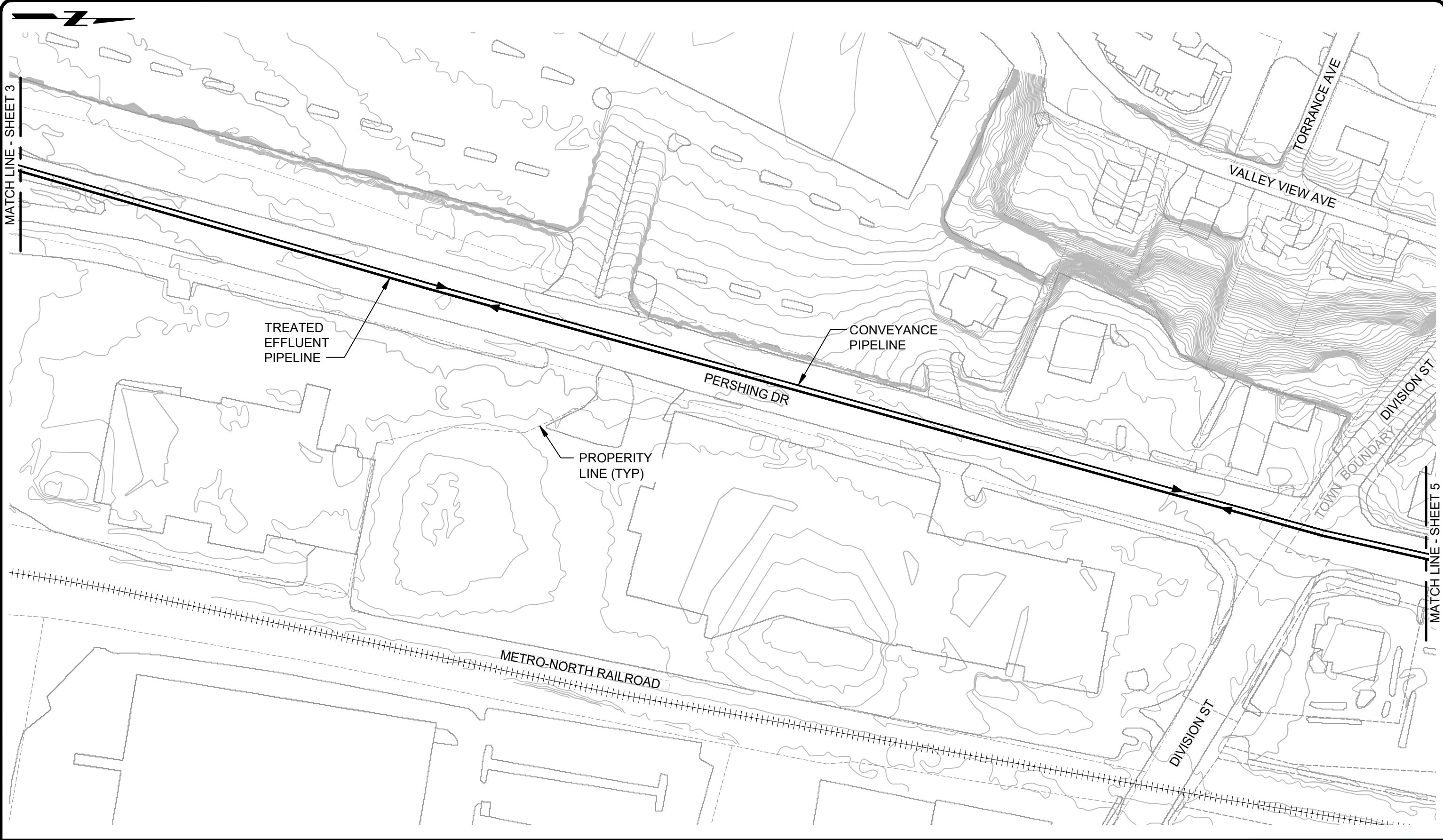
WASTEWATER REGIONALIZATION STUDY
CONVEYANCE PIPELINE - DERBY TO ANSONIA
TREATED EFFLUENT - ANSONIA TO DERBY





WASTEWATER REGIONALIZATION STUDY
 CONVEYANCE PIPELINE - DERBY TO ANSONIA
 TREATED EFFLUENT - ANSONIA TO DERBY





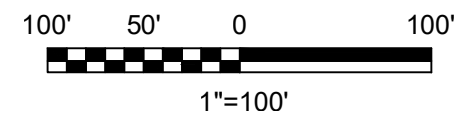
MATCH LINE - SHEET 3

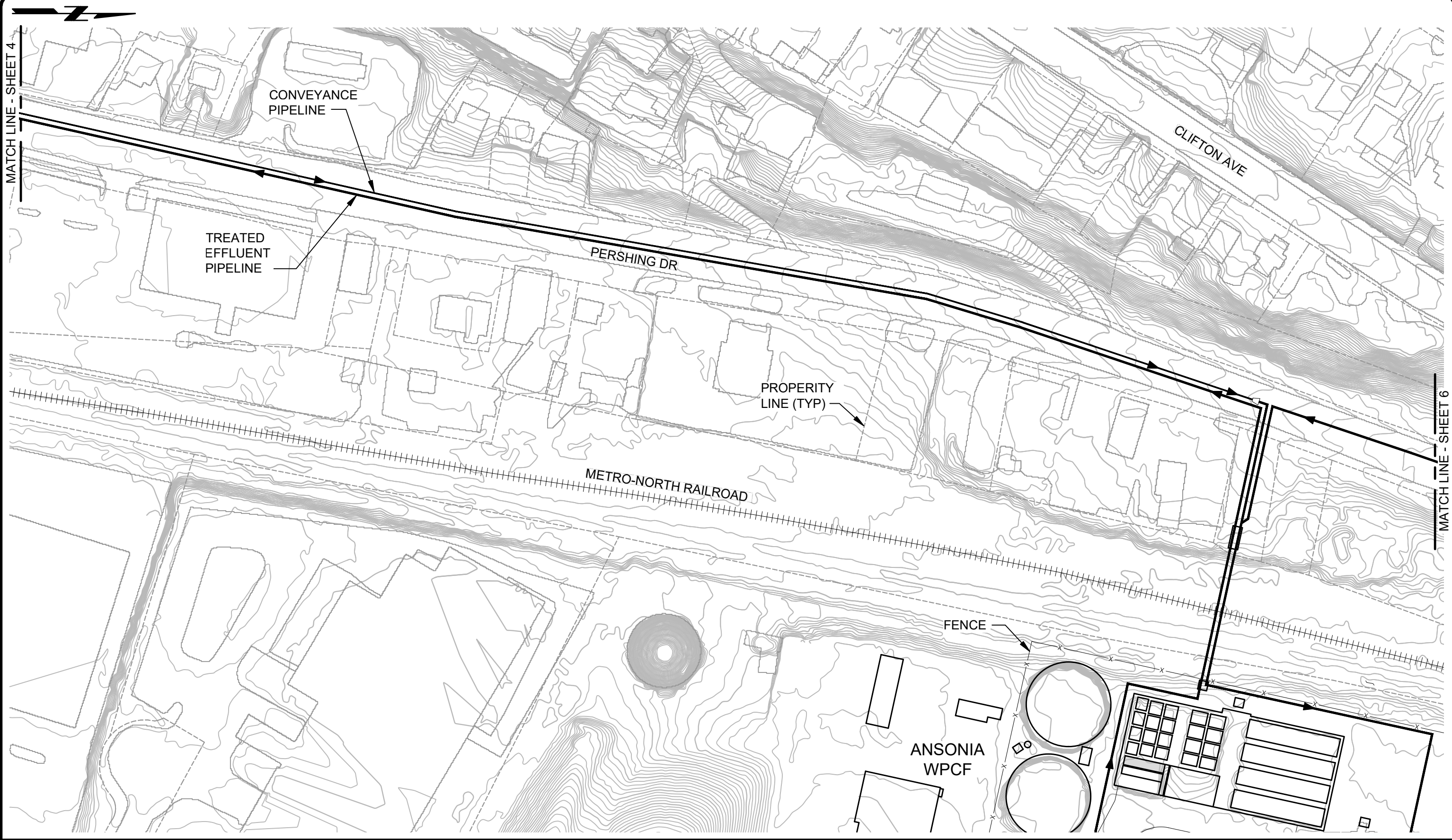
MATCH LINE - SHEET 5



WASTEWATER REGIONALIZATION STUDY

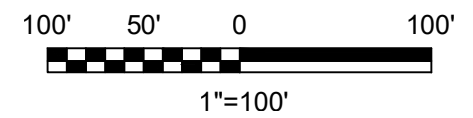
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TREATED EFFLUENT - ANSONIA TO DERBY

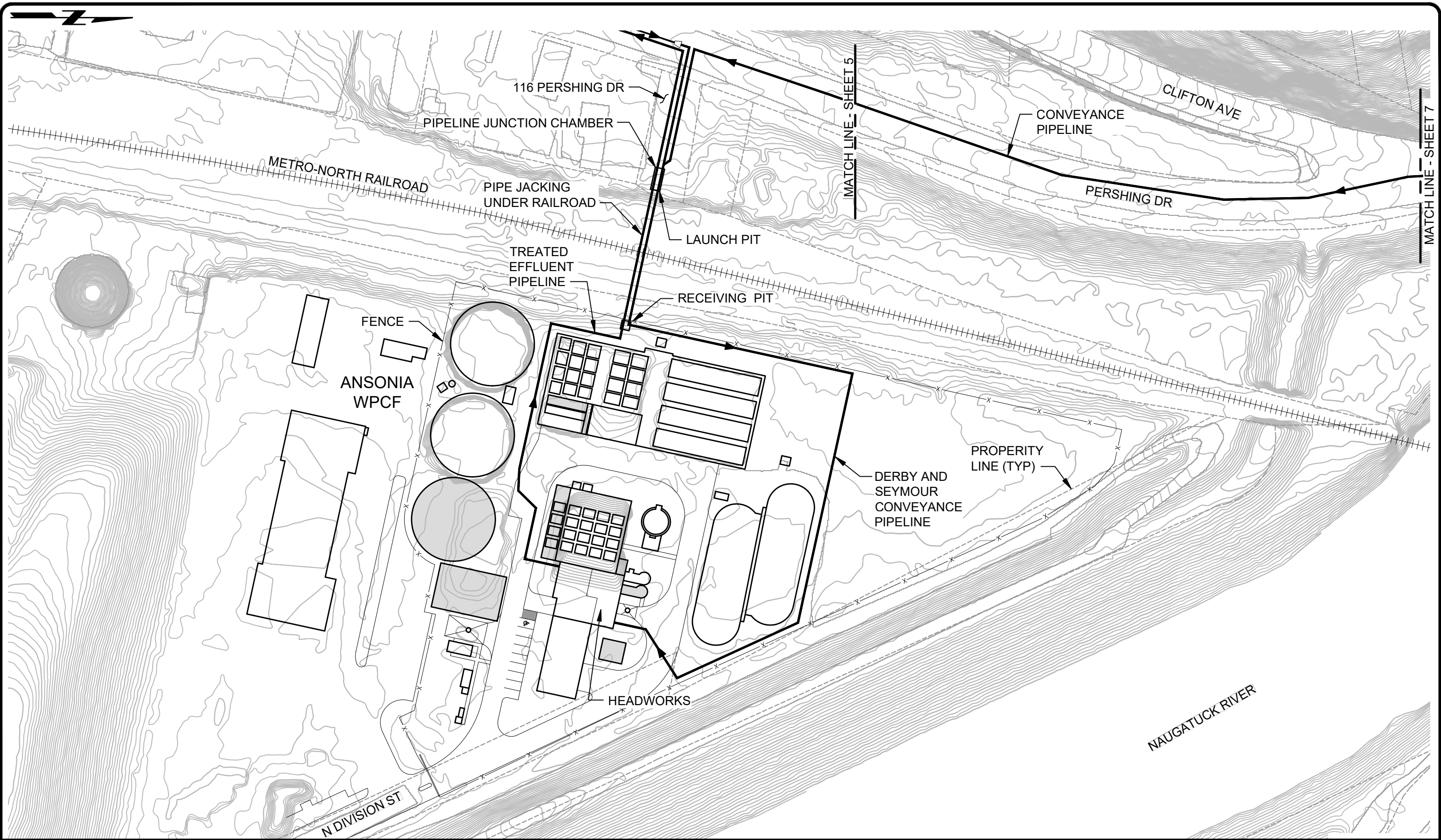




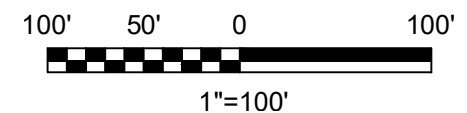
WASTEWATER REGIONALIZATION STUDY

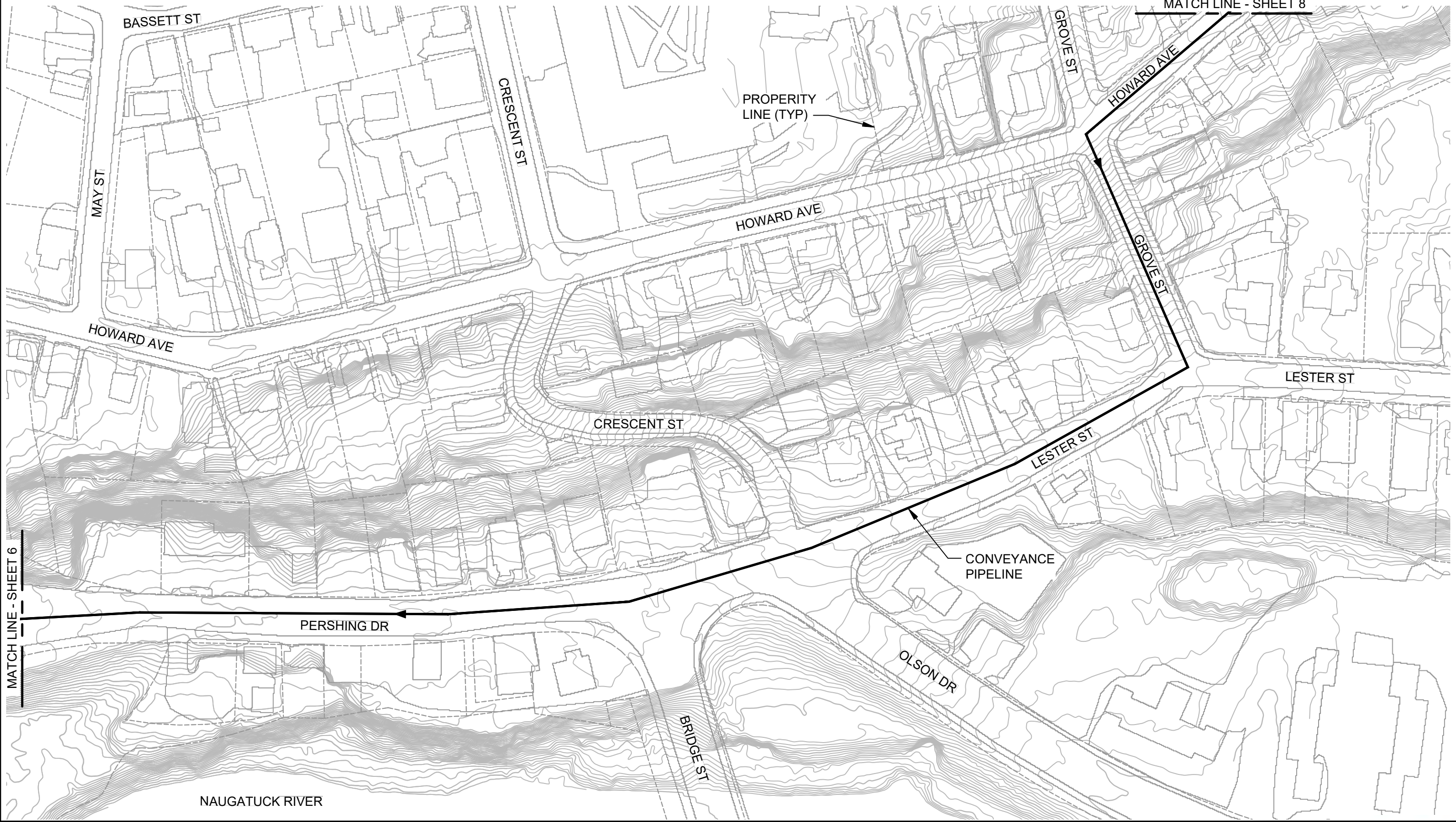
CONVEYANCE PIPELINE - DERBY TO ANSONIA
TREATED EFFLUENT - ANSONIA TO DERBY



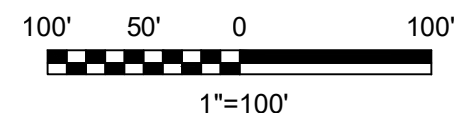


WASTEWATER REGIONALIZATION STUDY
CONVEYANCE PIPELINE - DERBY AND SEYMOUR
TERMINAL AT ANSONIA
TREATED EFFLUENT PIPELINE - ANSONIA TO DERBY



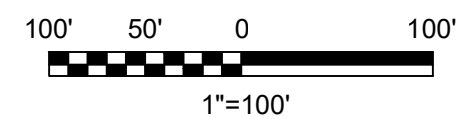


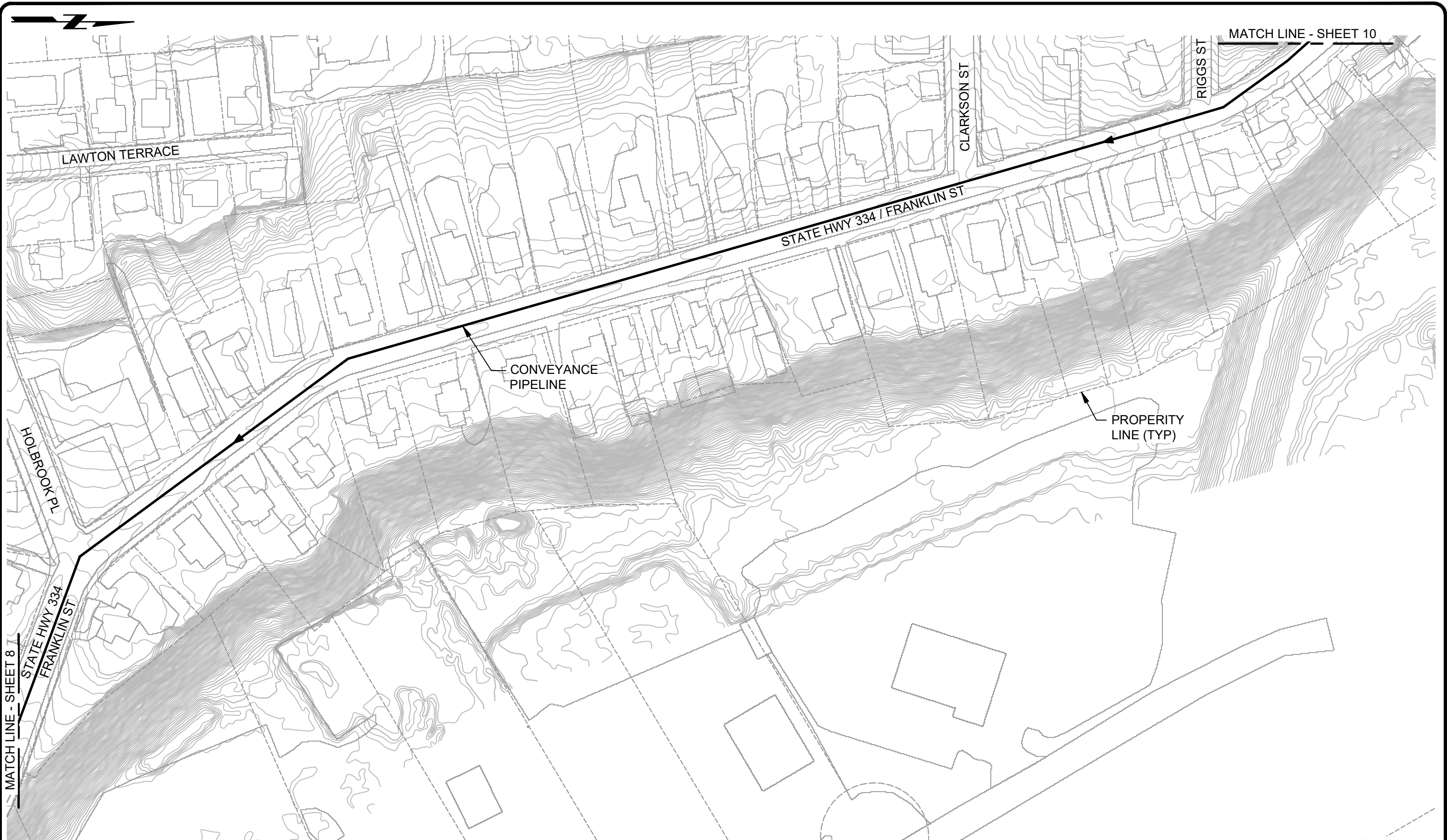
WASTEWATER REGIONALIZATION STUDY
CONVEYANCE PIPELINE - SEYMOUR TO ANSOANIA



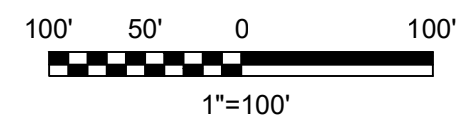


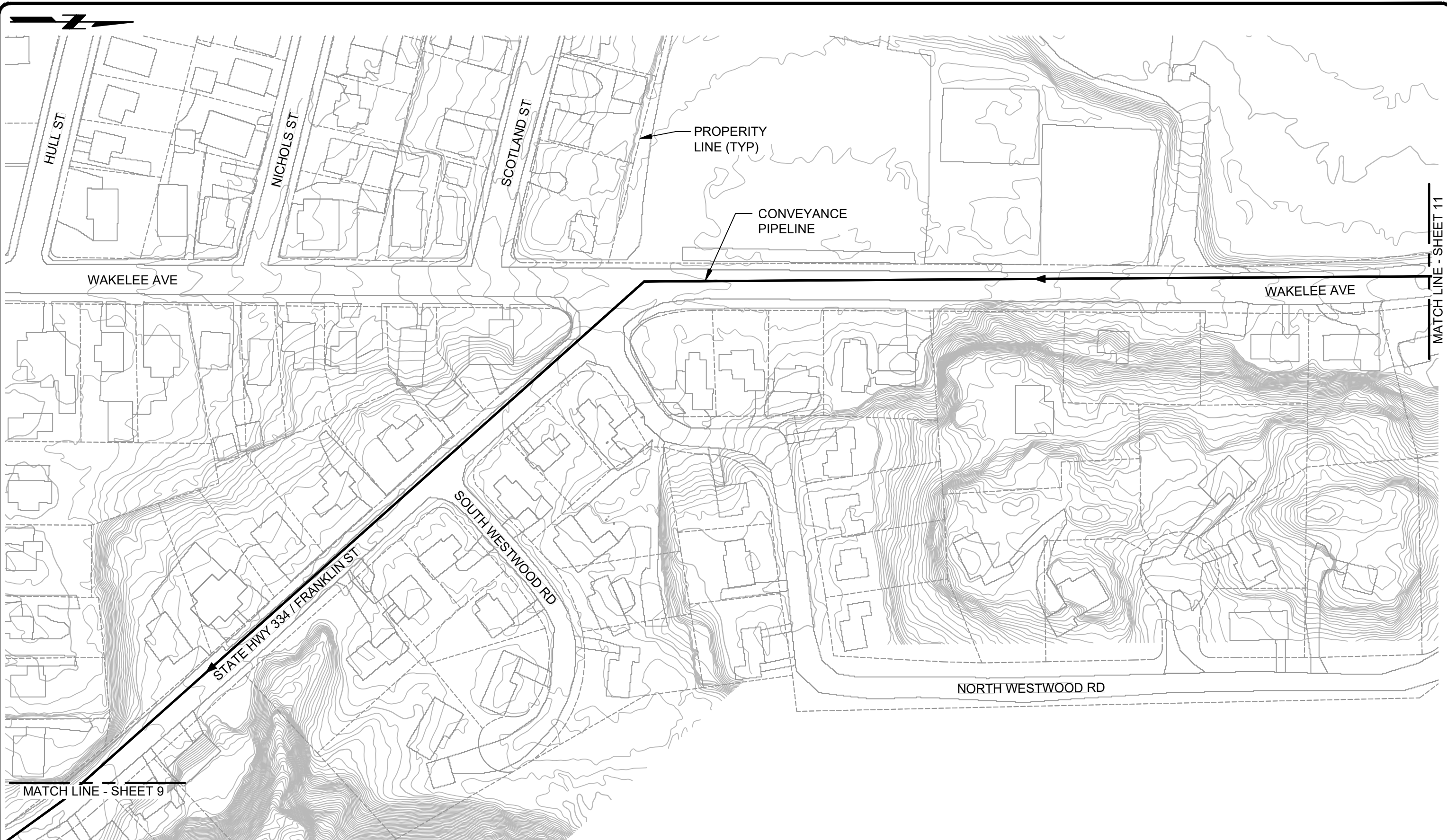
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CONVEYANCE PIPELINE - SEYMOUR TO ANSOANIA



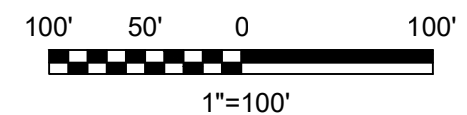


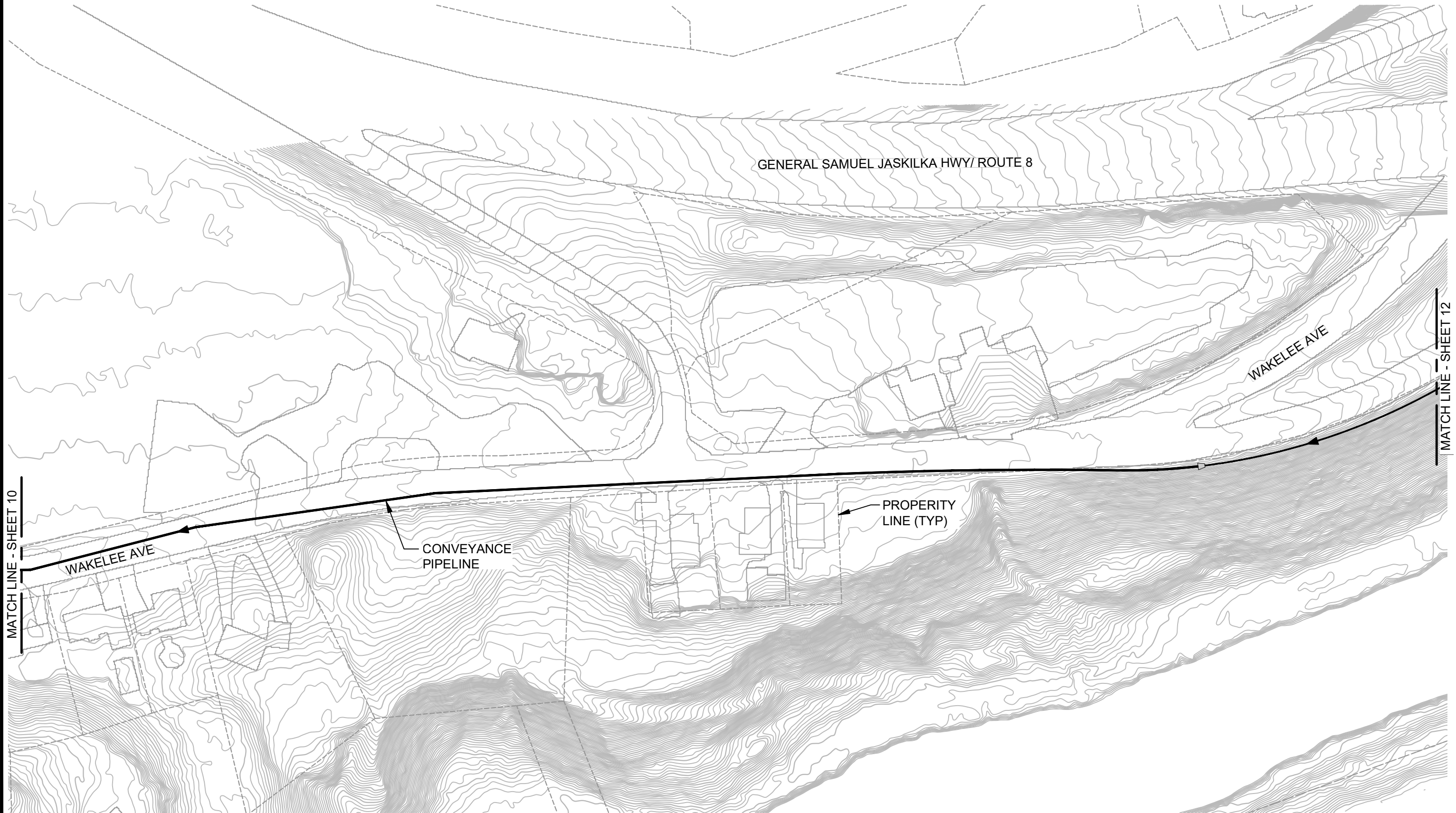
WASTEWATER REGIONALIZATION STUDY CONVEYANCE PIPELINE - SEYMOUR TO ANSOANIA



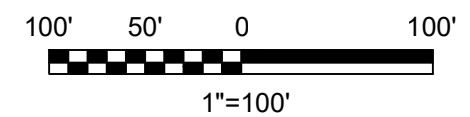


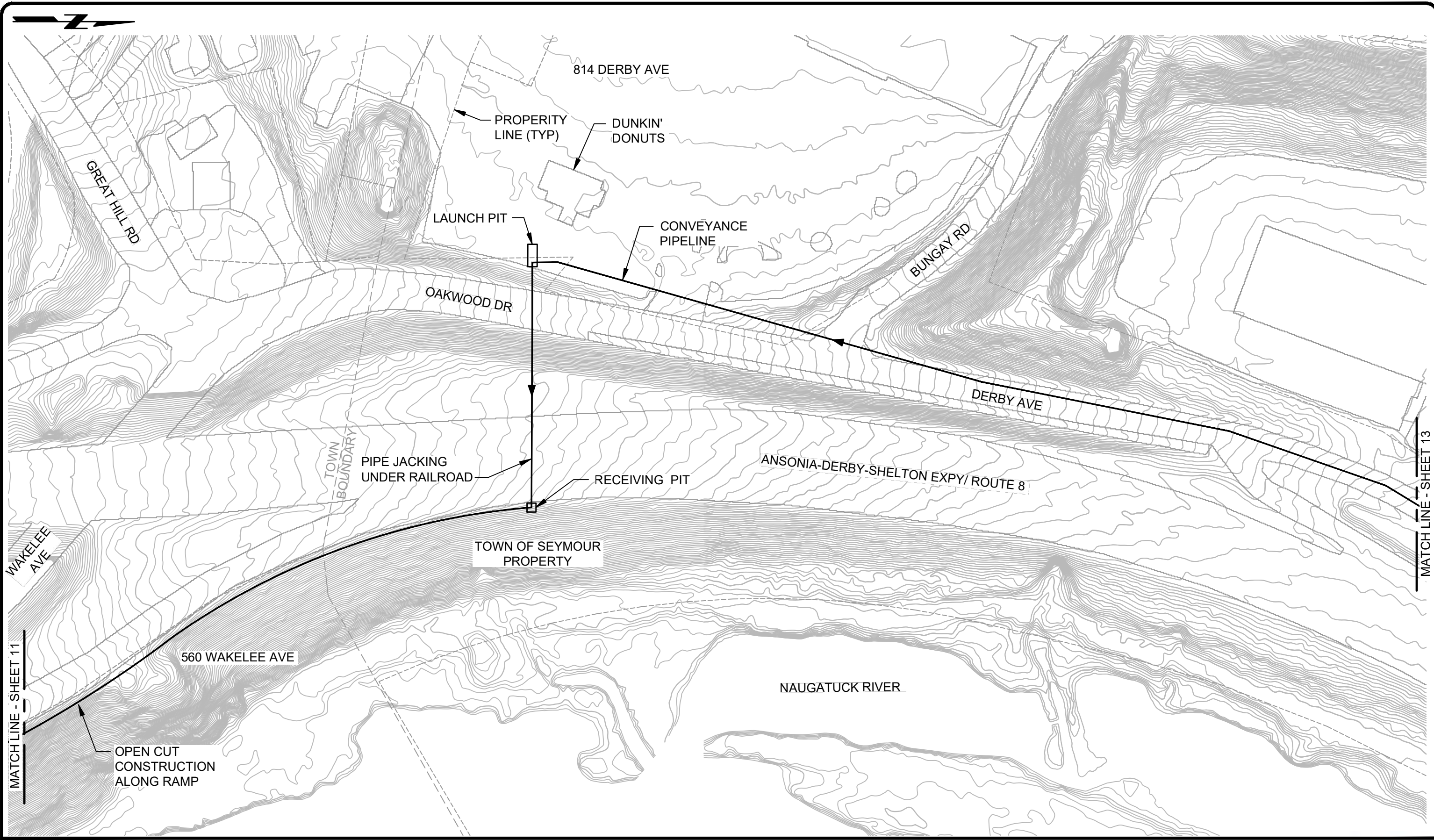
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CONVEYANCE PIPELINE - SEYMOUR TO ANSOANIA

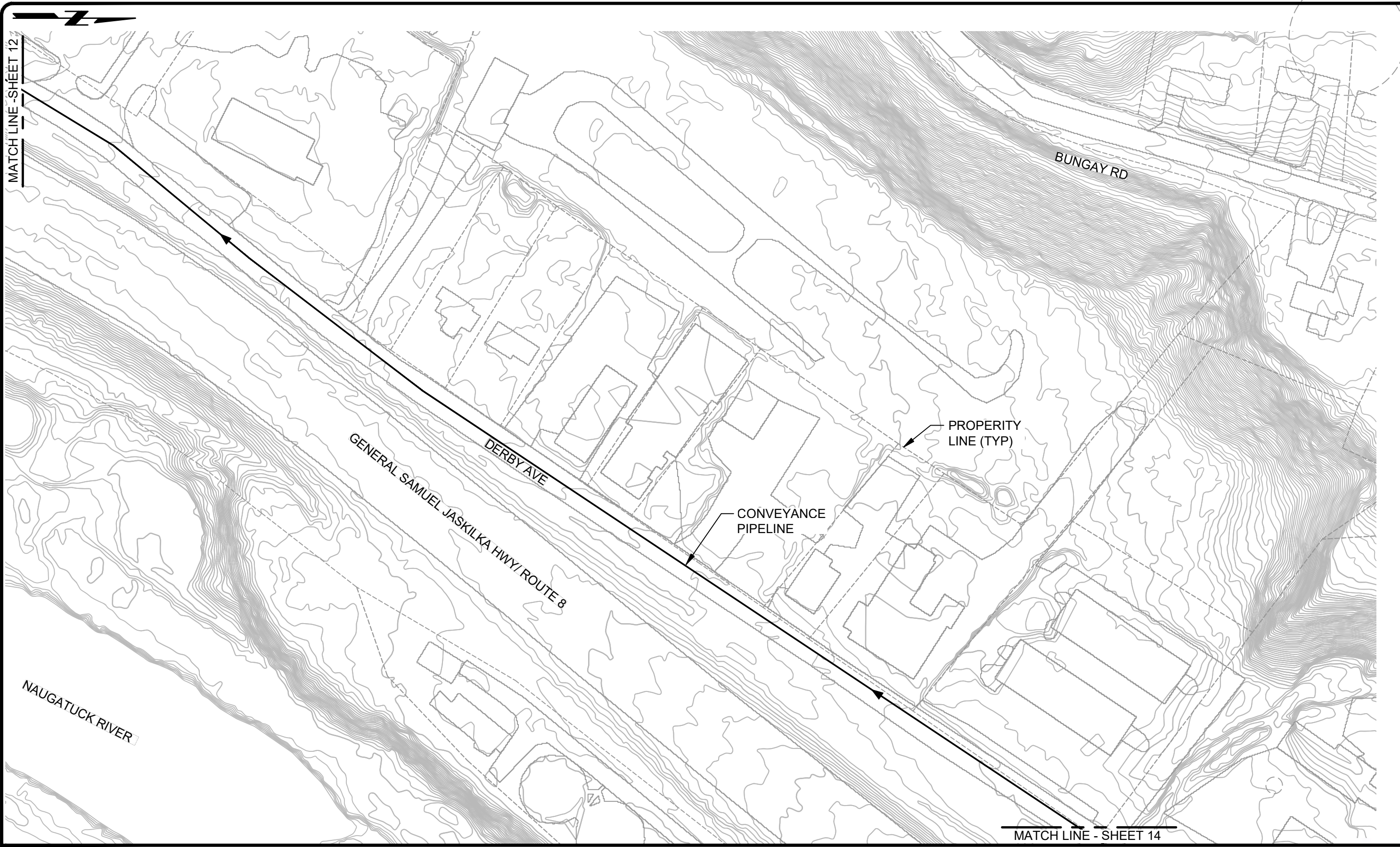




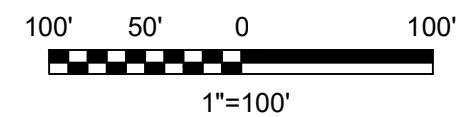
WASTEWATER REGIONALIZATION STUDY
CONVEYANCE PIPELINE - SEYMOUR TO ANSOANIA







WASTEWATER REGIONALIZATION STUDY
CONVEYANCE PIPELINE - SEYMOUR TO ANSOANIA





MATCH LINE - SHEET 13

ANSONIA-DERBY-SHELTON EXPY/ ROUTE 8

CONVEYANCE
PIPELINE

PROPERTY
LINE (TYP)

DERBY AVE

LAUNCH PIT

731 DERBY AVE

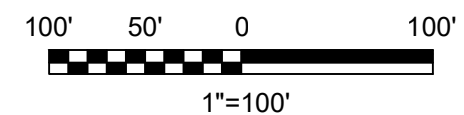
PIPE JACKING
UNDER HIGHWAY

RECEIVING PIT

SEYMOUR
WPCF



WASTEWATER REGIONALIZATION STUDY
CONVEYANCE PIPELINE - SEYMOUR TO ANSONIA



APPENDIX B

PHASE 1 REPORT: LONG LIST OF REGIONAL WASTEWATER SYSTEM ALTERNATIVES

- Technical Memorandum 1: Flows and Loads
- Technical Memorandum 2: Condition Assessment

REGIONAL WASTEWATER TREATMENT CONSOLIDATION STUDY

Phase 1 Report:

Long List of Regional Wastewater
System Alternatives

B&V PROJECT NO. 198910

PREPARED FOR

Naugatuck Valley Council of Governments

22 March 2019

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Appendices

A	Technical Memorandum 1: Flows and Loads (10/30/18)
B	Technical Memorandum 2: Condition Assessment (2/4/19)

EXECUTIVE SUMMARY

ES-1 Background and Objectives

- 1.1 The Naugatuck Valley Council of Governments (NVCOG) is undertaking a wastewater regionalization study that involves five municipalities in the region: Derby, Ansonia, Seymour, Beacon Falls and Naugatuck. A goal of this study is to identify the potential for economic efficiencies that regionalization may offer.
- 1.2 Under the existing setup, each of the five communities in the study has its own wastewater treatment plant, along with an associated collection system consisting of sewers and pumping stations. Regionalization alternatives, which would combine systems to reduce the number of treatment plants, offers the potential to reduce capital and operating expenses for the local communities through consolidating infrastructure and sharing staff resources.
- 1.3 The regionalization study is being performed in two phases. The first phase of the study (summarized here within the Phase 1 Report) developed the 20-year projected wastewater flows and loads for the five communities, and assessed the needs for capital expenditures over that planning horizon under the “base case” scenario of no regionalization.
- 1.4 The current report also identifies and describes a “long list” of regionalization alternatives that appear to have merit. The intent in Phase 2 is to screen the long list of alternatives, thereby creating a “short list” of favorable regionalization alternatives that can be further developed and compared to the base case, leading to final recommendations regarding regionalization.

ES-2 Population, Flow and Load Projections

- 2.1 The projected populations and flows and loads for the five communities were developed in Technical Memorandum No. 1, which is included as an appendix to this report.
- 2.2 Currently, the average flows to the treatment plants in each of the five communities are approximately half of the design permitted capacity.
- 2.3 According to the Connecticut State Data Center (CSDC), Ansonia and Derby are projected to grow by a total of approximately 6% by 2040; the other communities are projected to have lower growth. For the purpose of this study, CSDC projections were adjusted based on input from local officials to allow for modest, anticipated growth over the 20 year period of study.

2.4 With the exception of Beacon Falls, all of the communities in this study have older collection systems that are plagued with high infiltration and inflow (I/I). This results in very high peak flows to the treatment plants. The Derby treatment plant is unable to treat peak wet weather flows. Two of the communities are under Orders to reduce I/I from their collection systems.

ES-3 Condition Assessment of Existing Wastewater Infrastructure

3.1 A wastewater system condition assessment was conducted for each of the five communities. While some significant data gaps exist, this effort allowed for a high-level summary of the condition of existing wastewater treatment and collection system facilities, based on review of existing reports, interviews and site visits.

3.2 The Ansonia treatment plant is in good condition following a major upgrade completed in 2011. However, the other four plants are in fair to poor condition, and require major upgrades in the near future. In the case of Derby, this could approach full replacement of the plant.

3.3 In general, improvements to the collection system have been deferred for many years. As a result, this will require a period of catch-up for replacing and repairing pipes, followed by a sustained annual capital improvements program for buried infrastructure.

3.4 The condition assessment, which also projected planning-level capital costs for the 20-year planning study horizon under the base case of no regionalization, is summarized Technical Memorandum No. 2 which is provided as an appendix to this report.

3.5 The table below indicates the proposed capital expenditures that would be required for each of the five communities over the 20-year planning period, under the base case of no regionalization.

Projected 20-Year Wastewater Expenditures, Base Case (If No Regionalization)

	Derby	Ansonia	Seymour	Beacon Falls	Naugatuck	Total
Water Pollution Control Facility	\$ 70.0M	\$ 15.0M	\$ 40.0M	\$ 14.0M	\$ 55.0M	\$ 194.0M
Collection System	\$ 8.0M	\$ 10.3M	\$ 8.5M	\$ 3.1M	\$ 18.5M	\$ 48.4M
Large Pumping Stations	\$ 4.2M	\$ 3.0M	\$ 2.0M	\$ 0.5M	\$ 1.0M	\$ 10.7M
TOTAL	\$ 82.2M	\$ 28.3M	\$ 50.5M	\$ 17.6M	\$ 74.5M	\$ 253.1M

ES-4 “Long List” of Regionalization Alternatives

- 4.1 Phase 1 of this study also identified 12 regionalization alternatives (examples: sending all Derby flow to Ansonia; or sending all flow from Seymour and Ansonia to Derby). Some of those alternatives also included a variation involving a more aggressive approach to I/I reduction.
- 4.2 Planning level sewer pipeline corridors were identified for major system interconnection trunk sewers or force mains. These would allow the communities to interconnect for regionalization purposes. In some cases, multiple interconnection sewer or force main routing options were identified.
- 4.3 During the initial rough screening of the long list of alternatives, one of the regional wastewater alternatives was identified as clearly inferior to other alternatives, and therefore rejected from further consideration.
- 4.4 The remaining 11 alternatives are of potential interest, depending on the relative costs of construction and operation. During Phase 2 of this study, this list of 11 alternatives would be screened further to a shorter list of preferred alternatives, which then would undergo more detailed study and analysis. A recommendation(s) would be made at the end of Phase 2.

1.0 Purpose and Background

The Naugatuck Valley Council of Governments (NVCOG) is undertaking a regional wastewater treatment consolidation study comprising five municipalities in the region: Naugatuck, Beacon Falls, Seymour, Ansonia and Derby. Investigations on the viability of wastewater regionalization are not entirely new to the region, as there have been reviews and study on this subject in the past, involving several of the study communities.

Each of the five communities has their own wastewater system, where several of the wastewater treatment plants are in need of significant upgrade to replace major equipment and systems that have reached the end of their useful life, and also to incorporate new treatment systems for the reduction of phosphorous from their effluent discharge as a result of new regulations by the CT Department of Energy and Environmental Protection (DEEP). Prior studies of the wastewater collection systems for some of these communities have also revealed the need for upgrade and rehabilitation, including the abatement of infiltration and inflow (I/I). Some of the communities have Orders issued to them by DEEP and/or EPA requiring them to undertake capital and O&M improvements to provide for greater levels of wastewater treatment, improvements within the collection system and overall strengthening of systems reliability. Taken together, the plant and collection system upgrades will be a significant cost to these communities individually.

An important goal of this study is to identify the potential for economic efficiencies that regionalization may offer, as compared with each of the municipalities continuing to go it alone. Regionalization's attractiveness lies in its basis that sharing costs for wastewater infrastructure, operations and management will be less while meeting desired environmental objectives.

The regionalization study is being performed in two phases. The principal goal of Phase 1 (the current phase) is to define the practical universe of regional wastewater treatment alternatives, identifying a workable "long list" of alternatives that merit more in-depth evaluation and study in Phase 2. Screening criteria, to be used in Phase 2 to compare the regional alternatives are also defined in this Phase 1 report. These criteria include the following categories: technical feasibility, operations and maintenance, efficiency, community-based, environmental, schedule, regulatory and permitting, and cost.

Phase 1 work has developed 20-year projected wastewater flows and loads for the study communities. A planning level assessment of the wastewater treatment and collections systems of the five communities has also been undertaken. To the extent practical, this work has relied heavily on existing planning and engineering reports related to the technical needs and costs of the wastewater collection and treatment facilities in the five communities. However, significant gaps exist in the data to properly describe the infrastructure capital projects and costs of these communities out to 2040, the planning level horizon year of this study. On-site reviews and meetings with community representatives were utilized to help fill some but not all of the gaps in the available information. These assessments, identified as the *Base Case*, serve to identify the treatment and collection system needs and associated capital costs for each of the five communities. The Base Case will continue to be developed more fully in Phase 2 and will be compared against regional wastewater alternatives to assess cost effectiveness, reliability and compliance with environmental requirements. The flows and loads projections are included as Appendix A to this report, and the infrastructure Base Case determinations are included as Appendix B to this report.

This Phase 1 report identifies and describes a long list of regional wastewater alternatives that appear to have initial merit for regionalization. The intent is that this long list of alternatives will be carried into

Phase 2 where a screening-out evaluation of alternatives will be conducted to identify and eliminate the least promising alternatives which appear to have less attractive attributes as compared to other peer alternatives. This would result in a “short list” of regional alternatives. The screened list of regional alternatives, or short list in Phase 2, will be compared to the ‘Base Case’ alternative for final analysis and recommendation. Cost and non-cost criteria will be used to evaluate the regionalization alternatives versus the Base Case where each community acts individually to meet their wastewater infrastructure needs.

A more detailed condition assessment of treatment plant systems will also be undertaken in Phase 2. Targeted flow monitoring may be conducted within certain parts of the collection systems. This work, along with other analyses, will better define the flows, I/I contribution, costs, schedule, environmental and permitting requirements, as well as other pertinent complexities related to the regional short list of alternatives. Phase 2 will also present a preferred alternative from the short list of alternatives including its selection basis.

2.0 Identification of Alternatives

2.1 Approach

Regionalization is attractive because of the economies of scale in the cost of building, upgrading and operating wastewater treatment facilities. Therefore, regionalization allows for a sharing of physical infrastructure, as well as sharing management, operations, and administration. However, there are two additional cost factors which must be considered in a regional alternative. One is the cost of piping, and possibly pumping, to a regional facility; the other is the potential cost of providing higher levels of treatment at the regional facility due to discharging a greater wastewater load in a particular area of the receiving water. This latter scenario may require careful consideration as more wastewater discharged at a given point on the river may impose a greater impact on water quality for that specific area. Therefore, it is the purpose of this regional study to combine the above considerations into definable, regional alternatives, and to compare them to the base case on a life-cycle cost basis, taking into consideration environmental benefits and cost efficiencies, while working with NVCOG, DEEP and all stakeholders to determine if regional solutions have merit. These financial comparisons will be made in Phase II of this study.

2.2 Initial List of Regional Wastewater System Alternatives

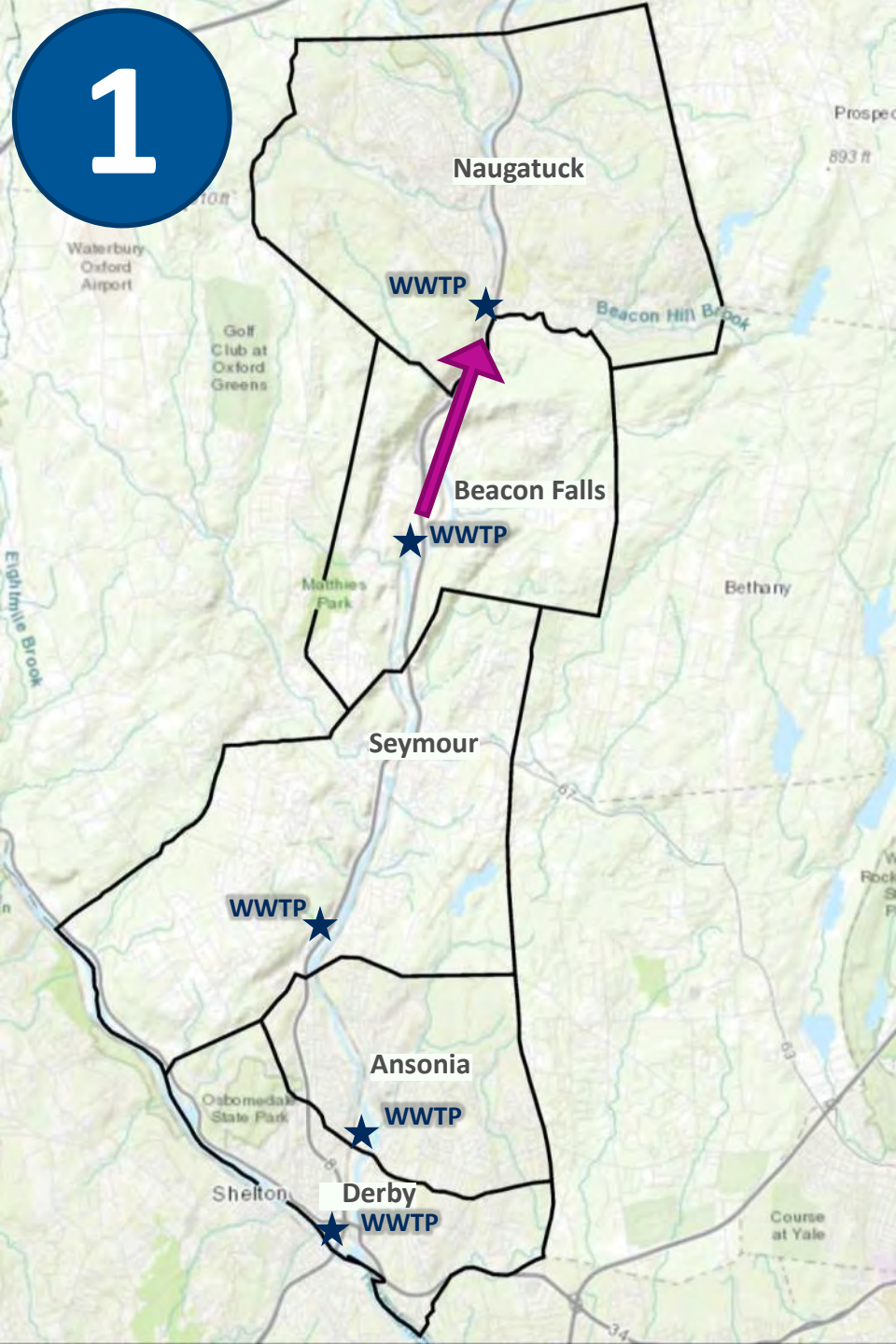
Regionalization alternatives among the five communities in this study will need to physically connect their sewerage collection systems with conveyance pipelines and pump stations. Constructing pipelines over long distances with pump stations capable of moving a community's wastewater miles away can be expensive. Thus, the cost of the connecting pipelines and pump stations will be considered in Phase II, when the regionalization alternatives are compared to the Base Case, where each community continues to act alone to invest and manage its existing wastewater infrastructure (i.e. plants and sewer collection system).

Identification of regional wastewater alternatives was not limited in any way as part of this report; however, the initial list of alternatives generally targeted adjacent communities (e.g. Seymour and Beacon Falls or Derby and Ansonia) as opposed to communities that are not adjacent or close to each other (e.g. Naugatuck and Derby). The initial list of regional wastewater alternatives is identified in Table 1. Some of the alternatives are variations of each other in that one alternative would convey current wastewater flows and its sister alternative calls for reduction in flow through implementation of an intensive I/I control program. Each alternative is presented in greater detail in tabular summaries following Table 1.

Table 1 Initial List of Regional Wastewater Alternatives

Alternative No.	Description	Abbreviated Description
1	Beacon Falls to Naugatuck	BF→N
2	Beacon Falls to Seymour	BF→S
2a	Beacon Falls to Seymour, I/I Reduction	BF→S, I/I
3	Derby to Ansonia	D→A
3a	Derby to Ansonia, I/I Reduction	D→A, I/I
4	Derby to Ansonia, Effluent Pumped to Housatonic	D→A→H
4a	Derby to Ansonia, I/I Reduction, Effluent Pumped to Housatonic	D→A→H, I/I
5	Derby and Seymour to Ansonia	D&S→A
5a	Derby and Seymour to Ansonia, I/I Reduction	D&S→A, I/I
5b	Derby and Seymour to Ansonia, Effluent to Housatonic	D&S→A→H
5c	Derby and Seymour to Ansonia, I/I Reduction, Effluent to Housatonic	D&S→A→H, I/I
6	Derby to Seymour and Ansonia	D→S, D→A
6a	Derby to Seymour and Ansonia, I/I Reduction	D→S, D→A, I/I
7	Derby to Seymour, Ansonia, and Derby	D→S, D→A, D→D
7a	Derby to Seymour, Ansonia, and Derby, with I/I Reduction	D→S, D→A, D→D, I/I
8	Ansonia to Derby	A→D
8a	Ansonia to Derby, I/I Reduction	A→D, I/I
9	Seymour and Ansonia to Derby	S&A→D
9a	Seymour and Ansonia to Derby, I/I Reduction	S&A→D, I/I
10	Seymour to Ansonia, Part of Ansonia to Derby	S→A, A→D
10a	Seymour to Ansonia, Part of Ansonia to Derby, I/I Reduction	S→A, A→D, I/I
11	Beacon Falls and Seymour to Ansonia, Part of Ansonia to Derby	BF,S→A, A→D
11a	Beacon Falls and Seymour to Ansonia, Part of Ansonia to Derby, I/I Reduction	BF,S→A, A→D, I/I
12	Beacon Falls, Seymour, and Ansonia to Derby	BF,S,A→D
12a	Beacon Falls, Seymour, and Ansonia to Derby, I/I Reduction	BF,S,A→D, I/I

1



Beacon Falls to Naugatuck

Treatment Plants

Naugatuck

~~Beacon Falls~~

Seymour

Ansonia

Derby

Overview

Under this alternative, the Beacon Falls WPCF would be decommissioned and replaced with a pumping station, to pump all wastewater flow to the Naugatuck WPCF.

Treatment Capacity

Since the Naugatuck WPCF has been operating at less than half of its annual average design capacity, that plant has available capacity to receive all of the flow from Beacon Falls (0.45 MGD annual average, 1.525 peak hydraulic flows) and to accommodate projected population growth, without further expansion.

Collection System

Naugatuck

Beacon Falls

Seymour

Ansonia

Derby

Collection System Rehabilitation

The base case assumes maintenance for each collection system necessary to improve the system to a basic level of reliability and performance consistent with the I/I reduction assumptions in the flows and loads analysis. This alternative does not include additional collection system improvements.

Operations & Maintenance

In this alternative, there could be minor savings in the maintenance of the collection system in Beacon Falls by relying on greater resources and potentially efficiency of Naugatuck O&M (Veolia), but this is not necessarily the case.

Lift Stations & Conveyance

Naugatuck

Beacon Falls

Seymour

Ansonia

Derby

Capacity

A lift station and force main would be required to convey flow from Beacon Falls to Naugatuck. It is assumed that the new Beacon Falls pumping station would be designed for the peak hydraulic flow (1.525 MGD), unless it is more cost-effective to reduce peak pumping capacity by providing some wet weather flow equalization storage at the pump station.

Conveyance Corridors

The elevation differential between Beacon Falls (138 ft) and Naugatuck (178 ft) is 40 ft, however, the terrain and existing rights of way pose significant constraints to this alternative.

Four preliminary alternative pipe routes were identified. The primary alternatives involve (a) going over or around Toby's Rock Mountain,

with a peak alignment elevation up to 780 ft or (b) following the Naugatuck River, either along the railroad or the Route 8 right of way. Pipe lengths could range from approximately 17,000 ft to 28,000 ft.

Potential routing corridors are discussed in greater detail in Section 3.

This alternative is not mutually exclusive with alternatives involving Seymour, Ansonia, and Derby.

2

Beacon Falls to Seymour

Treatment Plants

Naugatuck

~~Beacon Falls~~

Seymour

Ansonia

Derby

Overview

Under this alternative, the Beacon Falls WPCF would be decommissioned and replaced with a pumping station. All Beacon Falls wastewater would be pumped to the Seymour WPCF.

Treatment Capacity

Although the Seymour WPCF has available capacity to accommodate all flow from Beacon Falls under average annual and max month conditions, the peak flows due to I/I from Seymour and Beacon Falls would exceed the hydraulic capacity of the existing plant.

Increased plant capacity, high rate treatment, or storage (or some combination thereof) would be required.

Collection System

Naugatuck

Beacon Falls

Seymour

Ansonia

Derby

Collection System Rehabilitation

The base case assumes maintenance for each collection system necessary to improve the system to a basic level of reliability and performance consistent with the I/I reduction assumptions in the flows and loads analysis. This alternative does not include additional collection system improvements.

Operations & Maintenance

In this alternative, there could be minor savings in the maintenance of the collection system in Beacon Falls by relying on O&M resources and Seymour O&M (Veolia), but this is not necessarily the case.

Lift Stations & Conveyance

Naugatuck

Beacon Falls

Seymour

Ansonia

Derby

Capacity

It is assumed that the new Beacon Falls pumping station would be designed for the peak hydraulic flow (1.525 MGD), unless it is more cost-effective to reduce peak pumping capacity by providing some wet weather flow equalization storage at the pump station.

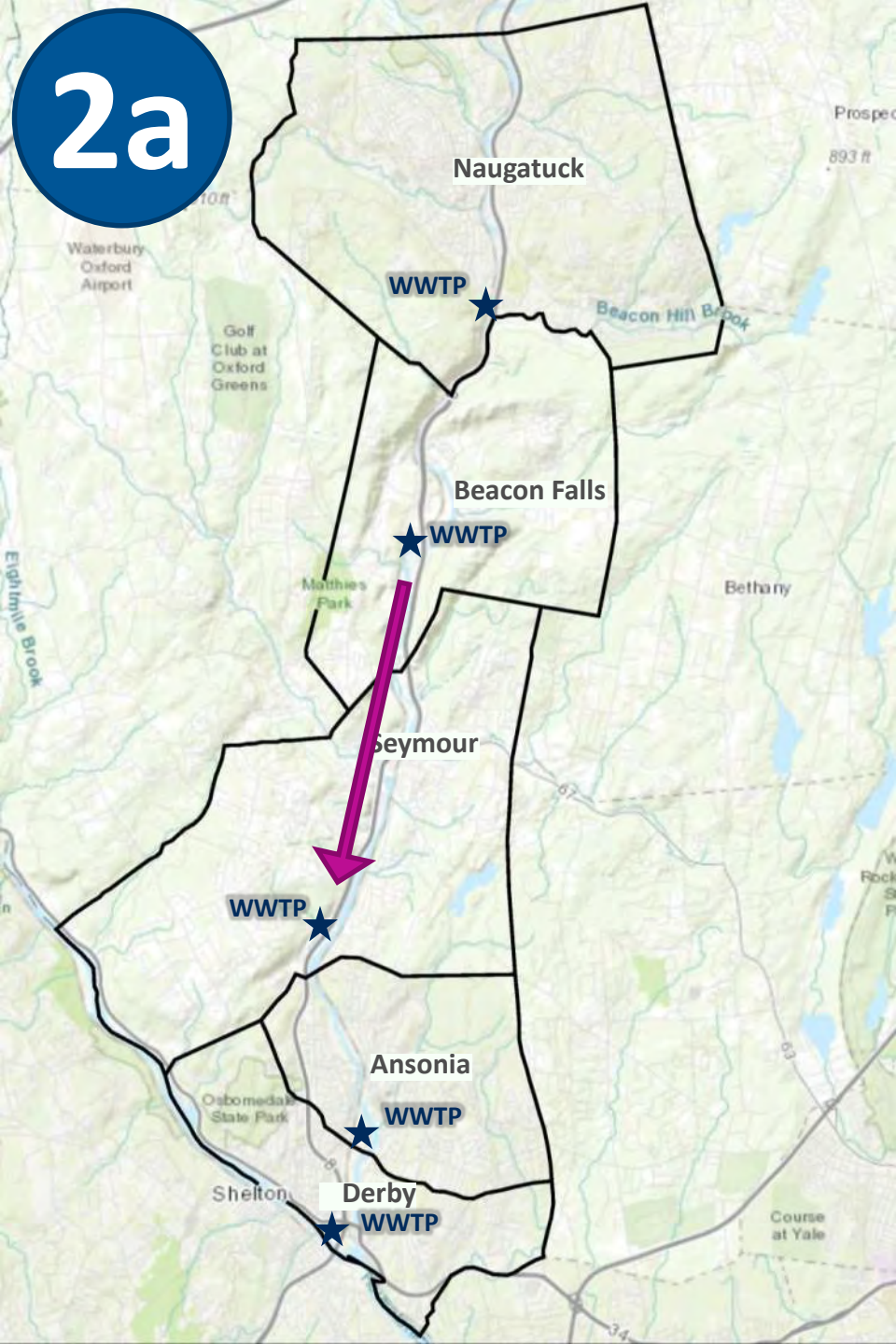
Conveyance Corridors

The elevation differential between Beacon Falls (138 ft) and Seymour (71 ft) is 67 ft. However, even with this elevation difference, the average slope of the shortest route (0.2%) would likely require a lift station.

Three potential routes were identified for comparison. The primary alternatives involve

(a) going over or around the hills west of the Naugatuck River, with peak alignment elevations of approximately 470 ft to 640 ft, or (b) following the Naugatuck River, either along the railroad or the Route 8 right of way. Pipe lengths could range from approximately 26,000 ft to 48,000 ft.

2a



Beacon Falls to Seymour, I/I Reduction

Treatment Plants

Naugatuck

~~Beacon Falls~~

Seymour

Ansonia

Derby

Under this alternative, the Beacon Falls WPCF would be decommissioned and replaced with a pumping station. All Beacon Falls wastewater would be pumped to the Seymour WPCF.

Treatment Capacity

Since the Seymour WPCF has been operating at less than half of its annual average design capacity, it has available capacity to accommodate all flow from Beacon Falls under average annual and max month conditions, through 2040. Depending on the extent of I/I removal, additional measures may be required

to accommodate peak hourly flows, such as increased wet weather treatment capacity at the Seymour WPCF or storage facilities.

Collection System

Naugatuck

Beacon Falls

Seymour

Ansonia

Derby

Collection System Rehabilitation

Implementing an aggressive I/I program in Seymour and Beacon Falls could reduce peak hydraulic flows from the combined collection systems to approximately 7.3 MGD, to reduce or eliminate the need for WPCF expansion to accommodate peak flows. Storage may also be considered to manage peak flows in conjunction with I/I removal.

Operations & Maintenance

In this alternative, there could be minor savings in the maintenance of the collection system in Beacon Falls by relying on O&M resources and Seymour O&M (Veolia), but this is not necessarily the case.

Lift Stations & Conveyance

Naugatuck

Beacon Falls

Seymour

Ansonia

Derby

Capacity

It is assumed that the new Beacon Falls pumping station would be designed for the peak hydraulic flow (1.525 MGD), unless it is more cost-effective to reduce peak pumping capacity by providing some wet weather flow equalization storage at the pump station.

Conveyance Corridors

The elevation differential between Beacon Falls (138 ft) and Seymour (71 ft) is 67 ft. However, even with this elevation difference, the average slope of the shortest route (0.2%) would likely require a lift station.

Three potential routes were identified for comparison. The primary alternatives involve

(a) going over or around the hills west of the Naugatuck River, with peak alignment elevations of approximately 470 ft to 640 ft, or (b) following the Naugatuck River, either along the railroad or the Route 8 right of way. Pipe lengths could range from approximately 26,000 ft to 48,000 ft.

3

Derby to Ansonia

Treatment Plants

Naugatuck

Beacon
Falls

Seymour

Ansonia

Derby

Under this alternative, the Derby WPCF would be decommissioned and replaced with a pumping station. All Derby wastewater would be pumped to the Ansonia WPCF.

Wet Weather Capacity

The Ansonia WPCF currently has sufficient capacity to accept the annual average flows from Derby. However, due to high wet weather

flows in both Ansonia and Derby, the Ansonia WPCF does not have the capacity to handle the combined peak flows. Therefore, this alternative would require upgrading the WPCF and/or providing storage or high rate treatment to increase wet weather capacity.

Phosphorus Treatment

The Ansonia WPCF discharges to the Naugatuck River, while the existing Derby WPCF discharges to the Housatonic River. Therefore, this alternative would involve an increase in costs associated with advanced treatment for Derby wastewater.

Collection System

Naugatuck

Beacon
Falls

Seymour

Ansonia

Derby

Collection System Rehabilitation

The base case assumes maintenance for each collection system necessary to improve the system to a basic level of reliability and performance consistent with the I/I reduction assumptions in the flows and loads analysis.

Lift Stations & Conveyance

Naugatuck

Beacon
Falls

Seymour

Ansonia

Derby

Capacity

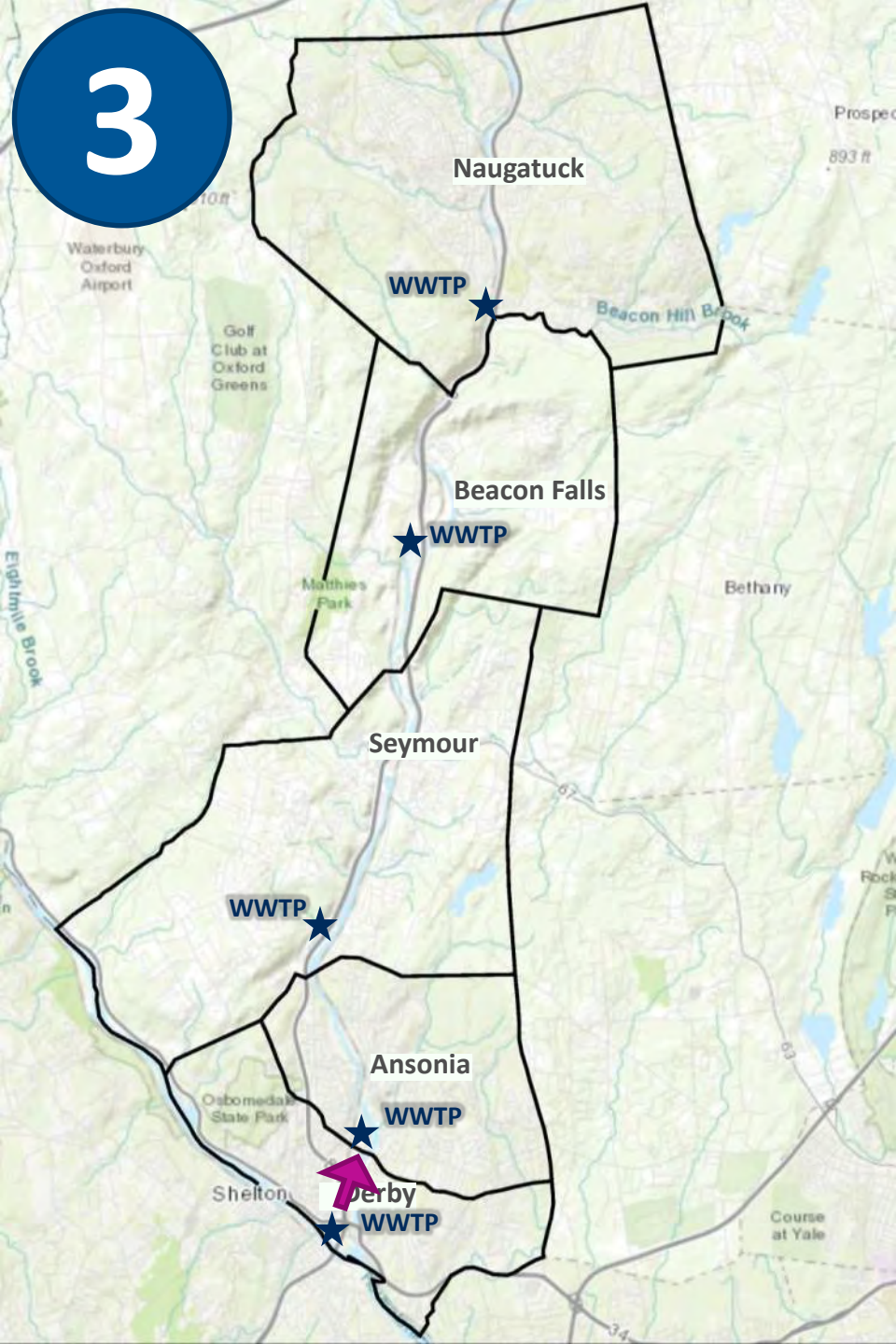
The new pumping station in Derby would be sized, along with any storage at that location, to handle anticipated peak hydraulic flow from the Derby wastewater collection system. This alternative might also include a headworks facility for grit and screenings removal at the new Derby pumping station..

Conveyance Corridors

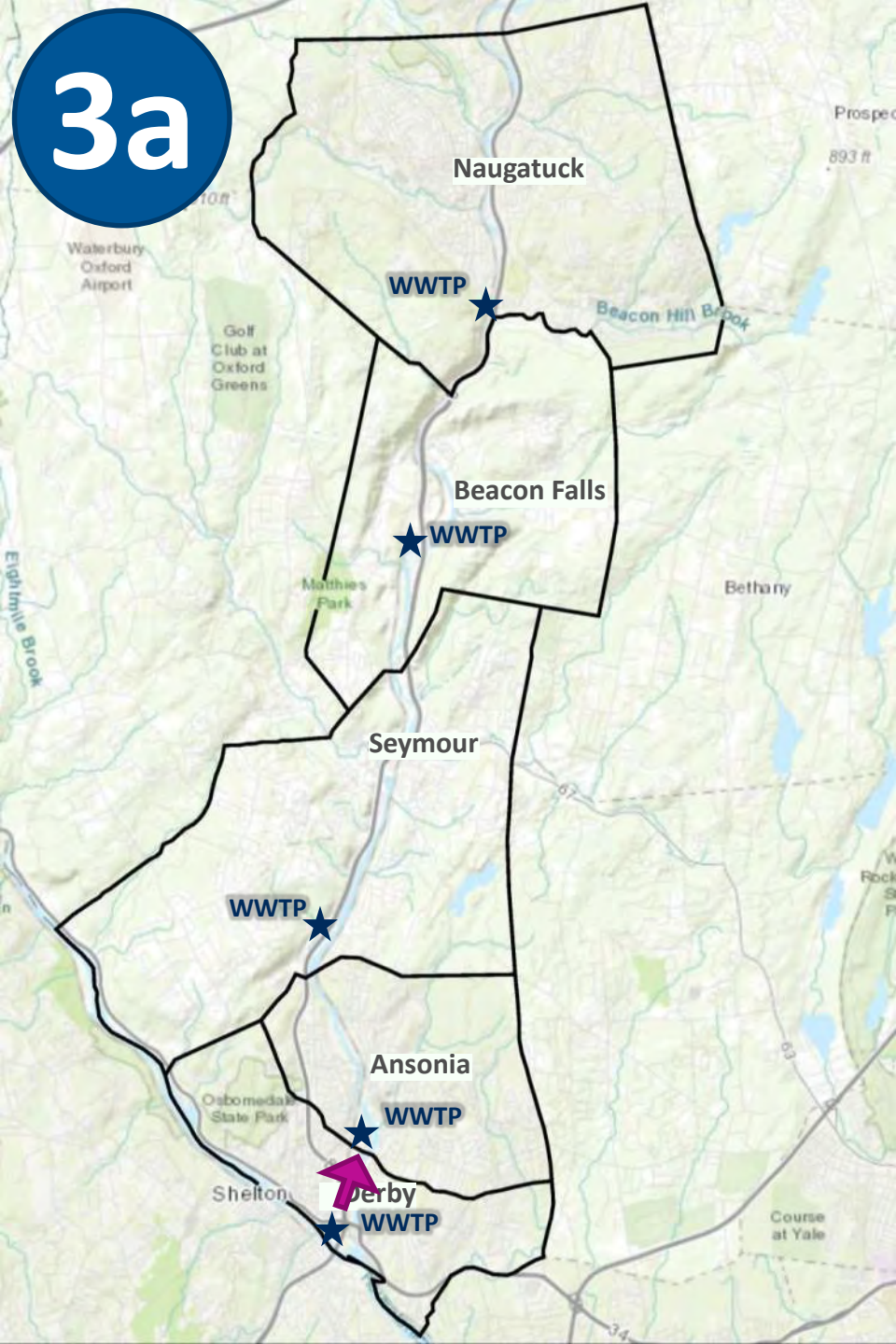
There is a nominal 5 ft elevation difference between the Ansonia and Derby WPCFs, and a distance between 8,000 to 9,000 ft.

Two routes were identified in preliminary review. The primary alternatives would be to follow the Naugatuck River from plant to plant, which would involve crossing wetlands, or

following existing town roads, which would include a high point of approximately 89 ft and require a pump station, but would not have as many permitting constraints.



3a



Derby to Ansonia, I/I Reduction

Treatment Plants

Naugatuck

Beacon Falls

Seymour

Ansonia

Derby

Under this alternative, the Derby WPCF would be decommissioned and replaced with a pumping station. All Derby wastewater would be pumped to the Ansonia WPCF.

Wet Weather Capacity

The Ansonia WPCF currently has sufficient capacity to accept the annual average flows from Derby. However, due to high wet weather

flows in both Ansonia and Derby, the Ansonia WPCF does not have the capacity to handle the combined peak flows. Therefore, this alternative would require upgrading the WPCF to increase wet weather capacity.

With aggressive I/I reduction in Derby and Ansonia, as well as storage and/or high rate treatment capacity, the required increase in

capacity could be reduced.

Phosphorus Treatment

The Ansonia WPCF discharges to the Naugatuck River, while the existing Derby WPCF discharges to the Housatonic River. Therefore, this alternative would involve an increase in costs associated with advanced treatment for Derby wastewater.

Collection System

Naugatuck

Beacon Falls

Seymour

Ansonia

Derby

Collection System Rehabilitation

Due to excessive peak flows in both Derby and Ansonia, aggressive I/I reduction would be required in both systems. I/I reduction could be mitigated with the use of storage. The extent of I/I reduction would need to be balanced with marginal cost of treatment and marginal cost of storage.

Lift Stations & Conveyance

Naugatuck

Beacon Falls

Seymour

Ansonia

Derby

Capacity

The new pumping station in Derby would be sized, along with any storage at that location, to handle anticipated peak hydraulic flow from the Derby wastewater collection system. With aggressive I/I reduction, this peak flow would be reduced. This alternative might also include a headworks facility for grit and screenings removal at the new Derby pumping station.

Conveyance Corridors

There is a nominal 5 ft elevation difference between the Ansonia and Derby WPCFs, and a distance between 8,000 to 9,000 ft.

Two routes were identified in preliminary review. The primary alternatives would be to follow the Naugatuck River from plant to plant, which would involve crossing wetlands, or

following existing town roads, which would include a high point of approximately 89 ft and require larger pumps, but would not have as many permitting constraints.

4

Derby to Ansonia Effluent Pumped to Housatonic

Treatment Plants

Naugatuck

Beacon
Falls

Seymour

Ansonia

Derby

Under this alternative, the Derby WPCF would be decommissioned and replaced with a pumping station. All Derby wastewater would be pumped to the Ansonia WPCF.

Wet Weather Capacity

The Ansonia WPCF currently has sufficient capacity to accept the annual average flows from Derby. However, due to high wet weather flows in both Ansonia and Derby, the Ansonia WPCF does not have the capacity to handle the combined peak flows. Therefore, this alternative would require upgrading the WPCF and/or providing storage or high rate

treatment to increase wet weather capacity.

Phosphorus Treatment

This alternative would include a new effluent discharge line back to Derby, for discharge to the Housatonic River, which could eliminate the need for phosphorus removal.

Collection System

Naugatuck

Beacon
Falls

Seymour

Ansonia

Derby

Collection System Rehabilitation

The base case assumes maintenance for each collection system necessary to improve the system to a basic level of reliability and performance consistent with the I/I reduction assumptions in the flows and loads analysis.

Lift Stations & Conveyance

Naugatuck

Beacon
Falls

Seymour

Ansonia

Derby

Capacity

The new pumping station in Derby would be sized, along with any storage at that location, to handle anticipated peak hydraulic flow from the Derby wastewater collection system. This alternative might also include a headworks facility for grit and screenings removal at the new Derby pumping station..

Conveyance Corridors

With a nominal 5 ft elevation difference between the Ansonia and Derby WPCFs, and a distance between 8,000 to 9,000 ft, a lift station would be required.

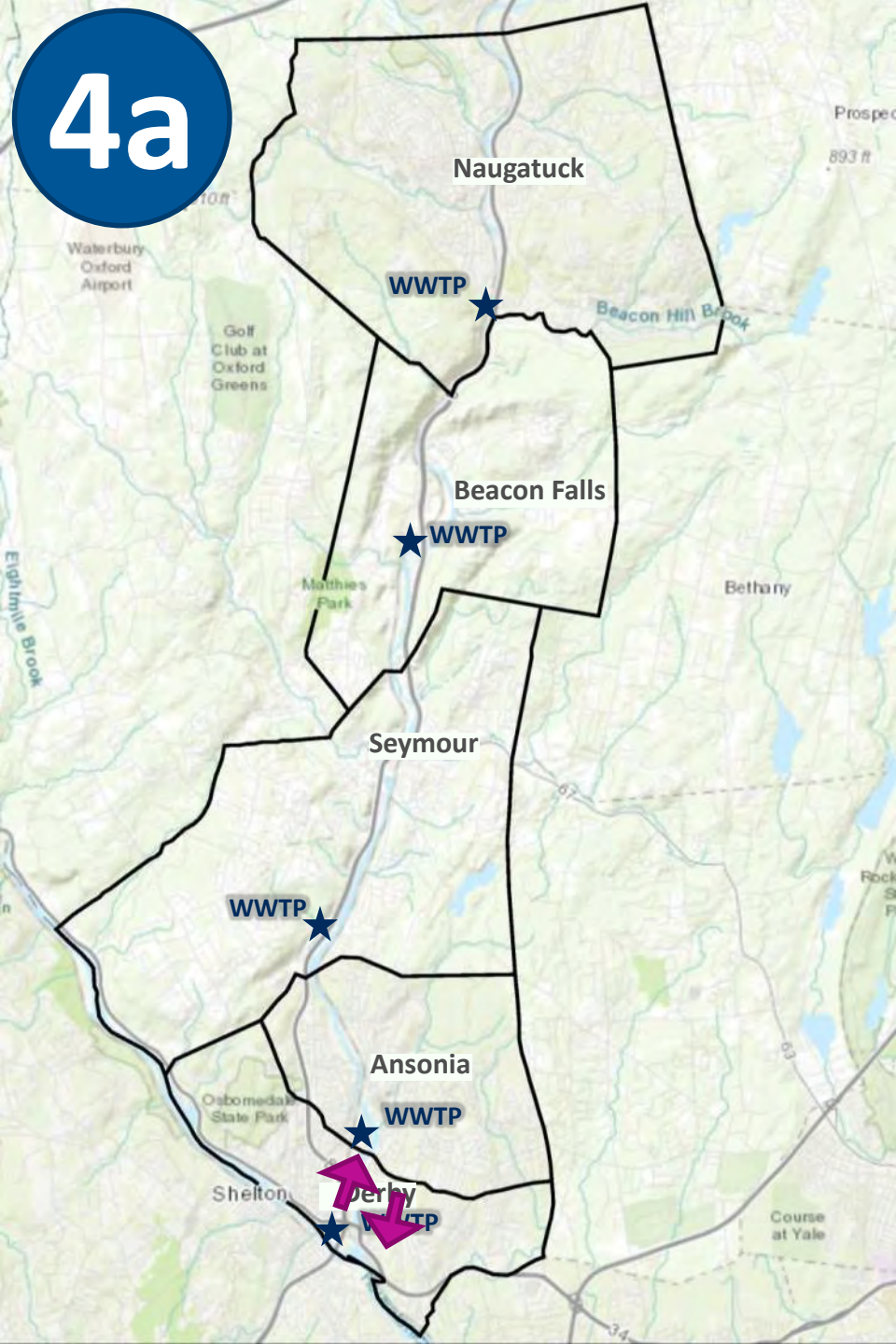
Two routes were identified in preliminary review. The primary alternatives would be to follow the Naugatuck River from plant to plant,

which would involve crossing wetlands, or following existing town roads, which would include a high point of approximately 89 ft and require larger pumps, but would not have as many permitting constraints.

Discharge to Housatonic

The same pipe route would be used for effluent discharge to the Housatonic River.

4a



Derby to Ansonia, I/I Reduction Effluent Pumped to Housatonic

Treatment Plants

Naugatuck

Beacon Falls

Seymour

Ansonia

~~Derby~~

Under this alternative, the Derby WPCF would be decommissioned and replaced with a pumping station. All Derby wastewater would be pumped to the Ansonia WPCF.

Depending on the extent of I/I removal, additional measures may be required to accommodate peak flows, such as increased wet weather treatment capacity at the Ansonia WPCF or storage facilities.

the Housatonic River, which could eliminate the need for phosphorus removal.

Wet Weather Capacity

The Ansonia WPCF currently has sufficient capacity to accept the annual average flows from Derby.

Phosphorus Treatment

This alternative would include a new effluent discharge line back to Derby, for discharge to

Collection System

Naugatuck

Beacon Falls

Seymour

Ansonia

Derby

Collection System Rehabilitation

Due to excessive peak flows in both Derby and Ansonia, aggressive I/I reduction would be required in both systems. I/I reduction could be mitigated with the use of storage. The extent of I/I reduction would need to be balanced with marginal cost of treatment and marginal cost of storage.

Lift Stations & Conveyance

Naugatuck

Beacon Falls

Seymour

Ansonia

Derby

Capacity

The new pumping station in Derby would be sized, along with any storage at that location, to handle anticipated peak hydraulic flow from the Derby wastewater collection system. With aggressive I/I reduction, this peak flow would be reduced. This alternative might also include a headworks facility for grit and screenings removal at the new Derby pumping station.

Conveyance Corridors

With a nominal 5 ft elevation difference between the Ansonia and Derby WPCFs, and a distance between 8,000 to 9,000 ft, a lift station would be required.

