DRAFT

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

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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.

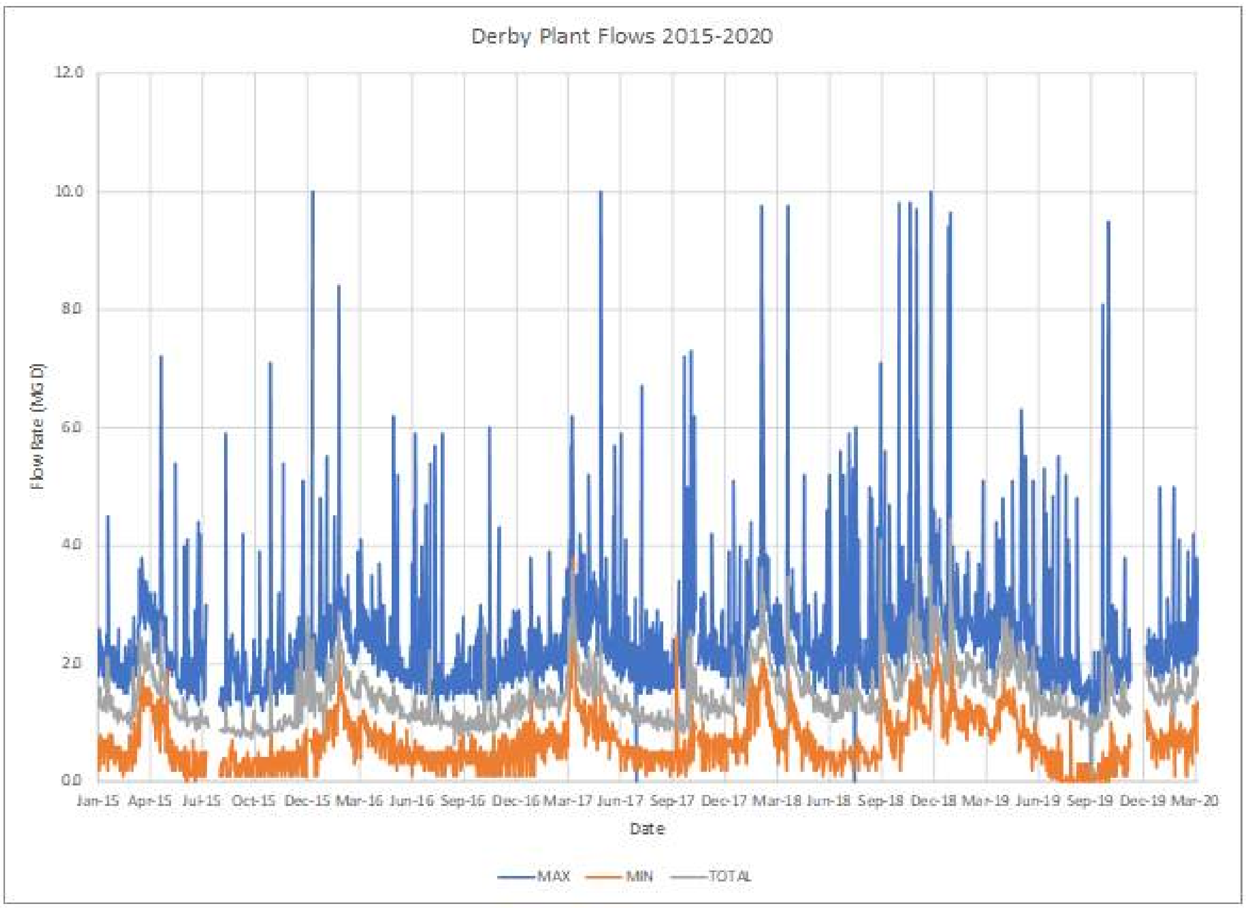

Figure 2-1 Derby Plant Flows 2015-2020

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.