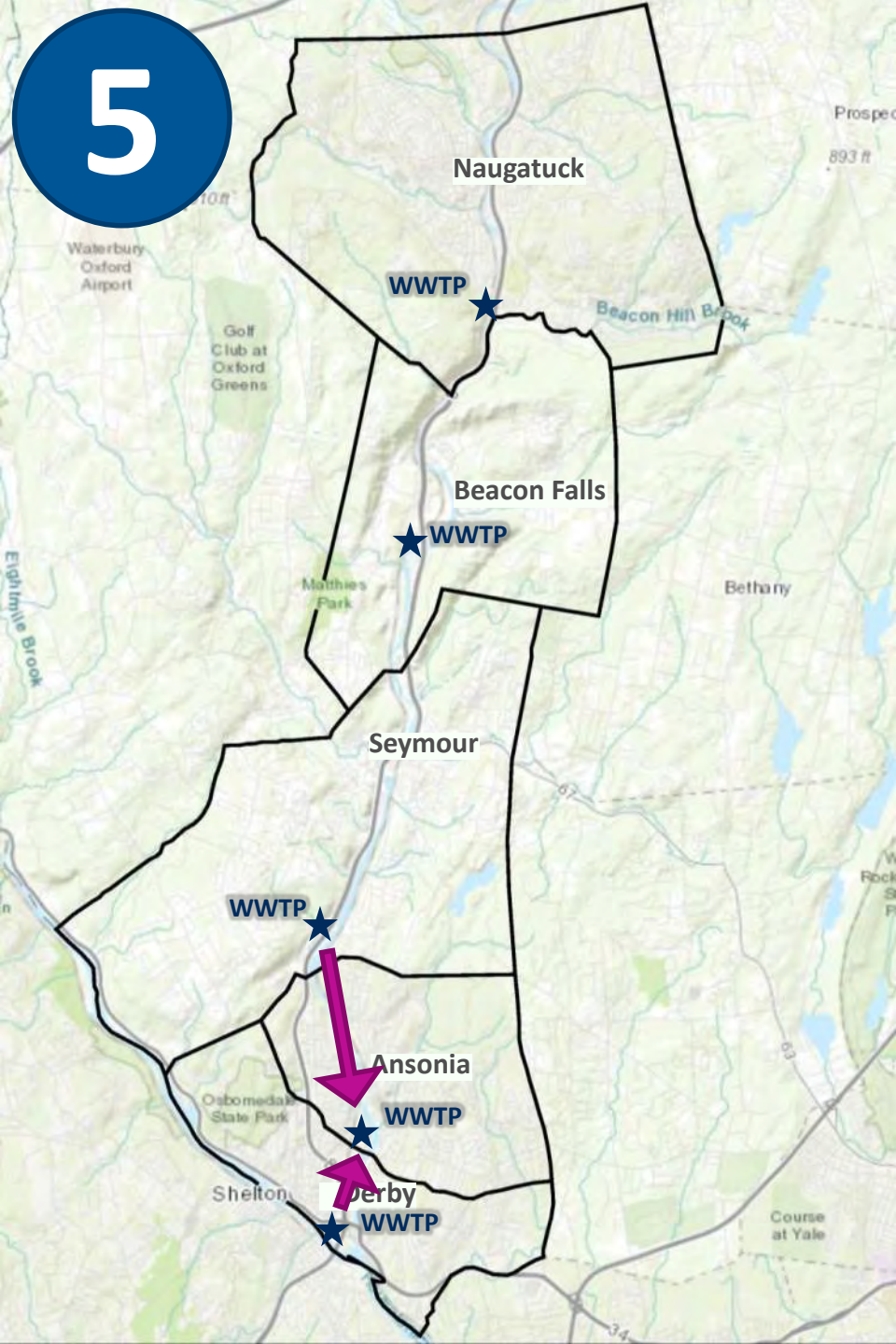
Two routes were identified in preliminary review. The primary alternatives would be to follow the Naugatuck River from plant to plant, which would involve crossing wetlands, or

following existing town roads, which would include a high point of approximately 89 ft and require a pump station, but would not have as many permitting constraints.

Discharge to Housatonic

The same pipe route would be used for effluent discharge pipeline to the Housatonic River.

5



Derby and Seymour to Ansonia

Treatment Plants

Naugatuck

Beacon Falls

~~Seymour~~

Ansonia

~~Derby~~

Under this alternative, the Derby WPCF and Seymour WPCF would be decommissioned and replaced with pumping to the Ansonia WPCF.

Treatment Capacity

The Ansonia WPCF would have to be upgraded from its current capacity of 3.5 MGD to a new design capacity of 4.9 MGD (annual average) to handle flows from all three towns through

2040. Furthermore, the treatment plant upgrade would need to accommodate significantly higher wet weather flows (10.4 MGD max month, 19.8 MGD peak day), likely also in combination with one or more wet weather storage facilities.

Phosphorus Treatment

The Ansonia WPCF discharges to the Naugatuck

River, while the existing Derby WPCF discharges to the Housatonic River. Therefore, this alternative would involve an increase in costs associated with advanced treatment for Derby wastewater.

Collection System

Naugatuck

Beacon Falls

Seymour

Ansonia

Derby

Collection System Rehabilitation

The base case assumes maintenance for each collection system necessary to improve the system to a basic level of reliability and performance consistent with the I/I reduction assumptions in the flows and loads analysis.

Lift Stations & Conveyance

Naugatuck

Beacon Falls

Seymour

Ansonia

Derby

Capacity

The new pumping stations in Seymour and Derby would be sized, along with any storage at that location, to handle anticipated peak hydraulic flow from their respective wastewater collection systems. This alternative might also include headworks facilities for grit and screenings removal at the new Seymour and Derby pumping stations.

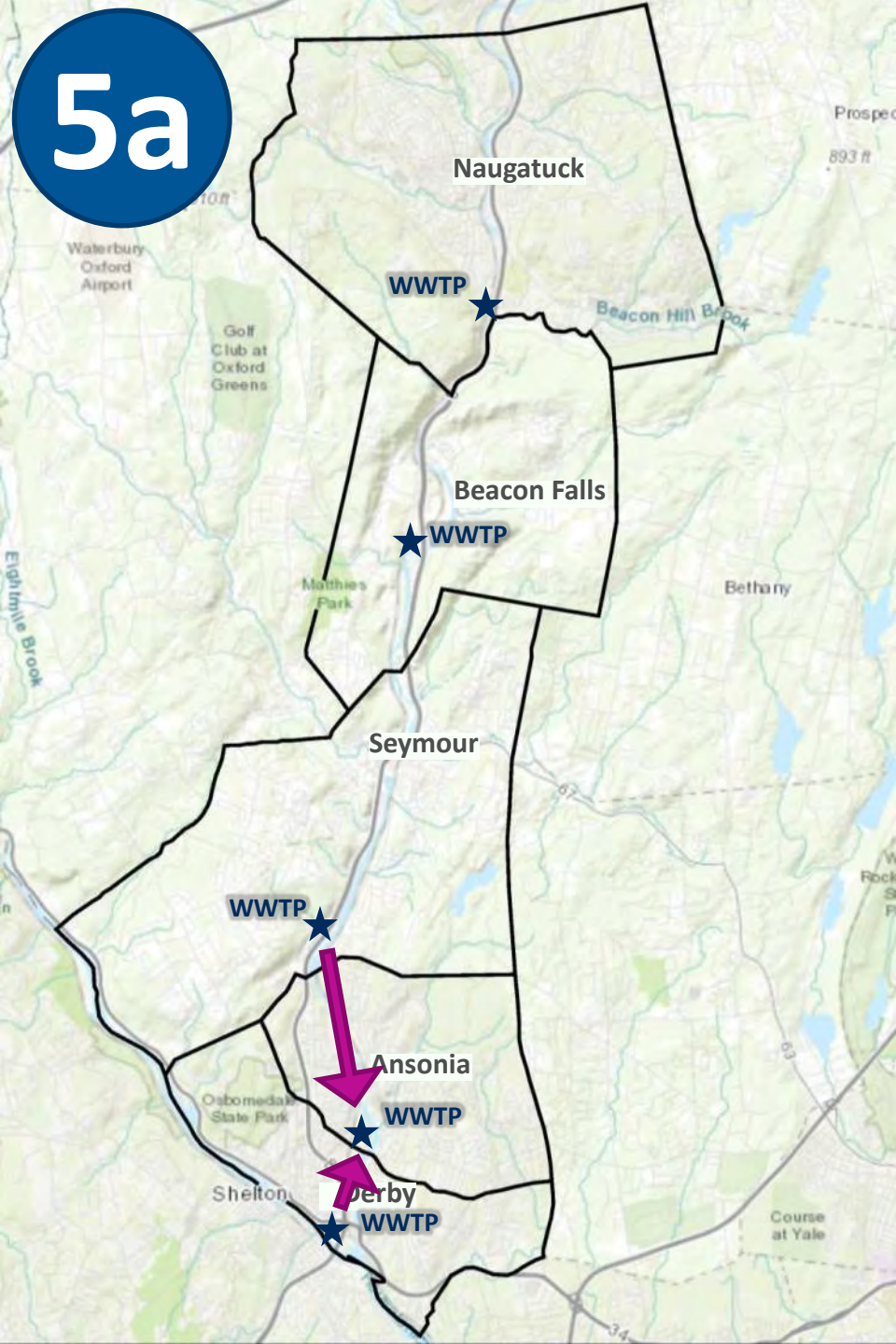
Conveyance Corridors

Two separate pipes would be required: Seymour to Ansonia and Derby to Ansonia.

Seymour to Ansonia could be routed along town roads without major topographic obstructions, with a maximum elevation of approximately 130 ft and a length of nearly 14,000 ft.

Derby to Ansonia has multiple potential routings of approximately 8,000 to 9,000 ft with a maximum elevation up to nearly 90 ft.

5a



Derby and Seymour to Ansonia I/I Reduction

Treatment Plants

Naugatuck

Beacon Falls

~~Seymour~~

Ansonia

~~Derby~~

Under this alternative, the Derby WPCF and Seymour WPCF would be decommissioned and replaced with pumping to the Ansonia WPCF. This alternative would include aggressive I/I reduction and storage to reduce peak flows.

Treatment Capacity

The Ansonia WPCF would have to be upgraded from its current capacity of 3.5 MGD to a new

design capacity of 4.9 MGD (annual average) to handle flows from all three towns through 2040. Furthermore, the treatment plant upgrade would need to accommodate significantly higher wet weather flows (depending on extent of I/I reduction, up to 10.4 MGD max month, 19.8 MGD peak day), possibly in combination with one or more wet weather storage facilities.

Phosphorus Treatment

The Ansonia WPCF discharges to the Naugatuck River, while the existing Derby WPCF discharges to the Housatonic River. Therefore, this alternative would involve an increase in costs associated with advanced treatment for Derby wastewater.

Collection System

Naugatuck

Beacon Falls

Seymour

Ansonia

Derby

Collection System Rehabilitation

Due to excessive peak flows in Derby, Seymour, and Ansonia, aggressive I/I reduction would be required in all three systems. I/I reduction could be mitigated with the use of storage. The extent of I/I reduction would need to be balanced with marginal cost of treatment and marginal cost of storage.

Lift Stations & Conveyance

Naugatuck

Beacon Falls

Seymour

Ansonia

Derby

Capacity

The new pumping stations in Seymour and Derby would be sized, along with any storage at that location, to handle anticipated peak hydraulic flow from their respective wastewater collection systems. This alternative might also include headworks facilities for grit and screenings removal at the new Seymour and Derby pumping stations.

With aggressive I/I removal, the necessary pumping capacity would be reduced.

Conveyance Corridors

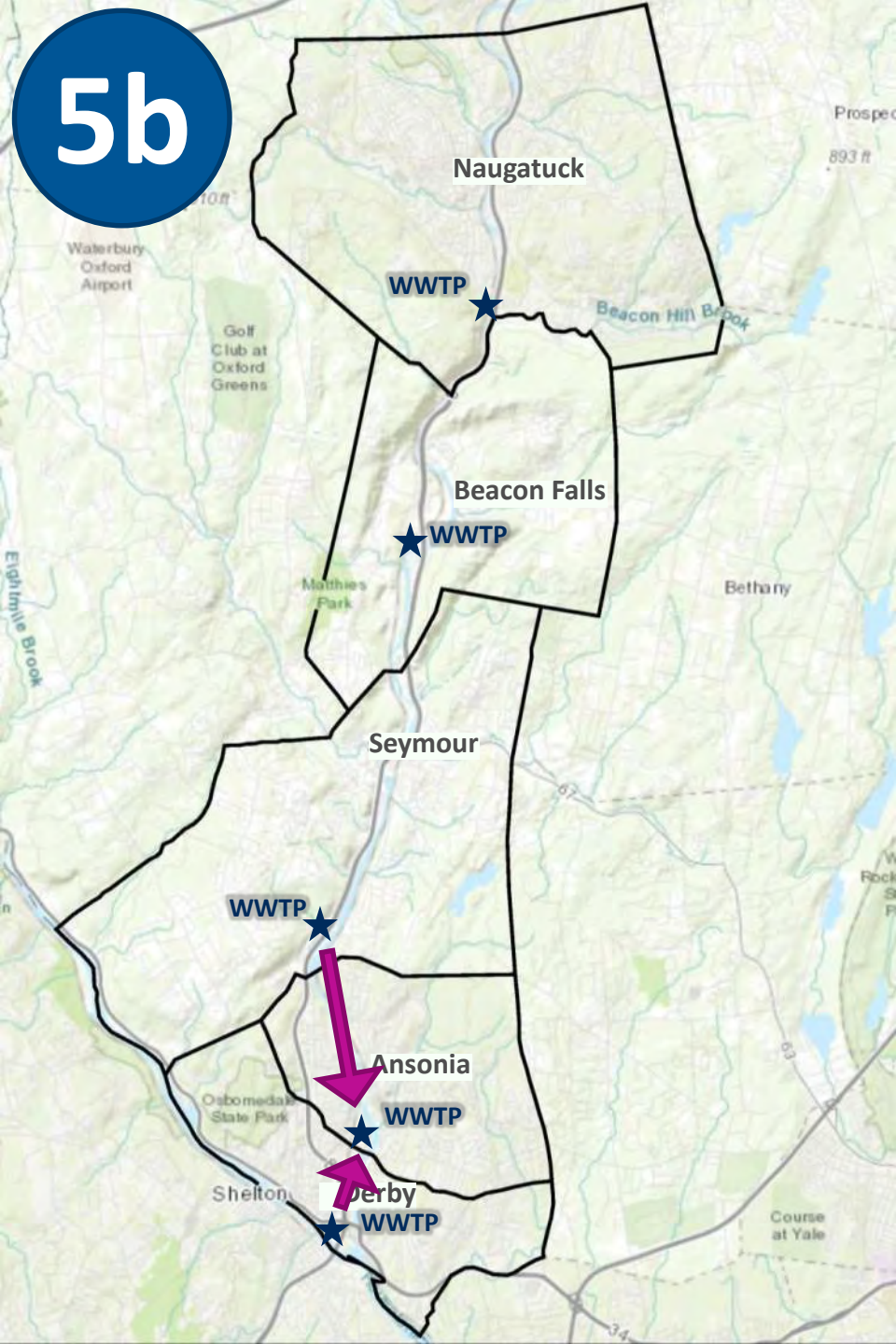
Two separate pipes would be required: Seymour to Ansonia and Derby to Ansonia.

Seymour to Ansonia could be routed along

town roads without major topographic obstructions, with a maximum elevation of approximately 130 ft and a length of nearly 14,000 ft.

Derby to Ansonia has multiple potential routings of approximately 8,000 to 9,000 ft with a maximum elevation up to nearly 90 ft.

5b



Derby and Seymour to Ansonia, Effluent to Housatonic

Treatment Plants

Naugatuck

Beacon Falls

~~Seymour~~

Ansonia

~~Derby~~

Under this alternative, the Derby WPCF and Seymour WPCF would be decommissioned and replaced with pumping to the Ansonia WPCF.

Treatment Capacity

The Ansonia WPCF would have to be upgraded from its current capacity of 3.5 MGD to a new design capacity of 4.9 MGD (annual average) to handle flows from all three towns through

2040. Furthermore, the treatment plant upgrade would need to accommodate significantly higher wet weather flows (10.4 MGD max month, 19.8 MGD peak day), likely also in combination with one or more wet weather storage facilities.

Phosphorus Treatment

This alternative would include a new effluent

discharge line back to Derby, for discharge to the Housatonic River, which could eliminate the need for phosphorus removal.

Collection System

Naugatuck

Beacon Falls

Seymour

Ansonia

Derby

Collection System Rehabilitation

The base case assumes maintenance for each collection system necessary to improve the system to a basic level of reliability and performance consistent with the I/I reduction assumptions in the flows and loads analysis.

Lift Stations & Conveyance

Naugatuck

Beacon Falls

Seymour

Ansonia

Derby

Capacity

The new pumping stations in Seymour and Derby would be sized, along with any storage at that location, to handle anticipated peak hydraulic flow from their respective wastewater collection systems. This alternative might also include headworks facilities for grit and screenings removal at the new Seymour and Derby pumping stations.

With aggressive I/I removal, the necessary pumping capacity would be reduced.

Conveyance Corridors

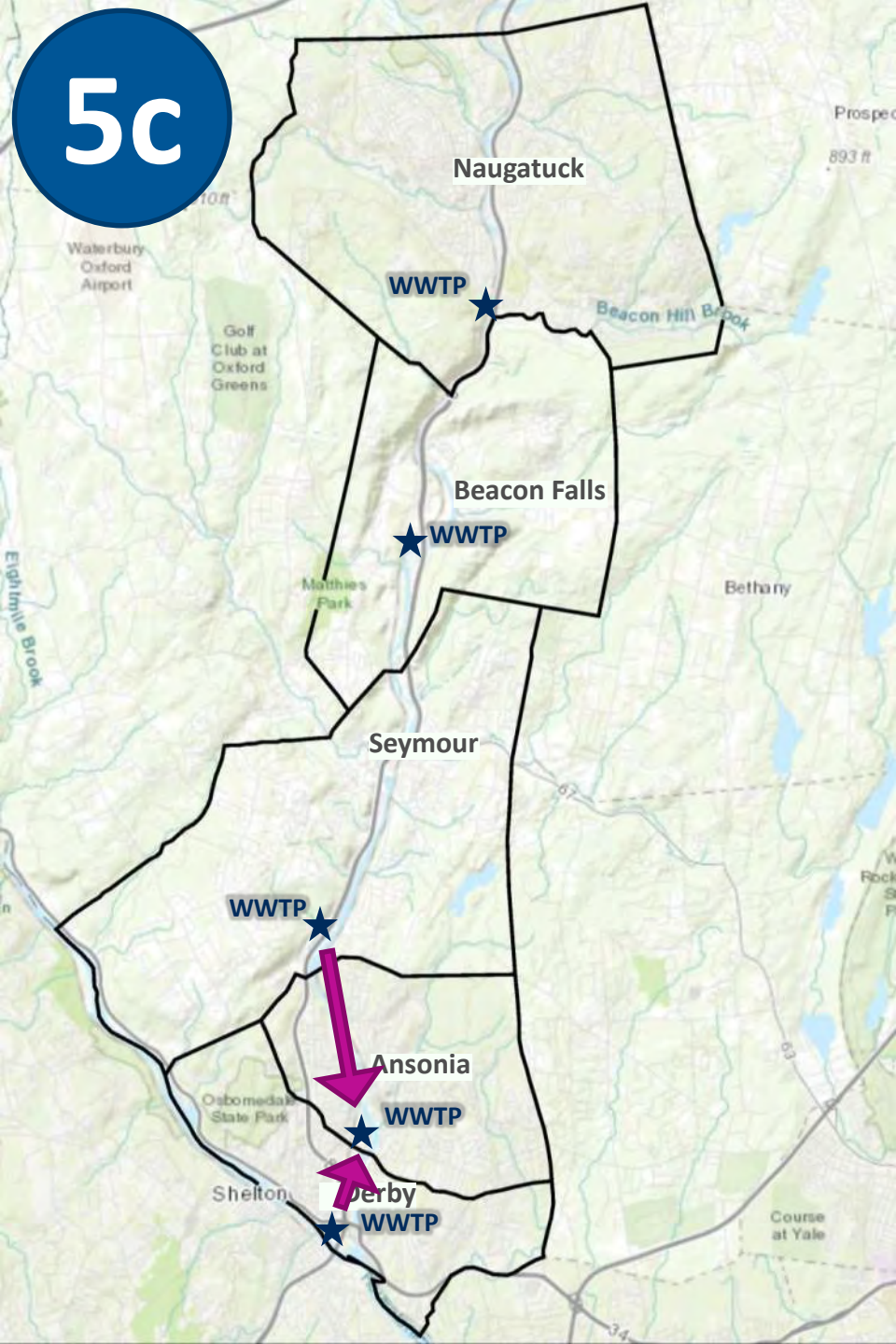
Two separate pipes would be required: Seymour to Ansonia and Derby to Ansonia. Seymour to Ansonia could be routed along town roads without major topographic obstructions, with a maximum elevation of approximately 130 ft and a length of nearly

14,000 ft. Derby to Ansonia has multiple potential routings of approximately 8,000 to 9,000 ft with a maximum elevation up to nearly 90 ft.

Discharge to Housatonic

The same pipe route would be used for effluent discharge to the Housatonic River.

5c



Derby and Seymour to Ansonia I/I Reduction, Effluent to Housatonic

Treatment Plants

Naugatuck

Beacon
Falls~~Seymour~~

Ansonia

~~Derby~~

Under this alternative, the Derby WPCF and Seymour WPCF would be decommissioned and replaced with pumping to the Ansonia WPCF. This alternative would include aggressive I/I reduction and storage to reduce peak flows.

Treatment Capacity

The Ansonia WPCF would have to be upgraded from its current capacity of 3.5 MGD to a new

design capacity of 4.9 MGD (annual average) to handle flows from all three towns through 2040. Furthermore, the treatment plant upgrade would need to accommodate significantly higher wet weather flows (depending on the extent of I/I removal, up to 10.4 MGD max month, 19.8 MGD peak day), possibly also in combination with one or more wet weather storage facilities.

Phosphorus Treatment

This alternative would include a new effluent discharge line back to Derby, for discharge to the Housatonic River, which could eliminate the need for phosphorus removal.

Collection System

Naugatuck

Beacon
Falls

Seymour

Ansonia

Derby

Collection System Rehabilitation

Due to excessive peak flows in Derby, Seymour, and Ansonia, aggressive I/I reduction would be required in all three systems. I/I reduction could be mitigated with the use of storage. The extent of I/I reduction would need to be balanced with marginal cost of treatment and marginal cost of storage.

Lift Stations & Conveyance

Naugatuck

Beacon
Falls

Seymour

Ansonia

Derby

Capacity

The new pumping stations in Seymour and Derby would be sized, along with any storage at that location, to handle anticipated peak hydraulic flow from their respective wastewater collection systems. This alternative might also include headworks facilities for grit and screenings removal at the new Seymour and Derby pumping stations.

With aggressive I/I removal, the necessary pumping capacity would be reduced.

Conveyance Corridors

Two separate pipes would be required: Seymour to Ansonia and Derby to Ansonia. Seymour to Ansonia could be routed along town roads without major topographic obstructions, with a maximum elevation of approximately 130 ft and a length of nearly

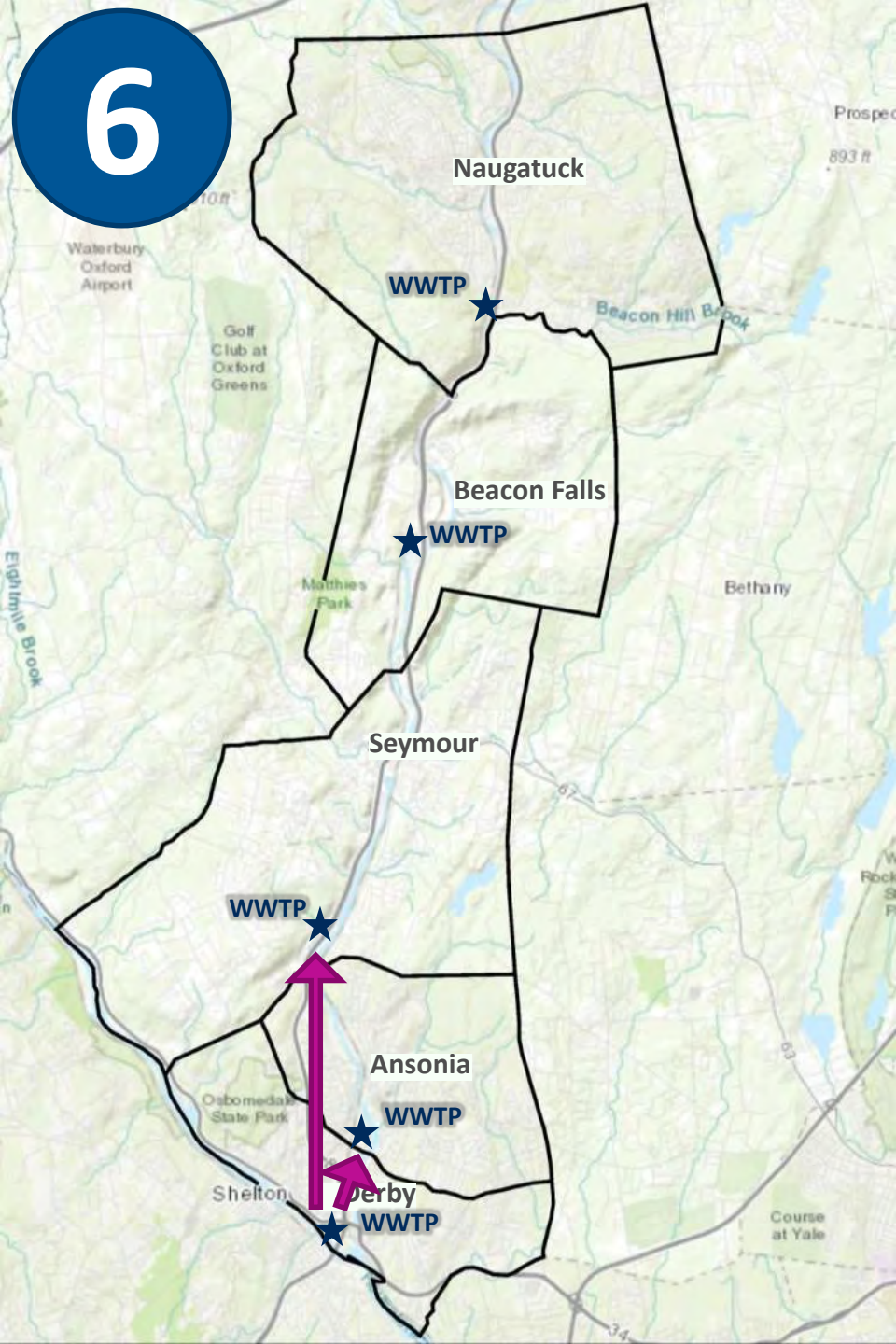
14,000 ft.

Derby to Ansonia has multiple potential routings of approximately 8,000 to 9,000 ft with a maximum elevation up to nearly 90 ft.

Discharge to Housatonic

The same pipe route would be used for effluent discharge to the Housatonic River

Derby to Seymour and Ansonia



Treatment Plants

Naugatuck

Beacon
Falls

Seymour

Ansonia

~~Derby~~

The Derby WPCF would be decommissioned and replaced with a pumping station. A portion of Derby's wastewater would be pumped to the Ansonia WPCF, with the remainder pumped to the Seymour WPCF.

Treatment Capacity

If approximately 40-60% of the Derby flow is pumped to the Seymour WPCF, with the

remainder pumped to the Ansonia WPCF, those two plants would have adequate capacity to handle the additional flows from Derby under average annual and max month conditions. However, the capacities of both plants would be exceeded during peak hydraulic flow conditions. Therefore, both treatment plants would need to be modified or upgraded to process higher wet weather flows,

possibly in combination with storage.

Phosphorus Treatment

Ansonia and Seymour discharge to the Naugatuck River, while Derby discharges to the Housatonic River. Therefore, this alternative would involve an increase in costs for advanced treatment for Derby wastewater.

Collection System

Naugatuck

Beacon
Falls

Seymour

Ansonia

Derby

Collection System Rehabilitation

The base case assumes maintenance for each collection system necessary to improve the system to a basic level of reliability and performance consistent with the I/I reduction assumptions in the flows and loads analysis.

Lift Stations & Conveyance

Naugatuck

Beacon
Falls

Seymour

Ansonia

Derby

Capacity

The new pumping station(s) in Derby would be sized, along with any storage at that location, to handle anticipated peak hydraulic flow from the Derby wastewater collection systems. This alternative might also include headworks facilities for grit and screenings removal at the new Derby pumping station(s).

Conveyance Corridors

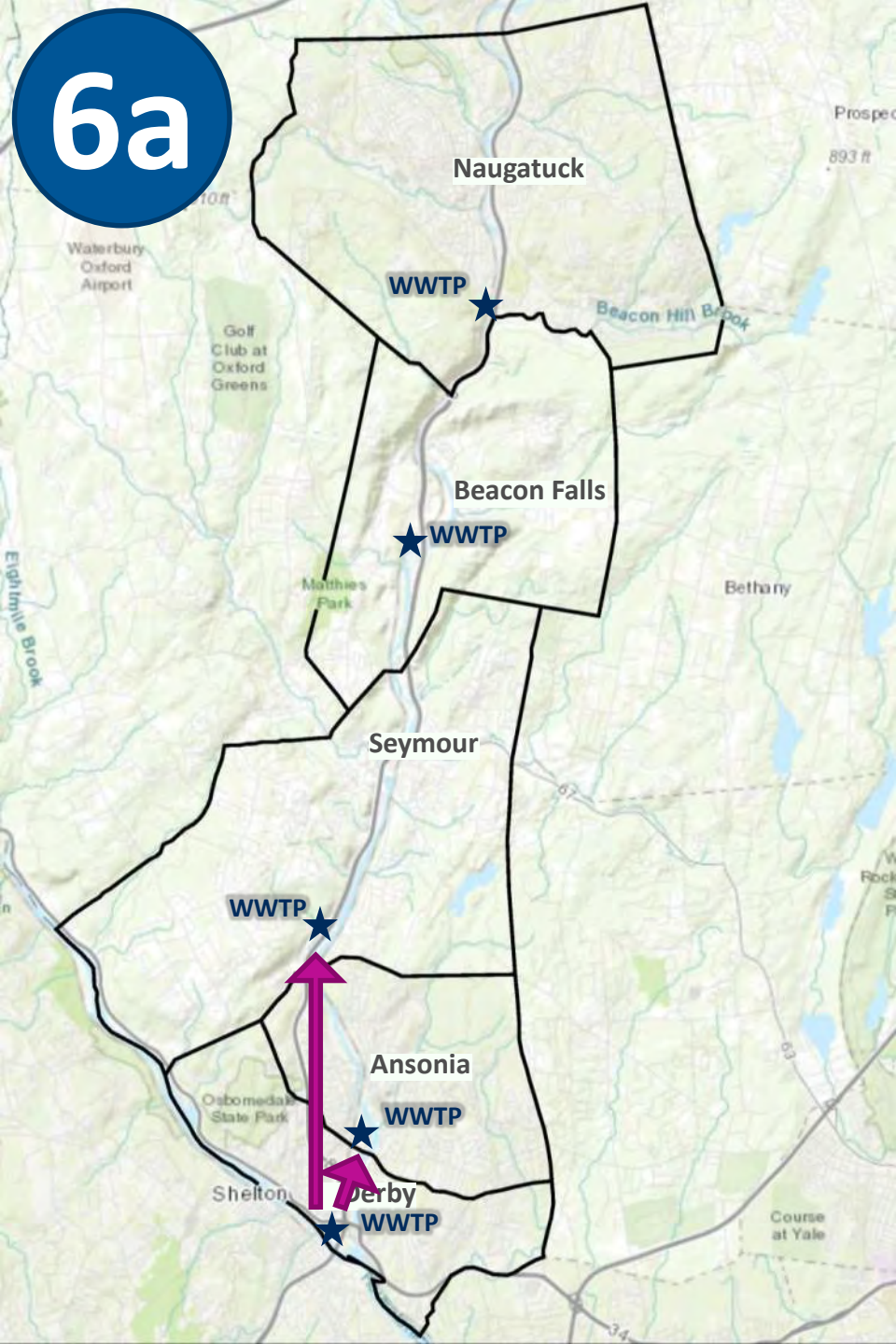
It would be possible to convey this flow in two stages: Derby to Ansonia and Ansonia to Seymour.

Seymour to Ansonia could be routed along town roads without major topographic obstructions, with a maximum elevation of approximately 130 ft and a length of nearly

14,000 ft.

Derby to Ansonia has multiple potential routings of approximately 8,000 to 9,000 ft with a maximum elevation up to nearly 90 ft.

6a



Derby to Seymour and Ansonia, I/I Reduction

Treatment Plants

Naugatuck

Beacon Falls

Seymour

Ansonia

Derby

The Derby WPCF would be decommissioned and replaced with a pumping station. A portion of Derby's wastewater would be pumped to the Ansonia WPCF, with the remainder pumped to the Seymour WPCF.

Treatment Capacity

If approximately 40-60% of the Derby flow is pumped to the Seymour WPCF, with the

remainder pumped to the Ansonia WPCF, those two plants would have adequate capacity to handle the additional flows from Derby under average annual and max month conditions. Depending on the extent of I/I removal, additional measures may be required to accommodate peak flows, such as increased wet weather treatment capacity at the WPCFs or storage facilities.

Phosphorus Treatment

Ansonia and Seymour discharge to the Naugatuck River, while Derby discharges to the Housatonic River. Therefore, this alternative would involve an increase in costs for advanced treatment for Derby wastewater.

Collection System

Naugatuck

Beacon Falls

Seymour

Ansonia

Derby

Collection System Rehabilitation

Due to excessive peak flows in Derby, Seymour, and Ansonia, aggressive I/I reduction would be required in all three systems. I/I reduction could be mitigated with the use of storage. The extent of I/I reduction would need to be balanced with marginal cost of treatment and marginal cost of storage.

Lift Stations & Conveyance

Naugatuck

Beacon Falls

Seymour

Ansonia

Derby

Capacity

The new pumping station(s) in Derby would be sized, along with any storage at that location, to handle anticipated peak hydraulic flow from the Derby wastewater collection systems. This alternative might also include headworks facilities for grit and screenings removal at the new Derby pumping station(s).

With aggressive I/I removal, the necessary pumping capacity would be reduced.

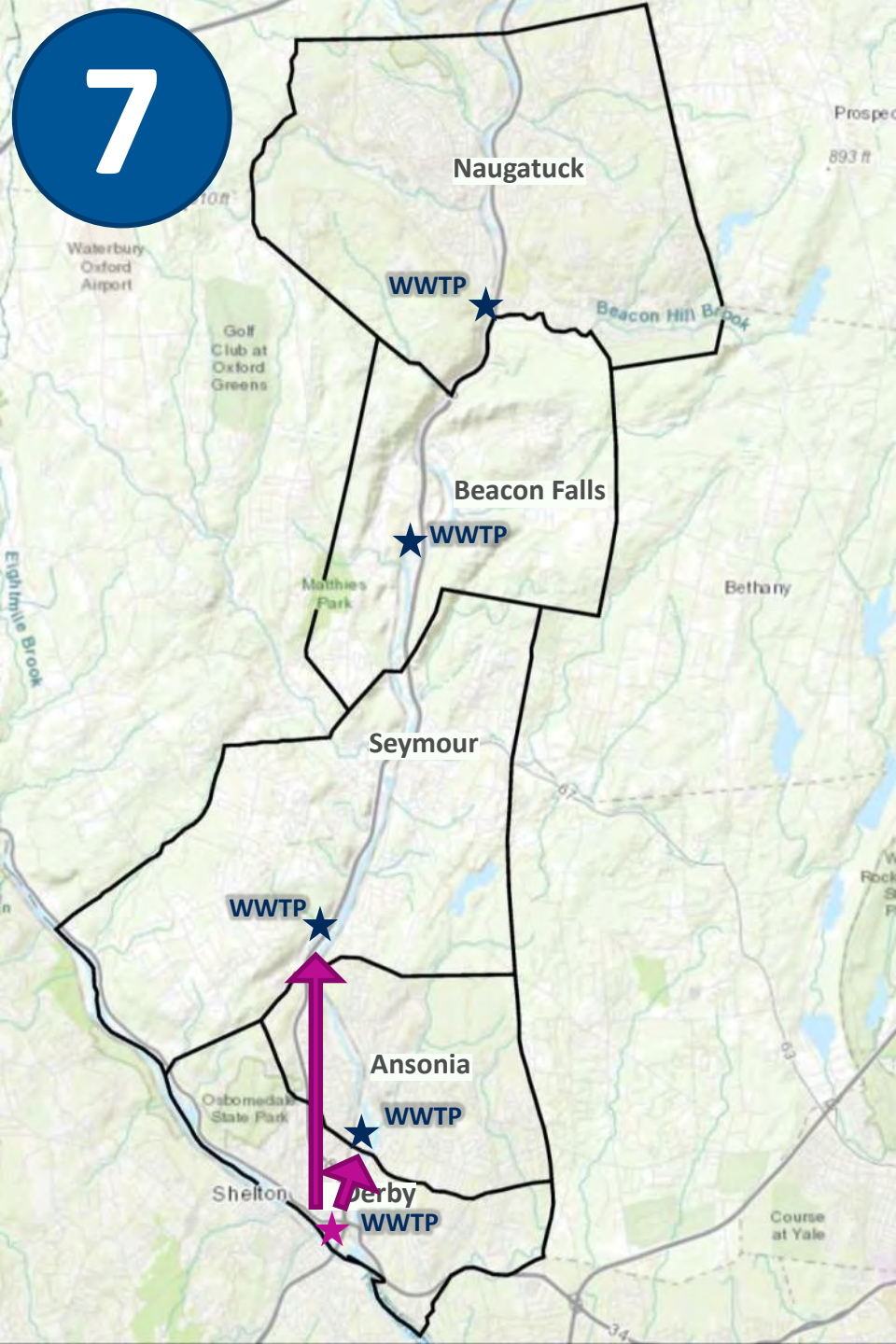
Conveyance Corridors

It would be possible to convey this flow in two stages: Derby to Ansonia and Ansonia to Seymour.

Seymour to Ansonia could be routed along

town roads without major topographic obstructions, with a maximum elevation of approximately 130 ft and a length of nearly 14,000 ft. Derby to Ansonia has multiple potential routings of approximately 8,000 to 9,000 ft with a maximum elevation up to nearly 90 ft.

7



Derby to Seymour and Ansonia and Derby

Treatment Plants

Naugatuck

Beacon
Falls

Seymour

Ansonia

Derby

There is no advantage to this alternative over the base case, if the Derby WPCF is not demolished. Therefore, this alternative is not considered further.

Collection System

Naugatuck

Beacon
Falls

Seymour

Ansonia

Derby

There is no advantage to this alternative over the base case, if the Derby WPCF is not demolished. Therefore, this alternative is not considered further.

Lift Stations & Conveyance

Naugatuck

Beacon
Falls

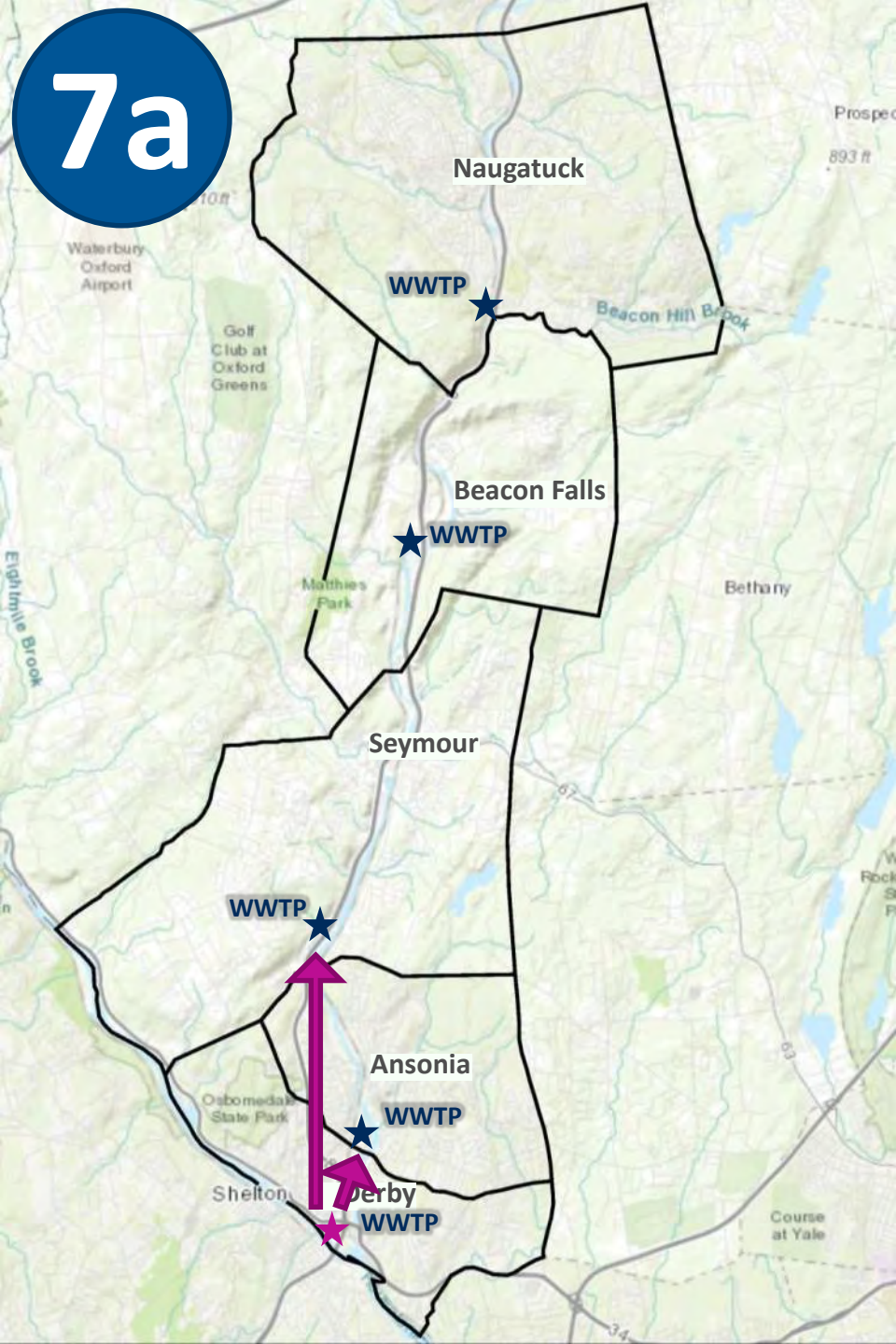
Seymour

Ansonia

Derby

There is no advantage to this alternative over the base case, if the Derby WPCF is not demolished. Therefore, this alternative is not considered further.

7a



Derby to Seymour and Ansonia and Derby, I/I Reduction

Treatment Plants

Naugatuck

Beacon
Falls

Seymour

Ansonia

Derby

There is no advantage to this alternative over the base case, if the Derby WPCF is not demolished. Therefore, this alternative is not considered further.

Collection System

Naugatuck

Beacon
Falls

Seymour

Ansonia

Derby

There is no advantage to this alternative over the base case, if the Derby WPCF is not demolished. Therefore, this alternative is not considered further.

Lift Stations & Conveyance

Naugatuck

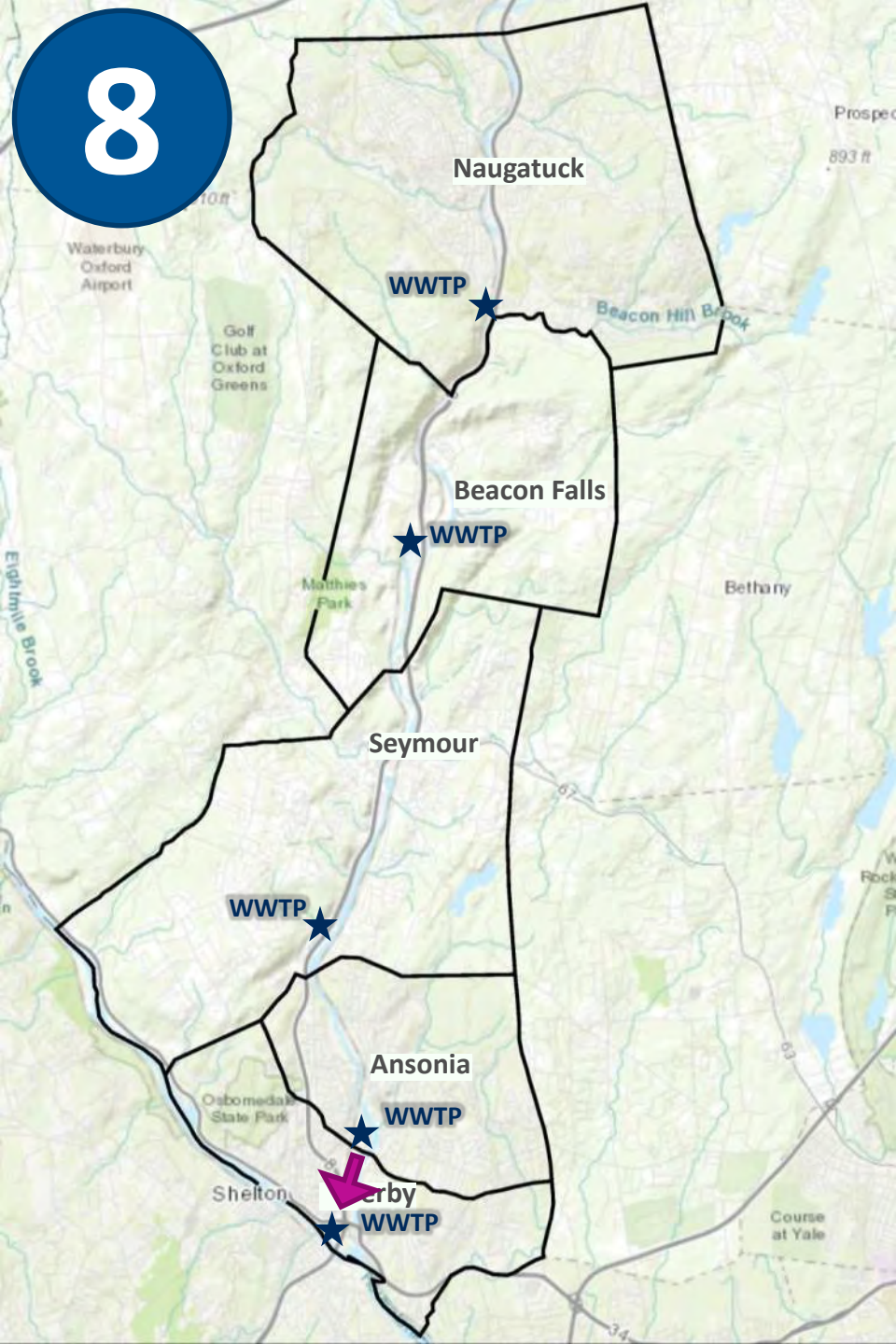
Beacon
Falls

Seymour

Ansonia

Derby

There is no advantage to this alternative over the base case, if the Derby WPCF is not demolished. Therefore, this alternative is not considered further.



Ansonia to Derby

Treatment Plants

Naugatuck

Beacon
Falls

Seymour

~~Ansonia~~

Derby

The Ansonia WPCF would be decommissioned and replaced with a pumping station. All Ansonia wastewater would be pumped to the Derby WPCF. Since the Ansonia WPCF was upgraded relatively recently (completed 2011), this alternative might not be fully implemented until closer to the end of the planning period.

Treatment Capacity

The Derby WPCF currently has sufficient capacity to accept the annual average flows from Ansonia. However, due to high wet weather flows, the Derby WPCF does not have the capacity to handle the combined wet weather flows. Therefore, this would require upgrading the Derby WPCF, possibly in combination with providing storage.

Phosphorus Treatment

Since the Derby WPCF discharges to the Housatonic River, where phosphorus removal is not required, this alternative would reduce some of the costs associated with advanced treatment for Ansonia wastewater.

Collection System

Naugatuck

Beacon
Falls

Seymour

Ansonia

Derby

Collection System Rehabilitation

The base case assumes maintenance for each collection system necessary to improve the system to a basic level of reliability and performance consistent with the I/I reduction assumptions in the flows and loads analysis.

Lift Stations & Conveyance

Naugatuck

Beacon
Falls

Seymour

Ansonia

Derby

Capacity

The new pumping station in Ansonia would be sized, along with any storage at that location, to handle anticipated peak hydraulic flow from the Ansonia wastewater collection system. This alternative might also include a headworks facility for grit and screenings removal at the new Ansonia pumping station.

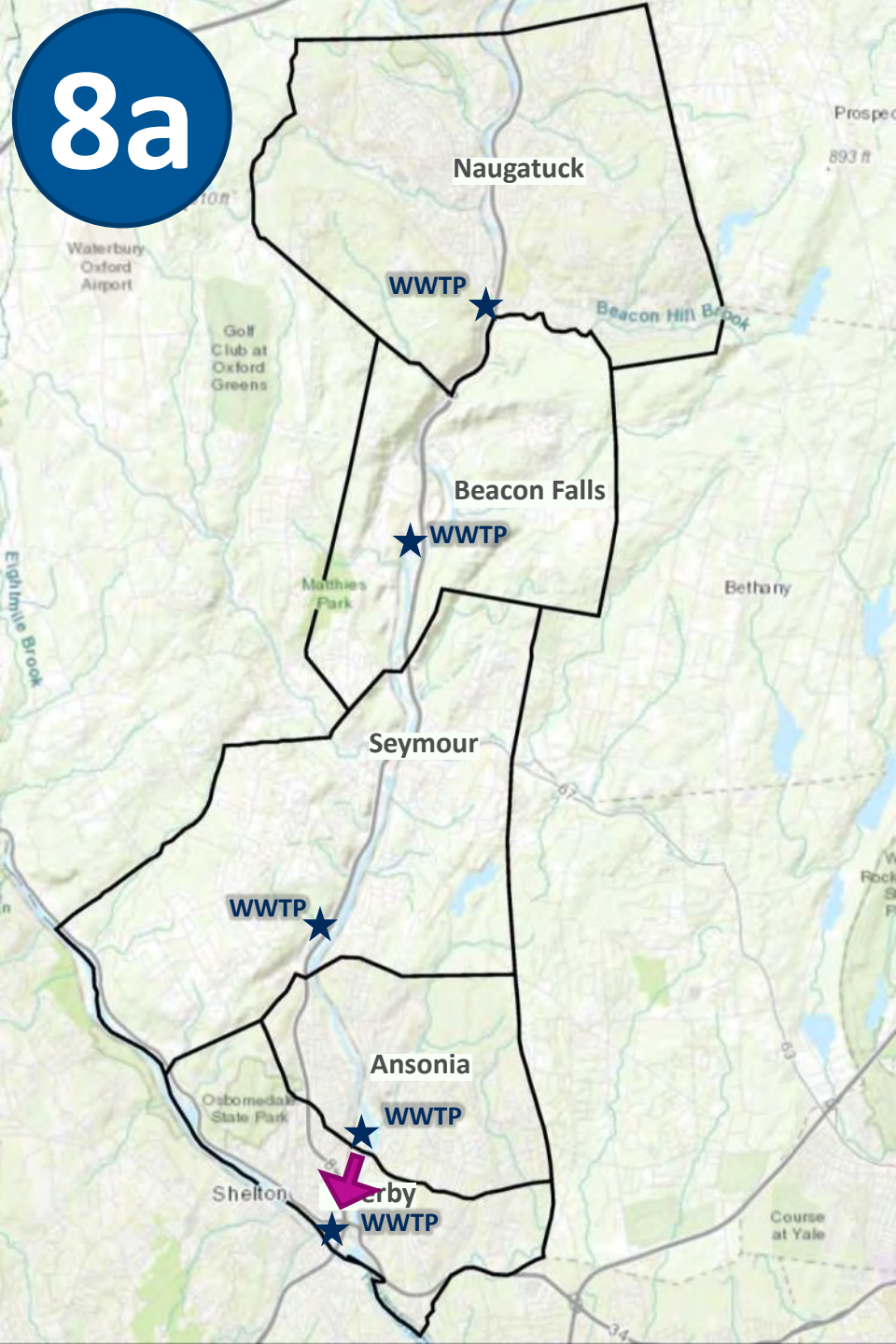
Conveyance Corridors

With a nominal 5 ft elevation difference between the Ansonia and Derby WPCFs, and a distance between 8,000 to 9,000 ft, a lift station would be required.

Two routes were identified in preliminary review. The primary alternatives would be to follow the Naugatuck River from plant to plant,

which would involve crossing wetlands, or following existing town roads, which would include a high point of approximately 89 ft and require larger pumps, but would not have as many permitting constraints.

8a



Ansonia to Derby, I/I Reduction

Treatment Plants

Naugatuck

Beacon
Falls

Seymour

~~Ansonia~~

Derby

The Ansonia WPCF would be decommissioned and replaced with a pumping station. All Ansonia wastewater would be pumped to the Derby WPCF. Since the Ansonia WPCF was upgraded relatively recently (completed 2011), this alternative might not be fully implemented until closer to the end of the planning period.

Treatment Capacity

The Derby WPCF currently has sufficient capacity to accept the annual average flows from Ansonia. Depending on the extent of I/I removal, additional measures may be required to accommodate peak flows, such as increased wet weather treatment capacity at the Derby WPCF or storage facilities.

Phosphorus Treatment

Since the Derby WPCF discharges to the Housatonic River, where phosphorus removal is not required, this alternative would reduce some of the costs associated with advanced treatment for Ansonia wastewater.

Collection System

Naugatuck

Beacon
Falls

Seymour

Ansonia

Derby

Collection System Rehabilitation

Due to excessive peak flows in Derby and Ansonia, aggressive I/I reduction would be required in both systems. I/I reduction could be mitigated with the use of storage. The extent of I/I reduction would need to be balanced with marginal cost of treatment and marginal cost of storage.

Lift Stations & Conveyance

Naugatuck

Beacon
Falls

Seymour

Ansonia

Derby

Capacity

The new pumping station in Ansonia would be sized, along with any storage at that location, to handle anticipated peak hydraulic flow from the Ansonia wastewater collection system. This alternative might also include a headworks facility for grit and screenings removal at the new Ansonia pumping station.

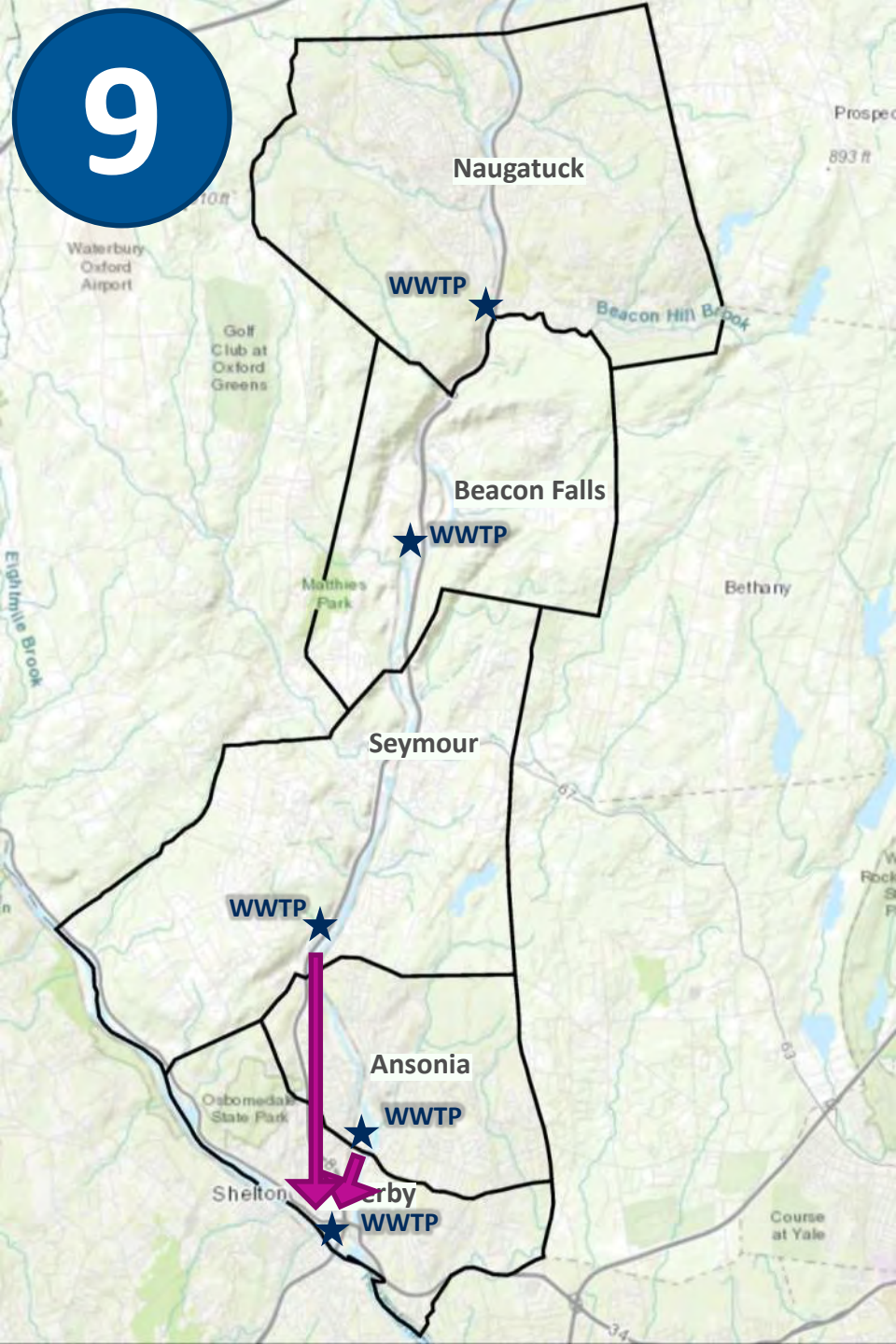
With aggressive I/I removal, the necessary pumping capacity would be reduced.

Conveyance Corridors

With a nominal 5 ft elevation difference between the Ansonia and Derby WPCFs, and a distance between 8,000 to 9,000 ft, a lift station would be required.

Two routes were identified in preliminary review. The primary alternatives would be to follow the Naugatuck River from plant to plant, which would involve crossing wetlands, or following existing town roads, which would include a high point of approximately 89 ft and require larger pumps, but would not have as many permitting constraints.

Seymour and Ansonia to Derby



Treatment Plants

Naugatuck

Beacon Falls

~~Seymour~~~~Ansonia~~

Derby

Under this alternative, the Seymour and Ansonia WPCFs would be decommissioned and replaced with pumping stations. All wastewater from those communities would be conveyed to the Derby WPCF for treatment. Since the Ansonia WPCF was upgraded relatively recently (completed 2011), the Ansonia to Derby part of this alternative might not be implemented until closer to the end of the planning period.

Treatment Capacity

The capacity of the Derby WPCF would have to be increased as part of a major upgrade to the facility. Due to high wet weather flows, this alternative would require significantly increasing the wet weather treatment capacity. This alternative also might storage facilities.

Phosphorus Removal

Since the Derby WPCF discharges to the Housatonic River, where phosphorus removal is not required, this alternative would reduce some of the costs associated with advanced treatment for wastewater from Seymour and Ansonia.

Collection System

Naugatuck

Beacon Falls

Seymour

Ansonia

Derby

Collection System Rehabilitation

The base case assumes maintenance for each collection system necessary to improve the system to a basic level of reliability and performance consistent with the I/I reduction assumptions in the flows and loads analysis.

Lift Stations & Conveyance

Naugatuck

Beacon Falls

Seymour

Ansonia

Derby

Capacity

The new pumping stations in Seymour and Ansonia would be sized, along with any storage at that location, to handle anticipated peak hydraulic flow from its respective wastewater collection system (for Seymour PS: 6.7 MGD peak day, 7.4 MGD peak hour; for Ansonia PS: 5.4 MGD peak day, 9.5 MGD peak hour). This alternative might also include a headworks

facility for grit and screenings removal at one or both of the new pumping stations.

Conveyance Corridors

It would be possible to convey this flow in two stages: Seymour to Ansonia and Ansonia to Derby.

Seymour to Ansonia could be routed along

town roads without major topographic obstructions, with a maximum elevation of approximately 130 ft and a length of nearly 14,000 ft.

Ansonia to Derby has multiple potential routings of approximately 8,000 to 9,000 ft with a maximum elevation up to nearly 90 ft.

9a

Seymour and Ansonia to Derby, I/I Reduction

Treatment Plants

Naugatuck

Beacon Falls

~~Seymour~~~~Ansonia~~

Derby

Under this alternative, the Seymour and Ansonia WPCFs would be decommissioned and replaced with pumping stations. All wastewater from those communities would be conveyed to the Derby WPCF for treatment. Since the Ansonia WPCF was upgraded relatively recently (completed 2011), the Ansonia to Derby part of this alternative might not be implemented until closer to the end of the planning period.

Treatment Capacity

The capacity of the Derby WPCF would have to be increased as part of a major upgrade to the facility. Depending on the extent of I/I removal, additional measures such as storage facilities may be required.

Phosphorus Removal

Since the Derby WPCF discharges to the

Housatonic River, where phosphorus removal is not required, this alternative would reduce some of the costs associated with advanced treatment for wastewater from Seymour and Ansonia.

Collection System

Naugatuck

Beacon Falls

Seymour

Ansonia

Derby

Collection System Rehabilitation

Due to excessive peak flows in Derby, Seymour, and Ansonia, aggressive I/I reduction would be required in all three systems. I/I reduction could be mitigated with the use of storage. The extent of I/I reduction would need to be balanced with marginal cost of treatment and marginal cost of storage.

Lift Stations & Conveyance

Naugatuck

Beacon Falls

Seymour

Ansonia

Derby

Capacity

The new pumping stations in Seymour and Ansonia would be sized, along with any storage, to handle anticipated peak hydraulic flow from its respective collection system (for Seymour PS: 6.7 MGD peak day, 7.4 MGD peak hour; for Ansonia PS: 5.4 MGD peak day, 9.5 MGD peak hour). This alternative might also include a headworks facility for grit and

screenings removal at one or both of the new pumping stations. With aggressive I/I removal, the necessary pumping capacity could be reduced.

Conveyance Corridors

It would be possible to convey this flow in two stages: Seymour to Ansonia and Ansonia to Derby.

Seymour to Ansonia could be routed along town roads without major topographic obstructions, with a maximum elevation of approximately 130 ft and a length of nearly 14,000 ft.

Ansonia to Derby has multiple potential routings of approximately 8,000 to 9,000 ft with a maximum elevation up to nearly 90 ft.

10

Seymour to Ansonia Part of Ansonia to Derby

Treatment Plants

Naugatuck

Beacon
Falls~~Seymour~~

Ansonia

Derby

Under this alternative, the Seymour WPCF would be decommissioned and replaced with a pumping station. All wastewater from Seymour would be conveyed to the Ansonia WPCF for treatment. Also, Bartholomew Pump Station in Ansonia would be decommissioned, and all flow going to it would be diverted to flow by gravity to the Derby WPCF.

If the hydraulic constrictions at the Ansonia WPCF are eliminated to restore its design peak flow capacity to 12.0 MGD, the Ansonia WPCF would be able to handle the annual average, max month flows, but not the peak flows from Seymour. Modifications to increase wet weather treatment capacity and/or storage would be needed at the Ansonia WPCF, and possibly also at the Derby WPCF.

Phosphorus Removal

Since the Derby WPCF discharges to the Housatonic River, where phosphorus removal is not required, this alternative would reduce some of the costs associated with advanced treatment for wastewater from Ansonia.

Collection System

Naugatuck

Beacon
Falls

Seymour

Ansonia

Derby

Collection System Rehabilitation

The base case assumes maintenance for each collection system necessary to improve the system to a basic level of reliability and performance consistent with the I/I reduction assumptions in the flows and loads analysis. This alternative does not include additional collection system improvements.

Lift Stations & Conveyance

Naugatuck

Beacon
Falls

Seymour

Ansonia

Derby

Capacity

The new Seymour pump station would be designed to pump all flow from Seymour (2040 design flows 6.7 MGD for peak day, and 7.4 MGD for peak hour), possibly with some storage provided at the pump station to mitigate peak flows and reduce max pumping capacity.

Conveyance Corridors

Seymour to Ansonia could be routed along town roads without major topographic obstructions, with a maximum elevation of approximately 130 ft and a length of nearly 14,000 ft.

Ansonia to Derby has multiple potential routings of approximately 8,000 to 9,000 ft

with a maximum elevation up to nearly 90 ft.

10a

Seymour to Ansonia, Part of Ansonia to Derby, I/I Reduction

Treatment Plants

Naugatuck

Beacon
Falls~~Seymour~~

Ansonia

Derby

Under this alternative, the Seymour WPCF would be decommissioned and replaced with a pumping station. All wastewater from Seymour would be conveyed to the Ansonia WPCF for treatment. Also, Bartholomew Pump Station in Ansonia would be decommissioned, and all flow going to it would be diverted to flow by gravity to the Derby WPCF.

If the hydraulic constrictions at the Ansonia WPCF are eliminated to restore its design peak flow capacity to 12.0 MGD, Ansonia WPCF would be able to handle the annual average, max month flows, but not the peak flows from Seymour. Depending on the extent of I/I removal, modifications to increase wet weather treatment capacity and/or storage may be needed at the Ansonia WPCF, and

possibly also at the Derby WPCF.

Phosphorus Removal

Since the Derby WPCF discharges to the Housatonic River, where phosphorus removal is not required, this alternative would reduce some of the costs associated with advanced treatment for wastewater from Ansonia.

Collection System

Naugatuck

Beacon
Falls

Seymour

Ansonia

Derby

Collection System Rehabilitation

Due to excessive peak flows in Derby, Seymour, and Ansonia, aggressive I/I reduction would be required in all three systems. I/I reduction could be mitigated with the use of storage. The extent of I/I reduction would need to be balanced with marginal cost of treatment and marginal cost of storage.

Lift Stations & Conveyance

Naugatuck

Beacon
Falls

Seymour

Ansonia

Derby

Capacity

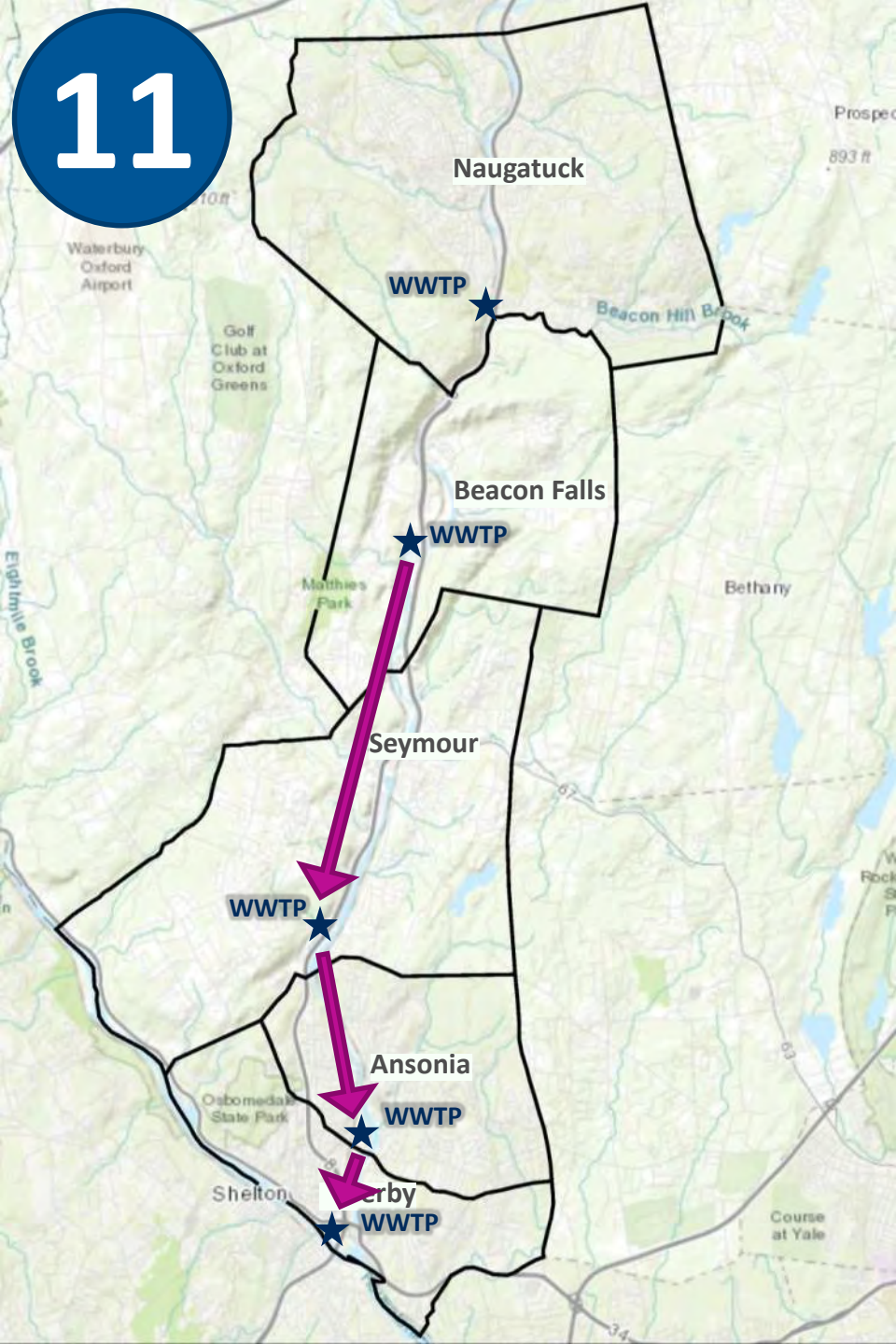
The new Seymour pump station would be designed to pump all flow from Seymour (2040 design flows 6.7 MGD for peak day, and 7.4 MGD for peak hour), possibly with some storage provided at the pump station to mitigate peak flows and reduce max pumping capacity.

With aggressive I/I removal, the necessary pumping capacity would be reduced.

Conveyance Corridors

Seymour to Ansonia could be routed along town roads without major topographic obstructions, with a maximum elevation of approximately 130 ft and a length of nearly 14,000 ft.

Ansonia to Derby has multiple potential routings of approximately 8,000 to 9,000 ft with a maximum elevation up to nearly 90 ft.



Beacon Falls and Seymour to Ansonia Part of Ansonia to Derby

Treatment Plants

Naugatuck

~~Beacon Falls~~~~Seymour~~

Ansonia

Derby

Under this alternative, the Beacon Falls and Seymour WPCF would be decommissioned and replaced with pumping stations. This wastewater would be conveyed to the Ansonia WPCF for treatment. Also, Bartholomew Pump Station in Ansonia would be decommissioned, and all flow going to it would be diverted to flow by gravity to the Derby WPCF.

If the hydraulic constrictions at the Ansonia WPCF are eliminated to restore its design peak flow capacity to 12.0 MGD, the Ansonia WPCF would be able to handle the annual average and max month flows from Seymour and Beacon Falls. However, additional measures such as treatment plant modifications to increase wet weather capacity at the Ansonia WPCF, possibly also combined with storage,

would be required to accommodate the peak day and peak hour flows.

Phosphorus Removal

Since the Derby WPCF discharges to the Housatonic River, where phosphorus removal is not required, this alternative would reduce some of the costs associated with advanced treatment for wastewater from Ansonia.

Collection System

Naugatuck

~~Beacon Falls~~~~Seymour~~

Ansonia

Derby

Collection System Rehabilitation

The base case assumes maintenance for each collection system necessary to improve the system to a basic level of reliability and performance consistent with the I/I reduction assumptions in the flows and loads analysis.

Lift Stations & Conveyance

Naugatuck

Beacon Falls

Seymour

Ansonia

Derby

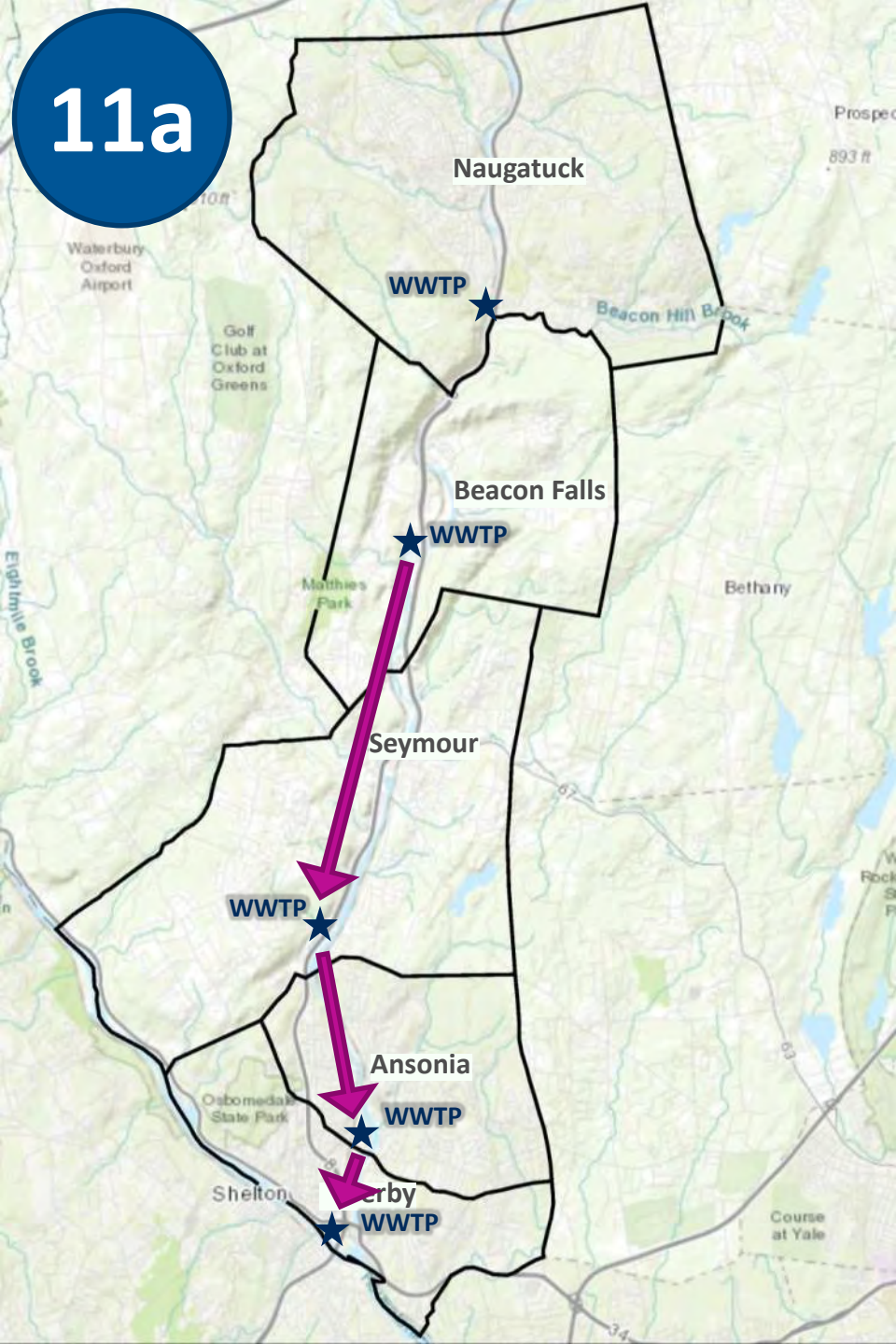
Capacity

The new pumping stations in Beacon Falls and Seymour would be sized, along with any storage at that location, to handle anticipated peak hydraulic flow from their respective wastewater collection systems. This alternative might also include a headworks facility for grit and screenings removal at one or both of the new upstream pumping stations.

Conveyance Corridors

Conveyance would be expected to utilize similar corridors to prior alternatives for each leg of conveyance.

11a



Beacon Falls and Seymour to Ansonia Part of Ansonia to Derby, I/I Reduction

Treatment Plants

Naugatuck

~~Beacon Falls~~~~Seymour~~

Ansonia

Derby

Under this alternative, the Beacon Falls and Seymour WPCF would be decommissioned and replaced with a pumping stations. This wastewater would be conveyed to the Ansonia WPCF for treatment. Also, Bartholomew Pump Station in Ansonia would be decommissioned, and all flow going to it would be diverted to flow by gravity to the Derby WPCF.

If the hydraulic constrictions at the Ansonia WPCF are eliminated to restore its design peak flow capacity to 12.0 MGD, the Ansonia WPCF would be able to handle the annual average and max month flows from Seymour and Beacon Falls. Depending on the extent of I/I reduction, additional measures such as treatment plant modifications to increase wet weather capacity, possibly also combined with

storage, may be required to accommodate the peak flows.

Phosphorus Removal

Since the Derby WPCF discharges to the Housatonic River, where phosphorus removal is not required, this alternative would reduce some of the costs associated with advanced treatment for wastewater from Ansonia.

Collection System

Naugatuck

Beacon Falls

Seymour

Ansonia

Derby

Collection System Rehabilitation

Due to excessive peak flows, aggressive I/I reduction would be required in all four systems. I/I reduction could be mitigated with the use of storage. The extent of I/I reduction would need to be balanced with marginal cost of treatment and marginal cost of storage.

Lift Stations & Conveyance

Naugatuck

Beacon Falls

Seymour

Ansonia

Derby

Capacity

The new pumping stations in Beacon Falls, Seymour and Ansonia would be sized, along with any storage at that location, to handle anticipated peak hydraulic flow from their respective wastewater collection systems. This alternative might also include a headworks facility for grit and screenings removal at one or more of the new upstream pumping

stations.

With aggressive I/I removal, the necessary pumping capacity would be reduced.

Conveyance Corridors

Conveyance would be expected to utilize similar corridors to prior alternatives for each leg of conveyance.

Beacon Falls, Seymour, and Ansonia to Derby

Treatment Plants

Naugatuck

~~Beacon Falls~~~~Seymour~~~~Ansonia~~

Derby

Under this alternative, the Beacon Falls, Seymour and Ansonia WPCFs would be decommissioned and replaced with pumping stations. All wastewater from those communities would be conveyed to the Derby WPCF for treatment. Since the Ansonia WPCF was upgraded relatively recently (completed 2011), this alternative might not be fully implemented until closer to the end of the

planning period.

The capacity of the Derby WPCF would have to be increased as part of a major upgrade to the facility, to provide sufficient treatment capacity for flows from all four communities through 2040. Storage may also be required for peak flows.

Phosphorus Removal

Since the Derby WPCF discharges to the Housatonic River, where phosphorus removal is not required, this alternative would reduce some of the costs associated with advanced treatment for wastewater from Beacon Falls, Seymour and Ansonia.

Collection System

Naugatuck

Beacon Falls

Seymour

Ansonia

Derby

Collection System Rehabilitation

The base case assumes maintenance for each collection system necessary to improve the system to a basic level of reliability and performance consistent with the I/I reduction assumptions in the flows and loads analysis.

Lift Stations & Conveyance

Naugatuck

Beacon Falls

Seymour

Ansonia

Derby

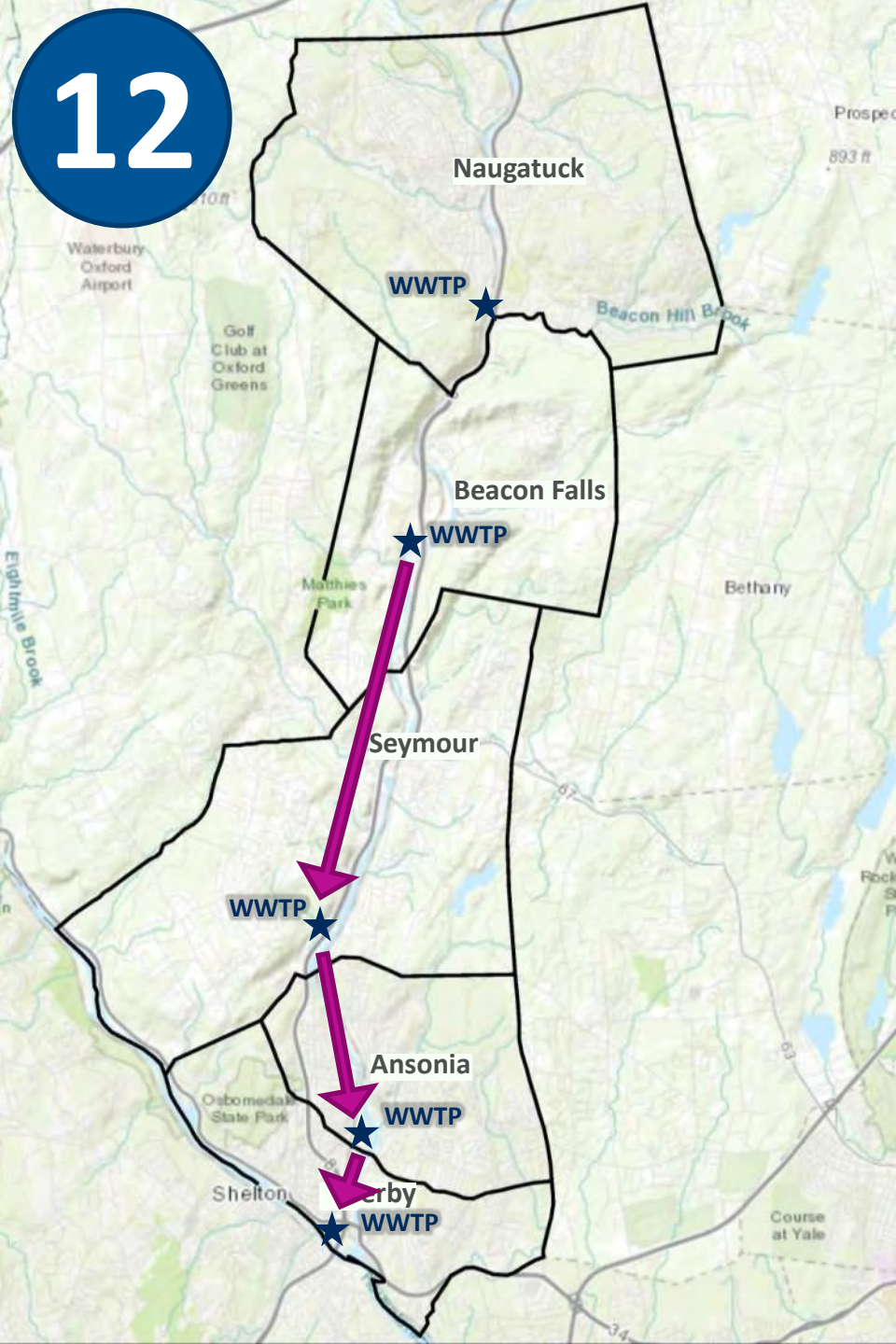
Capacity

The new pumping stations in Beacon Falls, Seymour and Ansonia would be sized, along with any storage at that location, to handle anticipated peak hydraulic flow from their respective wastewater collection systems. This alternative might also include a headworks facility for grit and screenings removal at one or more of the new upstream pumping

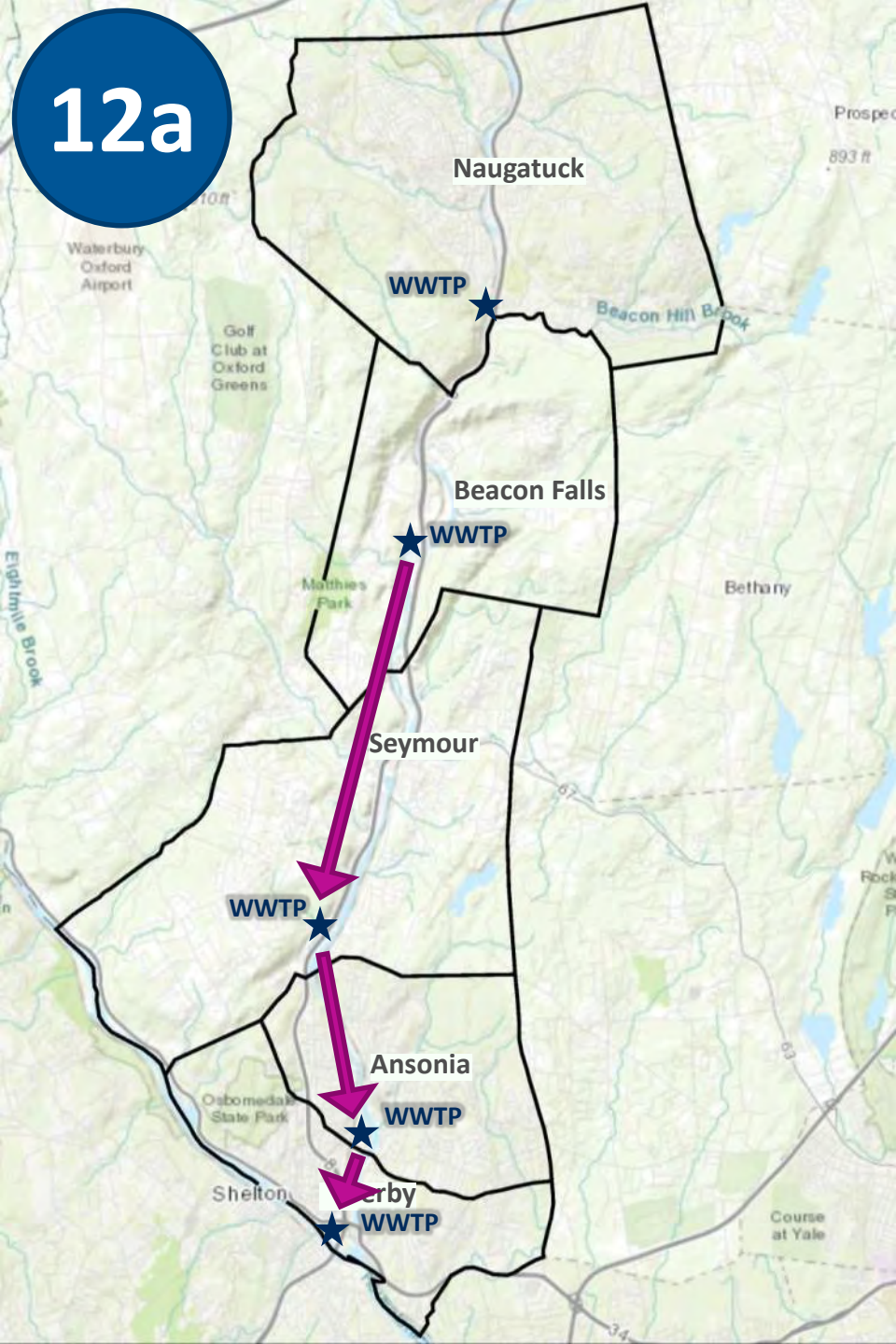
stations.