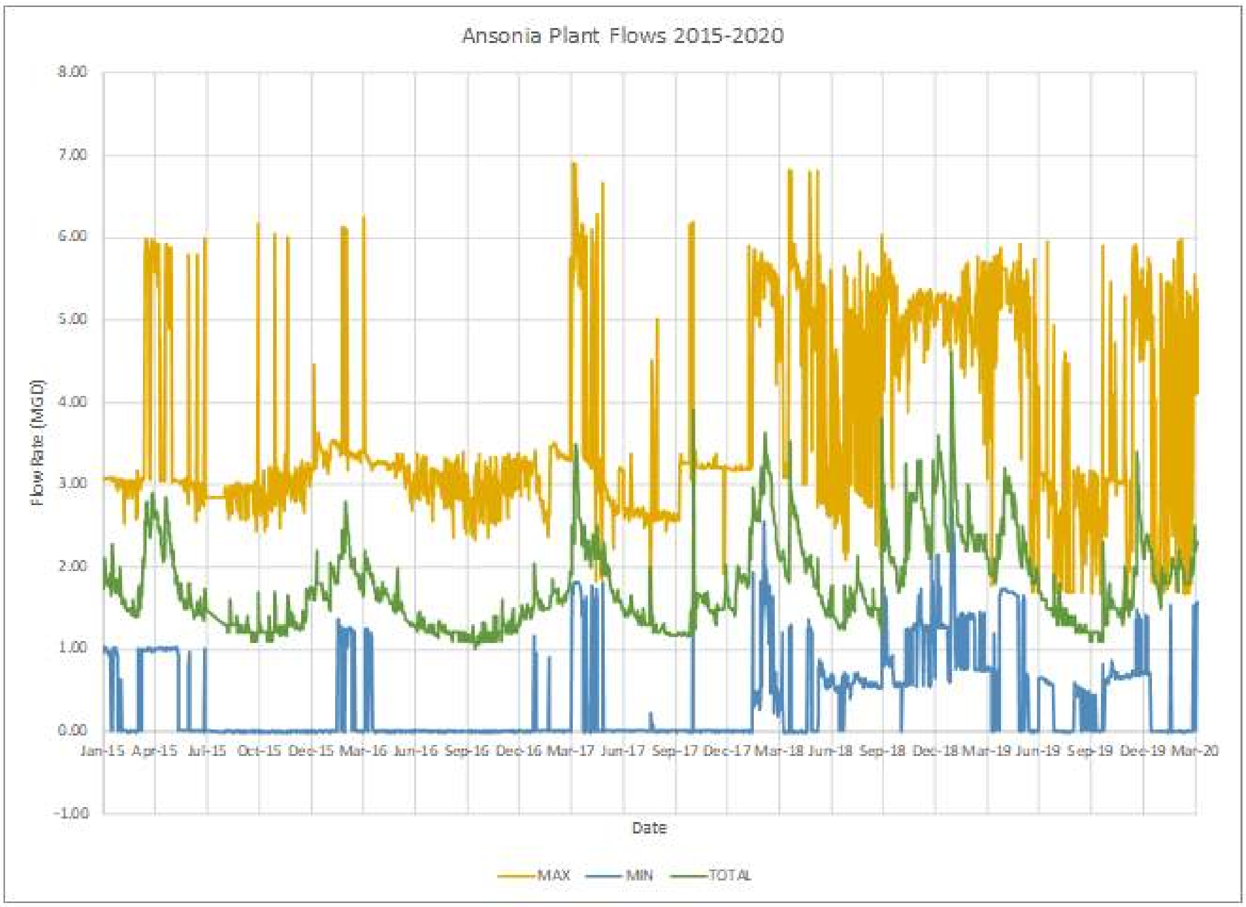

Figure 2-2 Ansonia Plant Flows 2015-2020

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.

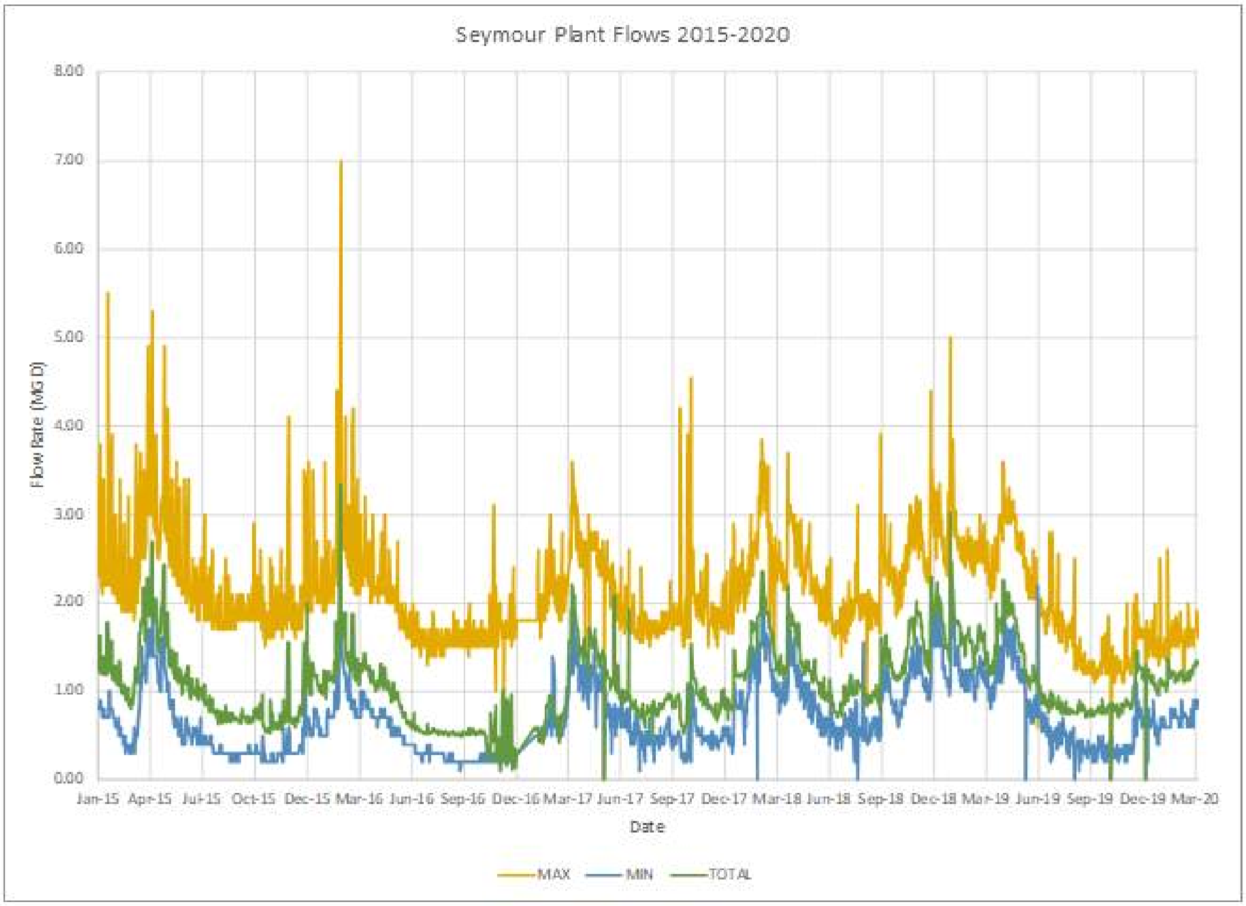

Figure 2-3 Seymour Plant Flows 2015-2020

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.

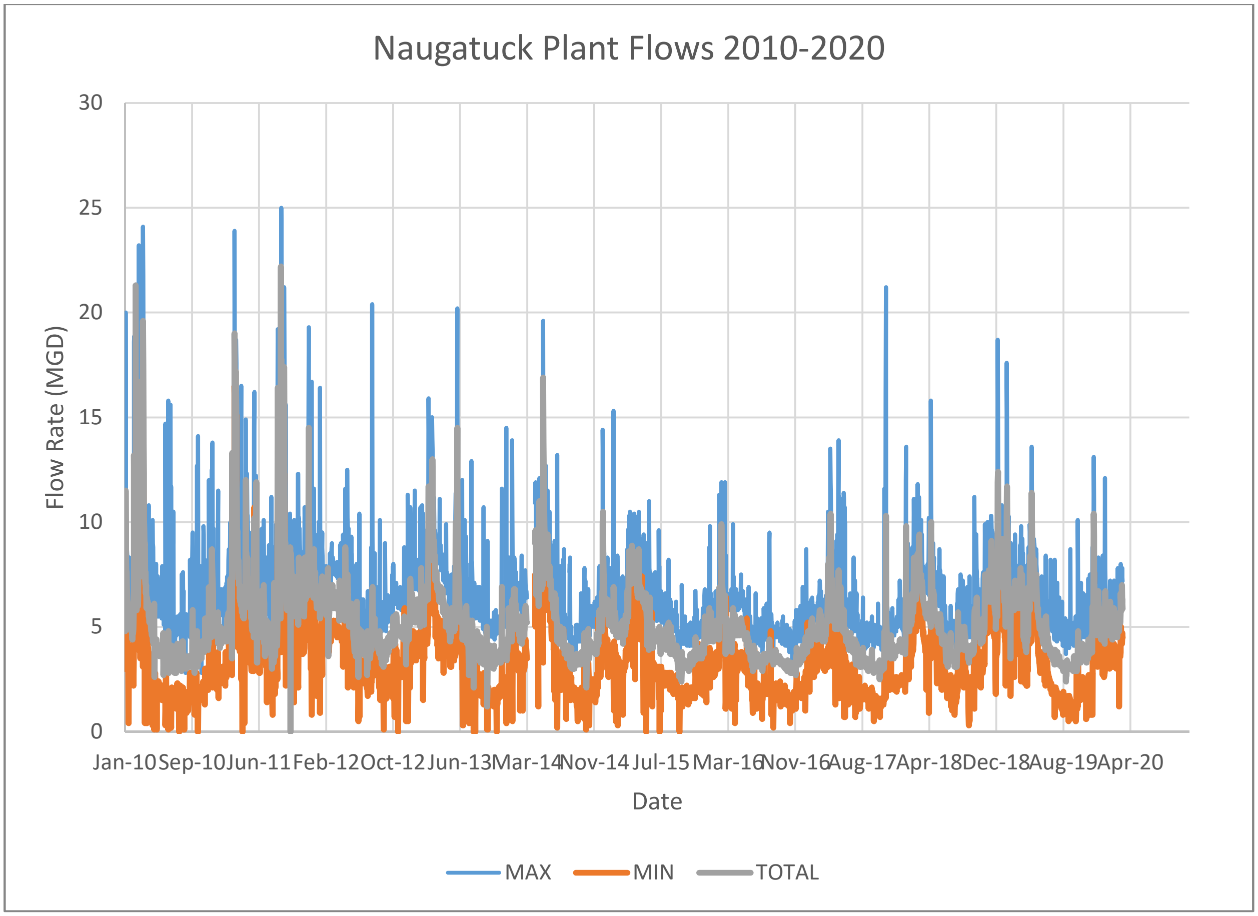

Figure 2-4 Naugatuck Plant Flows 2010-2020

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.

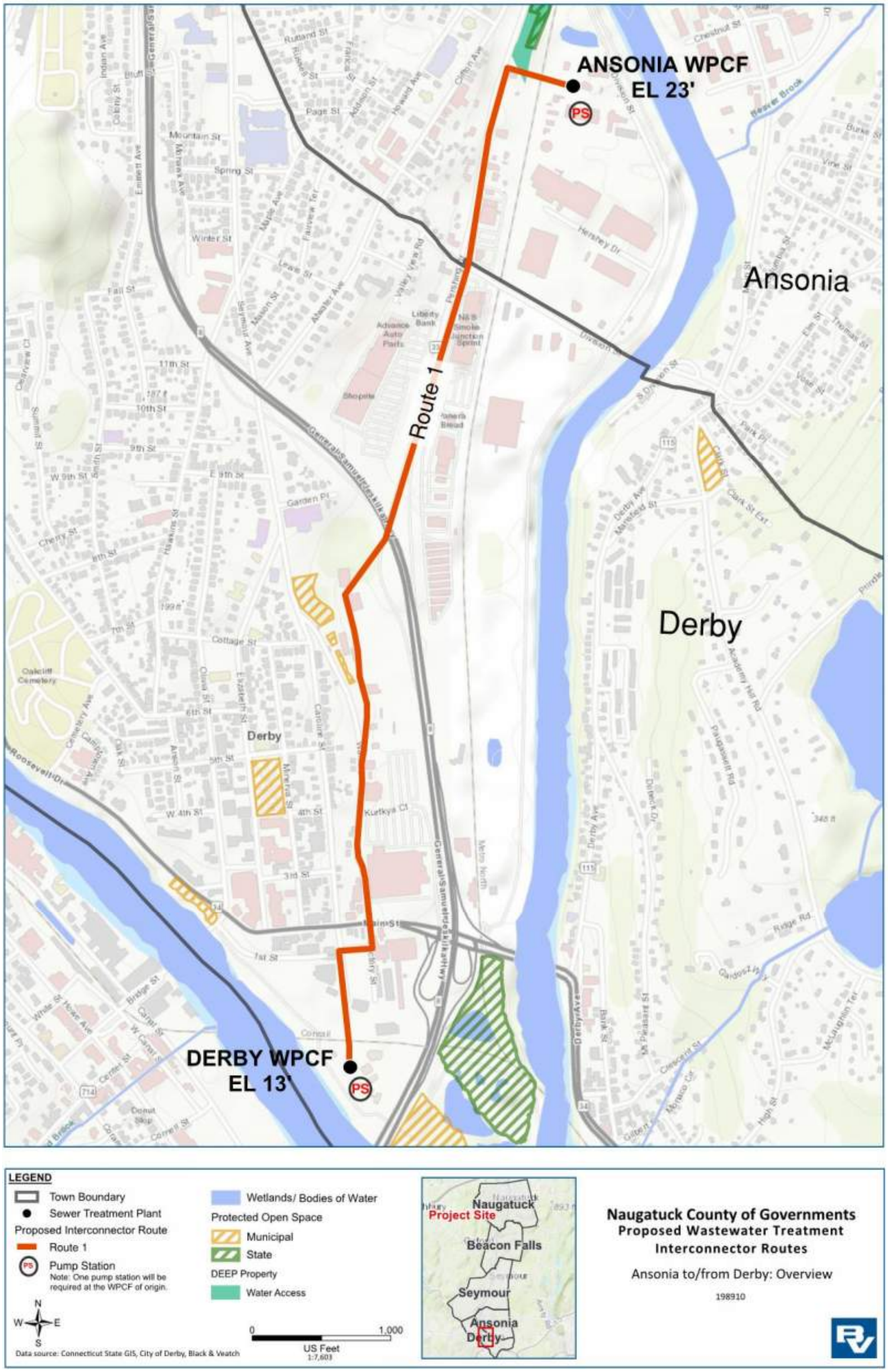

Figure 3-1 Ansonia to/from Derby Pipeline Route

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.