Conveyance Corridors

Conveyance would be expected to utilize similar corridors to prior alternatives for each leg of conveyance.



12a



Beacon Falls, Seymour, and Ansonia to Derby, I/I Reduction

Treatment Plants

Naugatuck

~~Beacon Falls~~~~Seymour~~~~Ansonia~~

Derby

Under this alternative, the Beacon Falls, Seymour and Ansonia WPCFs would be decommissioned and replaced with pumping stations. All wastewater from those communities would be conveyed to the Derby WPCF for treatment. Since the Ansonia WPCF was upgraded relatively recently (completed 2011), this alternative might not be fully implemented until closer to the end of the

planning period. The capacity of the Derby WPCF would have to be increased as part of a major upgrade to the facility, to provide for flows from all four communities through 2040. Due to high wet weather flows in the collection systems, particularly in Derby, Ansonia and Seymour, this alternative would require significantly increasing the wet weather capacity and possibly also storage.

Phosphorus Removal

Since the Derby WPCF discharges to the Housatonic River, where phosphorus removal is not required, this alternative would reduce some of the costs associated with advanced treatment for wastewater from Beacon Falls, Seymour and Ansonia.

Collection System

Naugatuck

Beacon Falls

Seymour

Ansonia

Derby

Collection System Rehabilitation

Due to excessive peak flows, aggressive I/I reduction would be required in all four systems. I/I reduction could be mitigated with the use of storage. The extent of I/I reduction would need to be balanced with marginal cost of treatment and marginal cost of storage.

Lift Stations & Conveyance

Naugatuck

Beacon Falls

Seymour

Ansonia

Derby

Capacity

The new Seymour pump station would be designed to pump all flow from Seymour (2040 design flows 6.7 MGD for peak day, and 7.4 MGD for peak hour), possibly with some storage provided at the pump station to mitigate peak flows and reduce max pumping capacity.

With aggressive I/I removal, the necessary pumping capacity would be reduced.

Conveyance Corridors

Conveyance would be expected to utilize similar corridors to prior alternatives for each leg of conveyance.

3.0 Interconnection Conveyance Route Options

3.1 Approach

Initial potential route options were investigated at a high level using available GIS data layers from the State of Connecticut. This analysis generally considered potential alignments from WPCF to WPCF following existing roads or rights of way. The intent of this investigation was not to optimize potential routes, but to identify options for routing that would help characterize the viability of each regionalization alternative. Route identification will be conducted in greater detail for selected alternatives as part of Phase 2.

Routes were identified as segments from WPCF to WPCF. Therefore, some alternatives that involve more than one community may include multiple segments of routes.

Additionally, some alternatives included routes that would not extend from plant to plant, such as from the Bartholomew pump station in Ansonia to the Derby WPCF or from the plant's outfall to the Housatonic River. These particular routes were not explored in this analysis and will be undertaken for short-listed alternatives in Phase 2.

The following GIS data layers were used as the basis for this analysis:

- Proposed Interconnector Routes measured length in Feet in NAD 1983 State Plane Connecticut
- Aquifer Protection Area
- Critical Habitat (USFWS)
- Protected Open Space
- Floodplains
- Hydrography
- Elevation

None of the conveyance route options are within any of the Aquifer Protection Areas.

At Derby, Protected Open Space is railroad property along the Naugatuck River.

3.2 Beacon Falls to Naugatuck

The Beacon Falls to Naugatuck route is likely the most challenging segment in the study. Steep slopes limit available space along the Naugatuck River, and Toby's Rock Mountain poses a major constraint for routing.

Four routing options were identified, as shown in Figure 3-1.

Options 1 and 3 attempt to follow the shortest path and minimize elevation constraints, but to accomplish this, they are aligned in major rights of way (ROW) for the railroad and Route 8. Options 2 and 4 recognize the difficulties of the rights of way and utilize routes around the mountain.

Following are brief summaries of each option.

3.2.1 Option 1: Railroad ROW

With a length of nearly 18,000 ft, this option would provide the most direct path with the smallest elevation differential. However, obtaining permission to use the railroad ROW may be insurmountable.

3.2.2 Option 2: Transmission ROW

It would likely be easier to obtain permission to use the existing power transmission right of way around Toby's Rock Mountain, but this route adds substantial length and multiple changes in elevation that would likely require multiple pump stations. The length of this alignment is nearly 28,000 ft, with a maximum elevation of 780 ft. Although this may be technically feasible, it is not expected to be the preferred option.

3.2.3 Option 3: Route 8 ROW

Although the length of this option, nearly 28,000 ft, is long, it provides an approach with less elevation differential than options around Toby's Rock Mountain. The route would for the most part follow the Route 8 right of way and use existing bridges for river crossing. The maximum elevation is approximately 270 ft. Although construction would be difficult in the major transportation right of way, this may be the most viable option.

3.2.4 Option 4: Around Toby's Rock Mountain

This option would rely on mountain roads to route past Toby's Rock Mountain. With a length of over 28,000 ft, the route has less overall variation in elevation than Option 2, as well as a somewhat lower maximum elevation of approximately 740 ft. Although this alignment would likely require less pumping than Option 2, the elevation differential is expected to lead to high pumping costs, and while possible, this is not expected to be the preferred option for routing.

3.3 Beacon Falls to Seymour

The Beacon Falls to Seymour route features similar constraints to the Naugatuck Route, but less extreme in terms of slopes and elevations. The railroad ROW and the Route 8 ROW still occupy central locations for the preferred routing. Three options were identified for routing and are presented in Figure 3-2.

3.3.1 Option 1: Avoid the Railroad ROW

This option recognizes that the railroad ROW is likely an insurmountable obstacle and provides the most direction option that avoids the railroad. It has a length of approximately 28,000 ft, and it has a significant hill, with a maximum elevation of approximately 470 ft. A significant portion of the alignment would be parallel to and likely use the Route 8 ROW.

3.3.2 Option 2: Use the Railroad ROW

This option is the most direct path with a length of just over 26,000 ft. The peak elevation of approximately 170 ft also provides the least elevation differential, which would be beneficial for pumping costs. However, the likelihood of using the railroad ROW is not good.

3.3.3 Option 3: Avoid both Railroad and Route 8 ROW

This option avoids the major rights of way, but that significantly increases the length of the alignment as well as the elevation differential. With a length of approximately 48,000 ft and a maximum elevation of approximately 640 ft, this route or similar possible routes do not appear desirable unless there is significant resistance to using the Route 8 ROW.

3.4 Seymour to/from Ansonia

The alignment from Seymour to or from Ansonia is less problematic than the routes to Beacon Falls because there are more town roads and less hills, so there is less need to use major rights of way. A typical route is shown in Figure 3-3, but there are multiple variations that are possible based on other factors, such as traffic disruption, planned road repairs, etc. The alignment shown is under 14,000 ft, and it has a maximum elevation of approximately 130 ft, making this alignment feasible.

3.5 Seymour to/from Derby

The alignment from Seymour to/from Derby would most likely be routed through Ansonia as shown in Figure 3-4. Alternative routes are possible if needed, but they are not likely to be preferred.

3.6 Ansonia to/from Derby

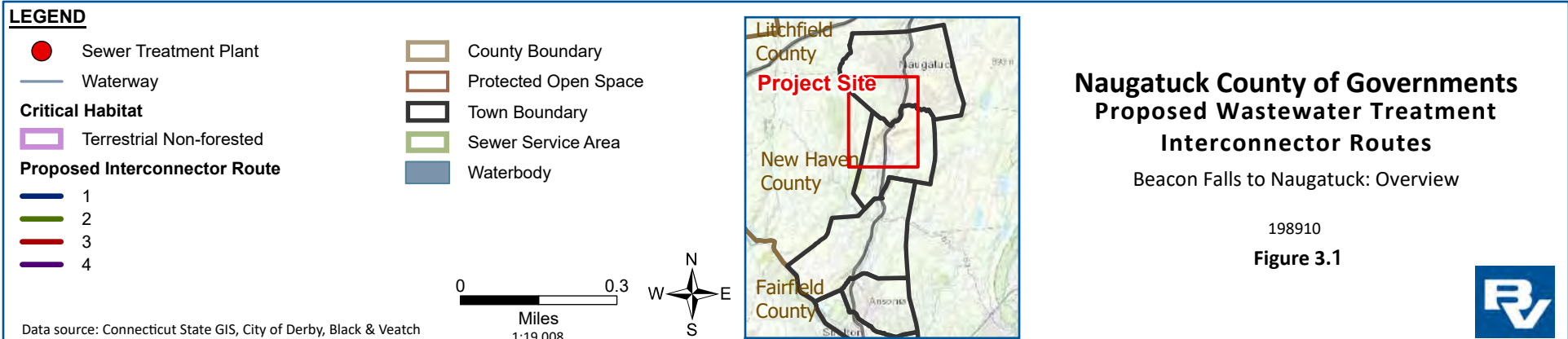
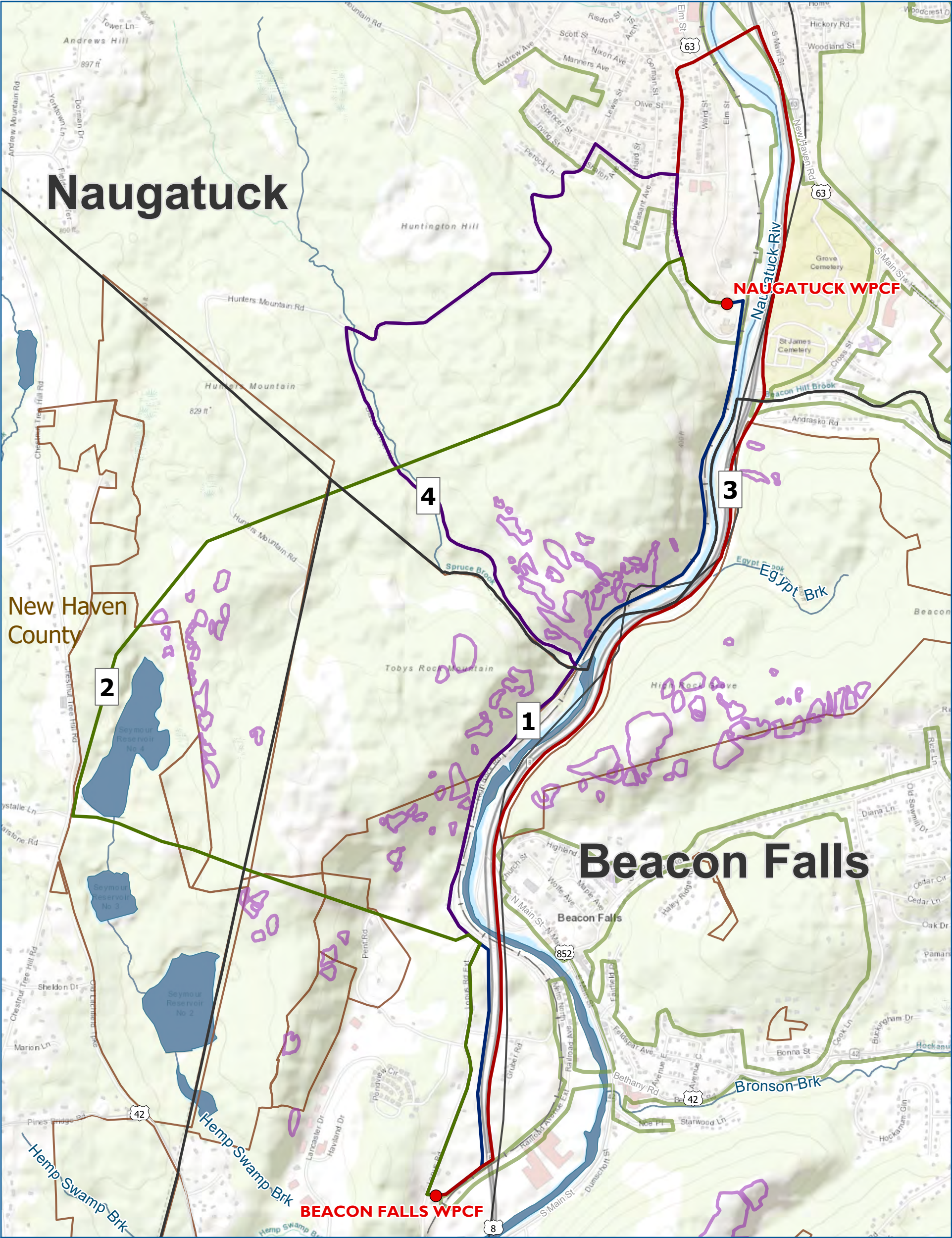
There are multiple potential routes between Ansonia and Derby that have been investigated previously. Two of the options are highlighted here and are shown in Figure 3-5.

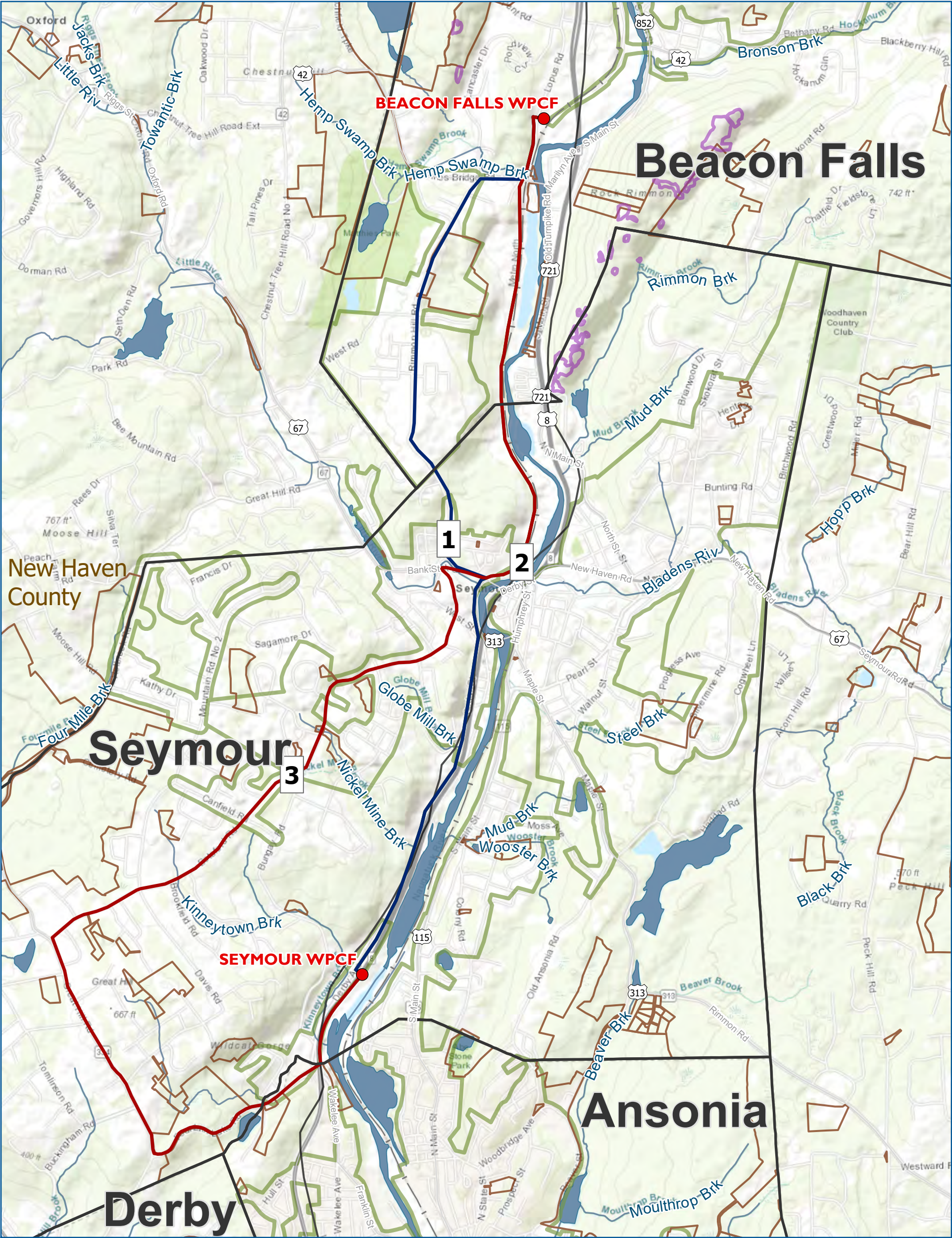
3.6.1 Option 1: Pershing Street and Town Roads

This route, with a total length of somewhat over 8,000 ft and a maximum elevation of approximately 90 ft, is likely the preferred route because it avoids conflicts with existing rights of way and wetlands. However, the potential disruption due to construction could be a factor favoring other routes.

3.6.2 Option 2: Naugatuck River

This route, with a total length under 9,000 ft and a maximum elevation under 50 ft, would provide more efficient pumping and less traffic disruption. However, construction along the river would have wetlands, flood control, and transportation obstacles that likely make this option less desirable.





LEGEND

● Sewer Treatment Plant

— Waterway

Critical Habitat

▭ Terrestrial Non-forested

Proposed Interconnector Route

1

2

3

▭ County Boundary

▭ Protected Open Space

▭ Town Boundary

▭ Sewer Service Area

▭ Waterbody

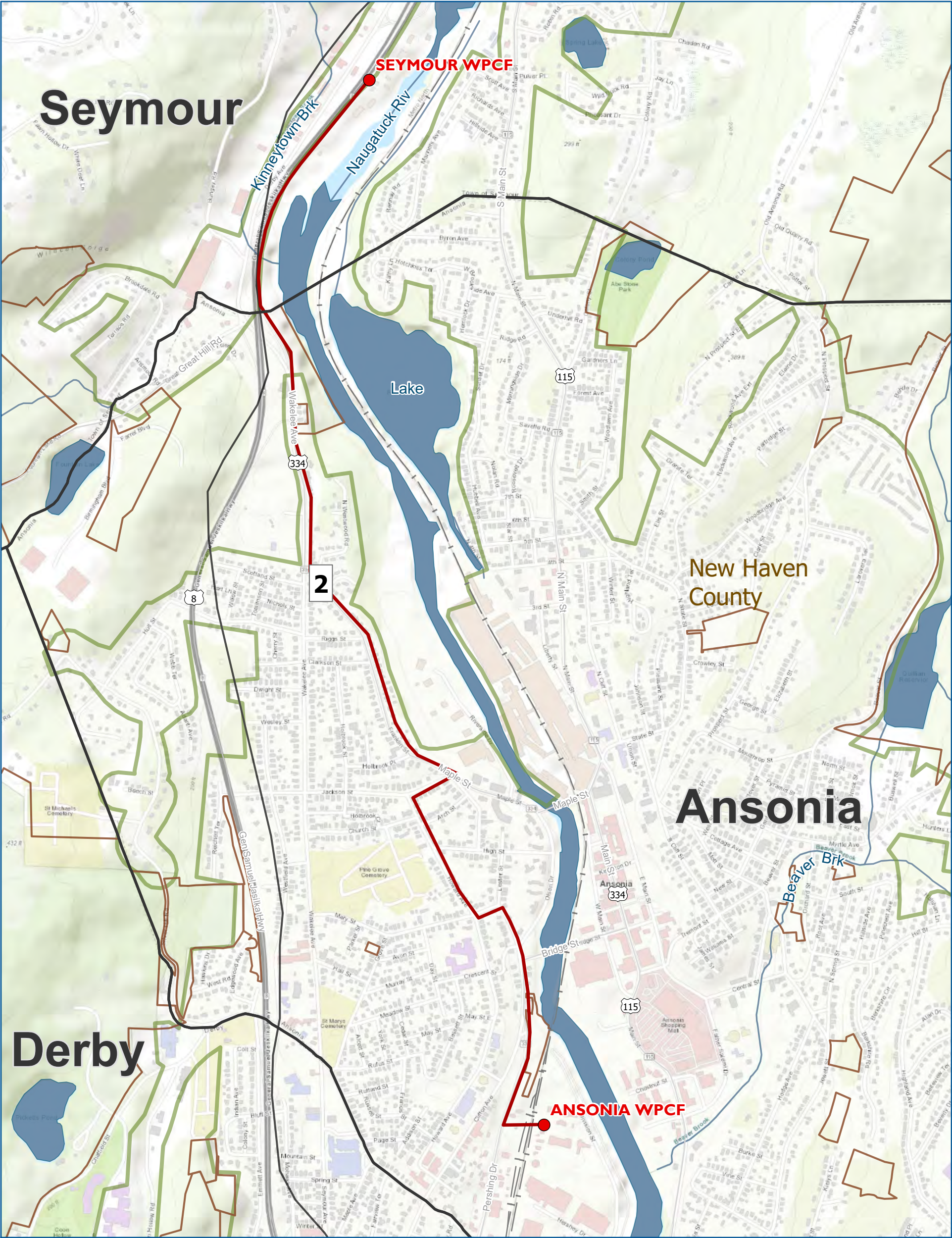
0 0.3 Miles 1:31,680

N
W E
S

**Naugatuck County of Governments
Proposed Wastewater Treatment
Interconnector Routes**

Beacon Falls to Seymour: Overview

198910
Figure 3.2



LEGEND

● Sewer Treatment Plant

— Waterway

Critical Habitat

▭ Terrestrial Non-forested

Proposed Interconnector Route

2

▭ County Boundary

▭ Protected Open Space

▭ Town Boundary

▭ Sewer Service Area

▭ Waterbody

0 0.15 Miles 1:12,165

N
W E
S

Litchfield County
Project Site

New Haven County

Fairfield County

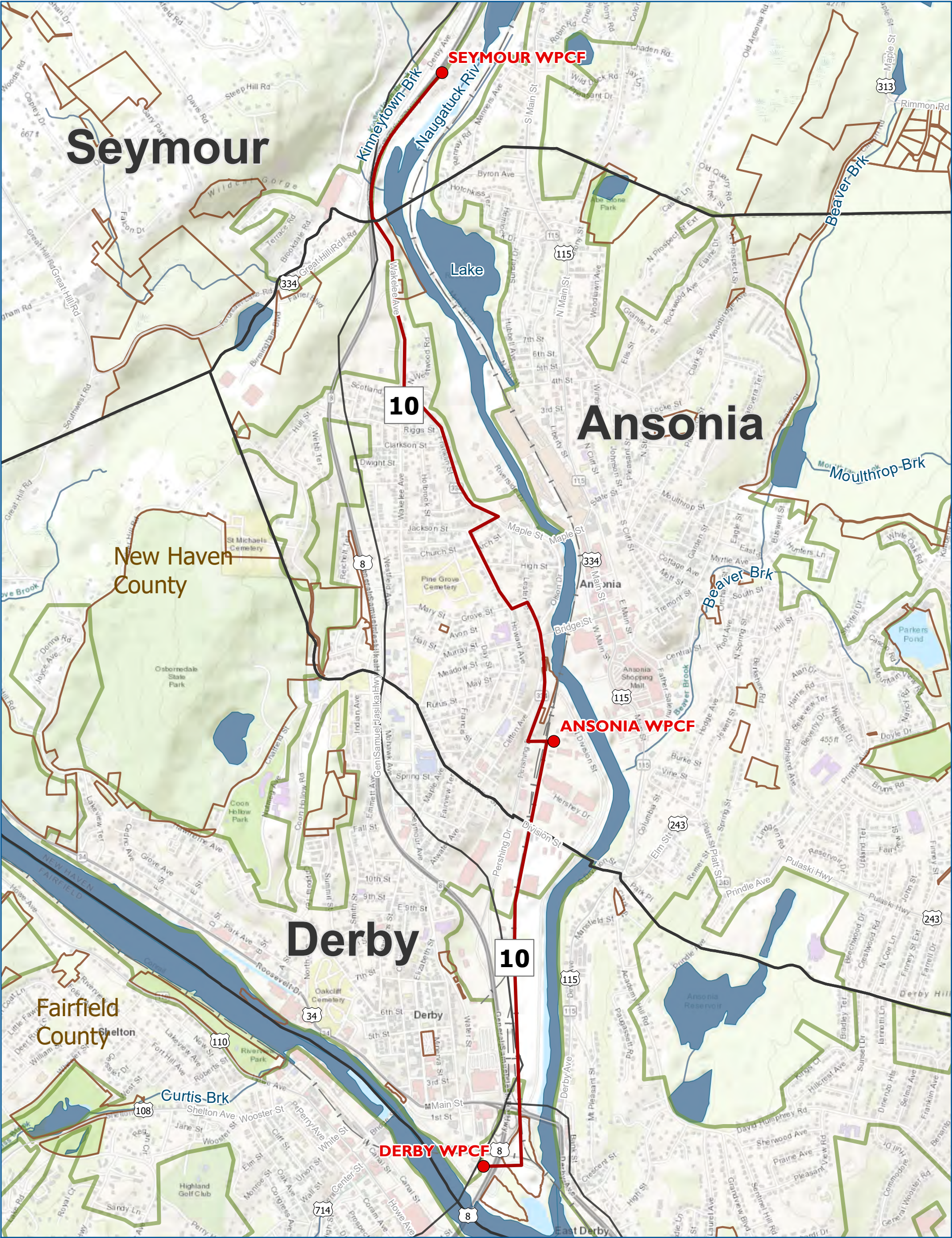
**Naugatuck County of Governments
Proposed Wastewater Treatment
Interconnector Routes**

Seymour to Ansonia: Overview

198910

Figure 3.3

Data source: Connecticut State GIS, City of Derby, Black & Veatch



LEGEND

Sewer Treatment Plant	County Boundary
Waterway	Protected Open Space
Critical Habitat	Town Boundary
Terrestrial Non-forested	Sewer Service Area
Proposed Interconnector Route	Waterbody
10	

0 0.3 Miles
1:19,008

N
W E
S

**Naugatuck County of Governments
Proposed Wastewater Treatment
Interconnector Routes**

Seymour to Derby
through Ansonia: Overview
198910
Figure 3.4



LEGEND

● Sewer Treatment Plant

— Waterway

Critical Habitat

▭ Terrestrial Non-forested

Proposed Interconnector Route

1

2

▭ County Boundary

▭ Protected Open Space

▭ Town Boundary

▭ Sewer Service Area

▭ Waterbody

0 0.15

Miles

1:9,504

N

W

E

S

Litchfield County

New Haven County

Fairfield County

Project Site

Naugatuck County of Governments

Proposed Wastewater Treatment Interconnector Routes

Ansonia to Derby: Overview

198910

Figure 3.5

Data source: Connecticut State GIS, City of Derby, Black & Veatch

4.0 Screening Criteria

The screening criteria presented below will be used to screen out the less preferred regional wastewater alternatives from the previously described initial list of regional wastewater alternatives. It is emphasized that this is a “rough screening” of the alternatives as the alternatives have not been adequately developed to apply the criteria with accuracy; however, the rough screening will allow for identifying fatal flaws among the alternatives. Those alternatives exhibiting fatal flaws will be omitted. Again, it is emphasized that the intent of this rough screening is to identify the long list of alternatives to carry into Phase II.

Additional development of the long list alternatives will be undertaken in Phase II along with further screening of the alternatives with the purpose of identifying a short-list of alternatives for more in-depth study and evaluation.

4.1 Identification of Screening Criteria

- Adequate space at the plant-site to accomplish required treatment
- Ease or difficulty of incorporating the treatment process at the WWTPs considering facilities layout and space requirements.
- Complexity in operation and maintenance. This will address the treatment plant, the collection and pump systems and the pumping and conveyance system required as a result of the regionalization.
- Implementation schedule
- Environmental restrictions
- Regulatory and permitting requirements
- Community benefits
- Capital and O&M costs, and overall life cycle cost (these costs will be macro-level and useful for comparative analysis only)
- Topographic or right-of-way constraints in interconnecting communities

4.2 Initial Screening of Alternatives

Each of the regional wastewater alternatives were compared for each criterion on three levels – green, yellow, and red – with green representing a most favorable rating and red representing a least favorable rating as compared to other alternatives.

Alt No.	Abbreviated Description	Space/Constraint	Existing Facilities	O&M	Schedule	Env	Reg	Benefits	Relative Cost
1	BF→N	●	●	●	●	●	●	●	●
2	BF→S	●	●	●	●	●	●	●	●
2a	BF→S, I/I	●	●	●	●	●	●	●	●
3	D→A	●	●	●	●	●	●	●	●
3a	D→A, I/I	●	●	●	●	●	●	●	●
4	D→A→H	●	●	●	●	●	●	●	●
4a	D→A→H, I/I	●	●	●	●	●	●	●	●
5	D&S→A	●	●	●	●	●	●	●	●
5a	D&S→A, I/I	●	●	●	●	●	●	●	●
5b	D&S→A→H	●	●	●	●	●	●	●	●
5c	D&S→A→H, I/I	●	●	●	●	●	●	●	●
6	D→S, D→A	●	●	●	●	●	●	●	●
6a	D→S, D→A, I/I	●	●	●	●	●	●	●	●
7	D→S, D→A, D→D	●	●	●	●	●	●	●	●
7a	D→S, D→A, D→D, I/I	●	●	●	●	●	●	●	●
8	A→D	●	●	●	●	●	●	●	●
8a	A→D, I/I	●	●	●	●	●	●	●	●
9	S&A→D	●	●	●	●	●	●	●	●
9a	S&A→D, I/I	●	●	●	●	●	●	●	●
10	S→A, A→D	●	●	●	●	●	●	●	●
10a	S→A, A→D, I/I	●	●	●	●	●	●	●	●
11	BF,S→A, A→D	●	●	●	●	●	●	●	●
11a	BF,S→A, A→D, I/I	●	●	●	●	●	●	●	●
12	BF,S,A→D	●	●	●	●	●	●	●	●
12a	BF,S,A→D, I/I	●	●	●	●	●	●	●	●

5.0 Conclusions and Recommendations

Based on the initial rough screening according to a qualitative assessment of the screening criteria for each alternative, there are potential advantages or benefits to many of the alternatives.

Alternatives with fatal flaws (identified as red in the assessment) were removed from the analysis.

In the initial rough screening, this meant that alternatives 7 and 7a were found to be clearly inferior to other alternatives and should be removed from consideration. The remaining alternatives are of potential interest depending on the relative costs of construction and operation and should be considered further. It is recommended that all other alternatives should be carried into Phase II analysis for more detailed study and analysis.

APPENDIX A

Technical Memorandum 1: Flows and Loads

TECHNICAL MEMORANDUM

Naugatuck Valley Regional Council of Governments
NVCOG Wastewater Regionalization Study
TM No. 1 – Flows and Loads

B&V Project 198910
B&V File 43.0010
October 30, 2018

1.0 PURPOSE AND BACKGROUND

The purpose of this technical memorandum is to establish the flows and loads to be used for each of the five wastewater treatment communities participating in the NVCOG Wastewater Regionalization Study. That includes both average and peak flows to be expected over a 20-year planning period, through 2040. This information will be used as a basis for evaluating the various regionalization alternatives in the study. The projections in this memorandum have been developed based on review of MOR data, rainfall records and population projections, as well as input from officials of the five municipalities.

In each of the five communities, the average flows received to the plant have been significantly below the plant's permitted capacity in recent years, as shown on Table 1-1 below.

Table 1-1 Annual Average (AA) Flow: Actual (2015-2017) vs. Permitted Capacity

Municipal WPCF	Average Annual (AA) Flow, 2015-2017 (MGD)	Permitted AA Design Capacity (MGD)	2015-2017 AA Flow as Percent of Permitted Capacity
Derby	1.3	3.5	37%
Ansonia	1.57	3.5	45%
Seymour	0.97	2.93	33%
Beacon Falls	0.31	0.71	44%
Naugatuck	4.61	10.3	45%

2.0 POPULATION PROJECTIONS FOR THE FIVE COMMUNITIES

The Connecticut State Data Center (CSDC) population projections for the five towns included in this study, through planning year 2040, are summarized in Table 2-1 below. This information was based on recent population projections published by the CSDC, as funded by the Office of Policy and Management (OPM). The US Census data is included in the table as well, for reference.

Table 2-1 CT State Data Center Population Projections to 2040

Municipality	US Census 2010	Based on CT State Data Center Population Projections (published August 31, 2017)				
		2020	2030	2035	2040	Percent increase, 2040 vs. 2020
Derby	12,902	13,251	13,803	13,959	14,082	6.3%
Ansonia	19,249	19,841	20,648	20,890	21,067	6.2%
Seymour	16,540	16,798	16,924	16,852	16,753	-0.3%
Beacon Falls	6,049	6,421	6,587	6,591	6,587	2.6%
Naugatuck	31,862	32,212	32,638	32,372	31,854	-1.1%
TOTAL	86,602	88,523	90,600	90,664	90,343	2.1%

The CSDC is projecting that this region, like most of the rest of the state, will experience very modest growth over the next twenty years.

3.0 APPROACH TO DEVELOPING CURRENT FLOWS AND LOADS

One of the most critical considerations in evaluating regionalization alternatives is peak flows to the wastewater treatment plants. All five of the communities in this study have older collection systems, with significantly higher flows during wet weather conditions, when infiltration and inflow (I/I) cause significant increases in flows to the treatment plants.

Typically, the most useful data to determine current condition baseline flows to the plant would be from the most recent years, as reported in the monthly operator reports (MOR's) submitted by the communities to the State of Connecticut. However, in this case the three most recent years of record (2015-2017) were characterized by unusually low rainfall, in comparison to the overall 2000-2017 period. It is not unusual to review a longer period of flow records when performing future projections; this was done here as explained below.

Rainfall data from local weather stations in the NOAA database were reviewed, to compare recent historic rainfall patterns with observed flows at the water pollution control facilities (WPCF's). Since the available data from the nearby Waterbury-Oxford Airport station had gaps in the period of interest, other nearby weather stations with more complete sets of rainfall data were considered.

The three weather stations in the region with the best data available for the continuous period since 2000 were at the following locations: Meriden Markham Municipal Airport, in Meriden; Tweed Airport, in New Haven; and Igor Sikorsky Memorial Airport, in Stratford. Where overlapping recent rainfall data was available from the Waterbury-Oxford Airport, that was evaluated as well to confirm correlation with weather in the Naugatuck Valley.

Data from the Tweed Airport weather station was very similar to the data from the Meriden Markham Airport station, showing 2015-2017 as an extraordinarily dry period, with 2015 and 2016 annual rainfall totals being the two lowest since the year 2000. Annual rainfall data

from the Sikorsky Airport station since 2000 also indicated that 2015-2017 was a relatively dry period. Available NOAA rainfall data from the weather station at Meriden Markham Airport is summarized on Table 3-1.

Table 3-1 Meriden Markham Airport Rainfall Data, 2000-2017

Year	Total Annual Liquid Precipitation (in.)	Peak Day Precipitation (in.)	Date of Peak Precipitation
2000	47.94	2.86	Jul-15
2001	36.21	2.77	Jun-17
2002	46.03	2.27	Aug-29
2003	58.75	3.20	Sep-28
2004	47.36	3.84	Sep-18
2005	(N/A)	(N/A)	(N/A)
2006	58.71	3.23	May-12
2007	45.02	3.03	Apr-15
2008	58.63	3.89	Sep-06
2009	45.39	1.92	Oct-03
2010	43.61	2.88	Dec-12
2011	54.28	2.87	Aug-28
2012	32.10	2.15	Jun-02
2013	37.86	2.97	Jun-07
2014	31.45	1.81	Dec-09
2000-2014 (Avg.)	45.95	2.84	
2015	21.70	1.47	Sep-30
2016	25.66	1.25	Jan-10
2017	36.07	3.25	Oct-29
2015-2017 (Avg.)	27.81	1.99	
2000-2017 (Avg.)	42.75	2.69	
2015-17 Avg. as % of 2000-2017 Avg.	65%	74%	

Observations from this data include:

- 2015-2016 was an extraordinarily dry period: the two driest years by far since 2000, with about half the typical rainfall.
- 2017 rainfall also was well below average for the 2000-2017 period.
- The 2015-2017 period, the three most recent years of record, had only about two-thirds of the average rainfall compared to the overall 2000-2017 period.

In contrast, regional weather reporting indicated particularly intense rainfall periods in 2007 and 2010. For example, CT DEEP identified the storm of April 15-16, 2007 as one of the five worst flooding events to strike the state in the past 100 years, with over 8 inches of rain falling in one 24-hour period in some places (Source: Connecticut's 2010 Natural Hazard Mitigation Plan Update, December 2010). Also, the rain gage at Stevenson Dam, 5 miles WNW of Derby/Ansonia, reported 8.39 inches of rainfall in the month of March 2010. While long-term weather patterns are difficult to predict, it is prudent to consider wastewater flows seen during periods of high rainfall such 2010-2011 as more representative of the high peak flows that could be experienced in the future.

Rainfall data for the first nine months of 2018 was reviewed from four local weather stations, to see if there was any new trend since the end of 2017 that might impact initial conclusions. The data indicates that 2018 may be more typical of the 2000-2014 period, in terms of higher total annual rainfall and higher days of peak rainfall (versus what was seen in the relatively drier period 2015-2017). This confirms the conclusion that the 2015-2017 period should not be considered typical for planning purposes.

The following sections of this Technical Memorandum address each of the five communities, developing the current condition and future (2040) design condition flows and loads.

4.0 DERBY FLOWS AND LOADS

Black & Veatch reviewed available MOR data from the past three years (January 2015 – January 2018). However, as noted previously this flow data was from an extraordinarily dry period. Therefore, wastewater flows from a period with more typical (higher) rainfall also were considered in developing representative baseline flows for Derby.

The 2014 Derby Wastewater Facilities Plan was developed based on flow data from the 2006-2011 period. This included years with higher rainfall, more representative of typical weather conditions seen over the past 20 years.

Approximately 95% of the Town of Derby is served by the wastewater collection system. One small portion of Seymour, along the Route 34/ Roosevelt Drive corridor, contributes flow to the treatment plant. An allocation of 140,000 gpd is reserved for that flow from Seymour. The plant also receives flow from some residences in Ansonia and in the Town of Orange.

The Derby Facilities Plan included future flow projections that assumed an aggressive development program for Derby characterized by significant population growth and several anticipated development projects being constructed. Since the time that the Facilities Plan was prepared, that picture has changed.

Black & Veatch met with Derby representatives (including WPCA chair Jack Walsh, plant supervisor Lindsay King and the mayor's economic development liaison Carmen DiCenso) on July 12, 2018. At that meeting, flow projections and future flows were discussed, in connection with the current local forecast for economic development and population growth. These discussions continued through July into early August, with input from Derby representatives.

Based on additional review by Derby representatives, the forecast of development and annual average flows was updated to reflect the most recent expectation of growth by the City. While Derby still foresees growth occurring over the next 20 years, the overall growth projection has been scaled back from what was forecast at the time of the 2014 Facilities Plan. For example, while a 300-500-unit development is anticipated on land adjacent to the treatment plant, Derby now considers it unlikely that the Hitchcock/ Hines Farm, Opera House, and Halo Projects will be

developed during the next 20 years. Also, if the Fountain Lake Industrial Park is developed, wastewater flow would be directed to the Ansonia WPCF rather than to Derby.

Taking the foregoing into account, Table 4-1 summarizes existing flow information and future projections for Derby. The last column on the right indicates the flows to the Derby treatment plant recommended to be used for the study horizon year 2040 in the NVCOG Regionalization Study. Maximum month, peak day and peak instantaneous flows were calculated from the average annual flow, using ratios developed in the Facilities Plan.

Table 4-1 Derby Wastewater Flows

Flow to Derby WPCF	Existing Condition		Future Condition	
	From 2015-2017 MOR Data	From 2014 Facilities Plan, Table 9-1	From Facilities Plan, Table 10-2 ⁽²⁾ , YR 2032	Revised Projection, YR 2040 ⁽²⁾⁽³⁾
Annual Average (MGD)	1.3	1.61	2.38	1.92
Maximum Month (MGD)	2.18	3.08	3.81	3.07
Peak Day (MGD)	3.59	8.1	9.5	7.7
Peak Hour (MGD)	10.0 ⁽¹⁾	10.0 ⁽¹⁾	12.5	10.0
⁽¹⁾ WPCF influent pump station capacity is currently limited to 10.0 MGD; collection system peak may be greater. Flows in excess of 10 MGD have been reported at the WPCF, and it has been reported that actual peak flows could be as high as 13 MGD.				
⁽²⁾ Based on aggressive I/I program implementation to reduce Peaking Factors as follows: MM/AA from current 1.91 to 1.6; PD/AA from 5.03 to 4.0; and PH/AA from 6.21 to 5.25; as projected in the Facilities Plan.				
⁽³⁾ AA flow based on Derby's revised development forecast.				

While the maximum recorded influent flow to the plant was 10.0 MGD, it is important to note that this reflects the maximum capacity of the treatment plant influent pump station (also 10.0 MGD). Therefore, it is assumed that during peak rain events, flow has been backing up in the collection system. The influent pump station recorded peak flows of 10.0 MGD during three months in 2010, as well as in January 2016 and May 2017.

As stated in the Facilities Plan, most of the wastewater collection system (150,000 LF out of a total 215,432 LF) consists of VC pipe. Prior inspections of limited sections of VC pipe in Derby have indicated unsatisfactory conditions; this may be characteristic of the VC pipe throughout the system. The 2014 Facilities Plan identified significant I/I issues in the collection system that need to be addressed. The City is currently under a Consent Order with DEEP and USEPA. Derby reportedly plans to spend \$270,000/year over the next 15 years to reduce I/I. Therefore, the Facilities Plan projected that the peaking factors for flows will be reduced in the future, as indicated on the projected flows table above. *We recommend that a collection system flow metering program be undertaken to obtain a more accurate estimate of the peak flows to the WPCF.*

Average values for influent wastewater to the plant over the past three years were: 202 mg/L BOD, and 226 mg/L TSS. These are values within the typical range for domestic sewage in an

area without significant industrial contributions. It is assumed that similar average concentrations will be seen in the future and can be used for planning purposes.

5.0 ANSONIA FLOWS AND LOADS

Virtually the entire City (approximately 98%) of Ansonia is served by the wastewater collection system. A small portion of Derby, as well as minor sections of Seymour and Woodbridge flow to the Ansonia collection system. These contributions from the adjacent communities are relatively minor, representing only about 3% of the flow to the Ansonia wastewater treatment plant.

The 2004 Facilities Plan reported that average plant flows were approximately 2.2 MGD (based on 1998-2002 data). That Facilities Plan also projected that average annual flows would increase in linear fashion through 2025, to 3.5 MGD. However, the actual trend since the time the Facilities Plan was prepared has shown flows to the plant decreasing since that time. This trend may reflect national trends where water consumption is decreasing due to residential water conservation resulting modern plumbing fixtures and Codes, combined with lower commercial/ industrial water use. The City Housing Authority demolished a multi-unit public housing project since the time that the Facilities Plan was developed, with no plans to replace that facility. The City plans for moderate adaptive reuse of industrial buildings in the central business district to multi-use residential development. It is also noted that it is Connecticut state policy to support new residential development near rail stations in all towns in this study.

Black & Veatch met with Ansonia WPCA representatives (including WPCA chairman Nunzio Parente, and Superintendent Brian Capozzi) on July 12, 2018 to visit the plant and to discuss flows and loads, and to review draft flows and loads for Ansonia that had been developed by Black & Veatch, in preparation for the meeting. The outcome of this meeting was agreement on the following points:

- a. No major expansion of the wastewater service area is anticipated.
- b. Average annual flow to the plant should be based on the most recent average annual flow data (2015-2017), increased through the year 2040 in proportion to the projected population growth for Ansonia (+6.2%, based on CT State Data Center).
- c. For maximum month, peak day and peak hour flows, similar adjustments should be made to data from the 2009-2010 period, which included greater rainfall and more high-intensity rainfall events.
 1. The peak flow projections for 2040 should also take into account the effect of collection system rehabilitation work performed during the past several years, to reduce I/I.
 2. While difficult to quantify, Black & Veatch will assume that the net effect of a 6.2% population increase offset by recent and future I/I reduction will be a slight net decrease (-5%) in max month and peak day flows, and a net decrease of 10% in peak hour flows.

Table 5-1 summarizes the existing and future flows to the Ansonia wastewater treatment plant. The middle column presents design flows that were provided in the Ansonia-Derby Interconnection

Feasibility Study (April 2014). The column at the far right presenting the proposed 2040 wastewater flows to be used in the NVCOG regionalization study.

Table 5-1 Ansonia Wastewater Flows

Flow to Ansonia WPCF	Existing Condition		Future Condition
	From 2015-2017 MOR Data	From 2014 Interconnection Study (2009-2011 Data)	Projection, YR 2040
Annual Average (MGD)	1.57	1.92	1.7
Maximum Month (MGD)	2.6	4.6	4.4
Peak Day (MGD)	3.9	5.73	5.4
Peak Hour (MGD)	6.91 ⁽¹⁾	10.5	9.5
⁽¹⁾ Ansonia plant staff report that although the influent pumps are designed for 12 MGD, in recent years the plant is limited to about 7 MGD due to hydraulic limitations between the UV channel and outfall.			

As noted in the footnote to the table above, in recent years there has been a hydraulic restriction at the back end of the Ansonia wastewater treatment plant that is limiting peak flow that the plant is able to treat. This is a situation that should be investigated further by Ansonia and corrected as soon as possible, so that the plant will be able to receive flows up to its full capacity during higher wet weather flow events. Also, correcting this problem will maximize the ability of the plant to take additional flow under regionalization alternatives that will be considered in Phase 2 of this study.

Average values for influent wastewater to the plant over the past three years were: 236 mg/L BOD, and 176 mg/L TSS. These are values within the typical range for domestic sewage in an area without significant industrial contributions. It is assumed that similar average concentrations will be seen in the future and can be used for planning purposes.

6.0 SEYMOUR FLOWS AND LOADS

The Nafis and Young Draft Engineering Report on WPCF Phosphorus Planning (May 31, 2016) stated that in addition to serving Seymour, the WPCF also serves parts of Oxford. The total sewered population sending flow to the WPCF is approximately 7,500, according to that report.

The Seymour Plan of Conservation and Development (POCD), dated September 8, 2016 noted that according to inter-municipal agreement, 7% of the total WPCF design capacity is allocated to Oxford. (Note that plant's permitted annual average design flow is 2.93 MGD.)

According to the CT State Data Center population projections, the population of Seymour is forecast to increase by approximately 0.8% (from 16,798 to 16,924) between 2020 and 2030. Thereafter a very slight decline in the local population through 2040.

In the absence of plans showing major expansion of the collection system to serve outlying areas or other significant development that would impact flows, Black & Veatch has assumed that future flows and loads for Seymour will increase in proportion to the projected population

growth forecast. On that basis, flows for planning year 2040 are developed from existing condition information in Table 6-1, and presented in the last column to the right.

Table 6-1 Seymour Wastewater Flows

Flow to Seymour WPCF	Existing Condition			Future Condition
	From 1/2015-2/2018 MOR Data	From 2010 MOR Data	Existing Condition (Nafis & Young) ¹	Projection, YR 2040 ⁽²⁾
Annual Average (MGD)	0.97	1.22	1.3	1.3
Maximum Month (MGD)	1.93	2.73		2.9
Peak Day (MGD)	3.34	6.3		6.7
Peak Hour (MGD)	7.0	7.2	7.3	7.4
¹ From e-mail communication from Nafis & Young to NVCOG, June 2018.				
² Population growth based on CT State Data Center forecast of 0.8% maximum increase. Flows escalated from existing condition AA and PH flows from Nafis & Young. MM and PD flows projected from 1.05 x 2010 MOR data.				

Black & Veatch met with Town of Seymour, WPCA and plant operations representatives (including Annmarie Drugonis, Ben Proto, Jon Livolsi of Seymour and the WPCA, and Walter Royals of Veolia Water) on August 22, 2017 to discuss flows and loads. At this meeting draft flows and loads developed by Black & Veatch in preparation for the meeting, were reviewed and discussed. The outcome of that discussion included the following points:

- Seymour officials confirmed that almost all of the septic tank issues in the Town have been addressed already. Therefore, they do not anticipate any significant increases in flows to the plant resulting from adding customers currently served by onsite disposal systems.
- According to the Plan of Conservation and Development for the Town of Oxford, the only developable land in that town that could be served by Seymour in the future is along the Route 67 corridor. Oxford has an existing agreement under which they have reserved up to 250,000 gpd of capacity at the Seymour WPCF.
- Seymour has been planning to implement an I/I study. To date, Phase 1 of that study, which represents an area of the Town, has been completed. Because this project is still in an early stage, at this time there is no information available to address the potential for I/I removal. However, it is noted that the peak flows seen at the plant (7.0+ MGD) are relatively high relative to annual average flows.

Average values for influent wastewater to the plant over the past three years were: 154 mg/L BOD, and 162 mg/L TSS. These are values on the lower side of the range for domestic sewage in an area without significant industrial contributions, which may reflect infiltration into the collection system. It is assumed that similar average concentrations will be seen in the future and can be used for planning purposes.

7.0 BEACON FALLS FLOWS AND LOADS

The past three years of MOR data for the Town of Beacon Falls water pollution control facility was reviewed. However, as indicated previously, this represented a period of below-average rainfall. Therefore, we have considered existing condition wastewater flow values provided by the 2015 Wastewater Facilities Plan, which were based on a wetter period (September 2009 to October 2012) to be more appropriate to use in this study; since they are more representative of longer-term weather patterns.

Table 7-1 provides a summary of existing flows, as well as the future (2040) flows that Black & Veatch is planning to use in this regionalization study for Beacon Falls.

Table 7-1 Beacon Falls Wastewater Flows

Flow to Beacon Falls WPCF	Existing Condition		Future Condition
	From MOR Data (6/2015-3/2018)	From Facilities Plan ¹	Projection, YR 2040 ²
Annual Average (MGD)	0.31	0.36	0.45
Maximum Month (MGD)	0.48	0.612	0.765
Peak Day (MGD)	0.69	1.01	1.263
Peak Hour (MGD)	1.24	1.22	1.525
¹ From 2015 Wastewater Facilities Plan, based on 9/2009-10/2012 data.			
² From DPC Engineering Memo on Beacon Falls WPCF Upgrades Summary, dated October 17, 2018, as basis of design for the proposed upgrade.			

Black & Veatch met with Beacon Falls WPCA representatives and their consultant DPC Engineering on October 18, 2018 to discuss flows and loads as well as plant upgrade plans. At this meeting Black & Veatch was provided with a copy of a memorandum on Beacon Falls WPCF Upgrades Summary, dated October 17, 2017. That memorandum provided estimated existing condition plant flows from the 2015 Wastewater Facilities Plan, as well as the basis of design flows used by DPC for the proposed WPCF upgrade. The basis of design flows contained in that memorandum, included here on Table 7-1 reflect additional sanitary flows, I/I reduction and anticipated water conservation.

Average values for influent wastewater to the plant over the past three years were: 180 mg/L BOD, and 239 mg/L TSS. The plant upgrade basis of design forecasts that at future average annual design flows, influent BOD will be 211 mg/L and TSS will be 199 mg/L on an average annual basis. The loading conditions used for the plant upgrade basis of design will be used for estimated loadings, for planning purposes.

8.0 NAUGATUCK FLOWS AND LOADS

The Naugatuck WPCF serves approximately 90% of Naugatuck, with the remaining 10 percent of the Borough served by on-site septic systems. The Naugatuck WPCF also receives flow from portions of Middlebury and Oxford. Lesser flows come from residences and developments in Beacon Falls and Prospect. The Naugatuck Wastewater Treatment Facilities Plan,

prepared Kleinfelder in December 2017, did not note septic tank failure issues in the community. Therefore, it has been assumed that the existing collection system will not be adding significant additional wastewater flow from residents currently served by on-site septic systems.

Future flows and loads (for year 2035) were developed in Section 3 of the Wastewater Treatment Facilities Plan. The Facilities Plan assumed minimal growth in Naugatuck through 2035 (net + 284 people, or + 0.9%); but significant growth in Middlebury (+22.4%). This would result in a net population increase within the collection system service area of +5.4%, as shown on Table 3-5 of that Facilities Plan.

The 2010-2015 period used in developing the existing condition flows includes a wetter period than the three most recent years, and therefore should more representative of the longer-term rainfall patterns. Therefore, Black & Veatch believes that the existing condition flows summarized in the Facilities Plan are appropriate to use for planning purposes in the current NVCOG Wastewater Regionalization study.

The SSES study undertaken by Naugatuck recommended three million dollars of I/I removal over a 4-year period, projected to remove 0.3 MGD of infiltration. The Facilities Plan assumed that the rate of I/I reduction would be offset by the rate of I/I increase over time, due to an aging collection system. Therefore, it assumed that flow peaking factors for future flows would remain the same as observed, at 3.9 x average flows, into the future.

Annual average (AA) flows developed by contributing area in Section 3 of the Facilities Plan are summarized below in Table 8-1. The third column presents future flows based on projected population growth in the service area; while the fourth (last) column includes the full flow allocations available for towns of Middlebury and Oxford.

Table 8-1 Naugatuck Wastewater Flows, by Area

Contributing Area	Existing Condition (Facilities Plan, Table 3-4)	Projected, 2035 (Based on Facilities Plan, Sec. 3.4.5)	Projected, 2035 (From Facilities Plan, Table 3- 6, including full allocations for Middlebury & Oxford)
AA Flow, Naugatuck Borough (MGD)	4.54	4.56	4.56
AA Flow, Middlebury (MGD)	0.62	0.78	1.8
AA Flow, Oxford (MGD)	0.083	0.28	1.0
AA Flow, Chentura (MGD)	0.16	0.16	0.16
AA Flow, Beacon Falls (MGD)	negl.	0.04	0.04
AA Flow, Prospect (MGD)	negl.	0.004	0.004
AA Flow, TOTAL (MGD)	5.4	5.82	7.56

The design capacity of the Naugatuck WWTF is 10.3 MGD. Therefore, the amount of additional flow that could be taken at the Naugatuck WWTF resulting from regionalization would depend in part on how the Middlebury and Oxford reserve allocations are addressed.

The Borough of Naugatuck has plans to foster new development in the Borough, within the planning period of this regionalization study, that go beyond what current State Data Center projections in Table 2-1 show. Based on a current population of 32,212 for the Borough of Naugatuck, with approximately 90% served by the wastewater collection system, the current estimated sewered population is 28,990. Based on input from local officials, we have added an allowance for 10% growth in the sewered population. That would add approximately 2,889 more people from Naugatuck Borough to the wastewater collection system by 2040. Assuming the resulting flow contribution is proportional to current flows, this would add an additional +0.45 MGD of average daily flow. Based on this, the reserve capacity available for additional flows resulting from regionalization would be in the range of 2.29-3.95 MGD, as annual average flows, depending on how the current reserve allocations from Oxford and Middlebury are addressed. This is summarized in Table 8-2 below.

Table 8-2 Naugatuck WPCF 2040 AA Capacity Available for Regionalization

Contributing Area	Existing Condition (Facilities Plan, Table 3-4)	Projected (Based on Facilities Plan, Sec. 3.4.5)	Projected (From Facilities Plan, Table 3-6, including full allocations for Middlebury & Oxford)
2035 AA Flow Total w/o Naugatuck Growth (MGD)	5.4	5.82	7.56
2040 AA Flow Total w/o Naugatuck Growth (MGD)		5.90	7.56
10% Naugatuck Growth Allowance (MGD)		0.45	0.45
2040 AA Flows, with 10% Naugatuck Growth (MGD)		6.35	8.01
Plant Capacity (MGD)		10.30	10.30
Maximum Daily Flow ¹	21.9	24.8	31.2
2040 AA Capacity Available for Regionalization Flows (MGD)		3.95	2.29
NOTE: ¹ Future peak flow in 2017 Wastewater Facilities Plan, section 3.4.5, was 29.5 MGD based on future average flow of 7.56 MGD. Future peak flows here use same assumptions for the future: no net change in I/I rate, and no net change in existing flow peaking factor of 3.9.			

APPENDIX B

Technical Memorandum 2: Condition Assessment

REGIONAL WASTEWATER TREATMENT CONSOLIDATION STUDY

Technical Memorandum 2: Condition Assessment

B&V PROJECT NO. 198910

PREPARED FOR

Naugatuck Valley Council of Governments

4 February 2019

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1.0 PURPOSE AND BACKGROUND

The purpose of this technical memorandum is to provide a high-level summary of the condition of existing wastewater treatment and collection system facilities of the five communities participating in this wastewater regionalization study. Information used in this assessment will include review of existing facilities plans and other reports, interviews with knowledgeable wastewater operations and management professionals in the five communities, and site visits by Black & Veatch engineers.

This report will discuss planning-level capital costs to upgrade the five treatment plants and their associated collection systems to meet current regulations, remove excessive I/I, and extend the life of the systems for the 20-year planning study horizon. Where capital costs to upgrade the wastewater facilities are available from previous work performed by the communities, these will also be included. ‘Placeholder’ type estimates will be assigned where capital costs are not available, and where available capital cost projections do not cover the entire planning period, through 2040.

This technical memorandum is intended to establish baseline conditions for wastewater treatment infrastructure in each of the five wastewater treatment communities in the NVCOG Wastewater Regionalization Study. The baseline conditions should reflect the budgetary level capital costs of infrastructure improvements that would need to be made during the planning period (through 2040), with no further regionalization implemented. This includes capital expenditures that would be required to replace aging infrastructure, to meet regulatory requirements, and to accommodate flow increases due to anticipated population increases within the service areas of the five communities.

During Phase 2 of this Wastewater Regionalization Study, the infrastructure needs and related cost projections associated with this Base Case scenario (no regionalization) will be reviewed, analyzed and updated further with more detail and with additional input from the communities. The Base Case for each of the communities then will be compared to the various regionalization alternatives under consideration.

2.0 DERBY WASTEWATER FACILITIES ASSESSMENT

2.1 DESCRIPTION OF EXISTING FACILITIES – DERBY WATER POLLUTION CONTROL FACILITY

The City of Derby water pollution control facility (WPCF), which discharges to the Housatonic River, was built in 1964. The plant was upgraded to secondary treatment in 1973. Limited upgrades undertaken since the 1973 upgrade include: sludge handling facilities (1986); a mechanical upgrade of the influent pump station and replacement of screenings/grinder (1996); electrical upgrade of the influent pumping station (1996); and new aeration system fine bubble diffusers (1997).

The most recent significant construction project (1998) included: electrical upgrades, replacing the main influent pumps, repairs to the grit basins, repairs to the primary settling tanks, modifications to the aeration basins aeration system, mechanical upgrade of the secondary clarifiers, a new bulk storage facility for sodium hypochlorite, and a new sodium bisulfate feed facility.

The WPCF serves approximately 95% of the residents of the City of Derby, plus a small portion of Orange that includes approximately 144 units in Fieldstone Village. The plant is a conventional secondary treatment plant designed for nitrogen removal via a Modified Ludzack-Ettinger (MLE) process configuration. Seasonal disinfection is provided by hypochlorite addition. Since the plant discharges directly to the Housatonic River, there is currently no permit limit for phosphorus (unlike for WPCF's discharging to the Naugatuck River).

Primary sludge and thickened waste activated sludge (WAS) are dewatered onsite, with the sludge cake trucked offsite for further treatment by incineration and ash disposal.

Black & Veatch reviewed the available drawings of the treatment plant, and the most recent Wastewater Facilities Planning Study (Weston & Sampson, March 2014). That facilities plan included an evaluation of the existing wastewater treatment plant and collection system, and developed a capital expenditures plan to address major anticipated upgrades required over a 20-year planning period. That study also looked at regionalization opportunities with other local communities.

Black & Veatch also visited the Derby WPCF on July 12, 2018, accompanied by Derby plant supervisor Lindsay King. A follow-up discussion at the plant site also included Jack Walsh (WPCA chairman) and Carmen DiCenso (the City's economic development liaison).

Peak flow to the plant is limited to approximately 10.0 MGD, based on the capacity of the plant's influent pump station. However, Derby has noted in the past that actual peak flow from the collection system may be up to 13 MGD. (This is documented in minutes of meeting with CT DEP on August 10, 2010, in Appendix H of the Facilities Planning Study.)

Overall, the treatment plant is very old, and in need of a major overhaul, or possibly a near-complete replacement of almost all major systems. The plant is difficult to operate, creating extraordinarily challenging working conditions for plant operations staff and impacting effluent quality.

The existing WPCF process configuration is described in Section 9.4 of the 2014 Wastewater Facilities Planning Study. An evaluation of the condition of each major system of the plant follows,

based on a review of existing reports, observation of the facilities and discussions with WPCA staff.

1. **Influent Trash Racks.** Flow enters the plant via two gravity sewer interceptors. Some screenings are captured with trash racks just upstream of the influent pumps on screens that are located several stories below-grade. The racks are cleaned manually and require manned entry into the inlet structure. The screenings are stored in bins within the inlet structure which are reportedly pulled up to grade when full. The inlet trash rack system is in poor condition in terms of process effectiveness, proper ventilation and safety, and should be corrected as a first priority. The lack of proper screens, grinders and grit removal upstream of the influent pumps results in additional wear and operating challenges for the pumps. A proper headworks should be provided upstream of influent pumping. However, lack of conceptual design makes it difficult to assess the footprint required for a properly functioning preliminary screenings facility. During follow-on study and design this should be investigated, along with the optimal (fine to medium) bar spacing that could be accommodated hydraulically.



Figure 2-1 Derby Trash Racks

2. **Influent Pump Station.** The influent pump station has two pumps with long, problematic shafts (motors on upper level); and one pump with a close-coupled motor. The newer close-coupled pump is the normal duty unit, because of issues with the other two extended shaft units. Due to age and problems at this facility, the influent pump station is in need of a major upgrade, and perhaps a complete replacement. All pumps, piping, valves and controls need

to be replaced at the influent pump station. This facility also needs to be able to handle peak flows from the collection system.



Figure 2-2 Derby Influent Pumps with Extended Shafts

3. **Aerated Grit Chamber.** Downstream of the pump station is a single aerated grit chamber, with no redundancy. Grit is removed from the tank using a clamshell bucket on an overhead monorail. The grit is discharged into an adjacent grit dumpster. The aerated grit tank is partially covered with a steel frame structure with a fiberglass canopy. Certain grating sections were compromised at the top of this structure. This arrangement is ineffective, difficult to operate and a safety concern as well. Overall, the grit system is in poor condition and needs to be completely replaced with an appropriate system that provides at a minimum, capability to bypass the grit removal system when extensive maintenance is required.



Figure 2-3 Derby Grit Facility Overhead Clamshell Hoist

4. **Channel-Mounted Comminutor.** Two channels direct flow from the grit chamber to the primary clarifiers. With a new headworks screening facility, as called for above, the

comminutors would no longer be required and therefore should be removed. This will allow for redundant channels to primary treatment.



Figure 2-4 Derby Comminutor Channel

5. **Primary Clarifiers.** The WPCF has two 90 ft. x 16 ft. x 10.5 ft. side water depth (SWD) rectangular primary settling tanks, which include chain-and-flight sludge collectors and tipping weir scum troughs. Due to the lack of proper headworks facilities, grit tends to collect in these clarifiers and cause operations challenges. It appears the plant has adequate primary clarifier capacity, at least under normal flows. One of the primary clarifiers was down for repair at the time of the site visit; the focus of the repair appeared to be the internal mechanism. Both clarifiers were constructed in 1964, and show some structural cracks due to their age and settlement. Complete replacement of the mechanisms at both clarifiers is recommended. These structures also need to be studied to determine the extent of repairs required.

Two plunger pumps located in the operations building convey primary sludge to storage.



Figure 2-5 Derby Primary Clarifiers

6. **Aeration Basins.** The plant has three basins for activated sludge secondary treatment. Each basin is configured in two passes, each pass being 100 ft. x 20 ft. x 15 ft. SWD.

Basins No. 2 and 3 were modified in the 1998 upgrade to operate in an MLE process configuration for nitrogen removal (with the first two-thirds of the first pass in each train being converted to an anoxic zone). The third basin, Basin No. 1, was left unmodified. Operations staff report that the two modified basins have provided sufficient capacity for plant wastewater flows. Based on issues discussed in the Facilities Plan, the aeration basin diffuser and blower system should be upgraded to improve overall energy efficiency, and for better DO control to optimize nitrogen removal. Additional investigation is required to confirm whether the existing off-line aeration basin needs to be upgraded.



Figure 2-6 Derby MLE Basins

7. **Aeration Blowers.** Process air to the aeration basins is provided from one variable speed positive displacement blower installed during the 1998 upgrade, and by an ABS variable-speed turbo blower purchased by the City in 2010. The newer, high-speed turbo blower is located in the same room with sludge pumps, which raises concern since the sensitive electronic controls of turbo blowers can be impaired by the presence of hydrogen sulfide.

Most of the blower system piping is outdated, and is leaking in several locations. The blower system should be updated at the same time that work in the aeration basins is being done in order to replace the aeration piping and to provide redundant blowers that are energy efficient. The blowers may need to be relocated to another building or in a new building if ventilation at the existing location cannot be positively corrected.



Figure 2-7 Derby Aeration Blowers

8. **Secondary Clarifiers.** Secondary settling is accomplished in two 60 ft. dia. x 10 ft. SWD secondary clarifiers with draft tube type sludge removal mechanisms. With only two clarifiers there is no redundancy if one unit is out of service. Also, the flow split between the two is uneven. New mechanisms and improved internal baffling are recommended for both secondary clarifiers, as well as hydraulic modifications upstream to improve flow splitting upstream of the clarifiers. The operations and performance should be reviewed after these modifications are implemented, to assess whether additional capital improvements will be required. The secondary clarifiers are served by three variable-speed centrifugal RAS pumps, all located in a dry pit.



Figure 2-8 Derby Secondary Clarifier

9. **Disinfection.** The Derby WPCF provides seasonal disinfection (May-September) with sodium hypochlorite, fed via peristaltic metering pumps. There are two parallel chlorine contact basins. The Facilities Planning Study noted that this system has been functioning properly overall, but recommended modifications to improve operational flexibility and to optimize the chemical dose. Dechlorination is accomplished by feeding sodium bisulfite. Since the chlorination system was installed over 20 years ago, plans for its renewal should be included as part of the overall plant upgrade.



Figure 2-9 Derby Chlorine Contact Basins

10. **Sludge Processing and Disposal.** Primary sludge is pumped manually to a sludge holding pit in front of the aerobic digesters. There are two rectangular aerobic digesters with coarse bubble diffusers, built in 1972 and located in a fiberglass enclosure with inadequate ventilation. Waste activated sludge (WAS) is thickened in a rotary drum thickener, located in the secondary control building. Thickened WAS is mixed with the primary sludge, and the mixed sludge is dewatered on a 1.5-meter gravity belt filter press, then trucked offsite by Synagro for further treatment and incineration.

The two circular anaerobic digesters at the plant were built in 1964 and no longer function as digesters, but have been used for sludge storage. There are also two rectangular aerobic digesters at the plant with coarse bubble diffusers, built in 1972 and located in a fiberglass enclosure with inadequate ventilation.

The Facilities Plan noted that the sludge processing equipment is over 30 years old, having served beyond the end of its useful life. That Plan recommended a complete upgrade of the sludge processing system, including rehabilitation of the old digesters and providing new sludge dewatering facilities, including a sludge cake storage area. However, Black & Veatch believes that the size of this plant is too small to justify this level of capital expenditure for solids processing. Average annual flow for 2015-2017 was only 1.3 MGD. For a plant of this size, we believe a more appropriate solution (one we expect will be lower in life cycle cost and easier to operate) would be to store thickened liquid sludge onsite without dewatering, and to haul it offsite in liquid form, in tanker trucks.

The approach we recommend would eliminate the need for: anaerobic digesters, aerobic digesters, sludge dewatering systems, sludge cake conveyance, and sludge cake storage and handling. Instead, all that would be required is WAS thickening, primary sludge thickening, thickened liquid sludge pumping, thickened liquid sludge storage and tanker truck loading facilities. The liquid sludge storage facility would require mixing and the ability to decant.



Figure 2-10 Derby Former Anaerobic Digesters

11. **Electrical System.** Most of the electrical equipment at the plant is over 30 years old. The plant upgrade should consider replacing all major MCC's and power and lighting panels.
12. **Plant Controls and SCADA.** The plant is largely operated in manual mode and does not have a functioning Supervisory Control and Data Acquisition (SCADA) system to monitor and control plant operations. A new SCADA system should be included as part of the plant upgrade.
13. **Odor Control.** Odor control will be an increasingly important issue at the plant, especially in view the plant's proximity to the Downtown area, and anticipated development on a site adjacent to the plant. Odor control facilities must be included with the plant upgrade and be integrated with other systems, particularly the headworks, sludge processing areas and other areas that are sources of odorous air.
14. **General, Site-wide Observations.** In addition to the condition assessment observations made related to specific systems, as noted above, there were also general observations made, related to the overall condition of the Derby WPCF.

Significant concrete spalling and rebar corrosion are noticeable at some of the structures, particularly in the headworks area. Also, there were a number of noticeable safety hazards at the plant. These included: open, unprotected areas above liquid surfaces; solids accumulated in walking areas; and poor ventilation in confined space type areas that had to be entered regularly by plant staff for maintenance (including manually raked bar screens in a lower level space at the headworks). The plant water system is at the end of its useful life and should be replaced with the next major plant upgrade.

The plant site is largely hemmed in with relatively little room to expand, especially with plans for development on adjacent property.

2.2 DESCRIPTION OF EXISTING FACILITIES – DERBY WASTEWATER COLLECTION SYSTEM

The Derby wastewater collection system, which serves approximately 95% of the properties in the City, dates from the late 1800's. The town's collection system is served by two major interceptors: one serving the area on the west side of the Naugatuck River, and the other serving the area on the east side. The subareas are broken out and described further in Section 5 of the Facilities Plan.

According to Derby's Collection System Capacity, Management, Operation & Maintenance (CMOM) Manual (November 2017), the Derby collection system has approximately 218,172 LF of gravity sewer and 6,770 LF of force main. Overall, sewer pipe sizes in the collection system range from 6-inch to 24-inch. The system also includes four inverted siphons.

From the Wastewater Facilities Planning Study, approximately 70% of the gravity sewers in the collection system consists of vitrified clay (VC) pipe. Based on a review of 20 years of television inspection tapes of existing sanitary sewers in Derby done by Weston & Sampson in 2012, representing approximately 45,600 LF of pipe, by far the more serious defects found in the system per foot were in the VC pipes (see Facilities Plan, section 5.1.3).

The 2014 Facilities Plan identified significant infiltration and inflow (I/I) issues in the collection system. The Phase II Sewer System Evaluation Survey (SSES) (April 2016) investigated 11 sewersheds or subareas of the wastewater collection system, utilizing television inspection, smoke testing, dye water testing, flow monitoring and other standard SSES techniques. Significant inflow was found in five of the subareas, and significant infiltration was found in eight of the 11 subareas. Of more than 160,000 LF of pipeline evaluated, approximately 16,000 LF were identified as candidates for cost-effective repair. A total of \$5.4 M in specific improvement projects (2015 dollars) was identified through the Survey, which also recommended additional investigations in the collection system and I/I removal on private property. Derby is committed to an ongoing I/I reduction program, in accordance with an ongoing Clean Water Act Consent Order with DEEP and USEPA (Docket No. CWA-AO-R01-FY16-02). As documented in a letter to CT DEEP on November 22, 2016 related to the Consent Order, the City plans to spend an average of \$270,000/year on I/I reduction over the next 15 years, to comply with the Order.

In 2017, Derby replaced 2,000 LF of sewer mains on Emmet Avenue. Other recent work on the collection system included isolating catch basins with indirect connections to the sewer system and replacing manhole covers. However, much additional work remains to be performed to upgrade the collection system.

A major upgrade/ rehabilitation of the downtown area (Route 34) of Derby is a state-funded project, with construction scheduled to begin in 2019. As part of this program, the roof drains and sump pump systems at 37 buildings in the downtown area will be separated and re-connected to a new storm water drainage system that will be constructed as part of the roadway rehabilitation project.

2.3 DESCRIPTION OF EXISTING FACILITIES – DERBY WASTEWATER PUMPING STATIONS

The Derby wastewater collection system has four pumping stations. These are described in Section 6 of the 2014 Facilities Planning Study, and are:

1. South Division Street PS – The upgrade to this pump station has been completed.
2. Burtville Avenue PS – The upgrade to this pump station has been completed.
3. Roosevelt Drive PS – Replacement of this pump station is under construction, scheduled to be completed in May 2019, at a budgeted cost of \$7.4M.
4. Patty Ann Terrace PS – This pump station, which was noted as deficient in the 2014 Facilities Planning Study, has been recently replaced by a new pump station.

The pumping stations are monitored through two inspections that occur each week. Each station has an alarm, which is transmitted by telemetry system to a pager. To date, these pump stations have not been on a SCADA system. The plan to add a new facility, the East Derby Pump Station, was recommended at the time of the 2014 Facilities Planning Study. However, Derby WPCA no longer considers this project, which was intended to eliminate a problematic siphon under the Naugatuck River, to be necessary. Therefore, following completion of the Roosevelt Drive Pump Station in 2019 there are no planned capital projects related to the wastewater pumping stations.

2.4 CAPITAL PROJECT NEEDS TO 2040 UNDER BASE CASE

This section summarizes the capital upgrades and improvements that would be needed for Derby to meet system needs throughout the planning period (to 2040), without regionalization.

The 2014 Facilities Planning Study developed a recommended plan for capital improvements over a 20-year planning period, summarized in Table 11-1 of that study. Derby WPCA officials updated items on that table related to projected collection system and WPCF capital improvements as part of the referendum passed in 2014.

2.4.1 Capital Projects to 2040 – Derby Water Pollution Control Facility

Based on Black & Veatch's review of the existing facilities at Derby, the following summarizes the improvements that we believe should be made at the WPCF. In view of the age and condition of the existing facilities, we believe that under the base case scenario (no regionalization for Derby), these improvements should be implemented in a single major plant upgrade. That upgrade should include the following components:

1. Replacement of the existing headworks, to provide a reliable medium- or fine-screening facility upstream of the influent pump station.
2. Replacement/ upgrade of the grit removal facility.
3. Complete mechanical and electrical upgrade of the influent pump station, replacing all pumps, motors, valves, piping, controls, etc. A major upgrade to the building housing the pump station also will be required.
4. Replacement of the existing primary clarifier mechanisms, which are beyond their useful life. The concrete tanks also need to be carefully reviewed in light of cracks in these structures, to assess the extent and cost of repairs required.
5. Complete mechanical upgrade of the sludge transfer pumping systems, including primary and secondary sludge pumping, thickened sludge pumping, and primary sludge grinders.

6. Simplify the sludge processing arrangement. Provide thickening for primary sludge and for waste activated sludge; and then store the thickened liquid sludges onsite, to be trucked offsite for dewatering and incineration. This approach would eliminate the need for anaerobic digestion, aerobic digestion, sludge dewatering and sludge cake transfer/ storage facilities onsite. This is a more cost-effective solution for a relatively small plant of this size, and would be simpler to operate and maintain.
7. Upgrade the site-wide electrical system, and provide a full plant SCADA upgrade. This would provide several operational advantages, such as allowing automatic or remote activation to switch to step-feed mode during wet weather events (as opposed to the current situation, which requires local manual switching).
8. An upgrade (as opposed to a total replacement) of the main operations building.
9. A full process upgrade of the secondary treatment system, to optimize performance of the BNR system and to improve energy efficiency. This would include adding additional high efficiency blowers and aeration distribution system, improving segregation and air supply to the blowers, replacing the RAS pumps, and hydraulic modifications to improve flow split to the secondary clarifiers.
10. The secondary clarifier mechanisms and internal baffles need to be replaced. Surface loading rates are high at current and future peak day and peak hour hydraulic loading rates, and the relatively shallow depths of the clarifiers (10 ft SWD) do not provide a great deal of operating cushion to protect the sludge blanket from being scoured during peak flows. It may be possible to mitigate this without adding a third clarifier by implementing other modifications, for example by adding sludge blanket baffles within the clarifiers. This will need to be confirmed with additional study of the clarifiers.
11. The plant water system should be replaced.
12. Other plant systems including disinfection, dechlorination and odor control, should be upgraded.
13. We do not see a justification for implementing a membrane-based treatment system in the future, as was suggested for a future Phase 3 Upgrade package, in the Facilities Planning Study. For this size facility, with the effluent limitations anticipated for the future, we believe the best long-term plan will be to stay with an activated sludge BNR-type system with conventional clarifiers. This will also be easier to operate and will have lower O&M costs compared to a membrane-based treatment system.

2.4.2 Alternative Sludge Processing Approach

The strategy of eliminating sludge dewatering, as proposed above, could include modifying existing tankage or installing two new steel storage tanks: one for thickened primary sludge (TPS) and one for thickened waste activated sludge (TWAS). Plant personnel would pump the thickened sludges to the storage tanks daily; then the thickened sludges would be transferred to tanker trucks for hauling to the offsite merchant facility.

Based on rough estimates of sludge produced at the Derby plant, it appears that two 40,000-gallon steel tanks, one for TPS and TWAS storage, would suffice. The tanks should provide for several days' worth of thickened sludge storage in the event of an interruption in the hauling schedule.

If required, the temporary sludge storage tanks could be silo-type with conical bottoms to minimize concerns with sludge settling out. They should also be covered to minimize the release of any odors that are produced during storage. Any new tanks required would be anchored to new concrete pads, and could be located near the anaerobic digester tanks; however, other locations could be made to work as well.

Storage tanks for the thickened sludges (TPS and TWAS) could be fed through new connections to the existing buried sludge lines. Sludge loading pumps would be required to transfer one truck's worth of sludge (6,500 gallons). These truck loading pumps would withdraw sludge through a connection at the bottom of the storage tanks.

Due to the raw nature of the stored sludges, odors associated with hydrogen sulfide formation may be produced, particularly in the TPS storage tank. To minimize these odors, ferrous chloride could be metered into the two thickened sludge streams ahead of the storage tanks. Odorous off-gases in the air spaces above the sludge liquid in the storage tanks could be treated by an activated carbon odor control system. A similar activated carbon system would be used to treat off-gases that are produced as trucks are filled.

2.4.3 Capital Projects to 2040 – Derby Wastewater Collection System

The following projects are scheduled for construction in 2019:

1. Route 34 gravity sewer replacement;
2. Hawthorne Avenue sewer lining and replacement; and
3. Force main extension and replacement, associated with Roosevelt Drive Pumping Station improvements.

The following projects that were included in Table 11-1 of the Facilities Planning Study have been deleted from the capital improvements program:

1. McConney Grove sewer system extension; and
2. Various planned development projects, including: Commerce Street/ Business Park, Hitchcock/ Hines, Derby Business Revitalization, HALO Project, and Derby Sterling Opera House.

As noted in prior reports, collection system peak flows can reach up to 13 MGD. While some work has been undertaken in the collection system, additional work is required to provide a reliable system. Investigations and prioritization is needed to maximize reliability and benefit.

2.4.4 Capital Projects to 2040 – Derby Wastewater Pumping Stations

The following projects that were included in Table 11-1 of the Facilities Planning Study have been completed (as of October 1, 2018):

1. South Division Street Pumping Station improvements;

2. Burtville Avenue Pumping Station improvements; and
3. Patty Ann Terrace Pumping Station improvements.

The Roosevelt Drive Pumping Station improvements project, which was included in Table 11-1 of the Facilities Planning Study, is scheduled for construction in 2019. Therefore, the only pumping station project included in the Facilities Planning Study that is yet to be constructed is the proposed new Division Street Pump Station.

2.5 PROJECTED CAPITAL AND O&M EXPENDITURES – 2040 BASE CASE

Projected expenditures for the WPCF, the wastewater collection system, and the wastewater pumping stations have been addressed as part of this early planning study. Budgetary capital and operating costs associated with the base case scenario for Derby outlined in this section are provided in Appendix A of this report. Since no engineering design has been undertaken for these proposed upgrades, the costs provided in that appendix are for higher-level budgeting purposes only, and have been based on typical parametric considerations, i.e. dollars-per-gallon, taking into consideration the size and age of the facility as well as the overall constraints of the site. Operations and maintenance costs have been based on current operating cost information provided by the City.

3.0 ANSONIA WASTEWATER FACILITIES ASSESSMENT

3.1 DESCRIPTION OF EXISTING FACILITIES – ANSONIA WATER POLLUTION CONTROL FACILITY

The Ansonia Water Pollution Control Facility (WPCF) was constructed as a primary treatment plant in 1968, and upgraded to secondary treatment in 1970. An extensive upgrade to the WPCF was completed in 2011. The WPCF serves approximately 98% of the residents of the City of Ansonia, a small portion of Derby, and minor sections of Seymour and Woodbridge. The plant is a secondary treatment plant in a four-stage Bardenpho process configuration for nitrogen removal, with oxidation ditch (carousel) aeration, and UV disinfection. The plant process also provides for seasonal phosphorus removal, to meet effluent requirements for discharge to the Naugatuck River.

As part of the condition assessment of existing facilities, Black & Veatch reviewed the Preliminary Design Report (October 2006) and the design plans for the plant upgrade. Black & Veatch also visited the WPCF on July 12, 2018 accompanied by plant superintendent Brian Capozzi. An assessment of each major system of the plant follows, based on a review of existing reports, observation of the facilities and discussions with WPCA staff.

1. **Mechanical screening.** The plant has only one mechanical bar screen, which was installed as part of the 2011 plant upgrade, along with the associated screenings process equipment. This is upstream of the influent pump station. There is also a second (manual) bar screen located at the lower level, which is more difficult to access.



Figure 3-1 Ansonia Mechanical Bar Screen

2. **Influent Pump Station.** The plant's influent pumping station has two smaller and two larger centrifugal pumps in a dry pit. All four pumps are new from the 2011 plant upgrade.



Figure 3-2 Ansonia Influent Pumps

3. **Vortex Grit Separation.** The plant has a new covered vortex grit chamber and grit system, also from the 2011 plant upgrade. There are provisions to bypass flow around the grit chamber when maintenance is required.
4. **Primary Clarifiers.** The chains and flights in the existing primary clarifiers were replaced during the 2011 upgrade. The clarifiers were full at the time of the visit, but appear to be in satisfactory condition based on staff input.



Figure 3-3 Ansonia Primary Clarifiers

5. **Primary Sludge Pumps.** The primary sludge pumps are air-driven diaphragm pumps, in a 4+1 arrangement. Pumps are FLSmidth slurry pumps, which are unusual in this type of application; those pumps are typically found in mineral slurry applications in the mining

and minerals industries, rather than for domestic wastewater sludge. It was reported that these are high-maintenance items, and that the ball checks need to be replaced relatively frequently. Without a high amount of maintenance for this system, these pumps would be unreliable. Ansonia is considering replacing these pumps with a pump more commonly used in primary sludge pumping applications.



Figure 3-4 Ansonia Sludge Pumps

6. **BNR Secondary Treatment.** The secondary treatment system features 2-stage anoxic zones, as well as first and second stage aeration. The old aeration basins were modified to become first stage anoxic zones. There appears to be some structural damage showing at these older tanks, including some cracks at the top of the walls.

The first stage of aeration is accomplished by two oxidation ditch (carousel or racetrack type) aeration basins operated in parallel, which were installed during the 2011 upgrade. Orientation of one of the ditches appears to be backwards relative to what it should be, and as a result there may be some short-circuiting. Since the plant is operating below its design capacity, this does not appear to be a problem at this time. However, it could become an issue if plant flows increase to the point where they approach the plant's design capacity.

Former rectangular secondary clarifiers were modified to become second stage aeration and second stage anoxic basins. New blowers and diffusers were installed for the second stage aeration system.



Figure 3-5 Ansonia Converted Anoxic Basins



Figure 3-6 Ansonia Oxidation Ditch Aeration Basin

7. **Secondary Clarifiers.** New circular secondary clarifiers were installed during the 2011 plant upgrade, along with new RAS and WAS pumping systems. Ansonia is adding alum ahead of the secondary clarifiers for phosphorus removal. Ansonia operations staff report that the phosphorus removal system is working well, and they have been meeting permit requirements for effluent phosphorus.



Figure 3-7 Ansonia Secondary Clarifier



Figure 3-8 Ansonia RAS and WAS Pumps

8. **UV Disinfection.** A new UV disinfection system was added to replace the chlorine contact tanks. Although the plant has only a single UV channel, some level of redundancy is provided since there is more than one bank of UV lamps in that channel.
9. **Effluent Pump Station.** The effluent pumping station, which is adjacent to the influent pump station, has two pumps in a duty/standby arrangement. The influent and effluent pumping stations are both designed for peak flows of up to approximately 12 MGD. However, according to plant staff the flow to the effluent pump station is limited to approximately 7 MGD. The cause of this limitation has not been fully investigated. However,

initial observations suggest there may be a hydraulic constriction in the conveyance system feeding into the effluent pump station wet well. As a result, the plant cannot handle peak flows greater than 7 MGD.