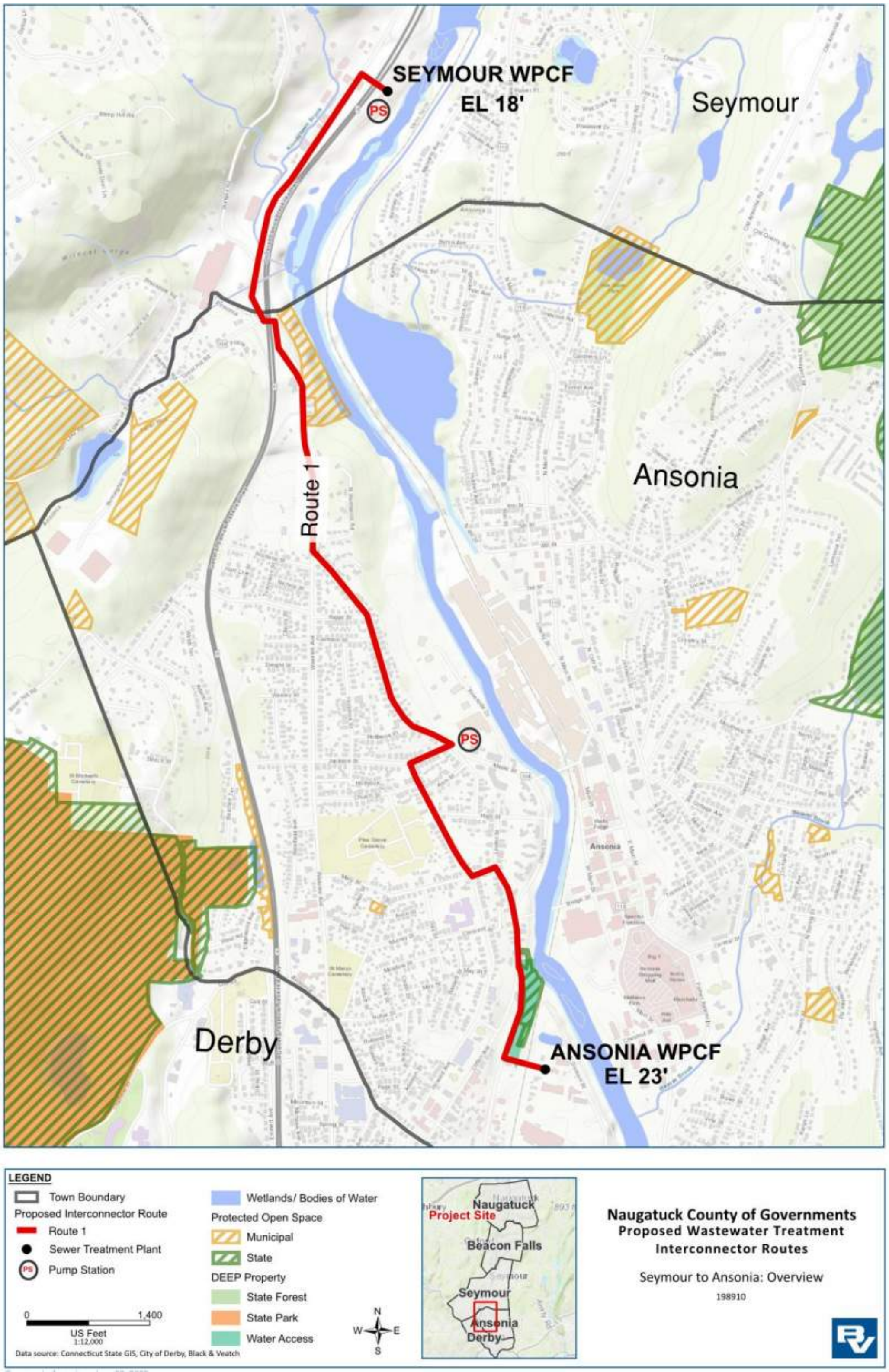

Figure 3-2 Seymour to Ansonia Pipeline Route

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.

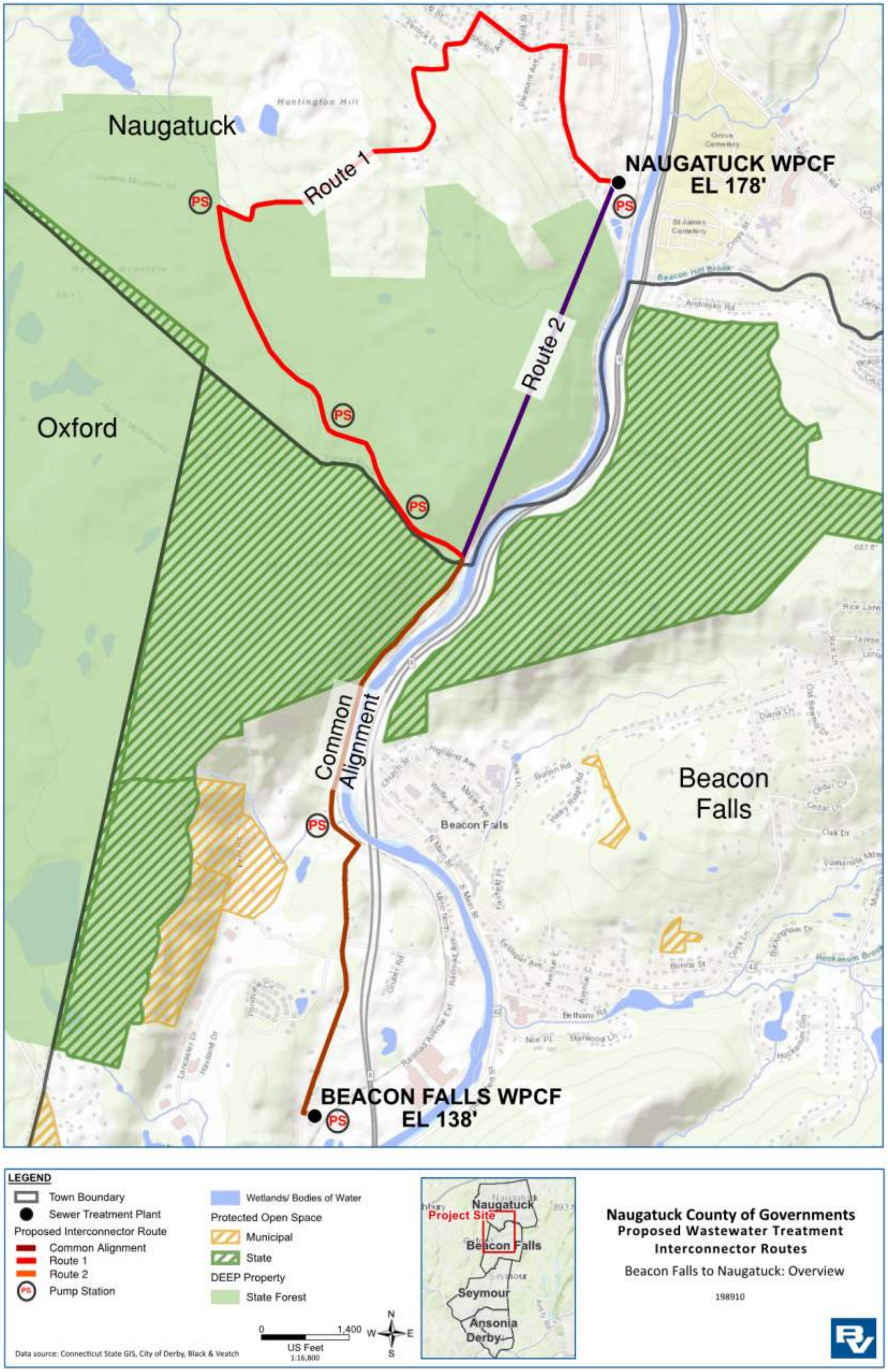

Figure 3-3 Beacon Falls to Naugatuck Pipeline Routes

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.

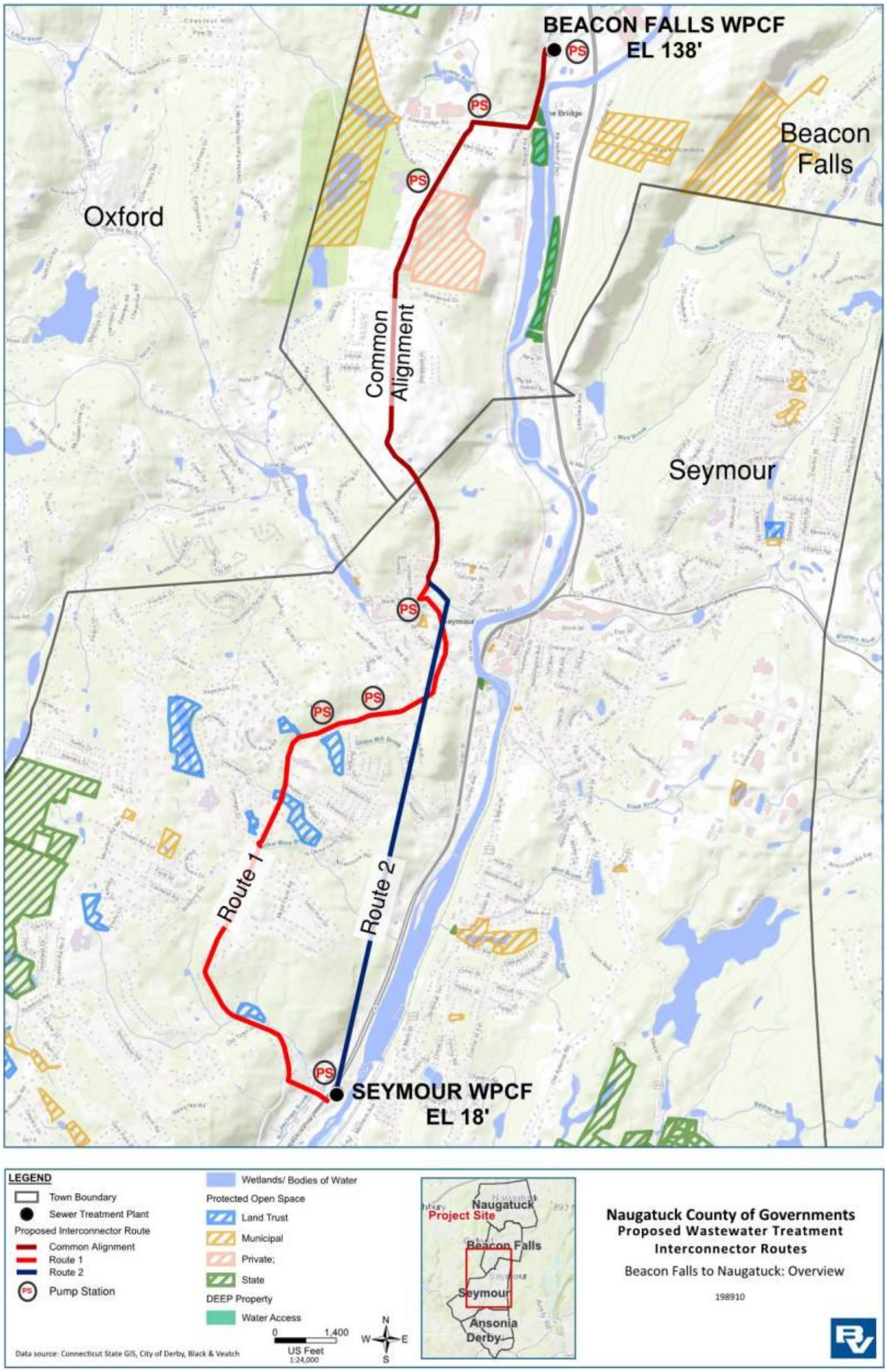

Figure 3-4 Beacon Falls to Seymour Pipeline Routes

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.