This is a problem that deserves immediate attention, and needs to be corrected as soon as possible, as historic peak flows to the plant as high as 10 MGD have been recorded. The current situation not only limits the plant's ability to handle peak flows from Ansonia, but also limits the facility's ability to receive wastewater flows from other communities as part of this regionalization study.

10. **Alkalinity Supplementation System.** The Merrick silo soda ash feed system was not being used at the time of the site visit, because the treatment process has not been requiring supplemental alkalinity. Plant operations staff noted that the layout of the pump system makes maintenance of this system very challenging.
11. **Thickened Sludge Storage.** WAS is thickened using rotary drum thickening. Thickened WAS is stored in one of two sludge holding tanks (two converted anaerobic digesters). Primary sludge from the primary clarifiers is pumped to the other storage tank. The sludge from these tanks is hauled away via tanker trucks to offsite incineration.

The sludge storage tanks do not have decanting ports. The City reports that having the ability to decant from the storage tanks would reduce the amount of water hauled off by the tanker trucks, thereby extending storage capability and reducing hauling costs.



Figure 3-9 Ansonia Former Digesters Used for Sludge Storage

12. **Overall Observations.** In general, the plant infrastructure appeared to be in good condition, since most of the mechanical systems and some of the basins had been replaced or overhauled as part of the major upgrade to the plant in 2011. Also, MOR effluent data

indicate very good, consistent treatment plant performance. Record effluent BOD and TSS is consistently in single digits, and the WPCF is meeting nitrogen and seasonal phosphorus removal requirements.

3.2 DESCRIPTION OF EXISTING FACILITIES – ANSONIA WASTEWATER COLLECTION SYSTEM

This assessment of the condition and needs of the Ansonia wastewater collection system is based on information contained in the 2004 Facilities Planning Study and discussion with WPCA staff.

The Ansonia collection system includes approximately 345,000 LF of sewers and includes three major interceptors: Two-Mile Brook interceptor, interceptors along the Naugatuck River, and an inverted siphon under the river. Much of the pipe is old, including vitrified clay (VC) pipe. Ansonia has undertaken several I/I reduction projects in recent years. However, while progress has been made, the collection system has I/I issues that contribute to high peak wet weather flows to the WPCF as noted in TM No. 1 – Flows and Loads.

As part of the 2004 Facilities Planning Study, televised inspections were performed for a significant number of pipes in the system. The videotapes of these inspections are still available, but summaries of the data were not developed, and some of the recommended improvements based on these videos were implemented. No further televised sewer inspections or other sewer system evaluation surveys (SSES) have been conducted since the 2004 study and associated construction activities.

Improvements recommended in the 2004 Facilities Planning Study were bundled into engineering and rehabilitation projects in May 2006. Two contracts were developed. The original design contract was \$891,000 to cover identified improvements including inflow control, point repairs, manhole cover replacements, etc. Insituform was selected for the second contract, which focused on pipe lining and other rehabilitation efforts, with a contract cost of \$2,934,000. The length of pipe and number of manholes rehabilitated in this project is not readily known. It was reported that because the pre-construction televised inspections required additional repairs, the lining project addressed approximately 60% of the recommended improvements identified in the 2004 Facilities Planning Study. No additional rehabilitation work has been performed on the collection system since the completion of these projects.

3.3 DESCRIPTION OF EXISTING FACILITIES – ANSONIA WASTEWATER PUMPING STATIONS

The City of Ansonia collection system has 14 wastewater pumping stations. Four of these are small “can” type stations that serve just a few homes; Ansonia hopes to eliminate up to three of these small stations by going with gravity systems instead. Of the remaining 10 larger stations:

- six have been upgraded within the past six years;
- the two largest two stations (Coe Pump Station, and Bartholomew or “Bart” Pump Station) were completely upgraded within the past ten years, including with new generators; and
- the other two stations were upgraded 6-10 years ago.

The WPCA staff is responsible for pumping station maintenance.

3.4 CAPITAL PROJECT NEEDS TO 2040 UNDER BASE CASE

This section summarizes the estimated capital improvements that would be needed for Ansonia to meet system needs throughout the planning period (to 2040), without regionalization

3.4.1 Capital Projects to 2040 – Ansonia Water Pollution Control Facility

Ansonia completed an extensive upgrade to the WPCF in 2011, and is consistently meeting permit requirements for all parameters, including nitrogen and seasonal phosphorus removal. The plant is overall in satisfactory operating condition. It is operating well under its design capacity, and is not projected to exceed that within the 20-year planning period. While the hydraulic restriction at the effluent pump station is a deficiency identified above that needs to be addressed immediately, at this point it has not yet been determined what the cause of that problem is, nor what capital expenditures would be necessary to correct it.

Otherwise, no additional major capital needs are foreseen in the near future as being required at the WPCF under the base case scenario (if no regionalization for Ansonia). Based on a 20 to 25-year average life for major mechanical systems that are well-maintained, and barring unforeseen changes in discharge requirements, the next major upgrade should be to replace recently-installed mechanical equipment when it comes to the end of its useful life. That would put the next significant mechanical upgrade cycle for the Ansonia WPCF in the 2031-2036 timeframe, near the end of the current planning period of this study.

3.4.2 Capital Projects to 2040 – Ansonia Wastewater Collection System

Increased investment in the collection system is needed to maintain appropriate service levels and meet regulatory requirements. A common industry approach in high-level analysis is to reference the estimated useful life of assets and estimate investment levels based on that useful life. For example, a 100-year useful life would require replacing an average of 1% of the system each year.

Initial activities would be focused on identified hot spots that have more frequent backups. The objective would be to focus on problematic areas and address them. The north end of downtown is believed to be the most problematic area at this time.

3.4.3 Capital Projects to 2040 – Ansonia Wastewater Pumping Stations

All ten of the larger pumping stations in the system have been upgraded within the past 10 years, and there are no plans to upgrade any of these stations in the near future nor to add new pumping stations. All of the pump stations in the system may be due for a major mechanical upgrade in 10-15 years. In the interim, it appears that the only capital expenditures foreseen related to the pumping stations would be for periodic upgrades and replacement of mechanical equipment and components that is typical for these types of facilities.

3.5 PROJECTED CAPITAL AND O&M EXPENDITURES – 2040 BASE CASE

Projected expenditures for the WPCF, the wastewater collection system, and the wastewater pumping stations have been addressed as part of this early preliminary study. Budgetary high-level capital and O&M costs associated with the work described in this section are provided in Appendix A of this report. In the absence of engineering estimates for specific capital projects, the cost information in that appendix represents high-level budgetary costs based on typical parametric

values such as dollars-per-gallon of treatment. Operations and maintenance costs have been developed from current operating cost information provided by the City.

4.0 SEYMOUR WASTEWATER FACILITIES ASSESSMENT

4.1 DESCRIPTION OF EXISTING FACILITIES – SEYMOUR WATER POLLUTION CONTROL FACILITY

The Seymour WPCF was built in the 1970s, with a significant upgrade implemented in the early 1990s. It serves a sewered population of approximately 7,500 that includes the Town of Seymour plus a small portion of Oxford. The Seymour WPCF is a secondary treatment plant in a Modified Ludzak Ettinger (MLE) process configuration, followed by chlorination/dechlorination disinfection and cascade reaeration prior to discharge to the Naugatuck River. Recently the plant has begun to provide enhanced seasonal phosphorus removal, via chemical addition.

The plant, which currently is operated by Veolia Water, is designed and permitted to treat 2.93 MGD on an average annual basis. However, in recent years (2015-2017) the average flow to the plant has been approximately 0.97 MGD.

As part of the condition assessment of existing facilities, Black & Veatch reviewed available documents, which included the May 2016 WPCF Phosphorus Planning draft engineering report, and some of the 1991 upgrade design drawings which were made available. Black & Veatch also visited the WPCF on August 22, 2018 accompanied by Veolia Water plant manager Walter Royals. An assessment of the major plant facilities follows based on a review of available documents, observation of the facilities and discussions with Veolia Water and WPCA staff.

1. **Influent Screening.** After Parshall flume flow measurement, the incoming wastewater flows through a coarse manual bar rake with 1.5-inch spacing. The flow then travels through a single mechanical bar screen located in a three feet wide channel. Bar spacing on the screen is 0.75 inches. A bypass channel allows for uninterrupted flow-through during times when the mechanical screen is down for maintenance. Captured screenings are lifted by a bucket elevator system to a dumpster at grade. The mechanical bar screen equipment and screenings handling system dates to the 1990s plant upgrade, and needs to be replaced with new equipment. This entire facility is located outdoors which makes operations and maintenance difficult, especially during the cold seasons.



Figure 4-1 Seymour Headworks Area

2. **Grit Removal.** A single rectangular aerated grit chamber receives the flow after screening. The tank is equipped with a submerged auger and bucket elevator for removal of grit. The grit is discharged into a classifier system prior to being conveyed into a dumpster. A bypass channel allows for flow to continue to pass through the plant during times when the grit chamber is down for service. The grit chamber facility and equipment was last upgraded in the early 1990s, and needs to be replaced with new equipment.
3. **Influent Pump Station.** The influent pump station is set up in a wetwell/drywell arrangement, and has three pumping units. The pumps are located at the lower level, with motors on the upper level connected by extended shafts. Each pump is rated at 2,700 gpm and the facility is reportedly rated at 5,000 gpm with two pumps operating and the third pump is a standby unit. All pumps are operated with variable frequency drives (VFDs). The pump station equipment was installed in the 1990s upgrade; however, some modifications have been made to the pumps since that time. Based on age of the equipment, this facility needs to undergo a major overhaul in the near future.
4. **Primary Settling Tanks.** The plant has four rectangular primary settling tanks. Two of the tanks date back to the original construction of the 1970s; the other two were constructed as part of the upgrade done in the early 1990s. The tanks include longitudinal sludge collectors with surface scum skimming. Effluent from the primary settling tanks flows to the secondary treatment influent box. Scum collected from the primary settling tanks is discharged to the primary scum reactor. The mechanisms on two of the four primary settling tanks require replacement. Metal within the tanks will require either replacement or sand blasting and recoating, depending on actual condition.



Figure 4-2 Seymour Primary Clarifiers

5. **Aeration Basins.** The biological treatment facilities include three rectangular extended aeration activated sludge basins. The basins are in an MLE configuration, to provide nitrification-denitrification. The middle basin, which serves as the anoxic zone, receives the primary effluent. Three submersible mixers keep this basin gently stirred. The outer basins are equipped with grid type fine bubble diffusers. The aerated basins are equipped with

effluent weirs which allow mixed liquor flow to the final clarifiers. Based on the age of the equipment, it is recommended that the aeration diffuser system in the aeration basins be programmed for replacement.



Figure 4-3 Seymour MLE Basin

6. **Aeration Blowers.** The aeration blowers include a newer magnetic bearing turbo blower unit which serves as the duty unit and two older style multistage centrifugal blowers. It is recommended that a new turbo blower be provided to match the operating conditions of the existing turbo blower. This will provide for more efficient operations and reliable back-up to the existing turbo blower. The older multi-stage blowers should be decommissioned. It is noted that the blowers are located within the same general space as sludge pumps. Turbo blowers have sensitive electronics that make them vulnerable to harsh environments, including sludge gasses that are prevalent at a wastewater treatment plant. The ventilation in the blower room space needs to be reviewed and modified accordingly such that the air supply to the blowers, including the space they occupy, is noncorrosive and conducive to their overall reliability. Relocating the blowers may be required if the ventilation system problem is not corrected.



Figure 4-4 Seymour Blower

7. **Final Clarifiers.** The two final clarifiers are 65-foot diameter, with 13-foot side water depth. The clarifiers are equipped with rotating suction type sludge collectors. The clarifiers were part of the early 1990s upgrade. Due to their age, the internal mechanisms in these tanks need to be replaced with new equipment. A detailed condition assessment of this equipment may show that sand blasting and recoating of all metal parts could be done as part of the upgrade.



Figure 4-5 Seymour Final Clarifier

8. **Phosphorous Removal System.** The Town recently installed and commissioned a chemical phosphorous removal system at the plant. The system utilizes alum, which is introduced into the process at the aeration basins effluent.



Figure 4-6 Seymour Alum Feed System for P Removal

9. **Disinfection.** Sodium hypochlorite is used for disinfection, with sodium bisulfite added post-disinfection, for dechlorination. From conversations with plant staff, it appears that the disinfection and dechlorination systems, including the chlorine contact tanks, are in satisfactory condition at this time.

10. **Sludge Processing and Treatment.** Sludge from the primary settling tanks is mixed with primary scum and pumped to a rotating drum thickener (RDT) for thickening. The waste activated sludge is co-thickened with the primary sludge at the RDT. The thickened combined sludge is discharged to a sludge holding tank located below grade. The thickened sludge is pumped to the belt filter press (BFP) for dewatering. The dewatered sludge cake discharges into a truck which hauls the material off-site to a merchant incineration facility for final treatment.

The RDT has reached the end of its useful life and needs to be replaced. The BFP has also reached the end of its useful life. From experience gained from working at other small plants, we believe that sludge processing should end with thickening at Seymour. The thickened sludge would then be hauled off-site for additional treatment at the merchant incineration facility. The two sludges should also be handled separately, and not combined; this is to minimize release of odorous compounds and to minimize corrosion of steel and concrete. This should be investigated further in lieu of proceeding with new sludge dewatering equipment.



Figure 4-7 Seymour Rotary Drum Thickener

11. **Electrical System.** The majority of the electrical panels and motor control centers at the plant are approximately 25 years old, dating back to the early 1990s upgrade. Some units are even older, from the 1970s project. This equipment is either at its end of usefulness or fast approaching its life expectancy. We recommend that all MCCs and electrical panels from prior to the 1990s upgrade be replaced with new gear. The electrical power and lighting panels and MCCs from the 1990s upgrade should be carefully evaluated and replaced as needed. This gear can be expected to reach the end of its useful life by 25 or 30 years after being put in service; on that basis Seymour should program for its replacements soon.
12. **SCADA.** The WPCF is operated for the most part in manual mode. It is manned one shift per day, five days per week, with alarms during off-hours going to operator phones. The Town should implement a new SCADA system at the facility. A SCADA system will provide for effective monitoring and also for automatic control. The SCADA system can be programmed to operate the plant with various degrees of automation. A new SCADA system would

improve reliability in operations and maintenance, and also would enhance accountability in O&M and in overall treatment performance.

13. **Valves & Gates.** The WPCF has numerous slide gates, sluice gates and valves. The sludge processing systems in particular have numerous valves critical to the operation of these systems. A close inventory and condition assessment of all slide and sluice gates and valves throughout the plant should be undertaken and these critical components should be replaced as needed.
14. **Odor Control System.** The existing biofilter, which draws odorous air from the sludge thickening and dewatering areas, does not work and needs to be replaced.

4.2 DESCRIPTION OF EXISTING FACILITIES – SEYMOUR WASTEWATER COLLECTION SYSTEM

This assessment of the condition and needs of the Seymour wastewater collection system is based on limited information provided by Nafis & Young Engineers, including information contained in GIS files. According to the GIS, the collection system comprises approximately 63 miles of pipe, with the primary materials being PVC (39%), asbestos cement (34%), and vitrified clay (23%).

No prior plans, condition assessments, or I/I investigation data were available. Prior conversations with WPCA representatives and consultants indicate that the sewer collection system has received limited capital investment over the years. There are no known engineering reports nor investigations available on the condition of the collection system. No information about prior repair history or collection system investments were available.

However, earlier this year Seymour started an initial flow monitoring plan on a section of the collection system. The initial focus of the I/I monitoring program is taking place in an older area that has more problems. Clay pipe is a major problem in the older parts of the system, due to structural integrity and I/I issues. The outlying areas of the town that were developed more recently tend to have newer, PVC pipes.

The WPCA staff is responsible for maintaining the wastewater collection system. At this time, Seymour has no annual sewer replacement program.

As noted in TM No. 1 – Flows and Loads, the collection system has significant infiltration and inflow (I/I) issues that cause high peak wet weather flows to the WPCF. Seymour had the second highest peaking factor of the towns in the study, indicating that the wastewater collection system may be in poor and deteriorating condition, and may have direct inflow connections as well.

4.3 DESCRIPTION OF EXISTING FACILITIES – SEYMOUR WASTEWATER PUMPING STATIONS

The two largest pump stations in the collection system are both located on Derby Avenue: the South Derby Pump Station and the North Derby Pump Station. It appears these two pump stations received significant upgrades approximately 10 years ago. There are also eight smaller pump stations, at least six of which are new stations with submersible pumps. The Seymour WPCA staff is responsible for maintaining the pumping stations. No upgrade and maintenance records were provided on these smaller pump stations.

4.4 CAPITAL PROJECT NEEDS TO 2040 UNDER BASE CASE

This section summarizes the estimated capital facilities that would be required for Seymour to meet system needs throughout the planning period (to 2040), without regionalization. It addresses the WPCF, the wastewater collection system, and the wastewater pumping stations.

4.4.1 Capital Projects to 2040 – Seymour Water Pollution Control Facility

Based on Black & Veatch's review of the existing facilities at Seymour, the following summarizes the improvements that we believe should be made at the WPCF. In view of the age and condition of the existing facilities, we believe that under the base case scenario (no regionalization for Seymour), these improvements should be implemented in a single major plant upgrade. That upgrade should include the following components:

1. Replacement of the existing screenings facility at the headworks, to provide a reliable medium- or fine-screening facility. This would include new mechanical screening equipment as well as associated screenings processing and conveyance systems. These systems should be enclosed.
2. Replacement of the grit removal facility.
3. Complete mechanical and electrical upgrade of the influent pump station, which would include replacing all pumps, motors, valves, piping, drives, controls, etc.
4. Replacement of the mechanisms in two of the four primary clarifiers. The concrete tanks also need to be carefully reviewed in light of their age to assess the extent and cost of repairs required.
5. Replacement of the aeration diffusers in the aeration basins, and other related modifications as needed to optimize BNR system performance and to improve energy efficiency. This would include replacing the older multi-stage blowers with a new turbo blower suitable to operate in concert with the existing turbo blower. Either fix the HVAC issues in the blower area, or consider relocating the blowers into another existing or new building as necessary to maintain an appropriate operating environment.
6. Replacement of the mechanisms on both circular secondary clarifiers.
7. Replacement of the rotary drum thickener (RDT), with similar equipment or other appropriate waste active sludge thickening systems.
8. Decommission the belt filter press (BFP). Instead of dewatering sludge onsite, provide thickened liquid storage onsite, with decanting capability, for trucking liquid sludge offsite for further processing and incineration. For a plant this size, this will be more cost-effective in the long term than dewatering onsite.
9. Upgrade of the site-wide electrical system, which would include replacing all MCC's as well as all of the older electrical panels, including power and lighting panels. Power cables should also be considered for replacement.
10. Provide a full plant SCADA system upgrade.

11. Review the condition of gates and valves throughout the plant, and replace those that are not functioning or which are at the end of their useful life.

4.4.2 Capital Projects to 2040 – Seymour Wastewater Collection System

No capital planning information was provided relating to planned investment in the collection system. It is assumed that increased investment in the collection system is needed to maintain appropriate service levels and meet regulatory requirements. A common industry approach in high-level analysis is to reference the estimated useful life of assets and estimate investment levels based on that useful life.

Because of the high I/I rate and the lack of prior investigation, it is assumed that 1.5% of the system will require replacement or rehabilitation per year to maintain the system. This corresponds to approximately 5,000 ft of pipe per year. This level of investment may not have a significant impact on reducing I/I in the system. It also appears that capital improvements for an initial period of time is also necessary to increase overall system reliability.

Some initial activities would be focused on identified hot spots that have more frequent backups. The objective would be to focus on problematic areas and address them.

4.4.3 Capital Projects to 2040 – Seymour Pumping Stations

There are two larger and eight small pump stations in the collection system. Based on age and condition, we would anticipate a major mechanical upgrade for the larger stations in 10-15 years. The smaller pump stations need to be investigated to determine investment requirements and timing. Lacking this information, it is assumed that these smaller pump stations require upgrade in the next 5 to 10 years.

4.5 PROJECTED CAPITAL AND O&M EXPENDITURES – 2040 BASE CASE

Projected expenditures for the WPCF, the wastewater collection system, and the wastewater pumping stations have been addressed as part of this early planning study. Budgetary capital and operating costs associated with the base case scenario for Seymour outlined in this section are provided in Appendix A of this report. Since no engineering design nor assessment work has been undertaken for the Town's wastewater treatment, collection and pump station infrastructure, the costs provided in that appendix are for higher-level budgeting purposes only, and have been based on typical parametric considerations such as dollars-per-gallon, taking into consideration the size and age of the facility as well as the other factors, such as plant site constraints.

5.0 BEACON FALLS WASTEWATER FACILITIES ASSESSMENT

5.1 DESCRIPTION OF EXISTING FACILITIES – BEACON FALLS WATER POLLUTION CONTROL FACILITY

The Beacon Falls WPCF is a small facility, with a permitted design flow of 0.71 MGD and recent annual average flow (2015-2017) of approximately 0.31 MGD. The 2015 Wastewater Facilities Plan estimated an average annual flow of 0.36 MGD and a peak day flow of 1.01 MGD. The plant solely serves residents of the Town of Beacon Falls, while other residents in town are served by septic systems and some wastewater flow is sent to the Naugatuck WPCF. The Beacon Falls WPCF was built in 1971 as a secondary treatment plant with primary clarifiers, activated sludge, secondary clarifiers and anaerobic sludge digestion (now used as a sludge holding tank).

The plant, which is subject to nitrogen limits, has been a net payer into the Long Island Sound nitrogen credits exchange program. The plant currently discharges approximately 50 pounds/day of nitrogen; the Town paid approximately \$16,000 into the nitrogen credits exchange program in 2017.

The most recent major upgrade to the WPCF was done in 1994, and included: new aeration blowers and diffusers, septage receiving station, sludge pumps, a new (12-foot side water depth) final settling tank, and modifications to the existing (8-foot side water depth) final settling tank. A UV disinfection system was added at the WPCF in 2006. Since much of the mechanical equipment is approaching 25 years in service and clearly nearing the end of its useful life, the plant is due for a major mechanical upgrade.

Since the plant is not at the southernmost (downstream) end of the collection system service area, most of the wastewater flow must be pumped to the plant.

Following a study by an engineering consulting firm which recommended an extensive upgrade to the WPCF, Beacon Falls retained DPC Engineering to develop a more streamlined plan for upgrading the facility. The projected capital cost for upgrading the Beacon Falls WPCF, included in the appendix to this report, is based on information provided by DPC Engineering.

Black & Veatch met with Beacon Falls WPCA members and operations staff at the Beacon Falls WPCF on August 22, 2018, and were given a tour of the facility at that time. The following summarizes observations made regarding condition of the existing facilities.

1. **Influent Pump Station.** The plant influent pump station features three constant speed centrifugal pumps in a wet pit/dry pit configuration. Based on the age and condition of the equipment, it appears that this pump station is structurally sound overall, but is due for a mechanical upgrade.



Figure 5-1 Beacon Falls Influent Pump Station

2. **Headworks.** The headworks, which is located downstream of the influent pump station, features a comminutor in parallel with a manually cleaned bar screen in the bypass channel. The equipment is at the end of its useful life and should be replaced.



Figure 5-2 Beacon Falls Comminutor and Bar Screen

3. **Primary Settling Tank.** There is only one primary settling tank at the plant, a rectangular basin mostly below grade, which dates back to the early 1970's. With the tank in service and most of it out of view, it was not possible to assess its condition. A condition assessment needs to be undertaken to determine whether structural repairs are needed. It was not clear whether this tank can be bypassed.



Figure 5-3 Beacon Falls Rectangular Primary Clarifier

4. **Aeration Basins and Blowers.** The secondary treatment system is activated sludge basins, with a grid of diffusers. These were installed during the 1994 upgrade, along with the three small conventional aeration blowers which are located in the basement of the Operations Building. Based on the age of these units, they should be replaced with more energy-efficient modern blowers. The aeration basins need to be modified as well, to improve nitrification and denitrification capability.



Figure 5-4 Beacon Falls Aeration Basin



Figure 5-5 Beacon Falls Aeration Blowers

5. **Secondary Clarifiers.** The Seymour plant has two rectangular concrete secondary settling tanks. The older, original clarifier is relatively shallow (8-foot depth); the second clarifier, added during the 1994 upgrade, is 12 feet deep. It was reported that a retrofit at the inlet to these tanks is required to optimize flow split and overall treatment performance.



Figure 5-6 Beacon Falls Secondary Clarifier Effluent Weirs

6. **Ultraviolet Disinfection.** A new two-bank outdoor ultraviolet disinfection system, installed in 2006, is reported to be in good working condition.



Figure 5-7 Beacon Falls UV Disinfection System

7. **Alkalinity Addition.** Soda ash is added at the headworks for alkalinity supplementation, to facilitate nitrogen removal.
8. **Sludge Processing.** The existing sludge pumps, which include plunger pumps for primary sludge and RAS/WAS pumps for secondary sludge, were installed in the 1994 upgrade and are due to be replaced. The solids processing system blends primary and secondary sludge, which is periodically decanted to a final concentration of approximately 2% solids. The sludge is trucked off-site to a regional sludge treatment merchant plant. The existing

anaerobic digester no longer functions as a digester, and is used for sludge storage. There is an abandoned sludge centrifuge onsite. Mechanical thickening should be provided to decrease sludge disposal costs.



Figure 5-8 Beacon Falls Plunger Pumps for Sludge

9. **Other Items.** The Beacon Falls WPCF Upgrade Summary memorandum provided by DPC Engineering, dated October 18, 2018 identified additional upgrades required at this facility. This includes: operations building roof replacement, site-wide electrical system upgrades, a new emergency standby generator, and miscellaneous safety-related improvements.

5.2 DESCRIPTION OF EXISTING FACILITIES – BEACON FALLS WASTEWATER COLLECTION SYSTEM

Most of the collection system (perhaps two-thirds) consists of pipe installed within the past 20 years, mostly PVC. The remaining one-third of the collection system is older than that. It is reported that the system has approximately 33 miles of sewer pipes overall. The Beacon Falls WPCA has recently taken over responsibility for maintaining the collection system. Most of the maintenance work is related to occasional blockages and root intrusion type problems. There is no annual program for pipe replacement in the system.

An I/I study was reported to have been conducted as part of the 2015 Wastewater Facilities Plan. It recommended further I/I investigation in the future, as well as limited I/I remediation work. At this time, I/I reduction is not a high priority for the Beacon Falls WPCF, and all future plans related to plant upgrades have assumed current levels of I/I wastewater flows.

5.3 DESCRIPTION OF EXISTING FACILITIES – BEACON FALLS WASTEWATER PUMPING STATIONS

All flow to the WPCF is pumped to the plant (none flows to the plant by gravity). There are three municipal pump stations in the collection system, plus one private pump station operated by a condominium developer. The three municipal pump stations typically require minimal maintenance work; it is anticipated that they will require their next major renewal/rehabilitation in approximately 10 years. The three pump stations are:

1. Railroad Avenue Pump Station – Located across from the WPCF, last upgraded 10 years ago; this station takes about 85% of the system flow. Consists of two pumps, each 1,000 gpm.
2. Pines Bridge Pump Station – Utilizes Tsurimi cutter pumps.
3. West Road Pump Station – A very small station, with a 3-inch force main.

5.4 CAPITAL PROJECT NEEDS TO 2040 UNDER BASE CASE

This section summarizes the estimated capital facilities that would be needed for Beacon Falls to meet system requirements throughout the planning period (to 2040), without regionalization. It addresses expenditures for the WPCF, the wastewater collection system, and the wastewater pumping stations.

5.4.1 Capital Projects to 2040 – Beacon Falls Water Pollution Control Facility

Proposed capital facility needs for the Beacon Falls WPCF are based on recommended upgrade items in a projected capital improvements program provided by DPC Engineering in the Beacon Falls WPCF Upgrades Summary memorandum dated October 17, 2018. The list of new or upgraded facilities programmed at the plant is identified below:

1. Influent Pumping System Upgrade
2. Headworks (Screen Building at Existing Pump Station)
3. Primary Clarifier (Convert to Anoxic)
4. Aeration System Upgrades and Instrumentation
5. Secondary Clarifier Upgrade/Expansion
6. Secondary Clarifier Anoxic Conversion
7. RAS/WAS Systems Upgrades
8. Gravity Thickener – Anoxic Recycle Conversion
9. Rotary Drum Thickener – Dewatering (In Existing Building/Finance)
10. Electric/Main Switchgear/Generator
11. Digester Cleaning, Replacement Roof and Mixer
12. Operations Building Replacement Roof
13. Safety Improvements

5.4.2 Capital Projects to 2040 – Beacon Falls Wastewater Collection System

There is no program for sewer replacement in Beacon Falls at this time. While no major new sewer projects have been identified, over time the system will need replacement of aging sewers on a long-term cycle. We have assumed the collection system improvements to be started and underway within the short-term (approximately 5 years).

5.4.3 Capital Projects to 2040 – Beacon Falls Pumping Stations

Beacon Falls has three relatively small municipal pumping stations in its collection system. The largest one, Railroad Avenue Pumping Station, will likely be due for a major upgrade in approximately 10 years. The other two stations will require periodic replacement of mechanical equipment and other repairs. With no additional information provided for these pump stations, it is assumed they will require upgrade in approximately 10 years.

5.5 PROJECTED CAPITAL AND O&M EXPENDITURES – 2040 BASE CASE

Projected expenditures for the WPCF, the wastewater collection system, and the wastewater pumping stations have been addressed as part of this early planning study. Budgetary capital and O&M costs associated with the base case scenario for Beacon Falls outlined in this section are provided in Appendix A of this report. Capital costs associated with upgrading the WPCF have been based on engineering cost information provided by DPC Engineering.

6.0 NAUGATUCK WASTEWATER FACILITIES ASSESSMENT

6.1 DESCRIPTION OF EXISTING FACILITIES – NAUGATUCK WATER POLLUTION CONTROL FACILITY

The Naugatuck Water Pollution Control Facility (WPCF) serves the Borough of Naugatuck and portions of adjacent communities: Middlebury, Oxford, Beacon Falls and Prospect. Recent average flows to the plant (2015-2017) have been approximately 4.6 MGD, which is significantly below the permitted design average flow for the plant of 10.3 MGD.

The original plant was upgraded to secondary treatment in the 1970's. The treatment process lineup includes influent pumping followed by primary sedimentation, 4-stage Bardenpho BNR for nitrogen removal, secondary clarification, and disinfection prior to discharge to the Naugatuck River. Disinfection consists of sodium hypochlorite addition at the head of a chlorine contact tank, with bisulfite addition at the end for dechlorination.

The lack of a headworks at the plant to remove grit and screenings presents an operational challenge at the primary settling tanks and downstream facilities.

The Naugatuck WWTF is also the site of a regional solids processing facility that includes bulk sludge delivery, liquid sludge storage, dewatering via centrifuge or belt filter press, and incineration. High strength sidestream flows from the regional solids processing facility to the WWTP contribute significantly to plant loading.

The Naugatuck WWTF Facilities Plan (December 2017) included a recent, detailed condition assessment of the existing facilities, and developed a capital improvements plan for projects that should be undertaken within the next 10 years to address the needs of the plant over a 20-year planning period. The Facilities Plan addressed the aging infrastructure that needs to be repaired or replaced, and included process changes to meet the new phosphorus limitations. The regional sludge incinerator was not included in the scope of the Facilities Plan.

The condition of the existing facilities at the Naugatuck WWTF is discussed in detail in Section 4 and Appendix D of the 2017 Facilities Plan.

Black & Veatch visited the Naugatuck WWTF on July 27, 2018 to observe major plant systems. The facilities include the following:

1. **Influent Pump Station.** The plant influent pump station consists of four pumps in a dry pit/ wet pit arrangement. There is no headworks upstream of the influent pump station; consequently, the influent pumps are subject to maintenance challenges related to both grit and screenings. Three of the four influent pumps were recently replaced with new Sulzer centrifugal pumps.



Figure 6-1 Naugatuck New Sulzer Influent Pump

2. **Primary Settling Tanks.** The plant has two operable rectangular primary settling tanks, each 120 ft x 30 ft x 12 ft SWD. Each tank has two parallel sections, with chain and flight sludge collectors and cross-collectors. The tanks are covered with fabric covers, for odor control. The scum collection system has not worked for several years.



Figure 6-2 Naugatuck Primary Settling Tanks, Covered for Odor Control

3. **BNR Biological Treatment.** Biological treatment is accomplished in two parallel trains, by a 4-stage Bardenpho process for nitrogen removal. The basins have internal curtain walls to segregate the zones, along with internal mixed liquor recycle to enhance denitrification. Air for the diffuser grids is provided by two Piller turbo blowers installed in 2013, with backup provided by positive displacement blowers installed in 1986. A number of deficiencies in the biological treatment system, and opportunities to improve performance, were noted in Section 4.6 of the 2017 Facilities Plan.

Naugatuck and Veolia are working to meet 0.4 mg/L effluent phosphorus while keeping chemical costs low. The high sidestream phosphorus loading from onsite sludge processing activities makes this more of a challenge than at more typical domestic wastewater treatment plants. To achieve phosphorus reduction in anticipation of more stringent permit limits scheduled to take effect late summer/early fall of 2019, Naugatuck has started to

implement low capital cost measures and chemical addition (PAC). These initial measures have resulted in effluent phosphorus reduction; however, additional capital cost improvements are planned in the upcoming months to reduce phosphorus levels and to meet the permit requirements.



Figure 6-3 Naugatuck Anoxic Basins, in Modified 4-Stage Bardenpho Process



Figure 6-4 Naugatuck Activated Sludge Aeration Basin



Figure 6-5 Naugatuck Piller Turbo Blower, for Aeration System

4. **Secondary Clarifiers.** The Naugatuck WPCF has three rectangular secondary clarifiers, each 150 ft x 40 ft x 12 ft SWD. Much of the mechanical equipment, including the collector drives, RAS pumps and WAS pumps, is from the 1970's and needs to be replaced. Flow from these three clarifiers is sent to a fourth polishing clarifier downstream of the other three.



Figure 6-6 Naugatuck Secondary Clarifiers

5. **Disinfection System.** Disinfection, which is provided by hypochlorite addition, is followed by bisulfite dechlorination. Since a significant portion of the plant's secondary effluent is used by the sludge incinerators, only a portion of the secondary effluent is disinfected and discharged to the Naugatuck River. The condition of the chlorine contact tanks is considered fair.



Figure 6-7 Naugatuck Chlorine Contact Basins

6. **Sludge Thickening and Dewatering.** Primary sludge and WAS are co-thickened in a gravity thickener. There are four covered gravity thickeners at the WPCF. Thickened sludge is stored in multiple sludge storage tanks onsite. Sludge dewatering is accomplished with two centrifuges (installed 2002) and two belt filter presses (installed in the 1970's). Due to the large amount of sludge being processed from other plants at the incineration facility, sludge dewatering generally takes place on a 24/7 basis.



Figure 6-8 Naugatuck Belt Filter Press

6.2 DESCRIPTION OF EXISTING FACILITIES – NAUGATUCK WASTEWATER COLLECTION SYSTEM

The existing sewer system is comprised of 156 miles of gravity sewer ranging from 6 inches to 24 inches in diameter and 0.8 mile of force main and is divided into 20 subsystems. In October 2017 Naugatuck received a Consent Order (No. CWA-AO-R01-FY17-07) relating to the collection system. The Order contains specific requirements for reporting and operations and maintenance of the

collection system, as well as an I/I Control Plan and a Capacity, Management, Operation and Maintenance (CMOM) program.

The Naugatuck WPCA has engaged in an active sewer system evaluation survey (SSES) and rehabilitation program, with recent engagements including the 2015 I/I analysis, which recommended two phases of SSES, the first of which was completed and documented in the SSES Plan Report in 2017. The 2017 SSES plan documented the investigation of priority I/I subbasins, and its recommendations included further SSES activities as well as cost-effective rehabilitation and I/I removal efforts. These recommendations were incorporated into the 2017 Facilities Plan. I/I reduction resulting from the proposed activities was estimated to be 0.3 MGD on average.

6.3 DESCRIPTION OF EXISTING FACILITIES – NAUGATUCK WASTEWATER PUMPING STATIONS

The Naugatuck collection system has five wastewater pumping stations. All were constructed in the 1970's or 1980's, and are relatively small facilities with submersible pumps. The condition of these facilities is discussed in Section 6 and Appendix J of the 2017 Facilities Plan. In all cases, it is reported that the pumps were recently replaced or rebuilt. All stations are generally reported to be in good condition, however the Inwood Pump Station has corrosion on some of the metal piping and valves, which may be due to hydrogen sulfide.

6.4 CAPITAL PROJECT NEEDS TO 2040 UNDER BASE CASE

This section summarizes the estimated capital facilities that would be required for Naugatuck to meet system needs throughout the planning period (to 2040), without regionalization. It addresses the WPCF, the wastewater collection system, and the wastewater pumping stations.

6.4.1 Capital Projects to 2040 – Naugatuck Water Pollution Control Facility

The recommended capital improvements for the Naugatuck WPCF are indicated on Table 8-2 of the 2017 Naugatuck Wastewater Treatment Facilities Plan, with the associated costs (in 2016 dollars) provided on Table 8-4 of that Plan. The capital costs for the WPCF presented in Appendix E have been developed based on that information.

The Facilities Plan accounted for capital costs that would need to be expended during the first ten years of the planning period, through FY 2026. During the later years of this regionalization planning period, which extends to 2040, replacement of mechanical equipment expected to wear out after 2026 also need to be included.

6.4.2 Capital Projects to 2040 – Naugatuck Wastewater Collection System

In the 2017 Facilities Plan, a program of collection system studies and improvements was identified, including a budget for emergency repairs, for the period of FY18 to FY26. This information has been used as a basis for the projected costs in Appendix E of this report.

6.4.3 Capital Projects to 2040 – Naugatuck Pumping Stations

Several relatively small recommended capital or repair type projects for the wastewater pumping stations are listed in Table 8-3 of the 2017 Facilities Plan. This includes generator replacement and miscellaneous repairs in the near term, and regular, scheduled equipment replacement in 11-20

years. Estimated costs for those projects are included in Table 8-4 of the 2017 Facilities Plan. Those costs have been used as the basis for the projected pumping station capital costs that are included in Appendix E of this report.

6.5 PROJECTED CAPITAL AND O&M EXPENDITURES – 2040 BASE CASE

Projected expenditures for the WPCF, the wastewater collection system, and the wastewater pumping stations have been addressed as part of this early planning study. Budgetary capital and operating costs associated with the base case scenario for Naugatuck outlined in this section are provided in Appendix A of this report. Capital costs associated with upgrading the WPCF have been based on engineering cost information provided in the 2017 Facilities Plan.

APPENDIX A

Projected Capital and O&M Expenditures to 2040 Under Base Case

APPENDIX A Projected Capital and O&M Expenditures to 2040 Under Base Case

This appendix provides planning-level capital and O&M costs for wastewater infrastructure for the five communities in this study (Derby, Ansonia, Seymour, Beacon Falls, and Naugatuck), under the base case scenario of no regionalization, through 2040. The costs presented in this appendix correspond to what will be required to address the existing conditions identified in the main body of this report, for each community.

In developing these costs, we have reviewed existing planning and engineering reports on wastewater infrastructure needs for the communities. However, for some of the communities there was little information available to properly capture the 20-year capital needs. As a result, very high-level estimates have been made based on experience with other comparable-sized facilities, on-site reviews, and parametric considerations (such as \$/gallon for treatment or \$/LF for collection system replacement).

The capital cost tables are broken down by the three main categories for wastewater infrastructure: treatment facilities, collection systems and large pumping stations in the collection system. The basis for the costs developed in for each community is provided in the discussion that follows.

A.1 Derby Capital Expenditures to 2040 – Summary

Projected wastewater infrastructure capital costs for Derby, under the base case scenario of no regionalization, through 2040, are summarized in Table A-1 below. The costs presented are based on 2019 dollars. Project costs shown include allowances for construction contingency as well as engineering, legal and administration.

Table A-1 Derby Wastewater Facilities Base Case Condition Capital Budgetary Needs

Derby Wastewater Capital Projects to 2040	Project Cost (2019 \$)
Water Pollution Control Facility (WPCF, Major Upgrade)	\$ 70,000,000
Collection System (CS)	
Subtotal for Years 1-5 (System Renewal @ 2.50%/yr. = \$654,000/yr.)	\$ 3,300,000
Subtotal for Years 6-20 (System Renewal @ 1.20%/yr. = \$314,000/yr.)	\$ 4,700,000
Large Pumping Stations (PS)	
Division Street New Pumping Station	\$ 2,200,000
Allowance for Other Pumping Station Upgrades through 2040	\$ 2,000,000
TOTAL: WPCF + CS + PS	\$ 82,200,000

Derby Base Case Capital Costs – Basis and Assumptions

The basis for the costs presented in Table A-1 is summarized below.

- Based on Black & Veatch’s observations of the facilities and supported by input of plant staff, the Derby WPCF is due for a major overhaul, approaching full replacement. We believe that the extent of the work that will required to upgrade this facility to meet requirements through 2040 was not fully captured in the 2014 Draft Facilities Plan. This estimate for WPCF upgrade needs was calculated based on a \$20/gal assumption considering the maximum month flow capacity of 3.5 MGD.
- Based on age and condition of the collection system (very old, approximately 70% VC pipe, high infiltration and inflow) system renewal costs were based on replacing 2.5% of the collection system per year during the first five years of a “catch up” period, followed by a sustained investment thereafter of replacing 1.2% of the system annually. Based on an overall collection system length of 41.2 miles of sewers, at an average 2019 replacement cost of \$120/LF, this would require approximately \$654,000/year for the first five years, and \$314,000/year thereafter, as shown on Figure A-1.

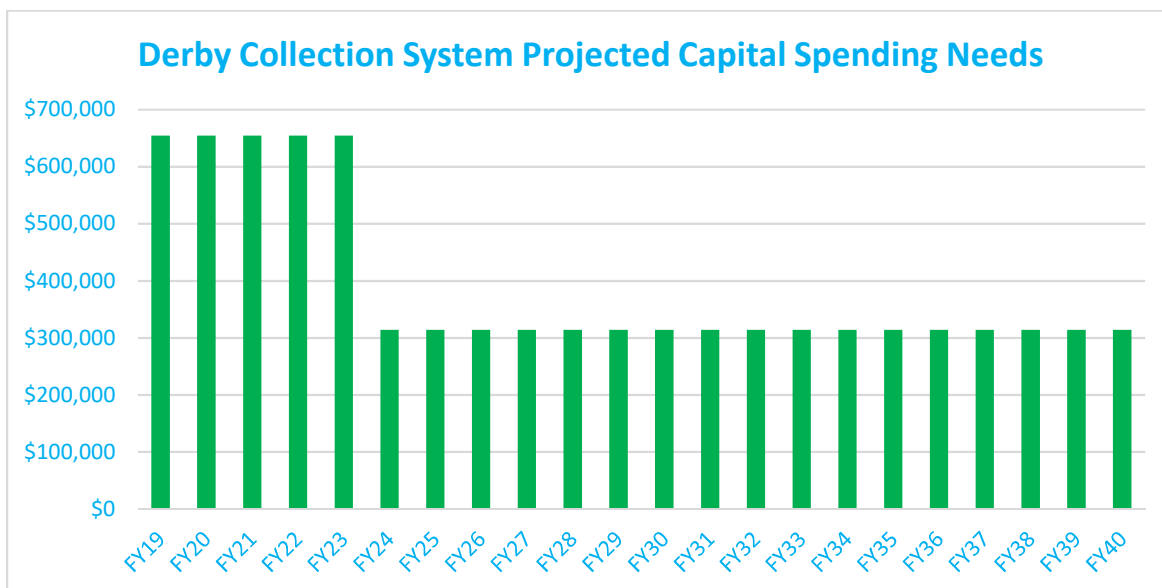


Figure A-1 Derby Collection System Projected Capital Spending Needs (2019 \$)

- Estimated project costs for new Division Street Pump Station were based on Table 11-1 of the Facilities Plan, escalated to 2019 and includes allowances for contingency as well as for engineering, legal and administration.

A.2 Capital Expenditures to 2040 – Ansonia Summary

Projected wastewater infrastructure capital costs for Ansonia, under the base case scenario of no regionalization, through 2040, are summarized in Table A-2 below. The costs presented are based on 2019 dollars. Project costs shown include allowances for construction contingency as well as engineering, legal and administration.

Table A-2 Ansonia Wastewater Facilities Base Case Condition Capital Budgetary Needs

Ansonia Wastewater Capital Projects	Project Cost (2019 \$)
Water Pollution Control Facility (WPCF)	\$ 15,000,000
Collection System (CS)	
Subtotal for Years 1-5 (System Renewal @ 2.0%/yr. = \$828,000/yr.)	\$ 4,100,000
Subtotal for Years 6-20 (System Renewal @ 1.0%/yr. = \$414,000/yr.)	\$ 6,200,000
Large Pumping Stations (PS)	
Allowance for Pumping Station Upgrades through 2040	\$ 3,000,000
TOTAL: WPCF + CS + PS	\$ 28,300,000

Ansonia Base Case Capital Costs – Basis and Assumptions

The basis for the costs presented in Table A-2 is summarized below.

- The Ansonia WPCF had an extensive upgrade completed in 2011, and the overall condition of the plant is good. While no major plant upgrades involving new tanks and structures are anticipated before 2040 under the base case scenario, it is likely that mechanical equipment upgrades would be required by approximately 2030, which is within the planning period of this study. Based on major mechanical equipment upgrades at approximately 20% of the cost of the prior upgrade, the planning level budget for capital expenditures is approximately \$15M in 2019 dollars.
- The Ansonia collection system is old and much of it is VC pipe. While there has been some I/I work done in the past, significant investment is still required. A higher system-wide renewal rate is recommended for the first five years for catch-up. Therefore, system renewal costs were based on replacing 2.0% of the collection system per year during the first five years of a “catch up” period, followed by a sustained investment thereafter of replacing 1.0% of the system annually. Based on an overall collection system length of 65.3 miles of sewers, at an average (2019) unit cost of \$120/LF (to cover average lining or replacement costs, manhole rehabilitation, related inspection and SSES activities), this would require an investment of approximately \$828,000/year for the first five years, and \$414,000/year thereafter, as shown on Figure A-2.

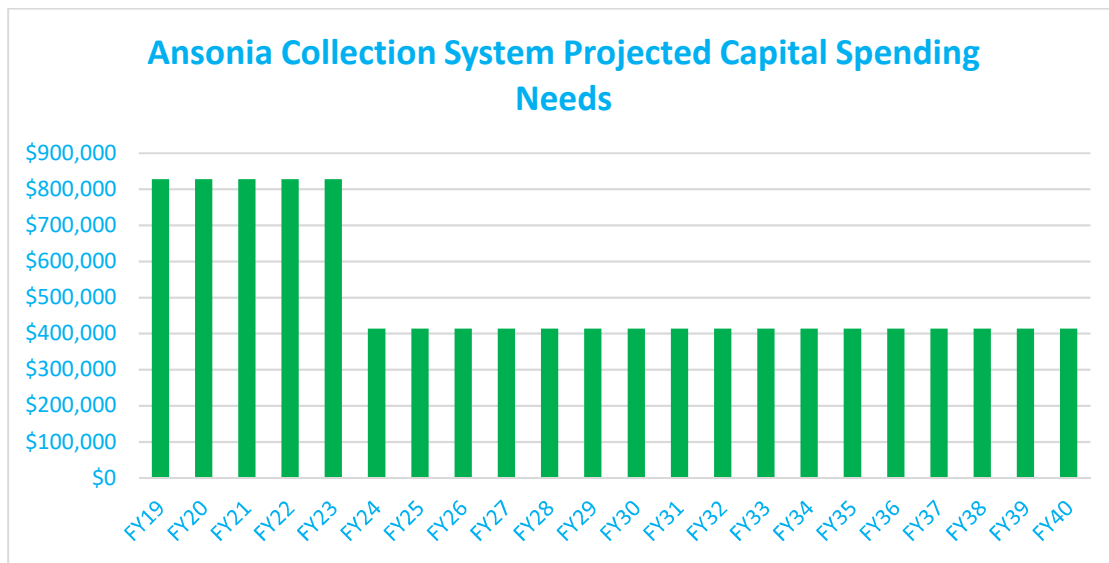


Figure A-2 Ansonia Collection System Projected Capital Spending Needs (2019 \$)

- All of the ten larger pump stations in Ansonia have been upgraded within the past 10 years, including the two largest stations (Coe and Bartholomew) which completely upgraded recently. Therefore, an allowance of \$2M has been provided for mechanical upgrades to each of the two larger pumping stations, which would be expected within the 20-year planning period.

A.3 Capital Expenditures to 2040 – Seymour Summary

Projected wastewater infrastructure capital costs for Seymour, under the base case scenario of no regionalization, through 2040, are summarized in Table A-3 below. The costs represent project are based on 2019 dollars. Project costs shown include allowances for construction contingency as well as engineering, legal and administration.

Table A-3 Seymour Wastewater Facilities Base Case Condition Capital Budgetary Needs

Seymour Wastewater Capital Projects	Project Cost (2019 \$)
Water Pollution Control Facility (WPCF)	\$ 40,000,000
Collection System (CS)	
Subtotal for Years 1-5 (System Renewal @ 2.0%/yr. = \$798,000/yr.)	\$ 4,000,000
Subtotal for Years 6-20 (System Renewal @ 0.75%/yr. = \$299,000/yr.)	\$ 4,500,000
Large Pumping Stations (PS)	
Allowance for South Derby and North Derby PS Upgrades through 2040	\$ 2,000,000
TOTAL: WPCF + CS + PS	\$ 50,500,000

Seymour Base Case Capital Costs – Basis and Assumptions

The basis for the costs presented in Table A-3 is summarized below.

- The Seymour WPCF is due for a major upgrade and overhaul of existing systems. However, no Facilities Plan has been commissioned to identify the facility needs in depth. Therefore, a high-level budgetary estimate for plant upgrade needs was based on a unit cost of \$14/gal and a maximum month design flow of 2.93 MGD, resulting in a project cost of approximately \$40M for the WPCF upgrade (based on 2019 dollars).
- Significant investment will be required for sewer replacement and repairs, based on age and anticipated poor condition of the system. Projected system renewal costs were based on replacing 2.0% of the collection system per year during the first five years of a “catch up” period, followed by a sustained investment thereafter of replacing 0.75% of the system annually. Based on an overall collection system length of 63 miles of sewers, at an average (2019) unit cost of \$120/LF (to cover average lining or replacement costs as well related inspection and SSES activities), this would require an investment of approximately \$798,000/year for the first five years, and \$299,000/year thereafter, as shown on Figure A-3.

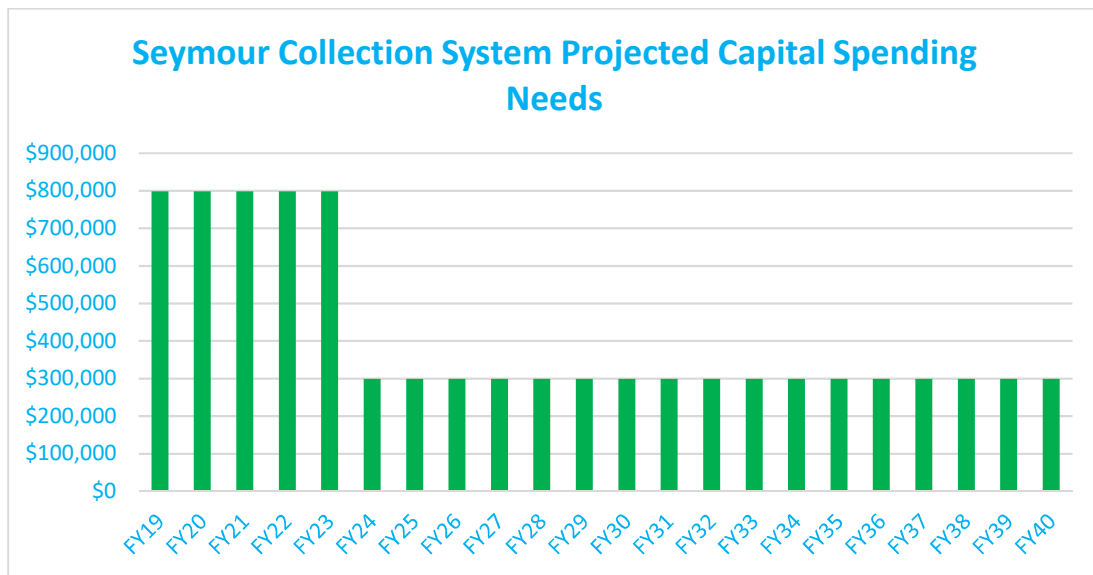


Figure A-3 Seymour Collection System Projected Capital Spending Needs (2019 \$)

- The two largest pumping stations in the system are on Derby Avenue: the South Derby Pumping Station and the North Derby Pumping Station. While these pumping stations are in good condition at this time, it is anticipated that they will require major mechanical upgrades within the planning period (before 2040). Therefore, a high level budgetary cost allowance for that work has been included in Table A-3 above, for renewal of these two pump stations.

A.4 Capital Expenditures to 2040 – Beacon Falls Summary

Projected wastewater infrastructure capital costs for Beacon Falls, under the base case scenario of no regionalization, through 2040, are summarized in Table A-4 below. The costs presented are based on 2019 dollars. Project costs shown include allowances for construction contingency as well as engineering, legal and administration.

Table A-4 Beacon Falls Wastewater Facilities Base Case Condition Capital Budgetary Needs

Beacon Falls Wastewater Capital Projects	Project Cost (2019 \$)
Water Pollution Control Facility (WPCF)	\$ 14,000,000
Collection System (CS)	
Total for Years 1-20 (System Renewal @ 0.75%/yr. = \$157,000/yr.)	\$ 3,100,000
Large Pumping Stations (PS)	
Allowance for PS Upgrades through 2040	\$ 500,000
TOTAL: WPCF + CS + PS	\$ 17,600,000

Beacon Falls Base Case Capital Costs – Basis and Assumptions

The basis for the costs presented in Table A-4 is summarized below.

- The Beacon Falls WPCF is due for a major upgrade and overhaul of existing systems. DPC Engineering is underway in preparing construction plans and specifications to upgrade the WPCF. DPC also prepared a construction cost opinion based on the design level of completion, which was summarized in a memorandum from Dave Prickett to Beacon Falls dated October 17, 2018. That memorandum outlined a program of proposed improvements through 2024, at a project cost of \$9.77M in 2018 dollars. For the current study we have escalated that cost to 2019 dollars and added an allowance for future upgrades through the end of the 20-year planning period.
- Approximately two-thirds of the Beacon Falls collection system sewer piping was installed within the past 20 years. However, no detailed engineering investigations have been done on the collection system in the recent past. Based on this being a relatively new sewer system, it is presumed that the system as a whole is good condition. Therefore, a relatively low annual investment should be required compared to other the communities in this study. Projected system renewal costs were based on replacing 0.75% of the collection system per year, throughout the 20-year planning period. Based on an overall collection system length of 33 miles of sewers, at an average (2019) unit cost of \$120/LF (to cover average lining or replacement costs as well related inspection and SSES activities), this would require an investment of approximately \$157,000/year throughout the planning period, as shown on Figure A-4.

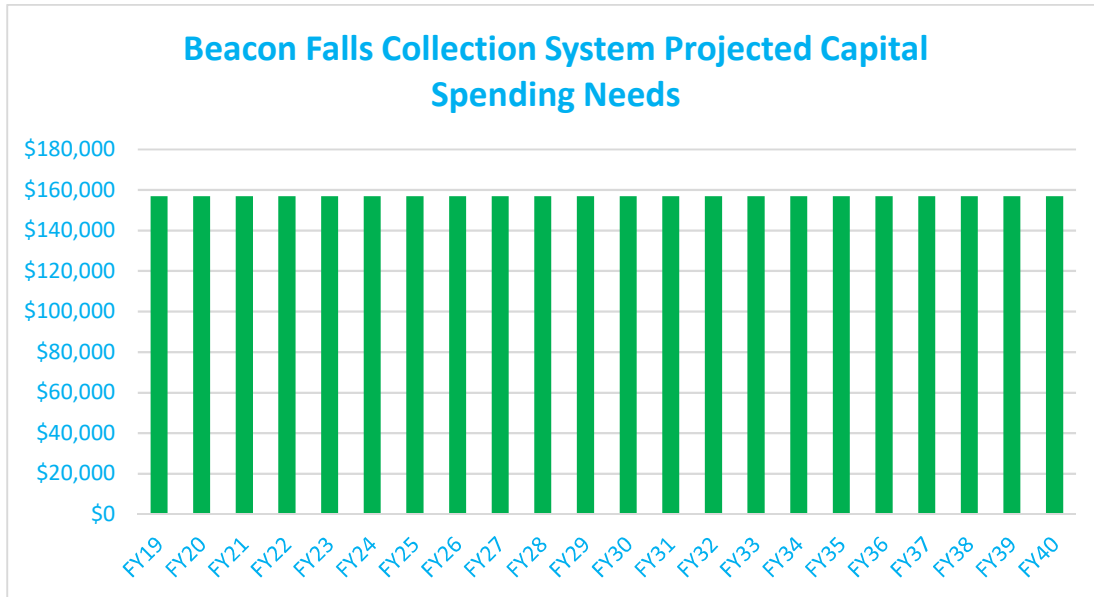


Figure A-4 Beacon Falls Collection System Projected Capital Spending Needs (2019 \$)

- There are three wastewater pumping stations owned by Beacon Falls that serve the collection system. The Railroad Avenue PS (upgraded about 10 years ago, which handles approximately 85% of the system's flow) is the largest; the two smaller stations are West Road PS and Pines Bridge PS. While all three stations are reported to be in good condition, it is anticipated that mechanical upgrades will be required in another 10 years. Therefore, an allowance for pump station upgrades has been included in Table A-4.

A.5 Capital Expenditures to 2040 – Naugatuck Summary

Projected wastewater infrastructure capital costs for Naugatuck, under the base case scenario of no regionalization, through 2040, are summarized in Table A-5 below. The costs represent project are based on 2019 dollars. Project costs shown include allowances for construction contingency as well as engineering, legal and administration.

Table A-5 Naugatuck Wastewater Facilities Base Case Condition Capital Budgetary Needs

Naugatuck Wastewater Capital Projects	Project Cost (2019 \$)
Water Pollution Control Facility (WPCF)	\$ 55,000,000
Collection System (CS)	
Subtotal for Years 1-5 (System Renewal @ 1.5%/yr. = \$1,480,000/yr.)	\$ 7,400,000
Subtotal for Years 6-20 (System Renewal @ 0.75%/yr. = \$741,000/yr.)	\$ 11,100,000
Large Pumping Stations (PS)	
Allowance for PS Upgrades through 2040	\$ 1,000,000
TOTAL: WPCF + CS + PS	\$ 74,500,000

Naugatuck Base Case Capital Costs – Basis and Assumptions

The basis for the costs presented in Table A-5 is summarized below.

- The Naugatuck WPCF is due for a major upgrade at a number of its significant treatment systems. The December 2017 Facilities Plan identified necessary upgrades with capital costs through 2026. In view of the age of the facility, additional capital expenditures have been programmed to address future upgrades and equipment replacement that will be needed during the 2027-2040 period. The capital costs shown include approximately \$46M for upgrades through 2026, and \$9M for replacements and upgrades for the period 2027-2040.
- The Naugatuck collection system is old and much of it is VC pipe. Significant investment will be required for sewer replacement and repairs, based on age and anticipated poor condition of the system. Projected system renewal costs were based on replacing 1.5% of the collection system per year during the first five years of a “catch up” period, followed by a sustained investment thereafter of replacing 0.75% of the system annually. Based on an overall collection system length of 156 miles of sewers, at an average (2019) unit cost of \$120/LF (to cover average lining or replacement costs as well related inspection and SSES activities), this would require an investment of approximately \$1,480,000/year for the first five years, and \$741,000/year thereafter, as shown on Figure A-5.

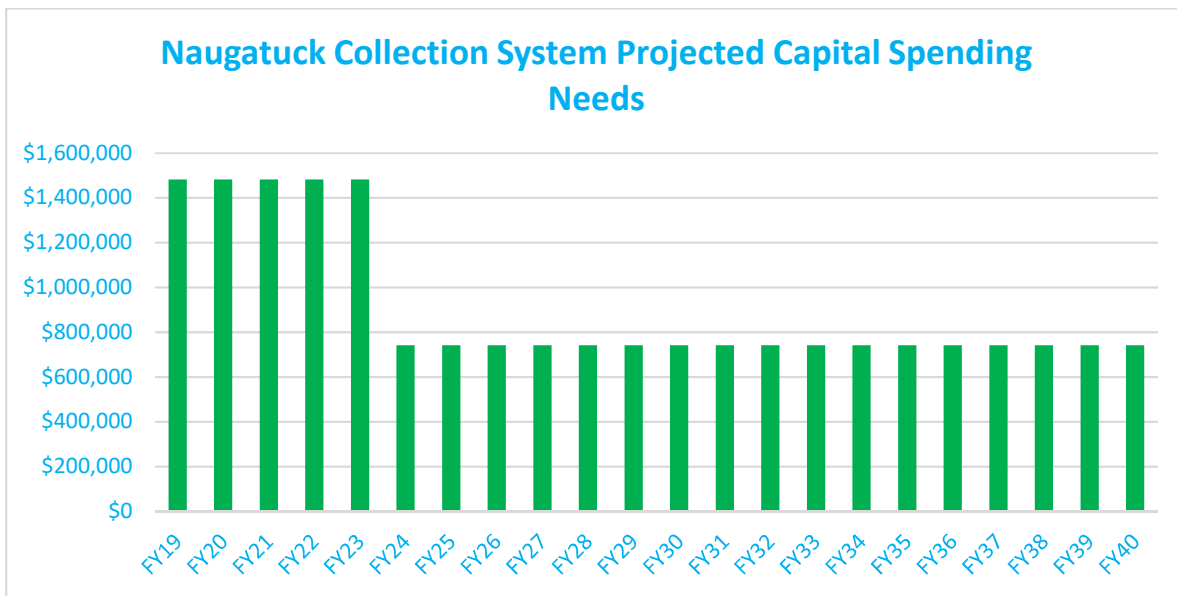


Figure A-5 Naugatuck Collection System Projected Capital Spending Needs (2019 \$)

- The Naugatuck collection system has five small to medium-sized pumping stations, each equipped with submersible pumps. These stations were built in the 1970's and 1980's, and are reported to be in fair condition. All will need equipment replacement (pumps, lighting, electrical, controls, generators, etc.) within the 20-year planning period. An allowance of \$1M has been provided for equipment replacement and upgrades through 2040.

A.6 Annual O&M Expenditures – Summary

Table A-6 below represents expected annual O&M costs for each of the five communities under the base case scenario, with no regionalization. Note that these costs represent the current O&M costs provided by the communities. O&M costs are not expected to change significantly with the upgrades of the WPCFs.

Table A-6 O&M Cost Summary

Estimated O&M Needs	Annual O&M Costs for Wastewater Systems (\$M/year, 2019 dollars)
Derby ¹	\$ 2.56
Ansonia ²	\$ 2.70
Seymour	\$ 1.55
Beacon Falls	\$ 0.68
Naugatuck ³	\$ 7.62
NOTES: 1. Derby includes \$0.634M/yr. debt service 2. Ansonia includes \$0.9M/yr. loan repayment to DEEP 3. Naugatuck costs have been increased by \$0.5M/year to account for chemicals associated with the phosphorus removal upgrade.	

APPENDIX C

TECHNICAL MEMORANDUM 3: SHORT LIST OF REGIONAL WASTEWATER ALTERNATIVES

REGIONAL WASTEWATER TREATMENT CONSOLIDATION STUDY

Technical Memorandum 3: Short List of Regional Wastewater Alternatives

B&V PROJECT NO. 198910

PREPARED FOR

Naugatuck Valley Council of Governments

9 June 2020

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1.0 PURPOSE AND BACKGROUND

The Naugatuck Valley Council of Governments (NVCOG) is undertaking a regional wastewater treatment consolidation study comprising five municipalities in the region: Naugatuck, Beacon Falls, Seymour, Ansonia, and Derby. Phase 1 of this work was completed in early 2019 and identified a long list of 23 wastewater system regional alternatives (regional alternatives) to investigate further. Phase 2 of the study will provide more in-depth evaluation of the Phase 1 long list regional alternatives. The process will start with a screening-out analysis of the long list resulting in a short list of regional alternatives. The short-list will undergo a more detailed analysis along with the 'base case' alternatives where each of the communities would continue to handle, treat and discharge their wastewater as they are currently doing. This analysis will allow for a recommended alternative(s) to be identified. After some refinement of the recommended alternative(s), an Environmental Impact Evaluation (EIE) would take place.

This technical memorandum (TM) summarizes the screening-out analysis undertaken on the long list of 23 regional alternatives coming out of Phase 1. As identified in our work plan, our goal is to cull the long list down to a total of six regional alternatives. These six will make up the short list of regional alternatives that will be developed and evaluated in greater detail in the follow-on work task. Table 1-1 identifies the 23 long list alternatives.

Table 1-1 Long List of Regional Alternatives

No.	Alternative Description
1	Beacon Falls to Naugatuck
2	Beacon Falls to Seymour
2a	Beacon Falls to Seymour, I/I Reduction
3	Derby to Ansonia
3a	Derby to Ansonia, I/I Reduction
4	Derby to Ansonia, Effluent Pumped to Housatonic River
4a	Derby to Ansonia, I/I Reduction, Effluent Pumped to Housatonic River
5	Derby and Seymour to Ansonia
5a	Derby and Seymour to Ansonia, I/I Reduction
5b	Derby and Seymour to Ansonia, Effluent Pumped to Housatonic River
5c	Derby and Seymour to Ansonia, I/I Reduction, Effluent Pumped to Housatonic River
6	Derby to Seymour and Ansonia
6a	Derby to Seymour and Ansonia, I/I Reduction
8	Ansonia to Derby
8a	Ansonia to Derby, I/I Reduction
9	Seymour and Ansonia to Derby
9a	Seymour and Ansonia to Derby, I/I Reduction
10	Seymour to Ansonia, Part of Ansonia to Derby
10a	Seymour to Ansonia, Part of Ansonia to Derby, I/I Reduction
11	Beacon Falls and Seymour to Ansonia, Part of Ansonia to Derby
11a	Beacon Falls and Seymour to Ansonia, Part of Ansonia to Derby, I/I Reduction
12	Beacon Falls, Seymour, and Ansonia to Derby
12a	Beacon Falls, Seymour, and Ansonia to Derby, I/I Reduction

1.1 METHODOLOGY

The long list of regional alternatives identified and selected in Phase 1 were defined and better developed in this task, such that they could be compared and so that the less implementable alternatives could be screened out. The development process included three major assessments below.

1. Aggressive I/I Evaluation. The plan was to review the feasibility of implementing aggressive inflow and infiltration (I/I) measures in the collection systems. Typical I/I programs are often beneficial to reduce wastewater flows to treatment facilities, and it is understood that these programs will continue as recommended and required in each of the NVCOG communities regardless of regionalization. The purpose of aggressive I/I control measures is also to reduce flows; however, these measures are not always cost effective when compared to simply pumping and treating those excess flows. The feasibility of aggressive I/I control was investigated to determine if the regional alternatives which include these measures should be considered further or if they should be removed from further study. This resulted in screening out approximately half of the long list of alternatives.

2. Conveyance Corridor Evaluation. This assessment evaluated the conveyance corridors/pipeline routes that will connect the treatment plants of communities that comprise regional alternatives. Several of the long list alternatives share common conveyance corridors and could therefore be investigated collectively based on feasible pipeline routing. This evaluation added important and necessary detail to the regional alternatives, enabling the review to identify significant issues with the implementation of some of the regional alternatives. Long list alternatives with pipeline routes that would be prohibitively costly and/or that lack reliability were screened out.
3. Plant Process and Site Layout Evaluation. This planning level analysis focused on the treatment requirements and associated facility needs for individual plants and regionalized plants. Treatment capacity of wastewater plants was analyzed to determine the general process and upgrade needs at the baseline level (i.e. single plant upgrade needs only) and for the different alternatives. Infeasible or redundant plant process and layout modifications were screened out.

Each of these assessments revealed the less attractive attributes of some of the regional alternatives that resulted in a screening-out of those alternatives. Through a progression of screening-out the less desirable regional alternatives, the TM culminates with the short-listed alternatives that will undergo more detailed development and cost effectiveness assessment in a subsequent task of the study. Alternatives that were screened out at each step are shown as gray strikethrough text in tables of alternatives under consideration in each corresponding section.

2.0 AGGRESSIVE I/I EVALUATION

2.1 DETERMINING THE RIGHT LEVEL OF I/I CONTROL

Infiltration and Inflow (I/I) is extraneous, undesired flow in the sewer system. It is typically relatively clean groundwater or storm water runoff that enters the collection system, potentially overwhelming pipe, pump, or treatment capacity, as well as increasing treatment and pumping costs. Defects resulting from aging, structural failure, lack of proper maintenance, and poor construction and design practices in sanitary sewer systems are the most common source of I/I. Defects can include conditions such as broken pipes; leaking joints; manhole lids with holes and/or poor sealing; and root infested sewer laterals. These conditions can compromise the structural integrity and contribute to excessive I/I during and after precipitation events, which can then lead to sewer surcharging and system overflows.

Many decades of industry experiences along with state-of-the-art methods indicate that integrated approaches to improving sewer condition and capacity is a prudent approach to managing I/I. Best practices developed by utility owners indicate that before investing in sanitary sewer capacity improvements to handle excessive I/I, it is critical to improve sewer system structural conditions to realize practical levels of I/I reduction first, followed by supplemental right-sized conveyance/storage and downstream treatment systems. It has also been proven that asset management approaches to sewer system rehabilitation are effective and adding I/I reduction criteria will assist prioritizing public investments.

The long list of regional alternatives each generally included at least two variants: option A included normal I/I control measures, while option B included aggressive I/I control measures, which typically include comprehensive rehabilitation of problematic sub-basins as well as private I/I removal. Some level of I/I control is recommended in all cases. This evaluation is focused on the potential benefit of adopting aggressive I/I control measures with the aim of reducing required transport and treatment capacity.

2.2 IDENTIFYING AND CHARACTERIZING I/I IN A COLLECTION SYSTEM

2.2.1 Flow Monitoring

The first step to controlling I/I is understanding the magnitude (how much flow), extent (where is it coming from), and nature (rapid inflow vs. gradual infiltration) of the problem. There are many different potential sources and patterns of I/I, ranging from discrete, identifiable sources to diffuse infiltration system-wide. The more widespread the problem is, the more expensive it will be to address. Flow monitoring is typically the first step in I/I management because it is cost-effective at characterizing each of the factors identified above. Having flow monitoring data from high groundwater periods and during storm events provides the ability to develop a strategy for successful I/I control.

2.2.2 Major Inflow Sources

Inflow sources typically provide very rapid response to rainfall, with a source of direct entry to the sewer system. These flows can lead to very high peaks that quickly overwhelm a sewer system. Where significant inflow is identified, it is typically the most cost-effective and beneficial approach to remove those sources. However, inflow sources are also frequently over-emphasized; solving these problems will reduce I/I, but there are many other sources as well, so it will not

reduce I/I to desired levels; it is the first step in I/I control. Common contributors of inflow include legal and illegal sources, including:

- Sump pumps
- Roof leaders
- Surface drainage to manholes
- Cross connections to storm sewers or catch basins

2.2.3 Infiltration Sources

Infiltration of various sorts are typically the predominant sources of I/I in most systems, especially when major sources of inflow have already been removed. Infiltration can be long term infiltration due to high groundwater in parts of the system, or it can be storm infiltration, which can be very rapid or gradual, depending on system defects as well as soil and surface characteristics. Infiltration is typically more difficult and more costly to manage than inflow.

2.2.4 Sewer System Evaluation Surveys

A sewer system evaluation survey (SSES) is used to identify potential sources of I/I and target appropriate repairs. The most common elements of SSES are identified below. There are multiple methods of inspection that vary in cost and precision.

- Smoke testing,
- Dye testing,
- Flow isolation monitoring,
- Manhole inspections, and
- Pipe inspections.

2.3 EXISTING I/I IN THE FIVE STUDY COMMUNITIES

2.3.1 Prior SSES activities

2.3.1.1 Derby

As a result of long-term lack of investment in the collection system, and violations of its discharge permit and the Clean Water Act, the City of Derby was placed under a Consent Order to develop and implement a program of improvements, including a Capacity, Management, Operations and Maintenance (CMOM) plan and an I/I control plan. These plans were developed and submitted in 2016, with approval in late 2017. Since that time, the WPCA has been moving forward implementing the I/I control plan. As part of the I/I control plan, Derby conducted extensive condition assessment and smoke testing activities throughout the collection system, resulting in recommended improvements to remove I/I from the system. Implementation of the I/I control plan has been broken into phases, with the first two phases designed to address indirect cross connections with storm sewer catch basins, estimated to remove 1.5 MGD of peak storm flow. These two phases were completed in 2019. The third and future phases will focus on removing infiltration sources from the system, with the third phase in progress (2020) and expected to remove 30,000 gpd of peak flow from the system. Future phases are expected to be similar to phase three and will continue for approximately 10 years. Finally, private I/I sources are

recognized as a significant source of peak flow, estimated at 3 mgd, and the WPCA will be developing a program to reduce this flow over time by implementing new policies and procedures as well as potential programs with property owners.

2.3.1.2 Ansonia

Ansonia has not had a city-wide I/I program done for over 15 years. Little is known about the sources or extent of I/I in the system, but based on review of MOR data, the system is subject to significant peaking factors, indicating that I/I is a problem to be addressed.

2.3.1.3 Seymour

Seymour has not had a city-wide I/I program done for over 15 years. Little is known about the sources or extent of I/I in the system, but based on review of MOR data, the system is subject to significant peaking factors, indicating that I/I is a problem to be addressed. It has been indicated that the Town is underway on some activities associated with I/I measurement in the collection system. Information on this ongoing work and future plans has been requested from Seymour.

2.3.1.4 Beacon Falls

Given the relatively newer state of the Beacon Falls collection system and, low observed peak flow factors (based on monthly MOR data), along with the comparatively low wastewater flow in the Beacon Falls system, aggressive I/I control is not considered necessary in this system and was not evaluated further.

2.3.1.5 Naugatuck

The Borough of Naugatuck received a Consent Order in October 2017 for nine discharges of untreated wastewater to the Naugatuck River and Hop Brook between 2012-2016, which were suspected to be caused by infiltration and inflow. In parallel with that order, Naugatuck undertook a sewer system evaluation survey in April 2015, with an update in 2017. The Borough is in the process of re-procuring its professional O&M services contract for the wastewater system, which will include tasks to control infiltration and inflow, as well as management, operations, and maintenance (MOM) planning and implementation of critical capital projects.

2.3.2 Plant Daily Flow Data (MORs)

Daily flow data at the treatment plants, as recorded in the monthly operating reports (MORs) constitute the best long-term flow information for each of the communities. The data contain the daily maximum, minimum, and average flow for each plant, which provides an approximation of peaking factors (maximum:average) as well as seasonal variation due to groundwater levels and plant uptake. This is useful for general evaluation of performance and potential problems, but it does not provide information about the potential sources of flow or how widespread any problem may be in the collection system network. Following is a brief description of the data review performed for each community. For purposes of I/I review, MOR plant data was evaluated looking at daily flow values to approximate average dry weather flow and instantaneous maximum flows. This is slightly different than the flows analyzed in the plant capacity evaluation which focused on annual average, max month, and peak day flows.

2.3.2.1 Derby

Daily flow data (maximum, minimum, average) from monthly operating reports from 2015 into 2020 were analyzed to characterize infiltration and inflow to the extent possible (Figure 2-1). The peak flow of 10.0 mgd reflects the flow meter capacity and was recorded three times during the analysis period, in 2016, 2017, and 2019. Twelve events had a peak flow of at least 9.0 mgd during the 2015-2020 period.

Average daily flow varied significantly during the analysis period, but it appears that average dry weather flow could reasonably be approximated at 1.0 mgd, such as occurred in September 2019. Even using the lower peak flow of 9 mgd, which occurs more than twice per year on average, the resulting peaking factor is 9, which is more excessive than in the Seymour or Ansonia systems. However, Derby has also initiated significant improvements in the collection system starting in 2019, which are expected to yield a reduction in peak inflows.

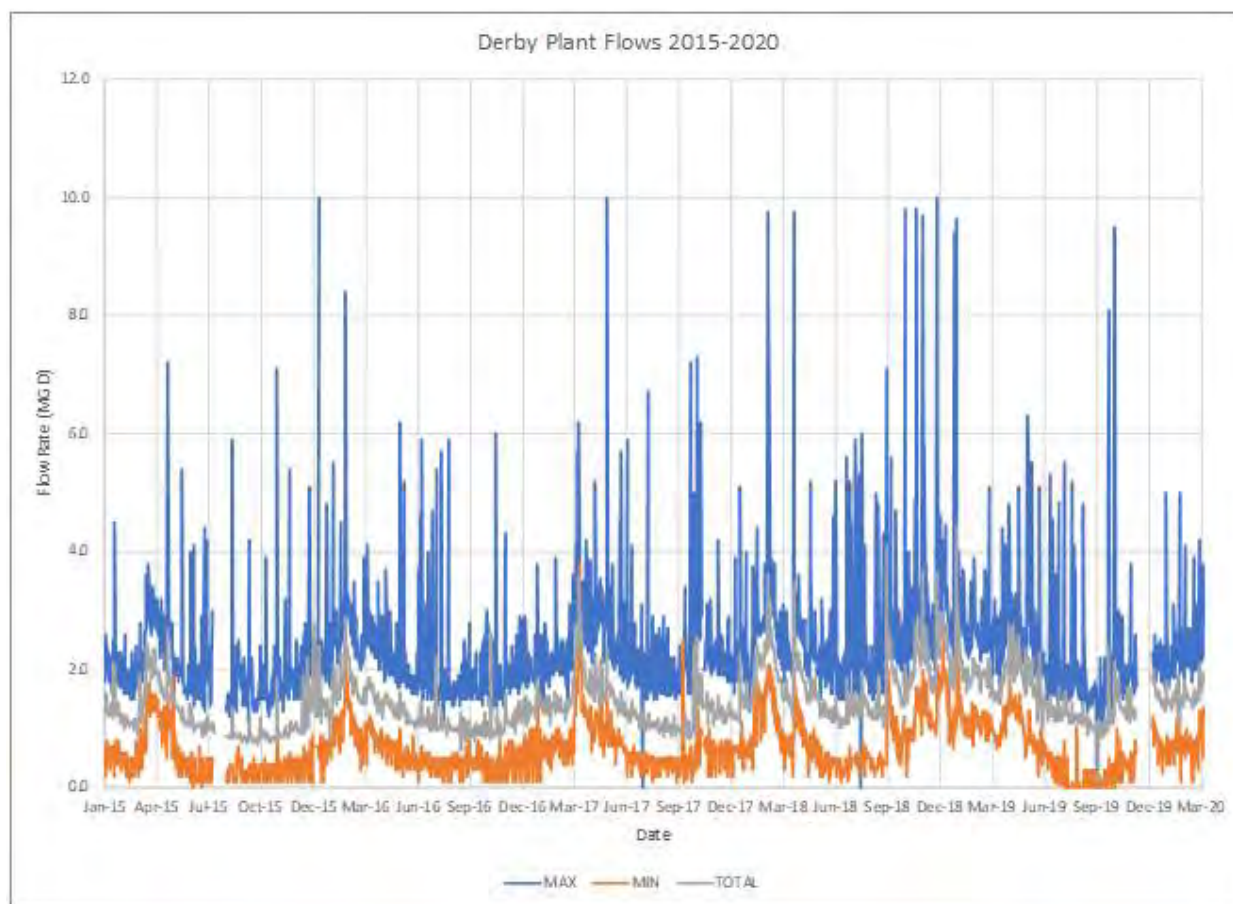


Figure 2-1 Derby Plant Flows 2015-2020

2.3.2.2 Ansonia

Daily flow data (maximum, minimum, average) from monthly operating reports from 2015 into 2020 were analyzed to characterize infiltration and inflow to the extent possible (Figure 2-2). Wetter conditions starting in 2018 are clearly identifiable in the figure, as well as a notable increase

in erratic maximum flow data, which appears to be due to the plant's influent pump station activating more regularly.

For the most recent five-year period starting in 2015, the peak flow was recorded on April 3, 2017 at 6.91 mgd. Thirty-six days were recorded with peak flows greater than 6 mgd, and there were many more instances where incoming flows to the plant were greater than 5.5 mgd. A quick review of documents from before 2015 indicate that even higher peak flows have occurred within Ansonia's system. This will be reviewed further.

Average daily flow during dry conditions is approximately 1.1 mgd, as indicated during September 2015-2017 and 2019. At 6.3, the ratio of peak flow to average dry weather flow is indicative of excess flow in the collection system and the need for significant I/I reduction. However, the data do not provide any information about the sources of I/I in terms of location or of defect type (e.g. infiltration vs. inflow). Given the high peaks that are sustained in 2018, it appears that both infiltration and inflow are significant.

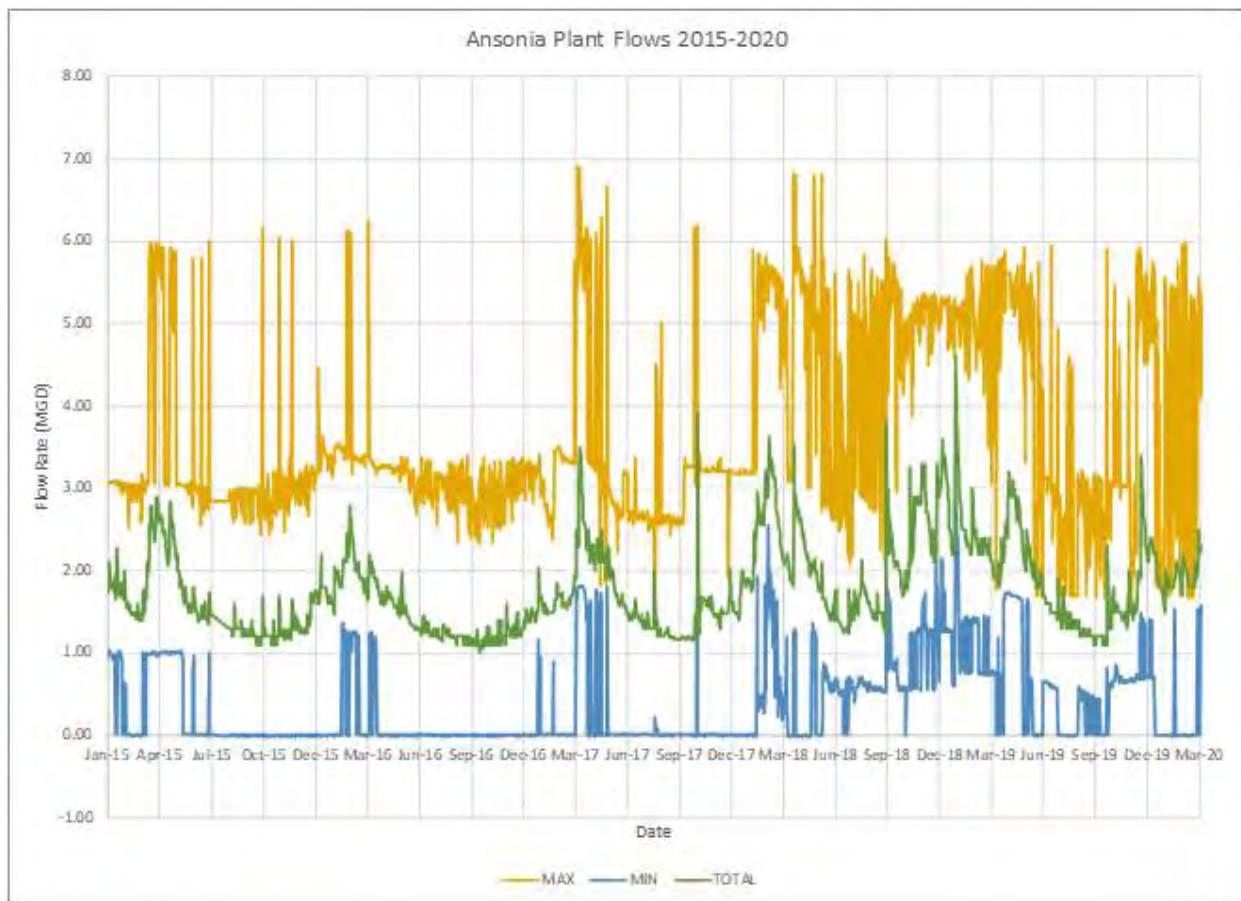


Figure 2-2 Ansonia Plant Flows 2015-2020

2.3.2.3 Seymour

Daily flow data (maximum, minimum, average) from monthly operating reports from 2015 into 2020 were analyzed to characterize infiltration and inflow to the extent possible (Figure 2-3). The

peak flow of 7.0 mgd was recorded on February 28, 2016 and was by far the highest recorded flow. Four events had a peak flow of at least 5.0 mgd during the 2015 through early 2020.

Average daily flow varied significantly during the analysis period, but it appears that average dry weather flow could be approximately 0.8 mgd, such as occurred in September 2019. Using the more common peak flow of 5 mgd, the resulting peaking factor is 6.3, which is considered excessive. Given the relatively lower frequency of peak flows, it appears that infiltration may be more significant in Seymour, but inflow is still likely a significant factor.

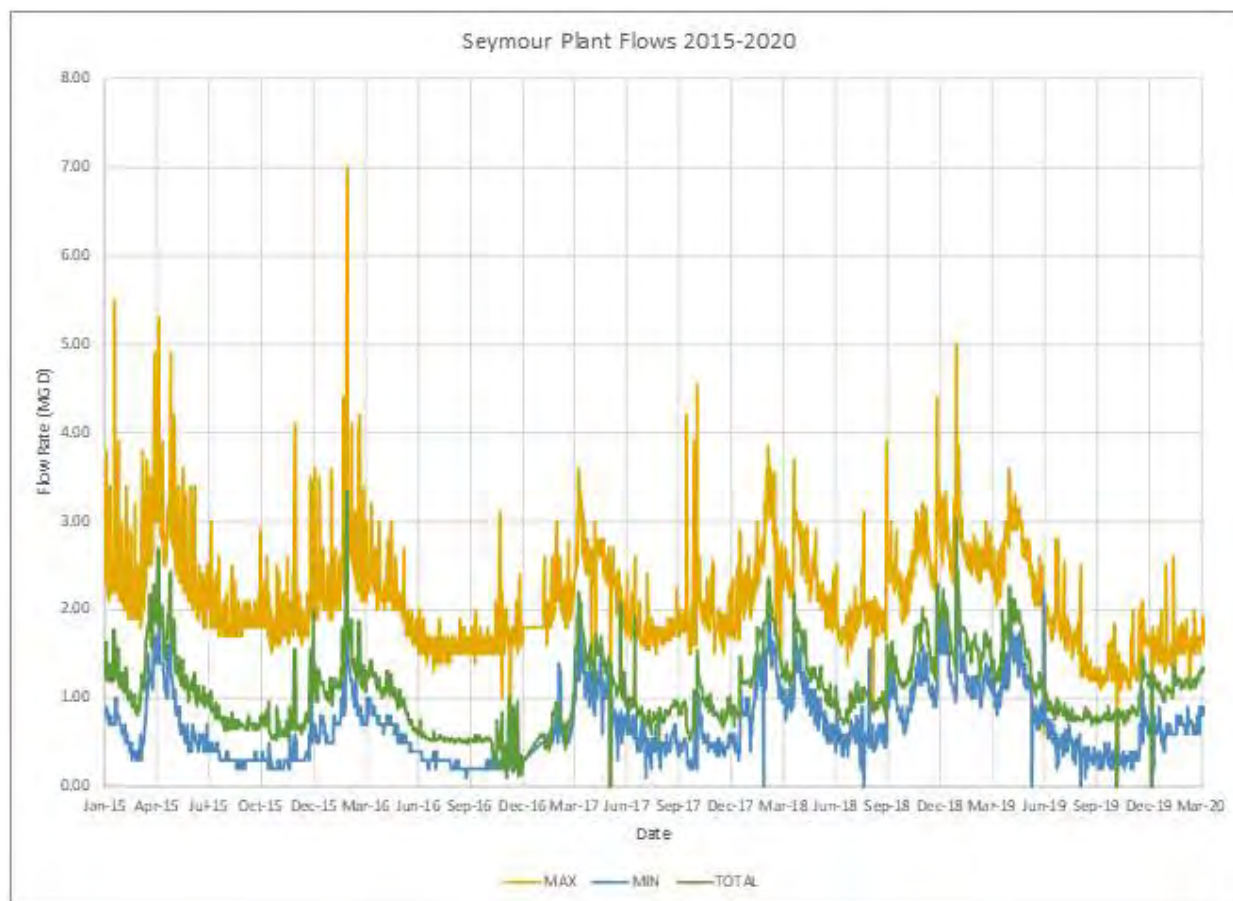


Figure 2-3 Seymour Plant Flows 2015-2020

2.3.2.4 Beacon Falls

Three years of MOR data from 2015-2018 were reviewed for the Town of Beacon Falls water pollution control facility. However, this represented a period of below-average rainfall. Therefore, existing condition wastewater flow values provided by the 2015 Wastewater Facilities Plan, which were based on a wetter period (September 2009 to October 2012) were determined to be more appropriate to use in this study, since they are more representative of longer-term weather patterns. Average daily flow was found to be 0.36 MGD, with a peak hour flow of 1.24 MGD.

2.3.2.5 Naugatuck

Daily flow data (maximum, minimum, average) from monthly operating reports from 2010 through early 2020 were analyzed to characterize infiltration and inflow to the extent possible (Figure 2-4). The peak flow of 25.0 mgd was recorded on August 28, 2011. Flows, particularly peak flow, in the years 2010-2011 were substantially higher than the remainder of the monitoring period. Nine events had a peak flow of at least 20.0 mgd during the 2010-2020 period.

Average daily flow varied significantly during the analysis period, but it appears that average dry weather flow could be approximately 3.3 mgd, such as in September 2019. Using the more recent peak flow of 21.2 mgd observed on October 30, 2017, the resulting peaking factor is 6.4, which is considered excessive.

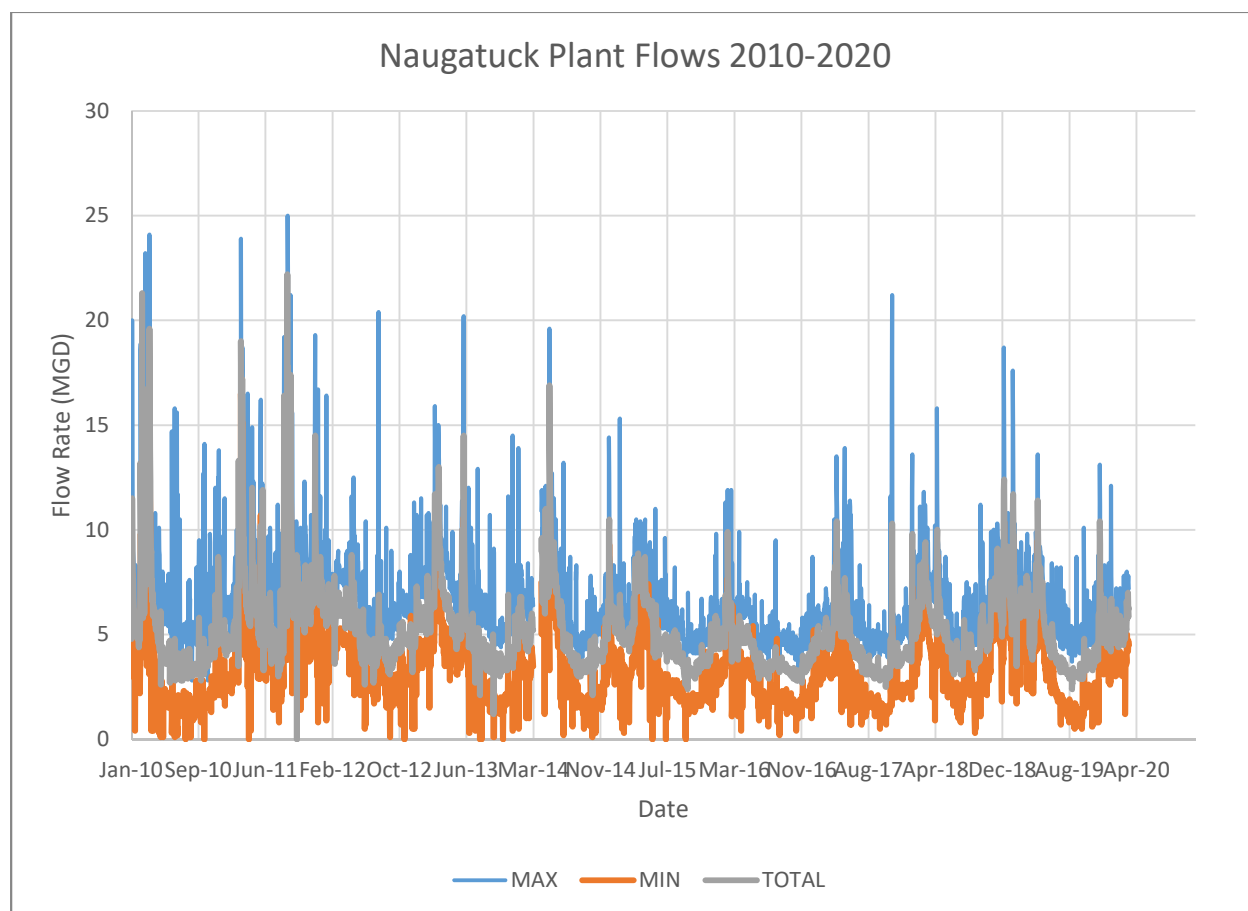


Figure 2-4 Naugatuck Plant Flows 2010-2020

2.3.3 Flow Monitoring

Flow monitors were installed in eight locations in Ansonia, Derby, and Seymour in April 2020 as part of this regionalization study. These flow monitors are still collecting data, so it is premature to draw conclusions from this data. This data will be used in the follow-on task where the short-listed regional alternatives will be studied further.

2.4 I/I REMOVAL

2.4.1 I/I Program Development

I/I programs are a standard part of wastewater management and are cost-effective at managing flows to the wastewater treatment plant over time. Implementation of an I/I program typically takes place in phases and over time – it is not uncommon that 10 or more years is required to fully implement community-wide I/I program, and I/I removal activities then continue indefinitely. I/I control results can be elusive due to the wide range of potential sources and environmental conditions, as well as the variety of control measures that can be implemented. Therefore, a strong commitment by the municipality to stay with the program is required. This is particularly the case as guideline assumptions of I/I removal may be optimistic, depending on the circumstances, and additional control may be required. This may occur for a variety of reasons, such as:

- Monitoring and SSES activities may not have identified all sources of inflow and infiltration, e.g. due to drier-than-normal conditions.
- Construction methods may not adequately seal the pipes, manholes, and related structures in the collection system to prevent I/I, or leaks that were sealed as part of the program may migrate to other cracks that were not producing leaks initially;
- Private I/I sources can be difficult to identify and control, and they may contribute a greater proportion of I/I than original estimates.

For these reasons and more, post-rehabilitation monitoring is important. The results characterize the effectiveness of I/I removal efforts and provide a basis for projecting future results. The results of post monitoring may also re-prioritize the capital plan and/or require additional testing prior to more implementation.

2.4.2 Aggressive I/I Control

Aggressive I/I control is described as the implementation of measures to remove additional I/I beyond what would typically be recommended based on standard cost-effectiveness analysis. This is particularly applicable in cases where treatment capacity may be limited, requiring major plant improvements or other measures, in which case, more aggressive I/I control can be cost effective.

Strategies employed in aggressive I/I control are most often applied after a conventional I/I program has been implemented and additional removal is desired. Measures employed in aggressive I/I include the following:

- Rehabilitating private laterals
- Comprehensive rehabilitation or replacement (vs. point repair)
- Standards for new pipe, repair, and replacement
- Regular monitoring and assessment

While it is clear that employing aggressive I/I control may remove substantial I/I from the collection system, it is difficult to predict the degree of I/I removal and the ultimate success of the program. It is often estimated that rehabilitation will remove 50% of the targeted I/I. This is often over-stated and is not always verified. The effectiveness of I/I removal programs in practice has varied from 0% to 90% or more, with comprehensive rehabilitation programs (including

private laterals) showing the greatest success. Given the uncertainty of success, as well as the uncertainty of flows, it is recommended to proceed by collecting data to better understand the flows in each system, and then to determine the appropriate level of I/I to target. An aggressive I/I control program is not considered to be reliably predictable for system planning at this time.

Furthermore, aggressive I/I control is most often undertaken in the context of reducing system flows to maximize existing treatment plant capacity and defer the need for plant expansion, whether to accommodate growth or due to existing capacity constraints. It is not typically cost-effective to undertake an aggressive I/I program in parallel with plant improvements. Given that treatment plant improvements will be required for a majority of the regional alternatives, and the lack of predictable level of control that aggressive I/I can achieve in the study communities, it is recommended that the eleven regional alternatives with aggressive I/I control be eliminated from further study at this time.

2.5 REGIONAL ALTERNATIVES SCREEN-OUT BASED ON AGGRESSIVE I/I

Each one of the five community plants included in this study will need improvements regardless of changes in flows and wastewater characteristics associated with regionalization. Therefore, regional alternatives which include aggressive I/I control measures were screened out from further evaluation as shown in Table 2-1. It is recommended that community-wide I/I programs be undertaken in all five of the communities, realizing that some of these are already underway. The results of these programs need to be regularly monitored. This will allow the communities to reevaluate the need and degree to implement aggressive I/I mitigation measures.

Table 2-1 Alternatives Screen-out Based on Aggressive I/I Evaluation

No.	Alternative Description
1	Beacon Falls to Naugatuck
2	Beacon Falls to Seymour
2a	Beacon Falls to Seymour, I/I Reduction
3	Derby to Ansonia
3a	Derby to Ansonia, I/I Reduction
4	Derby to Ansonia, Effluent Pumped to Housatonic River
4a	Derby to Ansonia, I/I Reduction, Effluent Pumped to Housatonic River
5	Derby and Seymour to Ansonia
5a	Derby and Seymour to Ansonia, I/I Reduction
5b	Derby and Seymour to Ansonia, Effluent Pumped to Housatonic River
5c	Derby and Seymour to Ansonia, I/I Reduction, Effluent Pumped to Housatonic River
6	Derby to Seymour and Ansonia
6a	Derby to Seymour and Ansonia, I/I Reduction
8	Ansonia to Derby
8a	Ansonia to Derby, I/I Reduction
9	Seymour and Ansonia to Derby
9a	Seymour and Ansonia to Derby, I/I Reduction
10	Seymour to Ansonia, Part of Ansonia to Derby
10a	Seymour to Ansonia, Part of Ansonia to Derby, I/I Reduction
11	Beacon Falls and Seymour to Ansonia, Part of Ansonia to Derby
11a	Beacon Falls and Seymour to Ansonia, Part of Ansonia to Derby, I/I Reduction
12	Beacon Falls, Seymour, and Ansonia to Derby
12a	Beacon Falls, Seymour, and Ansonia to Derby, I/I Reduction

3.0 CONVEYANCE CORRIDOR EVALUATION

3.1 BACKGROUND

The conveyance corridors identified in Phase 1 were conceptual in nature and served to connect communities, allowing wastewater system regionalization alternatives to be visualized. Some of the corridors included multiple possible routes to transport wastewater between the joining communities.

A closer study of the conveyance corridors has been undertaken to better understand the routes that are least implementable and that should be removed from further consideration. Of the routes that appear more implementable, additional planning level definition were provided, including approximate pipe size range and length, environmental concerns, possible construction methods, need for pumping, and easement/right-of-way issues.

3.2 INITIAL ROUTE IMPLEMENTABILITY REVIEW

Several of the corridors from Phase 1 included possible routes in the right-of-way (ROW) of railroads and Route 8. One of the routes was in the Eversource overhead transmission line ROW through the Naugatuck State Forest. These routes were mainly located between Beacon Falls and Naugatuck. Because of the conveyance corridor length and steep topography between these two communities, it was thought that routes requiring less development and that align with other infrastructure (e.g. a railroad) would be the most feasible. Locating the sewer line in an existing ROW offers practical engineering, construction, and maintenance solutions for service of the sewer pipeline.

Upon more detailed review, it is believed that installation of a sewer line within the ROWs of the railroad, Route 8, or the Eversource overhead transmission line will be very difficult to implement if feasible, and possibly not implementable. Uncertainties about the long-term viability of these ROWs is also cause for concern even where permission may be obtained in the near term. Therefore, pipeline routes along the highway, railroad and utility ROWs were not developed further as a part of this task. Summaries of these route reviews are described below.

Routes which do not require extensive ROW access were considered implementable for the purpose of this evaluation and were developed further.

3.2.1 Route 8 ROW

The Connecticut Department of Transportation (CT DOT) places high priority on safety. The CT DOT also does not want anything to negatively impact the traffic-carrying ability or the physical integrity of its highways. A buried pipeline within the highway's ROW can be seen to affect these characteristics. While buried pipelines have been allowed in state highway ROWs, there is also a history of pipeline utilities having to move and relocate their pipelines at their own expense when highway projects are required. Additionally, longitudinal pipelines along and within a ROW, identified as possible routes in Phase 1, are much more difficult to obtain approval for as compared to a utility line that crosses the highway. Connecticut regulations state that utility authority to use a highway ROW is subject to approval by the state Transportation Commissioner, noting that "if in the opinion of the Transportation Commissioner, it becomes necessary at any time to remove or

relocate any structure installed under permit, the removal or relocation upon notification by the Commissioner or his agent shall be made immediately”.

3.2.2 Railroad ROW

CTrail places high importance on safety, and a buried pipeline is generally viewed as a compromise to the railroad’s infrastructure and operations. From experience, obtaining permission to install a sewer line within the railroad ROW for the length required in this study (up to 25,000 ft) is very unlikely. Additionally, much of the Waterbury Branch of the Metro-North railroad is aligned along the Naugatuck River and borders protected open space which represents additional wetland permitting, flood control, and potentially the need for access roads for construction and maintenance.

3.2.3 Eversource High Transmission Line ROW

Our experience in New England is that it is generally very difficult to obtain approval to install a buried pipeline with the ROWs of electrical utility companies. The following points are made in this regard.

- Existing ROWs, especially in Connecticut and other parts of New England, are limited for future growth.
- Acquisition of new ROWs can draw a lot of public backlash; therefore, utilities have limited options and try to focus on future system upgrades within their existing corridors.
- Even if existing ROWs have available space for third party use, it is likely the utility will resist this option and try to hold out for any future opportunities (e.g. renewable tie-ins).
- Having a third party within a ROW provides added risk to the utility and limits future potential.
- There is a substantial application, review, and approval process associated with obtaining permission to use electrical transmission ROWs. Even when an agreement can be reached, it will take a long time. Electrical utilities can decide to de-commission and sell off that portion of its system for any reason; in these scenarios, a shared ROW with a buried pipeline would lessen the value of the electrical company’s asset.

3.3 ASSESSMENT OF IMPLEMENTABLE ROUTES

Some of the routes identified in Phase 1 along with, some new routes were developed as part of this task. Detail on the routes was obtained from existing sources including: State of Connecticut GIS data, aerial images, and the United States Fish and Wildlife Service (USFWS) National Wetland Inventory (NWI). The GIS data base was a key source, providing information on:

- Aquifer protection areas
- USFWS critical habitat and protected open space
- State wetlands
- FEMA flood zones
- Parcel data

- Topography

After the pipeline routes were established, the overall length of the pipelines was identified and the pipe sizes were calculated.

A summary of the pipeline routes/conveyance corridors that will connect the various regional alternatives is presented below.

3.3.1 Ansonia to/from Derby

Figure 3-1 shows the pipeline route/conveyance corridor that will connect Ansonia and Derby. This route would be the same for regional alternatives where Derby wastewater flows to Ansonia or vice versa for alternatives where Ansonia wastewater is discharged to Derby. Because of topography, the flow from either Derby to Ansonia or from Ansonia to Derby would need to be pumped part of the way. The pump stations would be situated on the sites of the treatment plants of the two cities.

For the most part, the pipeline would be routed in city streets but can be situated in nearby adjacent streets if desired. The alignment shown appears to have less topographical challenges as compared to other nearby routes. Additional feasibility analysis of this route will be undertaken as part of the follow-on short-list development task.

It is also noted that the pipeline which carries treated effluent from the Ansonia plant back to Derby for discharge to the Housatonic River at Derby's current discharge outfall, will also be situated in the same corridor as described above and as depicted on Figure 3-1. Table 3-1 summarizes some pertinent features of the pipeline route shown in Figure 3-1.



Figure 3-1 Ansonia to/from Derby Pipeline Route

Table 3-1 Ansonia to/from Derby Pipeline Routes Overview

		Derby to Ansonia	Ansonia to Derby
Physical Attributes	Total length (ft)	8,100	
	Pipeline	Force main	Gravity and force main
	Diameter (in)	18	16 and 18
	Pump stations	1	
	High point elevation (ft)	37	
Environmental	Within 100-year flood plain	Entire route outside flood plain.	
	Protected area impacts	None identified.	
	Within wetland buffer	Entire route outside wetland buffer.	
Easements/Private Land Taking	Private parcels	Approx. 20% of the route crosses private parcels.	

3.3.2 Seymour to Ansonia

Figure 3-2 shows the pipeline route/conveyance corridor that will connect Seymour to Ansonia. Because of topography, the flow would need to be pumped part of the way. Two pump stations are identified. One of these will be at the Seymour plant site and the other would be a lift station along the route in Ansonia.

The pipeline is largely routed in town/city streets. The pipeline can be routed in nearby adjacent streets to those shown on Figure 3-2 if desired. Additional feasibility analysis of this route will be undertaken as part of the follow-on short-list development task. Table 3-2 summarizes some pertinent features of the pipeline route shown in Figure 3-2.

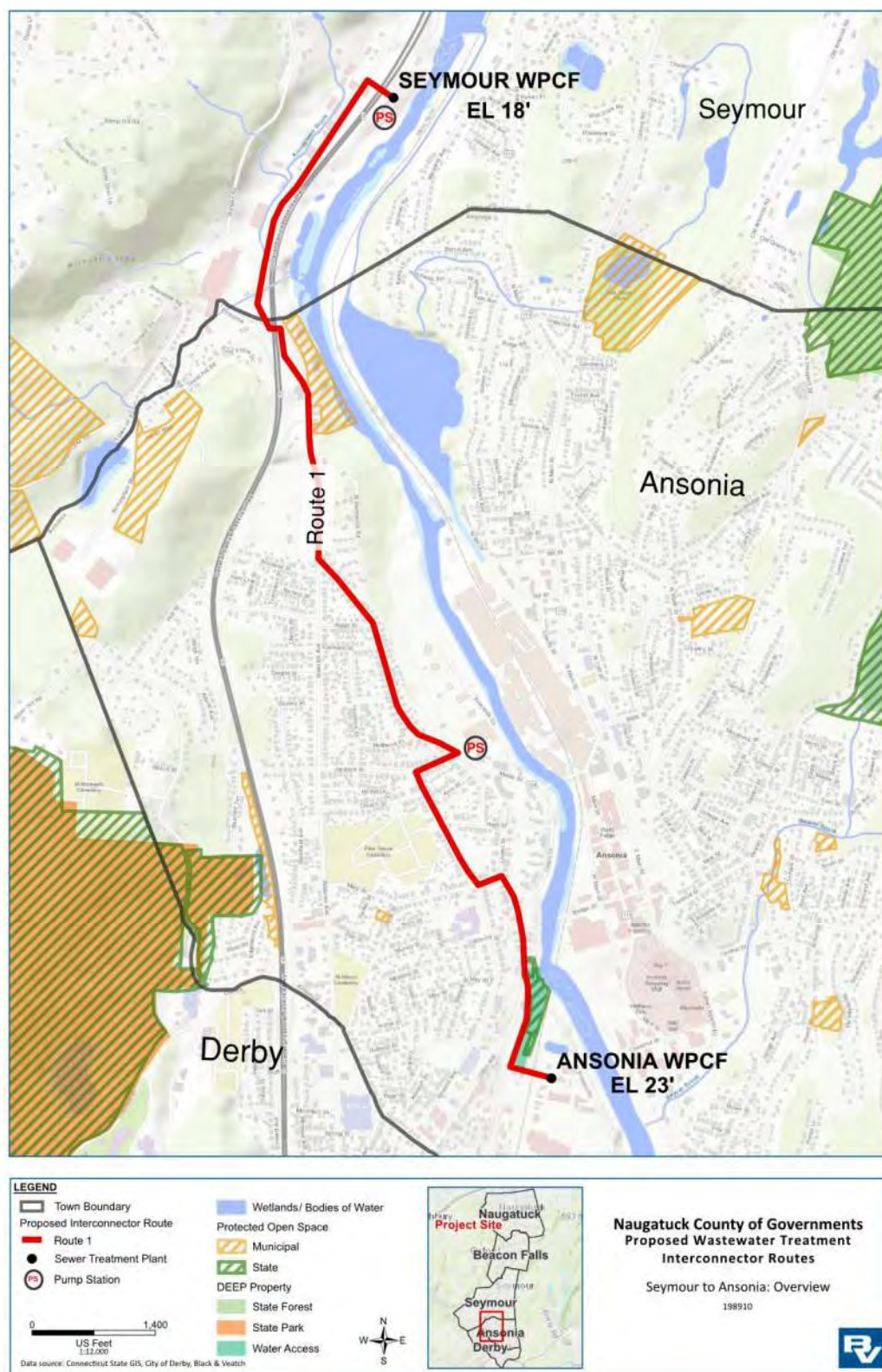


Figure 3-2 Seymour to Ansonia Pipeline Route

Table 3-2 Seymour to Ansonia Pipeline Route Overview

Route 1		
Physical Attributes	Total length (ft)	14,200
	Pipeline	Gravity and force main
	Diameter (in)	14 to 18
	Pump stations	2
	High point elevation (ft)	124
Environmental	Within 100-year flood plain	Generally not, except for two brook crossings.
	Protected area impacts	Borders two protected open space areas for small portions of the route.
	Within wetland buffer	Generally not, except for two brook crossings.
Easements/Private Land Taking	Private parcels	Approx. 10% of the route crosses private parcels.

3.3.3 Beacon Falls to Naugatuck

Figure 3-3 shows the pipeline route/conveyance corridor that will connect Beacon Falls to Naugatuck. The topography in this area is steep with large elevation changes in short distances, making this pipeline corridor very challenging to implement with significant cost implications. Two routes are offered to connect these two communities. Table 3-3 summarizes some pertinent features of the pipeline route shown in Figure 3-3.

Both routes begin with pump stations at the Beacon Falls WPCF and move north into the state forest just past the Naugatuck corporate boundary. From there the Route 1 alignment swings west through the state forest and follows a path along the base of Toby's Rock mountain before it leaves the state forest and follows Naugatuck roads generally north and east to the WPCF. This route is roughly 5.3 miles in length and will require a total of five pump stations. Three of these pump stations will be in the state forest and will require electric power feed supplied to them. A maintenance road will also be required to allow for regular inspection and maintenance of these pump stations and the pipeline.

From its split with Route 1, the Route 2 alignment proceeds straight in a northeast direction to the Naugatuck WPCF; however, because of the extremely high terrain, the pipeline will need to be tunneled for this section. The tunnel will be deep in the rock and is estimated to be roughly 7 to 8 feet in diameter. It will also take two deep shafts to build. While the overall length of Route 2, at 3.2 miles, is significantly shorter than Route 1, the tunnel will be expensive to construct and would make the Route 2 alignment prohibitive to implement. Route 2 would also require three pump stations. Both routes would require close coordination with the state because of their alignment in the state forest. Route 2 is envisioned to be less disruptive than Route 1 because the section through the forest will be tunneled.

As noted, both wastewater pipeline routes connecting Beacon Falls and Naugatuck will require multiple pump stations. Several of these pump stations will be situated in the state forest and will

be difficult to get to. Even with multiple equipment redundancy including dual electrical power feeds, the overall reliability of these two long pipeline routes is of significant concern.

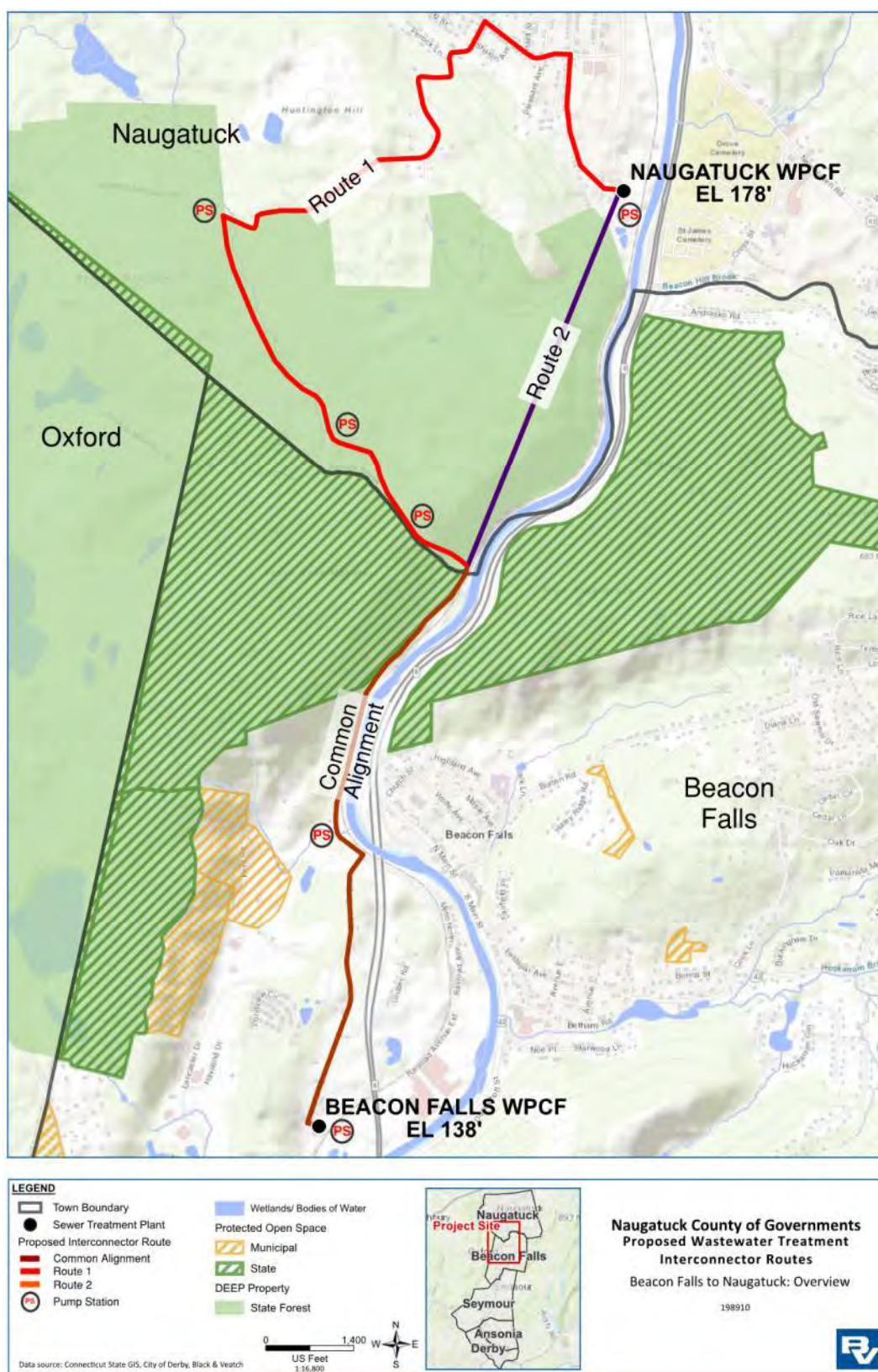


Figure 3-3 Beacon Falls to Naugatuck Pipeline Routes

Table 3-3 Beacon Falls to Naugatuck Pipeline Routes Overview

		Route 1	Route 2
Physical Attributes	Total length (ft)	28,100	16,500
	Pipeline	Gravity and force main	Gravity, force main, and tunnel
	Diameter (in)	10 and 12	10 and 12
	Pump stations	5	3
	High point elevation (ft)	737	608
Environmental	Within 100-year flood plain	Generally not, except for four brook crossings.	Generally not, except for three brook crossings.
	Protected area impacts	Borders protected areas for small portion of route.	
	Within wetland buffer	Generally not, except for brook crossings.	
Easements/Private Land Taking	Naugatuck State Forest	Approximately 65% of route through state forest.	

3.3.4 Beacon Falls to Seymour

Figure 3-4 shows two possible pipeline routes/conveyance corridor that will connect Beacon Falls to Seymour. The distance between these two plants is relatively far and the topography, while not nearly as steep as that between Beacon Falls and Naugatuck, is still challenging. As a result, the pipeline routes will require multiple pump stations. Table 3-4 summarizes some pertinent features of the pipeline route shown in Figure 3-4.

Both routes begin with pump stations at the Beacon Falls WPCF and head south on town roads in Beacon Falls and Seymour. After roughly 14,500 feet, the two routes split. The common alignment section will require two additional pump stations as a result of steep topography. From the split, Route 1 follows town roads south. As it approaches the Seymour plant, the pipeline route turns east, traversing private property prior to getting jacked/bored under Route 8 to the plant site.

From its split with Route 1, the Route 2 alignment turns east slightly and then proceeds straight in a southerly direction to the Seymour plant; however, because of the extremely high terrain, the pipeline will need to be tunneled in this section. The tunnel will be deep in the rock and is estimated to be roughly 7 to 8 feet in diameter. It will also take at least two deep shafts to build. While the overall length of Route 2 is roughly 4,500 feet shorter than Route 1 and has two fewer pump stations, the tunnel will be expensive to construct. Still, the cost of the tunnel would make Route 2 not feasible.

As noted, both wastewater pipeline routes connecting Beacon Falls and Seymour will require multiple pump stations, with Route 1 requiring six pump stations. While these pump stations are significantly more accessible for regular maintenance than those in the state forest on the Beacon Falls to Naugatuck corridor, equipment and electrical power supply redundancy are required to provide sufficient reliability to the pipeline, pump stations and related facilities.

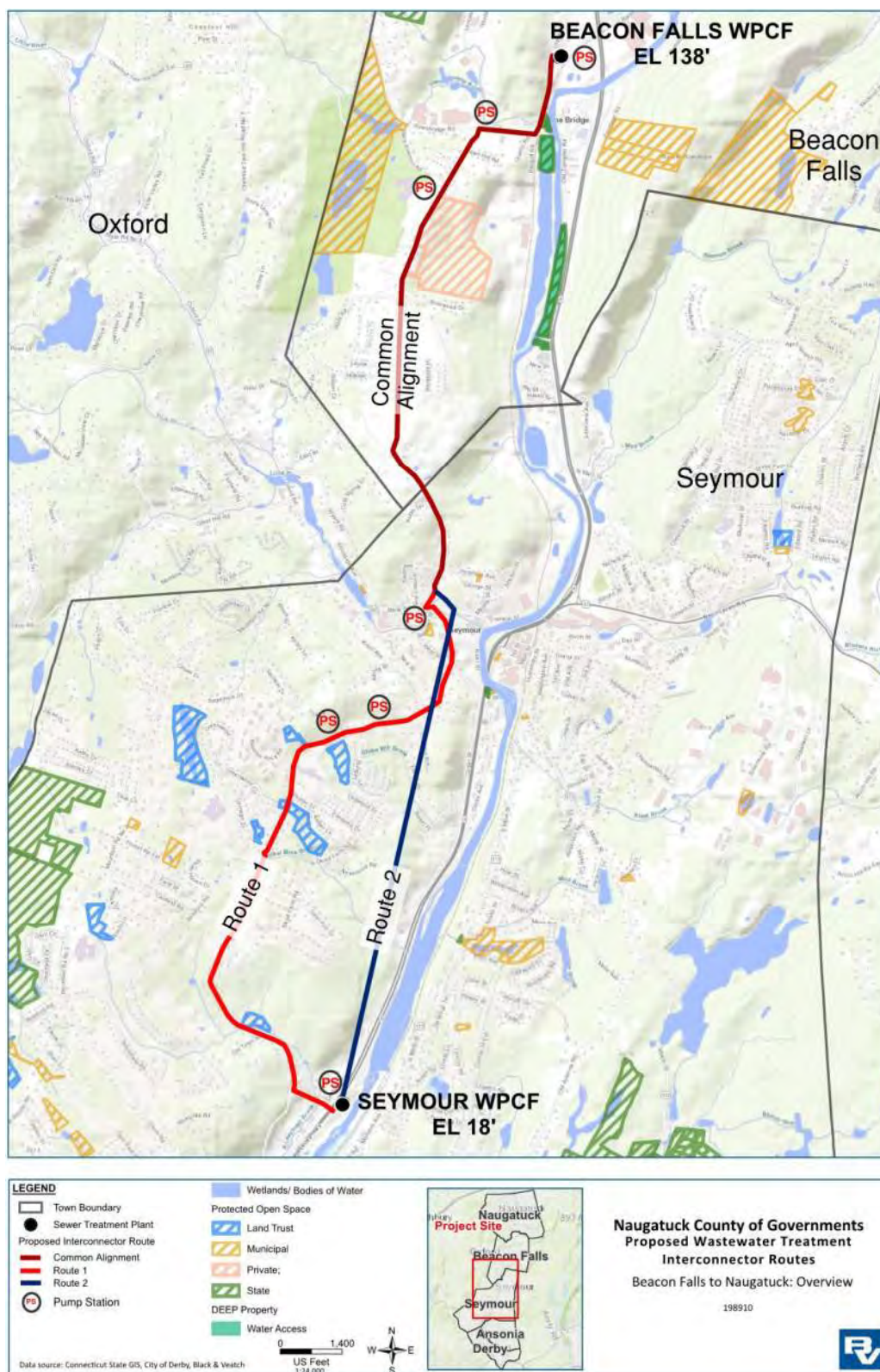


Figure 3-4 Beacon Falls to Seymour Pipeline Routes

Table 3-4 Beacon Falls to Seymour Pipeline Routes Overview

		Route 1	Route 2
Physical Attributes	Total length (ft)	31,000	26,500
	Pipeline	Gravity and force main	Gravity, force main, and tunnel
	Diameter (in)	10 and 12	10 and 12
	Pump stations	6	4
	High point elevation (ft)	500	463
Environmental	Within 100-year flood plain	Generally not, except for six brook crossings.	Generally not, except for four brook crossings.
	Protected area impacts	Borders protected areas for small portion of route.	Borders one protected open space parcel.
	Within wetland buffer	Generally not, except for brook crossings.	
Easements/Private Land Taking	Private parcels	Approx. 5% of the route crosses private parcels.	Approx. 45% of the route crosses private parcels.

3.4 ALTERNATIVES SCREEN-OUT BASED ON CONVEYANCE CORRIDORS ASSESSMENT

Upon review of implementable conveyance routes, it was determined that pipeline and pump station systems required to transfer wastewater from Beacon Falls to either Naugatuck or Seymour would be too costly on a capital cost basis. Additionally, these raw wastewater pipelines, with multiple pump stations are not considered to be sufficiently reliable to work in an uninterrupted manner on a regularly basis as would be expected. For these reasons, regional alternatives which include conveyance from Beacon Falls were screened out, shown in Table 3-5 along with the alternatives developed further in the plant process and site layout evaluation step.

Pipe routes for the other regional alternatives including Seymour, Ansonia, and Derby appear feasible from a conveyance basis. Those regional alternatives were evaluated further from a treatment plant facility perspective.

Table 3-5 Alternatives Screen-out Based on Conveyance Evaluation

No.	Alternative Description
1	Beacon Falls to Naugatuck
2	Beacon Falls to Seymour
3	Derby to Ansonia
4	Derby to Ansonia, Effluent Pumped to Housatonic River
5	Derby and Seymour to Ansonia
5b	Derby and Seymour to Ansonia, Effluent Pumped to Housatonic River
6	Derby to Seymour and Ansonia
8	Ansonia to Derby
9	Seymour and Ansonia to Derby
10	Seymour to Ansonia, Part of Ansonia to Derby
11	Beacon Falls and Seymour to Ansonia, Part of Ansonia to Derby
12	Beacon Falls, Seymour, and Ansonia to Derby

4.0 PLANT PROCESS AND SITE LAYOUT EVALUATION

4.1 BACKGROUND

Each of the regional alternatives involve at least two communities combining their wastewater treatment needs and a few of the regional alternatives involve three or more communities with combined treatment facilities. In order to identify the short-listed regional alternatives, these scenarios were evaluated from a plant process and site layout perspective to determine which alternatives appear implementable. During Phase 1, flow and load projections were developed based on projected population growth and available monthly operating report (MOR) data received from each of the plants. Because of the time elapsed since the majority of the Phase 1 work was accomplished, additional MOR data from several of the communities was compiled and reviewed together with the previous plant data obtained during Phase 1. The following items summarize the work performed as part of treatment facility and site development review.

1. Wastewater flows and loads data was updated and revised with additional MOR data available since Phase 1 was completed; this included data for the years 2018, 2019, and the first few months of 2020.
2. Plant data was analyzed using wastewater process methodologies to determine capacity and treatment facility requirements for individual plants and for each of the regional alternatives under consideration.
3. Process analysis resulted in conceptualized site layouts for each individual plant and the regional alternatives being considered. The development of the treatment plants allowed for additional perspective into which of the remaining regional alternatives should be eliminated now and which should move forward as the short-listed regional alternatives.

Alternatives that appear feasible from a plant process and site layout basis would be considered for the short list of regional alternatives.

4.2 PLANT DATA

Flows and loads data was updated to support the plant process evaluation. To achieve more realistic peaking factors of combined regional collection systems in the southern area of the Naugatuck Valley (i.e. Derby, Ansonia, and Seymour), daily MOR data was considered for these plants concurrently for at least 3 years of data when calculating influent flows and loads. This is particularly relevant for loads, as the peaking factors for the combined catchments would be lower than the peaking factors for individual catchments. This analysis was not done however for Beacon Falls as its contribution to the total Seymour or Naugatuck flows and loads is relatively low. In this data analysis for facility capacity, the annual average, max month (calendar), and peak day flows and loads were calculated as these are the critical parameters typically used in calculating treatment plant capacity. This is slightly different than the parameters analyzed in the I/I evaluation which focused on average dry weather flow and instantaneous maximum flows. This data is summarized in Appendix A of this TM.

It is noted that Derby is in the process of revising their future plant flow and loadings projections as part of updating their Facilities Plan. Preliminary indications are that these projections will be lower than those in the draft 2014 Facilities Plan. These new projections will be reviewed

in subsequent work tasks, particularly in light of treatment facility and conveyance requirements for Derby.

4.3 TREATMENT CAPACITY ASSESSMENT

A planning level process capacity review of the plants being considered for regional treatment was performed. This review focused mainly on the capacity of the major primary and secondary treatment units to processes the required flow and loads, and the need for new major tertiary process units to meet effluent permit limits. Primary and secondary treatment process units are footprint intensive; therefore, the evaluation focused on these to assess the feasibility of the existing sites to treat the required flow. Where existing unit process tankage is not adequate, the addition of major process units equal to existing units were considered with and without intensification alternatives. This assessment was performed with spreadsheet based steady state models and did not include a detailed assessment of the impact of the proposed loading changes and upgrades on BNR process performance. The assessment also did not include preliminary treatment processes, disinfection processes, residuals management/treatment, or general hydraulics. Some or all of these will be considered in greater detail as part of the short-listed alternatives in subsequent task work.

The treatment capacity assessment undertaken in this TM is described in Appendix B. The principal results and conclusions of that assessment is summarized below. There are numerous treatment technology terms and acronyms used in the planning level process assessment in Appendix A. These terms are also used in the summary below. As such, we provided a table of the technical terms and their acronyms in Table 4-1.

Table 4-1 Common Wastewater Process Abbreviations

AB	Aeration basin	N	Nitrogen
BioMag	Ballasted activated sludge (Evoqua BioMag®)	P	Phosphorus
BNR	Biological nutrient removal	PF	Primary filtration
BOD	Biological oxygen demand	PST	Primary settling tank
cBOD	Carbonaceous biological oxygen demand	SLR	Solids loading rate
CAS	Conventional activated sludge	SOR	Surface overflow rate
CEPT	Chemically enhanced primary treatment	SPA	State point analysis
HRT	Hydraulic residence time	SST	Secondary settling tank
IFAS	Integrated fixed film activated sludge	SVI	Sludge volume index
MLSS	Mixed liquor suspended solids	TKN	Total Kjeldahl nitrogen
MOR	Monthly operating report	TSS	Total suspended solids

4.4 CONCEPTUAL SITE LAYOUTS

Following the process analysis and associated facility requirements, conceptual wastewater plant site layouts were developed for the regional alternatives under consideration. The site layouts were developed to meet wastewater treatment requirements while balancing existing site constraints. These site layouts were used to approximate the feasibility of incorporating upgraded and new facilities on existing sites associated with the regional alternatives being considered. Development of the conceptual site plans were based on the following objectives and assumptions:

1. Major treatment facilities were placed entirely within plant parcels, assuming no additional land or easement acquisition would be required.
2. Preference was given to facility options that stayed within existing plant fence lines to minimize disruption and reduce the potential for property setback challenges and existing easement limit issues.
3. Conventional treatment methods were assumed where possible to implement. Chemically enhanced treatment and intensification options were selected for particular sites and regional alternatives where conventional treatment options appeared infeasible.
4. Facility layouts and descriptions were limited to major primary and secondary treatment technology and infrastructure required to meet treatment needs. Other treatment processes (preliminary treatment, pumping, effluent disinfection and residuals treatment/management) and related major support systems and equipment will be further defined for the short-listed alternatives in a subsequent task of this study.

The following subsections show the site layouts for the different regional alternatives under consideration. The overall planning level treatment requirements of the base case is also depicted to communicate the absolute minimum treatment facility needs if each community continues to go alone without regionalization. This work will continue to be refined for the short list regional alternatives in a subsequent work task.

4.4.1 Derby Conceptual Site Layouts

The Derby WPCF was originally constructed in 1964 and was upgraded to secondary treatment in 1973 with few significant upgrades since that time. Overall, the plant is old and needs a major overhaul if not near complete replacement of major treatment systems. Its major liquid treatment processes include two primary clarifiers, three MLE aeration basins (two active, one inoperable), and two secondary clarifiers. The plant is compact within the fence line, and the triangular site is confined on each side by a railroad to the north, route 8 to the southeast, and the Housatonic River levee to the southwest. Because of the confined nature of the site, treatment intensification options were considered for the Derby WPCF to increase treatment capacity within the existing parcel.

The site layout for Derby only (base case) is shown in Figure 4-1. This arrangement requires modification of the existing primary settling tanks to operate with CEPT and upgrade of the existing inoperable aeration basin.



Figure 4-1 Treatment Facility Requirements for Derby Only

The site layout for the Derby plus Ansonia regional alternative is shown as two options. Figure 4-2 shows the arrangement with BioMag and requires modification of the existing primary settling tanks to operate with CEPT, upgrade of the inoperable existing aeration basin, addition of one new aeration basin, and upgrades to add a magnetite feed and recovery system. Figure 4-3 shows the arrangement with IFAS and requires modification of the existing primary settling tanks to operate with CEPT, upgrade of the existing inoperable aeration basin, and addition of one new secondary settling tank.



Figure 4-2 Treatment Facility Requirements for Derby Plus Ansonia with BioMag



Figure 4-3 Treatment Facility Requirements for Derby Plus Ansonia with IFAS

The site layout for the Derby plus Seymour regional alternative is shown as two options. Figure 4-4 shows the arrangement with BioMag and requires modification of the existing primary

settling tanks to operate with CEPT, upgrade of the inoperable existing aeration basin, addition of one new aeration basin, and upgrades to add a magnetite feed and recovery system. Figure 4-5 shows the arrangement with IFAS which requires modification of the existing primary settling tanks to operate with CEPT, upgrade of the existing inoperable aeration basin, and addition of one new secondary settling tank.



Figure 4-4 Treatment Facility Requirements for Derby Plus Seymour with BioMag



Figure 4-5 Treatment Facility Requirements for Derby Plus Seymour with IFAS

The site layout for the Derby plus Ansonia and Seymour regional alternative is shown as two options. Figure 4-6 shows the arrangement with BioMag and requires modification of the existing primary settling tanks to operate with CEPT, upgrade of the inoperable existing aeration basin, expansion of the existing aeration basins, addition of one new aeration basin, upgrades to add a magnetite feed and recovery system, and addition of one new secondary settling tank. Figure 4-7 shows the arrangement with IFAS and requires modification of the existing primary settling tanks to operate with CEPT, upgrade of the existing inoperable aeration basin, and addition of two new secondary settling tanks.



Figure 4-6 Treatment Facility Requirements for Derby Plus Ansonia and Seymour with BioMag



Figure 4-7 Treatment Facility Requirements for Derby Plus Ansonia and Seymour with IFAS

4.4.2 Ansonia Conceptual Site Layouts

The Ansonia WPCF was originally constructed in 1968 and last upgraded in 2011. The major liquid treatment processes include four primary clarifiers, two BNR treatment trains each divided between two-stage anoxic zones and two oxidation ditch aeration zones in separate tankage, two secondary clarifiers, and a chemical phosphorus removal system. The plant is moderately compact within the fence line, with some open space to the north near the oxidation ditches. The site is triangular and confined by the Naugatuck River levee to the northeast, a railroad to the west, and the Ansonia transfer station to the south. Conventional treatment options were considered for the Ansonia WPCF where space for new and expanded facilities appeared to be available.

The site layout for Ansonia (base case) is shown in Figure 4-8, which, based on initial review does not require any additional facilities.



Figure 4-8 Treatment Facility Requirements for Ansonia Only

The site layout for the Ansonia plus Derby regional alternative is shown in Figure 4-9. This arrangement requires one additional primary settling tank, modification of the existing UV system, and addition of a tertiary treatment facility.



Figure 4-9 Treatment Facility Requirements for Ansonia Plus Derby

The site layout for the Ansonia plus Seymour regional alternative is shown in Figure 4-10. This arrangement requires one additional primary settling tank.



Figure 4-10 Treatment Facility Requirements for Ansonia Plus Seymour

The site layout for the Ansonia plus Derby and Seymour regional alternative is shown in Figure 4-11. This arrangement requires one additional primary settling tank, modification of the existing primary settling tanks to operate with CEPT, one additional secondary settling tank, modification of the existing UV system, and addition of a tertiary phosphorous treatment facility. Note that a tertiary phosphorus treatment facility would not be required if the treated effluent is conveyed back to the Derby plant site for discharge to the Housatonic River.



Figure 4-11 Treatment Facility Requirements for Ansonia Plus Derby and Seymour

4.4.3 Seymour Conceptual Site Layouts

The Seymour sewage treatment plant was originally constructed in the 1970s and last upgraded in the early 1990s. The major liquid treatment processes include four primary clarifiers, three MLE aeration basins, two secondary clarifiers, and a chemical phosphorus removal system. The plant is compact on a narrow site confined by route 8 to the west, the Naugatuck river to the east, and the Seymour public works facilities to the north. Space was left for a third secondary clarifier on the southern portion of the site. Conventional treatment options were considered for the Seymour sewage treatment plant where space for new facilities appeared to be available.

The site layout for Seymour only (base case) and Seymour plus Beacon Falls are the same. This is shown in Figure 4-12. This arrangement requires modification of the existing primary settling tanks to operate with CEPT and one additional secondary settling tank. Although the Seymour plant can accommodate flows from Beacon Falls without the need for significant additional facilities, regional alternatives which included flows from Beacon Falls were eliminated based on the conveyance corridor evaluation summarized previously.



Figure 4-12 Treatment Facility Requirements for Seymour Only and Seymour Plus Beacon Falls

4.5 ALTERNATIVES SCREEN-OUT BASED ON PLANT PROCESS AND SITE LAYOUT

The planning level plant process and site layout investigations performed as part of Task 2 show that treating flows at Derby or Ansonia is feasible with some upgrades and new facilities required, varying in degree by regional alternative. Along with details of their associated conveyance pipelines, these plant requirements and site layouts for each of the short list alternatives will be developed further in an upcoming study task.

Regional alternatives which include Ansonia effluent pumped to the Housatonic River will also be evaluated further in Task 3 to determine if any reductions in treatment requirements offset associated pipeline and pump station costs. The Derby to Seymour and Ansonia regional alternative (no. 6) was screened out because routing flow from Seymour to Ansonia would be more effective if that pipe route is confirmed to be feasible and recommendable. The screened-out regional alternatives are shown in Table 4-2 along with the regional alternatives that remain (the short-list); these remaining regional alternatives will be developed further in the subsequent work task.

Table 4-2 Alternatives Screen-out Based on Plant Process and Site Layout

No.	Alternative Description
3	Derby to Ansonia
4	Derby to Ansonia, Effluent Pumped to Housatonic River
5	Derby and Seymour to Ansonia
5b	Derby and Seymour to Ansonia, Effluent Pumped to Housatonic River
6	Derby to Seymour and Ansonia
8	Ansonia to Derby
9	Seymour and Ansonia to Derby
10	Seymour to Ansonia, Part of Ansonia to Derby

5.0 CONCLUSION AND RECOMMENDATIONS

5.1 SHORT LIST OF REGIONAL WASTEWATER ALTERNATIVES

Through progressive step evaluations looking at aggressive I/I, conveyance corridors, and plant facilities requirements, the short list of regional alternatives was established and is shown in Table 5-1. These are the regional alternatives recommended for further evaluation in Task 3.

Table 5-1 Short List of Regional Wastewater Alternatives

No.	Alternative Description
3	Derby to Ansonia
4	Derby to Ansonia, Effluent Pumped to Housatonic River
5	Derby and Seymour to Ansonia
5b	Derby and Seymour to Ansonia, Effluent Pumped to Housatonic River
8	Ansonia to Derby
9	Seymour and Ansonia to Derby
10	Seymour to Ansonia, Part of Ansonia to Derby

5.2 TASK 3 LOOK AHEAD

This TM summarizes the work conducted in Task 2 to develop the long list of NVCOG regional wastewater alternatives and define the short list of alternatives for further investigation. These conclusions and recommendations will be reviewed in a workshop (Workshop No. 1) with the NVCOG stakeholders where concurrence will be reached on the short list of regional alternatives. After Workshop No. 1 is complete, Task 3 activities will advance to further evaluate the short list of alternatives to reach the regional alternative(s), as recommendable.

The recommendations from Task 3 will be carried into the development of the recommended alternative(s) and preparation of the final technical report in Task 4.

APPENDIX A

WASTEWATER FLOWS AND LOADS DATA UPDATE

APPENDIX A WASTEWATER FLOWS AND LOADS DATA UPDATE

The data used for the analysis of the individual wastewater treatment plants came from sources including the monthly operating reports (MORs) and the individual plant facility plans. The data was used to determine design flows and loads for each individual plant and the combined capacity of several regional alternatives. On a planning level basis, the facilities were rated based on annual average, maximum month, and peak day data.

Many wastewater process terms used in Appendix A and Appendix B are abbreviated for clarity. Common term abbreviations are listed in Table A 1.

Table A 1 Common Wastewater Process Abbreviations

AB	Aeration basin	N	Nitrogen
BioMag	Ballasted activated sludge (Evoqua BioMag®)	P	Phosphorus
BNR	Biological nutrient removal	PF	Primary filtration
BOD	Biological oxygen demand	PST	Primary settling tank
cBOD	Carbonaceous biological oxygen demand	SLR	Solids loading rate
CAS	Conventional activated sludge	SOR	Surface overflow rate
CEPT	Chemically enhanced primary treatment	SPA	State point analysis
HRT	Hydraulic residence time	SST	Secondary settling tank
IFAS	Integrated fixed film activated sludge	SVI	Sludge volume index
MLSS	Mixed liquor suspended solids	TKN	Total Kjeldahl nitrogen
MOR	Monthly operating report	TSS	Total suspended solids

A.1 Individual Plants

A.1.1 Derby

The flow and loads coming into the Derby facility are listed below in Table A 2. This outlines the BOD, TSS, and TKN incoming to the plant based on 2015 to 2019 MOR data. Design projections are based on the Phase 1 2040 average flow projection when assuming flow and load peaking factors do not change.

It is noted that Derby is in the process of revising their future plant flow and loadings projections as part of updating their Facilities Plan. Preliminary indications are that these projections will be lower than those in the draft 2014 Facilities Plan. These new projections will be reviewed in subsequent work tasks, particularly in light of treatment facility and conveyance requirements for Derby.

Table A 2 Derby Current and Design Influent Flows and Loads

	Flow, MGD	BOD, mg/L	BOD lb/day	TSS, mg/L	TSS, lb/day	TKN, mg/L	TKN, lb/day
Current Influent							
Annual Average	1.42	221	2,621	197	2,333	28	335
Maximum Month	2.19	260	4,749	240	4,384	33	610
Peak Day	4.10	-	-	-	-	-	-
Design Influent							
Annual Average	1.92	221	3,544	197	3,155	28	453
Maximum Month	2.96	260	6,421	240	5,927	33	825
Peak Day	5.54	-	-	-	-	-	-

The following Table A 3 provides the current primary effluent and design primary effluent data for the Derby wastewater treatment facility. It is assumed that there is a 30 percent removal of BOD, 60 percent removal of TSS, and 10 percent removal of TKN.

Table A 3 Derby Current and Design Primary Effluent Flows and Loads

	Flow, MGD	BOD, mg/L	BOD lb/day	TSS, mg/L	TSS, lb/day	TKN, mg/L	TKN, lb/day
Current Primary Effluent							
Annual Average	1.42	155	1,835	79	933	25	302
Maximum Month	2.19	182	3,324	96	1,753	30	549
Peak Day	4.10	-	-	-	-	-	-
Design Primary Effluent							
Annual Average	1.92	155	2,481	79	1,262	25	408
Maximum Month	2.96	182	4,495	96	2,371	30	742
Peak Day	5.54	-	-	-	-	-	-

A.1.2 Ansonia

The flow and loads coming into the Ansonia facility are listed below in Table A 4. This outlines the BOD, TSS, and TKN incoming to the plant based on 2015 to 2019 MOR data. Design projections are based on the Phase 1 2040 average flow projection when assuming flow and load peaking factors do not change.

Table A 4 Ansonia Current and Design Influent Flows and Loads

	Flow, MGD	BOD, mg/L	BOD lb/day	TSS, mg/L	TSS, lb/day	TKN, mg/L	TKN, lb/day
Current Influent							
Annual Average	1.76	204	2,988	184	2,695	45	656
Maximum Month	3.06	187	4,772	191	4,874	41	1,046
Peak Day	4.60	-	-	-	-	-	-
Design Influent							
Annual Average	1.90	204	3,236	184	2,919	45	711
Maximum Month	3.31	187	5,167	191	5,278	41	1,133
Peak Day	4.98	-	-	-	-	-	-

The following Table A 5 provides the current primary effluent and design primary effluent data for the Ansonia wastewater treatment facility. It is assumed that there is a 30 percent removal of BOD, 60 percent removal of TSS, and 10 percent removal of TKN.

Table A 5 Ansonia Current and Design Primary Effluent Flows and Loads

	Flow, MGD	BOD, mg/L	BOD lb/day	TSS, mg/L	TSS, lb/day	TKN, mg/L	TKN, lb/day
Current Primary Effluent							
Annual Average	1.76	143	2,092	74	1,078	40	591
Maximum Month	3.06	131	3,341	76	1,950	37	942
Peak Day	4.60	-	-	-	-	-	-
Design Primary Effluent							
Annual Average	1.90	143	2,265	74	1,167	40	639
Maximum Month	3.31	131	3,617	76	2,111	37	1,020
Peak Day	4.98	-	-	-	-	-	-

A.1.3 Seymour

The flow and loads coming into the Seymour facility are listed below in Table A 6. This outlines the BOD, TSS, and TKN incoming to the plant based on the 2015 to 2017 MOR data. Design projections are based on the Phase 1 2040 average flow projection when assuming flow and load peaking factors do not change.

Table A 6 Seymour Current and Design Influent Flows and Loads

	Flow, MGD	BOD, mg/L	BOD lb/day	TSS, mg/L	TSS, lb/day	TKN, mg/L	TKN, lb/day
Current Influent							
Annual Average	0.97	140	1,133	146	1,181	33	269
Maximum Month	1.93	93	1,497	99	1,594	22	356
Peak Day	3.34	-	-	-	-	-	-
Design Influent							
Annual Average	1.30	140	1,518	146	1,583	33	361
Maximum Month	2.59	112	2,424	133	2,863	27	576
Peak Day	4.48	-	-	-	-	-	-

The following Table A 7 provides the current primary effluent and design primary effluent data for the Seymour wastewater treatment facility. It is assumed that there is a 30 percent removal of BOD, 60 percent removal of TSS, and 10 percent removal of TKN.

Table A 7 Seymour Current and Design Primary Effluent Flows and Loads

	Flow, MGD	BOD, mg/L	BOD lb/day	TSS, mg/L	TSS, lb/day	TKN, mg/L	TKN, lb/day
Current Primary Effluent							
Annual Average	0.97	98	793	58	472	30	242
Maximum Month	1.93	65	1,048	40	637	20	320
Peak Day	3.34	-	-	-	-	-	-
Design Primary Effluent							
Annual Average	1.30	98	1,063	58	633	30	325
Maximum Month	2.59	79	1,697	53	1,145	24	518
Peak Day	4.48	-	-	-	-	-	-

A.1.4 Beacon Falls

The flow and loads coming into the Beacon Falls facility are listed below in Table A 8. Beacon Falls data is based on the previous Black & Veatch projections in Phase 1 for influent flow and load characteristics.

Table A 8 Beacon Falls Current and Design Influent Flows and Loads

	Flow, MGD	BOD, mg/L	BOD lb/day	TSS, mg/L	TSS, lb/day	TKN, mg/L	TKN, lb/day
Current Influent							
Annual Average	0.31	211	546	199	514	50	130
Maximum Month	0.53	164	721	158	694	39	171
Peak Day	0.87	-	-	-	-	-	-
Design Influent							
Annual Average	0.45	211	792	199	747	50	188
Maximum Month	0.77	164	1,047	158	1,008	47	300
Peak Day	1.26	-	-	-	-	-	-

The following Table A 9 provides the current primary effluent and design primary effluent data for the Beacon Falls wastewater treatment facility. It is assumed that there is a 30 percent removal of BOD, 60 percent removal of TSS, and 10 percent removal of TKN.

Table A 9 Beacon Falls Current and Design Primary Effluent Flows and Loads

	Flow, MGD	BOD, mg/L	BOD lb/day	TSS, mg/L	TSS, lb/day	TKN, mg/L	TKN, lb/day
Current Primary Effluent							
Annual Average	0.31	148	382	80	206	45	117
Maximum Month	0.53	115	505	63	278	35	154
Peak Day	0.87	-	-	-	-	-	-
Design Primary Effluent							
Annual Average	0.45	148	382	80	299	45	170
Maximum Month	0.77	115	733	63	403	42	270
Peak Day	1.26	-	-	-	-	-	-

A.2 Combined Plants

Flow and loads for combined plants were calculated in one of two ways.

- For combinations of Derby WPCF, Ansonia WPCF, and Seymour WPCF influents, the combined flows and loads were calculated for each day with the MOR data that

was available. Five years of data (2015-2019) were available for Derby and Ansonia but only three years of data (2015-2017) was available for Seymour. This results in lower and more realistic peaking factors than are obtained by summing the max month or peak day conditions for individual facilities directly, because max conditions are less likely to happen concurrently in all collection systems; flow and load peaking factors tend to decrease with increasing catchment area or average flow.

- When combining the Beacon Falls with either the Seymour or Naugatuck, it was assumed that max Beach Falls flows and loads would occur concurrently. This assumption has a relatively small impact on the assessment because the Beacon Falls wastewater contribution is relatively small.

A.2.1 Derby Plus Ansonia

“Derby plus Ansonia” refers to the flows and loads from both facilities being treated at Derby using its existing treatment units. These flows and loads also apply to alternatives that have the combined systems treated at Ansonia.

Table A 10 Derby Plus Ansonia Current and Design Influent Flows and Loads

	Flow, MGD	BOD, mg/L	BOD lb/day	TSS, mg/L	TSS, lb/day	TKN, mg/L	TKN, lb/day
Current Influent							
Annual Average	3.17	212	5,598	195	5,155	38	994
Maximum Month	5.04	208	8,757	184	7,734	39	1,656
Peak Day	7.90	-	-	-	-	-	-
Design Influent							
Annual Average	3.82	212	6,749	195	6,215	38	1,198
Maximum Month	6.08	208	10,558	184	9,324	39	1,997
Peak Day	9.52	-	-	-	-	-	-

The following Table A 11 provides the current primary effluent and design primary effluent data for Derby and Ansonia wastewater treatment. It is assumed that there is a 30 percent removal of BOD, 60 percent removal of TSS, and 10 percent removal of TKN.

Table A 11 Derby Plus Ansonia Current and Design Primary Effluent Flows and Loads

	Flow, MGD	BOD, mg/L	BOD lb/day	TSS, mg/L	TSS, lb/day	TKN, mg/L	TKN, lb/day
Current Primary Effluent							
Annual Average	3.17	148	3,918	78	2,062	34	895
Maximum Month	5.04	146	6,130	74	3,094	35	1,491
Peak Day	7.90	-	-	-	-	-	-
Design Primary Effluent							
Annual Average	3.82	148	4,724	78	2,486	34	1,079
Maximum Month	6.08	146	7,390	74	3,730	35	1,797
Peak Day	9.52	-	-	-	-	-	-

A.2.2 Derby Plus Seymour

“Derby plus Seymour” refers to the flows and loads from both facilities being treated at Derby using its existing treatment units.

Table A 12 Derby Plus Seymour Current and Design Influent Flows and Loads

	Flow, MGD	BOD, mg/L	BOD lb/day	TSS, mg/L	TSS, lb/day	TKN, mg/L	TKN, lb/day
Current Influent							
Annual Average	2.28	205	3,893	185	3,518	32	607
Maximum Month	3.84	191	6,130	187	5,989	30	964
Peak Day	5.80						
Design Influent							
Annual Average	3.22	205	5,498	185	4,968	32	857
Maximum Month	5.42	191	8,658	187	8,458	30	1,361
Peak Day	2.54						

The following Table A 13 provides the current primary effluent and design primary effluent data for Derby and Seymour wastewater treatment. It is assumed that there is a 30 percent removal of BOD, 60 percent removal of TSS, and 10 percent removal of TKN.

Table A 13 Derby Plus Seymour Current and Design Primary Effluent Flows and Loads

	Flow, MGD	BOD, mg/L	BOD lb/day	TSS, mg/L	TSS, lb/day	TKN, mg/L	TKN, lb/day
Current Primary Effluent							
Annual Average	2.28	143	2,725	74	1,407	29	546
Maximum Month	3.84	134	4,291	75	2,396	27	868
Peak Day	5.80	-	-	-	-	-	-
Design Primary Effluent							
Annual Average	3.22	143	3,849	74	1,987	29	771
Maximum Month	5.42	134	6,060	75	3,383	27	1,225
Peak Day	8.19	-	-	-	-	-	-

A.2.3 Derby Plus Ansonia and Seymour

“Derby plus Ansonia and Seymour” refers to the flows and loads from all three facilities being treated at Derby with its existing capacity and treatment units. These flows and loads also apply to alternatives that have the combined systems treated at Ansonia.

Table A 14 Plus Ansonia and Seymour Current and Design Influent Flows and Loads

	Flow, MGD	BOD, mg/L	BOD lb/day	TSS, mg/L	TSS, lb/day	TKN, mg/L	TKN, lb/day
Current Influent							
Annual Average	3.89	218	7,079	179	5,807	39	1,265
Maximum Month	6.44	191	10,233	155	8,325	38	2,014
Peak Day	9.30	-	-	-	-	-	-
Design Influent							
Annual Average	5.12	218	9,321	179	7,646	39	1,666
Maximum Month	8.48	191	13,473	155	10,961	38	2,652
Peak Day	12.24	-	-	-	-	-	-

The following Table A 15 provides the current primary effluent and design primary effluent data for Derby, Ansonia, and Seymour wastewater treatment. It is assumed that there is a 30 percent removal of BOD, 60 percent removal of TSS, and 10 percent removal of TKN.

Table A 15 Derby Plus Ansonia and Seymour Current and Design Primary Effluent Flows and Loads

	Flow, MGD	BOD, mg/L	BOD lb/day	TSS, mg/L	TSS, lb/day	TKN, mg/L	TKN, lb/day
Current Primary Effluent							
Annual Average	3.89	153	4,946	72	2,323	35	1,139
Maximum Month	6.44	133	7,163	62	3,330	34	1,813
Peak Day	9.30	-	-	-	-	-	-
Design Primary Effluent							
Annual Average	5.12	153	6,525	72	3,058	35	1,499
Maximum Month	8.48	133	9,431	62	4,384	34	2,387
Peak Day	12.24	-	-	-	-	-	-

A.2.4 Ansonia Plus Derby

“Ansonia plus Derby” refers to the flows and loads from both facilities being treated at Ansonia using its existing treatment units. Refer to section A.2 Derby Plus Ansonia for the data summary.

A.2.5 Ansonia Plus Seymour

“Ansonia plus Seymour” refers to the flows and loads from both facilities being treated at Ansonia using its existing treatment units. From the MOR data, the cBOD to BOD ratio is 100 percent for Ansonia and Seymour.

Table A 16 Ansonia Plus Seymour Current and Design Influent Flows and Loads

	Flow, MGD	BOD, mg/L	BOD lb/day	TSS, mg/L	TSS, lb/day	TKN, mg/L	TKN, lb/day
Current Influent							
Annual Average	2.58	195	4,196	157	3,378	43	927
Maximum Month	4.37	165	5,977	124	4,519	39	1,403
Peak Day	6.13	-	-	-	-	-	-
Design Influent							
Annual Average	3.20	195	5,297	157	4,192	43	1,151
Maximum Month	5.42	164	7,417	124	5,608	39	1,741
Peak Day	7.61	-	-	-	-	-	-

The following Table A 17 provides the current primary effluent and design primary effluent data for Ansonia and Seymour wastewater treatment. It is assumed that there is a 30 percent removal of BOD, 60 percent removal of TSS, and 10 percent removal of TKN.

Table A 17 Ansonia Plus Seymour Current and Design Primary Effluent Flows and Loads

	Flow, MGD	BOD, mg/L	BOD lb/day	TSS, mg/L	TSS, lb/day	TKN, mg/L	TKN, lb/day
Current Primary Effluent							
Annual Average	2.58	137	2,937	63	1,351	39	835
Maximum Month	4.37	115	4,184	50	1,808	35	1,263
Peak Day	6.13	-	-	-	-	-	-
Design Primary Effluent							
Annual Average	3.20	137	3,645	63	1,677	39	1,036
Maximum Month	5.42	115	5,192	50	2,243	35	1,567
Peak Day	7.61	-	-	-	-	-	-

A.2.6 Ansonia Plus Derby and Seymour

“Ansonia plus Derby and Seymour” refers to the flows and loads from all three facilities being treated at Ansonia with its existing capacity and treatment units. Refer to section A.2

Derby Plus Ansonia and Seymour for the data summary.

A.2.7 Seymour Plus Beacon Falls

“Seymour Plus Beacon Falls” refers to the combination of flows and loads from both facilities being treated at Seymour with its current capacity and treatment units.

Table A 18 Seymour Plus Beacon Falls Current and Design Influent Flows and Loads

	Flow, MGD	BOD, mg/L	BOD lb/day	TSS, mg/L	TSS, lb/day	TKN, mg/L	TKN, lb/day
Current Influent							
Annual Average	1.28	157	1,678	159	1,696	37	399
Maximum Month	2.46	108	2,218	112	2,288	26	527
Peak Day	4.21	-	-	-	-	-	-
Design Influent							
Annual Average	1.8	158	2,310	160	2,330	38	549
Maximum Month	3.35	124	3,471	138	3,870	31	876
Peak Day	5.74	-	-	-	-	-	-

The following Table A 19 provides the current primary effluent and design primary effluent data for the Seymour wastewater treatment facility. It is assumed that there is a 30 percent removal of BOD, 60 percent removal of TSS, and 10 percent removal of TKN.

Table A 19 Seymour Plus Beacon Falls Current and Design Primary Effluent Flows and Loads

	Flow, MGD	BOD, mg/L	BOD lb/day	TSS, mg/L	TSS, lb/day	TKN, mg/L	TKN, lb/day
Current Primary Effluent							
Annual Average	1.28	110	1,175	64	678	34	359
Maximum Month	2.46	76	1,553	45	915	23	474
Peak Day	4.21	-	-	-	-	-	-
Design Primary Effluent							
Annual Average	1.75	111	1,617	64	932	34	494
Maximum Month	3.35	87	2,430	55	1,548	28	788
Peak Day	5.74	-	-	-	-	-	-

APPENDIX B

TREATMENT CAPACITY ASSESSMENT

APPENDIX B TREATMENT CAPACITY ASSESSMENT

B.1 Treatment Capacity Assessment

A planning level process capacity review of the plants being considered for regional treatment was performed. This review focused mainly on the capacity of the major primary and secondary treatment units to process the required flow and loads, and the need for new major tertiary process units to meet effluent permit limits. Primary and secondary treatment process units are footprint intensive; therefore, the evaluation focused on these to assess the feasibility of the existing sites to treat the required flow. Where existing unit process tankage is not adequate, the addition of major process units equal to existing units were considered with and without intensification alternatives. This assessment was performed with spreadsheet based steady state models and did not include a detailed assessment of the impact of the proposed loading changes and upgrades on BNR process performance. The assessment also did not include preliminary treatment processes, disinfection processes, residuals management/treatment, or general hydraulics. Some or all of these will be considered in greater detail as part of the short-listed alternatives in subsequent task work.

Refer to Table A-1 in Appendix A for a list of common wastewater terminology abbreviations used throughout this appendix.

B.1.1 Primary Settling Tanks

Primary settling tanks were assessed primarily on the basis of the surface overflow rate (SOR). NEIWPCC Technical Report 16 Guides for the Design of Wastewater Treatment Works (TR-16) allows an average surface overflow rate of 1,200 gpd/ft² and a peak hour surface overflow rate of 3,000 gpd/ft². If using chemically enhanced primary settling (CEPT), the peak SOR can be increased to in excess of 5,000 gpd/ft² while increasing the TSS and BOD removal across primary treatment. This will have added benefits to secondary capacity and potentially to energy costs depending on how primary and secondary sludge are managed. Though CEPT may allow for the existing primary settling tanks (PSTs) to treat higher flows in the regionalization alternatives, the PSTs will still likely need to be modified. An assessment of the PST internals would be required to determine if the higher flows could be treated and modifications would likely be required to ensure adequate residence times are available in the inlet structures and weir loading rates are adequately low. These assessments are beyond the scope of this evaluation.

B.1.2 Conventional Secondary Treatment

All facilities being evaluated currently use conventional activated sludge (CAS) in a modified Ludzack Ettinger process configuration for nitrogen (N) removal and chemical dosing for seasonal removal of phosphorus (P). CAS consists of aeration basins (ABs) and secondary settling tanks (SSTs). Because AB volume impacts the mixed liquor suspended solids (MLSS) concentration of operation which in turn impacts the solids loading rate (SLR) of the SSTs, AB and SST facilities must be considered together to determine secondary treatment capacity. TR-16 recommends the use of state point analysis (SPA) to evaluate this. In addition to this the SOR of the SSTs and the hydraulic residence time (HRT) of the ABs are considered. Secondary treatment systems were rated at max month conditions with the assumption that all major process units (ABs and SSTs) were online. The condition of one major process unit offline at average loads was however checked to ensure that this could be met for maintenance purposes.

To rate secondary treatment capacity, removals of 60% of TSS and 30% of BOD at the PSTs were assumed. These loads were applied to Black & Veatch's completely mixed activated sludge model to determine the MLSS concentration when operating at maximum month conditions and at an aerobic SRT of 9.2 days. This is the minimum design aerobic SRT when operating at a temperature of 12°C per Black & Veatch standards. An analysis of the data indicates that temperatures can drop below this, however design aerobic SRT should be adequate given that ammonia limits are not stringent. The impact of increased primary removal due to the use of CEPT was not considered in evaluating the secondary capacity as secondary and primary processes were considered separately.

SPA was then utilized to assess SST loading. An SVI of 120 mL/g was assumed for Derby and Ansonia facilities, but an SVI of 200 mL/g was assumed at Seymour. This is because Seymour experiences more extreme settleability issues than the other facilities. The settling parameters necessary for SPA were derived from the assumed SVIs and a commonly used design correlation, the same as is recommended in TR-16 (Daigger, 2016). To the settling flux curve, Ekama factors (safety factors) were applied (80% for Derby and Seymour and 85% for Ansonia); this is because the Ansonia plant has deeper and more modern SSTs. Later in this document, the SPA results are presented as percentages of capacity, defined as the required flux (the state point flux) divided by the limiting flux at the AB effluent MLSS concentration, visualized in Figure B 1. In addition to using SPA, a limit on the peak day SOR of ~1,200 gpd/ft² was utilized. In instances where a scenario just exceeded the secondary capacity, step feeding of primary effluent flow to of end of the ABs was considered and recommended if this allowed for SPA requirements to be met without the need for an additional AB or SST. This was limited to only step feeding flows in excess of 110% of the max month flow.

The plant secondary treatment capacity involves both the number of ABs and the number of SSTs. This secondary treatment capacity is a function of AB volume and SST area. There is thus a tradeoff between the two such that secondary capacity can be satisfied with different combinations of ABs and SSTs. We represent this secondary treatment capacity in the figures below presented for each of the plants and different scenarios (Figure B 2 for example). In these figures, below 100% indicates excess capacity and above 100% indicates insufficient capacity.

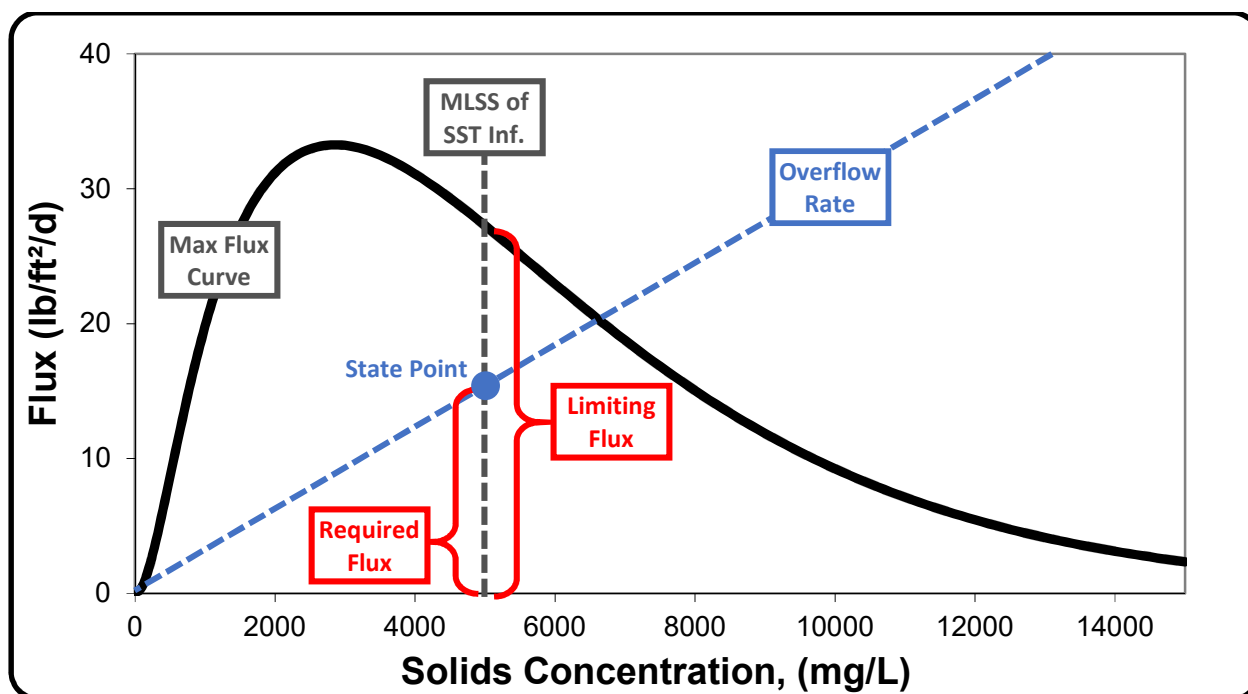


Figure B 1 Graphical Summary of Capacity Percentage Determined Through State Point Analysis

Aeration basins HRT was also considered when evaluating secondary capacity and the need for additional ABs in regionalization alternatives. A low limit of five hours aerobic HRT and 1.5 hours anoxic HRT were used as guidelines. More detailed biokinetic modeling will need to be performed to ensure that adequate aerobic and anoxic volumes are available for nitrification and denitrification. More generally, the evaluation performed as part of Task 2 has not considered impacts on denitrification performance which may result from regionalization. Higher loadings to ABs and changes in influent characteristics may reduce TN removal performance and require a reevaluation of BNR design, potentially including adjustments to the anoxic zone volume fraction of the ABs, adjustments to the mixed liquor recycle capacity, the addition of supplemental carbon feed systems, etc. These impacts will be investigated in later tasks.

B.1.3 Secondary Treatment Intensification

Several intensification alternatives were considered as alternatives to CAS, including primary filtration (PF) with CAS, integrated fixed-film activated sludge (IFAS), and ballasted activated sludge (Evoqua BioMag®). Aerobic granular sludge (AGS) was not considered as the available tankage onsite at all facilities is not adequate for retrofitting to AGS. Membrane bioreactor (MBR) was also not considered due to the high operational cost. These intensification options under consideration are described and the capacity rating basis for each is summarized in this section.

Ballasted Activated Sludge

The BioMag process adds magnetite to activated sludge in order to enhance settling rate and secondary sludge thickening characteristics. This allows for both an increase in SLR and SOR to the SSTs, meaning that both the flow and the MLSS (and therefore load) through secondary treatment can be substantially enhanced. Because the magnetite particles are hydrophobic, they readily bind to mixed liquor solids and can be recovered with a magnetic recovery drum.

Instead of using SPA to evaluate the ballasted activated sludge intensification options, design standards for MLSS, SLR, and SOR were utilized. The following published design values were used to assess the capacity requirements of the BioMag process system.

- MLSS: Maximum 10,000 mg/L (excluding ballast)
- SOR: Maximum 1,500 gpd/ft² Max Month, 2,500 gpd/ ft² Peak Hour
- SLR: Maximum 75 lb/day/ ft² Max Month, 100 lb/day/ ft² Peak Hour (excluding ballast)

Because all of these maximums cannot occur concurrently, the practice was to limit MLSS to ~5,000 mg/L, max day SOR to ~1,500 gpd/ft², and max day SLR to ~70 lb/day/ ft². Additional detail of the equipment requirements associated with BioMag will be described in the subsequent work task as the regional alternatives are developed further.

Integrated Fixed Film Activated Sludge

IFAS uses plastic biofilm carrier media in the mixed liquor to increase the inventory of the activated sludge process without increasing SST solids loading. Biofilm carrier media can be added to anoxic and aerobic zones and are retained with special retention sieves. The addition of media to the aerobic zones allows the minimum SRT for nitrification to be achieved at a lower operating MLSS, thereby reducing clarifier loadings. To assess this an extension of completely mixed activated sludge model was used which determines the media fill required to limit the MLSS to a target value. The reduced MLSS was used with SPA to assess capacity. Media characteristics assumed were consistent with Kaldnes K1 media and are as follows;

- Specific Surface Area: 500 m²/m³
- Void Ratio: 84%
- Maximum Fill Fraction: 65%

Only the addition of media to the aerobic zones was considered to determine feasibility. If this process is viable then more detailed biokinetic modeling can be done to assess the need for anoxic media to increase denitrification capacity (due to reduced suspended inventory) and to assess that nitrification rates are adequate for the reduced aerobic HRTs. IFAS upgrades will also require substantial equipment replacement including mixers and aeration systems; this will be described further in the subsequent work task,

Primary Filtration

Primary Filtration (PF) actually replaces the PSTs, however it was considered a secondary intensification alternative for purposes of this study. This is because PF can reduce the loading to secondary treatment enough, thereby in affect, increasing the secondary capacity. Other benefits include the potential for the whole facility to be more energy efficient by diverting carbon from the aeration basins (saving energy) to the residuals treatment/management processes (potentially producing energy if anaerobic digestion with biogas utilization is implemented). Primary filtration - can be implemented with cloth media filters which could be retrofitted into existing PSTs. Drawbacks include that additional primary sludge thickening will likely be necessary and carbon diverted from the secondary treatment process may limit nitrogen removal. To assess the capacity benefits of primary filtration, the assumed change in primary removal is adjusted to 85%

TSS removal and 45% BOD removal, which are typical of performance data provided by Aqua Aerobics, a major manufacturer of this process technology.