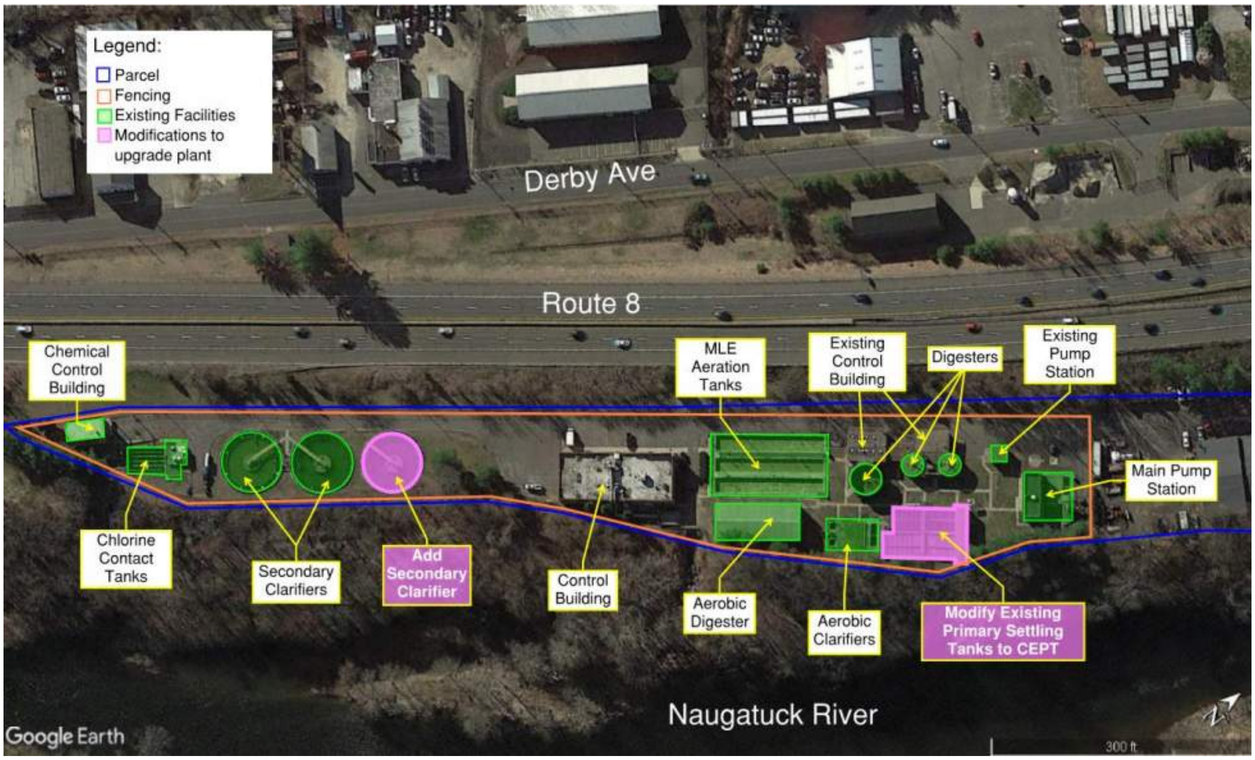

Figure 4-12 Treatment Facility Requirements for Seymour Only and Seymour Plus Beacon Falls

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/ft^2 and a peak hour surface overflow rate of 3,000 gpd/ft^2. If using chemically enhanced primary settling (CEPT), the peak SOR can be increased to in excess of 5,000 gpd/ft^2 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/ft^2 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.

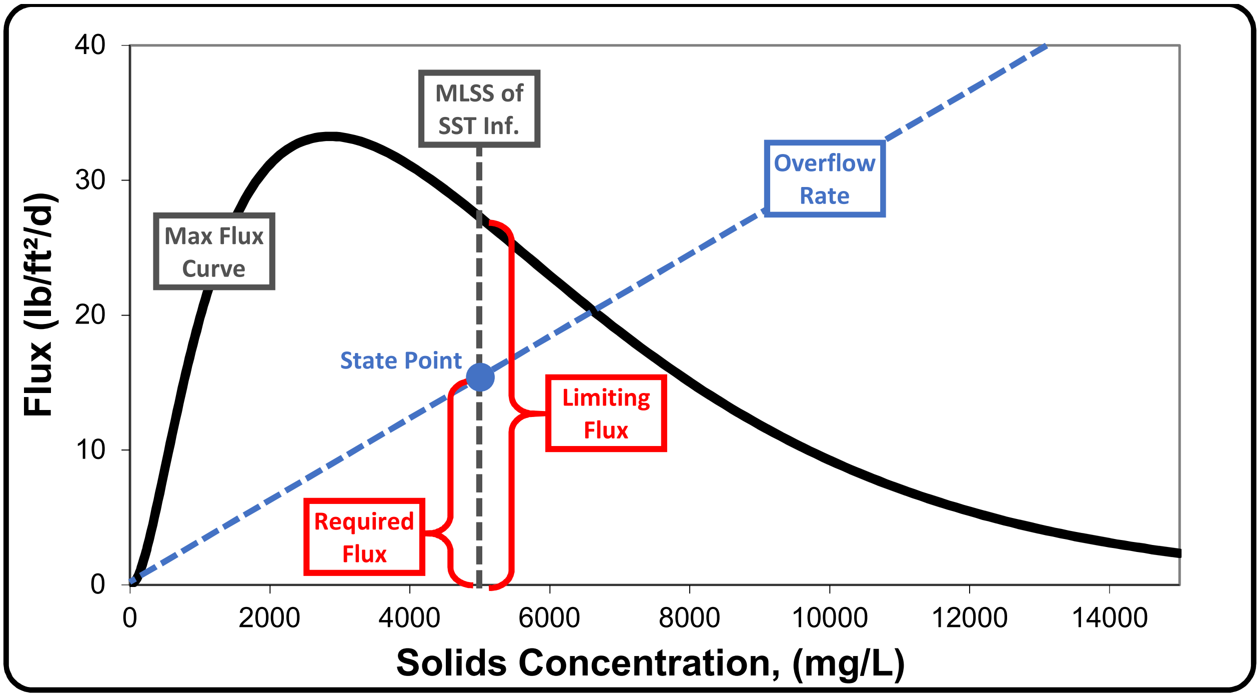

Figure B 1 Graphical Summary of Capacity Percentage Determined Through State Point Analysis

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/ft^2 Max Month, 2,500 gpd/ ft^2 Peak Hour
- SLR: Maximum 75 lb/day/ ft^2 Max Month, 100 lb/day/ ft^2 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/ft^2, and max day SLR to ~70 lb/day/ ft^2. 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 m^2/m^3
- 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/ft^2 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.