B.1.4 Seasonal Phosphorus Load Limits

All of the treatment facilities which discharge into the Naugatuck River (Ansonia, Seymour, Beacon Falls, and Naugatuck) have seasonal P limits, while the facilities that discharges into the Housatonic River (Derby only) do not have any P limits. For purposes of evaluating the regional alternatives, it is assumed that P load allocations will be transferred from one facility. Further, it is assumed that treated discharges to the Housatonic River at Derby will not have P load limits. Based on these assumptions, the seasonal load allocation is calculated for each regional alternative and used with the projected average 2040 flow to estimate the target effluent P concentration necessary to meet the seasonal load limit. If this is substantially low, that regional alternative will need to consider new facilities such as tertiary treatment and/or reduced SST loading. This assessment is made at a high level when considering these alternatives and a more detailed evaluation will be required for the short-listed alternatives. Potential changes to daily or monthly effluent P concentration limits will not be considered as the seasonal load limits usually dictate the treatment processes required.

B.2 Summary of Treatment Needs Alternatives Analysis

B.2.1 Treatment at Derby

Primary Treatment of Design Flows and Loads

The capacity of Derby's PSTs was assessed based on peak hour SOR. Average SOR was also considered but the peak condition was controlling in all cases due to the high flow peaking factors. The peak hour limit of 3,000 gpd/ft² was converted to a peak daily limit based on an assumed peaking factor. For Derby only (base case) the peak-hour to peak-day peaking factor is based on historical data, while a lower peaking factor (i.e. higher peak day SOR limit) is used for the regional treatment alternatives, as peaking factors tend to reduce in larger collection systems.

Based on these assumptions, Derby will need one to two PSTs in addition to the two existing ones to treat the design flows, depending on whether peaking factors have been reduced with recent I/I control measures. If treating the flow from either Ansonia or Seymour, two PSTs in addition to the two existing are needed. To treat the combined design flow for all three facilities (Derby, Ansonia, and Seymour), four PSTs in addition to the two existing are needed. Due to site limitations it is recommended that CEPT be the preferred option for primary clarifier capacity intensification. If utilizing CEPT, it is unlikely that additional PSTs would be required, though extensive modifications to the PST internals would be required both due to condition and to accept higher flows.

Table B 1 Derby Primary Settling Tank Overflow Rate Analysis at 2040 Design Conditions

	Derby	Derby +Ansonia	Derby +Seymour	Derby +Ansonia +Seymour
PH SOR Limit, gpd/ft ²	3,000 ⁽¹⁾	3,000 ⁽¹⁾	3,000 ⁽¹⁾	3,000 ⁽¹⁾
PH to PD Flow Peaking Factor	2.0-2.5 ⁽²⁾	1.6 ⁽³⁾	1.6 ⁽³⁾	1.6 ⁽³⁾
PD SOR Limit, gpd/ft ²	1,200-1,500	1,875	1,875	1,875
PD SOR with 0 New PSTs, gpd/ft ²	1,930	3,660	3,480	5,210
PD SOR with 1 New PSTs, gpd/ft ²	1,290	2,440	2,320	3,480
PD SOR with 2 New PSTs, gpd/ft ²	970	1,830	1,740	2,610
PD SOR with 3 New PSTs, gpd/ft ²	770	1,470	1,400	2,090
PD SOR with 4 New PSTs, gpd/ft ²	650	1,220	1,160	1,740
New PSTs Required	1-2	2	2	3-4
New PSTs Required (CEPT)	0	0	0	0
(1) Based on TR-16 Peak Hour SOR Limit of 3,000 gpd/ft ²				
(2) Based on 2015-2019 PD Influent Flow of 4.1 mgd and Peak Hour Flow of 8.0-10.0 mgd				
(3) Based on TR-16 Figure 2-1 for Facilities with Average Flows >3 mgd				

Secondary Treatment of Design Flows and Loads

City of Derby

Figure B 2 shows the percentage of the secondary treatment capacity requirement at 2040 max design condition, for various numbers of SSTs and ABs. SPA was used in this rating, where the percentage of the capacity is defined as the required SST flux divided by the limiting clarifier flux (see Figure B 1 above). MLSS was determined for the 2040 design maximum month conditions which was utilized with peak day flows. An SVI of 120 mL/g was assumed for Derby as settleability is typically good at the City's plant, though some improvements to settling may need to be explored. This indicates for example that with three SSTs (the new one being equivalently sized to the existing SSTs) and three ABs (through modification of the existing third AB), the 2040 flows from Derby only can be treated at the plant. Similarly, capacity could be met with four ABs and two SSTs. If the third AB is modified and there are only two SSTs, the system is at 114% of capacity. However, further state point analysis indicates that this difference can be made up through step feeding of wet weather flows. Table B 2 shows the results summarized for this base case scenario when using step feeding. The implications of step feeding this quantity of wastewater can be explored further through biokinetic modeling. Because capacity can be reached without new major process units, process intensification alternatives were not considered.

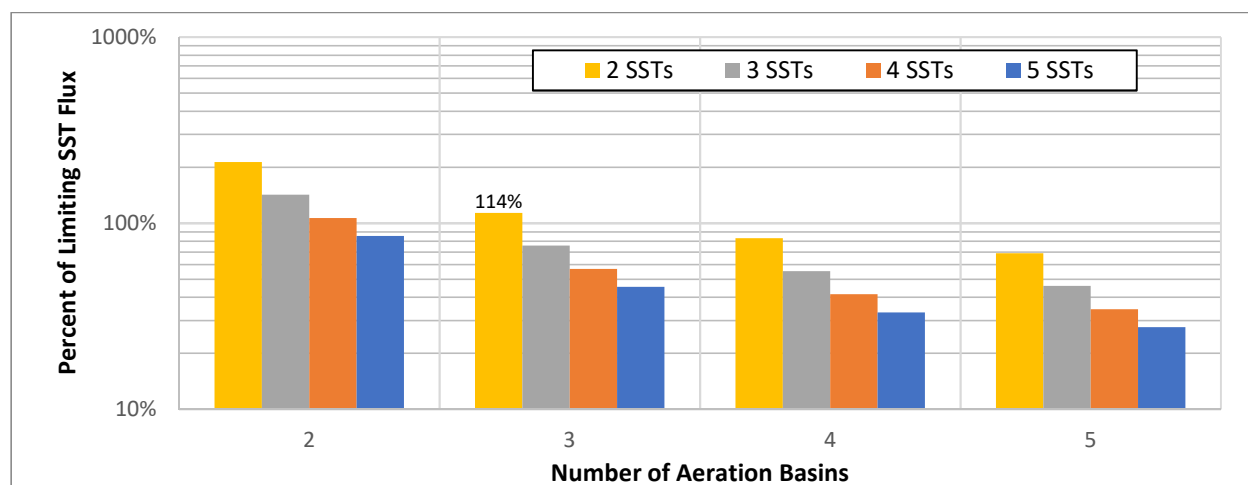


Figure B 2 Secondary Capacity Evaluation for Derby Treatment at Derby

Table B 2 Summary of Derby Treatment with Conventional Process Alternatives

CAS	
Number of Additional ABs	1
Number of Additional SSTs	0
Max Day Aerobic HRT, hours	7.3
Max Day Anoxic HRT, hours	3.6
Max Month MLSS, mg/L	5,364 ⁽¹⁾
Max Day SOR, gpd/ft ²	980
Max Day SLR, lb/day/ft ²	38
(1) Step Feeding of flows in excess of 4.7 mgd Reduces MLSS at Peak Flows to 3,200 mg/L	

Derby Plus Ansonia

As described above, Figure B 3 shows the percentage of the secondary treatment capacity requirement at 2040 max design condition, for various numbers of SSTs and ABs, with the same assumptions utilized as for the case when treating Derby wastewater only. Figure B 3 shows that four SSTs and three to four ABs are needed. With only three ABs, the system is at 134% of capacity. This could be managed through aggressive step feeding of wet weather flows, though the implications of step feeding this quantity of water would need to be explored further through biokinetic modeling if this alternative is promising. Table B 3 summarizes the SPA results for this scenario when using step feeding and compares this with intensification alternatives, which were explored further due to the difficulty of siting these additional major process units. IFAS and BioMag alternatives show promise as they can reduce the requirements to the modification of the third AB and the construction of one other major process unit. For IFAS, this will require an

additional SST while in the BioMag alternative this would be either an additional AB or an additional SST.

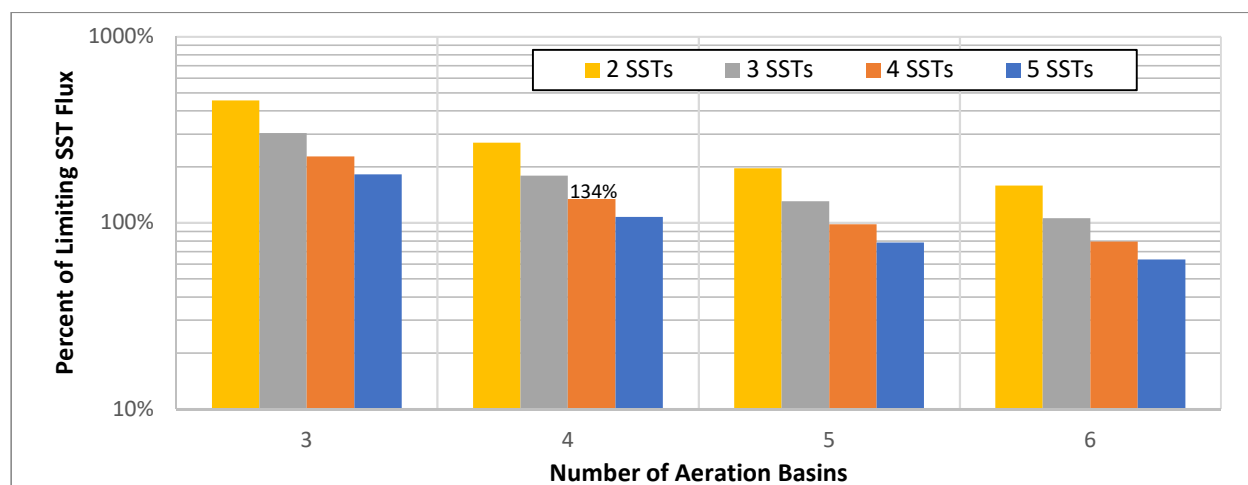


Figure B 3 Secondary Capacity Evaluation for Derby and Ansonia Treatment at Derby

Table B 3 Treatment of Combined Derby and Ansonia 2040 Design Maximum Flow and Loads at Derby WPCF with Conventional and Intensified Process Alternatives

	CAS	PF+CAS	IFAS	BioMag
Number of Additional ABs	2	1	1	2
Number of Additional SSTs	2	2	1	0
Max Day Aerobic HRT, hours	4.8	3.6	3.6	4.8
Max Day Anoxic HRT, hours	2.3	1.8	1.8	2.3
Max Month MLSS, mg/L	4,450 ⁽¹⁾	3,660	2,700 ⁽²⁾	4,450
Max Day SOR, gpd/ft ²	840	840	1,120	1,680
Max Day SLR, lb/day/ft ²	39.5	38.1	36.2	81.2
(1) Step Feeding of flows in excess of 6.5 mgd Reduces MLSS at Peak Flows to 3,600 mg/L				
(2) Suspended MLSS Limited to 2,700 mg/L with IFAS through 50% Media Fill (Kaldnes K1)				

Derby Plus Seymour

As described above, Figure B 4 shows the percentage of the secondary treatment capacity requirement at 2040 max design condition, for various numbers of SSTs and ABs, with the same assumptions utilized above. Because Seymour loads are lower than Ansonia loads, three SSTs and three ABs result in the systems being at 119% of capacity which can be managed through step feeding of wet weather flows. Table B 4 summarizes the SPA results for this scenario when using step feeding and compares this with intensification alternatives. Results are similar as in the analysis of the Derby plus Ansonia regional alternatives, except that only one additional SST is needed if primary filtration is employed.

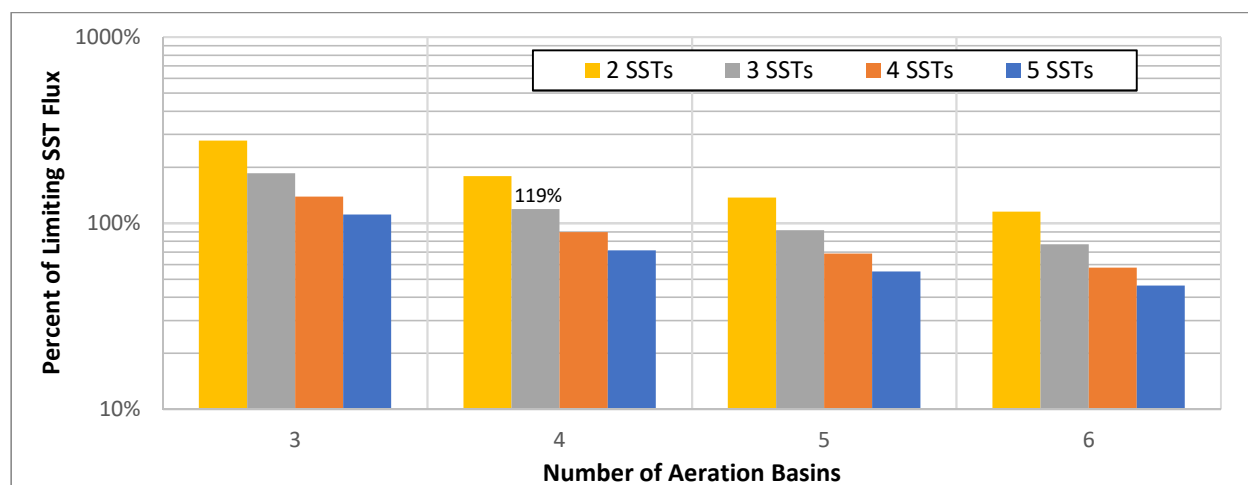


Figure B 4 Secondary Capacity Evaluation for Derby and Seymour Treatment at Derby

Table B 4 Summary of Combined Derby and Seymour Treatment at Derby WPCF with Conventional and Intensified Process Alternatives

	CAS	PF+CAS	IFAS	BioMag
Number of Additional ABs	2	1	1	2
Number of Additional SSTs	1	1	1	0
Max Day Aerobic HRT, hours	5.3	4.0	4.0	5.3
Max Day Anoxic HRT, hours	2.6	2.0	2.0	2.6
Max Month MLSS, mg/L	3,740 ⁽¹⁾	2,990	3,200 ⁽²⁾	3,740
Max Day SOR, gpd/ft ²	970	970	970	1,450
Max Day SLR, lb/day/ft ²	38.7	35.5	37.9	60.4
(1) Step Feeding of flows in excess of 6.3 mgd Reduces MLSS at Peak Flows to 3,300 mg/L				
(2) Suspended MLSS Limited to 3,200 mg/L with IFAS through 25% Media Fill (Kaldnes K1)				

Derby Plus Ansonia and Seymour

As described above, Figure B 5 shows the percentage of the secondary capacity requirement at 2040 max design condition, for various numbers of SSTs and ABs, again, with the same assumptions described above. With both Ansonia and Seymour loads, a total of four SSTs and six ABs are needed and still the system is at 132% of capacity. This could be managed through aggressive step feeding of wet weather flows or possibly mitigated through continued I/I reductions. Table B 5 summarizes the SPA results for this scenario when using step feeding and compares this with intensification alternatives. Results show a similar trend as in other regional alternatives, but additional major process units are needed in the regional alternative that has both Seymour and Ansonia wastewater treated at Derby.

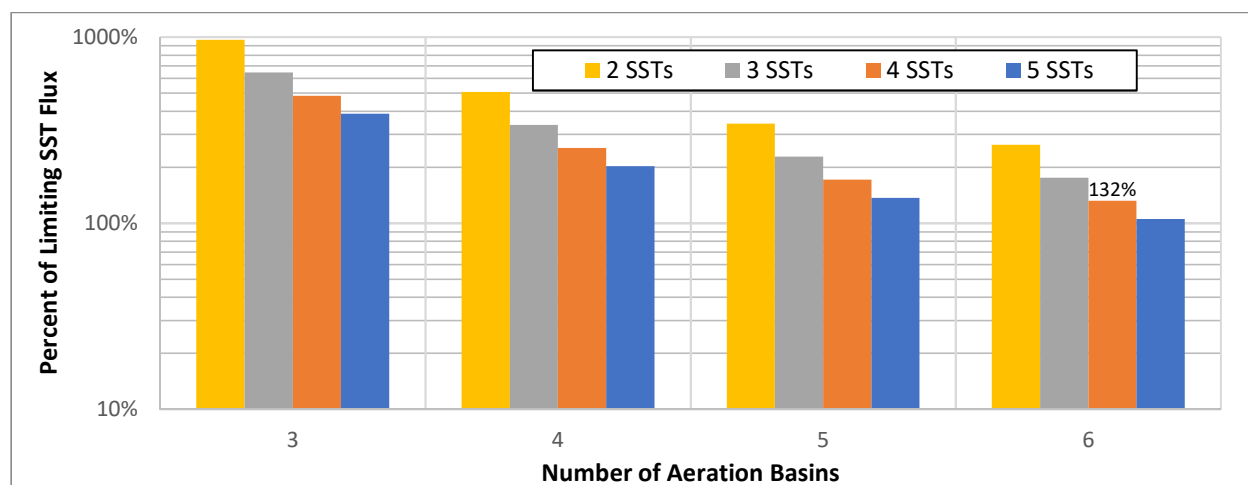


Figure B 5 Secondary Capacity Evaluation for Derby, Ansonia, and Seymour Treatment at Derby

Table B 5 Summary of Combined Derby, Ansonia, and Seymour Treatment at Derby WPCF with Conventional and Intensified Process Alternatives

	CAS	PF+CAS	IFAS	BioMag
Number of Additional ABs	4	2	2	3
Number of Additional SSTs	2	2	2	1
Max Day Aerobic HRT, hours	5.1	3.4	3.4	4.3
Max Day Anoxic HRT, hours	2.5	1.7	1.7	2.1
Max Month MLSS, mg/L	3,670 ⁽¹⁾	3,440 ⁽²⁾	2,900 ⁽²⁾	4,400
Max Day SOR, gpd/ft ²	1,080	1,080	1,080	1,440
Max Day SLR, lb/day/ft ²	38.3	38.0	37.4	68.3
(1) Step Feeding of flows in excess of 8.6 mgd Reduces MLSS at Peak Flows to 3,000 mg/L				
(2) Step Feeding of flows in excess of 9.4 mgd Reduces MLSS at Peak Flows to 3,000 mg/L				
(3) Suspended MLSS Limited to 2,900 mg/L with IFAS through 40% Media Fill (Kaldnes K1)				

Phosphorus Removal

Phosphorus limits are not required for discharge to the Housatonic River at Derby. As such tertiary phosphorus removal processes are not considered for regional alternatives that involve treatment and effluent discharge at Derby.

B.2.2 Treatment at Ansonia

Primary Treatment of Design Flows and Loads

The Ansonia plant's PSTs was reviewed for both Average SOR and peak hour SOR. This reviewed showed that the peak condition was controlling in all cases due to the high flow peaking

factors. The peak hour limit of 3,000 gpd/ft² was converted to a peak daily limit based on an assumed peaking factor. For the base case where the plant is handling Ansonia's wastewater flows only, the peak-hour to peak-day peaking factor is based on the peak flows reported in Phase 1 of this study, while a lower peaking factor (i.e. higher peak day SOR limit) is used for the regional treatment alternatives, as peaking factors tend to reduce in larger collection systems.

Based on these assumptions, Ansonia will not need any additional PSTs to treat its design flows under the base case. If treating the flow from either Derby or Seymour, one PST in addition to the four existing are needed. To treat the combined design flow for all three facilities, two to three PSTs in addition to the four existing PSTs are needed. Due to site limitations it is recommended that CEPT be the preferred option for primary clarifier capacity intensification. If utilizing CEPT, additional PSTs would not be required, though extensive modifications to the PST internals would be required to accept the higher flows.

Table B 6 Ansonia Primary Settling Tank Overflow Rate Analysis at 2040 Design Conditions

	Ansonia	Ansonia + Derby	Ansonia + Seymour	Ansonia + Derby + Seymour
PH SOR Limit, gpd/ft ²	3,000 ⁽¹⁾	3,000 ⁽¹⁾	3,000 ⁽¹⁾	3,000 ⁽¹⁾
PH to PD Flow Peaking Factor	1.8 ⁽²⁾	1.6 ⁽³⁾	1.6 ⁽³⁾	1.6 ⁽³⁾
PD SOR Limit, gpd/ft ²	1,670	1,875	1,875	1,875
PD SOR with 0 New PSTs, gpd/ft ²	1,030	2,170	1,950	3,090
PD SOR with 1 New PSTs, gpd/ft ²	820	1,740	1,560	2,470
PD SOR with 2 New PSTs, gpd/ft ²	690	1,450	1,300	2,060
PD SOR with 3 New PSTs, gpd/ft ²	590	1,240	1,120	1,770
New PSTs Required	0	1	1	2-3
New PSTs Required (CEPT)	0	0	0	0

(1) Based on TR-16 Peak Hour SOR Limit of 3,000 gpd/ft²

(2) Based on PD and PH flows reported in Phase 1

(3) Based on TR-16 Figure 2-1 for Facilities with Average Flows >3 mgd

Secondary Treatment of Design Flows and Loads

Based on our review, intensification processes would not likely be necessary for regional treatment at Ansonia. Figure B 6 below depicts the various regional involving treatment at Ansonia with different numbers of SSTs. Aerobic volume is adequate so only the impact of additional SSTs was considered. As with Derby, the SVI is assumed to be 120 mL/g as the Ansonia facility typically sees good settleability. Generally additional SSTs are not needed to treat Ansonia flows and loads or when bringing only Derby or Seymour flows and loads to Ansonia. For the Ansonia,

Derby, Seymour combined regional alternative, the existing system of two SSTs and two ABs is at 119% of capacity, something which could be managed with by step feeding peak flows if being consistent with the CAS evaluations at Derby. However, to meet effluent P targets in the Ansonia, Derby, and Seymour combined scenario, a lower effluent P concentration must be achieved. Given the concentration required, reducing SST SOR will reduce effluent solids and could allow the effluent target to be met without the need for tertiary filters; however, one additional SST is likely required. Tertiary treatment is assumed for the Ansonia plus Derby and Seymour alternative, and this will be evaluated further. Table B 7 summarizes the results.

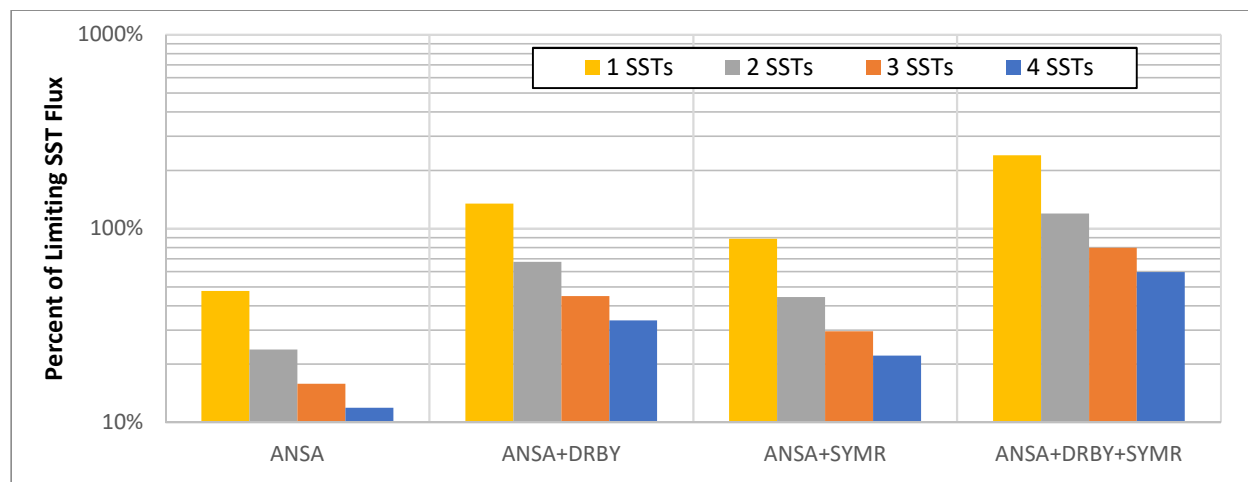


Figure B 6 Secondary Capacity Evaluation for Regional Treatment Alternatives at Ansonia

Table B 7 Ansonia Secondary Process Analysis at 2040 Design Conditions

	Ansonia	Ansonia + Derby	Ansonia +Seymour	Ansonia + Derby +Seymour
Number of Additional ABs	0	0	0	0
Number of Additional SSTs	0	0	0	1
Max Day Aerobic HRT, hours	13.9	7.6	8.5	5.4
Max Day Anoxic HRT, hours	8.0	4.4	4.9	3.1
Max Month MLSS, mg/L	1,498	2,833	1,935	3,515
Max Day SOR, gpd/ft ²	440	840	670	720 ⁽¹⁾
Max Day SLR, lb/day/ft ²	10	28	16	31
(1) Additional SST Provided to Keep SOR Lower for Improve TSS and TP Removal				

Phosphorus Removal

Table B 8 summarizes the effluent P concentrations which would need be targeted to meet seasonal P load limits for each regional alternative that has both treatment and effluent discharge

at Ansonia. Combined load limits are based on P load allocations of 11.92 and 7.54 lb/day for Ansonia and Seymour, respectively, and 0.0 lb/day for Derby. Seymour's load allocation is in line with Ansonia such that the treatment of Seymour and Ansonia together would require similar effluent targets to be met. However, because Derby does not have a total phosphorus (TP) load allocation the concentration targeted drops substantially in alternatives where Derby flow is diverted to the Ansonia plant. With just Ansonia and Derby, the TP limit is in the range where tertiary treatment should be considered. A more thorough evaluation of footprint constraints and operational cost should be undertaken to determine the appropriate tertiary treatment process if this proves to be a viable alternative.

Table B 8 Phosphorus Removal Requirements for Regional Treatment Alternatives at Ansonia

	Ansonia	Ansonia + Derby	Ansonia +Seymour	Ansonia + Derby +Seymour
Design 2040 Average Flow, mgd	1.90	3.82	3.20	5.12
Seasonal P Load Limit, lb/day	11.92	11.92	19.46	19.46
Required Avg. Effluent P, mg-P/L	0.75	0.37	0.73	0.46
Required Avg. Effluent TSS, mg/L	33 ⁽¹⁾	13.7 ⁽¹⁾	31.4 ⁽¹⁾	17.8 ⁽¹⁾
(1) Assuming Effluent Soluble Ortho-P = 0.1 mg-P/L & Effluent TSS is 2% P by Weight				

B.2.3 Treatment at Seymour

Primary Treatment of Design Flows and Loads

The capacity of Seymour's PSTs was assessed based on peak hour SOR. Average SOR was also considered but the peak condition was controlling in all cases due to the high flow peaking factors. The peak hour limit of 3,000 gpd/ft² was converted to a peak daily limit based on an assumed peaking factor. For the all cases, the peak-hour to peak-day peaking factor is based on the peak flows reported in Phase 1 of this study.

Based on these assumptions, to treat the 2040 design flows for Seymour (base case), only one new PST is needed in addition to the two existing ones. This is not changed if adding the relatively low flows from Beacon Falls. However, due to site limitations it is recommended that CEPT be the preferred option for primary clarifier capacity intensification. If utilizing CEPT, additional PSTs would not be required, though extensive modifications to the PST internals would be required to accept the higher flows.

Table B 9 Seymour Primary Settling Tank Overflow Rate Analysis at 2040 Design Conditions

	Seymour	Seymour + Beacon Falls
PH SOR Limit, gpd/ft ²	3,000 ⁽¹⁾	3,000 ⁽¹⁾
PH to PD Flow Peaking Factor	2.1 ⁽²⁾	2.1 ⁽²⁾
PD SOR Limit, gpd/ft ²	1,430	1,430
PD SOR with 0 New PSTs, gpd/ft ²	1,690	2,160
PD SOR with 1 New PSTs, gpd/ft ²	1,130	1,440
PD SOR with 2 New PSTs, gpd/ft ²	850	1,080
New PSTs Required	1	1
New PSTs Required (CEPT)	0	0
(1) Based on TR-16 Peak Hour SOR Limit of 3,000 gpd/ft ²		
(2) Based on PD and PH flows reported in Phase 1		

Secondary Treatment of Design Flows and Loads

Settleability is a significant problem at Seymour, with SVIs often greater than 200-300 mL/g. An SVI of 200 mL/g was assumed in this evaluation. Operators reported that at current loads, treatment can be challenging when settleability is poor and flows are high, with high sludge blankets encountered. Assuming an SVI of 200 mL/g, Figure B 7 shows the requirement for additional SSTs for treatment of Seymour's flows and loads only (base case) and for the addition of Beacon Falls. This shows that with two SSTs, the capacity is exceeded by peak flows and loads from the regional alternative that has Beacon Falls wastewater treated at Seymour. It is possible that process improvements such as better selector zone design, better dissolved oxygen control, or selective wastage could be utilized to improve settling; however, it is recommended for the purpose of this evaluation that an additional SST be identified as required. With an additional SST higher SVIs of 300 mL/g could be managed. Process intensification was not considered because of the relatively minor expansion required and because there is sufficient space near the existing SSTs for construction of one new SST. Table B 10 summarizes the results of this evaluation.

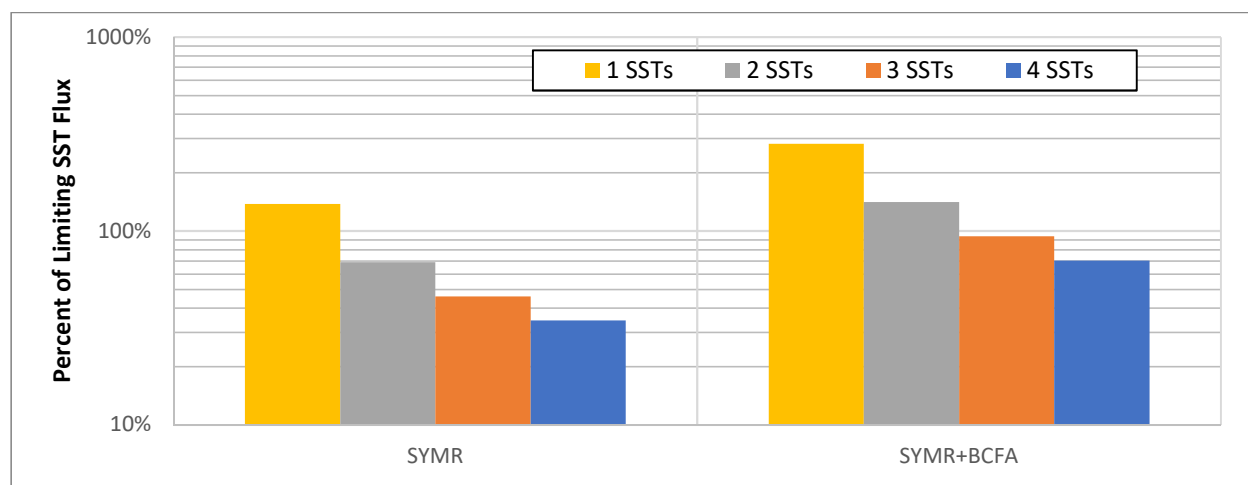


Figure B 7 Secondary Capacity Evaluation for Regional Treatment Alternatives at Seymour

Table B 10 Summary of Conventional Secondary Capacity for Regional Treatment Alternatives at Seymour

	Seymour	Seymour + Beacon Falls
Number of Additional ABs	0	0
Number of Additional SSTs	1	1⁽¹⁾
Max Day Aerobic HRT, hours	5.6	4.3
Max Day Anoxic HRT, hours	2.8	2.2
Max Month MLSS, mg/L	2,370	3,330
Max Day SOR, gpd/ft ²	450	580 ⁽¹⁾
Max Day SLR, lb/day/ft ²	12.6	26.5
(1) Additional SST Provided due to higher SVI Assumption (200 mL/g)		

Phosphorus Removal

Taking the same approach as evaluating the need for tertiary chemical P removal for regional treatment at Ansonia, Table B 11 shows the results for regional treatment at Seymour. Because both Seymour and Beacon Falls have TP load allocations which are in line with each other on a flow basis, the concentration targeted does not drop with the additional flow from Beacon Falls. At a effluent target of 0.7 mg-P/L, it does not appear that tertiary solids removal is required.

Table B 11 Phosphorus Removal Requirements for Regional Treatment Alternatives at Seymour

	Seymour	Seymour + Beacon Falls
Design 2040 Average Flow, mgd	1.30	1.75
Seasonal P Load Limit, lb/day	7.54	10.21
Required Avg. Effluent P, mg-P/L	0.70	0.70
Required Avg. Effluent TSS, mg/L	30	30
(1) Assuming Effluent Soluble Ortho-P = 0.1 mg-P/L & Effluent TSS is 2% P by Weight		

B.2.4 Treatment at Naugatuck

A general evaluation of the Naugatuck WPCF was performed, which showed that the plant has adequate capacity for its own current and future needs, and that it would the capacity to handle the added flow from Beacon Falls if the conveyance pipeline was feasible. The projected 2035 flows from the facilities plan with the addition of Middlebury and Oxford flows are 7.57 mgd average daily flow. With the addition of Beacon Falls 2040 projected average daily flow of 0.45 this is an average daily flow of 8.02 mgd. Currently the rated capacity of Naugatuck WPCF is 10.5 mgd average daily flow. Because Beacon Falls wastewater is also of typical domestic concentrations, this additional wastewater flow will be able to be treated at the Naugatuck plant without upgrades. For Naugatuck to treat projected or design flows while still maintaining the same degree of nitrogen removal may require some reconfiguration of BNR basin layout, however this is potentially an issue regardless of whether Beacon Falls is connected to Naugatuck or not. Because conveyance from Beacon Falls was determined to be infeasible for this study, an in-depth evaluation of the Naugatuck facility is not summarized here.

APPENDIX D

TECHNICAL MEMORANDUM 4: REGIONAL WASTEWATER ALTERNATIVES SHORT LIST DEVELOPMENT

REGIONAL WASTEWATER TREATMENT CONSOLIDATION STUDY

Technical Memorandum 4: Regional Wastewater Alternatives Short List Development

B&V PROJECT NO. 198910

PREPARED FOR

Naugatuck Valley Council of Governments

18 February 2021

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1.0 BACKGROUND AND PURPOSE

This technical memorandum (TM) No. 4 is a continuation of the regional wastewater treatment consolidation study being carried out by the Naugatuck Valley Council of Governments (NVCOG). The Task 2 work within this Phase 2 effort resulted in a total of seven regional alternatives to be carried forward into this Task 3, with an end goal of identifying the preferred alternative(s). The seven short-listed regional alternatives are identified below.

Table 1-1 Short List of Regional Wastewater Alternatives

No.	Alternative Description
3	Derby to Ansonia
4	Derby to Ansonia, Effluent Pumped to Housatonic River
5	Derby and Seymour to Ansonia
5b	Derby and Seymour to Ansonia, Effluent Pumped to Housatonic River
8	Ansonia to Derby
9	Seymour and Ansonia to Derby

This TM summarizes the work carried out in Task 3. The work effort is essentially synthesized in the following statements. The report is also organized to follow this structure.

1. More detailed development of the regional alternatives. The short-list regional alternatives have undergone more detailed analysis and development along with the 'base case' alternatives where each of the communities would continue to handle, treat and discharge their wastewater as they are currently doing. Treatment facility and wastewater conveyance systems infrastructure requirements are more fully defined. The collection systems are also addressed.
2. Budgetary cost development. Budgetary capital costs were assigned to each of the shortlisted regional alternatives and base case scenarios. Operations and maintenance (O&M) costs were also assigned for the regional alternatives and base case facilities.
3. Cost evaluation/analysis of the regional alternatives and base case facilities. This analysis will allow for a comparison of the regional alternatives and base case scenarios on both a capital cost and life cycle (present worth) basis. The present worth analysis allows for the capital and O&M costs to be converted, allowing for the alternatives to be compared on a present worth dollar basis.
4. Recommended alternatives. The forgoing information will allow for the preferred alternative(s) to be identified on a cost analysis basis. While other factors will contribute to the final decision, that is not part of this existing work task.

2.0 WASTEWATER TREATMENT PLANT DEVELOPMENT

2.1 INTRODUCTION

The technical study and engineering detail at the wastewater treatment plants was expanded further to define the planning level infrastructure needs associated with the short-listed regional alternatives and base case scenarios. Task 2 development efforts focused on wastewater process engineering on the parts of the plant that are traditionally more land intensive (i.e. primary and secondary treatment). Process related engineering and planning was continued in this task for each of the regional plants and base case facilities. This allowed for confirmation associated with the identified infrastructure needs (e.g. tanks) and the operating requirements for these systems (e.g. energy, chemical usage) that would be used in the present worth comparison of alternatives.

In addition, other critical parts of the plants that had not been addressed in Task 2 development were also better defined. These included: influent and effluent pumping systems, preliminary treatment (screenings and grit removal), effluent disinfection, sludge processing, treatment and disposal, plant administration facilities/buildings, and other major systems (e.g. electrical and SCADA).

General process considerations and data updates applying to the plants are described in Appendix A. Sludge management applying to the plant is described in Appendix B.

2.2 BASE CASES PLANTS

The base case wastewater treatment plant requirements for Derby, Ansonia and Seymour documented in Phase 1 and in Task 2 deliverables were developed further in this task. The resultant work has allowed for a more complete picture of the infrastructure needs at these plants and allowed for the associated upgrade costs to be established.

2.2.1 Derby

2.2.1.1 Performance

The historic effluent N loads at Derby are shown in Figure 2-1 below alongside the N General Permit waste load allocation (WLA). What this shows is that in some years the WLA is exceeded, and N credits must be purchased from the state, while in some other years the WLA is met. As established above, at current flows Derby would need to achieve an effluent TN of 6.0 mg/L-N on average in order to meet the WLA. With the current modified Ludzack-Ettinger process configuration, meeting this WLA is challenging during all months of the year and at various loadings. As the flows and loads increase during the study period, the required effluent concentration will decrease accordingly to approximately 5.0 mg/L-N. As this happens, either N credits will need to be purchased more frequently, or process upgrades such as the addition of a post-anoxic zone and supplemented carbon feed system will be required.

The addition of a post-anoxic zone would allow the facility to more consistently meet the N WLA even as flow increases in the future. Additionally, these post anoxic zones should be set up as swing zones, allowing the plant to operate these as:

- Un-aerated anoxic zones with carbon dosed for greater removal of N during warmer periods, and

- Aerated during colder months as needed in order to retain capacity.

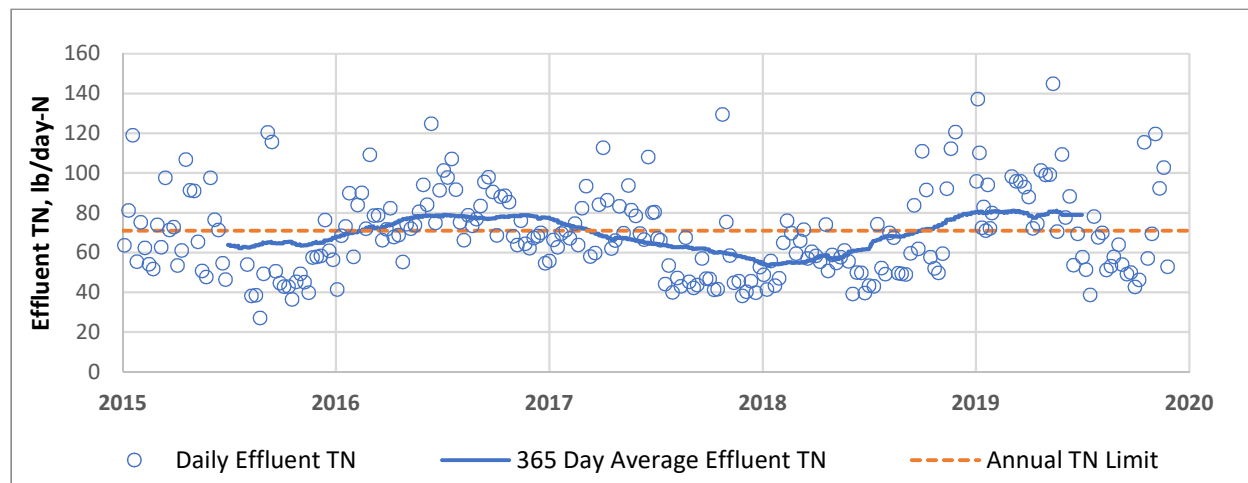


Figure 2-1 Derby Effluent N Loads

2.2.1.2 Capacity

In the previous Task, it was determined that to meet primary treatment capacity requirement at 2040 design conditions, additional primary clarifiers would not likely be needed; however, it was noted that there may be some reduction in primary solids removal performance as flows increase.

In the previous Task, it was determined that to meet secondary capacity requirements, the third aeration basin, which has not been upgraded to an MLE process for nitrogen removal as of 2020, would need to be upgraded to meet 2040 requirements. With the lower growth projections from the recent Derby Facility Plan (resulting in an annual average flow of 1.59 mgd), the need for this upgrade was reevaluated. The upgrade of the third basin is still recommended as it will meet capacity requirements without the need for step feeding. The facility plan recommends that an additional structure be built as a common reaeration zone. We believe that a 4-stage process with swing post-anoxic zones and reaeration zones can be incorporated into the footprint of the three existing aeration basins and that this additional external tankage is not necessary.

The facility plan also recommends the construction of two new deeper secondary clarifiers with the construction of a mixed liquor flow splitting structure and RAS pumping station. Additionally, the plan calls for the rehabilitation of one existing secondary clarifier and the demolition of the other. While newer and deeper secondary clarifiers would improve performance and add redundancy, we believe improving sludge settleability to be more important to achieve capacity and performance. The “addition” of the third aeration basin (upgrade of existing inoperable basin) should be adequate to meet the increase in loading with similar performance as is currently achieved. The additional aerobic volume along with settling rate enhancements and refurbishment of the existing secondary clarifiers, mixed liquor splitter, and RAS pumping system will allow the plant to achieve similar performance as it has historically at the projected future flow. Table 2-1 summarizes capacity parameters at Derby in 2040 based on the suggested upgrades.

Table 2-1 Derby Capacity Parameters in 2040

Facility Requirements and Capacities	2040 Annual Average ⁽¹⁾	2040 Max Month
Additional Primary Clarifiers	0	0
Primary Clarifier SOR, gal/day/ft ²	550	850
Additional Aeration Basins ⁽²⁾	1	1
Additional Secondary Clarifiers	0	0
Aeration Basin Total HRT, hrs	20.3	13.2
Aeration Basin MLSS, mg/L	1870	3430
Secondary Clarifier SOR, gal/day/ft ²	560	430
Secondary Clarifier SLR, lb/day/ft ²	15.6	24.9
(1) With one secondary clarifier offline at average loading conditions. (2) Derby has two existing aeration basins that are operable and one aeration basin that is inoperable. In the process evaluation, it was assumed that the inoperable basin would be upgraded to meet additional aeration basin needs.		

This subsection and the one proceeding it have addressed the need for additional primary and secondary liquid stream unit processes. In addition to the upgrades identified to these two process areas to increase capacity for 2040 flow and load conditions, there are numerous capital improvements required at other areas of the Derby plant. These are required to address poor condition, age/usefulness, inefficiencies and treatment bottlenecks throughout the plant and are highlighted in the next subsection.

2.2.1.3 Other Needed Upgrades

Table 2-2 lists the upgrades needed at the Derby wastewater plant based on the condition assessment performed as a part of Phase 1 and additional investigations conducted since that time. These upgrades, which cover both existing structures and equipment were identified as being needed from observations made during site visits and from information corroborated by Derby plant staff. These upgrades are required for the Derby base case; they were also carried into regional alternatives as applicable.

Table 2-2 Derby Facility Upgrades Needed

Area/Facility	Upgrades Required
Raw wastewater Screening Facility and Influent Pump Station	<ul style="list-style-type: none"> Replace manually cleaned trash racks with two mechanical screens for greatly improved process performance and redundancy. Include screenings washer/grinder compactor system Replace existing ventilation system with improved HVAC. Include odor control systems to remove H₂S gas and other influent odorous compounds Replace influent pumps, piping, valves, electrical components, VFDs and controls Repair damaged concrete, reconfigure intermediate platforms
Grit Removal Facility	<ul style="list-style-type: none"> Demolish existing aerated grit facility Construct new vortex grit removal facility
Primary Clarifiers	<ul style="list-style-type: none"> Replace mechanisms Remove channel mounted comminutor Repair damaged concrete
Aeration Basins	<ul style="list-style-type: none"> Replace air piping, diffusers, MLR pumps, mixers, valves, gates, and instrumentation for all three aeration basins (refer also to requirements described above)
Secondary Control Building	<ul style="list-style-type: none"> Replace blowers, aeration piping, and valves Replace RAS and WAS pumps, piping, and valves Partition sludge pumps off from blowers and upgrade HVAC system to protect blowers and controls from corrosive wastewater gases
Secondary Clarifiers	<ul style="list-style-type: none"> Replace mechanisms Upgrade flow splitter box to improve hydraulic balance between clarifiers
Disinfection	<ul style="list-style-type: none"> Upgrade sodium hypochlorite and sodium bisulfite chemical feed systems
Sludge Handling Facility	<ul style="list-style-type: none"> Demolish aerobic digesters Demolish sludge belt filter press and polymer feed system Construct new sludge handling facility to process thickened sludge for ultimate disposal (refer to appendix for detail)
Primary Control Building	<ul style="list-style-type: none"> Upgrade control building with remodeled interior and new HVAC system
Electrical and Control Systems	<ul style="list-style-type: none"> Replace all motor control centers and power/lighting panels Add a new plant supervisory control and data acquisition (SCADA) system
General	<ul style="list-style-type: none"> Add grating or platforms above water surfaces where needed Plant-wide structural concrete repairs Replace plant water system Replace underground process piping as needed

A brief description of some of the more significant facility upgrades in the above table is provided below

2.2.1.3.1 New Facilities and Reconfigured Existing Facilities

New facilities proposed for the Derby base case include a Grit Removal Facility and Sludge Handling Facility. Existing facilities that will undergo a major upgrade include the Influent Pump Station. Preliminary layouts of these facilities were developed for capital cost development. The reconfigured Raw Wastewater Screening Facility and Influent Pump Station is shown in Figure 2-2 and the new Grit Removal Facility is shown in Figure 2-3 (note that two vortex grit chambers are indicated, which applies to regional plants described later in this chapter; one vortex grit chamber is assumed for the Base Case); the new Sludge Handling Facility is shown in Figure B 1 in Appendix B.

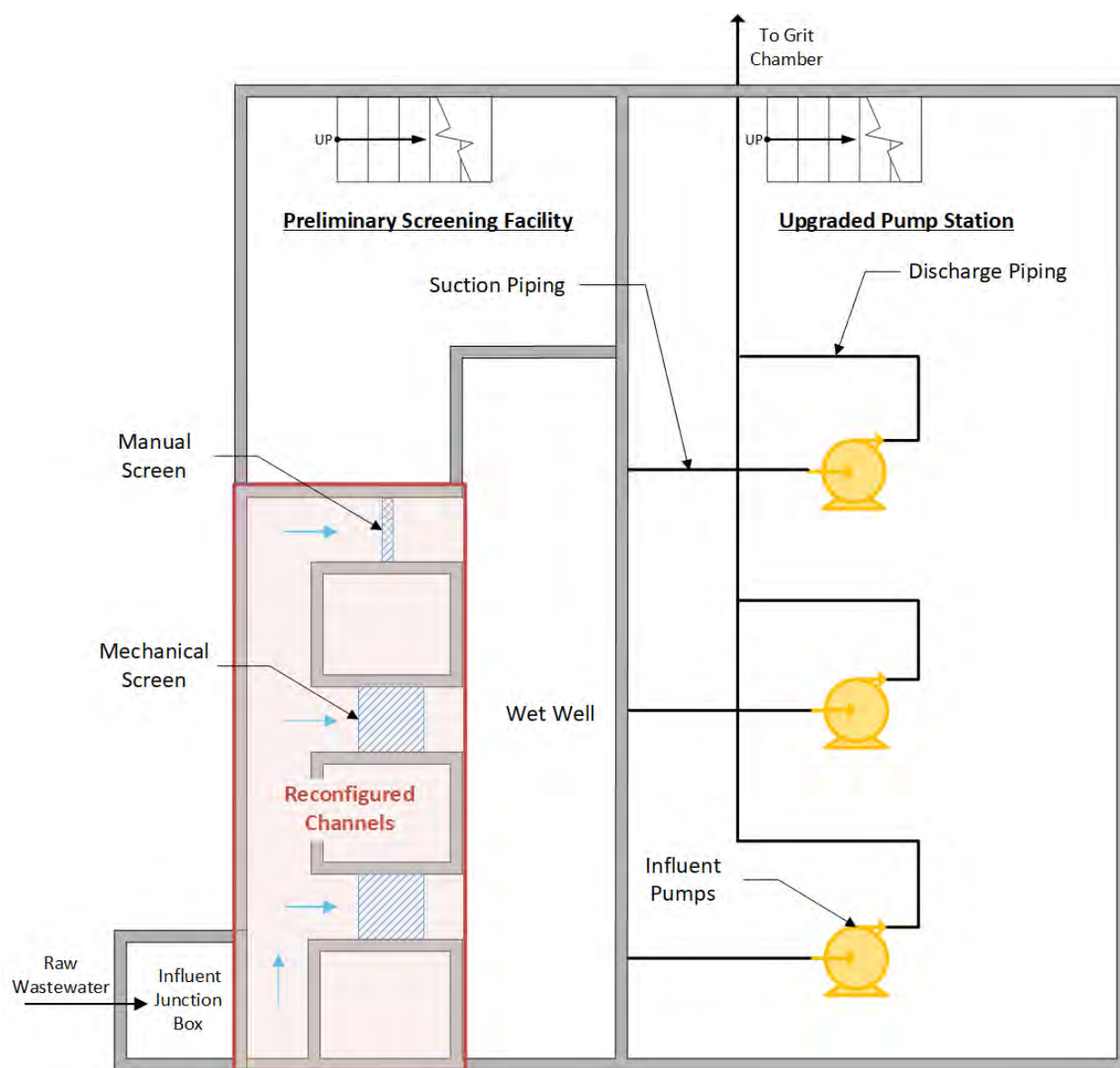


Figure 2-2 Preliminary Screening and Influent Pump Station for Derby

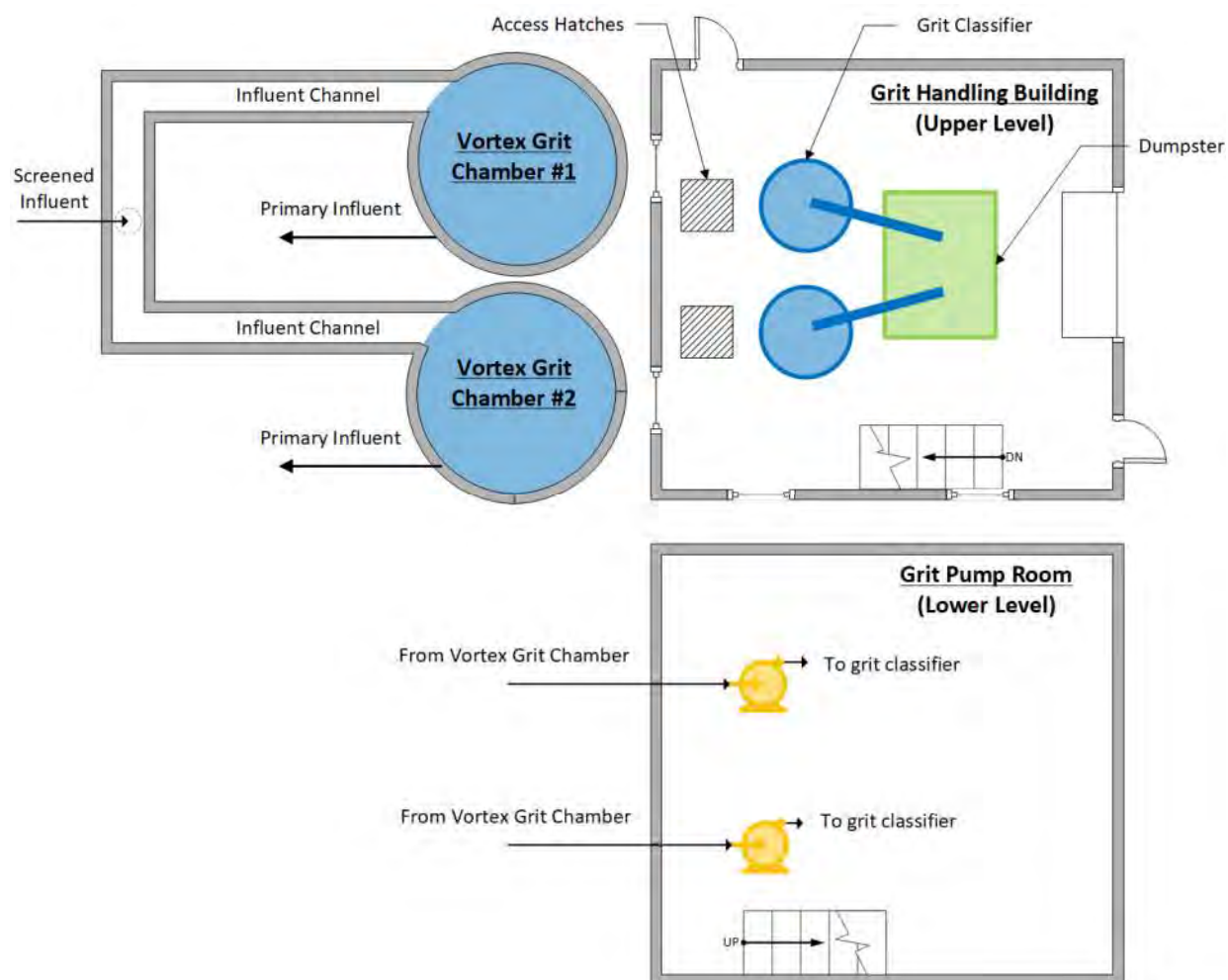


Figure 2-3 New Grit Removal Facility for Derby

Derby Base Case Site LayoutFigure 2-4 shows the conceptual site layout of the Derby WPCF base case.

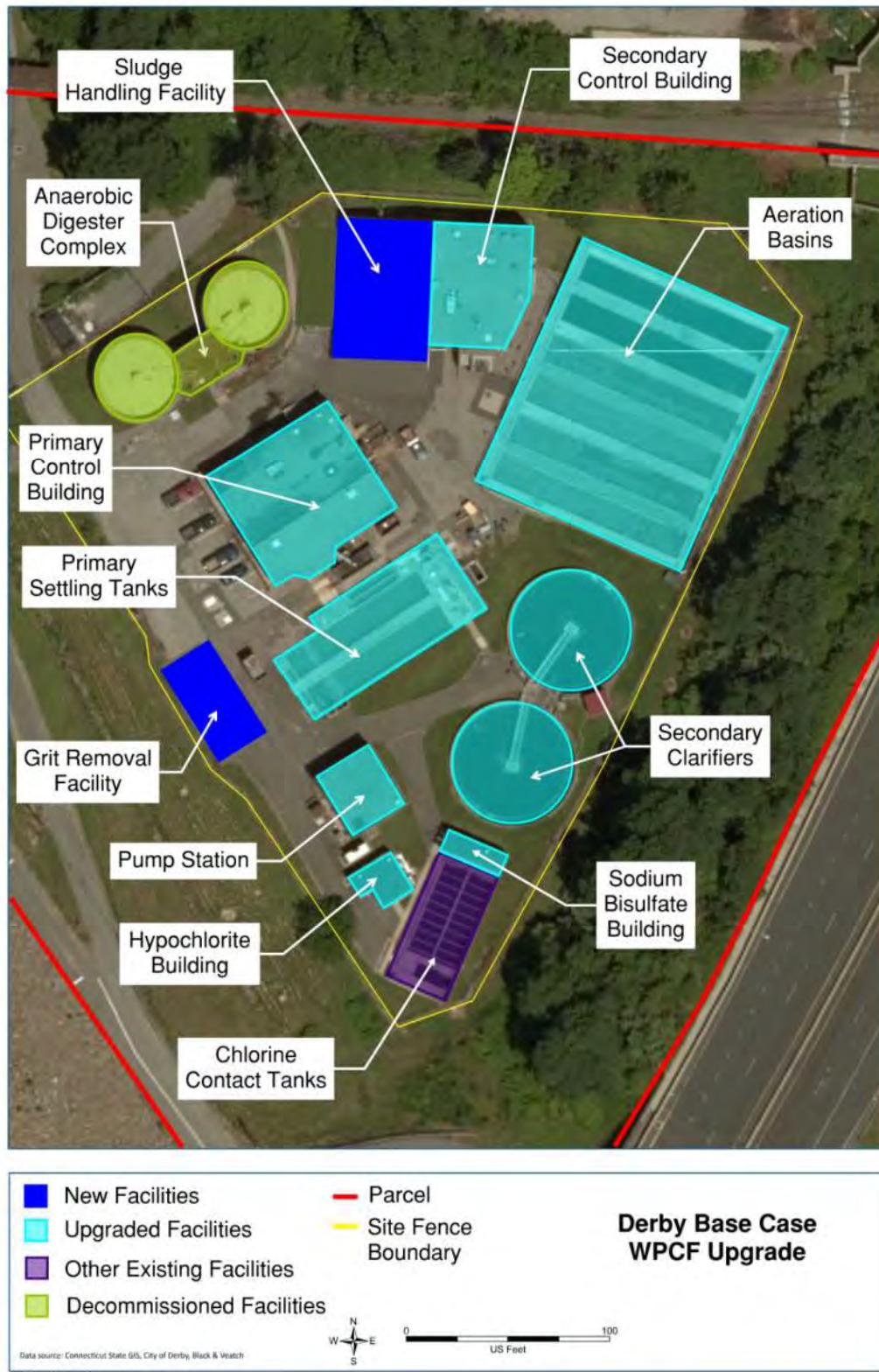


Figure 2-4 Derby Base Case Site Layout

2.2.2 Ansonia

2.2.2.1 Performance

The historic effluent N loads at Ansonia are shown in Figure 2-5 below alongside the N General Permit waste load allocation (WLA). The effluent N load is generally less than half of the WLA meaning that Ansonia is consistently a seller of N credits to the state credit trading program. As stated above, Ansonia's target effluent N concentration to meet the WLA is higher than those at Derby or Seymour with the average effluent TN to meet the load limit being 7.2 mg/L-N at 2040 flows. Despite the higher limits, Ansonia performs very well with average effluent TN of 3.3 mg/L-N. This is almost certainly due to the oxidation ditch configuration which operates at lower dissolved oxygen levels and achieves simultaneous nitrification and denitrification. Additionally, the facility has a 4-stage process, which has pre- and post-anoxic zones upstream and downstream of the oxidation ditch.

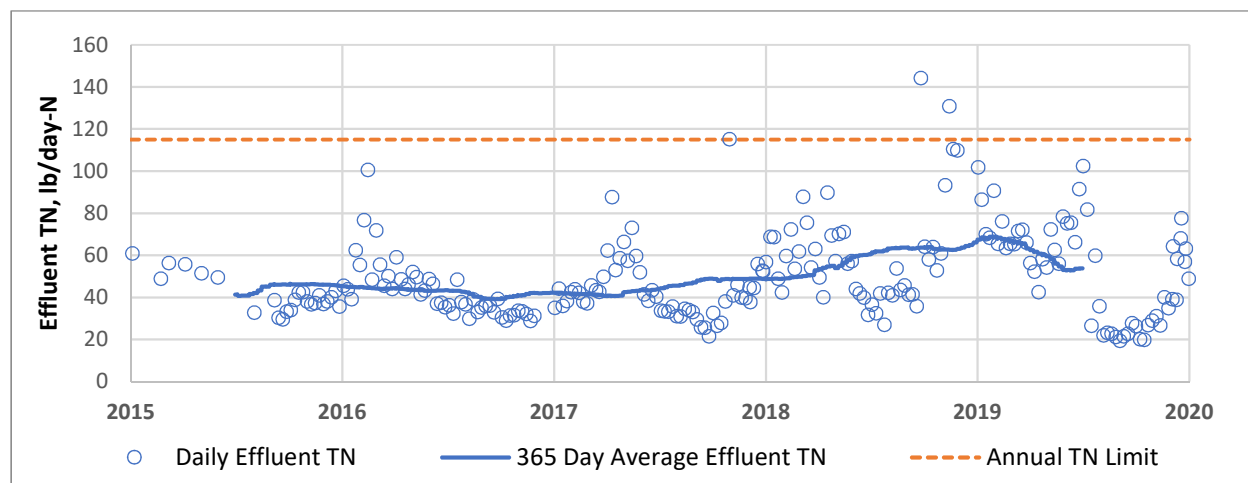


Figure 2-5 Ansonia Effluent N Loads

Ansonia is required to meet a seasonal phosphorus load limit of 11.92 lb/day from April through October. Based on average flows, this requires the facility to achieve a concentration of approximately 0.8 mg/L-P on average during that period, though the actual concentration required is greater than 0.9 mg/L-P as flows in the summer months are lower. Based on 2040 flows the requirement decreases to approximately 0.75 mg/L-P on average. These concentrations can be achieved through the dosage of chemical coagulant to the secondary process which has already been implemented at Ansonia.

2.2.2.2 Capacity

In the previous Task, it was determined that to meet primary treatment capacity requirements at 2040 design conditions, additional primary clarifiers will not likely be needed. Additionally, secondary capacity was determined to not be limiting at Ansonia in 2040. In this task, the capacity at Ansonia was revisited based on the refinements to the capacity and performance evaluation. Growth projection at Ansonia have not changed and CEPT is not necessary.

Based on biokinetic modeling, a more conservative SRT was selected for Ansonia. Additionally, a more thorough review of SVI data indicated that higher SVIs, corresponding to worse settling, should be assumed. If maximum month SVIs of approximately 200 mL/g is assumed,

state point analysis still indicates that there should be no issues in meeting the 2040 capacity requirements. Table 2-3 summarizes capacity parameters at Ansonia in 2040 without any additional primary and secondary treatment upgrades.

Table 2-3 Ansonia Capacity Parameters in 2040

Facility Requirements and Capacities	2040 Annual Average ⁽¹⁾	2040 Max Month
Additional Primary Clarifiers	0	0
Primary Clarifier SOR, gal/day/ft ²	390	680
Additional Aeration Basins	0	0
Additional Secondary Clarifiers	0	0
Aeration Basin Total HRT, hrs	38.1	21.9
Aeration Basin MLSS, mg/L	900	1,500
Secondary Clarifier SOR, gal/day/ft ²	5.0	7.8
Secondary Clarifier SLR, lb/day/ft ²	340	290
(1) With one secondary clarifier offline at average loading conditions.		

This subsection and the one proceeding it have addressed the need for additional primary and secondary liquid stream unit processes. In addition to the upgrades identified for these two process areas to increase capacity for 2040 flow and load conditions, there are capital improvements required at other areas of the Ansonia plant. These are required to address poor condition, age or usefulness, inefficiencies and treatment bottlenecks throughout the plant and are highlighted in the next subsection.

2.2.2.3 Other Needed Upgrades

Table 2-4 lists the upgrades needed at the Ansonia wastewater plant based on the condition assessment performed as a part of Phase 1 of this study and additional investigations conducted since that time. These upgrades, which cover both existing structures and equipment were identified as being needed from observations made during site visits and from information corroborated by Ansonia plant staff. These upgrades are required for the Ansonia base case; they were also carried into regional alternatives as applicable.

Table 2-4 Ansonia Facility Upgrades Needed

Area/Facility	Upgrades Required
Headworks	<ul style="list-style-type: none"> • Add second mechanical screen for added redundancy and ability to bypass the existing single screen • Replace ventilation and odor control system for headworks area to improve air quality and reduce corrosive H₂S gas concentrations
Disinfection	<ul style="list-style-type: none"> • Build new UV disinfection channel for added redundancy and ability to bypass the existing single UV channel
Effluent Pump Station	<ul style="list-style-type: none"> • Upgrade effluent pumps to meet peak flows (current system limited to 7 mgd)
Sludge Handling Facility	<ul style="list-style-type: none"> • Demolish existing sludge holding tanks • Build new sludge handling facility to process thickened sludge (refer to Appendix for details)
General	<ul style="list-style-type: none"> • Demolish non-functioning soda ash storage and feed system • Plant-wide structural concrete repairs

2.2.2.3.1 New Facilities and Reconfigured Existing Facilities

New facilities proposed for the Ansonia base case include a Sludge Handling Facility, shown in Figure B 1 in Appendix B. While no significant or large-scale facility retrofits are proposed for the Ansonia base case, as noted above, a number of moderate ones have been identified as being needed.

2.2.2.4 Ansonia Base Case Site Layout

Figure 2-6 shows the conceptual site layout of the Ansonia WPCF base case.

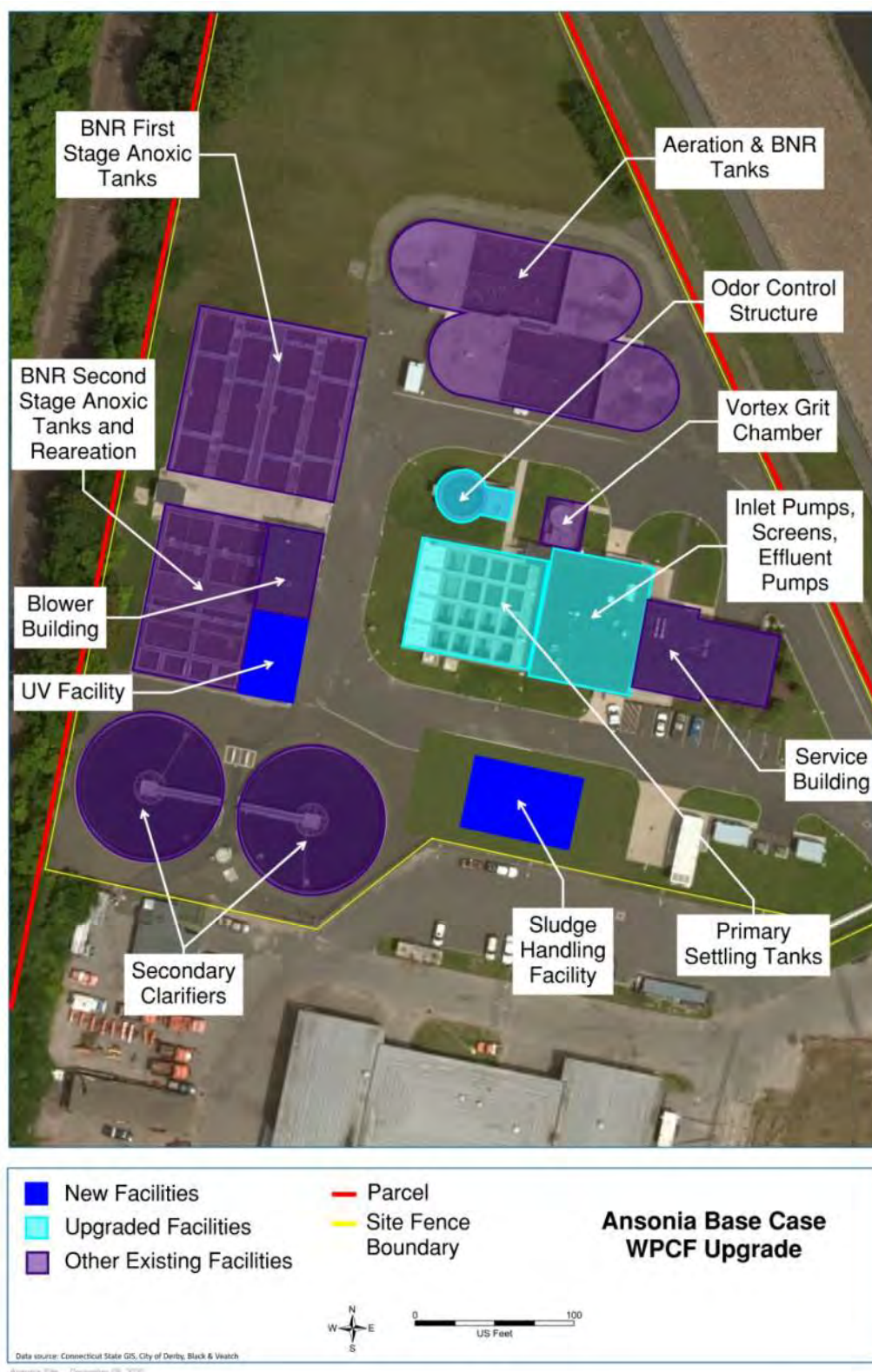


Figure 2-6 Ansonia Base Case Site Layout

2.2.3 Seymour

2.2.3.1 Performance

The historic effluent N loads at Seymour are shown in Figure 2-7 below alongside the N General Permit waste load allocation (WLA). What this shows is that in some years the WLA is exceeded, and N credits must be purchased from the state, while in some years the limits are met. As established above, at current flows Seymour would need to achieve an effluent TN of approximately 7.5 mg/L-N on average in order to meet the WLA. To meet this WLA, the current modified Ludzack-Ettinger process should be adequate provided that MLR pump system and anoxic zones are properly sized and that adjustments are made to improve the environment in the anoxic zone such that the process is more efficient. In addition to design, the influent C:N ratio impacts N removal performance, and the lower than average C:N ratio in the influent at this facility may be one of the issues to Seymour meeting its N target during various times of the year. Meeting this WLA will become more challenging as the flow and loads increase, with the required effluent concentration to meet the limit at 2040 flows being approximately 5.6 mg/L-N. As this happens, either N credits will need to be purchased more frequently or process upgrades will need to be explored. This could include increasing the MLR pump system size and adding a supplemental carbon feed system, or it could also mean the addition of a post-anoxic zone to create a 4-stage process as recommended at Derby. In summary, due to the relatively small scale of Seymour, and our observations at the plant, operational based changes with more moderate capital investment should be investigated to improve the system's N removal capability. If this is found to be effective, the Town, if needed, should expect to buy N credits when certain conditions arise when operational changes are not adequate to meet the N General Permit load allocation.

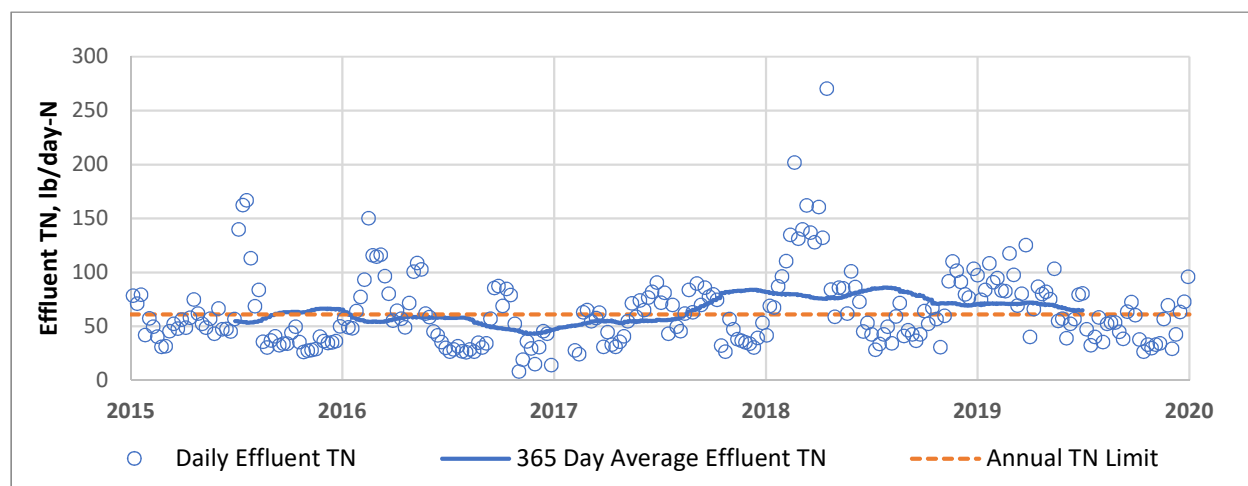


Figure 2-7 Seymour Effluent N Loads

With the recent permit renewal, Seymour is required to meet a seasonal phosphorus load limit of 7.54 lb/day from April through October. Based on annual average flows this requires the facility to achieve a concentration of approximately 0.9 mg/L-P on average during that period; however, the actual required concentration may be greater than 1.0 mg/L-P because plant records indicate that flows in the summer months are typically lower. Based on 2040 flow projections, the requirement decreases to approximately 0.7 mg/L-P on average. These concentrations can be achieved through

the dosage of chemical coagulant to the secondary process which has already been implemented at Seymour.

2.2.3.2 Capacity

In the previous Task, it was determined that to meet primary treatment capacity requirement at 2040 design conditions, additional primary clarifiers will not likely be needed. Additionally, secondary capacity was determined to not be limiting at Seymour under 2040 conditions. Our analysis during that Task also found that if the Seymour plant was to accept Beacon Falls wastewater flow then, process improvements to enhance settleability at Seymour would need to be implemented. However, because this long-list regional alternative (Beacon Falls to Seymour) was eliminated in Task 2, an additional settling tank at Seymour is not required. In this task, the capacity at Seymour was revisited based on the refinements to the capacity and performance evaluation. Growth projection at Seymour has not changed and CEPT is not necessary and so the conclusion for the Seymour base case remains largely the same as had been reported in Task 2. Table 2-5 summarizes capacity parameters at Seymour in 2040 without any additional upgrades to secondary treatment.

Table 2-5 Seymour Capacity Parameters in 2040

Facility Requirements and Capacities	2040 Annual Average ⁽¹⁾	2040 Max Month
Additional Primary Clarifiers	0	0
Primary Clarifier SOR, gal/day/ft ²	490	970
Additional Aeration Basins	0	0
Additional Secondary Clarifiers	0	0
Aeration Basin Total HRT, hrs	16.7	8.4
Aeration Basin MLSS, mg/L	1,420	2,370
Secondary Clarifier SOR, gal/day/ft ²	390	390
Secondary Clarifier SLR, lb/day/ft ²	10.0	16.6
(1) With one secondary clarifier offline at average loading conditions.		

This subsection and the one proceeding it have addressed the need for additional primary and secondary liquid stream unit processes. In addition to the upgrades identified for these two process areas to increase capacity for 2040 flow and load conditions, there are numerous capital improvements required at other areas of the Seymour plant. These are required to address poor condition, age/usefulness, inefficiencies and treatment bottlenecks throughout the plant and are highlighted in the next subsection.

2.2.3.3 Other Needed Upgrades

Table 2-6 lists the upgrades needed at the Seymour wastewater plant based on the condition assessment performed as a part of Phase 1 of this study and additional investigations

conducted since that time. These upgrades, which cover both existing structures and equipment were identified as being needed from observations made during site visits and from information corroborated by Seymour plant staff. These upgrades are required for the Seymour base case; they were also carried into regional alternatives as applicable.

Table 2-6 Seymour Facility Upgrades Needed

Area/Facility	Upgrades Required
Headworks	<ul style="list-style-type: none"> • Complete refurbishment of the preliminary treatment system which includes screening and grit removal • Replace mechanical screen and add redundant unit. Add washer grinder compactor • Replace grit removal equipment • Add enclosure structure, ventilation (with odor control) to improve operations and maintenance during all seasons of the year
Influent Pump Station	<ul style="list-style-type: none"> • Replace pumps, piping, valves and electrical components, VFDs and controls
Primary Clarifiers	<ul style="list-style-type: none"> • Replace mechanisms; modify for more efficient operation
Aeration Basins	<ul style="list-style-type: none"> • Replace air piping, diffusers, MLR pumps, mixers, valves, gates, and instrumentation
Old Digester Complex	<ul style="list-style-type: none"> • Add one additional aeration turbo blower for redundancy • Demolish existing multi-stage centrifugal blowers
Control Building	<ul style="list-style-type: none"> • Replace RAS and WAS pumps • Demolish rotary drum thickener, belt filter press, and polymer system • Add new gravity belt thickeners and associated systems to manage sludge as a thickened liquid • Upgrade HVAC and odor control systems to improve air quality and remove H₂S produced by sludge processing
Secondary Clarifiers	<ul style="list-style-type: none"> • Replace mechanisms (feed well, scrapers, skimmers, scum collector, baffles and weirs). Inspect bridge to determine need for refurbishment or replacement
Sludge Handling Facility	<ul style="list-style-type: none"> • Demolish aerobic digesters • Demolish sludge belt filter press and polymer feed system • Build new sludge handling facility to process thickened sludge (refer to Appendix for details)
Primary Control Building	<ul style="list-style-type: none"> • Upgrade/refurbish control building interior
Electrical and Control Systems	<ul style="list-style-type: none"> • Replace switchgear, motor control centers and power/lighting panels predating the early 1990s upgrade • Add a new plant supervisory control and data acquisition (SCADA) system
General	<ul style="list-style-type: none"> • Plant-wide structural concrete repairs

2.2.3.3.1 New Facilities and Reconfigured Existing Facilities

Major new/or significantly upgraded facility for the Seymour base case includes the sludge handling system to manage sludge as a thickened liquid instead of dewatered cake. This process configuration is depicted in Figure B 1 in Appendix B. Other significant upgrades involve the influent pump station, the preliminary treatment systems (both screening and grit removal), and new equipment at the primary clarifiers, the secondary clarifiers, and modifications at the aeration basins. Major upgrade is also needed on the plant electrical power system. A new

SCADA system is also required.

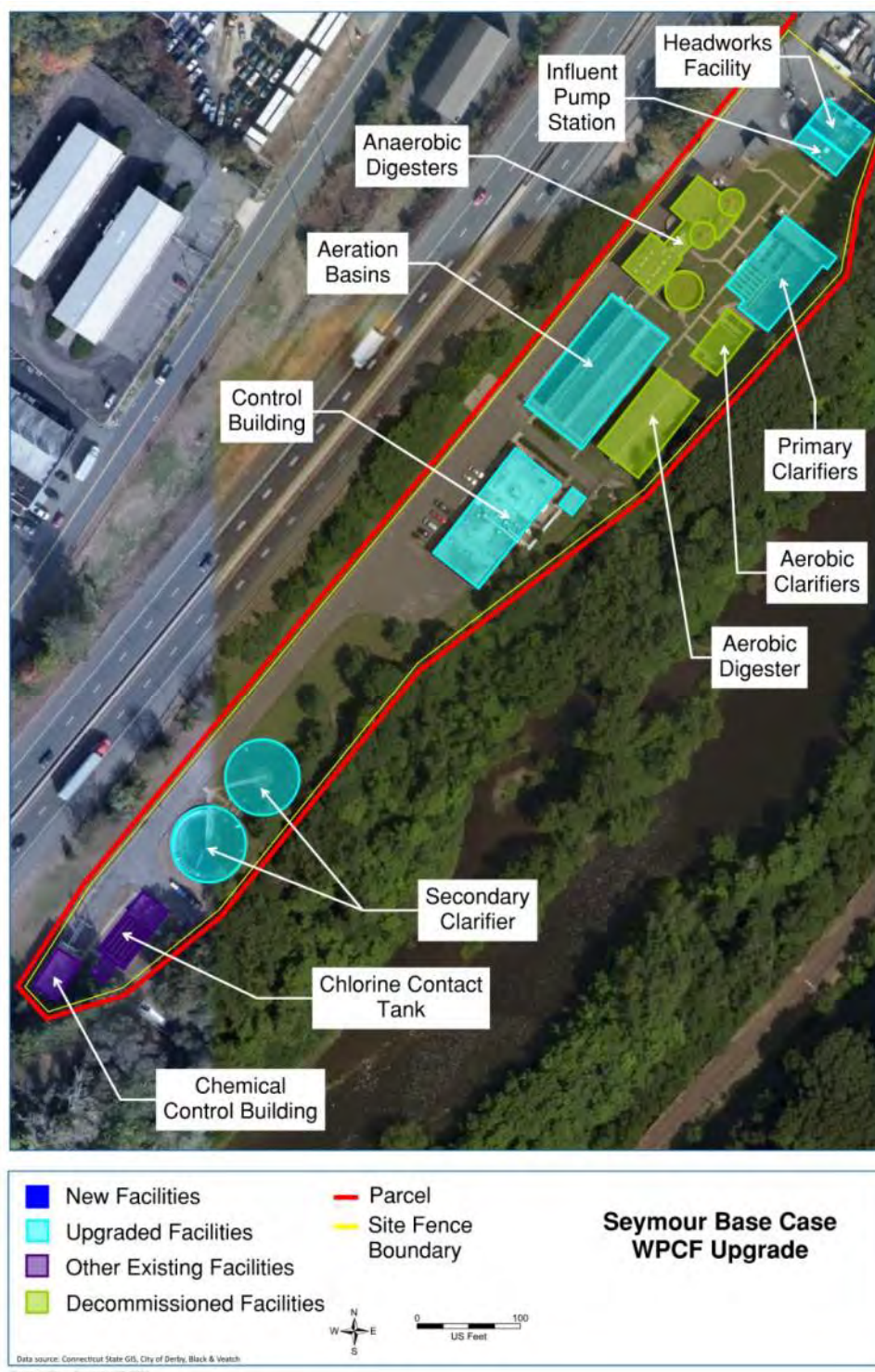


Figure 2-8 shows the conceptual site layout of the Seymour WPCF base case.

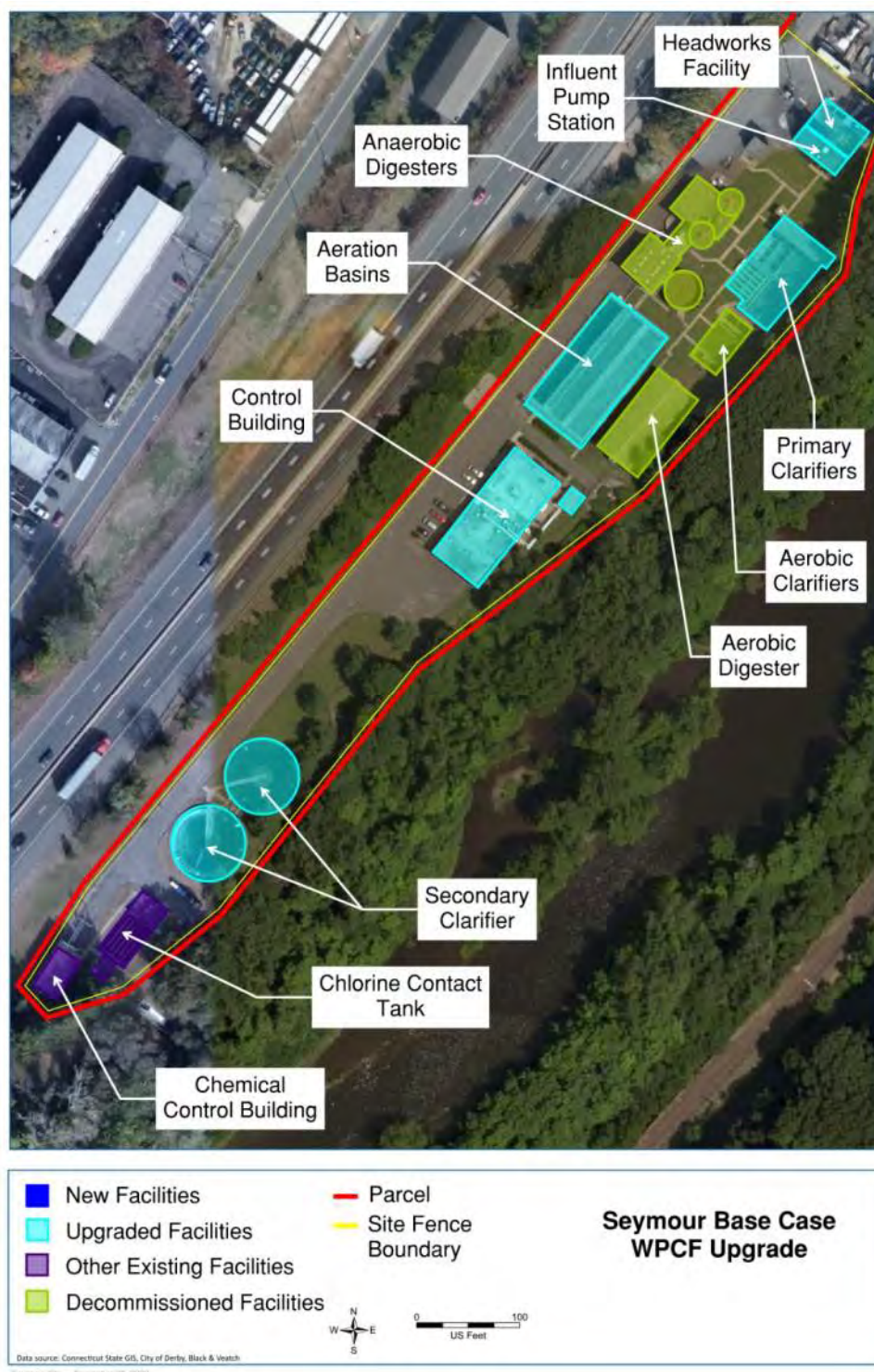


Figure 2-8 Seymour Base Case Site Layout

2.3 DERBY REGIONAL ALTERNATIVES

2.3.1 Issues

Task 2 evaluations of major unit processes for primary and secondary treatment indicated that regionalization at Derby was possible. However, due to the constrained site, both new treatment facilities and intensification technologies are required to treat the flows and loads associated with regionalization. For primary treatment, CEPT negates the need for additional primary settling tanks and reduces the loading to secondary treatment. Two intensification alternatives evaluated in Task 2 were assessed in greater detail; ballasted sedimentation and integrated fixed-film activated sludge (IFAS). These processes were evaluated in greater detail in this Task and with the load reduction associated with CEPT factored in by using Biokinetic modeling. This also allowed for estimation of planning level operational related costs. An advantage of regionalization at Derby is that discharge will be into the Housatonic. This discharge location does not have phosphorus limits associated with it and, as a result, does not require tertiary treatment which is in contrast to some of the regionalization alternatives at Ansonia.

2.3.2 Facilities

2.3.2.1 Ballasted Sedimentation Based Upgrades

Ballasted Activated Sludge enhances process capacity through the addition of high-density ballasting particles to the activated sludge process to increase the settling rate of the activated sludge flocs. The most widely adopted of this is the BioMag™ process by Evoqua in which magnetite is added and recovered through a magnetic recovery process. This enhances the settling rate and secondary sludge thickening characteristics allowing for an increased capacity both in terms of flow and loading.

Broadly, required equipment can be separated into two categories, the first being the equipment necessary for the feeding and recovery of magnetite which is generally part of the technology vendor scope of supply. Figure 2-9 shows how this equipment is incorporated into the activated sludge process. Specifically, magnetite feeding and recovery equipment includes:

- A magnetite storage silo
- A dry magnetite feeder and associated equipment
- A shear mill to separate magnetite from the floc
- A magnetic magnetite recovery drum
- A mix tank to incorporate recovered/make-up magnetite into a RAS slip-stream

The second concern with mechanical equipment is related to provisions made to prevent the settling of the ballasted mixed liquor in the process basins and channels, which is generally addressed during design. These considerations include:

- Supplemental mixing may be needed in aerated zones with efficient fine bubble diffusers (or alternately, coarse bubble diffusers can be utilized instead of fine bubble diffusers)
- Mixers in unaerated zones will require about twice the power as would be required for unaerated zones in conventional activated sludge systems
- Mixed liquor channels may need supplemental aeration or mixing to prevent settling

- Even with these provisions, more frequent cleaning of aeration basins should be accounted for by allowing for a basin to be offline at any time of operation

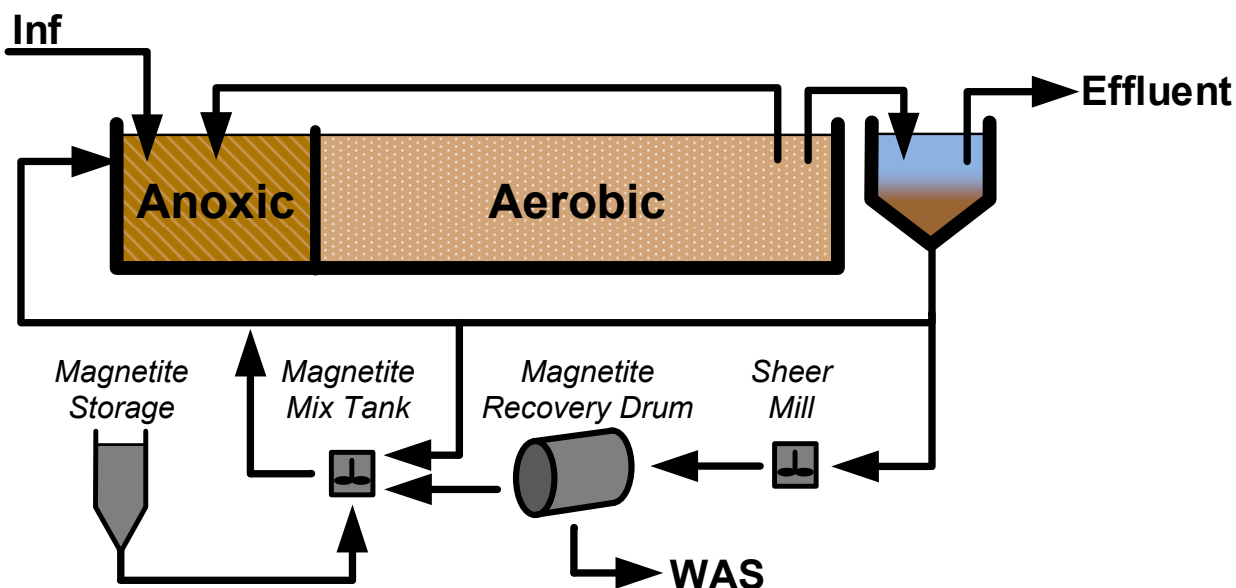


Figure 2-9 Magnetite Ballasted Sedimentation

Regionalization alternatives relying on BioMag were assessed in Task 2. These assessments have been further refined in this Task by accounting for lower loadings due to CEPT and the revised and lower Derby growth projections, which are counteracted by a higher design SRT and operational considerations.

As in Task 2, the capacity of the ballasted activated sludge process was assessed using clarifier loading rate guidelines from published design regulations. This includes that the SOR should not exceed 1,500 gpd/ft² on a Max Month basis or 2,500 gpd/ft² on a peak hour basis, and that the SLR should be limited to 75 lb/day/ft² on a Max Month basis or 100 lb/day/ft² on a Peak Hour basis (excluding ballast). Because the peak hour to peak day peaking factor is estimated to be around 1.5 to 1.7 in the regionalization alternatives, the peak day SOR and SLR were limited to 1,500 gpd/ft² and 70 lb/day/ft², respectively.

With this additional definition of system requirements, the capacity was again checked using yields resulting from biokinetic modeling. Based on these yields and the additional definition, the ballasted flocculation alternative would need one fewer aeration basin than indicated in Task 2 to meet the capacity requirements. However, due to the recommendation that the facility be able to operate with one basin offline if using BioMag, the same number of aeration basins are recommended for the regionalization alternatives. In the case of Derby treating Ansonia, two additional aeration basins and no additional secondary clarifiers are needed. In the full regionalization alternative of Derby treating Ansonia and Seymour, three additional aeration basins and one additional secondary clarifier is needed. Table 2-7 shows that the clarifier loadings meet the maximum limits for BioMag with one aeration basin offline.

Table 2-7 Recommended Capacity Parameters for Derby Regionalization Alternatives using BioMag

Facility Requirements and Capacities ⁽¹⁾	Derby + Ansonia	Derby + Ansonia + Seymour
Additional Aeration Basins ⁽²⁾	2	3
Additional Secondary Clarifiers	0	1
Design HRT, hrs	5.8	5.4
Design MLSS, mg/L	4,900	4,600
Peak Day SOR, gpd/ft ²	1,540	1,350
Peak Day SLR, ppd/ft ²	82.8	68.2
(1) Secondary clarifier capacities assume one aeration basin offline. (2) Derby has two existing aeration basins that are operable and one aeration basin that is inoperable. In the process evaluation, it was assumed that the inoperable basin would be upgraded to meet additional aeration basin needs.		

Aeration requirements and energy were also determined for the BioMag process based on planning level biokinetic modeling results for process oxygen requirements and Black & Veatch's gas transfer model. Because of the higher sludge density, BioMag process requires either supplemental mixing to aerobic zones with fine bubble diffusers or coarse bubble diffusers. To estimate airflow requirements and aeration energy requirements it was assumed that coarse bubble diffusers would be used.

2.3.2.1.1 Performance

The results of biokinetic modeling are shown in Table 2-8. The model was configured to reflect the proposed upgrades and operation. However, detailed influent characterization and model calibration were not undertaken, meaning that there is some uncertainty with regards to the nitrogen removal performance results of the model. The results however do confirm that the process has the capacity to effectively nitrify and denitrify to the required levels, though the amount of supplemental carbon needed will likely vary from the amounts projected. In this case, the average TN limits were met without the need for supplemental carbon. Based on historic N removal performance without CEPT, it is possible that this is an over prediction of N removal performance, however, as mentioned above, detailed influent characterization and model calibration were not undertaken.

Table 2-8 Biokinetic Modeling Results for Derby Regionalization Alternatives Using BioMag

	Derby + Ansonia	Derby + Ansonia + Seymour
Average NHx, mg/L-N	0.26	0.20
Max Month NHx, mg/L-N	1.00	0.58
Average NOx, mg/L-N	2.11	2.90
Max Month NOx, mg/L-N	2.64	4.39
Average TN, mg/L-N	3.76	4.48
Max Month TN, mg/L-N	5.21	6.44

2.3.2.2 Integrated Fixed-Film Activated Sludge Upgrade

Integrated Fixed-Film Activated Sludge (IFAS) is a hybrid suspended growth and fixed film technology. IFAS incorporates all the elements of conventional activated sludge but with the addition of the carrier media and retention sieves in order to increase the biomass inventory. Media is generally added to the aerobic zone though it can also be added to the anoxic zone as necessary. As with conventional activated sludge, the MLSS operates at 2,000 to 4,000 mg/L in IFAS. However, because of the fixed film biomass inventory associated with the media, the process can achieve the same inventory at a lower MLSS concentration, thereby increasing capacity, and/or achieving a higher inventory at the same MLSS concentration, thereby improving treatment. This makes IFAS a popular technology for retrofitting existing activated sludge processes which need to increase capacity and/or achieve stricter effluent nutrient limits. Figure 2-10 shows a schematic view of the IFAS process set-up. Generally, the technology vendor scope of supply includes the following:

- Plastic Biofilm Carrier Media
- Media Retention Sieves
- Diffuser Aeration System
- Scum Removal Systems

Additionally, the design must make the following considerations when designing for an IFAS system.

- Blowers must be sized for lower transfer efficiency due to the high DO operation associated with IFAS and the lower efficiency associated with coarse bubble diffusers
- The hydraulic grade line through the mainstream of the facility must account for the increased head loss through the IFAS process due to the media and media screens
- Screening at the influent must be fine enough to remove material that may blind the media retention screens

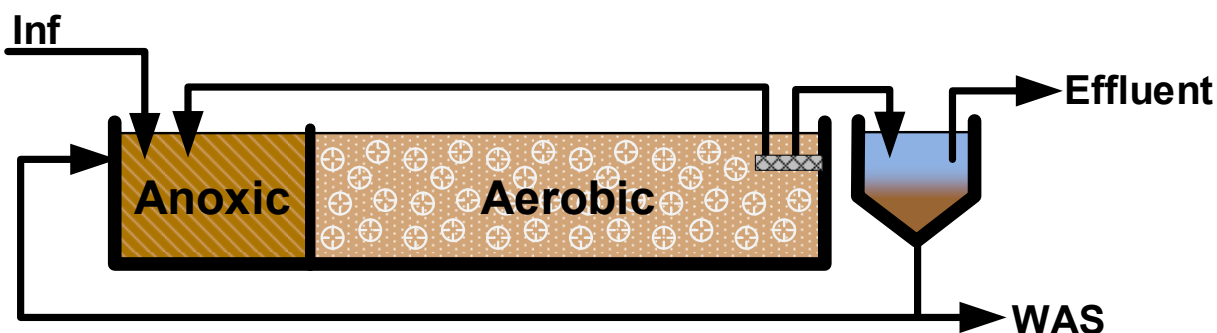


Figure 2-10 Integrated Fixed-Film Activated Sludge Process

Regionalization alternatives relying on IFAS were assessed in Task 2. These assessments have been further refined in this Task by accounting lower loadings due to CEPT and revised and lower Derby growth projections, which are counteracted by a higher design SRT and operational considerations. Performance and capacity were confirmed in this task using biokinetic models of the IFAS process with the revised secondary loadings. The resulting MLSS was used with state point analysis (SPA) to confirm clarifier capacity. One difference is that a closer examination of SVI has shown that SVI at Derby is routinely in the 150 to 200 mL/g range. There is no reason to expect that settleability will be markedly improved with the implementation of the IFAS process. The recommended number of new aeration basins, aeration basin fill fractions, number of secondary clarifiers, and clarifier loadings are summarized in Table 2-9 below.

Table 2-9 Recommended Capacity Parameters for Derby Regionalization Alternatives using IFAS

	Derby + Ansonia	Derby + Ansonia + Seymour
Additional Aeration Basins ⁽¹⁾	1	2
Additional Secondary Clarifiers	1	2
Media Fill, % ⁽²⁾	40	40
Design HRT, hrs	5.8	5.4
Design MLSS, mg/L ⁽³⁾	2,200	2,080
Peak Day SOR, gpd/ft ²	1,020	1,010
Peak Day SLR, ppd/ft ² ⁽³⁾	27.9	25.7
<p>(1) Derby has two existing aeration basins that are operable and one aeration basin that is inoperable. In the process evaluation, it was assumed that the inoperable basin would be upgraded to meet additional aeration basin needs.</p> <p>(2) Media characteristics assumed were consistent with Kaldnes K1 media and are as follows;</p> <ul style="list-style-type: none"> • Specific surface area: 500 m²/m³ • Void ratio: 84% • Maximum fill fraction: 65% <p>(3) Suspended only</p>		

Aeration requirements and energy were also determined for the IFAS process based on biokinetic modeling results for process oxygen requirements and Black & Veatch's gas transfer model. The IFAS process uses coarse bubble diffusers and so it was assumed that coarse bubble diffusers would be used when determining airflow and aeration energy requirements.

2.3.2.2.1 Performance

The results of biokinetic modeling are shown in Table 2-10. The model was configured to reflect the proposed upgrades and operation. However, detailed influent characterization and model calibration were not undertaken, meaning that there is some uncertainty with regards to the nitrogen removal performance results of the model. The results however do confirm that the process has the capacity to effectively nitrify and denitrify to the required levels, though the amount of supplemental carbon needed will likely vary from the amounts projected. In this case the average TN limits were met without the need for supplemental carbon. Based on historic N removal performance without CEPT, it is possible that this is an over prediction of N removal performance, however, as mentioned above, detailed influent characterization and model calibration were not undertaken.

Table 2-10 Biokinetic Modeling Results for Derby Regionalization Alternatives Using IFAS

	Derby + Ansonia	Derby + Ansonia + Seymour
Average NHx, mg/L-N	0.18	0.18
Max Month NHx, mg/L-N	0.62	0.73
Average NOx, mg/L-N	3.9	4.1
Max Month NOx, mg/L-N	3.5	4.8
Average TN, mg/L-N	5.8	5.9
Max Month TN, mg/L-N	5.8	7.3

2.3.2.3 New Facilities and Reconfigured Existing Facilities

Table 2-11 summarizes new and reconfigured existing facilities required for the Derby regional alternatives.

Table 2-11 New and Reconfigured Facilities for Derby Regional Alternatives

	Derby + Ansonia (BioMag)	Derby + Ansonia (IFAS)	Derby + Ansonia + Seymour (BioMag)	Derby + Ansonia + Seymour (IFAS)
Reconfigured Raw Wastewater Screening Facility and Influent Pump Station	Yes	Yes	Yes	Yes
New Grit Removal Facility	Yes	Yes	Yes	Yes
New Aeration Basin	Yes	No	Yes	Yes
Extension to Existing Aeration Basins	No	No	Yes	No
New Secondary Clarifier	No	No	Yes	Yes (2x)
New Sludge Handling Facility	Yes	Yes	Yes	Yes
New Ultraviolet Disinfection Facility	Yes	Yes	Yes	Yes

Preliminary layouts of these facilities were developed for the planning level capital costs. The new Sludge Handling Facility is shown in Figure B 1 in Appendix B. The reconfigured Raw Wastewater Screening Facility and Influent Pump Station, and new Grit Removal Facility are shown in Figure 2-2 and Figure 2-3, respectively, earlier in this chapter. The reconfigured aeration basin area with one new basin is shown in Figure 2-11; the reconfigured aeration basin area with one new basin and basin extension is shown in Figure 2-12. It is noted that the existing chlorine contact tank system currently in use at Derby will not be sufficient for the regionalization alternatives. With limited space to construct an additional chlorine contact tank, it will be necessary to replace chlorine disinfection and dechlorination with a new UV system for the regional alternatives at Derby. The new UV Disinfection Facility is shown in Figure 2-13.

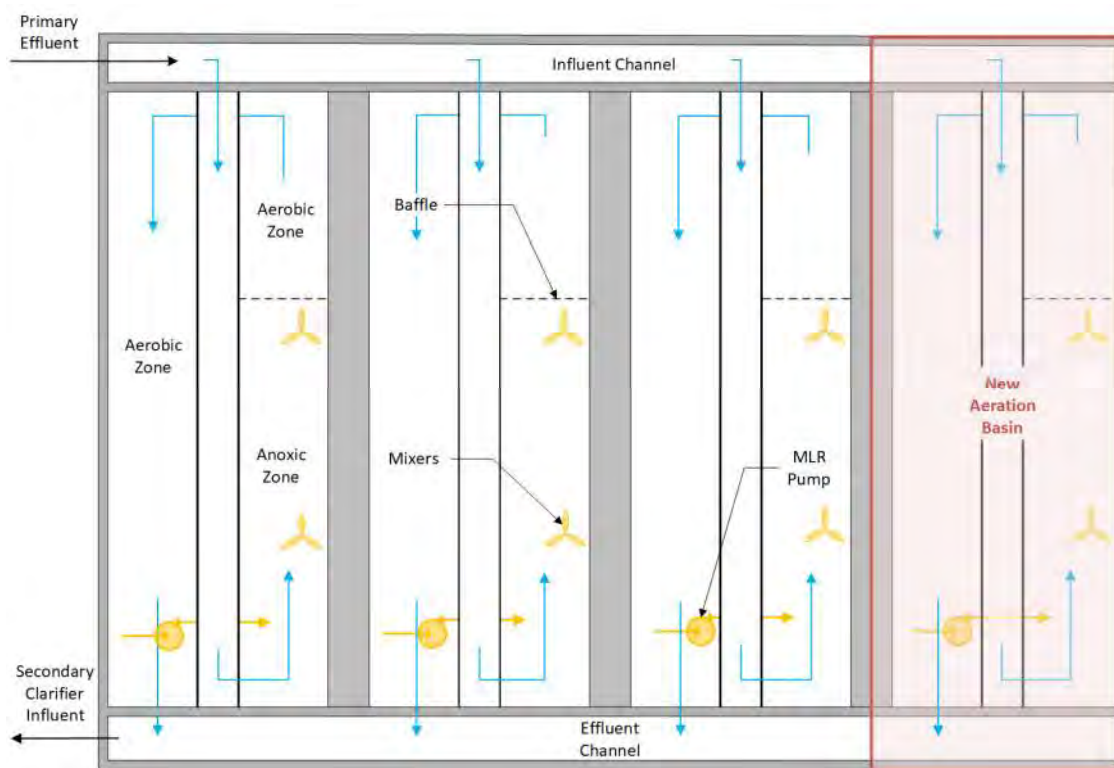


Figure 2-11 Reconfigured Derby Aeration Basins with One New Basin

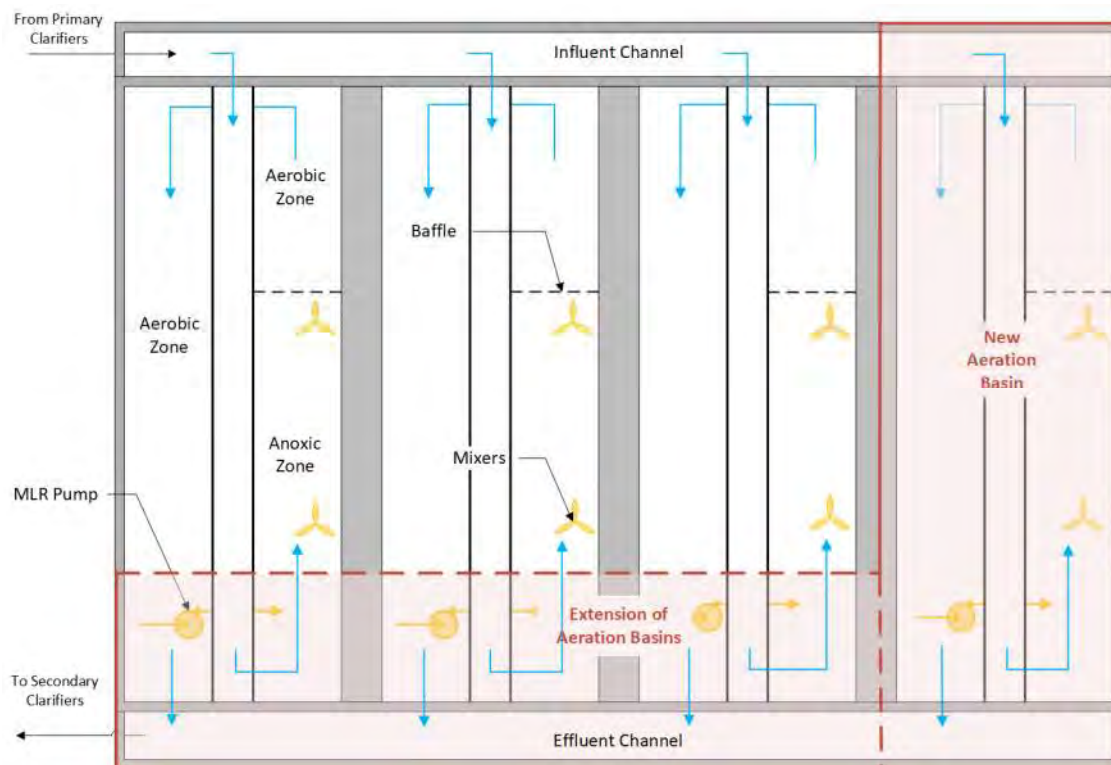


Figure 2-12 Reconfigured Derby Aeration Basins with One New Basin and Basin Extension

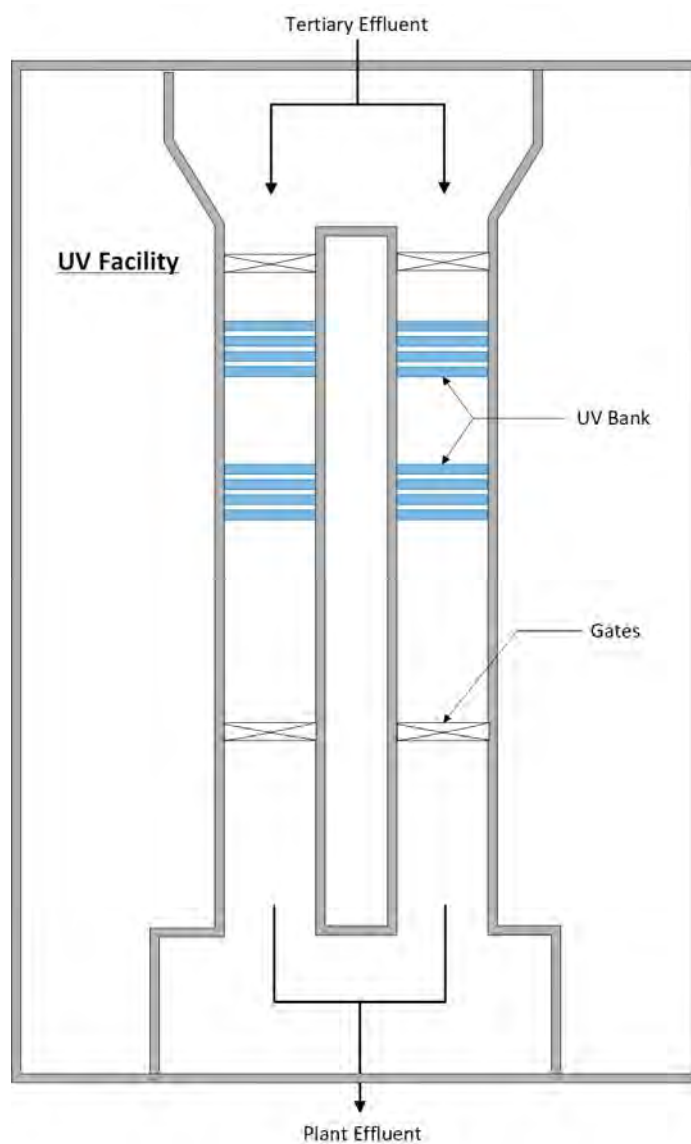


Figure 2-13 New UV Disinfection Facility

2.3.2.4 Derby Regional Alternative Layouts

Conceptual site layouts for the Derby regional alternatives are shown in Figure 2-14, Figure 2-15, Figure 2-16, and Figure 2-17.

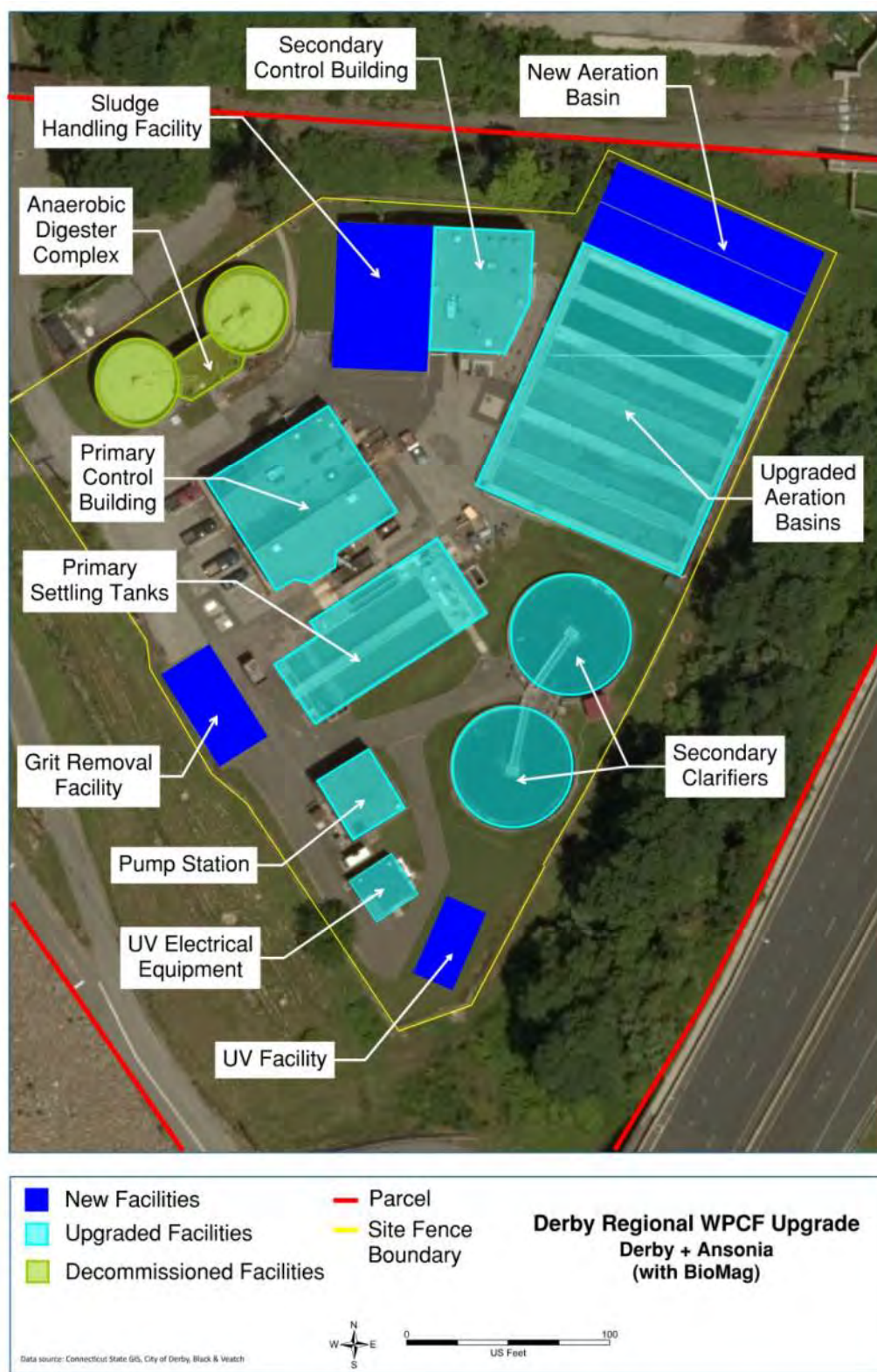


Figure 2-14 Derby Plus Ansonia with BioMag Site Layout

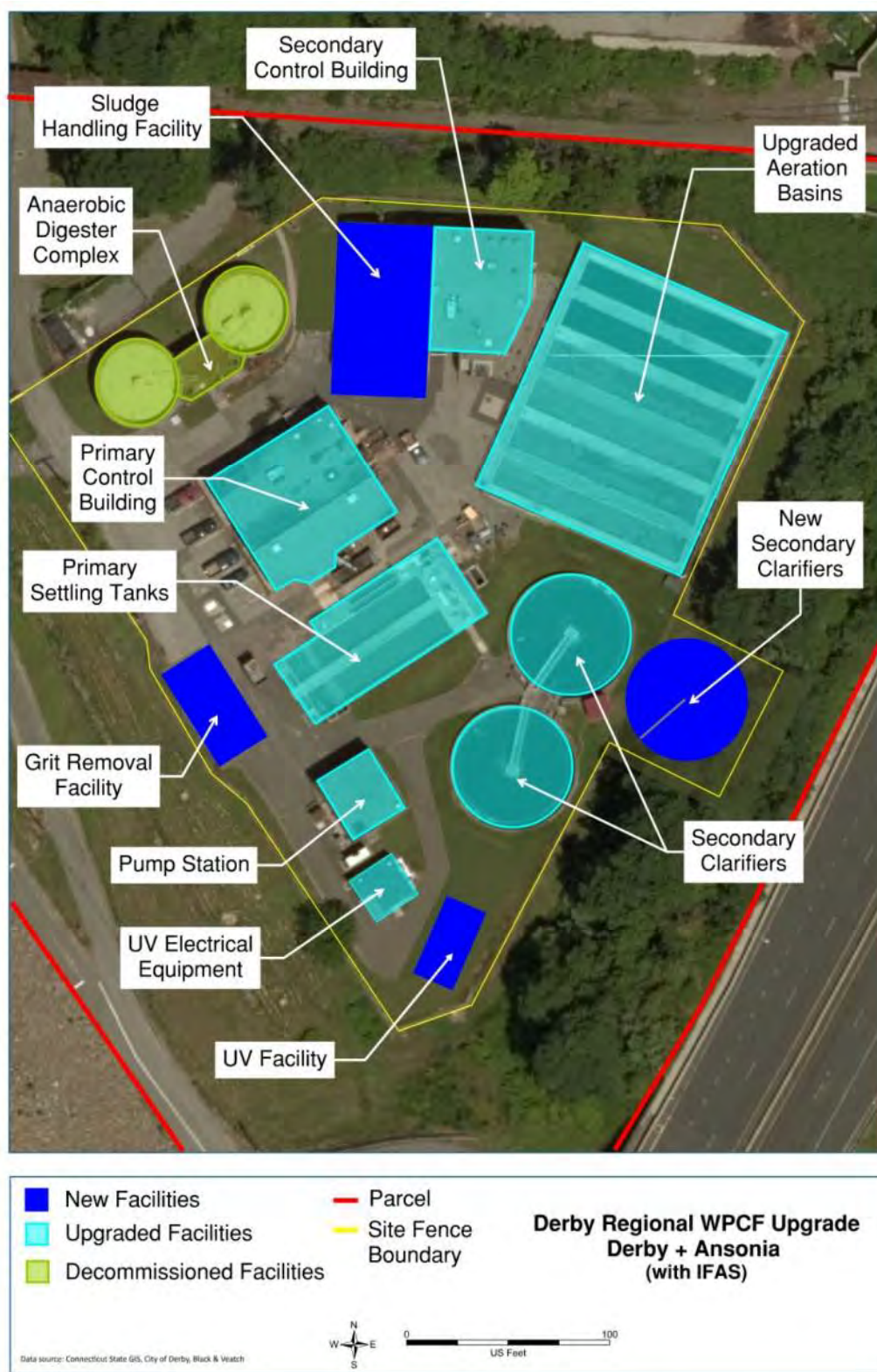


Figure 2-15 Derby Plus Ansonia with IFAS Site Layout

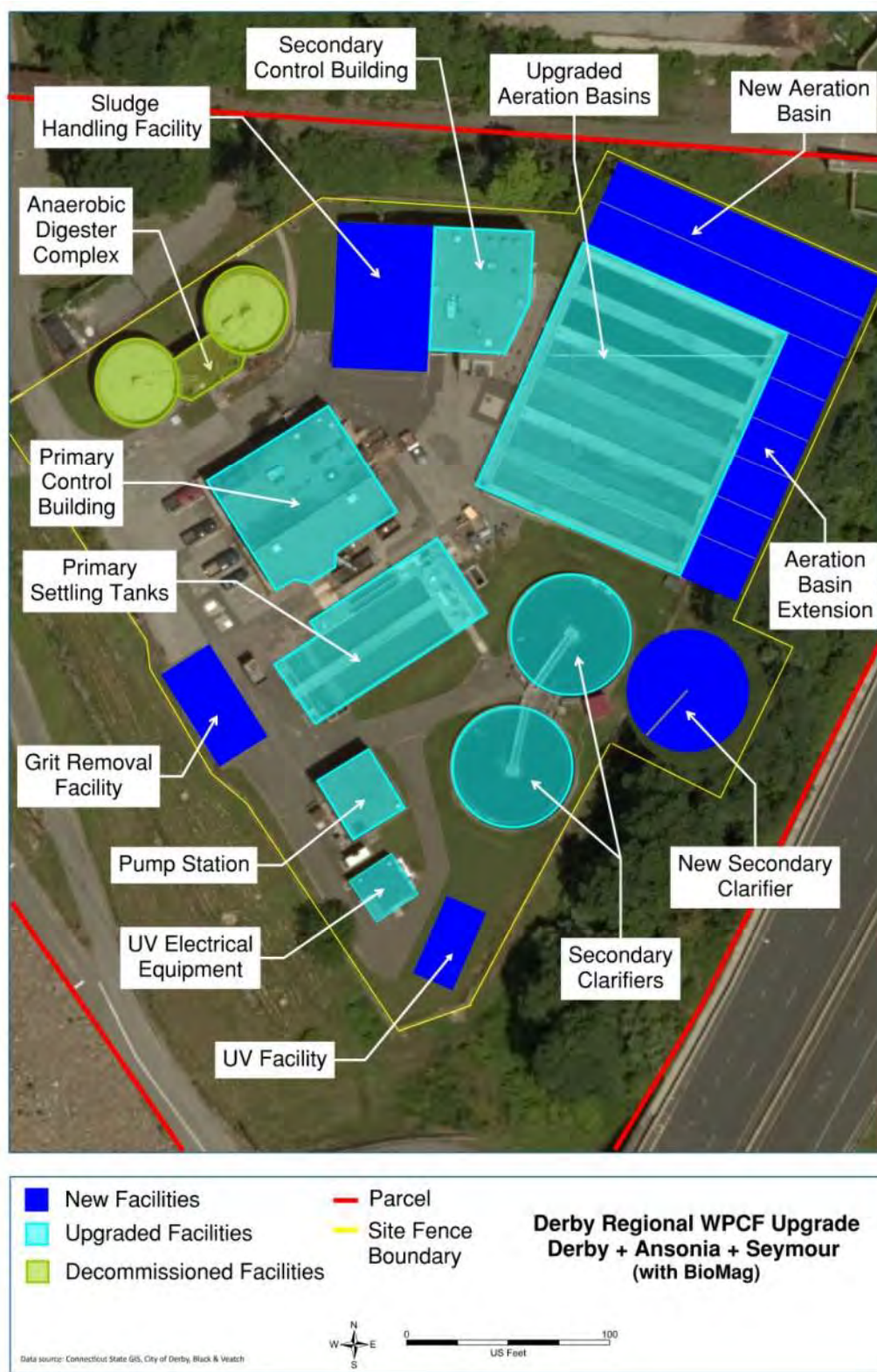


Figure 2-16 Derby Plus Ansonia and Seymour with BioMag Site Layout

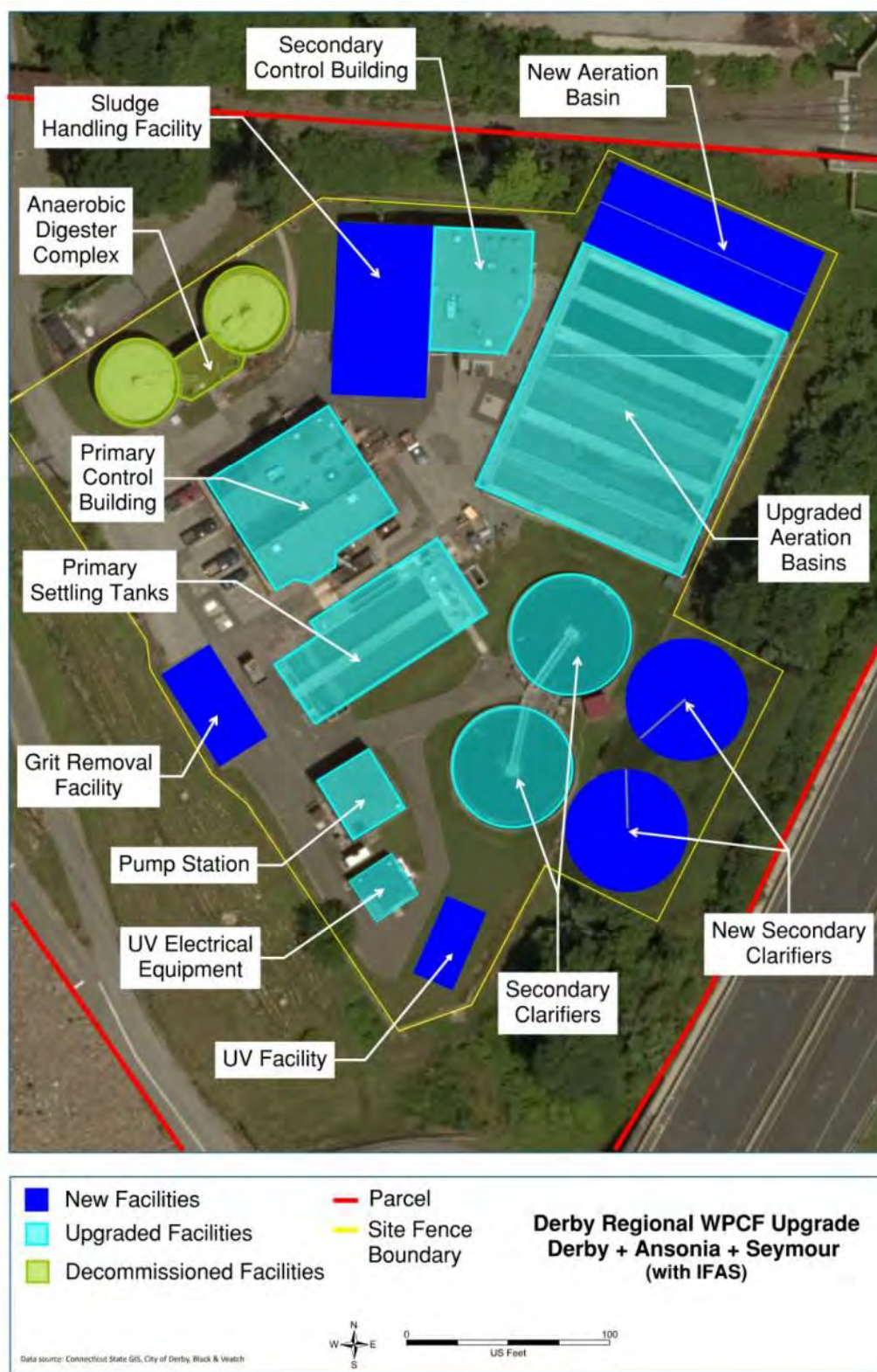


Figure 2-17 Derby Plus Ansonia and Seymour with IFAS Site Layout

2.4 ANSONIA REGIONAL ALTERNATIVES

2.4.1 Issues

Relative to Derby, the facilities at Ansonia have a higher safety factor in terms of treating the rated flows and loads. Additionally, the site is less constrained with more room to add major process units if needed. As a result, the previous task determined the regionalization alternatives were possible without using secondary process intensification, though primary settling tanks may require the ability to operate in CEPT at higher flows in some regionalization alternatives. The previous task also determined that, for regionalization alternatives in which Derby flows and loads are treated at Ansonia, tertiary treatment for phosphorus removal may be required. An alternative to this is secondary treatment and disinfection at Ansonia, with discharge at the Derby facility's outfall to the Housatonic (which currently has no phosphorus limits). Considering these issues, biokinetic modeling was used to provide additional planning level definition for the treatment upgrades required to meet capacity. This also allowed for estimation of planning level operational related costs for the regionalization alternatives at Ansonia, factoring in increased primary removals due to CEPT in the appropriate cases.

2.4.2 Facilities

Regionalization alternatives for Ansonia were assessed in Task 2. These assessments have been further refined in this Task by accounting for lower loadings due to CEPT and revised Derby growth projections which were decreased and, which are counteracted by a higher design SRT and operational considerations. Performance and capacity were confirmed in this task using biokinetic models with the revised secondary loadings. The resulting MLSS was used with SPA to confirm clarifier capacity. A closer examination of SVI has shown that while SVI at Ansonia average 130 mL/g, it routinely increases to approximately 200 mL/g in the spring. Based on these revisions to the design basis, it is still the case that no upgrades are needed if just regionalizing with Derby but that one additional secondary clarifier is necessary for a regionalization alternative involving Ansonia treating Derby and Seymour. The recommended number of new aeration basins, number of secondary clarifiers, and clarifier loadings are summarized in Table 2-12 below.

Table 2-12 Recommended Capacity Parameters for Ansonia Regionalization Alternatives

	Ansonia + Derby	Ansonia+ Derby+ Seymour
Additional Aeration Basins	0	0
Additional Secondary Clarifiers	0	1
Design HRT, hrs	12.7	9.0
Design MLSS, mg/L	2,670	2,950
Peak Day SOR, gpd/ft ²	790	690
Peak Day SLR, ppd/ft ²	25.0	25.1

The results of biokinetic modeling are shown in Table 2-13. The model was configured to reflect the proposed upgrades and operation. However, detailed influent characterization and model calibration were not undertaken, meaning that there is some uncertainty with regards to the nitrogen removal performance results of the model. In general, the model predicts lower N removal than observed historically. Again, this is likely due to the model not fully accounting for the SND occurring. In the model the average TN limits, which vary between 6.0 to 6.5 mg/L-N for the Ansonia regionalization alternatives, were met without the need for supplemental carbon which is likely to be the case at full scale.

Table 2-13 Biokinetic Modeling Results for Ansonia Regionalization Alternatives

	Ansonia + Derby	Ansonia + Derby + Seymour
Average NHx, mg/L-N	0.55	0.56
Max Month NHx, mg/L-N	0.43	0.36
Average NOx, mg/L-N	2.64	2.74
Max Month NOx, mg/L-N	4.48	6.06
Average TN, mg/L-N	4.32	4.48
Max Month TN, mg/L-N	6.36	7.89

As established in the previous task, there is some uncertainty regarding phosphorus limits in the Ansonia regionalization alternatives. If assuming that the load limits for P discharge into the Naugatuck River move with the facilities' share of the wastewater being treated, then when Seymour flows are treated and discharged at Ansonia, the effluent P concentrations for compliance are not substantially different; thus, tertiary phosphorus treatment is not needed to meet P limits. However, because Derby doesn't currently discharge into the Naugatuck River and therefore has no P load allocation to the Naugatuck River, incorporating Derby into the Ansonia regionalization alternatives substantially impacts the concentrations which must be achieved to meet P load limits in those alternatives. Table 2-14 summarizes these concentrations and shows that the alternatives with Derby being treated and discharged at Ansonia's outfall, the effluent TP would need to be less than approximately 0.5 mg/L-P for compliance. The concentration that can typically be expected to be achieved with chemical P removal in the secondary process is generally 0.5 mg/L-P. Lower values can be attained with good secondary clarifier performance (i.e. low effluent TSS) as is currently achieved at Ansonia. However, there will likely be some increase in effluent solids at higher loading rates and so at this stage of planning, it should be assumed that tertiary filtration is required to meet the P limits in the Ansonia regionalization alternatives which include Derby. At these effluent levels, the tertiary process is only needed for solids separation, i.e. chemical coagulant dosage to the tertiary process is not necessary. Given the limited footprint and process requirements, cloth media filters have been identified as an appropriate filtration technology, with the recommended filter requirements highlighted in Table 2-14.

Table 2-14 Effluent TP Loads and Filter Requirements for Ansonia Regionalization Alternatives

	Ansonia + Derby	Ansonia + Derby + Seymour
Seasonal P Load Limit, lb/day-P	11.92	19.46
2040 Annual Average Flow, mgd	3.49	4.79
Low Effluent P Required, mg/L-P	0.41	0.49
High Effluent P Required, mg/L-P ⁽¹⁾	0.44	0.52
Tertiary Treatment Required?	Likely ⁽²⁾	Possibly ⁽²⁾
Firm Cloth Filter Area Requirement, ft ²	1600	2200
Tertiary Filter Facility Requirements	2x15 Filter Discs ⁽³⁾	2x20 Filter Discs ⁽³⁾
<p>(1) Assuming April-October Average flow is 94% of annual average flow (based on historical averages).</p> <p>(2) Depends on effluent TSS achieved at higher flows. Need for tertiary filtration could be reevaluated after regionalization but before design conditions are reached.</p> <p>(3) Assuming the use of Aqua Aerobics Mega-Disc Filters.</p>		

2.4.2.1 New Facilities and Reconfigured Existing Facilities

Table 2-15 summarizes new and reconfigured existing facilities required for the Ansonia regional alternatives.

Table 2-15 New and Reconfigured Facilities for Ansonia Regional Alternatives

	Ansonia + Derby	Ansonia + Derby (Effluent to Housatonic)	Ansonia + Derby + Seymour	Ansonia + Derby + Seymour (Effluent to Housatonic)
Additional Grit Removal Unit	Yes	Yes	Yes	Yes
New Primary Clarifier	Yes	Yes	Yes	Yes
New Secondary Clarifier	No	No	Yes	Yes
New Phosphorus Removal Facility	Yes	No	Yes	No
Additional UV Channel	Yes	Yes	Yes	Yes
New Sludge Handling Facility	Yes	Yes	Yes	Yes

Preliminary layouts of these facilities were developed for the planning level capital costs. The new Sludge Handling Facility is shown in Figure B 1 in Appendix B. The new phosphorus removal facility is shown in Figure 2-18 (upper level) and Figure 2-19 (lower level).

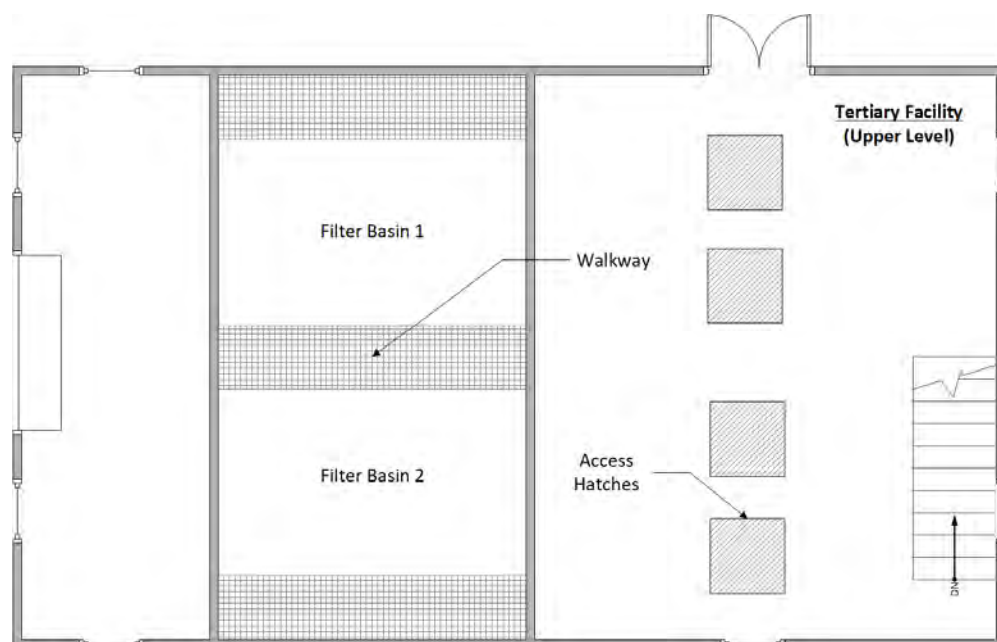


Figure 2-18 New Phosphorus Removal Facility (Upper Level)

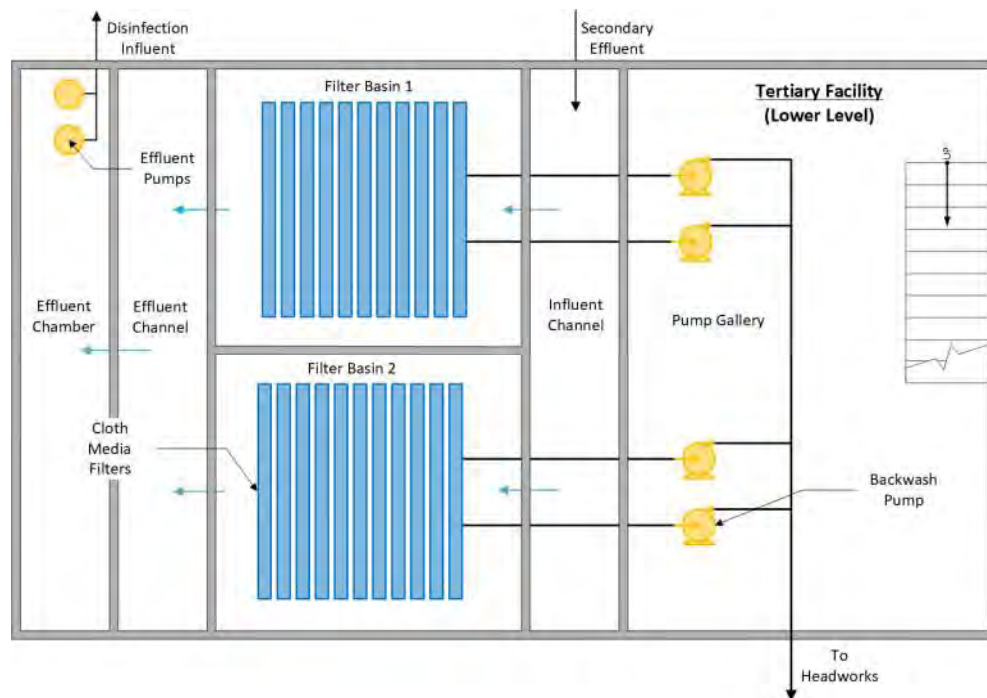


Figure 2-19 New Phosphorus Removal Facility (Lower Level)

2.4.2.2 Ansonia Regional Alternative Layouts

Conceptual site layouts for the Ansonia regional alternatives are shown in Figure 2-20, Figure 2-21, Figure 2-22, and Figure 2-23.

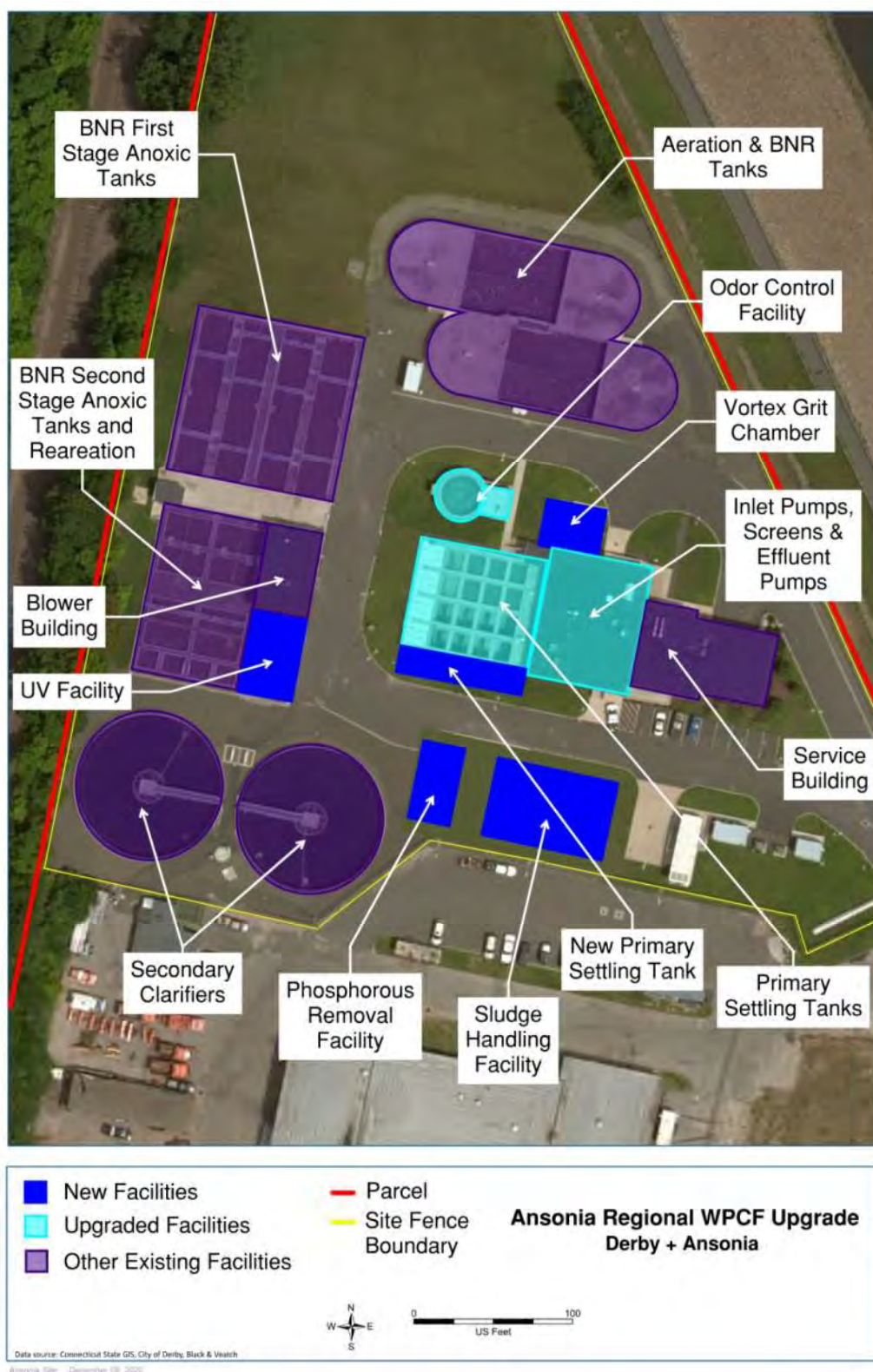


Figure 2-20 Ansonia Plus Derby Site Layout

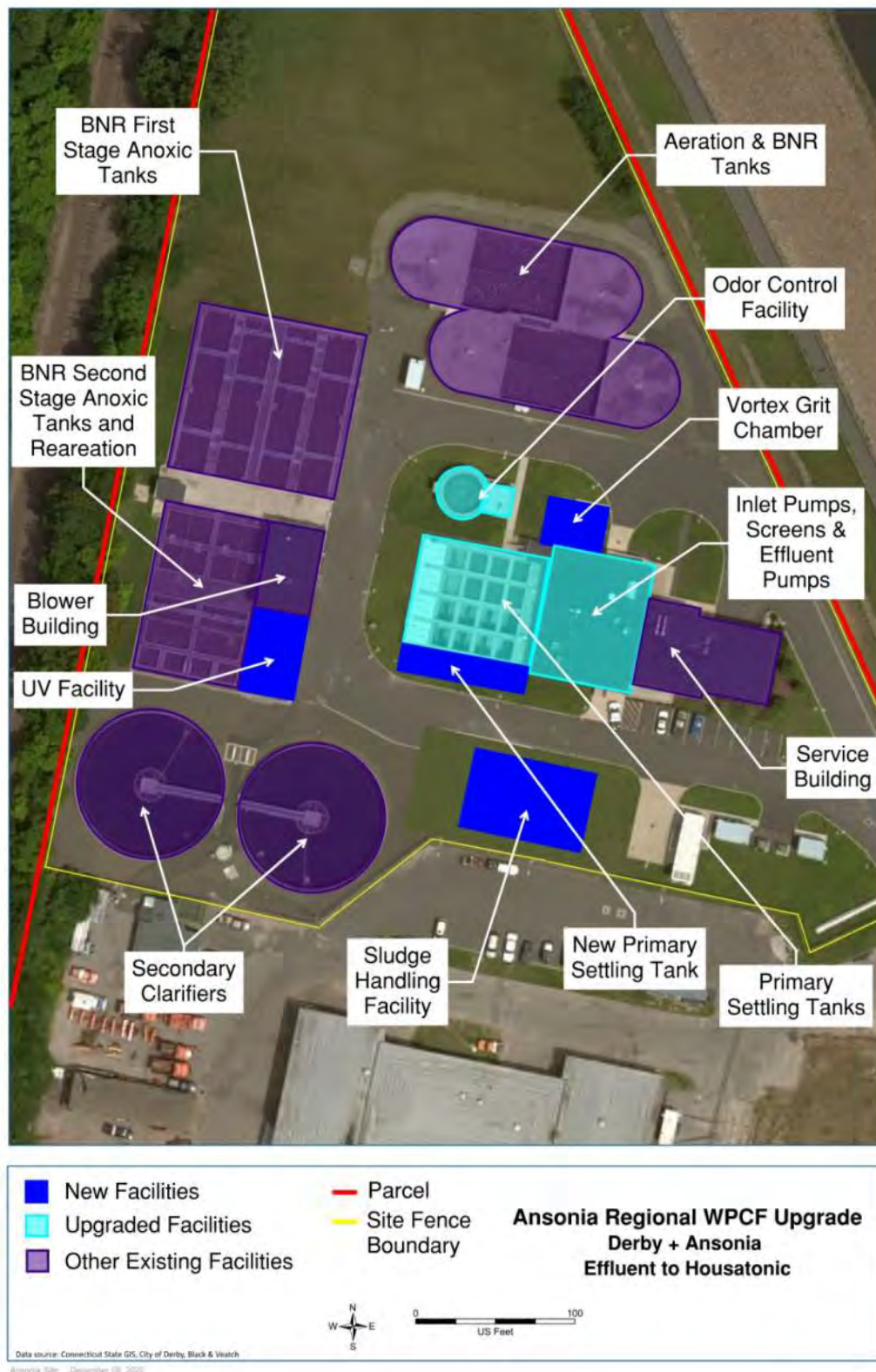


Figure 2-21 Ansonia Plus Derby with Effluent Pumped to the Housatonic Site Layout

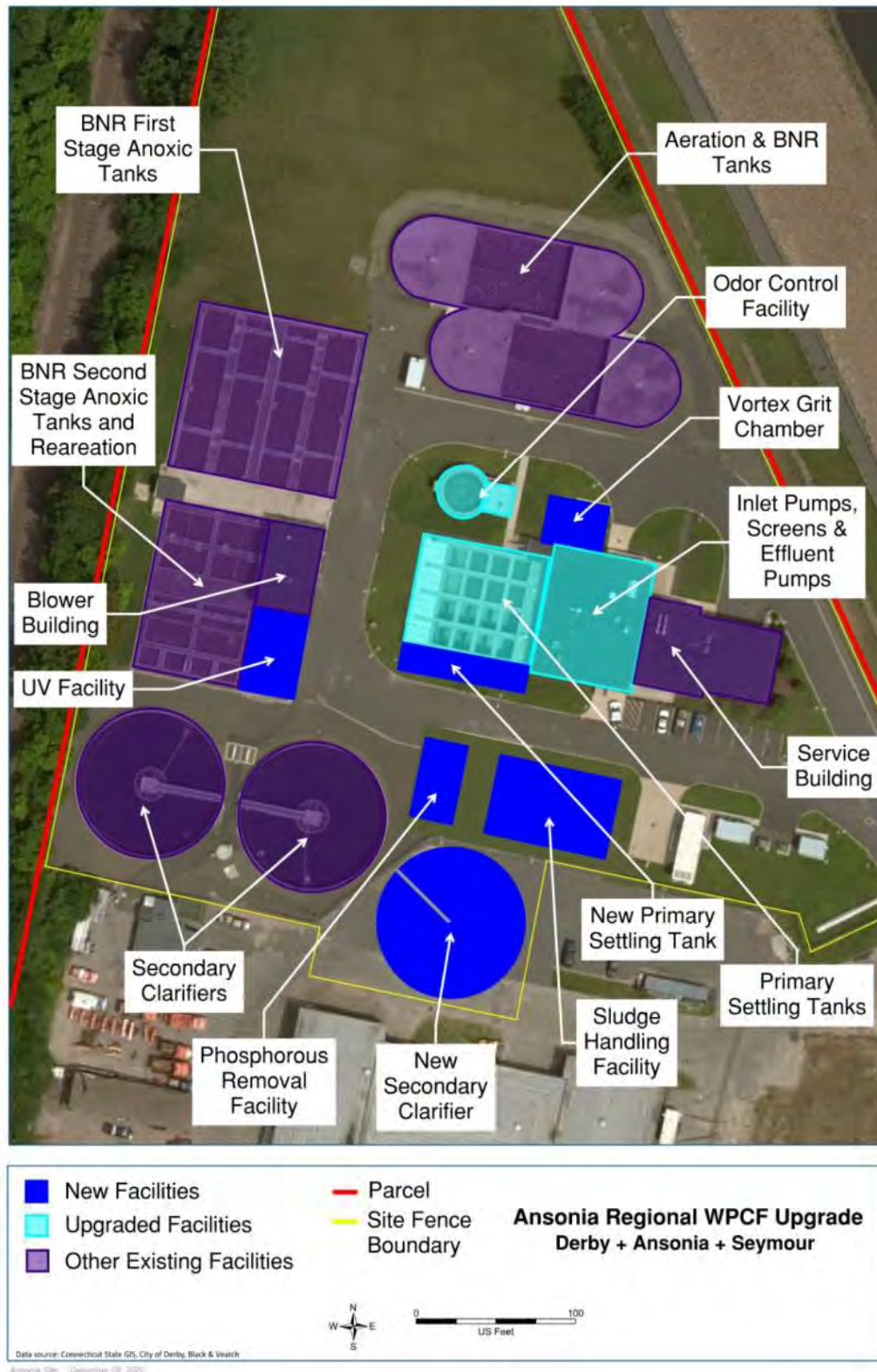


Figure 2-22 Ansonia Plus Derby and Seymour Site Layout

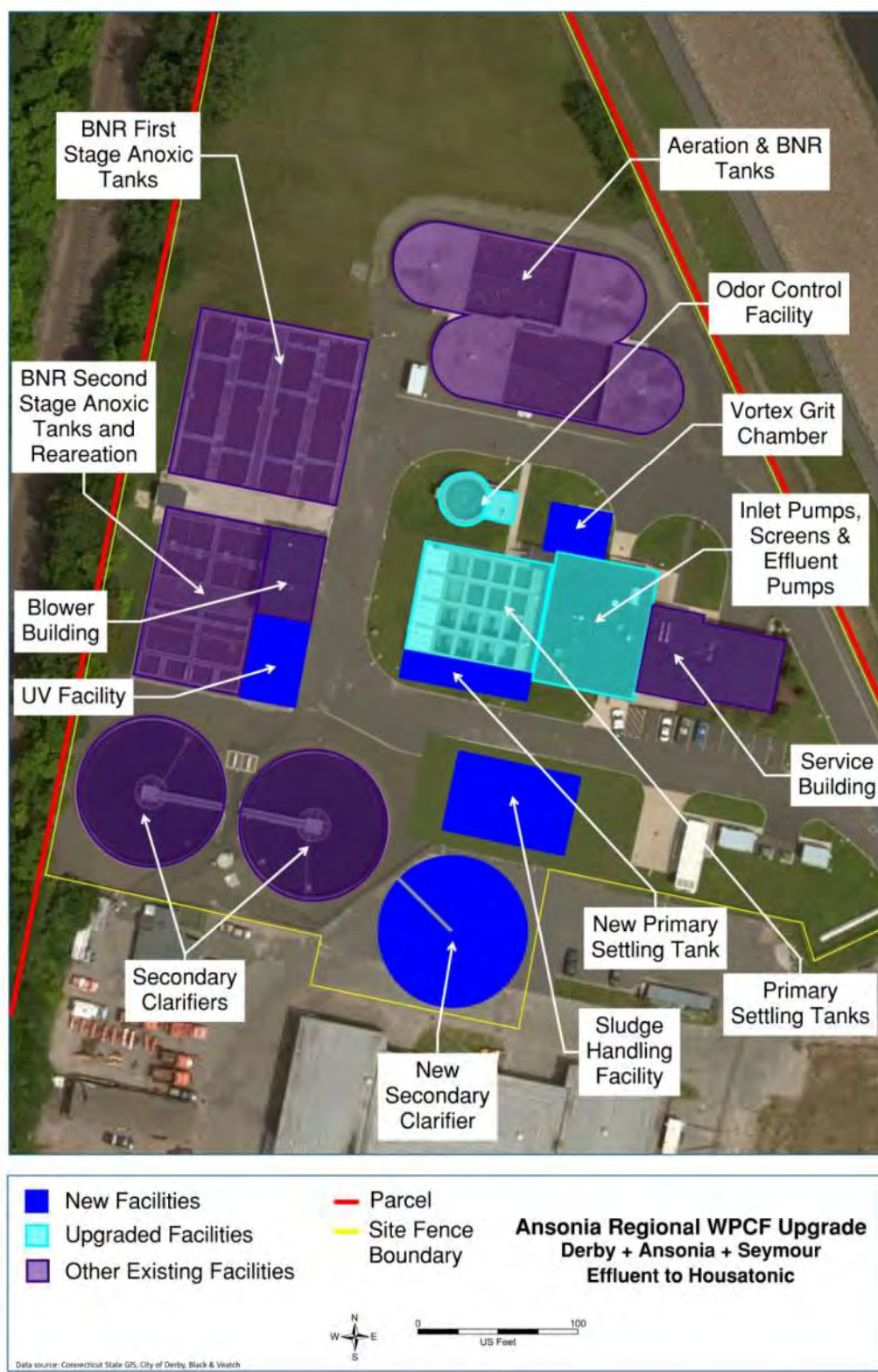


Figure 2-23 Ansonia Plus Derby and Seymour with Effluent Pumped to the Housatonic Site Layout



3.0 CONVEYANCE PIPELINES AND PUMPING

3.1 BACKGROUND

In Task 2, the wastewater conveyance corridors were developed on an initial basis and the alignments that were the least likely to be implemented (i.e. ones along river banks, railroad and/or state highway right-of-way) were removed from further consideration. The more attractive routes were carried forward into this Task where they were defined further in terms of general physical attributes, environmental concerns, easements/right of way issues, and pumping requirements. It is also noted that based on the short-listed alternatives resulting from Task 2, only the Derby to/from Ansonia and Seymour to Ansonia conveyance systems would receive focused study as part of this Task.

3.2 REGIONAL CONVEYANCE

The conveyance corridors between Derby and Ansonia and between Ansonia and Seymour were evaluated using the State of Connecticut GIS data, aerial imagery, and by on-site investigations of the streets where these corridors would be aligned. To mitigate the topographical challenges, variations to the routes were also investigated so as to minimize pumping.

A conceptual design of the pipelines based on flow rates, general topography, and associated pumping head requirements was developed; this included general characteristics of pipe diameters, pipeline lengths, and pump horsepower requirements. Requirements to screen and de-grit the raw wastewater prior to conveyance was also considered. This level of conceptual planning and design allowed for budgetary level capital costs to be identified for the conveyance pipelines associated with the short-listed regional alternatives.

3.2.1 General Considerations

3.2.1.1 Pipeline Routing

Pipeline routes were established based on environmental constraints and topography. Using Connecticut GIS, and to the extent possible, the routes were set outside flood plains, wetland buffers, and protected areas. Both conveyance corridors (the one from Seymour to Ansonia and the one that connects Derby and Ansonia) are routed along city streets and private property. At this level of planning, existing utility maps were not obtained/reviewed, but it is assumed that actual conveyance pipeline alignments can be adjusted to minimize impacts on existing buried utilities, highway structures (e.g. bridge abutments), or other existing infrastructure along the pipeline routes. Although the pipelines as depicted in this report are identified on specific streets and routing, it needs to be made clear that there are, in many reaches, two and even more alternate streets that the actual conveyance pipelines can be located in. The exact locations and property that pipelines will be routed through will be determined as part of a subsequent preliminary design where a more detailed analysis of existing utilities and land parcels would be performed.

While the conveyance corridors will not be located within the right-of-way (ROW) of the railroad or Route 8, they will cross these ROWs at different locations. ConnDOT allows transverse utility installations if they are underground, and any supporting structures are outside non-access highway lines and do not obstruct line of sight. To comply with these construction requirements, trenchless construction methods will need to be used for these segments of the routes.

Microtunneling was initially considered as a trenchless technology method where the conveyance corridor routes crossed the state highway or the railroad. Microtunneling requires, a microtunnel boring machine (MTBM) that is driven from a launch shaft to a receiving shaft. Excavated materials are carried to ground surface while lengths of pipe are added as the machine moves forward. Microtunneling is used when soil geology varies and long lengths need to be excavated. However, the lengths of these crossings are significantly shorter than for typical microtunneling projects and thus would be, cost prohibitive. Therefore, microtunneling at the crossings was dropped from further consideration and study.

Pipe jacking was also studied. In pipe jacking, casing sections are pushed using a boring machine while spoils inside the casing are simultaneously removed. Two jacking pits are required, and this method can be used in varying geological conditions for drives up to 1,000 feet. This method was determined to be a feasible and a cost-effective trenchless construction method for locations where these sections of the conveyance corridor pipelines crossed either the state highway or railroad ROWs.

3.2.1.2 Pumping Systems and Preliminary Treatment

Given the varying topography of the conveyance corridor routes, certain sections of the pipelines will flow by gravity while at others, the flow will need to be pumped. For all routes, the first leg of the conveyance pipeline will be pumped. This initial pump station would be located at the WPCF site where the wastewater flow will emanate. Existing influent pump stations would be upgraded and repurposed as wastewater conveyance pump stations. Due to the length and topographical extremes along the route from Seymour to Ansonia, an intermediate lift station will be required for that pipeline to convey flow to the Ansonia plant.

Before pumping, wastewater will be screened to minimize solids deposition and obstructions in the pipelines. Screening of the raw wastewater will also be required to better assure that pumps will not clog and serve reliably. Screening facilities at the plants will be upgraded as in the base case scenarios to remove debris prior to conveyance pumping. In addition to screening, it is ideal to remove grit prior to conveyance to minimize the potential for grit build up in the pipelines. Grit facilities require considerable space on the site to operate effectively, which is challenging when the hydraulic grade line is low at the plant influent. This is typically the case at the end of collection system. At Derby and Ansonia, implementing grit removal prior to pumping into the conveyance pipeline would be cost prohibitive and impractical for operations and maintenance given the deep hydraulic grade lines at those plants. Seymour has a shallower hydraulic grade line and grit removal upstream of the influent pumps; therefore, grit removal should be maintained ahead of raw wastewater being pumped into the conveyance pipeline.

Table 3-1 summarizes the upgrades required at current influent pump stations and screening and grit removal facilities to be converted into conveyance pump stations. These upgrades are comparable to the upgrades required for each of the plant Base Case scenarios.

Table 3-1 Required Upgrades to Convert Influent Pump Stations to Conveyance Pump Stations

	Derby	Ansonia	Seymour
Influent Pump Station	Replace pumps, piping, VFDs, and valves	Replace pumps, piping, VFDs, and valves (to meet hydraulic requirements)	Replace pumps, piping, VFDs, and valves
Screenings Facility	Reconfigure raw wastewater screening facility	Fix current mechanical screen and add a second mechanical screen	Replace mechanical screen and add second mechanical screen
Grit Removal	NA (grit removed at receiving plant)	NA (grit removed at receiving plant)	Replace equipment and upgrade grit aeration chamber
Miscellaneous	Pump station concrete repair, controls upgrade, and electrical equipment replacement	Headworks area odor control	Controls upgrade and electrical equipment replacement

An intermediate booster pump station will be required to convey flow from Seymour to Ansonia, shown schematically in Figure 3-1. This pump station would be located approximately two miles from the Seymour WPCF. Figure 3-2 identifies parcels where the pump station could be located. Exact location of the booster pump station would be determined in a later, more detailed engineering design phase if this regionalization alternative is selected.

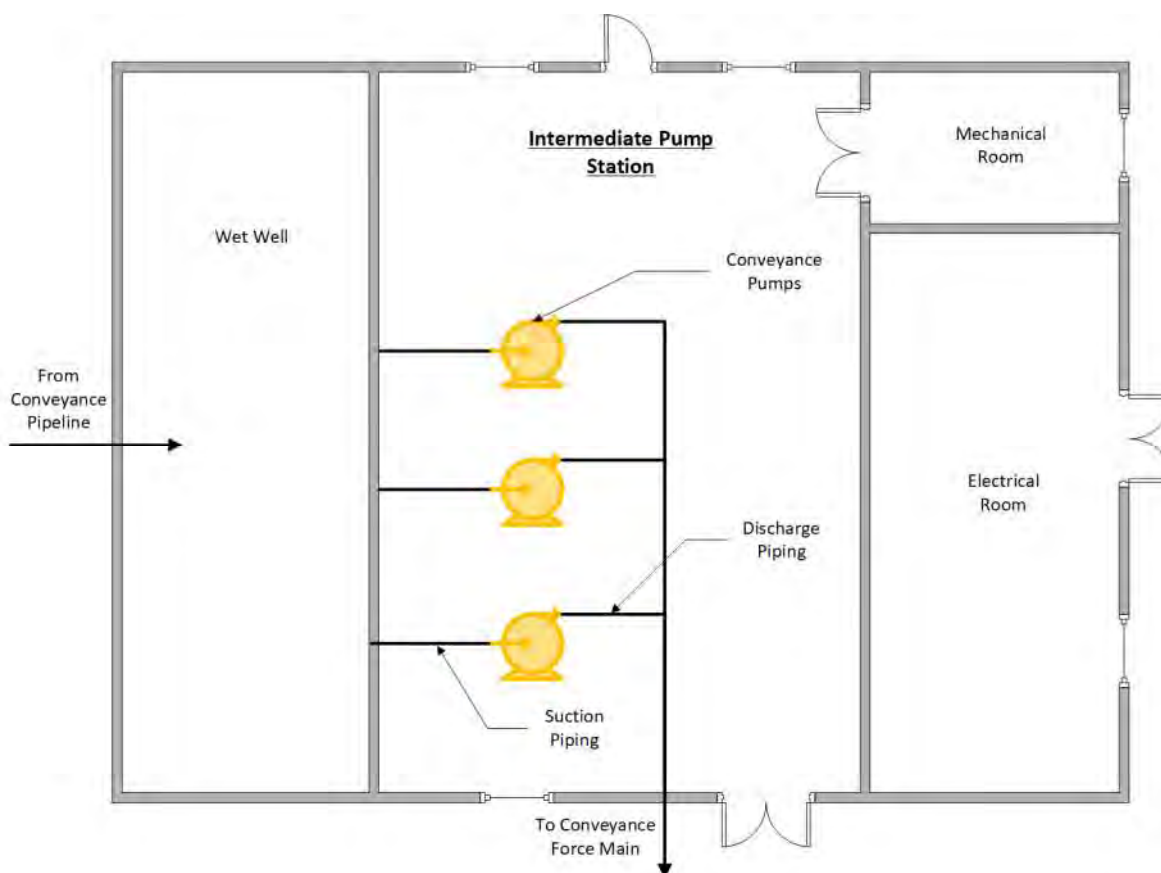


Figure 3-1 Seymour to Ansonia Intermediate Pump Station Schematic



Figure 3-2 Seymour to Ansonia Booster Pump Station Possible Site Locations

3.2.2 Derby to/from Ansonia

Figure 3-3 shows the conveyance corridor that will connect Ansonia and Derby. A route variation along North Division St was identified but at this point of the study, no decision could be made about which route is more optimal until further investigation on existing utilities and easements is undertaken as part of a subsequent preliminary design phase, if this regional alternative is selected.



Figure 3-3 Ansonia to/from Derby Conveyance Pipeline Route

The elevation difference between Derby WPCF (13 feet) and Ansonia WPCF (23 feet) is 10 feet with the highest elevation along the route at 37 feet. The elevation profiles from Derby to Ansonia and Ansonia to Derby can be seen in Figure 3-4 and Figure 3-5, respectively.



Figure 3-4 Derby to Ansonia Conveyance Pipeline Profile



Figure 3-5 Ansonia to Derby Conveyance Pipeline Profile

The existing influent pump station at the WPCF of origin will need to be upgraded as described in section 3.2.1.2 to convey wastewater. The conveyance corridor will be a combination of force main and gravity sewer trunk line. The pipeline will be routed along city streets and private property. Based on conceptual planning and design, the physical attributes of the conveyance corridor pipelines are summarized in Table 3-2.

Table 3-2 Derby to/from Ansonia Conveyance Pipeline Characteristics

	Derby to Ansonia	Ansonia to Derby	Ansonia + Seymour to Derby
Length (ft)	8,100		
Diameter (in)	14 and 20	14 and 21	16 and 24
Pump Stations	1		
High point elevation (ft)	37		
Private land taking	20% of route crosses private parcels		

Approximately 20% of the pipeline would be routed along private properties both in Derby and Ansonia. Through these properties, the preferred construction method will be open cut construction. The following parcels would be within the alignment of the conveyance corridor:

1. **205 Water St, 151 Water St, 139 Water St:** Between Water St and Route 8 ramp, the conveyance corridor will be routed for approximately 1,500 feet along the driveway of three businesses: Suburban Propane, Silktown Roofing, Inc., and A Quick Pick Crane & Rigging Services as seen in Figure 3-6.



Figure 3-6 Derby to/from Ansonia Conveyance Pipeline Property Crossings Near Derby WPCF

2. **112 Pershing Drive, 120 Pershing Drive, or 200 Pershing Drive:** Between Pershing Dr and Ansonia WPCF, the pipeline will either be routed along the driveway of an existing commercial building in 112 Pershing Drive, or through the empty vegetated lot of 120 Pershing Drive or 200 Pershing Drive as shown in Figure 3-7.



Figure 3-7 Derby to/from Ansonia Conveyance Pipeline Property Crossings Near Ansonia WPCF

3. **Waterbury Branch of the Metro North Railroad Crossing:** Leaving/going into Ansonia WPCF the pipeline will cross the railroad for approximately 75 feet as seen in Figure 3-7. This crossing will be constructed using pipe jacking with a pit at Ansonia WPCF and another pit in a private property on Pershing Avenue.

Additionally, the pipeline will cross several busy road areas that will require significant planning and traffic control. The preferred construction method through these intersections will be open-cut construction.

1. **Intersection between Factory Street/Water Street and Main Street:** This is an 85 feet long crossing through a four-way intersection in Derby. Main Street is just off Route 8 and is part of Route 34, a length of state highway that connects Newtown and New Haven. The intersection is shown in Figure 3-8.



Figure 3-8 Derby to/from Ansonia Conveyance Pipeline Intersection Crossing at Factory/Water and Main

2. **Merge ramp from Pershing Drive to Route 8:** The pipeline will be routed along a merge ramp in the southwest direction onto Route 8 for approximately 900 feet as shown in Figure 3-9.

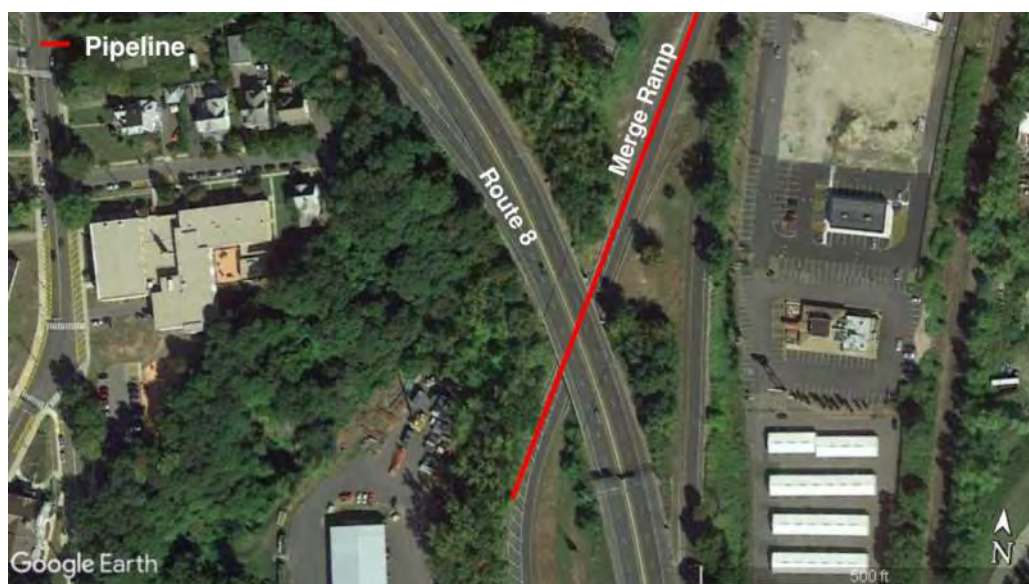


Figure 3-9 Derby to/from Ansonia Conveyance Pipeline Route 8 Crossing Along Merge Ramp

3. **Intersection between Pershing Drive and Division Street:** This is a 120 feet long four-way intersection in Ansonia. Division street is a five-lane, two-way street and Pershing Drive is a five-lane, two-way commercial and residential street that is part of Ansonia's State Route 727. The intersection is shown in Figure 3-10.



Figure 3-10 Derby to/from Ansonia Conveyance Pipeline Intersection Crossing at Pershing Dr and Division St

3.2.3 Seymour to Ansonia

Figure 3-11 shows the conveyance corridor that will connect Seymour and Ansonia. The conveyance corridor to connect these two communities, crosses Route 8 twice and is situated along a combination of town/city streets and private property.

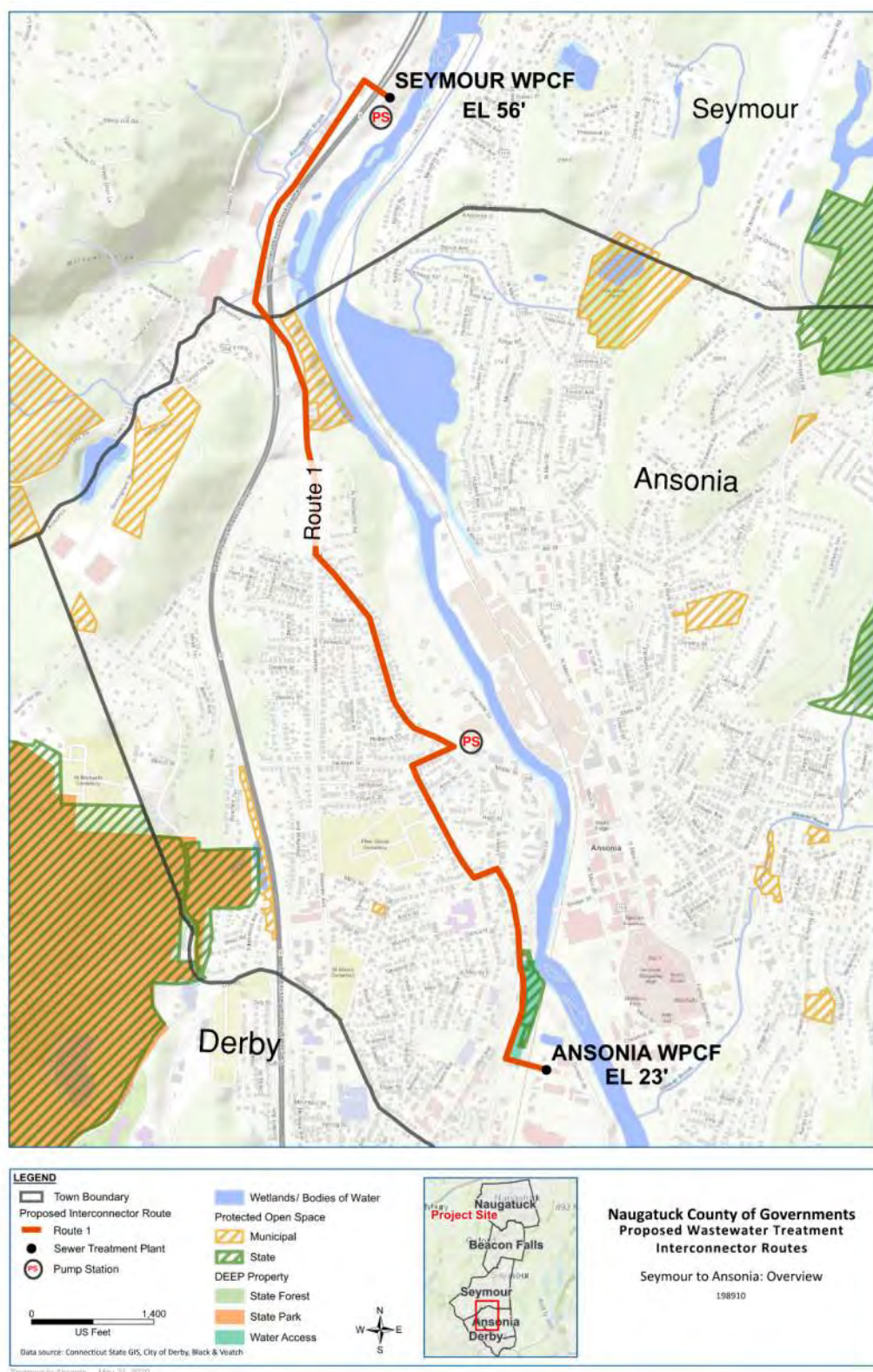


Figure 3-11 Seymour to Ansonia Conveyance Pipeline Route

Figure 3-12 shows the elevation profile of this route from Seymour WPCF to Ansonia WPCF. The elevation difference between Seymour WPCF (56 feet) and Ansonia WPCF (23 feet) is 33 feet with the highest elevation along the route at 135 feet. Route variations along different city streets were considered, with the route selected optimized based on length and topographical challenges. Due to the irregular topography, two pump stations will be required; one pump station will be located at Seymour WPCF and the other one will be a booster station along the pipeline route in Ansonia.