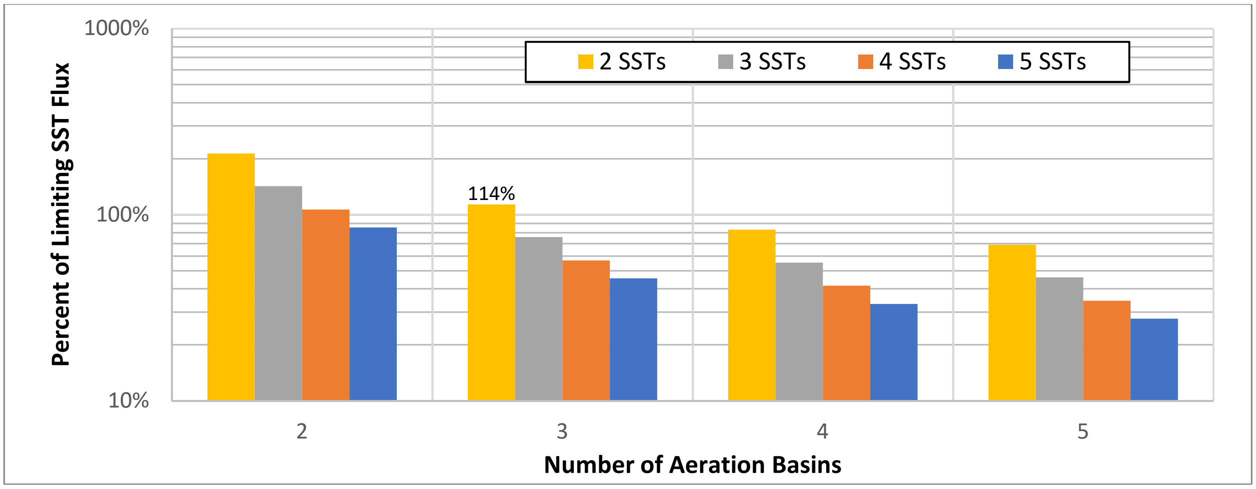

Figure B 2 Secondary Capacity Evaluation for Derby Treatment at Derby

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.

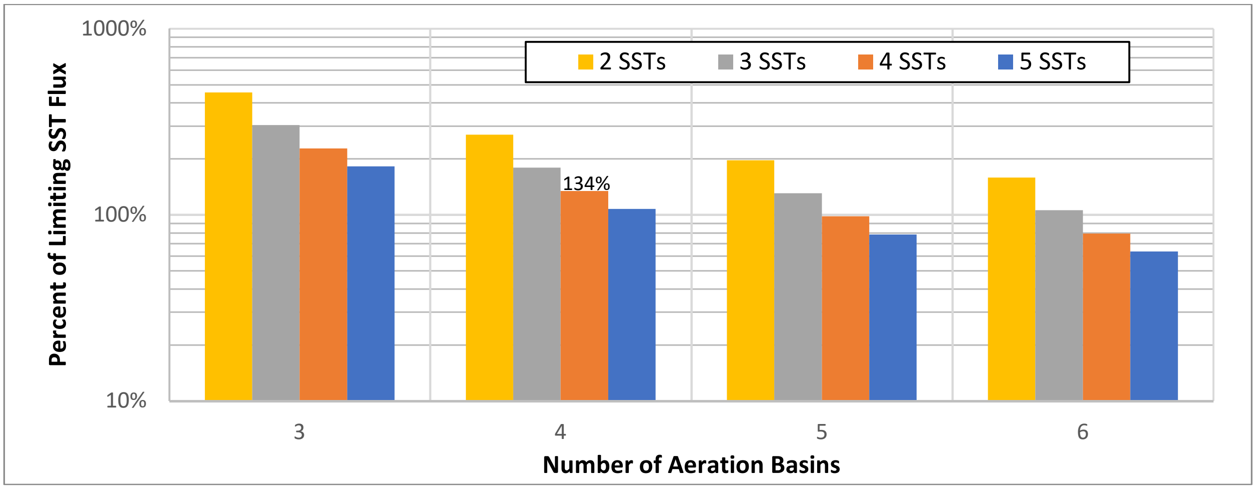

Figure B 3 Secondary Capacity Evaluation for Derby and Ansonia Treatment at Derby

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.

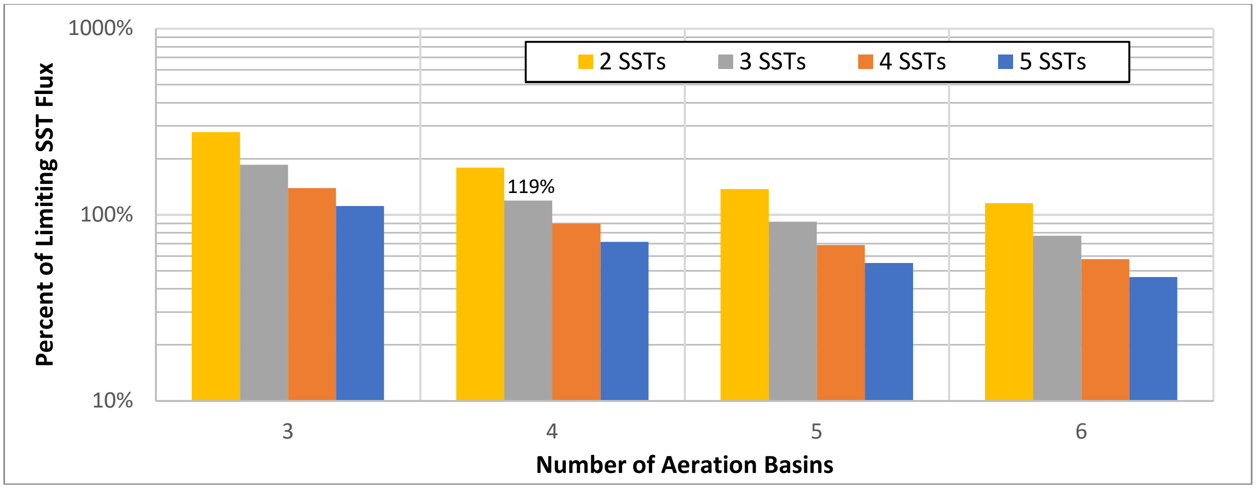

Figure B 4 Secondary Capacity Evaluation for Derby and Seymour Treatment at Derby

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.

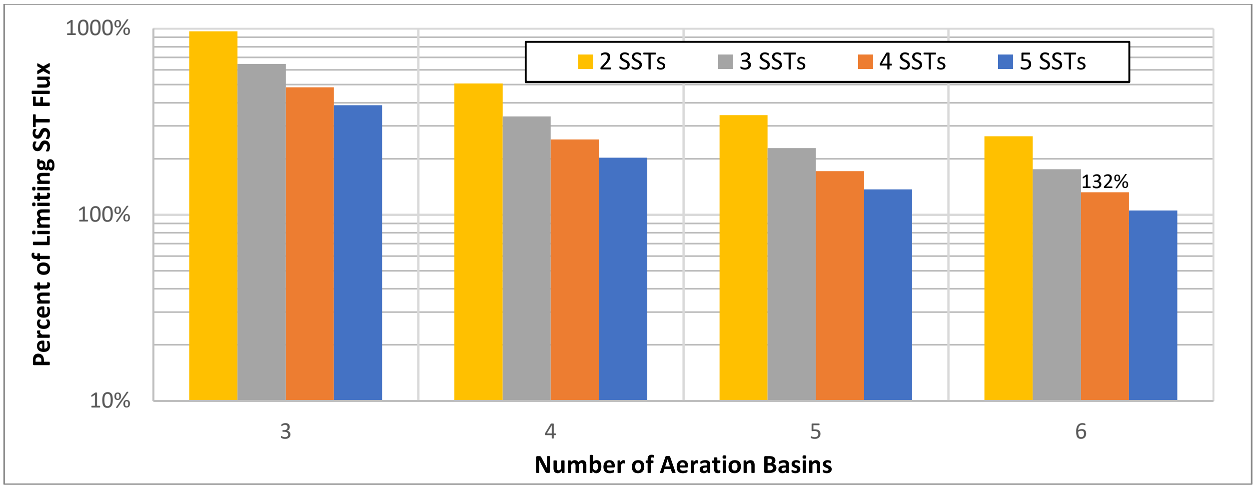

Figure B 5 Secondary Capacity Evaluation for Derby, Ansonia, and Seymour Treatment at Derby

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/ft²	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/ft²	1,670	1,875	1,875	1,875
PD SOR with 0 New PSTs, gpd/ft²	1,030	2,170	1,950	3,090
PD SOR with 1 New PSTs, gpd/ft²	820	1,740	1,560	2,470
PD SOR with 2 New PSTs, gpd/ft²	690	1,450	1,300	2,060
PD SOR with 3 New PSTs, gpd/ft²	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.

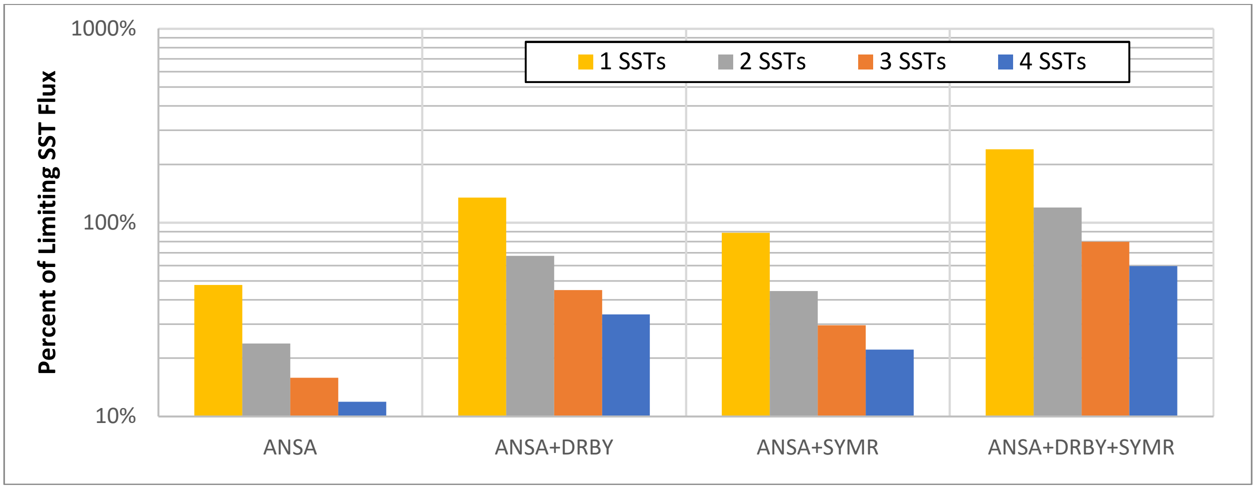

Figure B 6 Secondary Capacity Evaluation for Regional Treatment Alternatives at Ansonia

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/ft^2 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.

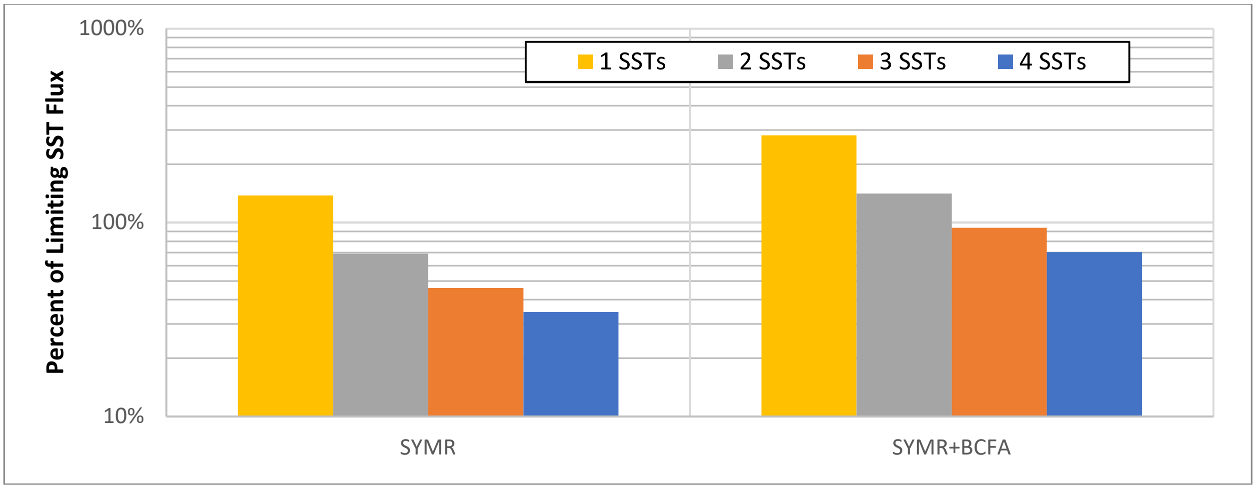

Figure B 7 Secondary Capacity Evaluation for Regional Treatment Alternatives at Seymour

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.