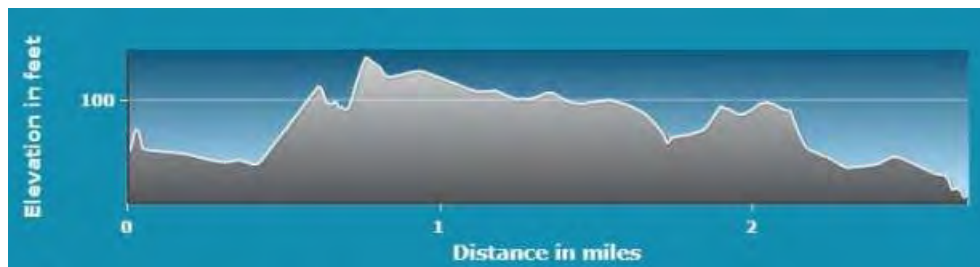


Figure 3-12 Seymour to Ansonia Conveyance Pipeline Profile

The pipeline will have different segments of force main and gravity sewer. Based on conceptual design, the physical attributes of the conveyance pipeline are summarized in Table 3-3.

Table 3-3 Seymour to Ansonia Conveyance Pipeline Characteristics

Seymour to Ansonia	
Length (ft)	14,200
Diameter (in)	14 and 20
Pump Stations	2
High point elevation (ft)	135
Private land taking	10% of route crosses private parcels

This conveyance corridor will cross Route 8 twice. At these locations pipe jacking will be required. The following segments of the wastewater conveyance pipeline will cross Route 8:

1. **From Seymour WPCF to Derby Avenue:** Figure 3-13 shows the approximately 200 feet long crossing from Seymour WPCF to Derby Avenue. One pit will be located at Seymour WPCF and the second pit at a private parcel on Derby Avenue.

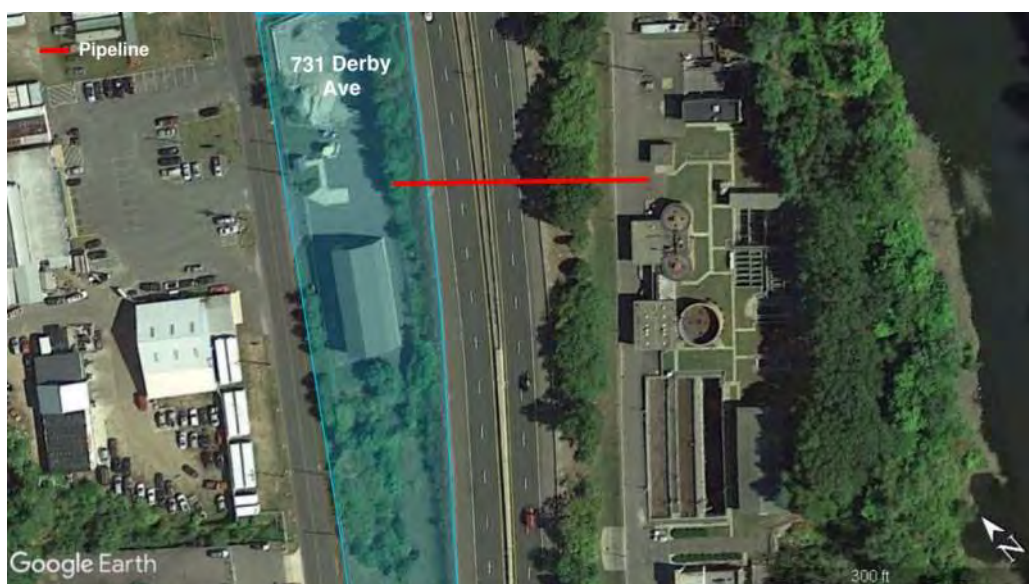


Figure 3-13 Seymour to Ansonia Conveyance Pipeline Route 8 Crossing from WWTP

2. **From Derby Avenue to Wakelee Avenue:** Figure 3-14 shows the first crossing from Derby Avenue to Wakelee Avenue. Because of the length and topographical changes, this crossing will have to be completed in two drives of approximately 600 feet each, requiring a launch pit, an intermediate pit, and a retrieval pit. The first drive will be from Derby Avenue to the median between Wakelee Avenue, Route 8, and merge ramp. The second drive will be from the median to a private parcel on Wakelee Avenue.



Figure 3-14 Seymour to Ansonia Conveyance Pipeline Route 8 Crossing near Wakelee Ave

Approximately 10% of the pipeline would be routed along private properties in Seymour and Ansonia. Through these properties, the preferred construction method will be open cut construction. The following parcels will be within the alignment of the conveyance corridor:

1. **731 Derby Avenue:** After crossing Route 8, the pipeline will cross 731 Derby Avenue, a property owned by the State of Connecticut shown in Figure 3-15.



Figure 3-15 Seymour to Ansonia Conveyance Pipeline 731 Derby Ave Crossing

2. **112 Pershing Drive, 120 Pershing Drive, or 200 Pershing Drive:** Between Pershing Dr and Ansonia WPCF, the pipeline will either be routed along the driveway of an existing commercial building in 112 Pershing Drive, or through the empty vegetated lot of 120 Pershing Drive or 200 Pershing Drive as shown in Figure 3-7 earlier in this chapter describing the Derby to/from Ansonia pipeline route.
3. **Waterbury Branch of the Metro North Railroad Crossing:** Leaving/going into Ansonia WPCF the pipeline will cross the railroad for approximately 75 feet as seen in Figure 3-7 earlier in this chapter describing the Derby to/from Ansonia pipeline route. This crossing will be constructed using pipe jacking with a pit at Ansonia WPCF and another pit in a private property on Pershing Avenue.

Most of the pipeline will be routed along busy city streets in Ansonia that will require significant planning and traffic control. The following intersections will need to be evaluated:

1. **Wakelee Avenue and Franklin Street:** Figure 3-16 shows the approximate 100 feet long three-way intersection in Ansonia. Wakelee Avenue and Westwood Road are both two-way residential streets. Franklin Street is a two-way residential street that is part of Route 334 in Connecticut, a state highway that runs from Seymour to Ansonia.



Figure 3-16 Seymour to Ansonia Conveyance Pipeline Intersection Crossing at Wakelee Ave and Franklin St

2. **Franklin Street/Maple Street and Jackson Street:** Figure 3-17 shows the approximate 100 feet long three-way intersection in the City of Ansonia. Franklin St (northeast) and Maple St are two-way streets both part of Route 334; Maple St and Franklin St (southwest) are both two-way residential streets.



Figure 3-17 Seymour to Ansonia Conveyance Pipeline Intersection Crossing at Franklin and Jackson St

3. **Lester Street/Pershing Drive and Olson Drive/Crescent Street and Bridge Street:** Figure 3-18 shows the approximate 300 feet long intersection in Ansonia. Lester Street and Olson Drive are two-way residential streets; Crescent Street is a one-way residential street; Pershing Drive is a two-way, five-lane commercial and residential street that is part of Ansonia's State Route 727; Bridge Street is a two-way street and is one of the main bridges connecting East and West Ansonia over the Naugatuck river.



Figure 3-18 Seymour to Ansonia Conveyance Pipeline Intersection Crossing at Pershing Dr and Bridge St

3.2.4 Effluent Discharge to Housatonic

Regionalization at Ansonia will require a new phosphorus removal facility to meet phosphorus discharge limits in the Naugatuck River. Regional alternatives 4 and 5b consider conveying and discharging fully treated secondary effluent from the regional plant in Ansonia to the Housatonic River at the Derby plant's existing outfall. The corridor for the effluent conveyance pipeline would be virtually the same as the regional conveyance pipeline from Derby to Ansonia for those alternatives, with two pipes installed in parallel, one from Derby conveying raw wastewater to Ansonia for treatment and the other from Ansonia conveying fully treated secondary effluent back to Derby. The physical attributes of the effluent discharge pipeline will be comparable to what is identified in Table 3-2. In these alternatives, the Ansonia effluent pump station would be modified to become a conveyance pump station; this only adds nominal costs as the effluent pumps at Ansonia would need to be upgraded in any case.

It is likely that the pipes will have to be installed at differing elevations to avoid interference with existing utilities. Moreover, installing two parallel pipes will result in longer construction times, wider easements, and increased traffic control requirements.

4.0 COLLECTION SYSTEMS

4.1 INFLOW AND INFILTRATION REDUCTION IN COLLECTION SYSTEMS

Infiltration and Inflow (I/I) is extraneous, undesired flow in the sewer system. It is typically relatively clean groundwater or storm water runoff that enters the collection system, potentially overwhelming pipe, pump, or treatment capacity, as well as increasing treatment and pumping costs. Defects resulting from aging, structural failure, lack of proper maintenance, and poor construction and design practices in sanitary sewer systems are the most common source of I/I. Defects can include conditions such as broken pipes; leaking joints; manhole lids with holes and/or poor sealing; and root infested sewer laterals. These conditions can compromise the structural integrity and contribute to excessive I/I during and after precipitation events, which can then lead to sewer surcharging and system overflows.

Each one of the five community plants included in this study will need improvements regardless of changes in flows and wastewater characteristics associated with regionalization. It is recommended that community-wide I/I programs be undertaken in all five of the communities, realizing that some of these are already underway. The results of these programs need to be regularly monitored. This will allow the communities to reevaluate the need and degree to implement aggressive I/I mitigation measures.

4.1.1 I/I Program Development

I/I programs are a standard part of wastewater management and are cost-effective at managing flows to the wastewater treatment plant over time. Implementation of an I/I program typically takes place in phases and over time – it is not uncommon that 10 or more years is required to fully implement community-wide I/I program, and I/I removal activities then continue indefinitely. I/I control results can be elusive due to the wide range of potential sources and environmental conditions, as well as the variety of control measures that can be implemented. Therefore, a strong commitment by the municipality to stay with the program is required. This is particularly the case as guideline assumptions of I/I removal may be optimistic, depending on the circumstances, and additional control may be required. This may occur for a variety of reasons, such as:

- Monitoring and SSES activities may not have identified all sources of inflow and infiltration, e.g. due to drier-than-normal conditions.
- Construction methods may not adequately seal the pipes, manholes, and related structures in the collection system to prevent I/I, or leaks that were sealed as part of the program may migrate to other cracks that were not producing leaks initially;
- Private I/I sources can be difficult to identify and control, and they may contribute a greater proportion of I/I than original estimates.

For these reasons and more, post-rehabilitation monitoring is important. The results characterize the effectiveness of I/I removal efforts and provide a basis for projecting future results. The results of post monitoring may also re-prioritize the capital plan and/or require additional testing prior to more implementation.

4.1.2 Flow Monitoring

The first step to controlling I/I is understanding the magnitude (how much flow), extent (where is it coming from), and nature (rapid inflow vs. gradual infiltration) of the problem. There are many different potential sources and patterns of I/I, ranging from discrete, identifiable sources to diffuse infiltration system-wide. The more widespread the problem is, the more expensive it will be to address. Flow monitoring is typically the first step in I/I management because it is cost-effective at characterizing each of the factors identified above. Having flow monitoring data from high groundwater periods and during storm events provides the ability to develop a strategy for successful I/I control.

Flow monitors were installed in eight locations in Ansonia, Derby, and Seymour from April 15 to May 28, 2020 as part of this regionalization study. Overall during this monitoring, flows were fairly typical for late-spring flows, with average flows moderately high compared with other times of the year. However, flow rates declined steadily throughout the monitoring period, indicating the drawdown of the water table throughout the study area. A total of 3.23 inches of rainfall was recorded during the monitoring period, using a rain gauge at the Ansonia treatment plant. Most of the rain fell as isolated, small, low intensity events with no significant change in flow. The two largest events were recorded on April 21 (0.25 inches) and April 30-May 1 (1.0 inches). These storm events are presented in more detail in subsequent discussion. Although ideal storm events were not recorded, given declining flows and lack of storm events, monitors were pulled on May 28, 2020 because significant storm responses were not expected.

Following are brief findings for each of the three monitored communities.

4.1.2.1 Derby Flow Monitoring

Flow rates from the monitoring period were compared with historical monthly operating report (MOR) flow data dating to 2015, as shown in Figure 4-1. The MOR data showed that flows in the monitoring period, while representing a high for 2020, were also substantially lower than prior years, like Seymour. Average flow during the monitoring period declined fairly rapidly over the course of the monitoring period.

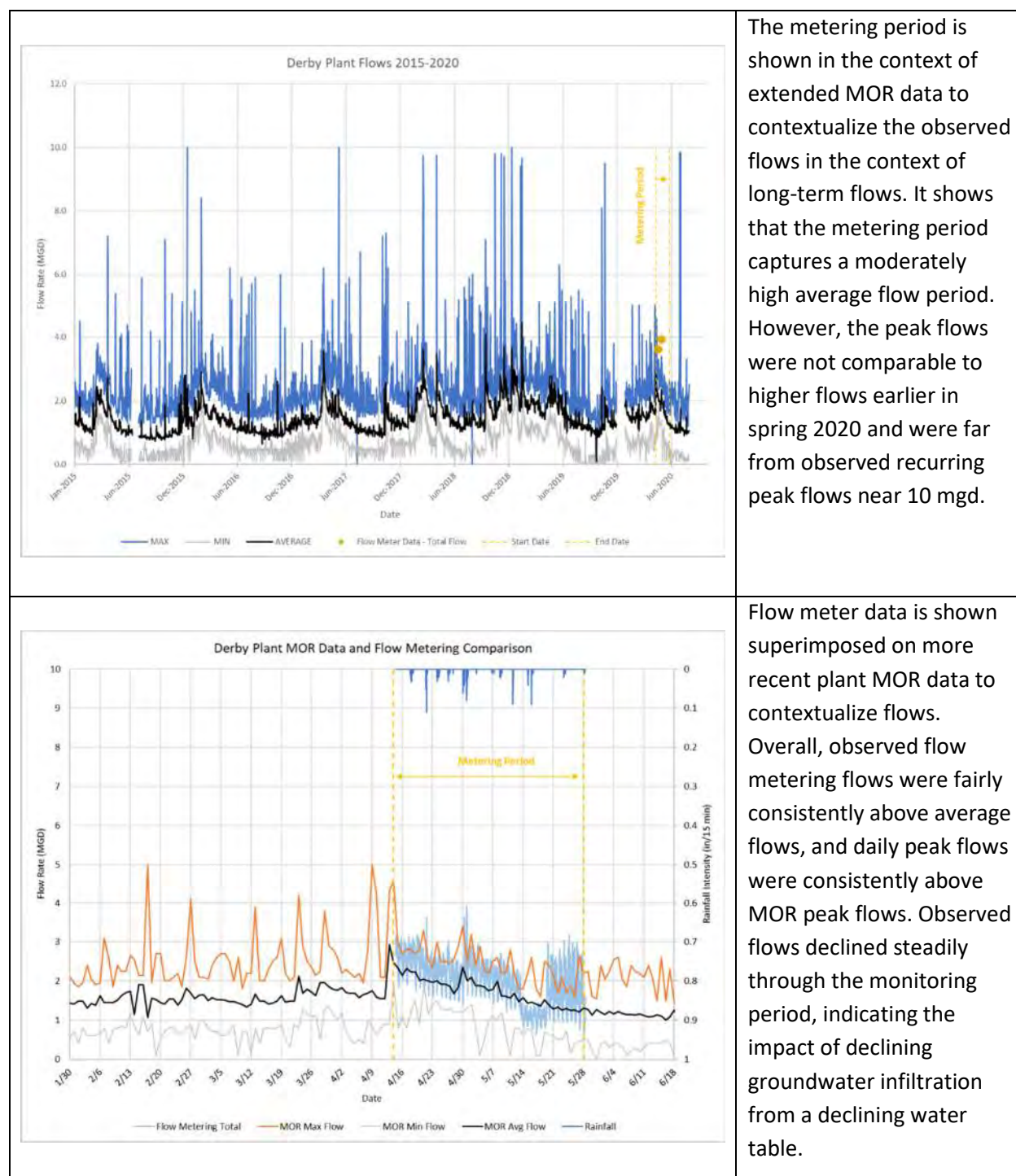


Figure 4-1 Derby Plant MOR Data and Flow Metering Data Comparison

Flow responses during the two storm events are presented in Figure 4-2, which shows that although there was an observable rainfall response, the response during larger storm events can be much larger than observed, as indicated by the frequency of events between 9 and 10 mgd since 2015. Further flow monitoring is recommended during wetter conditions before arriving at any conclusions regarding the low level of I/I observed in the system. Initially, it was

surmised that recent I/I removal activities had reduced the peak flows, but on July 3, a high peak flow was recorded again, indicating that there is still significant I/I.

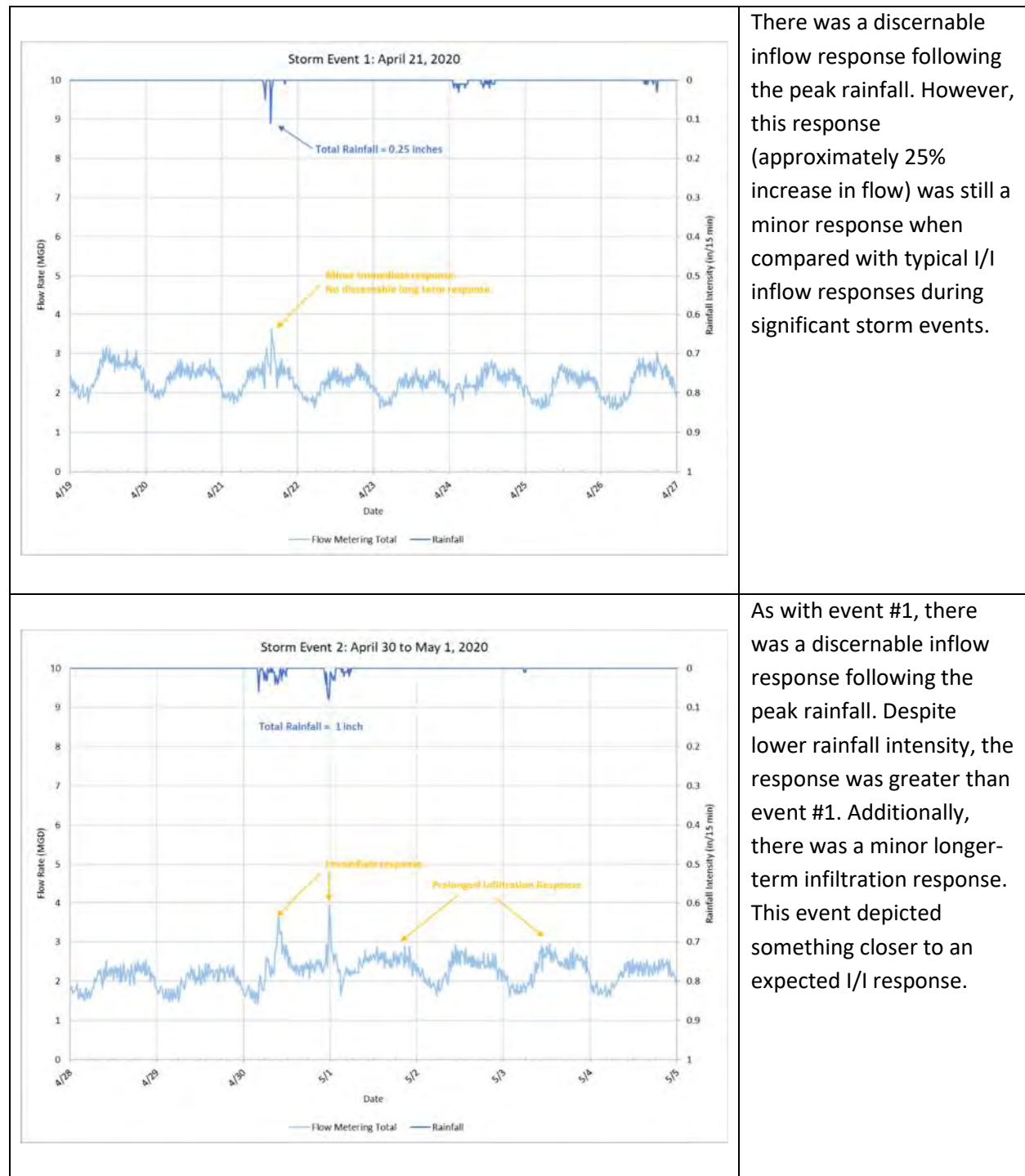


Figure 4-2 Derby Flow Metering Storm Event Observations

4.1.2.2 Ansonia Flow Monitoring

Figure 4-3 and Figure 4-4 present the MOR and flow meter comparisons and the storm event graphs, respectively. Like Derby, the average flow declined significantly during the monitoring period. Like Seymour, little to no storm response was observed in the monitoring period. Further flow monitoring is recommended during wetter conditions before arriving at any conclusions regarding the low level of I/I observed in the system.

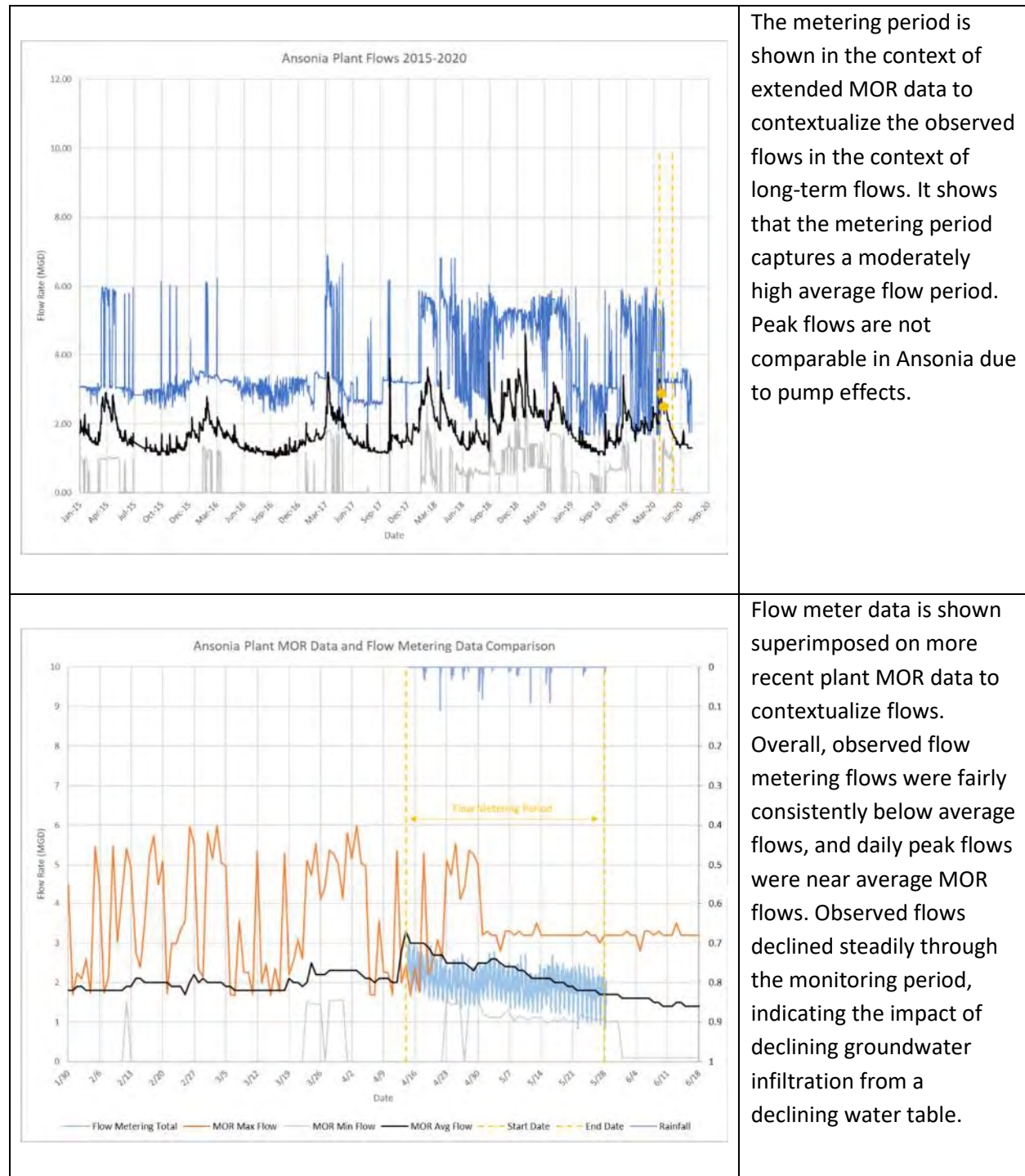


Figure 4-3 Ansonia Plant MOR Data and Flow Metering Data Comparison

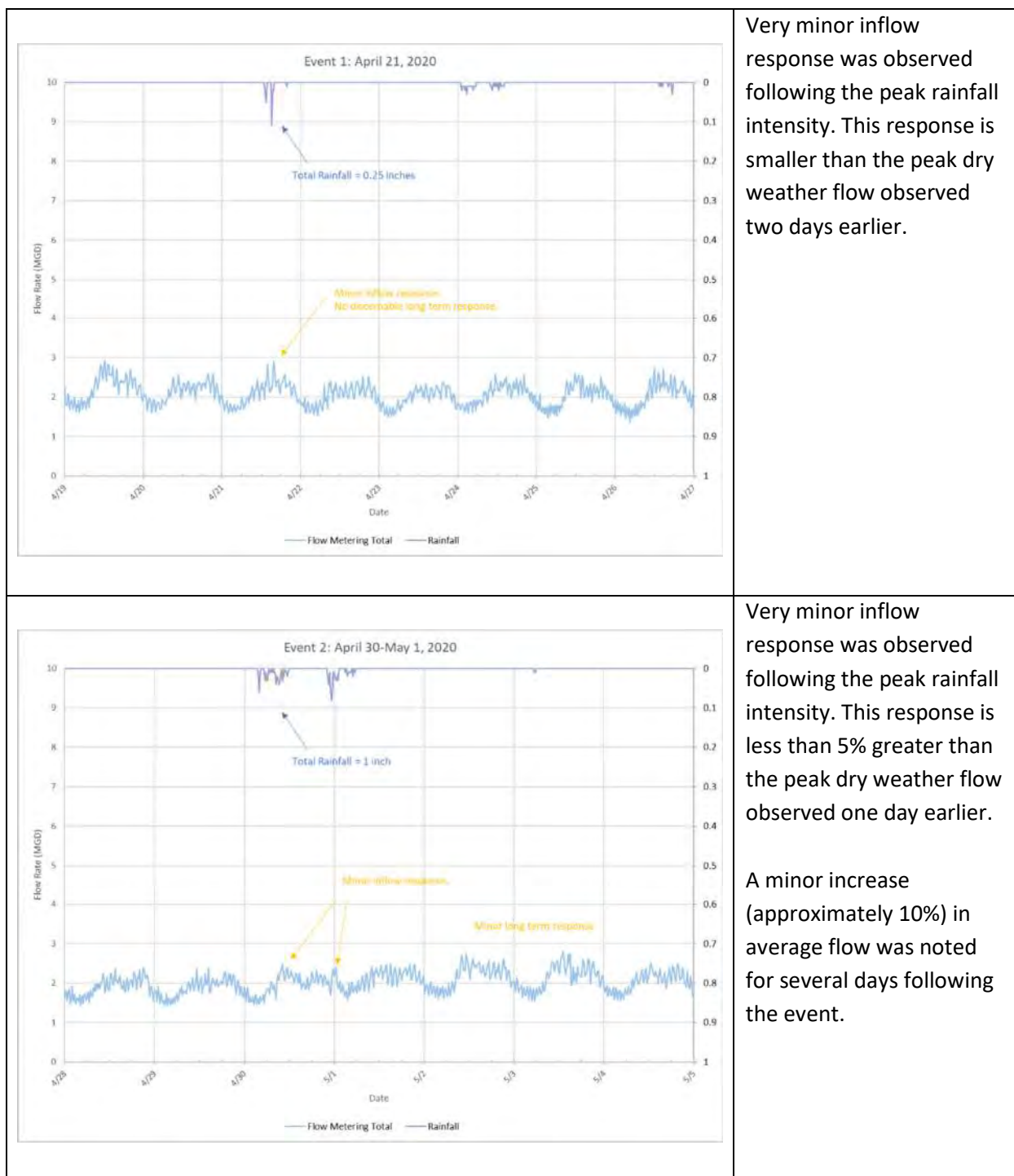


Figure 4-4 Ansonia Flow Metering Storm Event Observations

4.1.2.3 Seymour Flow Monitoring

Flow rates from the monitoring period were compared with historical monthly operating report (MOR) flow data dating to 2015, as shown in Figure 4-5. The MOR data showed that flows in the monitoring period, while representing a high for 2020, were substantially lower than

prior years. Data during the monitoring period were quite consistent. Flow responses during the two storm events are presented in Figure 4-6, which shows that there was little to no observable rainfall response in the system. Further flow monitoring is recommended during wetter conditions before arriving at any conclusions regarding the low level of I/I observed in the system.

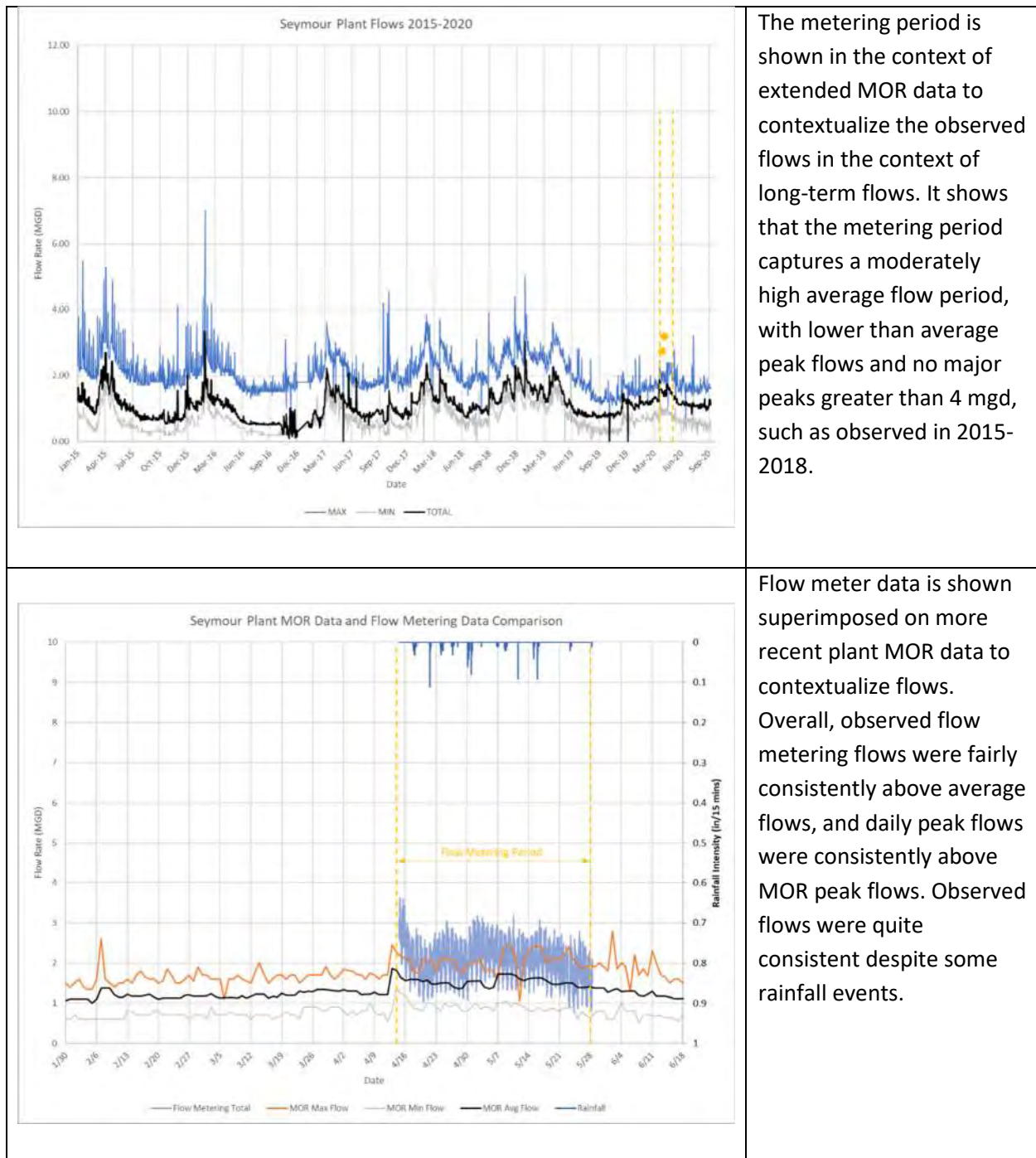


Figure 4-5 Seymour Plant MOR Data and Flow Metering Data Comparison

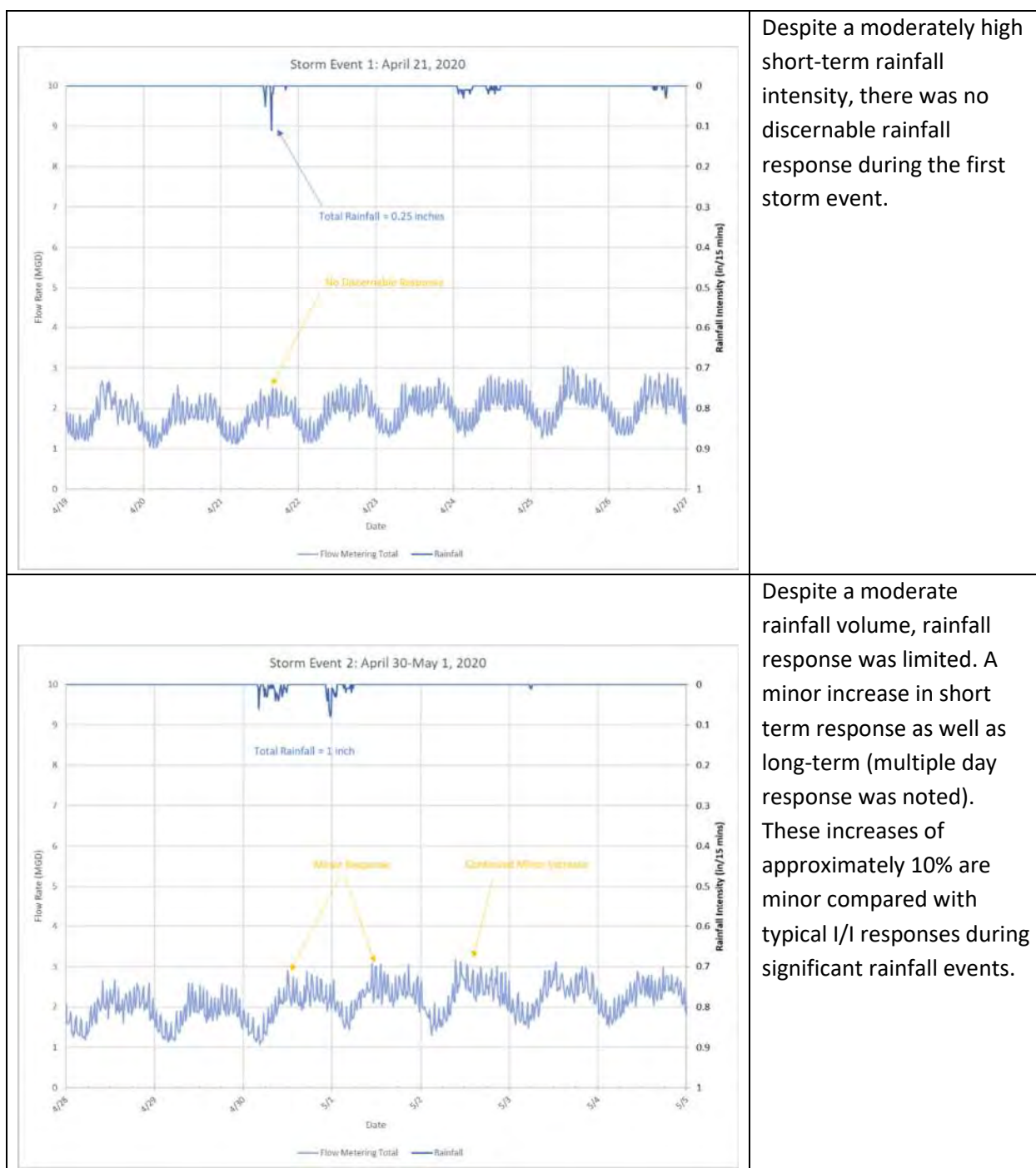


Figure 4-6 Seymour Flow Metering Storm Event Observations

4.1.3 Major Inflow Sources

Inflow sources typically provide very rapid response to rainfall, with a source of direct entry to the sewer system. These flows can lead to very high peaks that quickly overwhelm a sewer system. Where significant inflow is identified, it is typically the most cost-effective and beneficial approach to remove those sources. However, inflow sources are also frequently over-emphasized.

Solving these problems will reduce I/I, but there are many other sources as well. Therefore, it will not reduce I/I to desired levels but will be the first step in I/I control. Common contributors of inflow include legal and illegal sources, including:

- Sump pumps
- Roof leaders
- Surface drainage to manholes
- Cross connections to storm sewers or catch basins

4.1.4 Infiltration Sources

Infiltration of various sorts are typically the predominant sources of I/I in most systems, especially when major sources of inflow have already been removed. Infiltration can be long term infiltration due to high groundwater in parts of the system, or it can be storm infiltration, which can be very rapid or gradual, depending on system defects as well as soil and surface characteristics. Infiltration is typically more difficult and more costly to manage than inflow.

4.1.5 Sewer System Evaluation Surveys

A sewer system evaluation survey (SSES) is used to identify potential sources of I/I and target appropriate repairs. The most common elements of SSES are identified below. There are multiple methods of inspection that vary in cost and precision.

- Smoke testing,
- Dye testing,
- Flow isolation monitoring,
- Manhole inspections, and
- Pipe inspections.

5.0 COSTS

5.1 CAPITAL COSTS

While budgetary in scope, capital costs were estimated using standard industry methods and cost data. The estimated capital costs are considered AACE Class 4 with an accuracy range of $\pm 30\%$. Wherever possible, unit prices used for the capital costs were standardized to ensure consistency between the base case and regional alternatives.

Costs for wastewater plant facilities and upgrades, conveyance pipelines, and conveyance pump stations were estimated based on planning level quantities for comparative purposes and include equipment, construction installation, and startup. The costs also include general requirements, contractor overhead and profit, a 40% contingency, and a 20% allowance for engineering, legal, administration, and associated services during design and construction. These costs were escalated at a fixed rate of 3.5% through the anticipated midpoint of construction periods based on recent industry trends.

Costs for programmed collection system improvements were estimated on a set cost per linear foot of the collection systems, inflated at a fixed annual rate of 2% across the planning period based on historical trends.

5.1.1 Wastewater Treatment Plants

5.1.1.1 Base Cases

Plant upgrade capital costs for the base cases are summarized in Table 5-1, represented at the midpoint of anticipated construction.

Table 5-1 Plant Upgrade Capital Costs for Base Cases

Facility/Area	Derby	Ansonia	Seymour
Headworks and Influent Pumping	\$11,600,000	\$2,400,000	\$9,300,000
Primary Clarifiers	\$2,600,000	NA	\$1,500,000
BNR Process Upgrades	\$14,600,000	NA	\$4,100,000
Secondary Clarifiers	\$1,200,000	NA	\$2,200,000
Disinfection	\$900,000	\$2,200,000	NA
Effluent Pumps	NA	\$1,900,000	NA
Primary & Secondary Control Building Upgrades	\$4,200,000	NA	NA
Sludge Handling Facilities	\$8,700,000	\$8,700,000	\$2,500,000
Sitewide Electrical and Controls	\$5,300,000	NA	\$4,400,000
Sitework	\$1,400,000	\$900,000	\$400,000
General Upgrades/Miscellaneous	\$1,200,000	\$400,000	\$500,000
Total	\$51,700,000	\$16,500,000	\$24,900,000

5.1.1.2 Derby Regional Alternatives

Plant upgrade capital costs for the Derby regional alternatives are summarized in Table 5-2, represented at the midpoint of anticipated construction.

Table 5-2 Plant Upgrade Capital Costs for Derby Regional Alternatives

Facility/Area	Derby + Ansonia (BioMag)	Derby + Ansonia (IFAS)	Derby + Ansonia + Seymour (BioMag)	Derby + Ansonia + Seymour (IFAS)
Influent Pump Station and Screening	\$6,600,000	\$6,600,000	\$6,600,000	\$6,600,000
New Grit Removal Facility	\$7,600,000	\$7,600,000	\$7,600,000	\$7,600,000
Primary Clarifiers	\$2,600,000	\$2,600,000	\$2,600,000	\$2,600,000
BNR Process Upgrade and Fitout	\$26,200,000	\$30,700,000	\$27,200,000	\$34,200,000
New BNR Tankage	\$6,300,000	NA	\$10,400,000	\$6,300,000
New Secondary Clarifier	NA	\$4,700,000	\$4,700,000	\$8,400,000
Existing Secondary Clarifiers	\$1,200,000	\$1,200,000	\$1,200,000	\$1,200,000
New UV Disinfection Facility	\$4,100,000	\$4,100,000	\$4,100,000	\$4,100,000
Primary Control Building	\$2,700,000	\$2,700,000	\$2,700,000	\$2,700,000
Secondary Control Building	\$1,500,000	\$1,500,000	\$1,500,000	\$1,500,000
New Sludge Handling Facility	\$9,100,000	\$9,100,000	\$9,500,000	\$9,500,000
Sitewide Electrical And I&C	\$5,300,000	\$5,300,000	\$5,300,000	\$5,300,000
Sitework	\$1,600,000	\$1,600,000	\$1,600,000	\$1,600,000
General Upgrades/Miscellaneous	\$1,200,000	\$1,200,000	\$1,200,000	\$1,200,000
Total	\$76,000,000	\$78,900,000	\$86,200,000	\$92,800,000

5.1.1.3 Ansonia Regional Alternatives

Plant upgrade capital costs for the Ansonia regional alternatives are summarized in Table 5-3, represented at the midpoint of anticipated construction.

Table 5-3 Plant Upgrade Capital Costs for Ansonia Regional Alternatives

Facility/Area	Ansonia + Derby	Ansonia + Derby (Effluent to Housatonic)	Ansonia + Derby + Seymour	Ansonia + Derby + Seymour (Effluent to Housatonic)
Influent Screening	\$1,600,000	\$1,600,000	\$1,700,000	\$1,700,000
New Grit Removal Facility	\$4,100,000	\$4,100,000	\$4,100,000	\$4,100,000
Headworks Odor Control	\$800,000	\$800,000	\$900,000	\$900,000
New Primary Clarifier and CEPT	\$3,400,000	\$3,400,000	\$4,300,000	\$4,300,000
New Secondary Clarifier	NA	NA	\$7,500,000	\$7,500,000
Influent and Effluent Pumps	\$3,800,000	\$3,800,000	\$3,900,000	\$3,900,000
New Sludge Handling Facility	\$9,200,000	\$9,200,000	\$9,600,000	\$9,600,000
New Phosphorus Treatment Facility	\$14,200,000	NA	\$15,100,000	NA
UV Disinfection Facility	\$2,200,000	\$2,200,000	\$2,200,000	\$2,200,000
Sitework	\$900,000	\$900,000	\$1,000,000	\$1,000,000
General Upgrades/Miscellaneous	\$500,000	\$500,000	\$500,000	\$500,000
Total	\$40,700,000	\$26,500,000	\$50,800,000	\$35,700,000

5.1.2 Conveyance Pipelines and Pump Stations

Capital costs for the regional conveyance systems are summarized in Table 5-4, represented at the midpoint of anticipated construction.

Table 5-4 Regional Conveyance Pipeline Capital Costs

	Ansonia to Derby	Ansonia and Seymour to Derby	Derby to Ansonia	Derby to Ansonia, Effluent to Housatonic	Derby and Seymour to Ansonia	Derby and Seymour to Ansonia, Effluent to Housatonic
Conveyance Pipeline	\$13,600,000	\$34,200,000	\$13,000,000	\$19,600,000	\$33,200,000	\$39,900,000
Pump Stations, Screenings, Grit Removal	\$4,200,000	\$15,700,000	\$6,300,000	\$6,300,000	\$17,700,000	\$17,700,000
Total	\$17,800,000	\$49,900,000	\$19,300,000	\$25,900,000	\$50,900,000	\$57,600,000

5.1.3 Collection Systems

Capital costs for programmatic collection system improvements are summarized in Table 5-5, represented at the midpoint of the study period. These capital improvements costs are the same between the base cases and the different regional alternatives, as the existing collection systems will not change based on regionalization; however, it is imperative that these improvements are accounted for and implemented.

Table 5-5 Collection System Capital Costs

	Derby	Ansonia	Seymour
Collection System Length (miles)	41.2	65.3	63.0
System replacement rate (yr 1-5)	2.5%	2%	2%
System replacement rate (yr 6-25)	1.2%	1%	0.75%
System replacement cost (yr 1-5)	\$2,860,000	\$3,620,000	\$3,500,000
System replacement cost (yr 6-25)	\$7,030,000	\$9,280,000	\$6,720,000
Pump station replacement cost	\$4,380,000	\$3,150,000	\$2,100,000
Total	\$14,300,000	\$16,100,000	\$12,300,000

5.2 OPERATION AND MAINTENANCE COSTS

Relevant operational and maintenance (O&M) costs were estimated for the following key categories where O&M cost differences were anticipated between base cases and the regional alternatives: energy, chemicals, sludge disposal, disinfection, and labor. These costs were calculated based on unit costs applied to estimated O&M unit quantities. Unit costs were based on actual O&M cost data obtained from Derby, Ansonia, and Seymour plant staff for categories where cost information was readily available. To a limited extent, the unit costs from available market data were also reviewed. The calculated unit costs were escalated at a fixed annual rate of 2% across the planning period based on historical trends. Unit costs were standardized for all O&M costs to ensure consistency in analysis between the base case and regional alternatives. O&M unit costs are summarized in Table 5-6, represented at the midpoint of the study period.

Table 5-6 O&M Unit Costs

O&M Category	Unit Cost
Energy	\$0.18/kWh
Ferric	\$1.15/gallon
Polymer	\$19.21/gallon
Magnetite ballast	\$0.45/pound
Thickened sludge disposal, 4.5% solids	\$32.80/wet ton
Labor, superintendent	\$98.54/hr
Labor, O&M staff (blended)	\$69.38/hr

Startup factors were applied to totalized O&M costs to account for projected growth and associated increases to plant flows and loads over time. The startup factors are weighted averages of current 2020 loading to projected 2040 loading for each base case and alternative. O&M costs were totalized for 25 years, accounting for project implementation and startup within the first five years; for regional alternatives, base case O&M costs were carried for the first five years and the regional alternative O&M costs were carried for the subsequent 20 years.

Black & Veatch conducted an O&M staff evaluation to determine staff structures for the base cases and regional alternatives. This evaluation is discussed in Appendix C.

5.2.1 Base Cases

O&M costs associated with energy, relevant chemicals, disinfection, sludge disposal, and operations staff for the base cases are shown in Table 5-7, Table 5-8, Table 5-9, Table 5-10, and Table 5-11 below. Labor includes O&M costs of the plants and collection systems. A summary of annual and 25-year total O&M costs is shown in Table 5-12. Quantities were calculated based on the projected 2040 flows and loads; annual costs are expressed at the midpoint of the study period.

Table 5-7 O&M Energy Costs for Ansonia, Derby, and Seymour Base Case

	Ansonia	Derby	Seymour
Aeration Energy, hp	142	38.8	27.9
Influent Pump Energy, hp	19.1	19.3	13
RAS Pump Energy, hp	3.3	3.4	2.3
MLR Pump Energy, hp	9.5	9.6	6.5
Mixing Energy, hp	20	6	6
Fixed Energy Costs, hp	83.1	53.6	38.7
Total Energy, hp	277	130.7	94.4
Annual Energy Usage, kWh/yr	1,810,700	854,400	617,100
Startup Factor	0.92	0.89	0.75
Annual Electricity Costs	\$313,500	\$145,600	\$97,000
25-year Electricity Costs	\$7,838,000	\$3,639,000	\$2,424,000

Table 5-8 O&M Chemical Costs for Ansonia, Derby, and Seymour Base Case

	Ansonia	Derby	Seymour
Coagulant Dosage CEPT, gal/d	0	101	0
Coagulant Dosage P Removal, gal/d	225	0	127
Polymer Dosage CEPT, gal/d	0	4	0
Startup Factor	0.92	0.89	0.75
Annual Chemical Costs	\$90,900	\$66,700	\$46,600
25-year Chemical Costs	\$2,273,000	\$1,668,000	\$1,164,000

Table 5-9 O&M Disinfection Costs for Ansonia, Derby, and Seymour Base Case

	Ansonia	Derby	Seymour
Startup Factor	0.92	0.89	0.75
Annual UV Disinfection Costs	\$24,000	\$0	\$0
Annual Chemical Disinfection Costs	\$0	\$8,300	\$8,300
25-year Disinfection Costs	\$601,000	\$208,000	\$208,000

Table 5-10 O&M Sludge Disposal Costs for Ansonia, Derby, and Seymour Base Case

	Ansonia	Derby	Seymour
Total Sludge Production, dry tons/year	541	361	359
Thickened Sludge Production, wet tons/year ⁽¹⁾	12,018	8,019	7,980
Startup Factor	0.92	0.89	0.75
Annual Sludge Disposal Costs	\$379,200	\$249,000	\$228,500
25-year Sludge Disposal Costs	\$9,481,000	\$6,224,000	\$5,713,000
(1) Assumes a 4.5% solids concentration			

Table 5-11 O&M Labor Costs for Ansonia, Derby, and Seymour Base Case

	Ansonia	Derby	Seymour
Plant Superintendent, FTE	1.0	1.0	1.0
Plant Operations and Maintenance, FTE	4.0	5.0	4.0
Collection System O&M, FTE	2.0	2.0	2.0
Annual Labor Cost	\$1,070,800	\$1,215,100	\$1,070,800
25-year Labor Costs	\$26,771,000	\$30,378,000	\$26,771,000

Table 5-12 Total O&M Costs for Ansonia, Derby, and Seymour Base Case

	Ansonia	Derby	Seymour
Total Annual O&M Costs	\$1,878,000	\$1,685,000	\$1,451,000
25-year O&M Costs	\$47,000,000	\$42,100,000	\$36,300,000

5.2.2 Derby Regional Alternatives O&M Costs

O&M costs associated with energy, relevant chemicals, disinfection, sludge disposal, and operations staff for the Derby regional alternatives are shown in Table 5-13, Table 5-14, Table 5-15, Table 5-16, and Table 5-17 below. Labor includes O&M costs of the plants, conveyance systems, and collection systems. A summary of annual and 25-year total O&M costs is shown in Table 5-18. Quantities were calculated based on the projected 2040 flows and loads; annual costs are

expressed at the midpoint of the study period. Associated base case O&M costs were carried for the first five years to account for project implementation and startup.

Table 5-13 O&M Energy Costs for Derby Regional Alternatives

	Derby + Ansonia (BioMag)	Derby + Ansonia (IFAS)	Derby + Ansonia + Seymour (BioMag)	Derby + Ansonia + Seymour (IFAS)
Aeration Energy, hp	147.8	147.8	200.6	200.6
Influent Pump Energy, hp	38.3	38.3	51.4	51.4
RAS Pump Energy, hp	6.7	3.4	9	4.5
MLR Pump Energy, hp	19.2	19.2	25.7	25.7
Mixing Energy, hp	8	6	10	8
Fixed Energy Costs, hp	129.2	126.1	146.1	142.9
BioMag Recovery Energy, hp	33	NA	41	NA
Conveyance Pumping Energy, hp	8	8	60	60
Total Energy, hp	390.2	348.8	543.8	493.1
Annual Energy Usage, kWh/yr	2,550,700	2,280,000	3,554,700	3,223,300
Startup Factor	0.91	0.91	0.81	0.81
Annual Electricity Costs, /yr	\$438,000	\$391,500	\$579,700	\$525,700
25-year Electricity Costs	\$11,055,000	\$10,126,000	\$14,374,000	\$13,293,000

Table 5-14 O&M Chemical Costs for Derby Regional Alternatives

	Derby + Ansonia (BioMag)	Derby + Ansonia (IFAS)	Derby + Ansonia + Seymour (BioMag)	Derby + Ansonia + Seymour (IFAS)
Coagulant Dosage CEPT, gal/d	202	202	270	270
Polymer Dosage CEPT, gal/d	7	7	9	9
Polymer Dosage BioMag, gal/d	30	NA	42	NA
Ballast Usage, lb/d	174	NA	224	NA
Startup Factor	0.91	0.91	0.81	0.81
Annual Chemical Costs, \$/yr	\$355,900	\$127,800	\$460,300	\$160,000
25-year Chemical Costs, \$M	\$7,906,000	\$3,344,000	\$10,227,000	\$4,220,000

Table 5-15 O&M Disinfection Costs for Derby Regional Alternatives

	Derby + Ansonia (BioMag)	Derby + Ansonia (IFAS)	Derby + Ansonia + Seymour (BioMag)	Derby + Ansonia + Seymour (IFAS)
Startup Factor	0.91	0.91	0.81	0.81
Annual UV Disinfection Costs	\$38,300	\$38,300	\$42,800	\$42,800
25-year Disinfection Costs	\$928,000	\$928,000	\$1,059,000	\$1,059,000

Table 5-16 O&M Sludge Disposal Costs for Derby Regional Alternatives

	Derby + Ansonia (BioMag)	Derby + Ansonia (IFAS)	Derby + Ansonia + Seymour (BioMag)	Derby + Ansonia + Seymour (IFAS)
Total Sludge Production, dry tons/year	902	902	1,261	1,261
Thickened Sludge Production, wet tons/year ⁽¹⁾	20,038	20,038	28,018	28,018
Startup Factor	0.91	0.91	0.81	0.81
Annual Sludge Disposal Costs	\$627,000	\$627,000	\$832,600	\$832,600
25-year Sludge Disposal Costs	\$15,681,000	\$15,681,000	\$20,935,000	\$20,935,000
(1) Assumes a 4.5% solids concentration				

Table 5-17 O&M Labor Costs for Derby Regional Alternatives

	Derby + Ansonia (BioMag)	Derby + Ansonia (IFAS)	Derby + Ansonia + Seymour (BioMag)	Derby + Ansonia + Seymour (IFAS)
Plant Superintendent, FTE	1.0	1.0	1.0	1.0
Plant Operations and Maintenance, FTE	5.0	5.0	5.0	5.0
Collection System O&M, FTE	4.0	4.0	6.0	6.0
Annual Labor Cost	\$1,503,800	\$1,503,800	\$1,792,400	\$1,792,400
25-year Labor Costs	\$41,505,000	\$41,505,000	\$52,631,000	\$52,631,000

Table 5-18 Total O&M Costs for Derby Regional Alternatives

	Derby + Ansonia (BioMag)	Derby + Ansonia (IFAS)	Derby + Ansonia + Seymour (BioMag)	Derby + Ansonia + Seymour (IFAS)
Total Annual O&M Costs	\$2,963,000	\$2,688,000	\$3,708,000	\$3,354,000
25-year O&M Costs	\$77,100,000	\$71,600,000	\$99,200,000	\$92,100,000

5.2.3 Ansonia Regional Alternatives O&M Costs

O&M costs associated with energy, relevant chemicals, disinfection, sludge disposal, and operations staff for the Ansonia regional alternatives are shown in Table 5-19, Table 5-20, Table 5-21, Table 5-22, and Table 5-23 below. Labor includes O&M costs of the plants, conveyance systems, and collection systems. A summary of annual and 25-year total O&M costs is shown in Table 5-24. Quantities were calculated based on the projected 2040 flows and loads; annual costs are expressed at the midpoint of the study period. Associated base case O&M costs were carried for the first five years to account for project implementation and startup.

Table 5-19 O&M Energy Costs for Ansonia Regional Alternatives

	Ansonia + Derby	Ansonia + Derby (Effluent to Housatonic)	Ansonia + Derby + Seymour	Ansonia + Derby + Seymour (Effluent to Housatonic)
Aeration Energy, hp	239	239	331	331
Influent Pump Energy, hp	38.3	38.3	51.4	51.4
RAS Pump Energy, hp	6.7	6.7	9	9
MLR Pump Energy, hp	19.2	19.2	25.7	25.7
Mixing Energy, hp	20	20	20	20
Fixed Energy Costs, hp	113.6	113.6	123.3	123.3
Tertiary Facility Energy, hp	4	NA	4	NA
Conveyance Pumping Energy, hp	8	16	50	68
Total Energy, hp	448.8	452.8	614.4	628.4
Annual Energy Usage, kWh/yr	2,933,700	2,959,900	4,016,200	4,107,700
Startup Factor	0.91	0.91	0.81	0.81
Annual Electricity Costs, /yr	\$503,800	\$508,300	\$655,000	\$669,900
25-year Electricity Costs	\$12,371,000	\$12,461,000	\$15,880,000	\$16,178,000

Table 5-20 O&M Chemical Costs for Ansonia Regional Alternatives

	Ansonia + Derby	Ansonia + Derby (Effluent to Housatonic)	Ansonia + Derby + Seymour	Ansonia + Derby + Seymour (Effluent to Housatonic)
Coagulant Dosage CEPT, gal/d	202	202	270	270
Coagulant Dosage P Removal, gal/d	80	0	108	0
Polymer Dosage CEPT, gal/d	7	7	9	9
Startup Factor	0.91	0.91	0.81	0.81
Annual Chemical Costs, \$/yr	\$159,900	\$127,800	\$201,100	\$160,000
25-year Chemical Costs, \$M	\$3,985,000	\$3,344,000	\$5,042,000	\$4,220,000

Table 5-21 O&M Disinfection Costs for Ansonia Regional Alternatives

	Ansonia + Derby	Ansonia + Derby (Effluent to Housatonic)	Ansonia + Derby + Seymour	Ansonia + Derby + Seymour (Effluent to Housatonic)
Startup Factor	0.91	0.91	0.81	0.81
Annual UV Disinfection Costs	\$38,300	\$38,300	\$42,800	\$42,800
25-year Disinfection Costs	\$928,000	\$928,000	\$1,059,000	\$1,059,000

Table 5-22 O&M Sludge Disposal Costs for Ansonia Regional Alternatives

	Ansonia + Derby	Ansonia + Derby (Effluent to Housatonic)	Ansonia + Derby + Seymour	Ansonia + Derby + Seymour (Effluent to Housatonic)
Total Sludge Production, dry tons/year	902	902	1,261	1,261
Thickened Sludge Production, wet tons/year ⁽¹⁾	20,038	20,038	28,018	28,018
Startup Factor	0.91	0.91	0.81	0.81
Annual Sludge Disposal Costs	\$627,000	\$627,000	\$832,600	\$832,600
25-year Sludge Disposal Costs	\$15,681,000	\$15,681,000	\$20,935,000	\$20,935,000
(1) Assumes a 4.5% solids concentration				

Table 5-23 O&M Labor Costs for Ansonia Regional Alternatives

	Ansonia + Derby	Ansonia + Derby (Effluent to Housatonic)	Ansonia + Derby + Seymour	Ansonia + Derby + Seymour (Effluent to Housatonic)
Plant Superintendent, FTE	1.0	1.0	1.0	1.0
Plant Operations and Maintenance, FTE	5.0	5.0	5.0	5.0
Collection System O&M, FTE	4.0	4.0	6.0	6.0
Annual Labor Cost	\$1,503,800	\$1,503,800	\$1,792,400	\$1,792,400
25-year Labor Costs	\$41,505,000	\$41,505,000	\$52,631,000	\$52,631,000

Table 5-24 Total O&M Costs for Ansonia Regional Alternatives

	Ansonia + Derby	Ansonia + Derby (Effluent to Housatonic)	Ansonia + Derby + Seymour	Ansonia + Derby + Seymour (Effluent to Housatonic)
Total Annual O&M Costs	\$2,833,000	\$2,805,000	\$3,524,000	\$3,498,000
25-year O&M Costs	\$74,500,000	\$73,900,000	\$95,500,000	\$95,000,000

5.2.4 Conveyance Systems and Collection Systems

Labor and conveyance energy O&M costs associated with the conveyance systems and collection systems are carried under the plant O&M costs in the tables above.

5.3 PRESENT WORTH ANALYSIS

A common approach to comparing alternatives is to use a present worth (PW) analysis. The PW method allows for monetary costs associated with capital expenditures and O&M costs over the planning period to be expressed as a present equivalent value. The PW analysis allows for these costs to be expressed in common units enabling a comparison of distinct alternatives. The alternative with the lowest present worth cost is the most favorable as compared to the others. The PW analysis is often also referred to as a lifecycle analysis, which acknowledges the

useful lives of assets in the investment along with the impact on operations and maintenance (O&M) costs for the duration of the analysis.

A common approach to evaluating alternatives is to start the assessment with a high-level analysis of multiple alternatives and systematically reduce the number of alternatives through increasingly rigorous technical and financial review. For this study, there were 23 alternatives initially explored in Phase 1 and, through the subsequent analysis conducted in Task 2, these were reduced to six short-list regional alternatives. Using the present worth method, a detailed cost analysis was performed as part of this task to further reduce the short-listed alternatives down to the preferred ones to carry forward.

For this detailed cost analysis, inflation and escalation rates were applied to capital and O&M costs as noted previously in this chapter, calculated as averages across assumed PW durations. Table 5-25 shows the duration and timing of costs assumed for the present worth analysis. An annual interest rate of 2.2% was assumed over the 25-year evaluation period based on loans available through the CT DEEP Clean Water Fund program.

Table 5-25 Present Worth Analysis Time Assumptions

Present Worth Cost Category	Present Worth Start Year	Present Worth Duration
Derby and Seymour base case plant upgrades	3	Three years
Ansonia base case plant upgrades	4	Two years
Regional alternative plant upgrades	3	Three years
Conveyance pipelines and pumping	4	Two years
Collection system improvements	0	25 years (planning period)
Base case O&M costs ⁽¹⁾	0	25 years (planning period)
Regional alternative O&M costs ⁽¹⁾	6	20 years
(1) O&M costs for base cases associated with the regional alternative were carried for the first five years of the planning period.		

This analysis did not incorporate value of assets, salvage value or funding sources including cash, bonds, or grants. It is assumed that these values are comparable for each base case and regional alternative for the purposes of consistent evaluation. We also believe that these items will not change the results of the PW evaluation performed as part of this Task 3 work.

Table 5-26 summarizes the base case present worth costs and Table 5-27 summarizes the regional alternative present worth costs.

Table 5-26 Base Case Present Worth Costs

Base Case	Capital Present Worth Costs ⁽¹⁾	O&M Present Worth Costs	Total Present Worth Costs
Derby	\$58,400,000	\$32,100,000	\$90,500,000
Ansonia	\$27,300,000	\$35,800,000	\$63,100,000
Seymour	\$32,400,000	\$27,700,000	\$60,100,000
(1) Costs include wastewater treatment plant and collection system improvements.			

Table 5-27 Regional Alternative Present Worth Costs

No.	Regional Alternative	Capital Present Worth Costs ⁽¹⁾	O&M Present Worth Costs	Total Present Worth Costs
	Ansonia Regional Alternatives			
3	Derby to Ansonia	\$78,200,000	\$57,500,000	\$135,700,000
4	Derby to Ansonia, Effluent Pumped to Housatonic River	\$71,100,000	\$57,100,000	\$128,200,000
5	Derby and Seymour to Ansonia	\$125,800,000	\$74,200,000	\$200,000,000
5b	Derby and Seymour to Ansonia, Effluent Pumped to Housatonic River	\$117,900,000	\$73,800,000	\$191,700,000
	Derby Regional Alternatives	\$109,200,000	\$59,300,000	\$168,500,000
8a	Ansonia to Derby (BioMag)	\$111,900,000	\$55,400,000	\$167,300,000
8b	Ansonia to Derby (IFAS)	\$157,200,000	\$76,800,000	\$234,000,000
9a	Seymour and Ansonia to Derby (BioMag)	\$163,300,000	\$71,700,000	\$235,000,000
9b	Seymour and Ansonia to Derby (IFAS)	\$78,200,000	\$57,500,000	\$135,700,000
(1) Costs include wastewater treatment plant, conveyance systems, and collection system improvements.				

6.0 CONCLUSION AND RECOMMENDATIONS

6.1 BASE CASE AND REGIONAL ALTERNATIVES COMPARISON

Table 6-1 shows the comparison of regional alternative and base case present worth costs.

Table 6-1 Base Case and Regional Alternatives Comparison

No.	Regional Alternative	Regional Alternative Costs (millions)			Base Cases	Base Case Costs (millions)			Present Worth Difference (millions)
		Capital	O&M	Total		Capital	O&M	Total	
	Ansonia Regional Alternatives								
3	Derby to Ansonia	\$78.2	\$57.5	\$135.7	Derby + Ansonia	\$85.7	\$67.9	\$153.6	\$17.9
4	Derby to Ansonia, Effluent Pumped to Housatonic River	\$71.1	\$57.1	\$128.2	Derby + Ansonia	\$85.7	\$67.9	\$153.6	\$25.4
5	Derby + Seymour to Ansonia	\$125.8	\$74.2	\$200.0	Derby + Ansonia + Seymour	\$118.1	\$95.6	\$213.7	\$13.7
5b	Derby + Seymour to Ansonia, Effluent Pumped to Housatonic River	\$117.9	\$73.8	\$191.7	Derby + Ansonia + Seymour	\$118.1	\$95.6	\$213.7	\$22.0
	Derby Regional Alternatives								
8a	Ansonia to Derby (BioMag)	\$109.2	\$59.3	\$168.5	Derby + Ansonia	\$85.7	\$67.9	\$153.6	(\$14.9)
8b	Ansonia to Derby (IFAS)	\$111.9	\$55.4	\$167.3	Derby + Ansonia	\$85.7	\$67.9	\$153.6	(\$13.7)
9a	Seymour + Ansonia to Derby (BioMag)	\$157.2	\$76.8	\$234.0	Derby + Ansonia + Seymour	\$118.1	\$95.6	\$213.7	(\$20.3)
9b	Seymour + Ansonia to Derby (IFAS)	\$163.3	\$71.7	\$235.0	Derby + Ansonia + Seymour	\$118.1	\$95.6	\$213.7	(\$21.3)

6.2 RECOMMENDED ALTERNATIVES

Based on the present worth cost comparison, the Ansonia regional alternatives are financially more attractive than both the Derby regional alternatives and the base case scenarios. The Ansonia regional alternatives that convey treated secondary effluent back to Derby for discharge to the Housatonic River (4 and 5b) are the two most financially attractive alternatives and are recommended to be carried forward for final development in Task 4 and potential implementation.

Of these two preferred regional alternatives—*Derby to Ansonia, Effluent Pumped to Housatonic River*—is the more financially attractive; however, the other alternative that also includes Seymour—*Derby + Seymour to Ansonia, Effluent Pumped to Housatonic River*—has the added advantage of eliminating two wastewater treatment plant discharges completely.

These two regional alternatives benefit all participating communities; however, it is noted that the benefit is proportional to the current improvements needed at the respective plants.

6.2.1 Additional Notes

As noted in the present worth analysis summary, project funding has not been included in the cost analysis. Grants through the CT DEEP Clean Water Fund program are available for qualifying wastewater plant projects. These grants are prioritized for regional authorities; therefore, it is anticipated that more grant funds would be available for a regional alternative plant and related systems as compared to base case plants.

6.3 TASK 4 LOOK AHEAD

This TM summarizes the work conducted in Task 3 to develop the short list of NVCOG regional wastewater alternatives and define the recommended alternatives for final investigations. These conclusions and recommendations will be reviewed in a workshop (Workshop No. 2) with the NVCOG stakeholders where concurrence will be reached on the recommended alternatives. After Workshop No. 2 is complete, Task 4 activities will advance to further develop the recommended alternatives and the preparation of final technical report.

APPENDIX A

GENERAL PROCESS CONSIDERATIONS AND DATA UPDATES

APPENDIX A GENERAL PROCESS CONSIDERATIONS AND DATA UPDATES

A.1 Introduction

Several key assumptions and analysis methods were adopted as part of the added process evaluations conducted in this Task 3. These generally apply to each of the plants. These considerations are described below.

A.2 Chemically Enhanced Primary Treatment

An outcome of Task 2 was that the most practical way to maintain primary treatment capacity in the regionalization alternatives was to utilize chemically enhanced primary treatment (CEPT), as expanding primary clarifiers is challenging due to footprint constraints. This will result in additional removal of solids at primary treatment even at the higher loadings. In Task 2, primary TSS and BOD removals of 60% and 30% were assumed, which is typical of conventional primary treatment and supported by the MOR data at the facilities. In this task, Task 3, primary TSS and BOD removals of 75% and 37.5% will be assumed which are typical or even conservative for CEPT.

As a result of increased primary removal efficiencies, the secondary treatment capacity is increased. This was not fully accounted for during Task 2 evaluations of secondary processes as unit processes were being considered separately at that time. In this evaluation, the effect of CEPT on secondary capacity was evaluated. However, based on preliminary modeling it was decided that the nitrification safety factor for aerobic solids retention time (SRT) should be increased, which decreases capacity and mostly offsets the gains in secondary capacity resulting from CEPT.

A drawback of CEPT on secondary treatment is the detrimental impact on biological nutrient removal (BNR) process performance, which can be anticipated. This is accounted for in the present evaluation by assessing the performance through biokinetic modeling.

A.3 Flows and Loads

Wastewater flows and loads data was confirmed during Task 2 for the Derby, Ansonia, and Seymour plants. The Ansonia and Seymour flows and loads data is still current for this evaluation. New flows and loads data for the Derby plant was published in the City of Derby WPCA 2020 Wastewater Facilities Plan Supplement to the 2014 Wastewater Facilities Plant, dated June 2020. This information was reviewed against the process evaluations conducted to date, including coordination directly with the City and their engineer to confirm key assumptions. As a result of this data update, the wastewater flows and loads were reevaluated and adjusted as necessary, resulting in some changes to the Derby flows and loads. Flows and loads data is summarized for the Derby, Ansonia, and Seymour plants in Table A 1, Table A 2, and Table A 3, respectively.

Table A 1 Derby Current and Design Influent Flows and Loads

	Flow, MGD	BOD, mg/L	BOD lb/day	TSS, mg/L	TSS, lb/day	TKN, mg/L	TKN, lb/day
Current Influent							
Annual Average	1.42	221	2,621	197	2,333	28	335
Maximum Month	2.19	260	4,749	240	4,384	33	610
Peak Day	4.10	-	-	-	-	-	-
Peak Hour	10	-	-	-	-	-	-
Design Influent							
Annual Average	1.69	221	3,119	197	2,776	28	399
Maximum Month	2.61	260	5,651	240	5,216	33	726
Peak Day	4.88	-	-	-	-	-	-
Peak Hour	10	-	-	-	-	-	-

Table A 2 Ansonia Current and Design Influent Flows and Loads

	Flow, MGD	BOD, mg/L	BOD lb/day	TSS, mg/L	TSS, lb/day	TKN, mg/L	TKN, lb/day
Current Influent							
Annual Average	1.76	204	2,988	184	2,695	45	656
Maximum Month	3.06	187	4,772	191	4,874	41	1,046
Peak Day	4.60	-	-	-	-	-	-
Peak Hour	9	-	-	-	-	-	-
Design Influent							
Annual Average	1.90	204	3,236	184	2,919	45	711
Maximum Month	3.31	187	5,167	191	5,278	41	1,133
Peak Day	4.98	-	-	-	-	-	-
Peak Hour	9	-	-	-	-	-	-

Table A 3 Seymour Current and Design Influent Flows and Loads

	Flow, MGD	BOD, mg/L	BOD lb/day	TSS, mg/L	TSS, lb/day	TKN, mg/L	TKN, lb/day
Current Influent							
Annual Average	0.97	140	1,133	146	1,181	33	269
Maximum Month	1.93	93	1,497	99	1,594	22	356
Peak Day	3.34	-	-	-	-	-	-
Peak Hour	7.0	-	-	-	-	-	-
Design Influent							
Annual Average	1.30	140	1,518	146	1,583	33	361
Maximum Month	2.59	112	2,424	133	2,863	27	576
Peak Day	4.48	-	-	-	-	-	-
Peak Hour	9.4	-	-	-	-	-	-

A.4 Nitrogen Load Allocations

Total Nitrogen Annual Discharge Limits are currently dictated by the General Permit for Nitrogen Discharges. These limits are 115, 71, and 61 lb/day-N for Ansonia, Derby, and Seymour respectively, values which have not changed between the 2016-2018 General Permit and the current Proposed General Permit (2019-2023). Based on these annual limits, and the current and projected future annual average flows, target TN concentrations can be calculated as shown in Table A 4.

Table A 4 Target Effluent Nitrogen Concentrations for Base Case and Regionalization Alternatives

	Ansonia	Derby	Seymour	Ansonia + Derby	Ansonia + Derby + Seymour
Annual TN Limit, lb/day-N	115	71	61	186	247
Current Annual Avg Flow, mgd	1.76	1.42	0.97	3.17	3.89
Current Effluent N Target, mg/L-N	7.85	6.00	7.54	7.04	7.61
2040 Annual Avg Flow, mgd	1.90	1.69	1.30	3.59	4.89
2040 Effluent N Target, mg/L-N	7.25	5.04	5.63	6.21	6.05

Ansonia is consistently significantly below the load-based TN Discharge Limit, while Derby and Seymour have both been below the limit in some years and above the limit in others requiring these facilities to purchase nitrogen credits. Ansonia typically meets its limits as the concentrations required to meet the limit are >7.5 mg/L-N on average, because influent carbon to nitrogen is adequate, and because the facility has a 4-stage process with an oxidation ditch that is well suited to remove nitrogen. To avoid buying credits, Derby would need to achieve TN concentration of <6.0 mg/L-N and has a modified Ludzack-Ettinger (MLE) process which is not as well equipped to meet these limits consistently. Derby's recently completed facility plan recommends an upgrade to a 4-stage process which will have more anoxic and aerobic volume overall which will help it to meet its limits more consistently. Seymour needs to achieve <7.0 mg/L-N to meet its load-based limits, however it also operates an MLE process. The reason why the limit is not always met is likely due to the MLE process but could also be made worse by carbon-to-nitrogen (C:N) ratios which are on average lower at Seymour than the other facilities based on influent MOR data.

APPENDIX B

SLUDGE MANAGEMENT

APPENDIX B SLUDGE MANAGEMENT

B.1 Sludge Management

Sludge is currently managed for disposal differently at each of the plants. Prior to disposal, Derby and Seymour dewater their sludge and Ansonia thickens their sludge only. Sludge processing is summarized in Table B 1.

Table B 1 Existing Sludge Processing

Facility	Sludge Processing	Typical Solids Concentration
Derby WPCF	Dewatering	15%-17%
Ansonia WPCF	Thickening	3.5%-4.5%
Seymour WWTP	Dewatering	19%-21%

It is generally more cost effective to thicken sludge only instead of thickening and dewatering for wastewater treatment plants in the size range of the base case plants and the regional alternatives plants. This also takes into account the disposal and transport costs charged by area incinerator merchant plants. We have found that sludge dewatering clearly becomes cost effective at significantly larger plants than at any of the three base case facilities or the regional treatment facilities associated with the short-listed alternatives.

Sludge treatment and handling was evaluated on a planning level basis for the base cases and regional alternatives to determine what strategies were more cost effective for each of the plants. It was determined that sludge thickening only would be the most cost-effective strategy in every case, therefore this was carried forward for the regional alternatives and the base case scenarios. Figure B 1 shows a schematic depiction of a new sludge thickening facility that is envisioned for all three base case plants and the regional facilities at Derby and Ansonia.

Review of the existing sludge treatment/handling facilities at Derby indicated that these systems are old and inefficient and need to be replaced with systems as shown in Figure B 1. Additionally, the plant should cease using its aerobic digester when the new facilities are built. While the sludge treatment/handling systems at Ansonia are in better condition, they are not efficient. Therefore, on a planning basis, the sludge facilities at Ansonia are identified for replacement; this includes the base case regional alternatives plants. There may be opportunities to maximize the use of the existing sludge storage system at Ansonia in lieu of full replacement; this could be reviewed later for viability by others including if regionalization is agreed upon and selected.

The sludge treatment/handling facilities for Seymour are old, inefficient and in need of replacement. Per the planning level review noted above, the base case for Seymour calls for sludge thickening only with new facilities as shown in Figure B 1. It is noted that for Seymour, the new sludge facilities would be located within the existing sludge thickening and dewatering building.

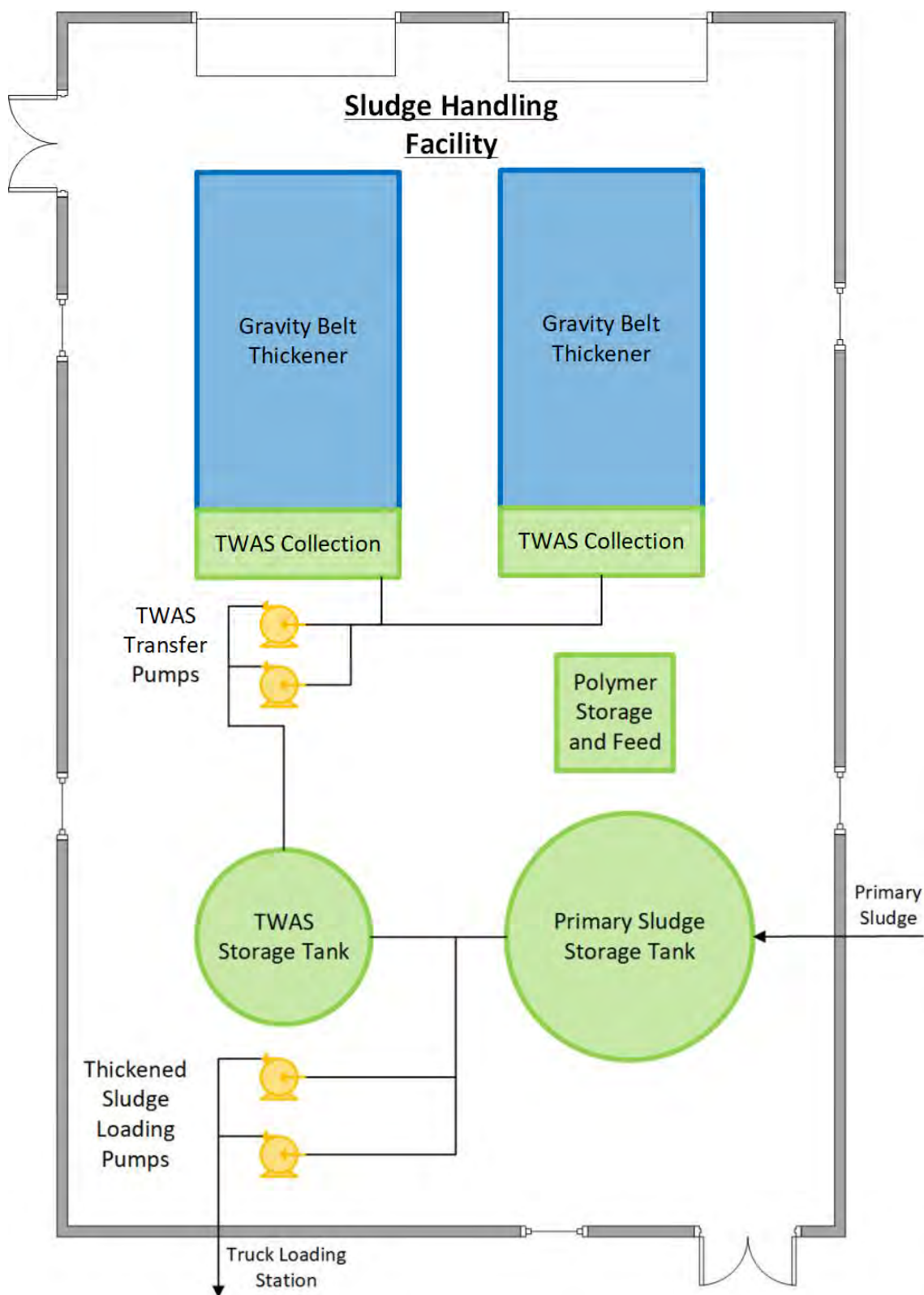


Figure B 1 New Sludge Handling Facility for Derby and Ansonia Plants

APPENDIX C

OPERATIONS AND MAINTENANCE STAFFING

APPENDIX C OPERATIONS AND MAINTENANCE STAFFING

C.1 Introduction

Staffing is a critical part of long-term operations and maintenance (O&M) of wastewater facilities and systems. A planning level O&M staffing assessment was conducted to determine staff needs across the study communities and establish comparable staff structures for the base cases and regional alternatives.

C.2 Current O&M Staffing

Staff structures, roles, and responsibilities were reviewed with Derby, Ansonia, and Seymour. All three plants operate on a basic single-shift, Monday through Friday schedule. Table C 1 summarizes the current wastewater facility staffing for the plants and collection systems.

Table C 1 Current Wastewater Facility Staffing

Facility	Plant O&M Full-Time Equivalencies ⁽¹⁾	Collection System O&M Full-Time Equivalencies
Derby WPCF	6	2
Ansonia WPCF	5	0 ⁽²⁾
Seymour WWTP	5	0 ⁽²⁾⁽³⁾
(1) Includes superintendent. (2) Plant staff at Ansonia and Seymour attend to pump stations in the collection system. (3) Plant staff at Seymour address problematic areas (“hot spots”) and handle emergencies in the collection system.		

C.3 Staffing Development

Black & Veatch relied on the expertise of our own O&M specialists to evaluate current staff structures and establish recommended staff structures for each of the base cases and regional alternatives for this evaluation. We also reviewed the New England Interstate Water Pollution Control Commission (NEIWPCC) Northeast Guide for Estimating Staffing at Publicly and Privately Owned Wastewater Treatment Plants to confirm our conclusions.

C.4 Conclusions

Table C 2 shows the recommended staff structure for the base case plants and Table C 3 shows the recommended staff structure for the regional alternatives. All operations would remain on a single-shift, Monday through Friday schedule.

Table C 2 Base Case Wastewater Facility Staffing

Facility	Plant O&M Full-Time Equivalencies ⁽¹⁾	Collection System O&M Full-Time Equivalencies
Derby WPCF	6	2
Ansonia WPCF	5	2
Seymour WWTP	5	2
(1) Includes superintendent.		

Table C 3 Regional Alternative Wastewater Facility Staffing

Regional Alternatives	Plant O&M Full-Time Equivalencies ⁽¹⁾	Collection System O&M Full-Time Equivalencies
Derby plus Ansonia	6	4
Derby plus Ansonia plus Seymour	6	6
Ansonia plus Derby	6	4
Ansonia plus Derby plus Seymour	6	6
(1) Includes superintendent.		

There may be opportunity to reduce the recommended staff by as much as 25% overall; however, we believe the staff numbers in the tables above are appropriate for comparison of the alternatives. A more in-depth study would need to be conducted on staffing after the planning has advanced beyond the scope of this initial study with both technical engineering studies and when the regional authority and structure is better defined.